Site Selection Process for a Tailings Storage Facility in Cacata, Angola.

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Abstract

A recently established mining company holds an exploration license for Phosphate in the Cabinda Province in the Republic of Angola. The company plans to establish a mine, beneficiation plant and export terminal for exporting phosphate from the Cabinda Province of Angola, Southern Africa and in particular from the Cacata area.

This paper will present the process taken to identify an appropriate site for the location of the TSF and ancillary infrastructure. Capital requirements, environmental and social impacts were factors that influenced the choice of site.

The recommendation to the client was to design the TSF at a site that will allow for a cost effective design and have the least impact on the surrounding environment and neighboring community. Furthermore, should the TSF fail it will fail away from the pit area.

Keywords: Tailings Storage Facility, Site Selection Matrix, Environmental, Social, Impact

1 Introduction

A recently established mining company holds an exploration license for Phosphate in the Cabinda Province in the Republic of Angola. The company plans to establish a mine, beneficiation plant and export terminal for exporting phosphate from the Cabinda Province of Angola, Southern Africa and in particular from the Cacata area.

Golder Associates Africa Pty Ltd (Golder) was appointed by the Client to carry out the prefeasibility study and option analysis for the design of a new tailings storage facility (TSF), in line with the life of new mine of Cacata Phosphate Project.

This paper will present the processes taken to identify an appropriate site for the location of the TSF and ancillary infrastructure where the challenge was to minimize capital expenditure and environmental and social impact.
The primary objectives of the TSF design were to:
- Identify and determine an additional alternative site to an existing site;
- Develop a concept design on the preferred site to a Prefeasibility level of engineering to allow an option analysis between a site previously identified and a site identified by Golder;
- The design of the return water management for decant and seepage water from the TSF;
- Determine access and service road routes;
- Determine slurry and return water pipe corridor; and
- Determine source for embankment construction material.

2 Site location and Description

The Cacata deposit is located approximately 49km east of Cabinda Town. The accompanying tailings material will be stored in a new TSF. The new TSF is required to accommodate the tailings to be produced for a planned mine life of 11 years.

The general topography of the site comprises large undulating plains with a thick concentration of bush and trees. The proposed tailings storage facility (TSF) sites and plant beneficiation plant site are Greenfields, with a number of cassava plantations scattered in the north-eastern portion of the site. The populace is largely reliant on subsistence farming, thus the project will have a social impact on the surrounding community.

The vegetation comprises a thick concentration of trees, small shrubs and grass, interspersed with large baobab trees. Movement through the trees by vehicles is generally difficult, with some foot paths used by the local people; the site investigation team relied on the tracks formed by the excavator to navigate the site. The block plan of the identified TSF sites, beneficiation plant and overburden stock pile area is shown in Figure 1 below.

Figure 1. Cacata Project block plan
3 Design Criteria

The design criteria that Golder adopted in the TSF design on the preferred site and related infrastructure within the battery limit is listed in Table 1 below.

Table 1. TSF Design Criteria

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Tailings material</td>
<td>Phosphate</td>
</tr>
<tr>
<td>2</td>
<td>Deposition rate</td>
<td>Option1, Dry 5.5y 15 943 tpm (average)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Option2, Wet 11y 17 039 tpm (average)</td>
</tr>
<tr>
<td>3</td>
<td>Rate of rise</td>
<td>2 m/year</td>
</tr>
<tr>
<td>4</td>
<td>Life of mine (LOM)</td>
<td>11 years</td>
</tr>
<tr>
<td>5</td>
<td>In-situ dry density (average)</td>
<td>1.3 t/m³(Preliminary)</td>
</tr>
<tr>
<td>6</td>
<td>Average specific gravity of tailings solids</td>
<td>2.8 (Preliminary)</td>
</tr>
<tr>
<td>7</td>
<td>TSF Freeboard</td>
<td>Minimum 0.5m plus 100 year 24-hour storm event</td>
</tr>
<tr>
<td>8</td>
<td>Return water from TSF to the process plant</td>
<td>Maximum amount of water to be returned to the process plant</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Designed for a freeboard of 0.5m above the expected precipitation plus a 24-hour 100 year return storm event</td>
</tr>
<tr>
<td>9</td>
<td>Return water dam sizing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Decision making criteria applicable to the lining / engineered barrier of the TSF facility</td>
<td>Assume worst classification waste requiring a clay liner.</td>
</tr>
<tr>
<td>10</td>
<td>TSF embankment design and outer slope stability factor of safety</td>
<td>Operational slope FOS = 1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Closure slope FOS = 1.5</td>
</tr>
<tr>
<td>11</td>
<td>TSF decant system</td>
<td>Gravity penstock or floating pump barge</td>
</tr>
<tr>
<td>12</td>
<td>Tailings deposition methodology</td>
<td>Upstream deposition with spigotted tailings</td>
</tr>
<tr>
<td>13</td>
<td>Overall TSF slope for closure and rehabilitation considerations</td>
<td>1V:4H</td>
</tr>
</tbody>
</table>

4 Site Selection

Two areas were identified as possible sites that would be suitable for the location of the TSF. Site 1 is located west of the pit area approximately 800 m from the pit, adjacent to the existing access road. This area is approximately 30 ha in size and gently slopes towards the southwest. Currently the area is used for cassava plantations by the local community. South of this site is a marsh area with intercepting streams. It is anticipated that water will flow in the southeast direction during the rainy season.
Site 2 is located approximately 1.7km east of the pit area. It is roughly 72 ha in size and is located within the mine boundary. A seasonal stream east of the site intercepts the site, generally flowing in the northern direction. This site area is enclosed by gently sloping hills with a marshy area at the lowest point. Apart from animal traps constructed by the local community, no use of the area was identified and no infrastructure is located on the footprint. The site positions in relation to the mine pit and plant area are shown in Figure 2 below.

Figure 2. TSF site options

5 Site selection option matrix

The site option selection was ranked according to the following criteria (Blight, 2010):
1. Engineering / Technical
2. Economic
3. Regulatory
4. Environmental
5. Social

Some sub-criteria were established for each of the main criteria in order to rank the sites. The most critical ones identified were sub-criteria for Engineering, shown in Figure 3; Economic, shown in Figure 4 and Environmental, shown in Figure 5.
5.1 Engineering

Water management in terms of stormwater diversion and the size of the contributing catchment areas were taken into consideration. The potential to expand the facility should the life of mine increase was also an important input into the matrix. The ease of operating the facility, especially the pool and freeboard management, and the storage capacity of each site was considered. The proximity to other mining infrastructure like the process plant and pipeline infrastructure and any interference with existing servitudes like powerlines had to be deliberated as part of the engineering criteria.

5.2 Economic

The most important element of the economic criteria was the capital expenditure (Capex) and Operational expenditure (Opex) requirements of each site and concept. The site and concept with the least Capex and Opex requirements were favoured. Part of the Capex requirement is
the bulk earthworks, especially the embankment of the starter wall. A larger wall would require more capital expenditure. The potential closure costs at the end of the facility’s life is also an important consideration and a concept that could allow the client to defer capital (like building the facility in phases) was rated higher than the others.

5.3 Environmental

![Diagram](image.png)

Figure 5. Environmental considerations

The environmental considerations were dust control, taking into account wind direction and surrounding infrastructure that could be impacted. Any ecological sensitive areas were identified by the environmental specialist and the site and concept that had the least impact on the ecology were preferred. The impact on surface water and groundwater was also considered, with the aim of minimising this impact. There was also a need to minimise the visibility of the facility from areas that are outside the mining area so that it is not an “eye sore” in the area.

A project specific site selection matrix was developed to assist with qualitative rating and ranking of the identified candidate sites. The rating of the candidate sites was based on the values given in Table 2 below.

Table 2. Site selection rating value

<table>
<thead>
<tr>
<th>Rating</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>5</td>
</tr>
<tr>
<td>Above average</td>
<td>4</td>
</tr>
<tr>
<td>Below average</td>
<td>2</td>
</tr>
<tr>
<td>Very poor</td>
<td>1</td>
</tr>
<tr>
<td>Fatal Flaw</td>
<td>F</td>
</tr>
</tbody>
</table>
6 Developing the Site 2 concept further

Conceptual models were developed for Site 2 in order to further assess the feasibility of the site to accommodate the tailings volume. Tailings volumes were modelled on the basis of 5.5 Mt airspace requirement.

Site 2, Option 1
The general layout for Option is shown in Figure 6 below. Option 1 has the highest TSF footprint of 78.8 ha, with a final height of 51 m and has the largest starter wall volume. The large area will allow for a low rate of rise (2 m per year after 18 months) giving the tailings enough time to consolidate.

![Figure 6. Site 2, Option 1 general layout](image)

Site 2, Option 2
Option 2 is constructed against the southern hill on the site and therefore has a reduced area of 65.2 ha. The final height of Option 2 is 49 m and the rate of rise is 2 m per year after 2.3 years of operation.

![Figure 7. Site 2, Option 2 general layout](image)
Site 2, Option 3
Similar to Option 2, Option 3 constructed against the southern hill on the site but will be constructed with an upstream self-raise. Option 3 will require an area of 64.5 ha and has smaller starter wall. The feasibility of the Option 3 will depend on the tailings characterisation. The final height of Option 3 is 49 m and the rate of rise is 2 m per year after 2.5 years of operation.

Figure 8. Site 2, Option 3 general layout

7 Site Selection Matrix
A site selection matrix was developed with the Client to assess which site would be the most suitable. The site selection matrix was weighted and the client indicated that the Environmental and Social impact criteria to weigh the most at 35% followed by the Economic Criteria. The matrix is shown in Table 3 below.
Table 3. Site Selection Matrix

<table>
<thead>
<tr>
<th>Site Options</th>
<th>Option Description</th>
<th>Engineering Concept</th>
<th>Overall Score</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Weighting</td>
<td>Engineering</td>
</tr>
<tr>
<td>Option 1</td>
<td>Site located to the west of the Provisional Tar Road</td>
<td>Self-raise conventional TSF</td>
<td>35%</td>
</tr>
<tr>
<td>(Western SRK Site)</td>
<td></td>
<td>1 2 1 1</td>
<td></td>
</tr>
<tr>
<td>Option 2</td>
<td>Site located 1.7 km east of the Pit Area in a low lying</td>
<td>Full footprint development Southern Hill (downstream</td>
<td>35%</td>
</tr>
<tr>
<td>(Eastern Site)</td>
<td>area.</td>
<td>development)</td>
<td>4 2 4 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Southern Hill (upstream development)</td>
<td>4 4 4 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 4 4 3</td>
</tr>
</tbody>
</table>

Site 2, option 2 was selected as the most feasible site option for this design for the following reasons:
- Site 2 is against a hill and will only require an embankment at the toe of the hill whereas Site 1 would require an embankment around the whole base of the TSF. Therefore, capital expenditure for Site 2 is less;
- Site 2 provides ample area to expand. Site 1 is restricted because of the surrounding community residences;
- Site 2 offers the option of staging the construction and therefore differing capital;
- Since Site 1 is quite far from mining operations, it will cost more to deliver tailings to Site 1 than to Site 2; and
- Site 1 is close to the community and therefore scored low on the environmental and social impact aspects of the matrix.

8 Conclusion

The recommendation to the client was to design the TSF at site 2 because the topography will allow for a cost effective design. Furthermore, should the TSF fail it will fail away from the pit area and because the site is remote there will be little impact to the surrounding community.

Option 2 arrangement proved to be the most suitable for the Site 2 development, it requires relatively lower capital investment because the starter wall is against a hill and gives adequate airspace for the tailings to be deposited and an option to extend the facility.
References


Converting a Mechanised Mine to a Conventional Mine and the Associated Challenges as Experienced at Lonmin’s Saffy Shaft

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Abstract

Lonmin’s Saffy Shaft is situated close to the town of Marikana in the North-West province of South Africa. Shaft sinking and infrastructure development were conducted during the period of 2000 to 2005. The Shaft was initially sunk in anticipation of being a fully mechanised mine. Lonmin embarked on a drive to mechanise some of the operations (i.e. Saffy). Between 2010 and 2012 the mining method was changed from fully mechanised to hybrid mining. This entailed conventional stoping whereas all of the on-reef cleaning was conducted with a mechanised fleet of equipment. Due to difficulties experienced with the optimisation of the hybrid mining method (50% conventional mining & 50% mechanised mining), it was opted to revert to a completely conventional mining method in an attempt to achieve the Shaft’s target production output. Both mechanised and hybrid mining was conducted on an essentially conventional breast layout due to geotechnical constraints.

Keywords: Conventional mine; mechanised mining; geotechnical constraints.

1 Introduction

Saffy Shaft, a Generation 2 Shaft is situated near Marikana in the North-West Province and forms part of the Western Bushveld Complex Figure 1. Saffy Shaft is currently utilising a conventional mining method (i.e. footwall waste development with on-reef stoping panels), following a conversion from mechanised mining and hybrid mining methods. Shaft sinking commenced in the year 2000 and was completed in 2005. Saffy Shaft was originally planned to be an entirely mechanised operation. Mechanised mining was practiced from 2005 to 2009. It was, however, deemed unsuccessful due to the underperformance of the mechanised equipment having difficulty negotiating major geological structures (i.e. rolling reef and faults). A decision was made in 2009 to revert to Hybrid Mining method.

Hybrid Mining consisted of conventionally breast mining with the cleaning operations conducted via strike gullies using Load haul dump (LHD) machines. After further challenges and difficulties with the unreliability of the trackless equipment, the production target of 200 000 tons a month were still not met. In 2012 a decision was made to change to a fully
operational conventional mining method. The production target was met for the first time in 2014.

![Figure 1. Location of Lonmin’s Marikana operation in relation to the Bushveld Complex.](image)

### 2 Reef type Exploited

On Lonmin, 3 shafts on the western side of Lonmin are mining both the Merensky and UG2 reef bands. As for Saffy shaft only the UG2 reef is being mined. On the eastern side of Lonmin the Merensky reef has a reef width of up to 10m and the PGM grading is very low. It will not be possible to extract a reef band of 10m in thickness with both mining methods. The UG2 reef width is approximately 1.2m wide. The dip of the reef varies between 11–13 degrees and dips towards the North. The UG2 Reef has a prominent parting plane (UG2A markers) situated in the hangingwall where commonly two chromitite stringers are present within the pyroxenite layer between the HW1A Pyroxenite layer and the HW2 Anorthosite layers. The parting plane of the UG2A markers defines a beam varying in thickness of between 8–12m which has to be supported.

![Figure 2. UG2 Reef stratigraphy (left) and UG2At Isopach plan (right).](image)
2.1 Problematic geological features intersected within the Saffy Shaft boundaries
The major geological features on Saffy Shaft are 3 major normal faults, 2 dykes, prominent jointing which can lead to instability and water intersection on the western side of the Shaft. The faults intersected are the Saffy East and Saffy West faults which strike in the NNW–SSE direction with displacements between 12–15m, dipping between 75–80 degrees in the NE direction. The Turfontein fault strikes NNE–SSW with displacements varying between 30–42m, dipping 85 degrees in the NW direction. Numerous strata control challenges are encountered between and approaching the Saffy East and Saffy West faults. Two major dykes Figure 3 are situated between the faults on the eastern and western side of the shaft. An aqua zone was intersected on the western side of the Shaft which has a major influence on the surrounding working places and mining as additional support has to be installed to ensure stability. The source of the water is still unknown and is exposed along jointing and fault intersections.

![Figure 3. Aeromagnetic survey indicating major geological structures in Saffy shaft. (More O’Ferrall, 2009).](image)

3 Saffy Shaft’s mining methods
Three different mining methods were applied on the Shaft. A brief background is provided in the next subsection to give an overview of each to highlight the limitations and challenges.

3.1 Mechanised Mining
Mechanised mining refers to all the mining activities being conducted with extra low profile (XLP) equipment. The mining layout is rigid and complex as it does not have the flexibility to change when adverse ground conditions or problematic geological features are intersected. A simple strategy such as establishing support pillars to provide additional stability would be challenging.

A major challenge mining faced were the unreliability of the machinery and the long period of time it took to repair breakdowns. This resulted in panels not being cleaned or support not being
installed on time resulting in production losses. This severely contributed to the shaft production target not being met month after month.

The equipment itself was constrained by travelling speeds and travelling distances. This limited the equipment mobility between panels and to ore tipping points. As mining faces advanced, lengthy travelling distances are created. The fleet of machinery, workshops, conveyor belt extensions and tipping points must therefore constantly be moved forward to reduce hauling and travelling distances.

The mechanised mining layout also had to accommodate larger mining spans. Most equipment has hefty turning curves. Also hangingwall slots had to be blasted in order for the machinery to tip the load into tipping points or onto conveyor belts. These areas required above normal support which has to cater for a discontinuous hanging wall beam. The pillar dimensions had to be increased along these areas as the excavation height has an impact on the width to height ratio of the support pillars.

Furthermore, the fleet of equipment had to be able to negotiate major geological structures. This created challenges. Areas with rolling reef, creates a steeper inclination where the reef trends into the footwall. Mechanised machinery is limited to the inclination it can operate in. Blocks of ore could therefore not fully be extracted or cleaned. Furthermore, steep gradients greater than 13 degrees with full loads are troublesome for trackless equipment. It increases the turn-around time and impacts on tyre wear and mechanical components.

The cleaning and support of the panels when optimising any mining layout should be conducted in a timely manner. The support design and support units must cater for equipment size and mobility. Support units should not be bumped out of place or damaged. E.g. tendon support units, where applicable, must be such that it is designed to be as flush as possible against the hangingwall to avoid damage caused by heavy machinery in a low mining environment.

### 3.2 Hybrid Mining

The Hybrid mining method is a combination of 2 mining methods, it refers to the conventional drilling of the panels. Winches clean these panels into advance strike drives (ASD). Broken ore is hauled by means of trackless equipment to tipping points at conveyor belts. Hybrid mining exposes more employees at the face area. However, due to conventional cleaning of the panels, support can be installed closer to the face improving the hangingwall stability in the face area.

The width of the ASD’s must, however, still cater for the trackless equipment. The support design must therefore consider wider spans across the stope panel to the pillar. The advantage of trackless cleaning in the ASD’s is that strike swings do not hamper the movement of the equipment. Trackless equipment is not bound to straight line movement, whereas with scraper cleaning it is of the utmost importance.

During strike swings or when rolling reef or slumps are intersected, the mechanised equipment cannot tolerate gradients along the footwall steeper than 13 degrees. As with mechanised mining it impacts on travelling time, tyre and equipment wear. Similar to fully mechanised mining drilling of the ASD’s is conducted with trackless drill rigs which are lengthy and challenging to maneuver after every drilling cycle. Steep gradients and sharp bends require skilled operators to move the equipment.

As trackless equipment require short travelling distances to travel, decent planning is required to ensure short turn-around time and workshops must be regularly moved forward to be as close to the working area as possible. Closure and convergence cannot be tolerated when utilising this equipment. Pillar designs must be such to cater for a stable pillar design.
3.3 Conventional Mining
Conventional mining consists of both off and on-reef development ends where stope panels are established through ledging processes. Off reef development consists of drilling, cleaning and support where all is conducted conventionally (by people). The conventional equipment consists of handheld drills for drilling purposes, and track bound locos with hoppers to haul the broken rock to a level-to-level ore pass system. Workshops and service excavations for the off reef development equipment is normally situated in the shaft pillar area and remain permanent for the life of mine on the specific level.

On reef development is conducted by means of handheld drills. Cleaning is done using electrical winches with scrapers. The broken rock is moved into ore passes down to the level below the reef horizon. Once placed in these ore passes, rail bound locos with hoppers draw the rock from these steep dipping ore passes and haul it to the main orepass system near the shaft.

Stoping panels are established by means of ledging. Mining is done using conventional handheld drills and cleaned by means of electrical winches using low profile scrapers to accommodate the mining height (approximately 1.2m).

Conventional mining is greatly dependant on a significant amount of off-reef development. Once these excavations have been established, then only the extraction of the reef can commence. This result in additional mining cost, as well as equipment and labor overheads. Off reef excavations must be designed and supported to cater for the reef mining which could take place over long periods of time. This could lead to increased support costs especially when adverse ground conditions are intersected and the development excavations must be kept open for long periods (life of the half level).

4 Support Strategies and Geotechnical constraints
Due to the fact that three different mining methods were applied on Saffy Shaft, various support strategies and challenges were experienced. Support standards and designs had to cater for various aspects and differences. The key challenges will be discussed in the following subsections.

4.1 Mechanised Mining
The pillar designs had to take the equipment dimensions in consideration. It had to accommodate the travelling of the drill rig from panel to panel. As a result the pillar holings had to be larger (4m x 4m). Also be able to allow tipping the ore at the tipping points. On Saffy Shaft an alteration layer Figure 4 is present along the top of the pillars. This results in the pillars behaving differently. To ensure long term stability this had to be incorporated in the design (du, Plessis, 2009). The inter pillar spans (panel lengths) was approximately 30m. The span had to accommodate equipment cycle times (support before the next blast). A 1.6m long tendon length was required to ensure that the resulting beam could be re-enforced and remain stable.
Due to the nature of the ground conditions on the shaft the support design had to ensure the stability of the UG2A markers situated 8–12m above the hangingwall Figure 2. A 1000 ton pack had to be developed to ensure the support requirements were met. The rock bolt support installed was Hydrabolts, a friction anchored tendon being pre-stressed with water. The Hydrabolts were effective in optimising the cycle times, however, allowed beam deflection which resulted in small-scale falls of ground occurring between the current support installed.

A breast mining layout has accommodated the equipment requirements. Breast mining also accommodated the orientation of the large scale problematic geological structures Figure 5, being mined on strike (E–W) ensures support for the geological features as the face advances Figure 5 and was therefore the preferred mining layout. The dip of the reef restricted the maneuverability of equipment. Dip access ways were therefore put on apparent dip.

One of the challenges with mechanised mining is that there is not any flexibility when poor ground conditions are intersected. Poor ground conditions may not encompass the entire length of the panel. Mining of only half a panel restricts the mobility of equipment as it cannot travel through restricted areas.
4.2 Hybrid Mining
Hybrid mining increased the exposure of employees in the face area. The support standard had to be redesigned as the support had to be closer to the face. The support requirement was the same as with mechanised mining in the development but the support standard in the panels had to be changed. Pillar holings were still 4m x 4m. This created a risk as the holings were used by the employees to travel through. If the holings weren’t being adequately supported it will impact on their safety.

Given the fact that the panels were being blasted and supported conventionally, elongate support could be introduced. Elongate support provides stiff-active support which is capable of supporting up to the UG2A markers. Grout packs were still being installed but as a supplementary support to prevent possible back breaks.

4.3 Conventional Mining
The majority of the Shafts on Lonmin Platinum are being mined conventionally. However, numerous challenges were experienced when the shaft was changed to a conventional mine. With the mechanised mining all the development as previously mentioned was on-reef, but conventional mining on and off-reef development was required.

Conventional mining required people to install the support in an unsupported area after a blast. The support standards had to change accordingly. Conventional mining also entailed a lot more people working on the face. The support spacing in the panels had to be closer to the face to ensure the safety of the employees. Figure 6 compares the support standard for conventional versus hybrid layouts to point out the significant differences. To ensure the stability of the conventional panels in problematic ground conditions the packs were installed 5m from the face after the blast to ensure stability of the hangingwall. The in-stope rock bolts changed from the 1.6m Hydrabolt (Friction anchored) to 1.6m Resin bolts which is a stiff active support unit as cycle times were no longer a limiting factor. The resin bolts being a stiff active support unit doesn’t allow any beam deflection and less falls of ground were experienced.

Even though the breast panel layouts were preferred, Lonmin specializes in split dip mining. This consists of the centre raise and two half panels being mined either side. Problematic ground conditions can be negotiated by only advancing one of the half panels. Also joint modelling (van Zyl, 2009) identified this layout to also accommodate the orientation of the problematic WNW joint orientation when the western panels leads the eastern panel.

The pillar designs could be adjusted as smaller pillar holings could be implemented (2m x 5m) which also meant less exposure to employees travelling through the holings. The challenge with holings is the fact that in most cases it doesn’t get supported after it’s been blasted. Even though the support standard was designed to ensure safety of employees and equipment the challenge of more people with a severe lack of discipline is problematic.
Figure 6. Hybrid layout (left) and Conventional Support Standard (right) (Saffy Shaft).

5 Conclusion

Saffy Shaft was converted from a fully mechanised operation to a conventional mine as a result of numerous different challenges. The complex ground conditions and unreliability of mechanised machinery did not allow for productive mining for both the mechanised or hybrid mining methods. Saffy achieved the target of 200 000 tons for the first time in 2014 with the conventional mining method. There are still daily challenges with the conventional mining method, however, conventional mining provides flexibility to adjust and accommodate the challenges experienced. The main focus of any mine is to create a safe working environment to ensure production requirements are achieved.

References

Routine GPR Mapping of UG2 Chromitite Triplets

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Abstract

GPR technology has been previously shown to be applicable for the mapping of chromitite triplets above the UG2 reef in the Bushveld Complex. A review of previous work on this topic and a theoretical evaluation of its applicability to the mapping of triplets was conducted. Electrical properties of the host and target rocks are also evaluated. The evaluation exercise came to the same historic conclusion that GPR technology is applicable for chromitite triplets mapping. The evaluation further determined the vertical resolution of 13cm, this being the width of the smallest detectable feature when using a 400 MHz antenna for the GPR system.

Keywords: GPR, UG2, Triplets, Mapping, Dielectric Constant.

1 Introduction

The application of Ground Penetration Radar (GPR) technology in the mapping of geological features in underground mines, especially in the Bushveld Complex (BC), was shown to be viable more than 10 years ago by Vogt et.al. (2005). Routine application of this GPR technology has however not expanded as anticipated based on the expected benefits of application.

This paper is aimed at helping to increase the application of GPR in the mining industry by providing technical content that will give confidence to mine rock engineers to use GPR technology, which is still mostly perceived to be a specialist’s tool.

Numerous underground geological features can be mapped using the GPR technology however this paper will evaluate the mapping of the chromitite triplets, which are just a few meters above the UG2 reef.

Knowledge of the middling between the UG2 reef and the triplets is critical in fall of ground management for the following two reasons:

• Where the triplets are too close to the reef they pose a fall of ground hazard, and
• Where the triplets are too far they pose a challenge to the support design.
A detailed knowledge of the triplet position is therefore crucial for fall of ground management on the UG2 reef horizon.

This paper provides a brief review of published work regarding GPR mapping of geological features in the BC. An explanation of how GPR technology is applied for underground geological feature mapping and an evaluation of GPR application to the mapping of chromitite triplets is given. This is done in order to give a better understanding and confidence in the application of GPR technology. Applicable parameter values for the host and target rocks applicable to UG2 triplet mapping within the BC are also given.

2 Preceding Published Work on GPR Application in the Bushveld Complex

Initiation of South African research into GPR routine mapping of geological features in the mines was started in the late 1980’s. However, it was in the 1990’s that research work into the feasibility of GPR hangingwall geological feature mapping in the gold and platinum industries was produced by ISS Geophysics (Pty) Ltd. This work laid the foundation for later research into BC GPR research (White et.al., 1999).

During the early 2000’s, the CSIR Mining Division did numerous progressive research on the application of GPR to map geological features in the BC (Vogt et.al., 2005). This work concluded that:

- ‘The electrical properties of the host rocks to the Merensky and the UG2 reefs indicate that radar is a viable technique within Bushveld platinum mines’
- ‘The GPR studies show that GPR is an effective tool for determining the distance to some of the important parting planes in the hangingwall of the UG2’

The work done by Vogt et al. (2005) also indicated that the hangingwall mapping of chrome triplets on the UG2 reef horizon meets all the requirements for a successful GPR scan as:

- The host rock (pyroxinite) has very low conductivity / high resistivity
- The target (chrome triplets) has a sharp contrast in dielectric constant with the host rock (pyroxinite)
- The target (chrome triplets) lies parallel / sub-parallel to the hangingwall which is the line of access

Additional research on measuring the radar frequency electrical properties of the Bushveld rocks was done by the CSIR Mining Division to aid in the application of the GPR technology. This work was published by Du Pisani and Vogt (2003) and Ngwenya et al. (2009). The Department of Electrical and Electronic Engineering at the University of Stellenbosch also did valuable and accurate work on the electrical properties of Bushveld rocks in 2006 (Rütschlin et al., 2007). The values of the electrical properties of the relevant host and target rocks are therefore known.

As part of the Platmine GPR project, ‘Implement and Optimise Ground Penetrating Radar’, published in 2004, the CSIR Mining Division also developed a GPR Application Guideline / Manual. These guidelines dealt with both the benchmarking and optimisation of GPR surveys, by identifying required items for planning and execution of in-mine GPR surveys (van Schoor, 2004).
3 GPR Theory

3.1 GPR Method and Survey
Ground Penetration radar is an electromagnetic method which operates at a frequency range of 1 to 1000 MHz. At these frequencies the electromagnetic field propagates as a wave through the rock. Electromagnetic energy pulses are sent from the antenna by the transmitter through the rock and arrives at the receiver (also inside the antenna) by wave reflection on a rock with different electrical properties, see illustration in figure 1 below (Dentith & Mudge, 2014).

![Figure 1. Schematic of Underground GPR Application. Adapted for underground situation from Dentith & Mudge (2014)](image)

In underground assessment of the hangingwall structures, the antenna is turned upside down against the hangingwall and the signal is sent up into the hangingwall rock as illustrated in figure 1 above.

In GPR surveys an electromagnetic pulse is transmitted as a discrete pulse of electromagnetic radiation, and the responses of the subsurface rocks are recorded. The method operates on a reflection-mode, where the pulse is reflected by the electrical property contrast in the subsurface rocks, and a record of the time difference in the reflected pulses is kept.

The most common configuration of a GPR survey is the reflection profiling mode, where the antenna direction is parallel to a survey line. The antenna is moved at near constant velocity along the survey line and the number of samples taken along the survey line affects the lateral resolution of the survey.

The depth to reflector indicated in figure 1 depends on the propagation velocity of the pulse through the host rock. The velocity and other electrical properties of the host and target rocks are determined in laboratory tests as will be indicated in the section on rock properties.

3.2 Radar Wave Propagation in Rocks
Important parameters in the design of GPR surveys is the velocity and attenuation of the electromagnetic waves in the rock medium. These parameters are dependent on the conductivity and dielectric properties of the host rock and they vary with the signal frequency outside the Radar window. Dentith & Mudge (2014) define the radar window as the frequency

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range where the attenuation and velocity are independent of the wave frequency. This is the range where the GPR system operates, between 1 MHz and 1000 MHz. They further indicate that to operate a GPR in the Radar window, the host rock’s conductivity must be about or less than 0.1 S/m (Dentith & Mudge, 2014).

According to Dentith and Mudge (2014), when operating within the Radar window, the most important parameter to radar wave response is the dielectric constant of the host and target rocks. It is therefore important that the radar window for a rock type and the correct values of dielectric constant are known.

The theoretical calculation of radar wave propagation parameters relevant to radar survey design / evaluation are discussed in the subheadings below as explained by Dentith & Mudge (2014) and Sensors & Software Inc. (1992):

3.2.1 Depth of Penetration
The penetration depth of EM waves into conductive materials is determined from the attenuation of the signal with depth, which is a function of conductivity, dielectric properties and frequency (Sensors & Software Inc.,1992).

Sensors & Software Inc. (1992) cited by Dentith & Mudge (2014) gives the following rule of thumb for estimating the maximum detection depth ($d_{max}$, in m) in terms of either attenuation ($\alpha$ in dB/m) or conductivity ($\sigma$ in S/m).

$$d_{max} < \frac{30}{\alpha} < \frac{0.035}{\sigma}$$

A conservative rule of thumb is that radar will be ineffective if the target depth is greater than 50% maximum depth calculated above.

3.2.2 Power Reflectivity Index
In order to determine whether a target will generate a response detectable by the antenna, the following factors must be considered:
- Is there enough contrast in the host and target dielectric constant
- Is the target physically adequate (size) to reflect a detectable amount of energy

Sensors & Software Inc. (1992) provides the following formula for estimating the Power Reflectivity ($Pr$), using the dielectric constant of the host rock ($\varepsilon_{host}$) and target rock ($\varepsilon_{target}$):

$$Pr = \left| \frac{\sqrt{\varepsilon_{host}} - \sqrt{\varepsilon_{target}}}{\sqrt{\varepsilon_{host}} + \sqrt{\varepsilon_{target}}} \right|^2$$

There are two rules of thumb for predicting success based on the above two considerations:
- The Power Reflectivity ($Pr$) should be at least 0.01 and above
- The ratio of target depth to smallest lateral dimension should not exceed 10:1

3.2.3 Resolution Size and Antenna Selection
For the GPR system, it is the wavelength of the pulse signal that determines the horizontal and vertical resolution. The wavelength of the signal is therefore determined by the antenna choice, since all GPR antennas have a predesigned central frequency (Sensors & Software Inc.,1992).

Sensors & Software Inc. (1992) cited by Dentith & Mudge (2014) gives the following formula as a rule of thumb for selecting the appropriate central frequency ($f$ in MHz) of the antenna, using the dielectric constant of the host rock ($\varepsilon_{h}$):
\[ f = \frac{150}{x \sqrt{\varepsilon_r}} \]  

(3)

\( x \) is the desired spatial resolution (m).

The spatial resolution gives the minimum width or size of a detectable target. This formula can therefore be reversed to calculate the minimum size detectable using an antenna with a particular central frequency.

4 Geological Setting

The UG2 reef at the Western BC has an average thickness of 70 cm. The hanging wall rock immediately above the UG2 reef is a 6m to 9m thick layer of pyroxinite, which is the host rock of interest. Within the pyroxinite host rock exists chromitite layers (target rocks) known as the Intermediate Chromitite Layer (ICL) and the chromitite “Triplets” which represent potential hazardous planes, see figure 1 for a generalised geological succession at Impala Platinum Mines (Impala Platinum Ltd, 1997).

![Generalised UG2 Geological Succession](image)

Figure 2. Generalised UG2 Geological Succession (Impala Platinum Ltd, 1997)

The distance to the lowermost triplet can be estimated to be from 1.0m to 4.5m and their combined thickness is generally 1.5m, which gives a required mapping vertical distance of 6.0m into the hangingwall. The chromitite triplets are each averaging 15cm in thickness and spaced at 50cm apart. The ICL is not well developed, however where it is encountered, it occurs between the UG2 Reef and the Triplets. The ICL is a single chromitite layer with a thickness of 0.5cm to 4cm (Balakrishna, 2006).

5 Electrical Properties of Host and Target Rocks

The electrical property measurement work for BC rocks done by Rütschlin et al (2007) is accurate and detailed and therefore its results can be used for the host and target rock properties. It should be noted that although the measurements were done at 25 MHz, the dielectric constant (permittivity), conductivity (inverse of loss tangent) and attenuation behave relatively constant over the GPR antenna frequency range of 25 – 1000 MHz (Dentith & Mudge, 2014).
Table 1. Electrical Properties of UG2 Rocks at 25 MHz (Rütschlin et al., 2007)

<table>
<thead>
<tr>
<th>Material</th>
<th>Sample</th>
<th>Permittivity</th>
<th>Loss tangent</th>
<th>Attenu. [dB/m]</th>
<th>Velocity [m/μs]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>εr</td>
<td>tan δ</td>
<td>α</td>
<td>CV α</td>
</tr>
<tr>
<td>Anorthosite</td>
<td>1</td>
<td>7.43</td>
<td>0.05</td>
<td>9.09</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>8.26</td>
<td>0.06</td>
<td>7.76</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8.31</td>
<td>0.05</td>
<td>5.01</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>8.21</td>
<td>0.05</td>
<td>11.65</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8.00</td>
<td>0.05</td>
<td>12.37</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>6, 7</td>
<td>12.16</td>
<td>0.10</td>
<td>5.95</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>7.75</td>
<td>0.04</td>
<td>10.60</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>11.67</td>
<td>0.11</td>
<td>5.03</td>
<td>0.8</td>
</tr>
<tr>
<td>Chromitite Triplets</td>
<td>6, 7</td>
<td>11.70</td>
<td>0.09</td>
<td>4.93</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>8.28</td>
<td>0.06</td>
<td>29.68</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>8.19</td>
<td>0.08</td>
<td>4.27</td>
<td>0.5</td>
</tr>
<tr>
<td>PFP</td>
<td>11</td>
<td>8.00</td>
<td>0.05</td>
<td>0.36</td>
<td>104.75</td>
</tr>
<tr>
<td>Melanotite</td>
<td>11</td>
<td>12.00</td>
<td>0.11</td>
<td>0.85</td>
<td>86.77</td>
</tr>
</tbody>
</table>

The coefficient of variation (CV) is the percentage ratio of standard deviation to mean of a particular.

The pyroxinite rocks above the UG2 reef is represented by sample 2 – 5 in table 1 above and the chrome triplet is represented by sample 3, 6 and 7. Table 2 below contains the average values for the rocks of interest determined from the table above:

Table 2. Properties of Host and Target Rocks (After Rütschlin et al, 2007)

<table>
<thead>
<tr>
<th>Rock types</th>
<th>Dielectric constant (εr)</th>
<th>Loss tangent (tan δ)</th>
<th>Attenuation (α) [dB/m]</th>
<th>Velocity (ν) [m/μs]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feldspathic Pyroxenite</td>
<td>8.00</td>
<td>0.05</td>
<td>0.36</td>
<td>104.75</td>
</tr>
<tr>
<td>Chromitite Triplets</td>
<td>12.00</td>
<td>0.11</td>
<td>0.85</td>
<td>86.77</td>
</tr>
</tbody>
</table>

6 Evaluation of GPR Suitability

Evaluating the appropriateness of the GPR method for the intended mapping of chromitite triplets can be accomplished using GPR theory equations (radar range equations). The evaluation will be conducted by providing replies to the questions below.

6.1 Is the target within the detectable range?

The way to answer this question is to calculate the maximum depth of penetration by using equation 1 given above and the attenuation (dB/m) of the host rock (Sensors & Software Inc., 1992). The average attenuation of the Feldspathic Pyroxenite from table 2 is 0.36 dB/m. The calculated maximum depth (dmax) is 83.33m. Using the rule-of-thumb of 50% given in section 3.2.3, the penetration depth in Feldspathic Pyroxenite is 41.66m.

Since the expected depth of the chromitite Triplets is from minimum 1.5m to maximum 9.0m, this will be within the detection range.
The equation used above is not universal, but is applicable when the attenuation is $> 0.1 \text{ dB/m}$ (Sensor & Software Inc., 1992), which is applicable in this case.

6.2 Will the target generate a detectable response?
To respond to this question, the Power Reflectivity must be calculated using equation 2 given above. Inputting the values of dielectric constant from table 2, equation 2 gives a Pr value of 0.0102. This value exceeds the rule-of-thumb value which states that for GPR success the Pr value should be above 0.01 (Sensors & Software Inc., 1992).

The lateral extend of the target exceeds the rule-of-thumb of 10:1 since the target extends over the whole UG2 reef and therefore would be detectable.

6.3 Is the width of the target adequate?
In order to determine if a chromitite layer of particular thickness will be detected by the GPR system, an inverse of equation 3 will be used to determine the spatial resolution of a chosen antenna. A common antenna central frequency of 400 MHz and the dielectric constant of the host rock of 8.0 was used.

Using these parameters, equation 3 gives a spatial resolution of 0.133m (13.3cm). The width of the ICL is 0.5cm – 4cm and that of Triplets is 15cm – 20cm, as given in section 4 above. The ICL width is far less than less than the spatial resolution determined and will therefore not be detected using the 400 MHz antenna, only the Triplet chromitite will be detected.

7 Conclusions
The mapping of chromitite triplets above the UG2 reef has been shown to be viable by the theoretical evaluation conducted. This accession is in agreement with conclusions by Vogt et al. (2005), that GPR technology is applicable for the mapping of geological features in the BC.

The electrical properties of the host (Feldspathic Pyroxenite) and target (Chromitite) rocks are available and adequate for use in predicting the range and resolution performance of the technology. This data can also be useful for radar system calibration on site.

The following can also be concluded from the evaluation of radar range equations:
- The average attenuation value of Feldspathic Pyroxenite is 0.36 dB/m.
- The depth of the triplet package is within the effective depth range of the GPR system in Feldspathic Pyroxenite of 41.66m.
- The chromitite triplets and ICL will reflect adequate energy to be identified by the GPR system, this was indicated by the calculation of the Power Reflectivity, this is ensured by the adequate difference in the dielectric constant value of the Feldspathic Pyroxenite and the chromitite.
- The triplets will be identifiable using an antenna of 400 MHz central frequency, while the ICL will not be easily detectable due to their small width. The vertical resolution of a 400 MHz antenna in the Feldspathic Pyroxenite is 13.33cm, therefore all features with a width less than 13cm will not be easily identifiable.
References


Elandsdrift Fault Zone (Graben Structure),
Managing the Complexity & Geotechnical Challenges

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Abstract

Ground conditions are assessed ahead of mine tunnelling by analysing drill cores (surface & underground diamond drilling), projections from the geological mapping, stratigraphy and structural modelling. This paper elaborates on the methods employed to extrapolate the fault position towards the various mine levels to project the faults’ line-of-intersections. Additional geotechnical drilling was done to quantify and express the geotechnical nature of the rock mass affected by the Elandsdrift Fault zone. Furthermore, an analysis of the geological conditions on the different mining levels reveal a range of inhomogeneity for the nature of the structure. These findings formed the basis of understanding rock mass behaviour and allowed for the design of suitable support. The case study shows that geological features and hazards identified can be predicted through continuous updating of the structural model by accurate and efficient field data. Geotechnical logging is used to depict the quality of ground mass by means of normal distribution curves.

Keywords: BC, Bushveld Complex; MK2, Farm Middle Kraal 2; SJ100 & MG4, in-house resin product; RQD, Rock Quality Designation.

1 Introduction

The Rowland operation faces the challenge of a depleting ore reserve for both UG2 and Merensky reefs within its current shaft boundaries. However, exploration assurance and feasible studies motivate miners led to the extension of Rowland operations haulages beyond the Elandsdrift Fault into the mineral resource of the Middlekraal 2 block in preference to accessing via a vertical shaft, and thus optimise life-of-mine plan.

Geological underground diamond drilling is used to predict the exact positions and nature of both western and eastern limbs of the Elandsdrift fault planes on elevated haulages. Diamond drilling becomes an advantage when geotechnical information is needed and to seal water and gas intersections with grouting techniques to prevent corrosion of support steel. Heterogeneous rock mass were characterized by lithology, number of joint sets, joint roughness, joint alteration (shearing & filling) and joint water reduction from underground boreholes. Ground conditions
were then assessed through geotechnical logging (Rock Quality Designation, RQD) and Q values rating per cover borehole was determined. The information was used to distinct poor ground mass from good by utilizing rock quality distribution curves.

2 Location

![Figure 1](image-url)

Figure 1. The locations of Lonmin’s South African Mineral Resources.

Lonmin's entire South African Platinum Group Element (PGE) Mineral Resources and Mineral Reserves are contained within the Bushveld Complex, located in the Republic of South Africa. The Marikana operations are situated some 30 km East of Rustenburg in the North West Province, on the southern portion of the Western limb of the Bushveld Complex. Rowland shaft's and MK2 project Mineral Reserves underlies an area measuring approximately 8.1km on strike and 4.2km on dip with most mining occurring at depths of between 0.77km to 1.03km (32 Level) below surface.

3 Geological Setting

The Bushveld Complex intruded into the supercrustal sedimentary sequence of the Transvaal Supergroup 2.06 billion years ago, and is the largest known layered intrusion on earth, underlying an area of approximately 66,000 km². Bushveld PGE Mineral Resources are frequently disturbed by geological conditions which may result in losses to the Mineral Resource area. The areas affected are classified as geological losses, which are commonly caused by potholes, faults, intrusive dykes and Iron Rich Ultramafic Pegmatite (IRUP) and the Pilanesberg Alkaline Intrusion Complex.

Both the Merensky and the UG2 reefs are well preserved in the Critical Zone which is one of the four major stratigraphic zones in the RLS. The strata and reefs strike in an approximately east-west direction and generally dip between 8°N in the west, gradually increasing to 13°N in the extreme east of the property. The UG2 Reef underlies the Merensky Reef by between 130 m and 210 m, the middling between the two reefs gradually increasing across the mining right from west to east. The layered nature of the Bushveld Complex makes it possible to identify different lithological and stratigraphic units, which facilitates the interpretation of geological
disturbances such as dykes, faults, potholes and IRUP. At the Marikana Operations, the UG2 Reef normally comprises a narrow tabular chromitite layer, which varies in thickness between 1.0m and 1.4 m. The Rowland Shaft block area is bounded by 2 major fault zones striking in a SSE-NNW direction, namely the Marikana fault zone on the western boundary and the Elandsdrift fault zone on the eastern boundary. The Elandsdrift fault zone has an estimated down throw displacement of 50–120 metres to the east.

This graben structure has a notable NNW orientation (Pilansberg Age Trend) with strike lengths up to 30km. Towards the northern extend of the shaft boundary the western and eastern limbs of the fault are associated with doleritic intrusive material suggesting that these lineaments were active during the intrusion of the BC, (Basson, 2008) solidification of magma ‘bridging structure’ emplacement.
4 Elandsdrift Fault Zone (Graben Structure) and mine tunnelling

At present additional development has established haulages beyond the Elandsdrift fault zone from 18 to 27 level towards the eastern boundary of Rowland Shaft. The strike and dip of the western and eastern fault limb were mapped in the haulages and were plotted on a stereonet. The Elandsdrift fault zone strikes in a NNW to NNE direction with the western limb dipping to the east and the eastern limb to the west (ranging from 65 to 80°). The haulages were designed to develop through fault zone with a critical angle of > 60° in order to negotiate the altered fault lineaments with the least exposure. The haulages were designed with a standard dimension of 3.5 x 3.0 metres and cubbies every 90 meters to drill underground boreholes.

22 level east was the first level to develop through the eastern limb of the Elandsdrift Fault zone and continual cover drilling assisted in pre-empting fault plane positions ahead of the advancing haulage. Ground conditions were of poor quality and phase two consolidation process was initiated (3.0m Spiraling Bolts, 1.5m Oslo Straps, Steel Sets and Shotcrete application). The haulage dimensions on the fault position changed to 4.2 x 4.0 metres to accommodate the steel set installation and clearance for services in the haulage.

As multi-blasting continued the 24 level east also developed through the eastern limb, but with only phase one consolidation process (3.0m Spiraling Bolts, 1.5m Oslo Straps). At the time of writing this paper the 26 level east also exposed the the eastern limb but ground conditions were a bit more challenging since there is an association with a 15 metres thick ‘altered’ dolerite dyke. This dyke were probed by a number of boreholes, but most of these holes caved when the dyke was intersected. Subsequently this is indicative of an unstable intrusion (“running dyke”) for mining purposes and alternative development layouts were considered through geotechnical logging results (RQD/ Q – Values).

The haulages are positioned 45 meters below the Merensky reef and 100 meters above the UG2 reef within the ‘Graben structure’. The haulages which already developed through the eastern limb of the lineament developed above the UG2 reef in the anorthositic norite of the Merensky footwall. The 24 level east haulage already exposed the UG2 reef with the UG2A marker. The marker is approximately 30cm thick chromitite layer with a 20cm thick pyroxenite. The UG2A marker overlays the UG2 chromitite reef with thicknesses from 1.0 metres and measures up to 8.5m towards the east of the shaft block. The lack of cohesion between the chromite grains
result in parting and hangingwall instabilities. From the information observed in the boreholes no additional altered layers were observed above the UG2 chromitite or in the vicinity of the UG2A markers.

5 Hazards Associated with Faulting

5.1 Water
Unexpected water ingress during tunnelling usually escalates project costs and causes critical delays. Water intersections locations in the haulages were correlated with surface topography from rivers and infrastructure features such as tailing dams, etc. This method gave some indication to the origin of the water, whether from meteoric, ancient or unnatural drainage sources.

Understanding the water origin will assist with the management strategy. Therefore, geological underground drilling should be planned and scheduled around the project area of interest. Water bearing structures and fissures were exposed in the Elandsdrift fault zone with water flows between 3 000 l/h to 21 000 l/h in some water bearing structures. Ring cover drilling methods were used to measure water flows above 10 000 l/h and to develop the faces within 15 metres of the known water intersection. The face should always be stopped in competent ground as directed from the borehole logging data. Once ring cover holes are drilled it’s preferred to test interconnectivity of water flow along the fissure between the boreholes. Good continuity promotes more effective sealing, and provides certainty that the resin penetrates effective in the fissure around the boreholes.

![Figure 5. Cover drilling delays.](image)

One of the disadvantages observed is to avoid latex injection in long boreholes exceeding 30 metres. The reason is that the latex tends to coagulate in the borehole and fails to penetrate into the local narrow fissures for adequate sealing process. Therefore, latex resin (SJ100) application is used to consolidate the formation only if caving was experienced in the borehole and it should be used with MG4 polyurethane resin.
Since reduction of production downtime is critical faster and more efficient water sealing methods were considered. Fissures with water flow below 10 000 l/h were sealed from underground boreholes through with a combination of MG4 & Cement. The MG4 being a fine grained product would initially penetrate hair fractures at a low pressure (4 Mpa) into the surrounding rock mass. Secondly, water was pumped into the borehole to serve as a buffer and finally cement thereafter (MG4/Cement ratio, 2:1). The resin mix was allowed to set for 24 hours and re-drilling continued to confirm if all possible fissures were sealed. The successful seal will only imply that the water was displaced to another location, since water flow to the point of least resistance.

5.2 Alteration
From the core sample observed the fault zone is highly fractured and mineralogically altered, with secondary minerals (serpentinite, chlorite and/or calcite) present in joints and shears (see figure 6). The fault zone exposed this far no alarming volumes of flammable gasses but water are present with traces (> 1ppm) of hydrogen sulphide (H2S). Lukewarm water percolating from the drilled boreholes and surrounding surfaces with a fair concentration of salinity. These type of water will have the tendency to corrodes the steel of roof bolts and may affect the haulages life.

Figure 6. Images of secondary minerals observed in recovered core samples.

5.3 Jointing
Joints are geological discontinuities along which no visible displacement occurred. They commonly have infilling material, which defines their cohesion and friction properties. Joints usually occur in sets, a set being defined as a group of joints with common orientation. Two, three or four joint sets usually occur in a rock mass with one of the sets being predominant in intensity in a specific area. Sets of joints were mapped in random haulages of the study area in the Elandsdrift fault zone. The data set were columned according to their strike, dip direction
and plunge. The histogram were created according to the frequency of joint strikes observed versus the range in degrees (quadrants).

![Joints Orientation](image)

Figure 7. Histogram indicates the most abundant joint orientation (90°–180°) in the fault zone.

Four sets of joints were observed within the Elandsdrift fault zone. This classification is based on their orientation, dip and characteristics. They are:

- **J1 & J3**: SW-NE striking joints, approximately vertical. These joints are characterized with high frequency of talc-chlorite-sericite infilling and may be closely spaced, creating very blocky ground conditions.
- **J2**: NNW-SSE striking joints, dipping ± 90°: These are the most prominent joints. These joints have the same orientation as the Elandsdrift fault zone. Calcite infill were also observed in these joints.
- **J4**: ESE-WNW (“E-W”) striking joints, dipping 60°–80°: These joints are closely spaced and it seems that they are related to the joints set two because the also appears in the same quadrant.

### 5.4 Rock Quality Designation

On 22 level east two geological cover boreholes were drilled on the left-hand side and right-hand side of the haulage to pre-empt the eastern limb of the graben lineament. The borehole on the left of the haulage collard in anorthositic norite and advanced into the fault zone. The fault zone material were not competand and drilling couldn’t proceed due to continues caving of the borehole. The borehole was stopped and a second borehole commenced on the right-hand side of the haulage. This borehole also collard from the same footwall and advanced through the fault zone into the MK2 block.

Geotechnical logging was performed on these two boreholes (see figure 9) to determine the rock quality designation (RQD) by using simple geotechnical parameters introduced by Deere in 1963. The RQDs were measured from the core recovered and were applied to a quasi-quantitative rock mass classification system (Q-system by N. Barton, R. Lien and J. Lunde).

Although geotechnical and structural information is available, strategic decision making will still be a challenge without reasonable data interpretation methods. As a competent person in the mining industry we feel more confident to visualize data sets as to compare objectives with parameters.
The Q-value which derived from the geotechnical parameters were applied on a method to signify ‘bell-shaped curves’ for diverse strata control applications. Therefore, rock quality distribution curves were generated to distinct poor ground conditions from good in jointed or sheared rock formations. The rock quality distribution curves may be applied as a tool to classify stability and support estimates of mine tunneling.

The simulated distribution curves footprint depicts a positive skewed curve (Q-value < 5) for poor ground conditions and negative skewed curve (Q-value > 5) for good ground conditions in relation with the mean of a data set. The skewness of a distribution is a measure of the asymmetry of the probability distribution of a real-valued random variability about its mean (see figure 8). Not all bell-shaped curves are normal therefore the unique footprint of distribution curve maybe related to the degree of adverse ground conditions. Normal bell-shape curves exhibit the same relationship between the mean and standard deviation as stated by their mathematical expression. Underground boreholes 4V0501 and 4V0514 were drilled to explore the virgin ground around the fault in the direction of haulage.

The borehole on the right-hand side (borehole 4V0501) has a tendency to the right which implies that the core recoverd is of poor to intermediate ground conditions with a Q–value of 3.8 comparing to the distribution curves. The trendline in the scatter diagram decreases as a result of the fault zone of 61% frequency of poor ground mass.
Figure 9. Rock Quality Designation Distribution Curves.
The borehole on the left-hand side (borehole 4V0514) probed from poor quality ground mass into competent ground conditions beyond the fault zone position. The data obtained from the geotechnical parameters represents a Q-value of 1.82 which results in a positive skewed curve.

The results from both the boreholes are of poor ground conditions therefore direction of mining was taken into consideration and support standards were recommended accordingly. Because, the ground mass around the fault zones were of poor quality therefore phase one and two consolidation were recommended. Due to these ground conditions mine planning and scheduling will be delayed and expenditures will increase. Therefore it is vital for miners to reveal the expected ground conditions in order for them to plan accordingly.

6 Conclusion

The objective for this case study is to develop mining infrastructure through unfavourable ground conditions without any unplanned delay or lost time injury. To achieve this goal an inherent understanding and knowledge of the ore body and its associated ground conditions are essential to allow for accurate technical recommendations. This paper confirms that geological cover drilling is vital to execute the above mentioned objectives. The parameters of the inhomogeneity of the Elandsdrift fault were identified through geological cover drilling observations and geotechnical logging. It was learnt during the preparation of this paper that distribution curves can assist in identifying poor or good ground conditions.

References

Geotechnical Block Modelling for the 3-Dimensional Visualisation of Rock Mass Quality in the Mining Environment

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Abstract

The collection and analysis of geotechnical data forms the basis for understanding the geotechnical characteristics and the overall quality of the rock mass in a mining environment. This data in turn makes available a number of empirical methods for the evaluation of stability, the design of support and the selection of mining methods in underground and open pit operations, which allow safe mining to take place.

As rock mass quality is often presented as averages over large domains it can be difficult to form a visual impression of the quality of the rock mass. This paper therefore focuses on the creation of geotechnical block models that provide a 3-dimensional visual representation of rock mass data which estimates rock mass conditions (with varying levels of confidence) across the planned mining area. This concept is illustrated using case studies where geostatistics is adopted to estimate the rock mass quality between boreholes by applying the appropriate geostatistical methodologies.

From this study it can be concluded that while geotechnical block models can provide insight in the variability of the rock mass conditions in the mining area, these models should not be used in a prescriptive manner to design rock support on a local scale. Instead, a geotechnical block model should provide insight on areas where potential instabilities can occur, allowing for the opportunity to address these instabilities. Overall, once a geotechnical block model is created, it should also be updated on a continuous basis as more data is gathered as mining takes place.

Keywords: geotechnical block modelling, geostatistics, rock mass quality, geotechnical data
1 Introduction

A detailed understanding of rock mass conditions is essential for safe, productive mining to take place. To gain insight on the quality of a rock mass, boreholes are usually drilled, geotechnically logged and analysed prior to and during mining operations. During this process, data is often assessed using rock mass classification systems. While the results from the use of these systems provide an impression of the rock mass conditions, it can be difficult to form a 3D visual impression of the quality of the rock mass across the mining area. To account for this, spatial variability in rock mass data can be estimated and assessed using 3-dimensional geotechnical block models. This paper presents case studies where geotechnical block models have been created to allow for a 3-dimensional visual representative of the rock mass conditions, where the identification of data deficient areas and potentially poor ground conditions are outlined. Similar work has also been carried out by Jenkin and Seymour (2009), Bye (2006), Luke and Edwards (2004) as well as by other authors, which may also be used as a reference point when conducting 3-dimensional geotechnical block modelling.

2 Kipushi Geotechnical Block Model

Kipushi Mine (Kipushi) is a high-grade underground copper-zinc mine located adjacent to the town of Kipushi in the southern Haut-Katanga Province in the Democratic Republic of Congo. Kipushi is currently investigating the potential to mine a high grade zinc orebody known as the big zinc (MSA, 2016). Major lithologies in the mining area are sphalerite (orebody), the kakontwe dolomite formation (comprising of the upper, middle and lower Kakontwe dolomite) and the shales, siltstones and sandstones of the Grand Lambeau Formation, all of which fall within the Central African Copper Belt. For the Pre-feasibility stage of the project, a geotechnical block model was created for Kipushi, with the aim to determine the variability in rock mass quality across the project area and to identify gaps in the data set.

2.1 Rock Mass Quality

Data input into the Kipushi geotechnical block model is based on rock mass quality information which was determined with the use of Barton et al’s (1974) Norwegian Geotechnical Institute’s Q-System (Barton et al, 1974). This system was applied to a total of 90 geotechnical borehole logs which were identified across the project area. A Q value was determined for each geotechnical interval for every available borehole. As Q values are expressed on a log scale, all Q values were converted to rock mass rating (RMR) values using Barton’s equation \( \text{RMR} = 15\log(Q) + 50 \). RMR values range between zero and 100, whereby the higher the RMR the better the quality of the rock (Table 1). The converted RMR values were used to populate the block model. A histogram illustrating the rock mass classification results (uncomposited) is presented in Figure 1(left).

<table>
<thead>
<tr>
<th>RMR</th>
<th>Rock Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 20</td>
<td>Very Poor</td>
</tr>
<tr>
<td>21 – 40</td>
<td>Poor</td>
</tr>
<tr>
<td>41 – 60</td>
<td>Fair</td>
</tr>
<tr>
<td>61 – 80</td>
<td>Good</td>
</tr>
<tr>
<td>81 - 100</td>
<td>Very Good</td>
</tr>
</tbody>
</table>
Following the rock mass classification, a weighted averaging method known as compositing was applied to the data to produce geotechnical intervals of equal lengths, allowing for statistical analysis. This operation was performed using the software package, LEAPFROG. A compositing length of 3 m was chosen for the data as this was the typical core run length. Rock mass classification results based on the composited data are presented in Figure 1 (right) and Figure 2.

![Figure 1. Histogram of RMR from Q – not composited (left) and composited (right)](image)

2.2 Geotechnical Domains

On analysis of the rock quality across the project area, it was observed that overall the rock mass quality is lower to the north of the project area compared to the south (Figure 2). It was therefore decided to separate the data into two domains, domain A and domain B. As the poorer quality rock in the north may be due to the more fractured nature of the rock in the north (upper Kakontwe dolomite), the boundary between the middle and upper Kakontwe was used to separate the domains (Figure 3).
The distribution of RMR values for Domain A and Domain B are presented in Figure 4. Both domains share the bimodal negatively skewed distribution of the total dataset however; the first peak in domain A has a higher kurtosis than the first peak in Domain B. Furthermore, whilst the mean RMR values for Domain A and Domain B are similar (80 and 77), the distribution of RMR results illustrate that there is very little data with an RMR of less than 60 (RMR >60 = good rock) for Domain A (43 samples) compared with Domain B (872 samples). Domain A and Domain B were thus modelled separately to highlight areas with the poorer quality rock (Domain B) without distorting the good quality rock (found in Domain A) and to honour the observed differences across the middle to upper Kakontwe stratigraphy.

2.3 Geotechnical Model Creation
For the creation of the Kipushi geotechnical block model, use was made of the Datamine Studio RM and Isatis software packages. The process followed in creating the model is described briefly below.

2.3.1 Variograms
The anisotropy of the RMR values was assessed through a semi-variogram map, which showed moderate anisotropy. It was observed that the data has the longest range of continuity in the
vertical direction, and the shortest along the north-south axis. For the creation of the model, variograms were required and thus created in 3 orthogonal directions to gain an impression of the spatial continuity of the data across the project area (Figure 5). Based on these results, a variogram model was created for Kipushi (Table 2).

Figure 5. Modelled semi-variograms
Table 2. Variogram model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Domain</th>
<th>X</th>
<th>Y</th>
<th>Z</th>
<th>Nugget</th>
<th>Sill</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Parameter</td>
<td>Domain A</td>
<td>68</td>
<td>53</td>
<td>69</td>
<td>35</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Domain B</td>
<td>68</td>
<td>53</td>
<td>69</td>
<td>31</td>
<td>13</td>
</tr>
<tr>
<td>Second Parameter</td>
<td>Domain A</td>
<td>200</td>
<td>120</td>
<td>230</td>
<td>35</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>Domain B</td>
<td>200</td>
<td>120</td>
<td>243</td>
<td>31</td>
<td>39</td>
</tr>
</tbody>
</table>

2.3.2 Prototype

To create a block model, a model prototype is required. The prototype defines the location and dimensions of the block model prior to the adding data to the model. The block size is 5 x 5 x 5 m, which was chosen to match the grade estimation block model.

2.3.3 Statistical Approach

Two methods were employed for the creation of the block model:
- Nearest Neighbour
- Ordinary Kriging

To honour the data within the boreholes, the nearest neighbour method was applied to a 5 m radius from each sample. This method does not involve weighting sample values. Instead, each cell is assigned the value of the 'nearest' sample, where 'nearest' is defined as a transformed or anisotropic distance which takes account of any anisotropy in the spatial distribution of the RMR values.

Kriging is the geostatistical method for estimating the value of a volume and involves the assignment of weights to the surrounding data. The calculation of the kriged weights is based on the modelled semi-variogram, which describes the correlation between two samples as a function of the distance between them. One of the major advantages of kriging is that the weights are calculated in order to minimize the error variance. When minimizing the error variance, kriging takes into account the spatial location of the samples relative to each another. Hence, if several samples are clustered together, this will be taken into account when the weights are calculated and the weights reduced accordingly.

There are two variations of kriging i.e. ordinary kriging and simple kriging. For ordinary kriging, a weight is calculated for each sample, and the sum of these weights is 1. For simple kriging a weight is calculated for each sample and a weight of (1 - ΣW) is assigned to the mean, therefore the sum of the sample weights, plus the weight assigned to the mean equals 1. Simple kriging is not as responsive as ordinary kriging to local trends in the data, since it depends partially on the mean, which is assumed to be known, and constant throughout the area. Ordinary kriging is therefore the most commonly used method of kriging and was thus applied to the Kipushi data.

Ordinary kriging was applied to the Kipushi data using a three search pass strategy, where the distance from the data was incrementally increased for each search pass (Table 3). This was done to increase the smoothing of the block model as the distance from the data increased, while locally honouring the nearby data. The ranges chosen for each search pass was based on the variogram results (Table). For each search pass, a minimum and maximum number of samples to be utilised was defined. Note that where more than the maximum number of samples within search volume exist, the nearest samples are selected.
Table 3. Search pass parameters

<table>
<thead>
<tr>
<th>Search Pass</th>
<th>Range (m)</th>
<th>Minimum no. of samples</th>
<th>Maximum no. of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>60</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>6</td>
<td>20</td>
</tr>
</tbody>
</table>

2.4 Results

Figure 6 illustrates the confidence in the block model, which decreases as the distance from the boreholes increase. As there is no data available in the far east of the project area, this was not modelled. A horizontal section through the Kipushi block model, showing the estimated RMR values, is presented in Figure 7.

Based on the block modelling it was established that the rock mass conditions are generally good to very good, especially in the south of the mining area. The block model also highlights that there is little to no information to the east of the mining area. As footwall development is planned in the east, further drilling is recommended here to confirm the rock mass conditions in this area (Figure 2).

Figure 6. Block model confidence (Plan view at 1352 m)

Figure 7. Horizontal section at 1307 m
3 Platreef Geotechnical Block Model

Ivanplats (Pty) Ltd. has undertaken an investigation to assess the feasibility of developing a 4Mtpa vertical-shaft accessed underground platinum mine known as the Platreef project. The project is located on the Northern Limb of the Bushveld Complex in South Africa, near the town of Mokopane, approximately 280 kilometers northeast of Johannesburg. Geologically the Platreef is a complex PGE deposit subject to various processes over the course of its genesis. Major lithologies across the project are from the Upper Critical Zone stratigraphy which has been locally divided into the uppermost Norite Cyclic Unit (NC1), the Turfspruit Cyclic Unit (TCU), a footwall Norite Cyclic Unit (NC2), the UG2 (hangingwall chromitite and harzburgitic footwall) and the lowermost mafic and ultramafic magmatic units of the Lower Zone. The TCU hosts the two dominant ortho-magmatic mineralized zones (orebody). A major fault known as the Tshuduku fault also traverses the project area from the north to the south. For the feasibility stage of the project, a geotechnical block model was created for Ivanplats, with the aim to determine the variability of the rock mass quality across the planned mining area and to highlight the poor ground caused by the presence of the Tshukudu fault.

3.1 Rock Mass Characterisation

As discussed in Section 2.1, the Norwegian Geotechnical Institute’s Q-System was utilised to facilitate the derivation of Q values for the rock mass per geotechnical interval per stratigraphic unit. A total of 83 borehole geotechnical logs were assessed using this system and thereafter the Q values were converted to rock mass rating values (RMR) using the equation described in Section 2.1. The compositing process was accomplished using the software package Datamine Studio RM. A 10 m interval (compositing) length was applied to the data, as this was the block size chosen for the z-axis.

3.2 Geotechnical Block Model Creation

The following processes describe a summary of the development of the Platreef block model:

- Conversion of the Barton Q values into Rock Mass Rating values (RMR).
- Importing of the geotechnical borehole collar, survey and RMR data into the software package (Datamine Studio RM).
- Compositing (regularising) the RMR data within the borehole to 10 m lengths.
- Importing the Tshukudu fault wireframe to creates a zone of influence (poor ground).
- Defining the model extents based on the lithological wireframes.
- Assigning a RMR value of 20 (very poor ground conditions) to a 5 m zone around the Tshukudu fault wireframe and a RMR value of 40 (poor ground conditions) to a 30 m buffer zone around the fault wireframe.
- Estimating the RMR data within the model extents based in the inverse distance squared algorithm with a 3 pass estimation neighbourhood.
- Creation of the geotechnical block model based on the resultant data from the above processes.

3.2.1 Statistical Approach

The method employed for the creation of the Platreef block model was the inverse distance squared algorithm as outlined in the summary. Inverse Distance Squared Weighting is a type deterministic method for multivariate interpolation with a known scattered set of points. The values that are assigned to unknown points are calculated with a distance weighted average of the values available at the known points.

Anisotropic search ranges were chosen based on the orientation of the major structures in the area, as the expected maximum continuity of weak zones is anticipated to align with these. A block size of 20 x 20 x 10 m was chosen, and the blocks were informed in a three pass search strategy. The first pass was very restrictive, in order to ensure the estimates honoured the local data, using only 1 sample and a very short range (the nearest neighbour type estimate used in
section 2.2.3). The second pass utilised a longer range and a maximum of five samples in the estimate. All remaining blocks, not estimated in the first two passes were assigned a RMR value of 62 (the average RMR from our dataset). The confidence in the third pass is naturally low, as there is insufficient data to inform the estimates. Search pass parameters are presented in Table 4.

<table>
<thead>
<tr>
<th>Search Pass</th>
<th>Range (m)</th>
<th>Minimum no. of samples</th>
<th>Maximum no. of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td><strong>Assigned Average RMR = 62</strong></td>
<td></td>
</tr>
</tbody>
</table>

### 3.3 Results

A summary of the values determined using the statistical function imbedded into the Studio RM programme for RMR are presented as Table 5 for search pass 1 (highest confidence) and search pass 2 (medium confidence).

<table>
<thead>
<tr>
<th>RMR Search Pass 1</th>
<th>RMR Search Pass 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>65</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>8</td>
</tr>
<tr>
<td>Min</td>
<td>31</td>
</tr>
<tr>
<td>Max</td>
<td>82</td>
</tr>
<tr>
<td>Mean + Std Deviation</td>
<td>73</td>
</tr>
<tr>
<td>Mean – Std Deviation</td>
<td>37</td>
</tr>
<tr>
<td>Number of Samples</td>
<td>42 980</td>
</tr>
</tbody>
</table>

These search passes are also illustrated in Figure , and can be considered as a proxy for the confidence in the estimates. In contrast to the Kipushi model, the geotechnical data on the Platreef project are less densely clustered, and so the search passes appear more like the classic spotted dog. The diagram illustrate that the first and second passes represent reasonable confidence in the estimates, while the third search pass highlights areas that are poorly informed, and require additional data to model.

Based on the block modelling exercise, it is recommended that further drilling is conducted in areas where there is insufficient data. For the planning process, the geotechnical block model should be used to identify areas of “poor” ground to ensure that placement of permanent structures are avoided in these areas (eg. in the vicinity of the Tshukudu fault).
4 Conclusions

Geotechnical block models were successfully created for Kipushi Mine and the Platreef project to provide a 3-dimensional visual impression of the rock mass conditions in the planned mining areas. While these models provide insight on areas where potential instabilities may occur, such models should not be used in a prescriptive manner to design rock support on a local scale. Instead, they should be used to create awareness and provide the opportunity to address potential rock mass instabilities that each mine may be faced with during the excavation process. As the proposed mining at Kipushi and Platreef has not commenced, it should be noted that the block models serve only as a platform that should be continually built and improved upon as more data is gathered as mining takes place.

Based on this study it was determined that geotechnical block models may be utilised successfully for various mining applications that require a detailed understanding of the variability in rock mass conditions. Creating such models not only allow for the assessment of the spatial variability in the rock mass information, but in addition allows for the identification of data-deficient and high risk areas. The use of geostatistics with geotechnical datasets has also highlighted the specific challenges which come with geotechnical data such as a combination of background values (undisturbed rock mass) and planar features (such as faults and lithological boundaries) which require specific consideration and domaining.

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Support Strategies to Safely and Effectively Develop Through the Elandsdrift Fault Zone

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Abstract

Rowland Shaft is situated on the Western Limb of the Bushveld Complex and plays a vital role in the production profile of Lonmin’s Marikana mining operations. Being a second generation shaft, the majority of the development has reached the shaft boundaries with the ore reserves rapidly depleting. As a strategy to access the adjacent mineable reserves to the East utilising the current shaft infrastructure, off-reef haulages are planned to be developed through the 100m–120m down-throw NNW-SSE striking Elandsdrift Fault Zone (EFZ). With adverse ground conditions anticipated, due consideration must be made in order to develop and implement the most appropriate, yet cost effective support strategy. The support design should be versatile in order to combat the probable rock related hazards anticipated and should safeguard the development ends from instability for the life of mine. Geotechnical data collected were used to estimate the anticipated zone of influence and prevailing conditions at various stages of development. This paper discusses the challenges encountered as the haulages develop through the EFZ and the support strategies implemented to alleviate these challenges.

Keywords: Mining through fault zone, support strategies

1 Introduction

Rowland Shaft is critical in Lonmin’s sustainable production profile. With the current shaft’s ore reserves rapidly depleting, the ore reserves ahead of the shaft boundary will extend the life of the shaft if accessed with the existing shaft infrastructure.

Rowland Shaft is situated along the Western Limb of the Bushveld Complex. The Marikana operations are located approximately 80km West of Pretoria in the North West Province (refer to Figure 1). The Elandsdrift Fault Zone (EFZ) is a natural shaft boundary dissecting the mining operations occurring towards the East (Hossy and Newman Shafts) and West (Rowland Shaft) respectively (refer to Figure 2). The position of this prominent geological structure has been largely construed historically by both geological surface and underground drilling, underground mapping and aeromagnetic surveys conducted across Marikana operations. As mining advanced towards the fault zone, additional geological information gathered has led to a more definite understanding of the local position and nature of the zone of influence.
These excavations (haulages) form the linkage between the stoping excavations and the shaft infrastructure, for ventilation, the removal of ore and for the transport of personnel and material. The stability is therefore important to ensure a safe and effective mining process.

The stability of tunnels is largely defined by the rock mass environment; however, problematic ground conditions can be controlled with the application of effective support systems. The appropriate design and layout of tunnels (size, shape, orientation and excavation technique) within a particular geological environment can maximize the intrinsic stability of tunnels. Support and rehabilitation costs can therefore be kept to a minimum (Jager and Ryder, 1999).

The objective of this paper is to highlight the various challenges encountered during the haulage (tunnel) development phase through the EFZ and elaborate on the support and geotechnical strategies used to alleviate the various challenges. Some of the challenges include, and are not limited to, water and gas intersections, exposure of the fault and adverse ground due to increased joint density and sympathetic faulting. Developing through large areas of altered infill material including the unravelling dyke posed further challenges.
Levels 19, 24 and 26 are emphasized in order to demonstrate the different types of abnormalities experienced when the fault was exposed at different positions and the subsequent support implications in these areas.

2 Characterization of the EFZ

The EFZ is characterized as a graben structure. The Eastern Limb of the graben structure is defined by a prominent NNW-SSE striking normal fault dipping between 60-80 degrees towards the West. The downward displacement of the stratigraphic layers ranges between 40m-50m respectively. Along the fault zone, altered material consisting of chlorite, talc and calcite occur. The fault zone varies between 16m to 30m in thickness.

The Western Limb of the graben structure is characterized by a prominent NNW-SSE striking normal fault dipping at approximately 40-50 degrees towards the East. The infill material consists of brecciated material contained in the fault plane with altered forms of chlorite, talc and calcite similar to the Eastern Limb.

A 15m wide dolerite dyke striking NW-SE intruded the stratigraphic layers with a near vertical dip. The dyke intersected the EFZ on the Eastern Limb and when secondary re-activation of the fault structure occurred, the dyke was displaced towards the North in the region of 400m down dip from its original position.

The dyke structure at the fault intersection is severely fragmented due to the excessive movement and reactivation of the fault that occurred. A large amount of groundwater is present along the dyke.

3 Variations of Rock Conditions at Different Intersections

3.1 Western Limb area of influence

Intersecting the fault zone from the Western side focuses on 19, 24 and 26 level Haulages deliberating the different ground conditions and complications encountered during the development phase.

3.1.1 19 Level

19 East haulage (refer to Figure 3), is situated approximately 573m below surface. This Haulage experienced extreme challenges when it intersected the fault. Here, the fault zone has an estimated width of 28m. The most prominent joints observed were:

- J1: NNW-SSE striking and,
- J2: ESE-WNW striking.

Both joint sets have high inclination angles of between 60-80 degrees, with calcite and chlorite infilling in the joint planes. The deteriorated footwall unit is highly jointed and sheared with chloritized alteration (Barnard, 2016).

19 East Haulage was developed through Footwall 2 Norite up to the position of the Western Limb of the EFZ where Rowland Shaft’s boundary was intersected in 2002.

From borehole drilling during the prospecting phase, fault breccia was present in the core intersected in the vicinity of 19 Level Haulage. During the investigation of the fall of ground on 19 Level which occurred due to time dependent deterioration, the borehole information was re-evaluated and examined. The underground site investigation revealed striations and
brecciated material as observed on the borehole core which ultimately could be used as pre-cursors to the fault intersection.

Figure 3. 19 East Haulage re-establishment.

Support strategies in the form of steel arches with void filling were applied in the vicinity of the now collapsed face. The approach of obtaining areal coverage with a fair yield capacity and high load bearing capability in the form of steel arches with every 1m advance proved futile as the brecciated material continue to unravel. The excavation collapsed around and ahead of the support (Figure 4).

Figure 4. Fault zone collapse ahead of the steel arches on 19 East Haulage.

Further geological data was required to establish a more suitable position along this potentially unstable geological structure. The aim was to orientate the long axis of the excavation perpendicular to the strike of this prominent plane of weakness. This orientation would limit the length of intersection with the unstable secondary structures associated with this prominent geological weakness and subsequently maximize the stability of the excavation. History has shown that deformation in this fault zone without adequate support systems may result in rock mass failure, if left unsupported for a significant period of time.
Before strategic re-establishment of the haulage, all geological information (Figure 4 and 5) available (from surface and underground) were re-evaluated and utilized to obtain a more conclusive understanding of the rock mass characteristics, and the in this zone.

![White brecciated fault material from surface borehole core correlates to the material exposed underground.](image)

Figure 5. Surface borehole core, drilled into the Western limb (19 Level) of the EFZ

The Haulage was re-established South of the current face position. From cover drilling operations it is estimated that the thickness of the fault zone in this Western Limb is approximately 28m wide. Support strategies to account for the deficiencies in the first support design will be discussed in Section 4.

The dyke had no influence in this area as it is situated approximately 220m North from 19 Level.

### 3.1.2 24 Level Haulage

![Western Limb](image) ![Eastern Limb](image)

Figure 6. 24 East Haulage in relation to EFZ.

24 East Haulage is situated 753m below surface (Figure 6). Here the fault zone is approximately 4m wide (as illustrated in Figure 7 and 8), with slight alteration. Minor water dripping, but no
gas was intersected during the development phase. The host rock conditions were fair to good in relation to the other two haulages.

The fault zone was competent during drilling and blasting operations and no failure had occurred. The dyke had no influence on this area as it is situated further towards the North of the shaft.

Figure 7. 24 East Haulage Fault intersection

Figure 8. 24 East cover-drilled core showing fault intersection.
3.1.3 26 Level

Figure 9. 26 East Haulage illustrating re-establishment

26 East Haulage (Figure 9) is situated 825m below surface. The 15m wide NW-SE Dolerite dyke (Figure 10 (a) and (b)) contributed to the adverse ground conditions intersected in the vicinity of the fault. The dyke fragmented due to the re-activation and shear movement along the EFZ across a large area and displaced the dyke towards the North by a distance of 200m. Dyke material (Figure 11) was observed to be within the fault zone and, as it was sheared along the fault plane during secondary reactivation resulted in blocky ground conditions in the vicinity of the dyke. The properties of the dyke material in close proximity of the fault can be described as that of a unravelling dyke. The visual appearance of the dyke is clay-like and when in contact with water, it unravels totally.

Should Rock Mechanics principles be followed, by intersecting the dyke at an acute angle, this Haulage will then remain in the dyke for a long period of time. The haulage has to be turned towards the North in order to target the shortest distance through the dyke. This will cause the haulage to be developed through the area of the dyke which has been sheared along the fault plane.

Excessive water and gas were intersected during both the core drilling and development phase. The acidity levels of the water are exceptionally high as corrosion on steel and galvanized items were present.

Figure 10. 26 East Level fault Intersection a) and Dolerite Dyke intersection b)
4 Support Strategies

4.1 19 Level Haulage

Due to the unravelling of the Haulage, which resulted in total closure, the initial decision was to change the direction of the Haulage towards the South, parallel to the fault structure. From this Haulage, core drilling was conducted in order to determine both the extent of the fault zone and the rock condition, within the fault zone. Once the position had been obtained from core analysis, the Haulage was turned and developed through the fault zone.

To combat the self-mining challenges, spiling bolts were installed at a $60^\circ$ angle ahead of the face position. Resin was injected into the holes containing the spiling bolts, to permeate into the fractured rock to seal and subsequently re-inforce the strength of the rock mass in the vicinity.

A layer of 50mm thick shotcrete was applied after every blast for areal coverage on both the hanging wall and sidewalls down to the footwall of the Haulage. Steel sets which have a high load bearing capability were installed 1m apart in the direction of mining. Void filling was installed above the sets for two reasons, firstly the steel sets are not flush with the hanging wall
and secondly, should sagging of the rock occur, the cement based void fill will act as a stiff yielding type of support and tolerate the load.

4.2 24 Level Haulage
The influence of the fault zone including the infilled material was negotiated with lesser support than both on 19 Level and 26 Level. The fault infilling was supported by means of steel mats pinned with 3m long spiling bolts injected with grout. The spiling bolts were install 0.5m apart and installed 90° to both the hanging wall and sidewall.

4.3 26 Level Haulage
As both knowledge and experience have been obtained during the first attempt to develop through the dyke with no success, rock consolidation has been introduced during the second attempt in the Haulage. The rock consolidation process was introduced which commenced 6m before the dyke intersection. The consolidation process comprised of steel sets spaced 1m apart installed in the direction of mining up to the face position, cementitious void fill had been installed above the steel sets flash to the hanging wall.

Blasting was conducted by means of short 1m blast holes using low density explosives and smoothex on the perimeter holes. Spiling bolts with resin injection had been installed in the full perimeter of the development end at angles of 70° from the edge of the walls of the development end and 3m into the host rock. After every blast, the development end face was supported by means of 50mm thick shotcrete to seal the fragmented dyke material and to avoid it from unravelling into the space created by blasting. The support installation process was repeated which resulted in a time consuming, 4 day cycle blast process.

5 Water Interception
During prospect drilling on 19 East and 26 East Haulages, water was intersected at medium flow rates. This posed additional mining challenges which included the possibility of Haulages flooding and the inability of blasting using anfex, as anfex is highly soluble in water. The cover holes were sealed with concrete to divert the water to enable primary blasting agents in the form of Anfex to be used. The sealing of holes was very time consuming and created unplanned delays.

6 Conclusion
During the development phase and rock consolidation process through the EFZ it was realized that the amount of information available regarding the structure played a major role in decision making. Decisions regarding support application, change in mining layout and the level of risks which had to be negotiated were dependent on the information obtained as the development was in progress. Prospecting by means of core drilling provides minute information at a point in relation to the complex geological deformation.

It is essential to conduct accurate geotechnical core logging in terms of joint orientation, alteration, mineral composition, and overall rock mass characteristics. It may be used as precursors to the more prominent Geological structures and associated adverse rock mass conditions. This in turn would assist in the planning of support strategies and enable the costs, time and overall feasibility to be accommodated for.
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Abstract

It is essential to understand the properties and behaviour of rock mass, mainly in a hydrogeological context as well as various geotechnical applications. However, most of these processes occur as bulk fluid flow in the unsaturated zone and single fracture flow becomes imperative in understanding these processes, including, the identification of fluid flow mechanisms. The research presented in this paper investigates the influence of aperture, in a single smooth parallel plate model, on flow mechanisms under conditions of variable saturation. Visual experiments illustrate that liquid migration and to a lesser extent flow structures, are affected by varying the fracture aperture. The results indicate that the width and the smooth joint have a significant effect on the interaction between capillary and gravitational forces. Narrow apertures provide more contact between fluid and joint surface, thus, favouring capillary forces. In contrast, wider apertures provide less contact area between water and surface, favouring gravitational forces.

Keywords: fracture aperture, variable saturation, smooth parallel plate model.

1 Introduction

Improved understanding of variably saturated fracture flow in unsaturated rock masses has numerous implications, especially when considering the complexity, heterogeneity and anisotropy of the fractured vadose zone. Specifically, better quantification of groundwater flow has application in; permeability assessments for dam foundation rock masses, water inflow into deep mines, subsurface tunnels, and large rock cavern (hydro-electric and storage facilities, which need to be quantified to create safe operational conditions (Berkowitz, 2002). Previous research has illustrated that flow capacity of the fractured media depends on a few parameters such as the joint or fracture orientation, joint aperture, type of infill material, and continuity (persistence) of the fractures in the direction of flow (Eid, 2007). Moreover, aperture is cited as one of the most vital fracture properties governing flow (e.g. Barton and de Quadros, 1997; Hakami and Larsson, 1996).
The successful application of numerical methods have proved to produce accurate and reliable results. This is attainable through extensive simulations of non-equilibrium flow processes based on elaborate field data which aid in comprehending and determining pronounced effects of heterogeneities including the effects of fractures on flow (Jing, 2003). Equally important, are physical and empirical models, particularly single fracture (SF) experimental studies, primarily in the disciplines of hydrology and petroleum engineering. Though simplified, they have provided the basis upon which fracture flow behaviour is understood (Louise, 1951; Qian et al., 2005). Experimental models have generally been used as a basis to develop fundamental conceptual models of fracture flow due to the inherent complexities associated with rock masses (Lomize, 1951). Although, SF studies are limited by the number of variables permissible to be incorporated into an experimental model, which is normally in plane strain (i.e., 2D), noteworthy results have been gathered, providing practical insight on fracture flow. Examples of some of these studies include the illustration of scale-dependent hydraulic conductivity through experimental single fracture studies (Qian et al., 2007) and effects of isolated fractures on flow (Barton et al., 1995). Furthermore, the adoption of SF studies has successfully allowed for the identification of different fluid flow regimes in the unsaturated zone including flow regimes arising from two-phase flow systems. Adding to the already complex system are the influences of varying joint parameters, in particular – aperture, which conditions the fracture flow tortuosity and flow channelling (Hakami and Larsson, 1996).

The research presented in this paper aims at assessing the influence of different apertures on flow through a discrete, smooth, open, parallel discontinuity. The aim is achieved through the development of an experimental model tested at three different apertures, that of 0.18 mm, 0.5 mm and 2 mm. This allowed for the identification of possible flow regimes or flow structures associated with the varying aperture’s tested. The results are anticipated to provide valuable insight into more complex experimental models in order to further address the influence of fracture properties of variably saturated flow processes.

2 Materials and Methods

The model comprises of two rectangular plexiglass plates with dimensions 400 mm x 290 mm x 10 mm, as shown in Figure 1, to simulate a discrete, smooth, open, vertical fracture. Due to the choice material opted for, the model assumes an impermeable rock matrix, hence, excludes effects of matrix imbibition. Notwithstanding, the transparency of the model allows for the visual assessment of flow paths within the fracture. A water inflow container is placed at 1 335 mm from the base of the fracture, with the length of the inlet pipe measuring 1 310 mm. Food colouring is placed in the inflow tank in order to colour the water. In order to simulate the differing fracture apertures, the two plexiglass sheets are separated by plexiglass strips, which act as spacers. The thickness of the spacers dictates the 3 apertures tested, (i.e., 0.18 mm, 0.5 mm and 2 mm).

Tests are conducted by the introduction of water into an initially dry fracture to gradually wet the fracture. The amount of fluid entering the fracture is controlled manually by means of an adjustable valve illustrated in Figure 2. This is done under constant head conditions, with water flowing into the fracture through a 6 mm diameter point source. In order to control the flow rate of fluid into the fracture, the tap was rotated in order to gradually increase the amount of fluid flow from 13% at position 1, to 100% at position 8. The position of the tap also served to represent each test that is conducted per experiment (e.g. Position 1 = Test 1) for the experiment run. Each test is summarised in Table 1. Due to small differences in the inflow velocity for the assumed flow percentages for the different test, and in order to highlight noticeable differences in the experiments, only the Tests 3, 5 and 8 are discussed. A grid is placed on the outside of the opposite Plexiglass sheet, facing inwards, so that the geometry of the flow mechanisms can be assessed.
Figure 1. Experimental set-up.

Figure 2.  a) Valve with 6 mm diameter openings on either side.  
b) Top view of the valve further illustrating the numbering of the ridges. The rectangular red blocks served as points to where the ridge needs to align when opened.

Table 1. Ridges corresponding to flow rate, $Q$.

<table>
<thead>
<tr>
<th>Position on Tap</th>
<th>% Flow assumed</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>38</td>
<td>3</td>
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<td>4</td>
<td>50</td>
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<td>75</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>88</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>100</td>
<td>8</td>
</tr>
</tbody>
</table>

A video camera is placed perpendicular to and approximately 1 m from the fracture, to record each experiment. Each experiment is initiated by commencing recording on the camera. Thirty (30) seconds thereafter, the tap on the valve is rotated to each position. Each test was concluded once a 2l of fluid had passed through the fracture. Once the experiment is competed the recordings are analysed and snapshots taken. Each image is annotated by: test number; aperture;
and time into the test that the snapshot was taken. Between subsequent experiment, the experimental materials were separated and dried.

3 Results

3.1 0.18 mm Aperture Fracture

The results of each test on the experiment containing the 0.18 mm aperture fracture are shown in Figure 3, 4 and 5 illustrating fluid flow through the fracture. At the lowest flow rate (Test 3), a wide wetting front is seen emerging from the inlet source, moving vertically through the centre of the fracture. Simultaneously, break-away rivulets are observed as fluid flows laterally from the inlet. In some instances, localised rivulets breakaway from the main fluid pool and creates new isolated pathways. Similar observations are observed during Test 5 with an increase in the flow rate, as shown in Figure 4. A wide fluid pond bulges out from the point source, subsequently forming different sized break-away rivulets with the main neck-sized rivulet confined to the centre. Some break-away rivulets travel laterally towards the boundary walls of the fracture. At the highest flow rate of Test 8, as shown in Figure 5, these lateral break-away rivulets are wider, resulting in an increased percentage of the fracture being saturated.

Figure 3. Water flow movement at 38% flow (Test 3).

Figure 4. Water flow movement at 63% flow (Test 5).
3.2 0.5 mm Aperture Fracture
During Test 3, a narrow rivulet emerges from the point source with flow occurring directly down the centre of the vertical fracture, as illustrated in Figure 6. Some minor break-away rivulets emerge at the base of the fracture, saturating upwards.

However, with an increase in the flow rate, a wider plume forms from point source during Test 5 and Test 8, with a larger percentage of the fracture saturating and some break-away rivulets observed in unsaturated portions of the fracture. Faster fluid flow is observable on the edge of the plume and along some of the smaller rivulets. In addition, a network of rivulets emerges upwards from the base of the fracture. This occurs as a phreatic surface is created due to faster inflow conditions than outflow of fluid at the base of the fracture.
3.3 2 mm Aperture Fracture

The experiment conducted at the lowest flow rate (Test 3) on the widest aperture (2 mm) show a thinner rivulet emerging, flowing in an abrupt and sinuous manner (Figure 10). Throughout this flow rate, the rivulet is unstable, and continues to oscillate in this sinuous manner. Similar flow behavior is also seen in Test 5 (Figure 10) and Test 8 (Figure 11) as the flow rate is gradually increased. Throughout these increased flow rates the thin rivulet remains unstable and oscillates aggressively.

Figure 9. Water flow movement at 38% flow at an aperture of 2 mm (Test 3).

Figure 10. Water flow movement at 63% flow at an aperture of 2 mm (Test 5).
4 Analysis

Darcy and Cubic Law equations, e.g. Singhal and Gupta (2010) were used to calculate the Linear Flow Velocity as well as the Hydraulic Gradient, $K$, for each test per model experiment. The results are presented in Figure 12 and 13 respectively. Quicker water outflow is expected through fractures with narrower apertures, as shown in Figure 12. Which corresponds to water either filling up the fracture bottom or the fracture centre. Conversely, a greater amount of water flow is expected in wider fractures, as shown in Figure 13.

Figure 11. Water flow movement at 100% flow at an aperture of 2 mm (Test 8).

Figure 12. The change in linear flow velocity with a change in aperture.

Figure 13. The influence of aperture ($e$) on fracture conductivity ($K_f$).
5 Discussion

Narrow aperture experiments (0.18 mm and 0.5 mm) are characterised by plume formation or water bulging. In some instances, the narrowest aperture fracture (0.18 mm), is characterised by ponding at the base of the fracture, whilst in the less narrow aperture fracture (0.5 mm), water flow is confined to break-away rivulets forming from the main plume. Upon an increase in flow rate, there is an increase in the percentage of the fracture that is saturated and in each experiment water outflow is confined to the position of the plume or rivulet.

In the wider 2 mm aperture experiments, the rivulet travels in a sinuous manner, with the flow path/position continuously oscillating. It continues to advance vertically with numerous liquid snaps within the fracture. As opposed to the narrower aperture, no break away rivulets emerge from the main rivulet.

The visual experiments mainly illustrate the relationship between capillary, gravity and viscous forces which are a function of the fracture surface and aperture. These are consistent with the observations of Tokunaga and Wan (1997) as well Sue et al (1999). In this study, the aforementioned forces rather influence the liquid flow migration and much lesser the resulting flow structures. The plume forming in the narrower apertures is a result of capillary forces dominating over gravity forces. This is due to the availability of a greater surface area which translates to a wider portion of the fracture being saturated. Notwithstanding, no liquid snaps are seen from rivulets travelling through the fracture, which is suggestive of the rivulets supplied by high flow rates as presented by Sue et al (1999).

As the available surface area is reduced through increase in aperture, capillary forces gradually become ineffective and gravity forces begin to dominate, hence the unstable, aggressively sinuous flows and numerous rivulets snaps or cessations observed in wider aperture fracture experiments under different flow rates.

6 Conclusions

The observations presented in this study are limited to an over simplification of natural conditions. Notwithstanding, the results provide further basis for future physical models that ultimately seek to investigate unsaturated flow mechanisms and flow regimes. Further research needs to investigate the influence of other joint properties in order to ultimately make the model more representative of natural fracture conditions.

The visual experiments in this study illustrate that the width and orientation of the smooth joint have a significant effect on capillary and gravitational forces. Narrower apertures provide more contact between fluid and joint surface, thus, favouring capillary forces. Whereas, wider apertures provide less contact area between water and surface, favouring gravity forces.

References


Yield and Critical Strength Ratios for South African Gold Tailings

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Abstract

A database of 69 laboratory triaxial compression tests on South African tailings was collected to determine the yield and critical strength ratios. Only tests results of specimens that showed contractive behavior was included in this study. The data used in this study yielded yield and critical strength ratios that are close in approximation to those calculated in previous laboratory tests and back-calculated from liquefaction flow failure case histories. It was found that the data used in this study confirms the yield and critical strength ratio concepts. Furthermore, this paper provides a database of yield and critical strength ratios for South African gold tailings.

Keywords: undrained shear strength, liquefaction, triaxial tests, yield shear strength ratio, critical shear strength ratio, critical state

1 Introduction

Over the past few decades research on static liquefaction has grown tremendously. Soils that are subjected to liquefaction are commonly subjected to various modes of shear (see Figure 1). Laboratory tests are commonly used to model these modes of shear. These laboratory tests includes the triaxial compression test, the triaxial extension test and the direct shear test. Olson (2001) proposed a yield strength ratio based on liquefaction case history analysis to evaluate the potential of liquefaction in a sandy, contractive soil. Olson and Stark (2002, 2003a) back-calculated numerous case histories where liquefaction has occurred to estimate the yield and liquefied shear strength ratios mobilized during failures. The liquefied shear strength, referred to as the critical shear strength in this paper \((s_u(crit))\), is the shear strength mobilized at large deformations after liquefaction is triggered in a saturated, contractive, sandy soil (Olson, 2001). The yield strength, \(s_u\) (yield) is the peak (maximum) shear strength available during undrained loading of a saturated, contractive, sandy soil (Olson and Stark, 2003a).
Drainage conditions are prohibited in the laboratory, therefore the term “undrained” applies. However, drainage condition cannot be prohibited in the field and many flow failures experience drainage (Stark and Mesri, 1992). Yield strength ratios and critical strength ratios are calculated by normalizing the strength with the vertical effective stress at failure, $\sigma''_o$, (Olson 2001). However, in the laboratory the yield strength ratio are calculated by determining the peak shear strength of the sample and normalizing it with the consolidation effective stress, $\sigma'_c$, (Olson, Mattson 2008). The critical strength ratio on the other hand is calculated by determining the critical shear strength and normalizing it with the consolidation stress, $\sigma'_c$, (Olson, Mattson 2008). There are many yield and critical strength ratios for different sands reported by various researchers. Hanzawa (1980) found the typical $s_u$ (yield)/$\sigma'_c$ values determined from isotropically consolidated triaxial test for samples from the Persian Gulf approximately equal to 0.31. Hanzawa also measured $s_u$ (yield)/$\sigma'_c$ values for Kisarazu, Valgrinda and Sengenyma sand tested in triaxial compression to be in the ranges of 0.18-0.32, 0.11-0.27 and 0.23-0.41, respectively.

Olson and Stark (2003b) compiled a database of triaxial compression tests to evaluate the concepts of the yield and liquefied shear strength ratios. The data generally supported these concepts. Furthermore, Olson and Mattson (2008) collected more triaxial test data on contractive soils and determined the yield and shear strength ratios. Again these ratios corresponded to the back-calculated strength ratios which Olson and Stark (2003) calculated. However, the strength ratios calculated from triaxial compression tests generally plot above the upper range of case history data. The main purpose of this paper is to evaluate whether the yield and critical strength ratio concept proposed by Olson (2001) is applicable on South African gold tailing. This paper provides a brief theoretical background for yield and critical strength ratios from triaxial tests. This paper also provides typical yield and critical strength ratios for South African gold mines.

2 Terminology and concept discussions

A soil will exhibit one of three stress-strain responses when it is subjected to static loading indicated in Figure 2. Castro (1969) defined these three responses as follows: type A response (purely contractive), upon loading, the sample will undergo strain-hardening until the sample reaches a peak (yield) shear strength, after this yield shear strength is mobilized, the sample will undergo strain-softening to a minimum shear strength, also called the critical shear strength or steady state shear strength (Poulos, 1981).

- Type A response can be described as a contractive/liquefaction/strain softening behavior and occurs in loose sandy soils, it is most susceptible to liquefaction. Defined as a soil mass which displays an increase in pore water pressure reaching a constant value and displaying a peak in deviator or shear stress within a few initial percent of axial strain followed by a
rapid and sudden decrease to a constant residual strength (critical strength) (Papageorgiou, 2004).

- Type B response can be described as a contractive then dilative behavior that occurs in loose sands. It is referred to as limited liquefaction. It is a similar response to type A response, but instead of undergoing strain-softening while exhibiting the critical shear strength, the sample will undergo strain-hardening. Type B response can be defined as a soil which displays a temporary decrease in shear strength followed by an increase in shear strength as the axial strain increases (Papageorgiou, 2004).

- Type C response (dilative) is a strain-hardening behavior which reaches neither a yield nor a critical shear strength. This response is generally not susceptible to liquefaction. Type C behavior are generally observed in dense sands. Type C response can be defined as a soil mass which displays an initial increase followed by a continuous decrease in pore water pressure and a continuous increase in shear strength with increasing strain (Papageorgiou, 2004).

This study is limited to Type A laboratory response.

Figure 2. Type A, B and C undrained stress-strain response in sands (from Verdugo, 1992).

Where $e$ is the void ratio after consolidation; $q$ is the deviator stress; $\sigma^\prime_{1c}$ is the consolidation effective stress; $s_u$ (min) is the critical shear strength; $s_u$ (yield) is the peak shear strength. When a contractive soil is sheared under undrained conditions, the effective stress decreases as the soil attempts to contract under a constant volume or constant void ratio ($e_c$). The soil will reach a peak shear strength referred to as the yield shear strength, $s_u$ (yield). This yield shear strength represents the liquefaction flow failure triggering condition (Olson and Mattson, 2008). When the yield shear strength is reached and straining continues, excess pore pressure increases at a high rate, consequently strain-softening occurs until no volume change is longer possible. This is called the ‘critical state’, the soil strains at a constant volume, constant shear stress and constant effective stress, as described by Casagrande (1936).

The initial fabric of a soil has a great influence on the behaviour of that soil. There are different methods for reconstitution of soil samples for laboratory tests such as tamping methods, dry funnel deposition and water sedimentation, slurry deposition and mixed dry deposition and air pluviation method. Therefore, the sample preparation of a soil sample is a critical factor that will affect the way that a soil sample behaves during triaxial compression. Mulilis et al. (1977) as well as Taksuoka et al. (1986) show that the mode of reconstitution of a sand sample has a great influence on its resistance to cyclic shear and liquefaction properties. Canou (1989) and Vaid et al. (1999) found that the moist tamping technique seems to favour the initiation of liquefaction instability under monotonic shear with respect to dry pluviation. Chang (2009) concluded that neither moist tamping nor slurry deposition can fully replicate the behaviour of undisturbed samples. Chang (2009) recommended that slurry samples be used rather than moist tamped samples as the slurry samples replicate the behaviour of undisturbed samples better.
than moist tamped samples. Chang (2009) show that gold tailings specimens prepared using moist tamping show contractive behaviour where the same gold tailings specimens prepared using slurry deposition and the undisturbed gold tailings specimens showed dilative behaviour. However, Papageorgiou (2004) stated that the condition of the critical state of a sand is uniquely determined irrespective of the initial soil fabric created by different methods of sample reconstitution.

In the isotropically consolidated triaxial compression tests data collected for this study, the mean consolidation stress, \( \sigma'_{\text{mean,c}} = (\sigma'_{1c} + \sigma'_{2c} + \sigma'_{3c})/3 \), is equal to \( \sigma'_{1c} \), where \( \sigma'_{2c} \) and \( \sigma'_{3c} \) are the intermediate and minor principle stresses and \( \sigma'_{1c} \) is the major principle consolidation stress (Olson and Mattson, 2008).

Olson et al stated that the yield strength ratio and critical strength ratio can by calculated from isotropically consolidated undrained triaxial compression tests as follows:

\[
\frac{s_u(\text{yield})}{\sigma_c} = \frac{(\sigma'_{1c} - \sigma'_{2c})_{\text{yield}}}{2\sigma'_{\text{mean,c}}} \quad (1)
\]

and

\[
\frac{s_u(\text{crit})}{\sigma_c} = \frac{(\sigma'_{1c} - \sigma'_{2c})_{\text{crit}}}{2\sigma'_{\text{mean,c}}} \quad (2)
\]

Olson (2001) proposed a relationship to estimate the yield and critical shear strength ratios, using normalized cone penetration tests (CPT):

\[
\frac{s_u(\text{yield})}{\sigma_v} = 0.205 + 0.0143(q_{c1}) \mp 0.04 \quad (3)
\]

and

\[
\frac{s_u(\text{crit})}{\sigma_v} = 0.03 + 0.0143(q_{c1}) \mp 0.03 \quad (4)
\]

Where the liquefied shear strength is equivalent to the critical shear strength used in this paper and \( q_{c1} \) is the corrected tip resistance and has units of MPa.

The critical state line (CSL) can be defined as the locus of void ratio and effective stress pairs at critical state, which means when the soil has reached a minimum resistance. Casagrande (1936) expressed this relation as follows:

\[
e_{cs} = \Gamma_{cs} - \lambda_{cs}\log(\sigma'_{cs}) \quad (5)
\]

Where \( \Gamma_{cs} \) is the intercept at 1 kPa on the \( e \) - \( \log(\sigma'_{\text{mean}}) \) curve and \( \lambda_{cs} \) is the slope of the CSL. Been and Jefferies (2006), amongst others, stated that the critical strength ratio are related to the critical-state line (CSL) as shown below:

\[
\frac{s_u(\text{crit})}{\sigma_c} = 10^{-\psi_{cs}/\lambda_{cs}\sin(\phi'_{cs})} \quad (6)
\]

Where \( \psi_{cs} = e_c - e_{cs} \) as described by Been and Jefferies (1985) as the state parameter. A negative state parameter corresponds to dilative behaviour and in contrast, a positive state parameter corresponds to contractive behaviour. The larger the value of \( \psi_{cs} \), the more strain-softening a sample will undergo as the soil approaches the CSL (Been and Jefferies, 1985). As indicated in Equation 6, the critical strength ratio will decrease with an increase in state parameter. When a soil mass is loose the initial state of a soil is above the CSL it will tend to contract during shearing (reduce in volume). When a soil mass is dense and the initial state of the soil is below the CSL, it will tend to dilate during shearing (increase in volume). Figure 3
indicates that the state of a soil is a function of the void ratio and confining pressure prior to shear.

Figure 3. stead state line concept and behavior of loose and dense specimens under undrained and drained conditions (adopted from Kramer 1996). a) is arithmetic scale and b) is on a logarithmic scale.

3 Triaxial database for South African gold tailings

George Papageorgiou (2004) performed triaxial compression tests on Merriespruit gold tailings. Information about the sample sizes, sample gradings and testing procedure can be found in his Ph.D. thesis “Liquefaction Assessment and Flume Modelling of the Merriespruit Gold and Bafokeng Platinum Tailings”. Papageorgiou (2004) tested four gradings of Merriespruit gold tailings in the triaxial compression tests. The tailings were obtained from the Merriespruit flow failure scar. The original tailings (referred to as Merriespruit – 60% fines) were sieved to three additional gradings: Merriespruit - 0% fines, Merriespruit - 20% fines and Merriespruit - 30% fines. Figure 4 represents the grading curves for the Merriespruit samples.

Figure 4. Grading curve for the Merriespruit samples, (Papageorgiou, 2004)
Papageorgiou (2004) found that the moist tamping technique was the most suitable sample preparation technique for liquefaction investigation. All the samples showed contractive behavior and were therefore selected for this study. There are a total of 45 test results on gold tailings obtained from Papageorgiou. An additional 24 test results were obtained from the University of Pretoria (UP) on various gold tailings where all samples exhibited contractive behavior. All tests were isotropically consolidated and undrained with pore-water pressure measurements. The test data obtained from UP did not have sample gradings available.

The suffix GT after each test number in Table 1 indicates gold tailings. For each test, the following data were collected where possible: end-of-consolidation void ratio (\(e_c\)), state parameter (\(\psi_{c}\)), major and minor effective stresses after consolidation, yield shear stress (\(s_u(\text{yield})\)) and the critical shear stress (\(s_u(\text{crit})\)).

The database are big and to conserve space the raw data from the triaxial tests done by Papageorgiou (2004) is referenced and omitted from this paper. However, the data obtained from UP cannot be referenced and is included in this paper. Table 1 summarizes the specification of the triaxial compression test results obtained from UP.

The samples from the data obtained from UP, were prepared by the compaction method (CM) or the moist tamping (MT) method. Table 1 indicates the preparation method for each test. The cylindrical triaxial compression test specimens were 50.88 mm in diameter and 101.6 mm tall. A back-pressure saturation procedure recommended by Bishop and Henkel (1962) was followed on all samples to dissolve any remaining air and saturate the specimen until a Skempton B value of at least 0.97 was observed. Figure 3.3 presents the stress paths and stress-strain plots for tests 7GT, 8GT and 9GT.

The stress paths are constructed in t-s’ space where \(t = 0.5 (\sigma_{1c} - \sigma_{3c})\) and \(s' = 0.5 (\sigma_{1c} + \sigma_{3c})\). The peak strength and critical strengths are indicated on the stress path. The stress-strain plot is constructed using the deviatoric stress (q) where \(q = \sigma_{1c} - \sigma_{3c}\) and the axial strain is presented as a percentage of the initial sample height.

![Stress Path Curve in t-s’ space](image)

![Stress-Strain Curve](image)

**Figure 5.** a) Stress Path and b) Stress-Strain diagrams for 7GT, 8GT and 9GT.

The stress paths shown in Figure 5 a) all indicate contractive response. From Figure 5 b) it is evident that after the yield shear strength was mobilized, strain-softening occurred until the critical shear strength was reached. All the specimens in Table 1 showed similar behavior. To calculate the yield strength ratio for 9GT, the peak shear strength (\(s_u(\text{yield})\)) was normalized by the major consolidation effective stress (\(\sigma_{1c}\)). Yielding a yield strength ratio of 0.38. To calculate the critical strength ratio for 9GT, the critical shear strength (\(s_u(\text{crit})\)) was normalized by the major consolidation effective stress (\(\sigma_{1c}\)). Yielding a critical strength ratio of 0.22.
Table 1. Specification of the triaxial compression test results obtained from the University of Pretoria.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>( \sigma_{1c} ) (kPa)</th>
<th>Preparation Method</th>
<th>( e_c )</th>
<th>( S_u(yield) ) (kPa)</th>
<th>( S_u(crit) ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1GT</td>
<td>50</td>
<td>CM</td>
<td>0.82</td>
<td>15.1</td>
<td>7.5</td>
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</table>

3.1 Yield strength ratios from laboratory data

The laboratory test database discussed in the previous paragraphs are used to calculate the yield shear strength ratios for each test by using Equation 1. Figure 6 presents the calculated yield shear strength ratios for the gold tailings database collected from the University of Pretoria. The yield strength ratios for these specimens ranged from 0.12 to 0.39. Figure 7 presents the calculated yield shear strength ratios for the gold tailings database available from Papageorgiou (2004). The yield strength ratios for these specimens ranged from 0.1 to 0.35. In Figure 7 it is evident that the 0% fines specimens generally yielded the highest yield strength ratio when a trend line was plotted through those data points. It is also noticeable that the grading has an influence on the yielded yield strength ratios. The influence of percentage fines are not included in the scope of this study. However, Youcef et al (2016) found that the peak shear strength of a soil specimen tend to decrease with an increase in fines content at the same over-consolidation ratio. Youcef et al (2016) concluded that liquefaction resistance of a soil mass decreases with an increase in fines content. These findings by Youcef et al (2016) are supported by various other researchers (Shen et al, 1977, Troncoso and Verdugo (1985)).
3.2 Critical strength ratios from laboratory data

As with the yield shear strength ratio, the laboratory test database are used to calculate the critical shear strength ratios for each test by using Equation 2. Figure 4.5 presents the calculated critical shear strength ratios for the gold tailings database collected from UP. The critical shear strength ratios for these specimens ranged from 0.02 to 0.26. Figure 4.6 presents the calculated critical shear strength ratios for the gold tailings database available from Papageorgiou (2004). The critical shear strength ratios for these specimens ranged from 0.02 to 0.19.
Summary of yield and critical shear strength ratios

Table 2 provides a summary of the calculated yield and critical strength ratios for the different tailings tested in the triaxial compression tests used in this study.

As discussed in section 2 of this paper, Olson (2001) proposed a relationship between the yield strength ratio and corrected tip resistance from CPT testing. Similarly, Olson (2001) also proposed a relationship between the liquefied shear strength ratio (the same as critical shear strength ratio in this paper) and the corrected tip resistance. See Equations 3 and 4 respectively. Louis Alberto Cruz (2001) stated that the $q_{c1}$ value for the Merriespruit dam was equal to 0.92 MPa. By utilizing Equation 3 and 4 respectively as proposed by Olson (2001) the back-calculated yield strength ratio range for Merriespruit is 0.18 – 0.26 and the critical shear strength ratio range is 0.01 – 0.1. Olson and Mattson (2008) published yield and critical strength ratios calculated using triaxial compression tests and field case histories and are also given in Table 2.
Table 2. Summary of yield and critical strength ratios from triaxial compression tests used in this study and collected from Olson and Mattson (2008).

<table>
<thead>
<tr>
<th>Laboratory database</th>
<th>Yield strength ratios</th>
<th>Critical strength ratios</th>
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<tbody>
<tr>
<td>University of Pretoria gold tailings</td>
<td>0.12 – 0.38</td>
<td>0.02 – 0.26</td>
</tr>
<tr>
<td>Merriespruit gold tailings</td>
<td>0.1 – 0.35</td>
<td>0.02 – 0.19</td>
</tr>
<tr>
<td>Back – calculated from CPT results</td>
<td>0.18 – 0.26</td>
<td>0.01 – 0.1</td>
</tr>
<tr>
<td>Previous laboratory data (Olson and Mattson, 2008)</td>
<td>0.18 – 0.43</td>
<td>0.01 – 0.23</td>
</tr>
<tr>
<td>Field case histories (Olson and Mattson, 2008)</td>
<td>0.23 – 0.31</td>
<td>0.05 – 0.12</td>
</tr>
</tbody>
</table>

4 Interpretation and discussions

Olson and Mattson (2008) collected 386 individual laboratory tests, performed almost exclusively on reconstituted specimens, to calculate the yield and critical shear strength ratios. Olson and Mattson (2008) found that the range for the yield shear strength ratios from triaxial compression tests were 0.18 – 0.43. Similarly, the critical shear strength ratios yielded a range of 0.01 – 0.23. These ranges all correspond to type A behavior of the samples. The yield strength ratios ranges calculated in this study are enveloped by the ratios found by Olson and Mattson (2008).

The yield shear strength ratio range for the Merriespruit gold tailing specimens was 0.1 – 0.35. By using Equation 3, the yield shear strength ratio range was found to be 0.18 – 0.26, which falls within the range calculated from the triaxial compression tests. The critical shear strength ratio range for the Merriespruit gold tailings specimens was 0.02 – 0.19. By using equation 4, the critical shear strength ratio was found to be 0.01 – 0.1. Again Equation 4 yielded a range that falls below the upper-bound calculated from the triaxial compression tests.

The yield strength ratios calculated from laboratory triaxial compression tests in this study ranged from 0.1 – 0.38. As mentioned before, Hanzawa (1980) found yield strength ratios that ranged from 0.11 – 0.41. The difference in the lower-bound yield strength ratio is 0.09% and the difference in the upper-bound yield strength ratio is 0.07%. Therefore the strength ratios calculated in this study correspond to the strength ratios found by Hanzawa (1980).

Olson and Mattson (2008) stated that on average the triaxial compression tests will yield a higher yield and critical strength ratio envelope than back-calculated yield and critical shear strength ratios. This is because the yield strength is mobilized at a smaller strain in the triaxial compression test. Olson and Stark (2003) stated that at small strains, the mobilized shear strength is a function of the mode of shear. The back-calculated strength ratios calculated in this study are smaller than the strength ratios calculated from the triaxial tests and therefore corresponds to the finding of Olson and Mattson (2008).

5 Conclusion

The ranges for the yield shear strength ratios found in this study are close to those from earlier laboratory tests, with a 33% difference in the lower-bound and 11% in the upper-bound. Similarly, the critical shear strength ratio ranges in this study are fairly close to those found from previous laboratory tests (see Olson and Mattson, 2008), with a difference of 50% in the lower-bound and again 11% in the upper-bound. These differences may be due to the rounding of numbers by Olson and Mattson (2008), laboratory errors and difference in reconstitution.
methods. The laboratory data used in this study are from South African gold tailings. This study confirms that the yield and critical strength ratio concepts described by Olson and Stark (2003a) are valid on South African gold tailings. The data collected in this study also shows that the strength ratio ranges yielded from triaxial compression tests envelopes the strength ratio ranges back-calculated from liquefaction flow failure case histories.

References


Desludging Wastewater Ponds Using Geotextile Dewatering Bags

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Abstract

Several mechanical desludging options are available as solutions for onsite desludging, but are costly for wastewater treatment facilities and as well as contractors that have limited budgets. The objective of this case study is to show the feasibility of geotextile bags (geobags) as a dewatering option for several desludging projects with ease of operation, retention of solids, and cost effectiveness as advantages over alternative methods. The filtration properties of geotextiles, with their ability to allow liquids to pass through whilst retaining solids, make them desirable in dewatering sludge material.

Geobags, manufactured from woven geotextiles, are incorporated in the desludging process into which materials are pumped directly, containing the solids and providing a relatively quick dewatering method compared to alternatives (Gaffney, Martin, Mahir, and Bennert, 1999).

Keywords: Wastewater, Sludge, Tailings, Geotextiles, Dewater, Geobag

1 Introduction

When wastewater treatment ponds reach their capacity, as a result of increasing population, the options available are either to build new facilities or to empty the existing ones. Desludging of these ponds has to be carried out in order to make additional space available for the waste. This is further intensified by the strict regulations and laws set by environmental bodies to improve the functionality of wastewater treatment works.

Due to the constraints in funding and the limited space available at existing wastewater facilities to dispose of the extracted material through the desludging process, a feasible solution has to be found. The use of geotextile dewatering bags was seen as the most efficient, cost effective method that could be implemented to achieve the required desludging process.
2 Background

Wastewater treatment is a process used to convert wastewater - which is water no longer needed or is unsuitable due to its most recent use - into an effluent that can either be returned to the water cycle with minimal environmental impact. The solid particles (sludge) can then be separated from the effluent and transported to a designated safe disposal site, or stored in a tailings storage facility (TSF) at the wastewater treatment plant. This should strictly follow the regulations set in Section 40 of the National Water Act (Act No. 36 of 1998) for on-site disposal and Section 20 of the Environment Conservation Act (Act No. 73 of 1989) for offsite land disposal of the sludge.

According to the Guidelines for the Utilization and Disposal of Wastewater Sludge (2007), with an ever increasing sludge accumulation, the current management methods are unsustainable thus becoming a challenge for many wastewater tailings facilities worldwide. In South Africa, the Bothaville, Denaissance, Fochville and Villers wastewater treatment plants were identified as having reached their maximum storage capacity, and have subsequently been worked on. Figure 1 shows excess sludge accumulation at a wastewater tailings facility.

![Sludge accumulation at a wastewater tailings facility](image)

Figure 1. Sludge accumulation at a wastewater tailings facility

Innovative solutions need to be sought after to create opportunities that provide a wide spectrum of options to the management of wastewater sludge, which had to be disposed of either offsite or into a new tailings dam. However, the option of building new tailings dams was too expensive. In addition, the poor construction of the tailings storage facilities, most without any adequate lining system, also led to an environmental concern through possible leakage of toxic effluent into the ground.

The project entailed removing the sludge from the existing tailings facilities, with the further intention of carrying out repairs to the lining system and installation of new lining systems in the facilities that previously had none - as stipulated by the new Department of Water Affairs regulations.
3 Sludge Removal

Dredging is by far the most common form of sludge removal, with a few other different methods available, all of which involve mechanically removing sludge from the tailings dam. Once the sludge is removed, it is dried and transported to either a landfill or a land application facility. The dredging method is however an extremely difficult, time consuming and costly process. In addition, the process of dredging could damage the lining system which would compromise the structural integrity of the sludge pond with seepage of wastewater into the environment. Figure 2 shows the removal of excess sludge from a wastewater tailings facility by the mechanical dredging method.

![Figure 2. Mechanical dredging method of excess sludge from a wastewater tailings facility.](image)

With the limited budget and time sensitive need to desludge the wastewater ponds, alternative methods had to be sought after. The more economical solution chosen involved using geotextile dewatering bags on-site.

4 Project Design

In the project under review, dewatering bags comprising UV-stabilized woven Polypropylene (PP) geotextile material with high strength seams for a strong and durable dewatering system are used to capture the dredged sludge whilst minimizing water loss. The thread used to stitch the bags has a higher breaking strength than the geotextile itself providing sufficient tensile strength to the geobags to withstand the stresses associated with pumping the material at high pressures.

The dewatering bags allow water to flow out through the porous geotextile fabric of the bags, while filtering any solids. The concern of the woven fabric opening size appearing to be slightly larger than the particle size of the dredged sludge material is countered by the formation of a filter cake on the inside of the fabric, with a resultant effective retention of the solids. This thus
creates an equivalent two-stage filter with filtration efficiencies above 98% for fine-grained material filtered through high strength woven geotextile bags (Bindra, 2004).

In order to accommodate pipe connections, 2 inlets per bag of standard size 200 mm were allowed for that will accommodate any pipe size up to and including 200 mm diameter. At the end of the filling process, these inlets can be tied off very easily once the pipe is removed. The pipe was inserted approximately 2/3 of the way into the injection port and secured with tension strapping.

The pumping rate was constantly monitored and kept below 40 m³/h, however if it did exceed this rate a manifold would have to be used to split the flow and reduce the inlet speed per bag. The throughflow property of the geotextile determines the rate at which the effluent flows out of the geobag. As such, any increase in the pumping rate would not result in an increase in outflow. This moderate rate also ensured the bags kept their structural integrity throughout the operation.

Selection of the final disposal site for the dewatering geotextile bags should be as close to the generation area as possible in order to minimize transportation costs. In addition, the recommended selection procedure for the suitable area should:

- Ensure that the disposal site is not located in a sensitive area where disposal is not permissible.
- Ensure that the sludge disposal site is located as far as possible from the area where the final effluent is discharged to limit possible contamination of the final effluent, as well as to limit possible contribution of contaminants to the water resource.
- Allow for the maximum buffer zones, greater than 400 m from surface water.
- Consider the slope of the disposal site to minimize run-off, erosion and ponding.
- Ensure the disposal site is not within the 1:100 year flood line.

Prior to placement and filling of the geobags, the dewatering area was lined with a 1 mm thick Geomembrane layer that meets the SANS 1526 (2015) specifications for thermoplastic sheeting, in order to prevent local erosion and to collect all the effluent, which was released from the geobags, thus preventing any seepage of the wastewater into the ground. The effluent would then be channeled back into the dam or taken for further treatment. Figures 3 and 4 show preparation of the ground prior to and placement of the Geomembrane liner and geobag on the dewatering area respectively, following the guidelines stated in SANS 10409 (2005).

Figure 3. Preparation of ground
No flocculants are necessary in the process as there is enough time available for the material to dewater by gravity, which also leads to a further cost reduction. Flocculants are generally used to group solid particles, however the tailings dams worked on consisted of solid dry waste. In order to ease the removal of the waste, water is pumped into the dams to agitate the sludge. Transfer of the sludge from the tailings dam into the bags is through pumps fitted with an impeller on the inlet that cuts down bigger particles as the pumping is conducted.

The duration of the dewatering and consolidation period always varies depending upon the type of geotextile, the bag sizes, fill material, and the site conditions. On average it takes about a week for the bags to drain. Figures 5 and 6 show the filling of and subsequent filtration of water through the dewatering bags respectively. The solids are retained in the geobags and can then be safely disposed of or used as fertilizer. The geobags are also disposed of as they are not reusable.
5 Analysis

The selection of the geobag size is dependent on the volume that needs to be removed from the tailings facility and the space available on site where the geobags shall be placed. The largest sized geobag that was used in this dewatering process had a 15 metre circumference and was 60 metres in length, which equates to an approximate volume of 720 cubic metres. The costing of the bag is determined by the woven geotextile material, the high strength thread and the stitching process. The indicative price for the geobag is thus R60.00 per cubic metre, however in comparing this method to other techniques further considerations additional transport, labor, construction and installation costs.

The ability to measure the volume of waste that can be extracted from the tailings dams per month per bag would have to be determined with a further detailed study. In addition, the measurements are influenced by the particle sizes and densities that vary with sites and waste composition of the facilities.

6 Conclusions

"According to Bindra (2004)”, with the growing population worldwide, more and more waste will be generated and there shall be less space to dispose of it. The traditional methods of waste containment usually require large amounts of space, have increasingly difficult environmental permits and demand frequent dredging. These methods are also not affordable to many municipalities, thus making them unfeasible solutions.

This innovative technology incorporating geotextile dewatering bags is capable of competing economically with other alternative dewatering techniques. In addition, this technique is passive and does not require extensive or constant monitoring and maintenance of equipment. As such geotextile dewatering bags are 50 – 70 % cheaper than other sludge removal methods, when all factors are considered including but not limited to construction time, labor and machinery costs.

The dewatering bag system is a cost effective solution that simplifies the sludge removal process, and subsequently increases the space available in the wastewater tailings facilities for
the additional waste. These geobags are an environmentally friendly solution and are suitable for use in aquaculture, industrial lagoons, sedimentation ponds, and wastewater plants.

The geobags are effective in retaining the fine-grained materials as found in sewage sludge whilst allowing the water to filter through. The effluent that passes through the woven fabric meets the regulations set by the Department of Water Affairs, and can then be transferred to a designated safe disposal site.

References


Designing with Geosynthetics in Capping

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Abstract

When a landfill reaches its full waste capacity, safety measures and environmental recovery are undertaken using a capping system. Traditional soil cover capping systems can be costly solutions. By introducing geosynthetics, costs can be drastically reduced. This paper focuses on the design aspects using geosynthetics to conceive an effective and economical capping system. A geocomposite drain can be used as an alternative to the thick, selected granular, seepage collection layer. Reinforced geomats can be used in soil veneer design to reduce the cover thickness. When using different geosynthetics, extensive knowledge on a product’s properties is required to understand its behaviour and performance to conceive an appropriate design. Two case histories are presented that illustrate an effective geosynthetic solution for upgrading an existing capping system and a failed slope to demonstrate the importance of complying with soil veneer design principles when using geosynthetics.

Keywords: landfill; capping; geosynthetics; drainage; design

1 Introduction

The proper disposal of waste materials in suitable areas to ensure the protection of the environment from the harm of waste material is an important requirement for an advanced society. This results in the demand for greater landfill capacity, requiring proper management, planning and engineering to uphold the surrounding environmental conditions, keeping the waste secluded for a certain design life. Once a landfill reaches its full capacity dictated by the license given to the site, it can no longer hold waste and must be closed using a capping system that must shield the waste from harming the surrounding environment, reducing rainwater infiltration and the associated generation of leachate to a minimum level.

The term “capping” means a complex design system (which is able to satisfy a set of requirements) aimed at achieving landscape, health and environmental protection minimum requirements (Scotto & Napoleoni, 2007).

1.1 Traditional Capping Solutions

A traditional cover capping system requires high volumes of natural resources, requires labour intensive installation and is susceptible to erosion, making it a costly solution. For covering a
landfill, several types of capping systems can be used. Some can be used for hazardous landfills, and some for non-hazardous landfills. Depending on the landfill classification, different capping solutions may be used. Figure 1 illustrates an example of a typical capping layer using a traditional solution with mostly natural materials.

![Figure 1. Example of typical traditional capping solution](image1)

A capping system can consist of an intermediate cover layer (7) above the waste material (8) followed by a grading layer (6). For an impermeable barrier layer (5), commonly, compacted clay is used as cap; however, there is a chance of cracking if the clay dries out. Therefore, a Geomembrane comprising of LLDPE or PVC (which is able to sustain higher values of deformation, due to the consolidation of the waste body) is often used in place of the clay. The aim of the drainage layer (4) is to intercept, collect and dispose of the infiltrating rainwater, and to avoid waterlogging of the cover. The correct operation of this system is also essential for the growth of vegetation (1), as the moisture content in the soil (2) must be maintained at a level that sustains growth. Traditionally, the drainage layer may consist of a selected gravel with a thickness of up to 0,5m (according to European standards) and is accompanied generally by a geotextile (3) to perform the function of separation and filtration, allowing water to pass through to the drainage layer while retaining the fines from the soil layer above. The outer layer (2) consists of top soil and the thickness must be sufficient to enable the growth and development of an adequate vegetative cover. As per European Standards (Figure 2), the thickness used until now on flat surfaces varied between 0,5 and 1,0m and hardly ever exceeded 0,5m on slopes.

![Figure 2. Capping example as per EN Standardization (Scotto & Napoleoni, 2007)](image2)

Although it is not relevant to this paper, and for this reason not shown in Figure 1, it should be noted that the grading layer can be covered by a biogas drainage layer and a waterproofing layer where biogas needs to be collected for recovery or venting. These layers are subject to
the same design principles as are described in this paper, for the outer layers (layers 1 to 4 in Figure 1).

1.2 Geosynthetics in Landfill Capping
The use of geosynthetics in a capping system, can significantly reduce costs. Figure 3 illustrates the use of geosynthetics in the outer layer.

Figure 3. Inner and Outer layer of capping (Scotto & Napoleoni, 2007)

Drainage geosynthetics such as geocomposite drains illustrated in layer 2 in Figure 3 can be used as an alternative to a typical granular drainage layer. Geogrids or reinforced turf mats can also be introduced to increase the stability of the soil cover (Layer 1) and erosion control blankets/mattresses can be used to not only protect the soil surface from erosion, but also to assist in the sustainable growth of vegetation (Layer 3).

When using different geosynthetics and design approaches extensive knowledge on a product’s characteristics is required to understand its behaviour and performance in order to develop an appropriate design.

2 Design Approach for Multi-Layered Capping

The properties and the behavior of the waste influence the performance of the cap, and must considered in the design and in construction (Shukla, 2012). One of the typical problems which the designer must deal with when using multi layered materials (soil or geosynthetic) is the stability of the layer above on the layer below. An adequate factor of safety against sliding is required. The analysis procedure is called veneer design and is controlled by the interface friction angle between the materials which can be soil-soil, geosynthetic-geosynthetic or soil-geosynthetics. The interface friction angle is characterized by a dry value and wet value. Wet friction angles can reach values below 10° for soil-geomembrane (e.g. in a cover where infiltrating rainwater is not suitably and effectively drained or in a base barrier where accumulation of leachate brings about saturation). The drainage design is therefore important in ensuring stability.

2.1 Soil Veneer Design
In a soil slope, a stability calculation is required to assess the factor of safety for sliding on the barrier layer. In a landfill, there are four situations scenarios that should be considered:

- Landfill liners with leachate collection sand or gravel above them.
- Surface impoundment liners where the cover soil is placed over the geomembrane to shield it from ultraviolet light, heat degradation, and equipment damage.
- Landfill covers that have topsoil and protection soil placed over the geomembrane.
• General slopes and embankments containing geotextiles or erosion control materials being covered with a layer of soil. In all four of the above-mentioned scenarios the soil layer is relatively thin (0.3 to 1.0 m), hence the sliding stability of such a veneer of cover soil is the issue (Koerner, 2012).

Each of the layer interfaces must be in equilibrium with an adequate factor of safety between the forces destabilizing the cover layer and the stabilizing forces. These forces are shown in by a diagram in Figure 4.

Figure 4. Diagram of forces in soil veneer (Koerner, 2012)

A limit equilibrium equation is taken from this diagram which enables the use of geosynthetic reinforcement to be taken into account inside the soil layer with a thickness H.

To determine whether reinforcement with geosynthetic materials is required, a quick evaluation can performed by checking the compliance of the equilibrium condition between the tangent of the friction angle for the soil/geosynthetic interface (δ) and the slope angle (ω), as per Equation 1:

\[
\frac{\tan \delta}{\tan \omega}
\]  

(1)

When the value of Equation 1 is higher than 1.3 (proposed value), mechanical reinforcement of the soil cover is not necessary. However, less than 1.3, mechanical reinforcement is necessary.

If the equilibrium condition is not complied with, it is possible to calculate the working strength value for the reinforcement by using the following Equation 2 (Scotto & Napoleoni, 2007).

\[
\frac{W \cdot \cos \omega \cdot \tan \delta + C_a \cdot L + R_{\text{reinforcement}}}{W \cdot \sin \omega} = FS > 1.3(\text{proposed value})
\]  

(2)

Where:

- \( W \) = weight of the mass placed above the geosynthetic material and/or of the waterproof package = \( \gamma H \);
- \( \omega \) = slope angle
- \( \delta \) = soil/geosynthetic interface friction angle (experimental value)
- \( L \) = Length of slope
- \( C_a \) = soil/geosynthetic material adhesion (experimental value usually assumed to be zero)
- \( R_{\text{reinforcement}} \) = long-term tensile strength of the reinforcement member

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The working strength defines the long-term strength of a product. An evaluation is done to determine the effects of viscous deformation phenomena such as creep and all reduction factors that take into consideration mechanical damage and chemical-environmental damage. These findings are used to determine the long term strength a geosynthetic is usually characterized by.

Figure 5 illustrates a multi layered capping system (on an infinite slope) where a mechanical tensile check should be carried out for the individual layers of the materials which have been selected and used. This aims to verify that the stresses induced by the overloads and transmitted between the various layers in accordance with the interface friction angle between the various layers of geosynthetics and the angle of the slope are compatible with the mechanical strength of the various geosynthetic materials.

Figure 5. Exploded view of liner system

As different geosynthetic materials possess different interface friction angles, the upper and lower faces of a layer are subjected to different forces. This can cause an unbalance between the two tangential stress components applied to the layer, and the one that prevails is that applied to the upper face, the upper face will be subject to tensile forces. It will therefore be necessary to check that this is compatible with the mechanical strength of the component.

It is considered good practice not to apply tensile forces to the geomembrane and to ensure that the balance of the stresses on the two interfaces is always neutral ($F_4 = F_5$) or the geosynthetic is in compression ($F_4 > F_5$). In the case this is not possible, the component that induces compressive stresses and not tensile stresses in the membrane should prevail. Some designers suggest that stresses that can lead to a deformation of 2.5-4%, should not be applied to the membrane, which is equivalent to applying a safety factor of approximately 4 with respect to the yield state of a HDPE membrane (Scotto & Napoleoni, 2007).

It is important to take into account the pore pressures and presence of a drainage geocomposite (which is always recommended) and of a highly permeable layer which is able to effectively drain away rain water.

2.2 Drainage Design

In order to avoid the build-up of water on/in the cap, a drainage layer is usually installed in the system above the geomembrane. In German landfills, the minimum gradient of the cap drainage layer is 5% and in long slopes, the maximum inclination should not be steeper than 1(Vertical) in 3(Horizontal). The maximum slope is limited for practical reasons with respect to landscaping and maintenance work (GGS, 1991).
The design of drainage for a landfill cap begins with the calculation of input flow rate. Although this is project specific, it is possible to provide indications only for the most common and simple cases. The input flow from rainfall onto a sloping surface is of interest in many situations such as landfill capping. Figure 6 illustrates the scheme for the calculation of the input flow rate in the case of rainfall on a sloping surface based of Darcy’s Law.

Figure 6. Scheme for the calculation of the input flow rate

For all applications, the available flow rate of the geocomposites shall be obtained by applying a set of Reduction Factors (Cancelli & Rimoldi, 1989) which take into account all the phenomena that may decrease the flow rate over the entire design life compared to the short term flow rate measured in the tests according to EN ISO 12958:2010 or ASTM D4716 -08(2013) standard and in accordance with equation 3:

\[ Q_a = \frac{Q_L \cdot F_{lr}}{RF_{in} \cdot RF_{cr} \cdot RF_{cc} \cdot RF_{bc}} \]  

(3)

Where:

- \( Q_a \) = available long term flow rate for the geocomposite;
- \( Q_L \) = short term flow rate obtained from laboratory tests;
- \( RF_{in} \) = Reduction Factor for the intrusion of filter geotextiles into the draining core;
- \( RF_{cr} \) = Reduction Factor for the compressive creep of the geocomposite;
- \( RF_{cc} \) = Reduction Factor for chemical clogging of the draining core;
- \( RF_{bc} \) = Reduction Factor for biological clogging of the draining core;
- \( F_{lr} \) = Empirical factor to be applied when the test results for \( Q_L \) are available for contact conditions different from the project conditions.

The Reduction Factors shall be set considering the specific conditions of each project, taking into consideration the experience and/or research on similar conditions of use.

Once the design input flow \( Q_D \) has been calculated, the available input flow \( Q_a \) shall be calculated for one or more geocomposites. The final Factor of Safety \( F_{SG} \) afforded by the design with each geocomposite is given by:

\[ F_{SG} = \frac{Q_a}{Q_D} \]  

(4)
Only those geocomposites for which $FS_G \geq 1$ are suitable for the project. The final selection of the geocomposite shall be made from the geocomposites for which:

$$FS_G \geq 1.00$$  \hspace{1cm} (5)

taking into consideration also costs and availability.

2.3 Anchoring

The “fixing” of the geomembranes and other layers at the top of a slope is a fundamental problem and is a requirement to all geosynthetic materials to which tensile forces are applied. The most commonly used form of fixing is by excavating a trench at the top of each slope inside which the geosynthetic material (this term is also used to indicate a geomembrane and any other type of related product) will be fixed. The trench is filled with adequately compacted soil or aggregate. Some designers propose a concrete crown to the trench (sometimes lightly reinforced) whilst others prefer to use loose material considering that it is preferable for the anchored geosynthetic material to slide out of the trench rather than break. If a double membrane layer is used, or there is a large number of geosynthetics to fix at the top or the level of stresses applied to the various materials is high, use of a double anchor trench is recommended as shown in the following Figure 7:

![Figure 7. Single-anchor trench (left) and separate anchor trenches (right)](image)

The design of the trench is normally carried out with a method which should safeguard the design both in terms of pull-out and breakage. Figure 8 illustrates the design layout in the presence of frictionless pulleys which transmit the entire tensile force acting on the geosynthetic material are assumed, enabling the equilibrium condition to be satisfied. More detailed information regarding anchor trench design can be found in literature (Scotto & Napoleoni, 2007 and Qian et al. 2002).

![Figure 8. Anchor trench design layout](image)

3 Case Study

3.1 Tubatse Ferrochrome Mine, South Africa (Dode & Legg, 2016)

In Dode and Legg, 2016, the solution to upgrading the capping system of a ferrochrome smelter to comply with the local legislation is discussed. The facility was originally capped in 1999 with concrete filled geocells. The cells began to crack on the outer slopes due to the settlement of the waste body as shown in Figure 9.

![Figure 9. Tubatse Ferrochrome Mine](image)
To overcome the stability challenge associated with a steep side slope 1V:1.5H, an engineering solution using geosynthetics was applied to effectively upgrade the cap. The upgraded capping system design is presented in Figure 10 and Figure 11 and consists of the following layers (from bottom up):

a) Existing concrete surface to be repaired locally to remove any sharp protrusions and to fill any voids;
b) Non-woven, needle punched geotextile and seams overlapped and heat-bonded, placed onto the existing concrete surface;
c) 1.5 mm thick HDPE geomembrane liner (single textured, facing down);
d) Geocomposite drainage layer;
e) Geogrid reinforced geomat; and
f) 300 mm thick topsoil with grass and biodegradable erosion blanket.
3.2 Rigoloccio Mine, Italy (Favalli et al. 2002)
The first capping of a mining landfill (tailings) took place in the Rigoloccio mine, located in Gavorrano (GR) in Tuscany Italy. Detailed information regarding this project can be found in Favalli et al. 2002. The original solution for the sloped areas consisted of 0.5m of vegetated soil, a drainage geocomposite (two geotextiles and a geonet. The barrier layer consisted of a textured HDPE membrane (2mm thick), 0.4m of clay over the waste. However, due to soil veneer problems during construction, the original design was modified to include an erosion control blanket and a reinforced turf mat over the geocomposite drain. Further, the soil thickness was reduced to 0.3m (Figure 12).

Based on the modified design, the Factor of Safety calculated for the reinforced solution was 1.77. Shortly after completion of construction, during a period of heavy rainfall, 500m² of a 18 000m² job had failed (Figure 13).

A site investigation was conducted and the analysis found that:
- The layer of soil was 30-40% thicker than the one designed.
- The friction angle considered had reduced significantly during the heavy rainfall and presumably the soil weight increased due to saturation.
- Failure of the reinforced section occurred predominantly at the end of the slope confirming that collapse was due to a lack of mechanical strength and, in few other sections, at different points along the slope.
- It was difficult to understand how failures occurred at other points in the middle of the mat (probably due to poor connection).

Considering the parameters obtained from the analysis, a design analysis was done in which under the field conditions, the factor of safety was found to have been reduced to 0.72, hence the result of a failure. From this experience, it can be learned:
- As the failure occurred in the reinforced mat, the textured membrane in this case did not influence the reinforcement and therefore irrelevant in the stabilization of the soil; a smooth membrane could have been used instead, reducing the costs;
- The interaction factors between the different geosynthetics are a key point; these values can varied between layers depending on the different situations and must be considered;
Thickness and shear strength of the soil placed on the reinforced mat are relevant issues:
- The thickness (especially when small) is difficult to manage and it is likely that thicker layers will be placed in practice and thereby overload the interface with the geosynthetic;
- The unit weight (density) and also the shear strength of the soil are not homogenous and are often different from those assumed. Changing the soil unit weight and/or shear strength can also increase the loads and/or decrease the shear strength.

4 Conclusions

A brief introduction to the design of landfill capping solutions has been given. Although there are different standards around the world, these are merely a guideline to minimum requirements for the design of a capping system. Ultimately the designer should choose the most viable and economical solution without reducing the factor of safety. In general, the use of geosynthetics can reduce the total cost of capping.

Consistent design methodology is found throughout literature. However, it is important to highlight that the long-term properties of a product should be used in the design. Particularly the effects of creep which ultimately affect the long-term performance of a product must be considered.

The case studies show the importance of following the design specifications. It can be concluded from these studies that:
- The design of an erosion protection system is a real structural project and the input data must be consistent with the real situation;
- The use of a textured membrane can in some cases be insufficient and ineffective;
- The reinforcement strength must be appropriate for short term as well as long term conditions in any operating situation (dry/wet);
- Soil properties and the variations such as apply in the field are important;
- Installation should follow the design instructions strictly (e.g. Layer thickness must neither increased or decreased).

References
