



Risk-based slope design: Insights from the Thabazimbi failure

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Abstract

Certain degrees of safety, economic, and financial risk are implicit in any mining operation. At Thabazimbi Mine, slope stability was one of the major sources of risk, largely due to data uncertainties. Consequently, a risk-based approach in slope design was followed.

On 6 June 2015, a slope failure occurred in Thabazimbi Mine's Kumba pit, involving approximately 65 million tonnes of rock failing into the pit and the valley below. Although the failed mass moved rapidly and resulted in a run-out distance of approximately 900 m, controls had been put in place to successfully evacuate all personnel. Risk assessment during the onset of failure supported the decision to not compromise the safety of personnel in any way, leading to unavoidable equipment loss.

This contribution presents the failure as a case study in the context of a risk-based approach to slope design as well as the importance of implementing geotechnical controls to effectively manage slope instability risk. Furthermore, a comprehensive back-analysis was performed, on a fully constrained 3D model, utilising 3DEC software, to gain insights into the failure mechanism.

In conclusion, lessons from the Thabazimbi slope failure are valuable as they demonstrate the importance of following a risk-based approach in slope design to effectively manage safety and financial risk in open pit mines.

Keywords

slope failure, risk, geotechnical controls

Introduction and background

Thabazimbi ("mountain of iron" in Setswana) Mine has been operating since 1932 and is currently in the closure phase. It is situated approximately 250 km north of Johannesburg in the Limpopo Province of the Republic of South Africa (Figure 1). Thabazimbi mine primarily produced high-grade haematite ore (more than 62% iron (Fe) content), which is particularly low in contaminants and sold exclusively to ArcelorMittal South Africa. The Thabazimbi area has a recorded mean annual precipitation (MAP) of 645 mm, of which 90% falls between October and April. The area is characterised by high peak temperatures, with daily maximum temperatures that may rise above 40 °C in summer.



Figure 1—Location of Thabazimbi Iron Ore Mine and the distribution of the Transvaal Supergroup

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Geological setting

Thabazimbi Iron Ore Mine is hosted by the Neo-Archaean (ca. 2.5 Ga) Transvaal Supergroup, which occurs on the Kaapvaal Craton of Southern Africa. The Transvaal Supergroup comprises three sub-basins: the Transvaal Basin in the east, the Griqualand West Basin in the west, and the Kanye Basin to the north in southern Botswana (Figure 1). The amalgamation of several Archaean greenstone belts, including the Witwatersrand Basin, and their underlying crystalline trondhjemite-granodiorite-granitoid (TGG) basements led to the formation of the Kaapvaal Craton (Poujal et al., 2003; Zeh et al., 2013), and formation of prominent suture zones referred to as the Colesburg lineament (De Wit et al., 1992) and the Thabazimbi-Murchison lineament (Zeh et al., 2013).

The Transvaal Supergroup is composed of basal chemical sediments and is separated from overlying volcanoclastic sediments by a regional unconformity (e.g., Eriksson et al., 2006). At Thabazimbi Mine, the simplified stratigraphy from the base upwards consists of three broad packages, although there is a significant tectonic imbrication of portions of these: 1) thick, basal dolomites of the Chuniespoort Group (Malmani Subgroup); 2) a thin shale unit that grades into a thick banded iron formation (BIF) of the Penge Formation and 3) overlying siliciclastic sediments of the Pretoria Group (Basson, Koegelenberg, 2017). These sequences were intruded by ca. 2.05 Ga mafic sills of the Bushveld Complex (Rajesh et al., 2013), which have been subsequently deformed, and late, relatively undeformed, ca. 1.1 – 1.3 Ga Pilansberg Complex dykes (Van Niekerk, 1962; Allsopp et al., 1967).

Structural setting

Thabazimbi Mine occurs in the Mochlapitsi Fold-and-Thrust

Belt (Good, De Wit, 1997) of the Thabazimbi-Murchinson lineament. Close to the northern margin of the Transvaal Basin, the Transvaal Supergroup has been folded and imbricated along a segment of the regional E-W trending Thabazimbi-Murchinson lineament (Gutzmer, Beukes, 1988; Good, De Wit, 1997; Basson, Koegelenberg, 2017). Basson and Koegelenberg (2017) suggest that Thabazimbi Mine exploits localised hematite enrichment along an E-W trending, 20 km long, structurally complex horizon. High-grade hematite mineralisation is typically confined to the base of a sequence of banded iron formation (BIF), on three prominent, E-W trending mountain ranges that are locally referred to as the Northern, Central and Southern Ranges as illustrated in Figure 2 (Basson, Koegelenberg, 2017). Kumba pit is situated at the extreme western end of the Northern Range.

The regional structure is due to uplift and duplication of the stratigraphy along two north-verging, E-W trending, oblique-slip thrusts, namely the Belt-of-Hills Thrust and the Bobbejaanswater Thrust (Du Plessis, Clendenin, 1988; Du Plessis, 1990). Duplication is accentuated by positively-weathering BIF, which defines significant scarp slopes along the Northern and Southern Ranges. The northernmost boundary of the Mochlapitsi Belt is marked by the Belt-of-Hills Thrust that juxtaposes Malmani Subgroup dolomites over younger units of the Transvaal Supergroup and Waterberg Group. The southernmost margin is marked by Pretoria Group sediments that dip gently southwards below the ca. 2054 Ma Bushveld Complex. The reader is referred to Basson and Koegelenberg (2017) for a more detailed overview of the structural setting of the Thabazimbi Mine area, and for insight into the 3D geometry of various mining areas. Table 1 serves a summary of the main deformational features of the Thabazimbi Mine region.

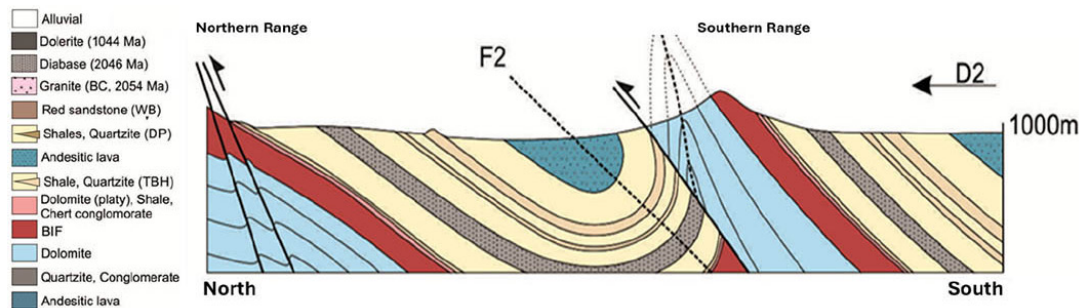


Figure 2—Simplified cross-section through the Thabazimbi mining area with Kumba pit located in the Northern Range (Kumba internal report)

Table 1			
Summary of structural features in the Thabazimbi area. Modified from Basson and Koegelenberg (2016) and references therein			
Event		Structural features	Comments
Extension	D ₁	Gentle, southwards- dipping Transvaal Supergroup. E-W trending listric faults.	Crustal extension and sagging during emplacement of the Bushveld Complex and contemporaneous deposition of the Waterberg Group.
	D ₂	Inversion of listric faults to thrusts. Gentle-to-open, km-scale, shallowly plunging F2.	Regional top-to-the-N thrusting and steep reverse faulting. Mine-scale to mesoscale D ₂ thrusts and folds show predominantly northwards kinematic transport and imbrication “pop-ups” are bound along their southern sides by S-verging reverse faults, along broad fold hinges.
Compression	D ₃	Oblique, sinistral strike-slip duplexes.	Reactivation of D ₂ structures.
	D ₄	NW-SE trending normal faults.	Mesozoic extension during sedimentation and eruption of Karoo Basalt, forming half-grabens.

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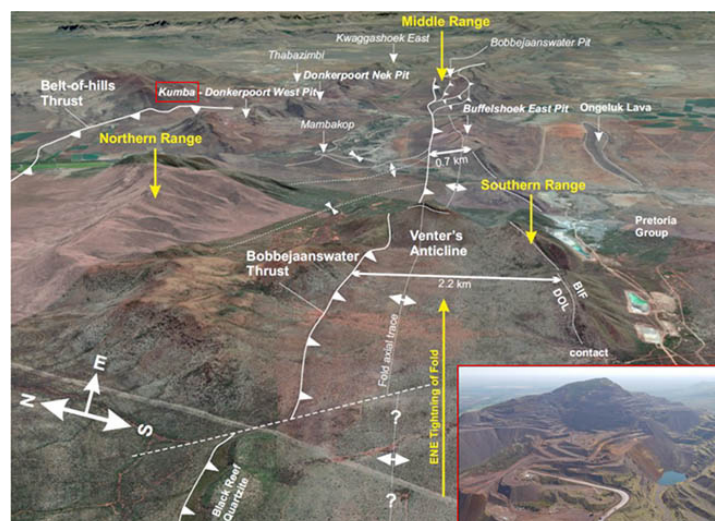


Figure 3—Overview of operational pits [satellite image and relief sourced from Google Earth Pro (2015)]

Table 2

Ore body characteristics

Orebody	Average thickness	Dip	Strike (0° North)	Planned mining depth below pit crest
Buffelshoek West	10 – 50 m variable	40 – 50°	075°	200 m
Donkerpoort Neck	5 – 30 m variable	50 – 55°	090°	200 m
Kumba	5 – 54 m variable	30 – 50°	170°	490 m

Mineralisation

Mineralisation at Thabazimbi Mine may be categorised into two main types, largely based on tectonostratigraphic position (Basson, Koegelenberg, 2017). Ore is categorised as *normal* or *typical* if it is constrained to the base of Penge Formation, i.e., between the base of BIF and a lower carbonaceous shale layer that directly overlies the Malmani dolomite. Shale shows variable thicknesses due to both thrusting, shearing, and flexural-slip or buckle folding (Basson, Koegelenberg, 2017), whereas ore categorised as ‘atypical’ or ‘Thabazimbi-type’ is situated stratigraphically higher, relative to the top of the dolomite, and is enclosed in BIF. Atypical ore often runs parallel to the sheared contacts of a laterally-persistent diabase sill or series of sills. Based on its crosscutting orientation, ore and diabase sills are hosted by northwards-climbing thrust planes. Kumba pit is known for containing one or more of these ‘atypical’ ore zones.

Orebodies mined

Before closure, mining was carried out in three pits (Buffelshoek, Donkerpoort Nek, and Kumba) using conventional open pit mining methods (Figure 3). Open pit mining operations by Kumba Iron Ore at Thabazimbi Mine ceased in September 2015 and current activities consist of decommissioning, reclamation, small scale mining, and monitoring.

The geometry and thickness of the basal orebody, which overlies the shale, is controlled by dolomite karst topography, the intensity of which decreases from north to south. Wad is expected to be present as dislocated localised pockets on the dolomite contact. Wad is a generic name for (often poorly crystalline) soft manganese oxides/hydroxides, often containing significant amounts of hydroxides/oxides of other metals and adsorbed metals (Ni, Co, Cu, Fe, and other transition metals, alkali elements, etc.). The upper orebody, overlying a diabase sill, is cusped in nature and dips generally increase towards the outer edges of orebodies as they pinch out.

Based on the available drillhole data and mapping, ore bodies pinch and swell along both dip and strike directions, resulting in variations in orebody thickness and dip. Tabulated in Table 2 are the general characteristics of the ore bodies or the portions thereof.

Most of the mining activities took place above the groundwater table and the pits were predominantly dry. The Thabazimbi operations are situated in mountainous terrain, with the Crocodile River draining the southern section of the area. Groundwater flow contours indicate drainage from higher lying, mountainous areas towards the higher-yielding dolomitic aquifers and the Crocodile River and Rooikuispruit stream.

Slope design and risk assessment

Kumba Iron Ore utilises a standardised geotechnical risk management process that employs risk controls at all levels, including slope design, slope monitoring, operational controls, and evacuation procedures (Figure 4).

Results from geomechanical analyses conducted in the Kumba pit, indicated that the design adhered to industry and company acceptability criteria. Figure 5 shows the locations of cross-sections used in the initial stability analysis and Table 3 presents a summary of the factor of safety (FoS) and the model probability of failure (PoF) of the cross-sections that intersected the failure surface. This was completed for the initial design in 2008, and the follow-up study was conducted in 2014.

Subsequent to the geomechanical analysis, a comprehensive geotechnical risk assessment for Thabazimbi Mine was conducted for the Life of Mine plan. Fault and event tree analyses were incorporated into the Kumba Iron Ore Hazard Identification and Risk Assessment (HIRA) criteria, in order to enable the comparison of slope failure risk with other assessed risks on the mine. This was performed routinely, according to the mine planning cycle.

Thabazimbi Mine had limited geological / structural and geotechnical data, and taking into account the full failure history of the mine, the resultant geotechnical uncertainty had to be

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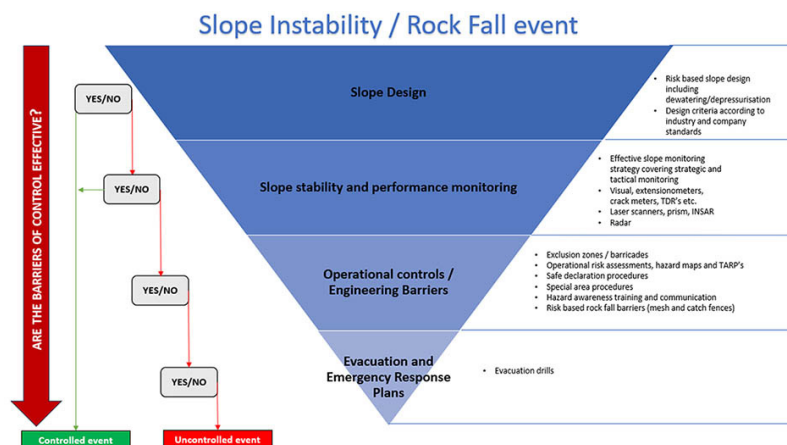


Figure 4—Geotechnical controls at Kumba Iron Ore

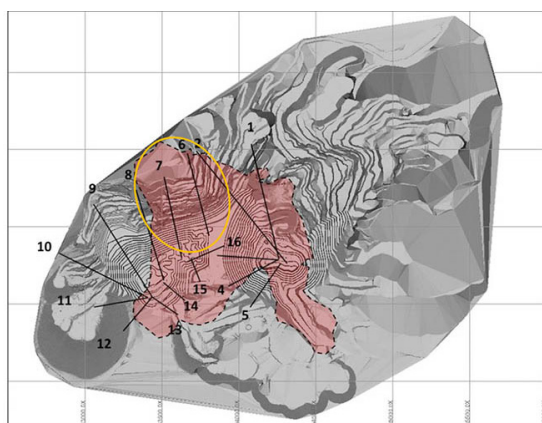


Figure 5—Cross-sections analysed for the Kumba pit with the failure area indicated

incorporated in the fault tree analysis. Subsequently, and despite the fact that the resulting fault tree analysis indicated higher probability of slope failure, the safety risk (HIRA) that was determined, using event tree analysis, was considered to be acceptable and defensible. This was dependent on all geotechnical risk control measures – including slope radar monitoring, survey prism monitoring, operational controls, and evacuation procedures – being adequate, in place, and effective.

Slope failure

On 6 June 2015, a significant slope failure occurred at the Kumba open pit. An approximate volume of 65 million tonnes of rock failed into the pit and the valley below it, inundating the production area of the pit. Although the failure was significant in terms of production impact, the trigger action response plan informed by slope monitoring data, allowed for early warning and safe evacuation of personnel.

Sequence of events

The sequence of events leading to, and subsequent to the onset of the slope failure, is shown in Figure 6: the initial slope stability radar monitoring deformation alarm triggered on 1 June 2015 (Day 1), 6 days before failure.

The sequence shows continued increase in displacement rates over 6 days leading up to the failure. Displacement rates increased to 20 mm/hr by Day 3. On Days 4 and 5, displacement rates continued to increase, to 75 mm/hr immediately prior to failure. The onset of the failure was at 22:00 on Day 5, with the failure event occurring at 02:45 on Day 6 (6 June 2015).

Slope stability monitoring

The Kumba pit slope monitoring strategy included a Leica GeoMos Survey Prism System for slope performance monitoring and GroundProbe Slope Stability Radars (SSRs) for safety-critical monitoring. Figure 2 illustrates the Kumba pit with locations of the two SSRs, survey robotic theodolite stations, and prisms installed on the highwall. Regular visual inspections of the pit were carried out, with photographic and written records being kept for each geotechnical inspection. This monitoring strategy was considered adequate for the assessed Kumba pit slope stability risk.

Both automated monitoring systems, GeoMos and SSR's, successfully detected the onset of the failure and provided early warning, facilitating the evacuation process. The GeoMos prism monitoring data is illustrated in Figure 8, while the SSR monitoring data is depicted in Figure 9. Deformation rates and velocities exceeded 500 mm/day leading up to the failure, with a total displacement of approximately 2 metres before the event occurred. Figure 10 shows a comparison of photos before and after the failure.

Back-analysis

Geotechnical conditions leading to the failure were manifold, involving several inputs with varying degrees of data confidence. Variations in the geological, structural, rock mass, mining, and

Table 3
Summary of geomechanical modelling results for corresponding sections in the failure area

Kumba Iron Ore Analysis, 2008			SRK Consulting Analysis, 2014		
Section	FoS	PoF	Section	FoS	PoF
6	1.59	< 10%	S15	1.47	< 10%
7	1.73	< 10%	S18	1.48	< 10%

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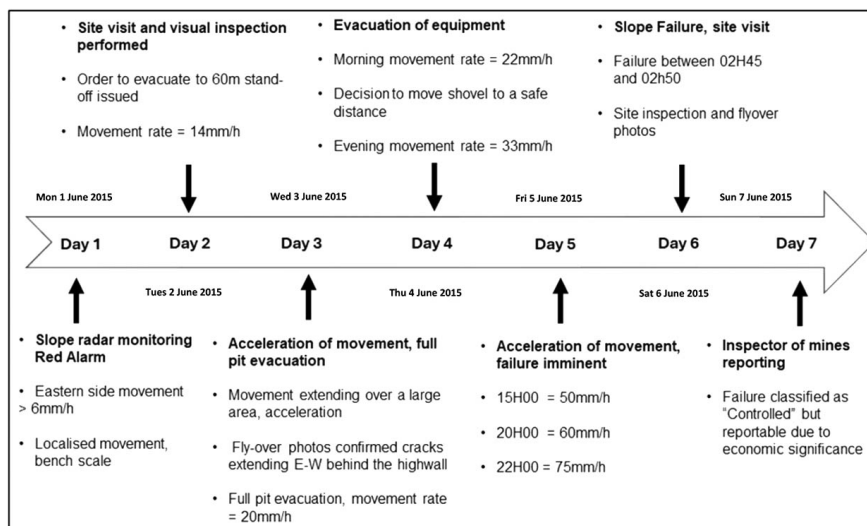


Figure 6—A summary of the sequence of events leading to the June 2015 slope failure

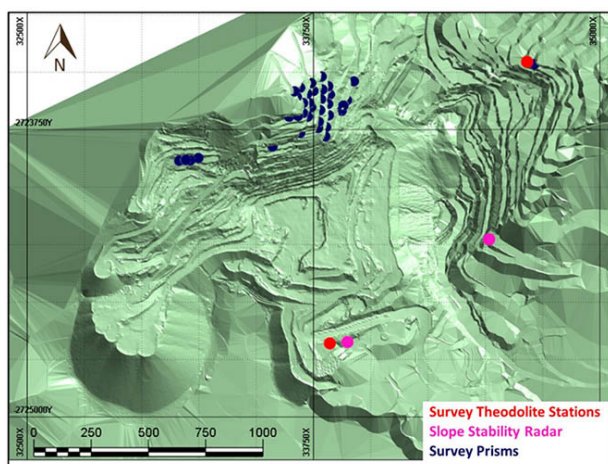


Figure 7—Slope monitoring in the Kumba pit

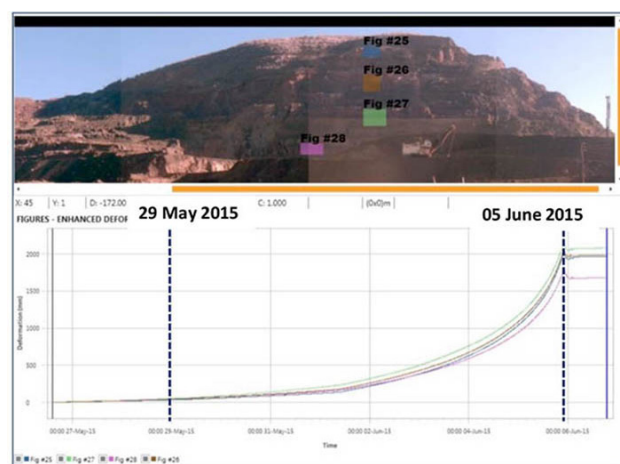


Figure 9—Slope Stability Radar data from 27 May 2015 to 5 June 2015

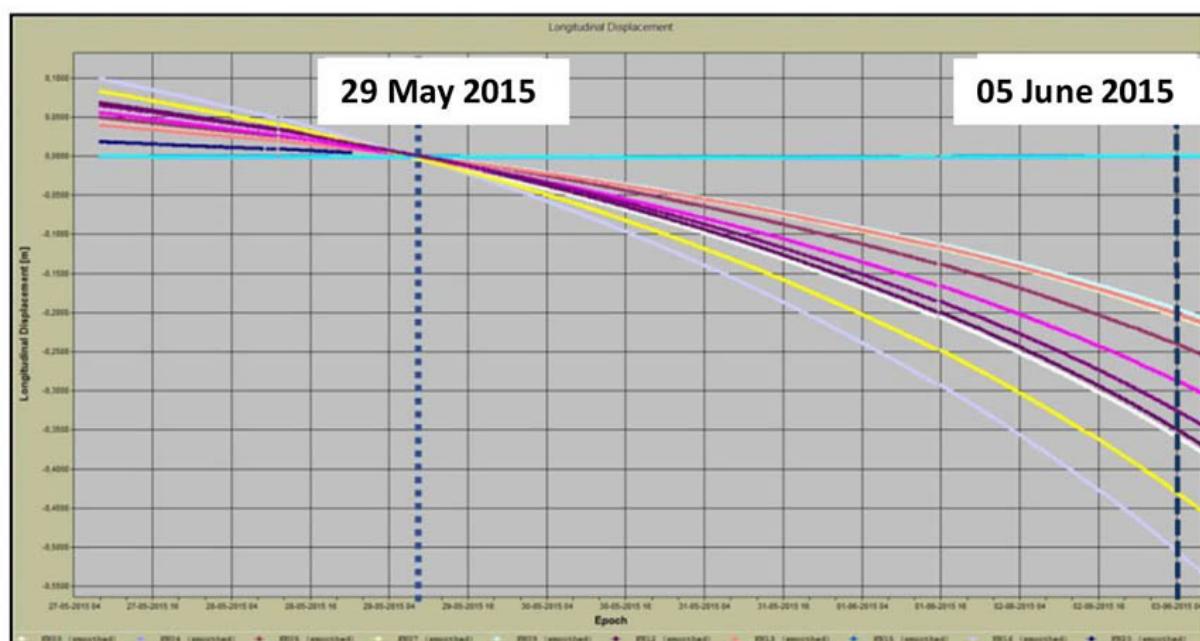


Figure 8—GeoMoS data from 27 May 2015 to 5 June 2015 showing displacement

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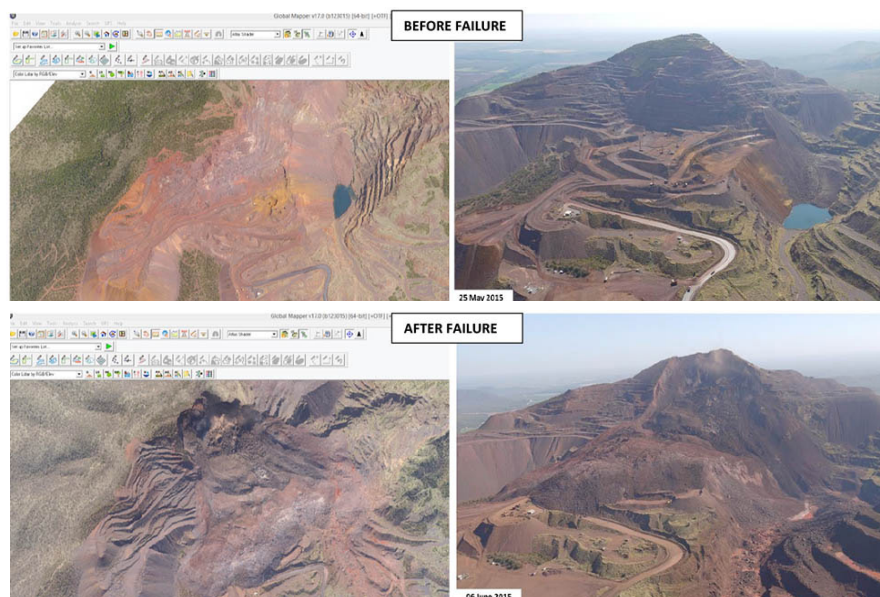


Figure 10—The Kumba pit before (top) and after (bottom) the slope failure event

groundwater conditions individually and jointly contributed to triggering the failure. The three-dimensional nature of the pit slope, combined with the surrounding topography and geotechnical conditions, required a phased approach to the analysis. Each phase introduced increasing levels of complexity, allowing for a deeper understanding of the failure mechanisms as the investigation progressed. For this reason, the back-analysis comprised geological back-analysis, 2D and 3D numerical modelling.

Geological back-analysis

Geological back-analysis of the slope failure requires a fully-constrained three-dimensional (3D) model. Although the original 3D model of the Kumba pit existed at the time of failure, this version contains an updated structural interpretation that considered all, albeit limited, historical mapping data, an up-to-date drillhole database, and available downhole structural data. Consequently, it provides a fit-for-purpose 3D model for failure simulation and is considered to be a significant improvement over the previous 3D model(s). The 3D model was built using a workflow approach, with the following basic steps:

- Drillhole database import and validation;
- Cleaning (e.g., compositing) of drillhole data and intersections;
- Point/structural data import (in this case face mapping data);
- Fault network construction based on explicitly-mapped points and offsets of contacts between adjacent drillholes;
- Establishment of a fault chronology (cross-cutting relationships and fault activation to create fault-bounded blocks);
- Lithological surface construction;
- Establishment of surface chronology;
- Surface chronology activation to construct lithological volumes.

Fault network construction

The generally cross-cutting fault network at Kumba pit consists of two dominant orientations: 1) NE-trending normal or reverse faults that have exploited fold-related cleavage; and 2) NNW-trending strike-slip faults, often with a component of vertical offset. In general, NE-trending faults terminate against NNW-trending faults,

with the latter continuing to the model boundary. 'Activation' of the fault network results in a total of 57 fault-bounded blocks (Figure 6), each with their own average bedding and lithological contact orientations. A total of 48 NE-trending faults were constructed. A decision was made to construct the NE-trending faults with a predefined average orientation of $330^{\circ}/75^{\circ}-85^{\circ}$, modified based on locally-available data. A total of 7 NNW-trending faults is represented.

Bedding and contacts

Dolomite and shale volumes are generated from surfaces that employ measured lithological contacts and structural discs as modelling parameters. Plotting these orientations on stereonet produces contoured maxima of $153^{\circ}/36^{\circ}$ and $156^{\circ}/34^{\circ}$ for dolomite and shale contacts, respectively (Figure 12).

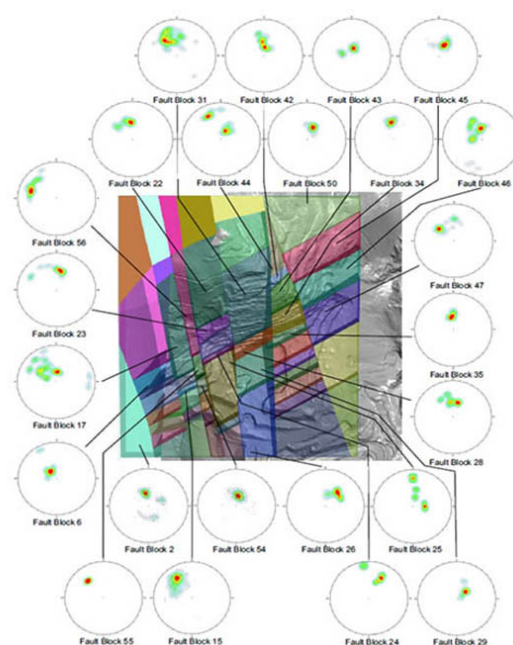


Figure 11—Bedding – fault network and intervening blocks, with contoured bedding orientations

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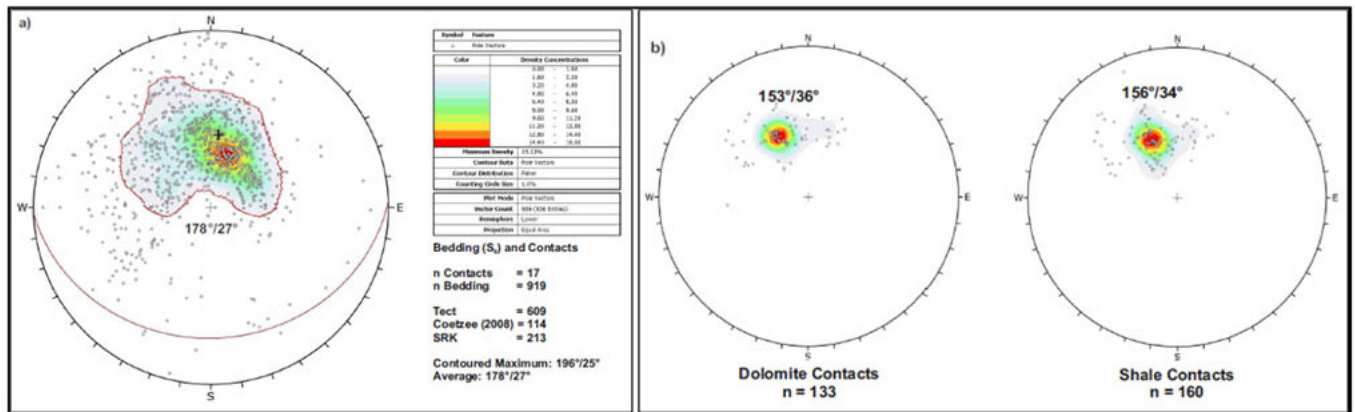


Figure 12—Structural data summary: bedding and contacts

This partially overlaps with the contoured maximum of bedding but not necessarily the orientation of contacts derived from pit mapping (196°/25°; Figure 12). This may denote an acute angular relationship between bedding and contacts, or some form of rotational deformation between the two, which is entirely possible as the Kumba pit occurs close to a lateral ramp within the Northern Range.

Banded iron formation (BIF)-diabase and BIF-shale-dolomite contacts are tectonised, characterised by top-to-the-N kinematics. To reiterate, from a geotechnical or back-analysis viewpoint, these (particularly dolomite-shale-BIF contacts) should be modelled as broad, weak zones. In the case of the BIF-shale dolomite transition, this has a combined thickness of 40-50 m, straddling the modelled lithological contact in the 3D model. Furthermore, the thrust upper contact between the lower diabase sill and BIF is defined by a finely-laminated and sheared clay-calcite-rich zone with occasional fault gouge and quartz schlieren, as illustrated by Figure 13 (Basson, Koegelenberg, 2017), over a maximum thickness of approximately 1 m. Consequently, there are numerous sites of weakness, focused along major contacts, distributed throughout the stratigraphy or tectonostratigraphy, combined with locally-variable, block-specific bedding orientations.

Three-dimensional geological model discussion

Figures 14 and 15 are cross-sections through an area that is suspected to be the focal point of the slope failure at Kumba pit. Included are previously-modelled dolomite surface as well as the pit surface prior to failure.

In the focal point, the steep angle (~61°) of dolomite (2014 dolomite surface) between adjacent drillholes does not honour the bedding, as the dip direction/dip of the two respective fault-

bounded blocks (Block 34 and 35) is constrained by mapping and downhole data: 170°/37° for Block 34, and 171°/28° for Block 35, respectively. Regarding the updated 3D model, a nearby mapping point for a fault, just off-section, has been recorded (330.3°/71.8°) and as such, informs the position and orientation of modelled Fault NE07 that is locally of high confidence. Consequently, the updated model supports brittle offset rather than steepening of the relevant dolomite contact. Notably, the local effects of fault drag cannot be excluded due to lack of data resolution, which otherwise has caused localised steepening of bedding in other parts of the pit.

Regardless of the lack of resolution of the updated 3D model, it does however strongly suggest that the sheared dolomite/shale and BIF contact, in Block 34, is vertically higher and closer to the pit surface, compared to the previous 2014 model. The original, explicitly-modelled dolomite contact (2014) is approximately 93 m from/below the pit surface, whereas the revised contact in Block 34 (this model) extends to within 50 metres of the toe of the slope.

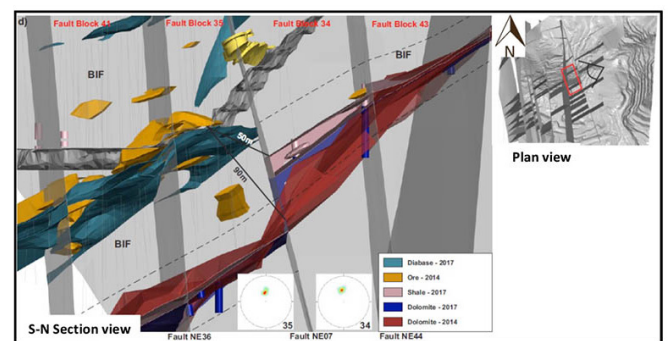


Figure 14—S-N geological section showing the suggested focal point of the June 2015 failure

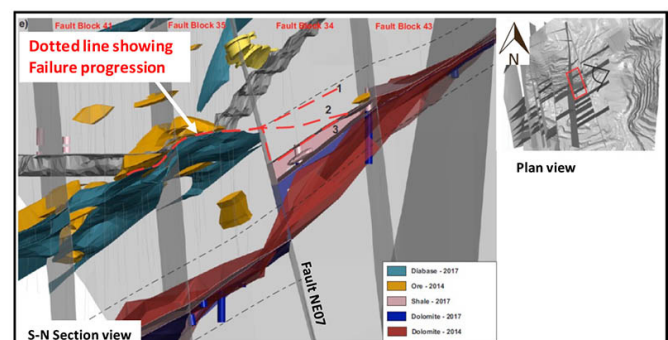


Figure 15—S-N geological section showing possible failure propagation paths

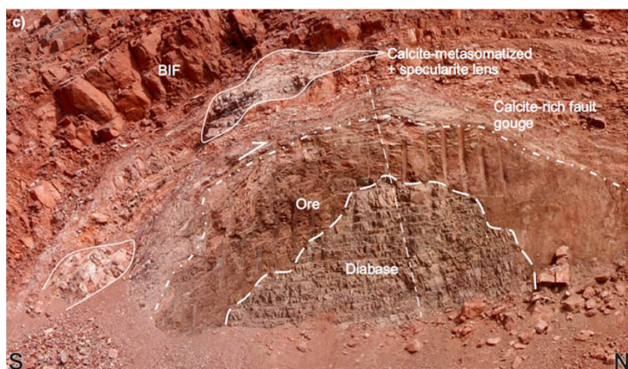


Figure 13—Thrust contact of folded diabase with ore and overlying BIF

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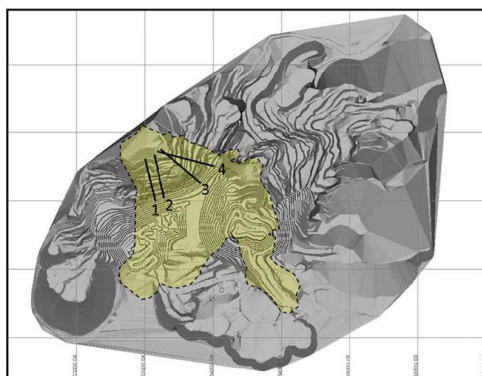


Figure 16—Locations of back-analysed cross sections on the final pit design

previous studies. Sections 3 and 4 intersected the diabase sill and 'atypical' ore, described in the previous section, towards the toe of the slope.

Geomechanical parameters

In general, geotechnical data levels at Thabazimbi Mine were low, with limited core drilling and laboratory strength test data available in the Kumba pit area. The quality of core drilling was poor, with core recoveries below 60%, due to difficult drilling conditions experienced in the Kumba pit. A geotechnical testing programme, delivering limited results, was completed prior to the geomechanical analysis of the 2008 Life of Mine pit layout. Therefore, geomechanical properties were derived mainly from surface mapping and from the limited laboratory testing data.

Table 4

Summary of rockmass parameters used in the 2D back-analyses

Geotechnical Unit Description	Code	KIO, 2008					SRK, 2014				
		Unit Weight (kN/m ³)	UCS (MPa)	GSI	mi	D	Unit Weight (kN/m ³)	UCS (MPa)	GSI	mi	D
Banded Iron Formation	BIF	32.74	165	40	9	0.7-1	31	244	43	14.7	0.7-1
Dolomite	DOL	28.55	81	40	9	0.7-1	28	106	53	13.1	0.7-1
Shale	SHALE	28.01	27	26	6	0.7-1	28	27	26	6	0.7-1
Diabase	DIA	28	28	34	15	0.7-1	28	28	34	15	0.7-1

There is another combination of features that may have had a significant effect on the strength of the toe of the slope (Figure 15); a thin sliver of 'atypical' ore, directly in contact with a sill beneath it, which runs into Fault NE07 from the south, that has been downthrown due to the abovementioned apparent reverse movement along Fault NE07. This feature is extremely close to Benches 1 and 2 of the pre-failure pit and immediately down-dip of the area wherein the dolerite surface was close to the pre-failure pit surface. These features are reasonably considered in the context of a broad (40-50 m) tectonised zone. The outline of this zone is shown on Figure 15 as a dashed, line and it is apparent that this overlaps with the fault-juxtaposed atypical ore zone. As a first approximation and working concept, Figures 15–18 highlights several proposed paths of propagation for the failure, aided by lateral, NNW-trending faults that acted as release planes.

Two-dimensional back-analysis

In order to validate the root cause of the slope failure, a two-dimensional (2D) stress-strain deformation analysis was first conducted using the RS2 Finite Element Analysis program by Rocscience. The analysis considered two sets of rock mass parameters described in previous geotechnical design reports, as well as dolomite/banded iron formation (DOL/BIF) contact properties determined in previous back-analyses of slope failures in Kumba pit.

A previous back-analysis of a failure in a different sector recommended that slopes in the Kumba pit should have a minimum BIF thickness of 50 m between the slope toe and the DOL/BIF contact. Although located in a separate geotechnical design sector from the Kumba South pit, this criteria was considered appropriate for the failure area due to the similarity of the lithologies present.

The location of the four sections utilised for the back-analysis is illustrated in Figure 16. Sections 1 and 2 were found to have a pit slope toe-to-DOL thickness of 61 m and 70 m, respectively, which exceeded the 50 m of minimum thickness recommended in the

The rock mass strength was characterised using the Hoek-Brown criterion, with equivalent Mohr-Coulomb parameters calculated for the bedded units based on the Barton-Bandis criterion. A summary of the rockmass strength parameters is included in Table 4.

The DOL/BIF contact was considered to be the dominant failure mechanism and was the focus of this analysis. The contact properties were selected from previous reports to show how modelled slope stability changed as new insights into the contact strength properties were introduced. A summary of the contact parameters used in the first set of analyses in this study is shown in Table 5.

RS2 models were created for four cross sections spaced across the failure surface. A 5 m thick DOL/BIF contact was added to the models explicitly, with its associated strength parameters considered to cater for the contact as well as a continuous shale layer at the contact. The shale is not ubiquitous in drilling results, but due to its physical properties and the nature of the percussion drilling done in most areas, it has been assumed to be present throughout. Three contact strength scenarios were modelled for each set of rock mass parameters, including cases where there was no contact, a weak contact ($c = 1.5$ kPa, $\phi = 16^\circ$) and a moderately strong contact ($c = 9.1$ kPa, $\phi = 46^\circ$).

The moderately strong contact represents strength parameters for the shale, assumed to be present along the dolomite contact, while the weak contact values simulate a mix of shale and weathered dolomite (Wad) along the contact. Prior to the failure Wad was

Table 5

DOL/BIF contact shear strength properties

Report	Cohesion (KPa)	Friction Angle (°)
2008	No contact	
2011	9.1	46
2012	1.5	16

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Table 6
Summary of RS2 analysis results

Section	SRK 2008			SRK 2014		
	No Contact	Contact C= 9.1 kPa, $\phi = 46^\circ$	Contact C= 1.5 kPa, $\phi = 16^\circ$	No Contact	Contact C= 9.1 kPa, $\phi = 46^\circ$	Contact C= 1.5 kPa, $\phi = 16^\circ$
Final Pit Profile						
TBZ_1	1.39	1.38	0.78	1.98	1.72	0.85
TBZ_2	1.41	1.42	0.71	2.2	1.71	0.7
TBZ_3	1.29	1.21	0.49	1.39	1.6	0.5
TBZ_4	1.75	1.23	NC	2	1.31	0.49
Current Pit Profile						
TBZ_3	-	1.33	0.56	-	1.68	0.59
TBZ_4	-	1.83	1.06	-	2.61	1.0

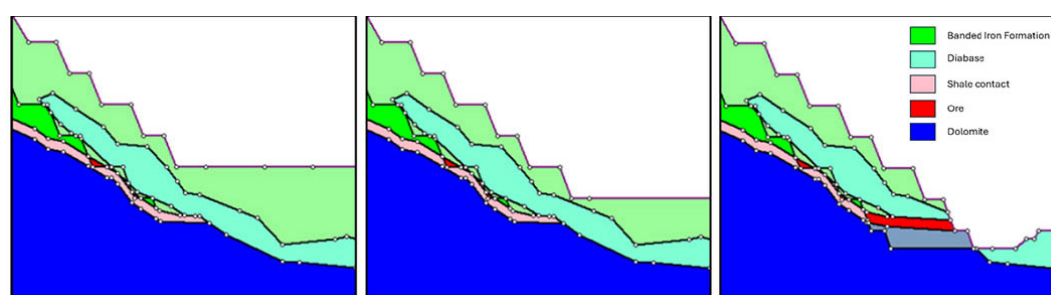


Figure 17—Simplified Section 3 pit layout variations: (left) pit profile prior to failure, (middle) intermediate pit profile, (right) final pit profile

expected to be present as dislocated localised pockets. However, the failure surface revealed a much greater extent of Wad present along the dolomite contact. In reality the back-analysed rock mass values for the failure are likely to fall between the moderately strong contact and weak contact values. A sensitivity analysis was carried out to define the likely range of material parameters in the area of the failure.

Analysis results

The first analysis was carried out using both the final pit profile for all four sections and the pit profile at the time of failure for Sections 3 and 4. A summary of the analysis is included in Table 6. All four sections were found to be stable when there was both no contact and a contact of moderate strength when the final pit profile was used. The stability of these sections increased when the pit profile at the time of failure was used (Sections 3 and 4). When a weak contact was present, all four sections were found to be unstable when modelled with the final pit profile. However, when the pit surface at the time of failure was used, Section 4 was found to be marginally stable and Section 3 was unstable.

A sensitivity analysis was completed for Sections 3 and 4 to investigate the influence the contact strength and toe burden had on slope stability. These sections were of particular interest, as slope monitoring data indicated that initial slope movement began in this area. Three burden depths were considered to simulate the influence of mining (and indirectly blasting) on slope stability. The three slope profiles shown in Figure 17 were analysed with contact strengths varying between $c = 1.5 - 9.1$ kPa and $\phi = 16^\circ - 46^\circ$ for each case. The results are illustrated in Figure 18.

The results suggested that the distance to the shale contact (toe burden) had a significant influence on slope stability. The variation of contact strength parameters indicated that slope stability was

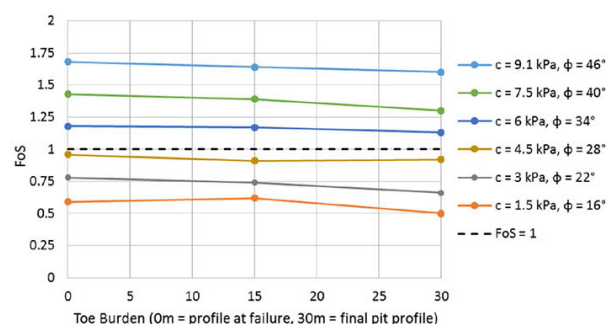


Figure 18—Section 3 sensitivity analysis results

also dependent on the contact strength. The models suggest that the actual contact strength parameters were approximately $c = 4.5$ kPa, and $\phi = 28^\circ$.

Three-dimensional analysis

Although the 2D analysis provided an initial understanding of the shear strength at the DOL/BIF contact prior to failure, it did not fully capture the complex interactions inherent to the 3D slope geometry. Factors such as geological structures, material anisotropy, and the step-path failure mechanism required a more comprehensive 3D approach to accurately back-analyse the failure.

Therefore, a three-dimensional analysis was performed using a Distinct Element Code, 3DEC by Itasca, to back-analyse the failure. 3DEC allows important structural features (e.g., faults, contacts) to be modelled explicitly and other minor structures to be included implicitly. Figure 19 shows that four stages of the pit development were modelled, which included: 1) pre-mining stage to achieve the initial stresses properly; 2-4 years 2012 to 2014 as pre-failure stages to accumulate displacements.

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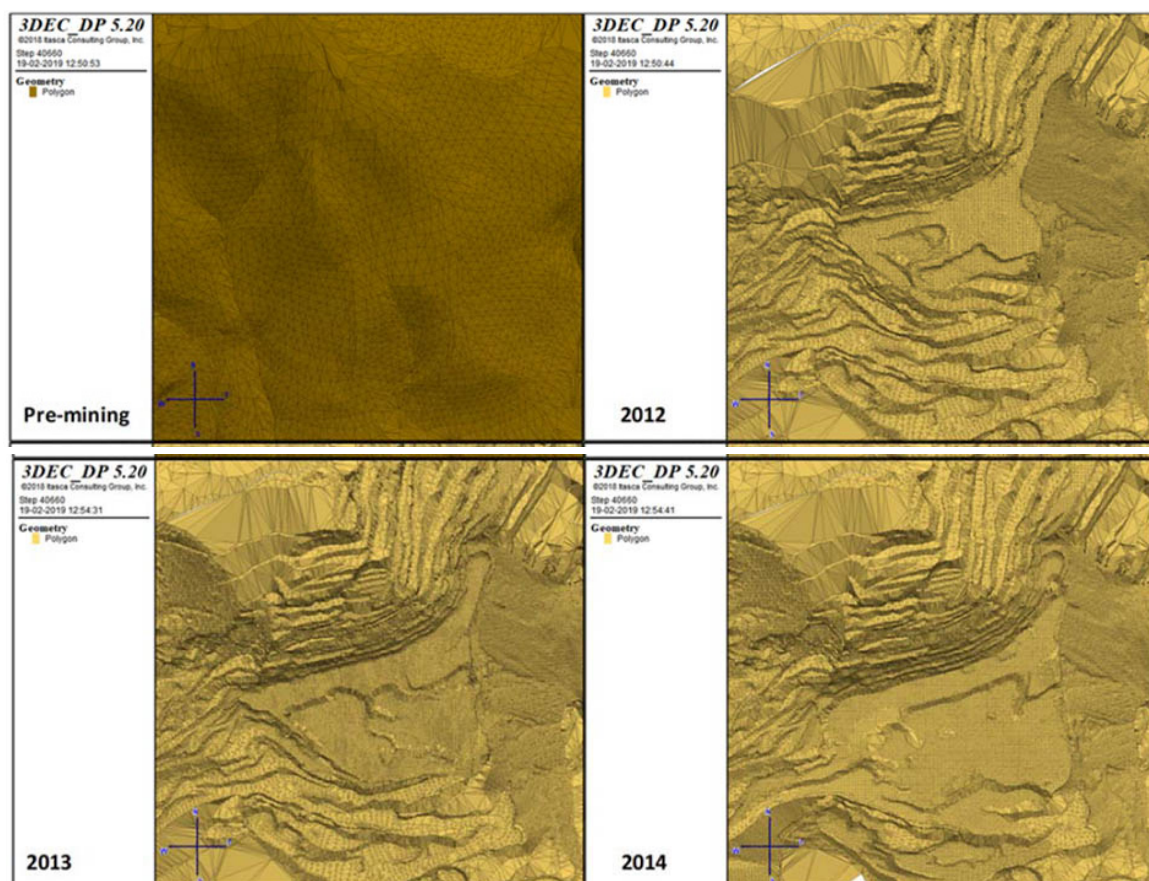


Figure 19—Topography for 1) pre-mining, 2) 2012, 3) 2013, and 4) 2014 stages

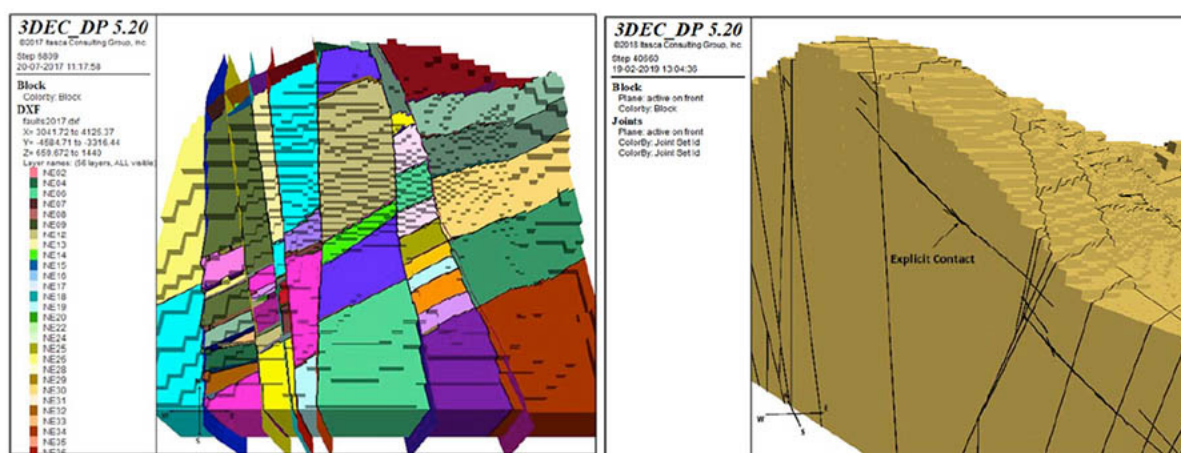


Figure 20—3DEC model of the Kumba pit showing principal faults (left) and shale-dolomite contact (right)

In the area of analysis, all principal faults and the contact between shale and dolomite material were modelled explicitly, as shown in Figure 20. Figure 21 shows the location of the contact as well as the distribution of all the materials in the numerical model.

Anisotropic rock mass properties

An important element in the failure back-analysis is the anisotropic behaviour of some of the lithologies, specifically banded iron formation (BIF) and shale (SH), on a rock mass scale. The anisotropy orientation in BIF is variable relative to the position in the wall, as presented in Figure 22. At present, it is not practical to simulate the explicit bedding within a large-scale 3DEC pit model due to the large computational requirements. For this reason,

equivalent continuum representation was employed to ensure reasonable computation times and in order to simulate the effects of anisotropic rock mass strength and deformation behaviour on pit slope stability.

Itasca has developed a ubiquitous joint rock mass (UJRM) modelling technique to account for rock mass anisotropy and scale effects in 3DEC. The ubiquitous joint constitutive model was developed to simulate well-defined strength anisotropy due to embedded planes of weakness within an isotropic continuum material. When used to simulate rock mass strength and deformation behaviour under unconfined compression, the UJRM model represents the progressive degradation of matrix cohesion and ubiquitous joint failure at various stages of loading.

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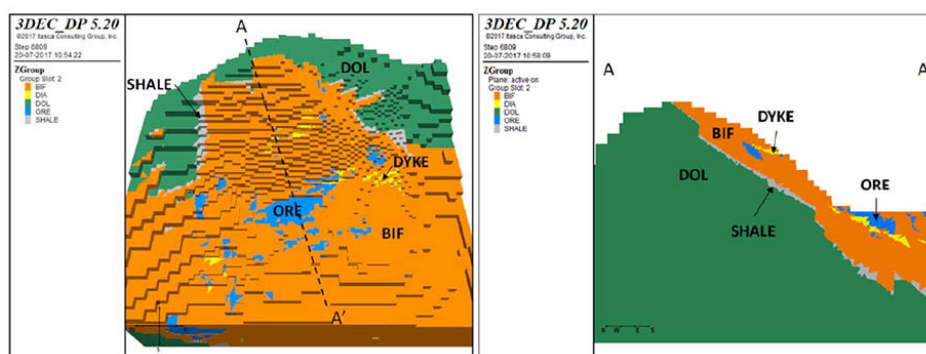


Figure 21—3DEC model of the Kumba pit showing plan view (left) and A-A' cross-section (right) of the lithology

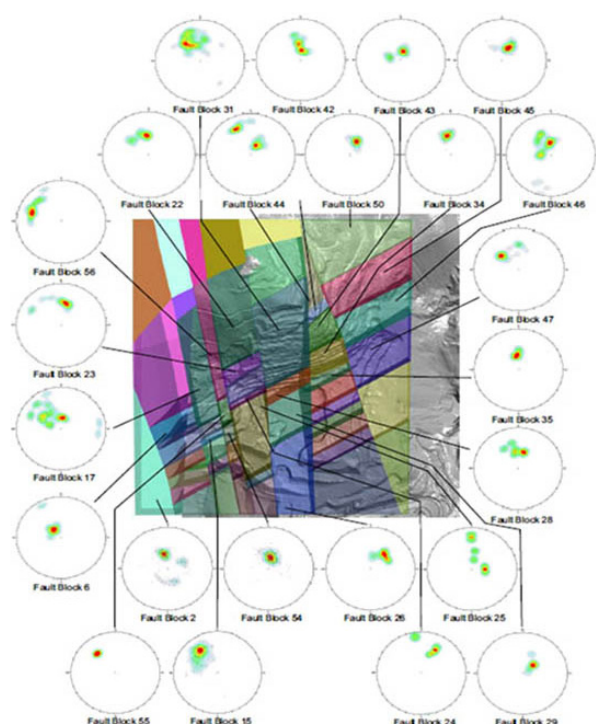


Figure 22—Anisotropy orientations per fault-bounded block in BIF

Table 7

Ubiquitous properties for BIF and SH units (Cabrera, Lorig, 2017)

	Cohesion (kPa)	Friction (°)	Tensile strength (kPa)	Dilation (°)
BIF structures	2.2	36	0	5
Shale structures	1.9	28	0	5

The anisotropic analysis was presented in a previous report for Kumba Iron Ore's Sishen Mine (Cabrera, Lorig, 2017). For back-analysis, the anisotropic behaviour for both BIF and shale material was used, with the ubiquitous joints' properties for these materials presented in Table 7. In the numerical model this has been assigned using different dips and dip directions for every zone, depending on its position. The anisotropy orientation in shale is parallel to the shale-dolomite contact.

Major structures included in the analyses were implemented explicitly in the 3DEC models. Their behaviour is represented by a linear Mohr-Coulomb failure criterion, and the residual values are

Table 8

Shear strength properties for the faults and shale-dolomite contact

	Cohesion (kPa)	Friction (°)	Tensile Strength (kPa)
Principal faults	10	23	1
SH - DOL Contact	0	25	0

$c_{res} = 0$ (cohesion), $\phi_{res} = \phi_{peak}$ (friction). Shear strength parameters of the structures (including principal faults and shale - dolomite contact) are presented in Table 8.

Brittle rock mass properties

It was decided to use an advanced constitutive model (CaveHoek) for considering strain-softening rock mass behaviour. The CaveHoek model was developed specifically by Itasca Consulting Group to describe a Hoek-Brown rock mass behaviour under caving conditions, but this has lately been used in open pit mining. The constitutive relation (shown simplified in Figure 23), allows representation of the nonlinear weakening behaviour of materials based on prescribed variations of properties as a function of the plastic shear strain.

The CaveHoek model allows for representation of elastic modulus softening, density adjustment, variable dilation, dilation shutoff, scaling properties to zone size, tension weakening, cohesion weakening, and frictional strengthening. In cases where the open joints controlling rock mass strength have a preferred orientation (as in Kumba pit), ubiquitous joints can be activated within the model with orientations that reflect their true orientation distribution. When the stress has exceeded the elastic limit, the rock mass yields by fracturing without losing all its cohesion. A full description of the Itasca CaveHoek model is provided in a paper by Lorig and Pierce (2000).

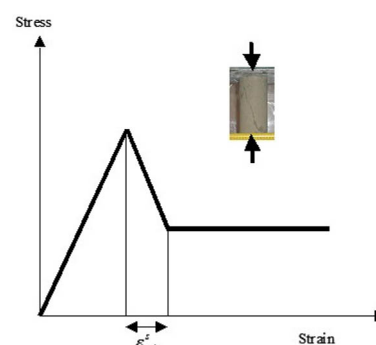


Figure 23—3DEC Strain softening constitutive behaviour

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Table 9

Geomechanical properties for the 3DEC analysis

	UCS (MPa)	GSI	m_i	E_i
Banded iron formation	92	27	15	38
Shale	27	26	6	51
Dolomite	106	53	23	52
Diabase	26	34	15	36
Hematite ore	27	26	6	51

CaveHoek residual properties must be defined in terms of Hoek-Brown parameters m_b , a and s . Fitting these parameters to the Barton shear strength envelope at 30% porosity results in the following values: $m_b = 2.0$, $a = 0.77$ and $s = 0$. The peak parameters must be defined in terms of σ_{ci} , GSI, m_i , and E_i . The values used in the analyses were obtained from the previous stability analyses and are shown in Table 9.

Analysis results

The numerical model results are analysed in terms of numerical velocity. An unstable condition is assumed in the model when, after a significant number of calculation steps, the numerical velocity remains aligned and above a certain value (different in each model). Figure 24 schematically shows the definition of stable/unstable related to numerical velocities.

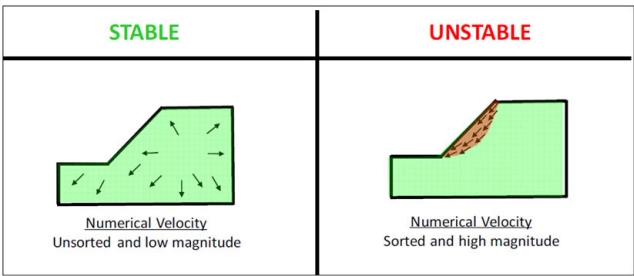


Figure 24—Definition of stable / unstable behaviour according to numerical model velocities

Using the original rock mass properties, the analyses showed stable conditions. Some rock mass properties had to be reduced, as listed in the following, to accurately simulate the failure:

- Contact between shale and dolomite: $c = 0$ and $\phi = 15^\circ$
- Principal faults: $c = 10 \text{ kPa}$ and $\phi = 25^\circ$
- BIF anisotropy: $c = 0$, $\phi_{\text{peak}} = 25^\circ$ and $\phi_{\text{residual}} = 20^\circ$
- Shale anisotropy: $c = 0$, $\phi_{\text{peak}} = 18^\circ$ and $\phi_{\text{residual}} = 16^\circ$

These properties were found to be realistic to simulate the failure numerically.

Figure 25 shows the calculated numerical velocities for 2012 to 2014 mining stages. Blue numerical velocity represents stable zones. Green numerical velocities represent yielding zones. Red numerical

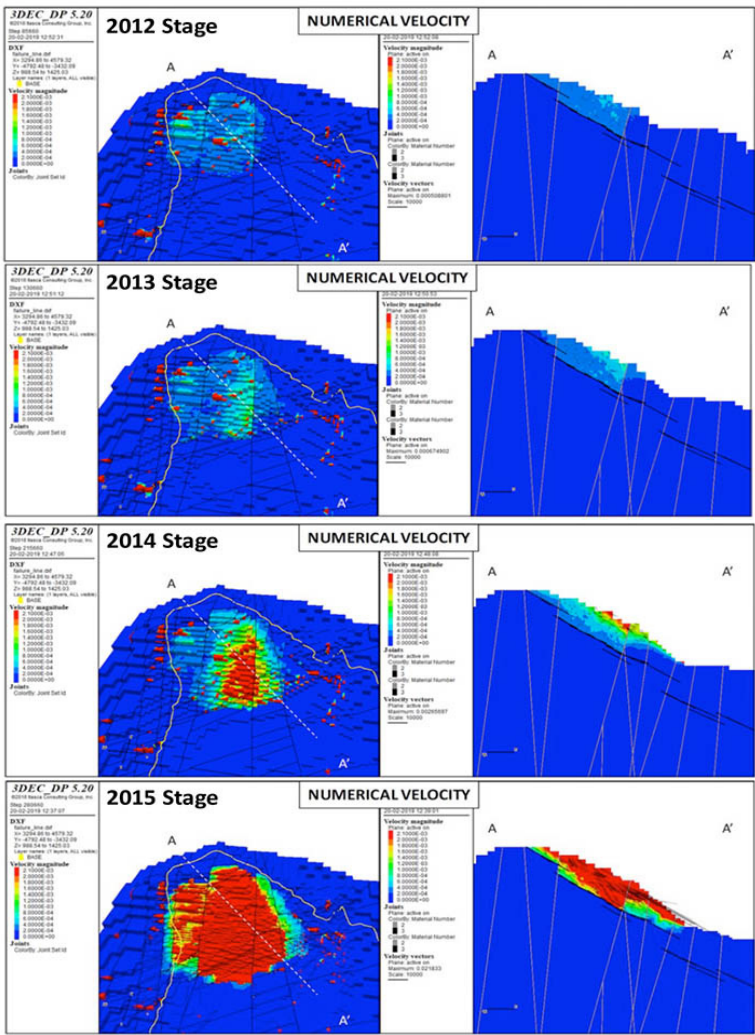


Figure 25—3DEC numerical model velocities for 2012, 2013, 2014, and 2015 (time of failure) pit development stages

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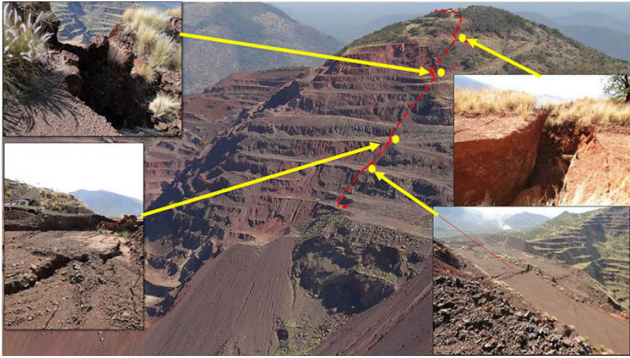


Figure 26—Failure progression

velocities represent local and shallow failures at the center of the slope. The yellow line represents the observed limits of the 2015 failure.

The numerical model is considered well calibrated as most of the slope presents high numerical velocities. The model showed stable conditions in 2014 (with some local failures at bench scale) and globally unstable in 2015 extending down to the shale-dolomite contact in some points. The mechanics of the movement is complex, sliding along the shale-dolomite contact and failing because of the BIF anisotropy. The western limit of the global failure is better achieved than the eastern one, because of the N-S principal faults orientation.

Because of the size and complexity of the numerical model, the use of a small strain assumption was mandatory. This assumption means that further degradation of the slope after the initial failure was not fully captured, as the nodes are not actually moving, just numerically.

In conclusion, the 3DEC modelling properties for the shale-dolomite contact, faults, and BIF-shale anisotropy had to be reduced to simulate the observed failure as accurately as possible. The resulting model closely aligns with the 2D back-analysis using RS2, which highlighted that stability was highly dependent on the contact strength. A logical conclusion is that the shale-dolomite contact was significantly weaker than initially expected, likely due to weathering, Wad formation, and possibly elevated pore pressures from surface water ingress.

Discussion

Based on the detailed back-analysis presented in the previous sections, the Kumba pit failure mechanism can be explained by the following sequence of events:

- **27-29 May 2015:** Mining activities at the eastern toe of the slope exposed a thin layer of atypical ore in direct contact with an underlying sill, which had been displaced by faulting. This faulting also brought the shale-dolomite and BIF contact in Fault Block 34—just north of the toe—closer to the pit surface. Initially, the modelled dolomite contact was located approximately 93 m below the pit surface; however, after revision, the dolomite contact in Block 34 was found to be within 50 m (or less) of the slope's toe. This shift triggered the initial relaxation at the eastern toe, marking the onset of a complex step-path failure.
- **Friday, 29 May 2015:** Prism monitoring indicated movement on the eastern side of the slope.
- **Monday, 1 June 2015:** Failure propagated along a north-south fault up the slope, triggering radar alarms.
- **Tuesday, 2 June 2015:** The failure then progressed from east to west, with cracks observed to be opening along faults and the shale-dolomite contact from the eastern highwall across towards the west (Figure 26). A north-south fault on the western side of the slope acted as another release plane, facilitating the failure's propagation down the western side of the slope. This effectively created a large, 'loose' block of ground on the BIF-shale-dolomite contact.
- **Wednesday, 3 June 2015:** Visual, ground-based, and helicopter inspections revealed alarming signs (Figure 27). The authors conducted an on-site inspection of the pit in the presence of the Mine Manager. During the visit, dust trails were observed rising along fault lines on both sides of the pit, accompanied by audible cracking noises emanating from the slope. These observations confirmed significant instability, prompting the immediate decision to evacuate the pit.
- **Thursday, 4 June 2015:** The rate of acceleration temporarily slowed, but despite the authors' inverse velocity calculation from the previous day predicting a failure at approximately 23:00 on Friday, strict no-entry orders remained in effect, except for moving a shovel to a safer distance during the morning.
- **Friday, 5 June 2015:** Continued unravelling of the slope along faults was observed. Radar monitoring indicated rapid acceleration over the entire slope area (Figure 28).
- **Saturday, 6 June 2015:** Failure occurred at 02:45 with a run-out of approximately 900 m in less than 5 minutes (Figure 29). The anisotropic nature of BIF—characterised by steeply dipping bedding on the western side, and therefore non-daylighting in the slope—contributed significant strength to

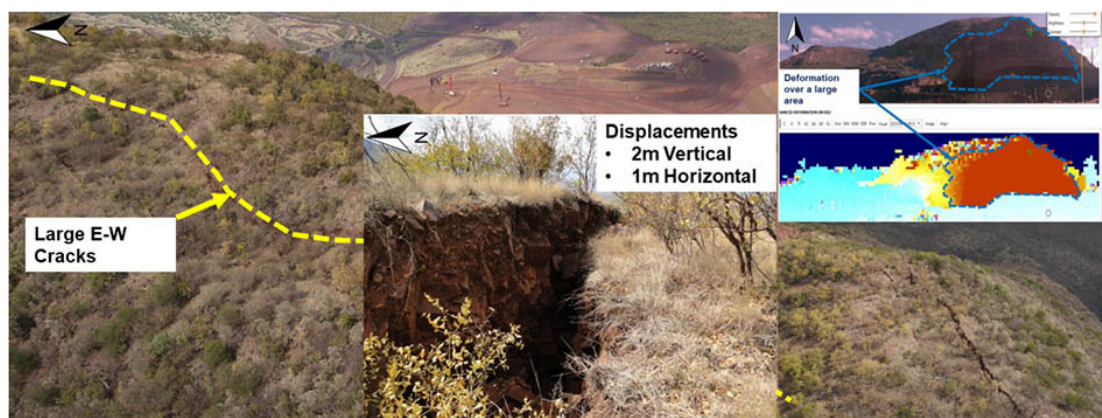


Figure 27—Deformation was observed over a large area

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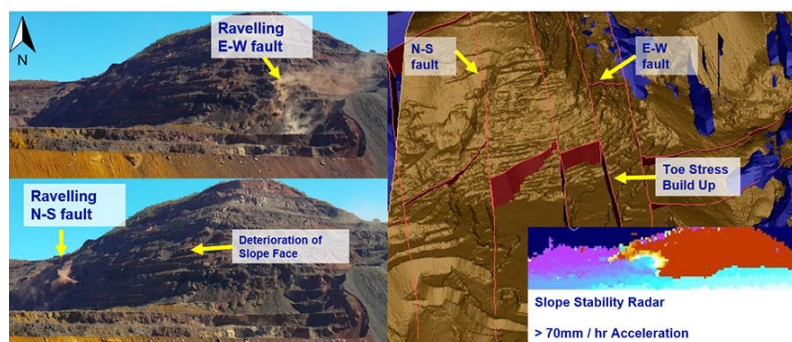


Figure 28—Rapid acceleration observed on Friday



Figure 29—Slope failure showing run out distance and the area from where approximately 280 000 m³ of water was ejected

the entire BIF rock mass. This strength allowed a buildup of stress at the toe of the failure surface. It is inferred that the accumulated stress triggered the violent nature of the failure, which also accounts for the extensive run-out distance. It may also be inferred that, had the BIF featured daylighting bedding (weaker orientation), failure would likely have shown movement at the toe earlier and exhibited slower movement, akin to the earlier Kumba West failure (a slow moving smaller failure that occurred earlier).

From a safety perspective, the failure was well managed, with no personnel exposed and no potential for unwanted safety consequences. However, from a financial standpoint, the event effectively sterilised the remaining ore, resulting in force majeure consequences for Kumba pit, which in turn precipitated closure proceedings for Thabazimbi Mine.

This raises an intriguing question: What if every uncertainty had been known prior to the commencement of mining? While hindsight offers perfect clarity, it is essential to consider the broader economic context that informs risk-reward decisions. The Kumba pit was actively mined for six years, up until 2015, supplying the domestic iron ore market under cost-plus contractual agreements. During this period, production from Kumba pit effectively 'freed up' tonnage from Sishen and Kolomela Mines for export, generating significant foreign exchange and contributing to Kumba Iron Ore's financial success, as well as to South Africa's economy through royalties and taxes.

In retrospect, had the full extent of geotechnical uncertainties been known—along with the costs and practical challenges of obtaining this data—it is likely that the pit would have been deemed uneconomical. The additional waste removal required to ensure stability could have rendered the project financially unviable, resulting in the premature closure of Thabazimbi Mine, along with the associated loss of jobs.

Therefore, every tonne extracted from the Kumba pit added significant value on the reward side of the equation, contributing

to both the local and national economy. At the same time, diligent geotechnical risk management ensured that no harm came to personnel, effectively balancing economic gains with the highest safety standards.

Conclusion

The development and operation of Kumba pit exemplified a complex balance between economic rewards and the inherent risks of mining. While the pit's eventual failure had significant financial implications, its six years of production played a vital role in supporting both domestic iron ore demand and the generation of foreign exchange through exports. At the same time, robust geotechnical risk management strategies ensured that personnel safety remained uncompromised, even in the face of uncertainty.

Key lessons learned from the Kumba slope failure are summarised as follows:

- When dealing with uncertainty, it is crucial to incorporate probabilistic analysis, along with event and fault tree analysis, to inform decision-making, rather than relying solely on model factors of safety.
- The significance of an accurate conceptual failure model as the foundation for all geotechnical design work cannot be overstated.
- Always calibrate limit equilibrium stability analysis with numerical modelling.
- When geological contacts are tectonised, they should always be explicitly modelled as broad, weak zones to assess the sensitivity of stability to their presence. If found to be a potential driver of instability, it should trigger targeted data acquisition to improve confidence in its parameters to improve slope design reliability.
- **Slope performance monitoring vs. critical monitoring:** While critical monitoring tools like slope stability radars (mitigation controls) ensure the safety of personnel, long-

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term strategic monitoring (preventative control) is essential for verifying slope performance and ensuring that measured movements remain within design tolerances.

- **Slope design verification:** Ongoing face mapping of each newly exposed bench is vital for reconciling the design with all four components of the geotechnical model, enabling early identification of any deviations or anomalies.
- One cannot be too conservative when it comes to run out distance assessment.

Ultimately, the decision-making process at Kumba pit reflects the careful navigation of risk and reward, where every mined tonne carried both economic value and the full implications of effectively managing potential geotechnical hazards.

Acknowledgements

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