LONG TERM STABILITY ASSESSMENT OF A STREAM DIVERSION AND WETLAND AREAS FOR PLANNED UNDERMINING AT GLENCORE'S GOEDGEVONDEN COLLIERY, MPUMALANGA PROVINCE

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by Carel de Beer, Bare Rock Consulting

1. INTRODUCTION

The Goedgevonden Colliery plan to mine the no 2 Seam and no 4 Seam under a rerouted tributary of the Zaaiwater Spruit and a wetland area on the eastern boundary of the property. Before the mining can commence an Environmental impact assessment of the proposed amended mine plan have to be executed.

Jacana Environmental Consultants contracted Bare rock Consulting to conduct an independent assessment of the risk that undermining of the Zaaiwater Spruit rerouted tributary, or the wetland area will pose for the environment after closure.

The impact to consider is surface subsidence and the safety risk that water can enter along preferred pathways into the underground workings, resulting in potential flooding. A secondary impact will be the starvation of the downstream aquatic environments that will be deprived of waters from upstream.

1.2. AIM

The aim of this assessment is to determine the likelihood that pillar failure after closure can occur and that the resultant surface subsidence will have an impact on the natural environment. The impacts considered are:

- Stream capture- where subsidence cause surface water ingress into the underground workings, resulting in reduced run-off from the surface drainage and the consequential impact of downstream streamflow
- Increased permeability of the sub surface area under the wetland due to pillar failure or roof collapse and possible draining of the wetland as a result.

1.2. LIMITATIONS

The information provided in this specialist report is based on information provided by the client and/ or the client's representatives, published scientific literature, maps, and information published in the public domain and that collected by Bare Rock Consulting (Pty) Ltd during the site investigation in the area.

1.3. AUTHOR'S CREDENTIALS AND DECLARATION OF INDEPENDENCE

The Author of this report Carel J de Beer is a professional engineering geologist, registered with the South African Council of Natural and Scientific Professions (Pri. Sci. Nat # 400211/05). Carel has 25 years' experience in the mining and civil industries and is a member if the South African Institute of Rock Engineers.

The compilation of the report, and any other work done by Bare Rock Consulting (BRC) for the Client Company, is strictly in return for professional fees. Payment for the work is not in any way dependent neither on the outcome of the work, nor on the success or otherwise of the Company's own business dealings. As such there is no conflict of interest in BRC undertaking the study as contained in this document.

1.4. SCOPE OF WORK

The scope of the project will be to model the pillar strength and the deterioration of the pillars over time to determine whether the pillars can fail and the likelihood that a pillar run can occur.

If a pillar run occurs, there will be a reduction in volume and due to the relative movement of the rock and soil, preferred pathways may develop along which water ingress can occur.

The outcome of the geotechnical assessment will be to determine the post closure risks with respect to surface expression of the underground mining. In order to achieve this goal, modelling on the likelihood, extent, and magnitude of potential subsidence and water ingress risk in the longer term will be required. The results of this study will feed into the calculation of the quantum for financial provisions for rehabilitation and closure.

The two areas considered in this assessment is the GGV Central Block that runs below the rerouted tributary of the Zaaiwater Spruit and the Eastern Underground Block along the eastern boundary of the property. The two areas are indicated in red in figure 1 below.

2. AVAILABLE INFORMATION CONSIDERED

The following data were supplied or requested and used in this assessment:

- Layout plan of the proposed mining areas and the current underground workings.
- Previous study to define the stability of the old workings under the additional load of waste rock dumps
- PDF file with borehole logs
- PDF plan showing available borehole positions relative to the proposed workings
- Presentations and figures indicating the contours of the roof of the no 2 seam and the no 4 seam in the areas of interest.
- Site visit on 26 October 2021

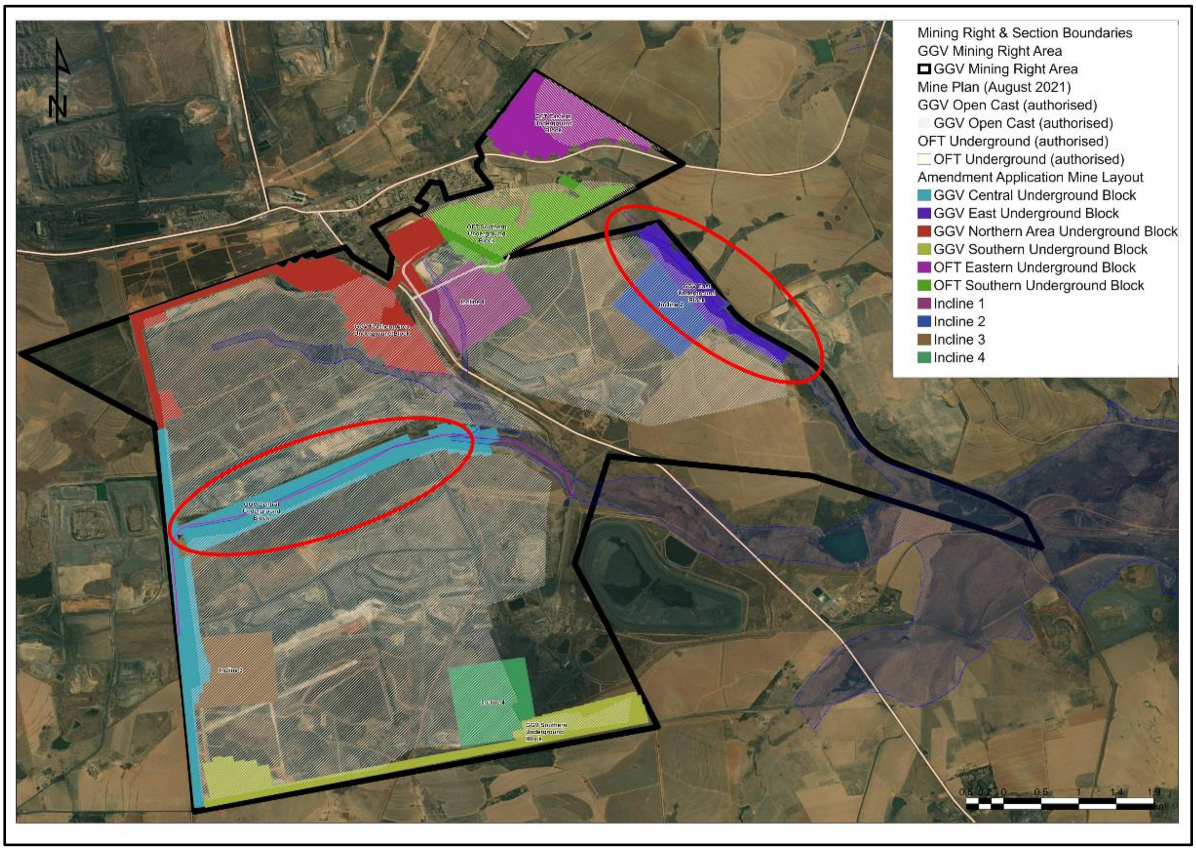


Figure 1: Undermined areas considered

3. SITE ASSESSMENT

3.1. PRINCIPALS CONSIDERED IN THE STABILITY ASSESSMENT

Surface subsidence due to undermining occurs when a long wall panel is allowed to close under controlled conditions or when the pillars underground fail and this failure led to a progressive failure of panels within a mining block or larger area.

The failure of a single squat pillar is unlikely to result in a surface subsidence.

Considering that surface settlement and fracturing of the overburden above a room and pillar mined area depend on the time dependent stability of the underground pillars, the strategy to determine whether there will be surface subsidence and potential ingress of water into the underground workings under the diverted stream and the wetland area (figure 2) depend on the stability of the pillars left behind when the 2 Seam and 4 Seam are mined. If the pillars collapse under the overburden load, the rock mass above the span between the pillars will start to fail in tension and compression resulting in angled and vertical cracks forming in the overburden. These cracks will form conduits along which surface water can enter the rock mass, over time, erosion can occur. This erosion will allow increased infiltration, resulting in flooding of the old workings. This infiltration will also rob the downstream aquatic system of water. The stability of the pillars of both the 2 and 4 seam mining areas, after mine close is paramount to the survival and functioning of the drainage and wetland area. Refer to figure 1 for the areas under consideration where the mine plan to extract coal from both seams.

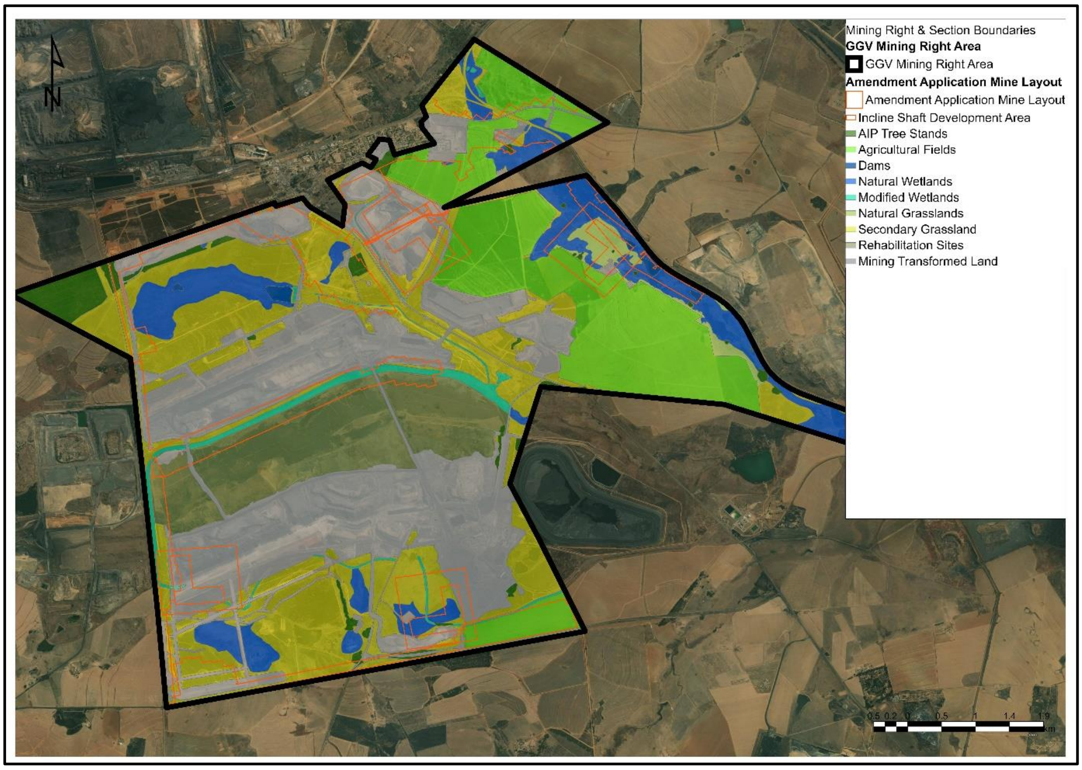


Figure 2:Amended application areas and indicated surface land use

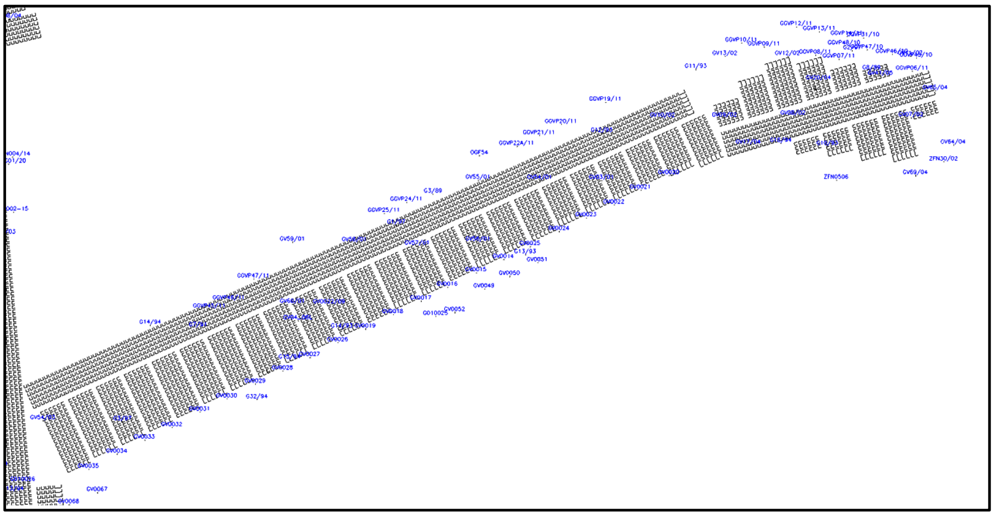


Figure 3: Available boreholes in the central underground area

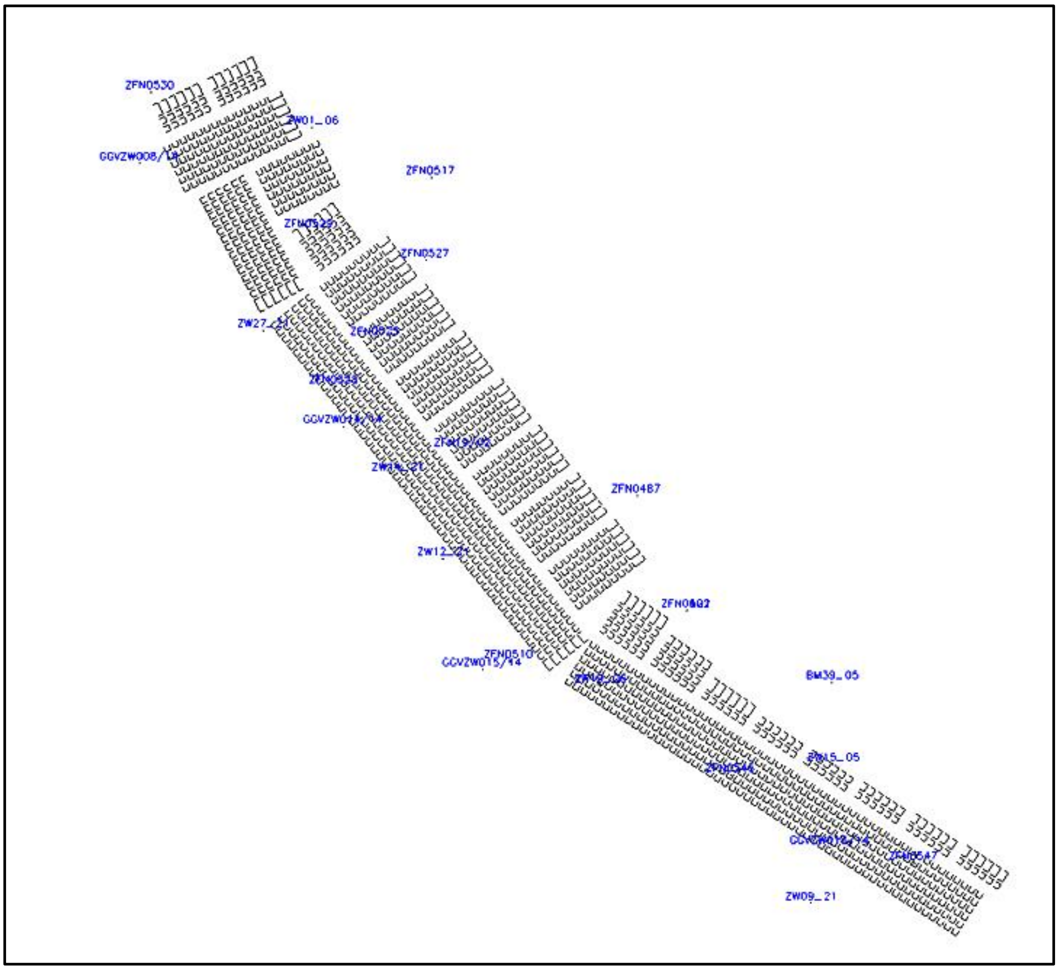


Figure 4: Boreholes in the eastern underground block

The borehole information (figure 3 an4) provided as pdf list by the mine was used to draw up the contour plans (also provided by the mine) from the roof of 2 seam to surface and the roof of 4 seam to surface as shown in figures 5 and 6.

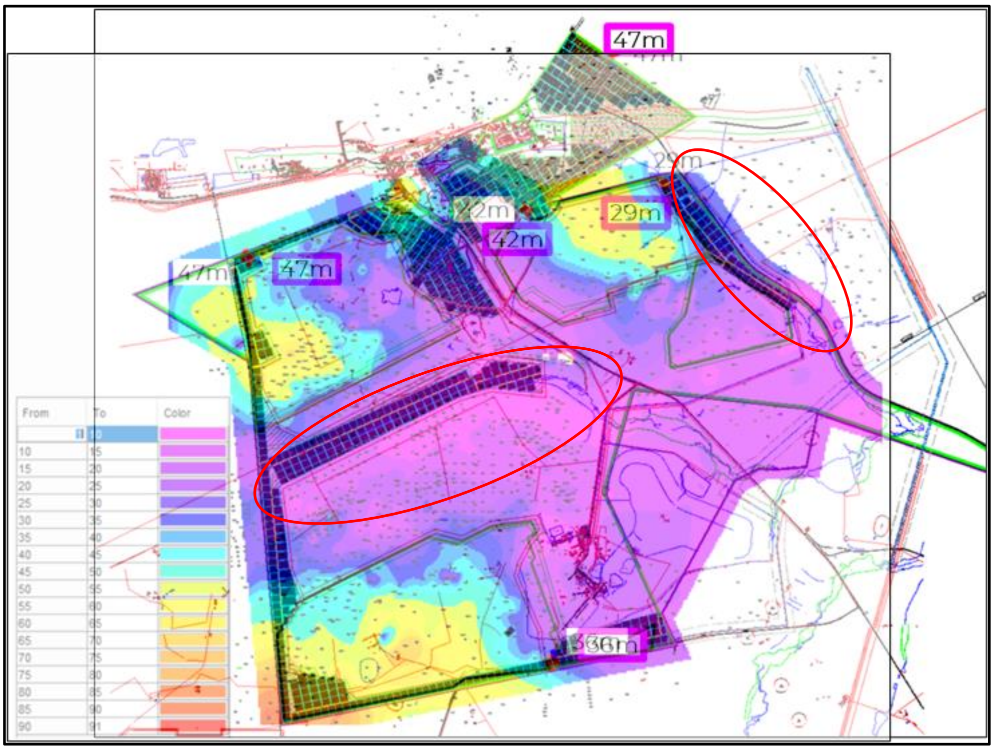


Figure 5: Overlay of the depth of the roof of the no 4 Seam within the study area

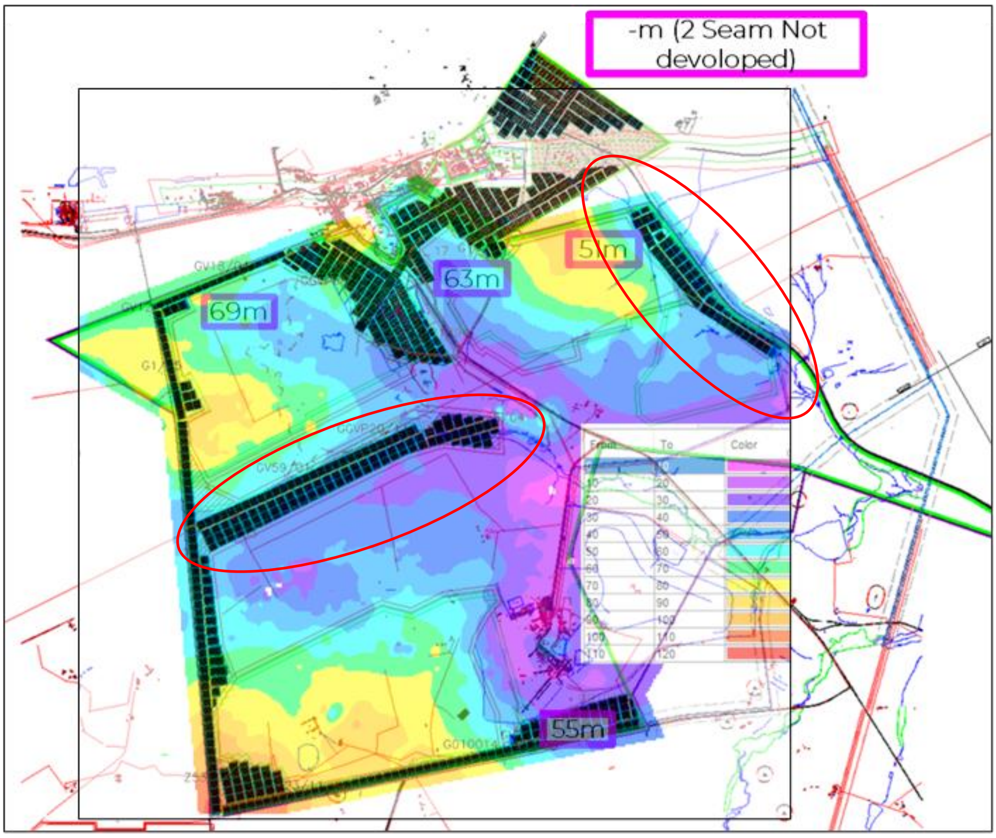


Figure 6: Depth of 2 seam roof below surface.

3.2. PROPOSED MINING LAYOUT

The no 4 seam varies in depth of between 10 and 25 m with the average being 15m below surface in the area under the diverted river. In the eastern area where the wetland is located, the roof thickness of the no 4 seam varies between 15 and 30m with the average thickness of 20m

The roof of the no 2 seam under the river diversion varies in thickness between 40m in the west and 20m in the east. In the eastern area under the wetland the seam varies in depth from north to south from 60m to 40m below surface

Table 1:Geometry of pillar design and depth below surface for the different areas.

Area and Seam	Mining Height	Pillar Width	Centre to centre distance	Mining depth below surface
	m	m	m	m
Central 2S min	3.5	11.5	18	20
Central 2S ave	3.5	11.5	18	30
Central 2S max	3.5	11.5	18	40

Central 4S min	3.5	11.5	18	10
Central 4S ave	3.5	11.5	18	15
Central 4S max	3.5	11.5	18	25
East 2S min	3.5	11.5	18	40
East 2S ave	3.5	11.5	18	50
East 2S max	3.5	11.5	18	60
East 4S min	3.5	11.5	18	15
East 4S ave	3.5	11.5	18	20
East 4S max	3.5	11.5	18	30

3.3. SQUAT PILLAR DESIGN BACKGROUND AND THEORY

3.3.1 Background to current design

Following the Coalbrook disaster in 1960, it was realized that a method to define coal pillar strength was urgently required, as no satisfactory method existed at the time. It was soon seen that laboratory tests on coal specimens would not result in an acceptable outcome. The main problem was that strength is size-dependent and transforming the laboratory test results to real coal pillars was not sufficiently accurate.

It has been shown since the very early times that the strength of a rock pillar,σ_p, can be described by the following fundamental equation:

$$\sigma_P = kw^{\alpha}h^{\beta}, \qquad [1]$$

Where,

k= constant related to the material strength

w= pillar width

h= pillar height

α and *β* are constants related to material type.

The constant β consistently has negative value. The strength of a pillar is thus directly proportional to its width and inversely proportional to height.

Salamon and Munro (1967) overcame the size obstacle by using cases of failed pillars underground in a statistical analysis to determine the critical parameters. What they essentially did was to set up two databases: one for failed cases (27 cases) and one for unfailed cases (98 cases). The selection criteria are described in the reference. The analysis was thus based on real pillars and not laboratory-sized specimens. Note that the database of failed cases represented the full population of known failures, while the unfailed database was a representative sample of the unfailed cases.

It was reasoned that if both the strength of pillars and the loads imposed on them, σ_L are known, it would be possible to have a measure of stability. The simplest measure of stability is merely the ratio of pillar strength to the load imposed on it, the safety factor (SF):

$$\mathrm{SF} = \frac{\sigma_P}{\sigma_L} \qquad [2]$$

The pillar load is simply assumed to be caused by the weight of the overlying strata, distributed equally over the pillars, or

$$\sigma_L = \rho g H \frac{(w+B)^2}{w^2} \qquad [3]$$

Where *H* = depth to floor of the workings

B = bord width.

(Note for clarification - in the context of this paper, *pg* is often replaced by 0.025 MN/m^3, and by the pillar centre distance, *C).*

The maximum likelihood estimation method was used to determine the parameters k, α, and *β* to be used in Equation [1]. The maximum likelihood function as it was used is based on the assumption that pillars failed at a load equal to their strength, *i.e.* at SF = 1. The method then results in values for k, α, and *β* that cause the frequency distribution of SF to be as densely concentrated around a value of unity as possible.

The maximum likelihood function also takes account of the existence of unfailed pillars in the sense that the two fundamental requirements in the calculation are that for the failed database, SF = 1 and for the unfailed data base, SF > 1. Yet, the method is much more reliant on the failed cases than the unfailed ones as the first requirement dominates the process.

Although this is not claimed by the authors, criticism of the method includes the salient assumption that at the time of mining, the pillars contained in the failed database had dimensions resulting in SF = 1 - either intentionally or by chance. This is not the case, as pillar sizes were determined by operational requirements more than stability and pillars could thus have any size suiting those operational requirements as long as failure did not occur during the mining operation.

It furthermore assumes that pillar dimensions remained constant over the period until they failed, which is also known not to be the case. It does not explain why pillars fail at vastly different ages, ranging from just a few months to more than 50 years. The authors also noted that it was very likely for different coal seams or regions to have different strength constants, but there was simply not sufficient data to perform the analysis for different areas.

Nonetheless, the constants that Salamon and Munro determined were successfully used in the mining industry for several decades, and while more pillar failures have occurred since the introduction of the formula, there has not been a repeat of the Coalbrook disaster. The additional failures that occurred do not imply that the formula as it was used was incorrect. If anything, they demonstrate the nature of variability, and with that, the power of using a probability of failure approach to pillar design.

Salamon and Munro found the following:

K = 7.2 MPa

α = 0.46

β = -0.66

It has to be noted that while k in terms of the equation equals the strength of a cubic metre of coal, it is not a laboratory-determined unit strength: it is a statistically determined number related to the material strength. For this reason, k should be seen against the background of the linked group of constants. It is incorrect to develop a new strength formula by merely changing the value of k without at least investigating the impact on α and β.

3.3.2. Pillar Strength Analysis

From van der Merwe (2016) the amount of scaling by which pillar width is reduced, *dT,* after a time T, is:

$$dT = mh^x T^{1-x}, \qquad [6]$$

Where

h = mining height (m)

m= constant, 0.1799

x= constant, 0.7549

T= age of pillars (years).

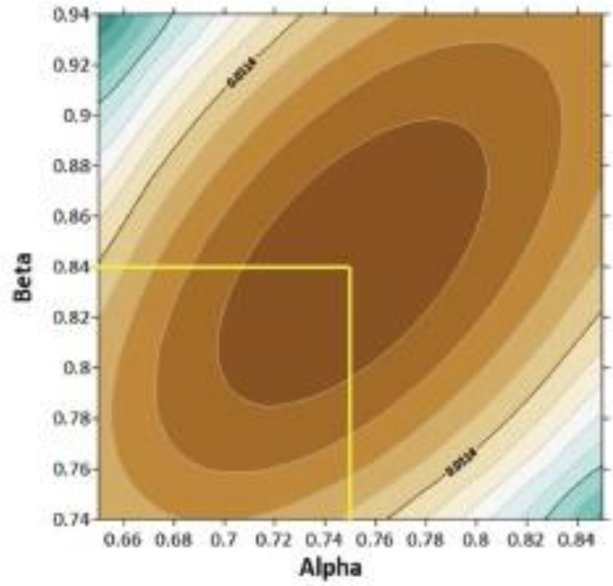
At any given point in time the reduced pillar width, w_T, is then

$$w_T = w_0 - dT \qquad [7]$$

Where w_0 is the as-mined pillar width or the design pillar width

and w_T is the pillar width at time T

In essence, the values for the parameters α and β are found by iteration to result in the combination that displays the smallest overlap between the frequency distributions of the logarithms of the failed and unfailed cases. Figure 2 is a contour plot of the areas of overlap for the final round of iterations. It was seen that α = 0.74 and β = 0.85 were the optimal values. It was then found that k_T= 10.2 MPa satisfied the criterion that the median of the SF of failed cases should be unity. It is important to note that the material strength constant, k is in the same range as that found by the direct strength tests for which the dimensions at the time of failure were exactly known.



Graph 1: Relationship between alpha and Beta values

(After van der Merwe 2016)

Equations [6] and [7] were then applied to the pillars in the databases. For the failed database, the ages of the pillars at the time of collapse were used. For the unfailed database, the year 2012 was used as the base since the database was set up in 2012.

The equation for pillar strength, which now incorporates the effects of time, is then as follows:

$$\sigma_{S,T} = k_T \frac{w_T^{\alpha}}{h^{\beta}} \quad \text{MPa} \qquad [8]$$

In the expanded form the full equation can be written as

$$\sigma_{S,T} = k_T \frac{\left(w_0 - m h^x T^{1-x}\right)^{\alpha}}{h^{\beta}} \quad \text{MPa} \qquad [9]$$

Note that the pillar load to be used in conjunction with Equation [3] for the determination of the safety factor, should also be based on the reduced pillar width. The load is then expressed by

$$\sigma_{L,T} = \frac{0.025 H C^2}{w_T^2} \quad \text{MPa} \qquad [10]$$

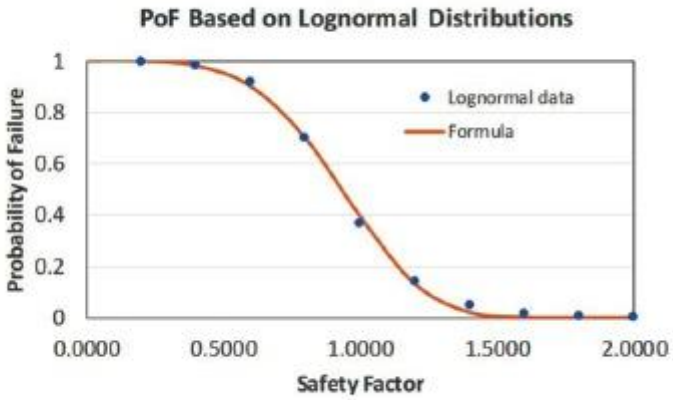
Where *H* is the depth to the floor of mining and *C* is the pillar centre distance, which is not affected by time. The safety factor at any given point in time is then

$$SF_T = \frac{\sigma_{S,T}}{\sigma_{L,T}} \qquad [11]$$

3.3.3. Probability of Failure

It is well known that the probability of failure is a more rational indicator of pillar stability than the safety factor due to the variability of the input parameters.

The best way to determine the probability of failure is to compare the number of failed cases in the database to the total number of cases in each interval of safety factor.



Graph 2: Probability of failure vs Factor of safety.

The probability of failure as a function of the safety factor is:

$$\text{PoF} = \exp\left(-0.93SF_T^{4.28}\right) \qquad [12]$$

According to Equation [12], the PoF has a value of 0.5 at a safety factor of 0.93, which is close to the statistical expectation of 1.0.

Equation [12] is based on the ever-reducing pillar width due to ongoing pillar scaling. It therefore results in the probability of failure at any given point in time. The probability of failure increases over time as the pillar width reduces.

3.3.4. Pillar life index

With the new equation for pillar strength and the more realistic value of safety factor at the point where a failure probability of 50% is reached, the equations for the critical scaling distance had to be adapted. This will have a downstream effect on the pillar life index as reported by van der Merwe (2016).

Following the same line of argument as in van der Merwe (2016), the new equation for the critical scaling distance should calculate the scaling distance that will result in a failure probability of 50%, which implies a safety factor of 0.93. The equation which is then derived from Equations [9] and [10] and with substitution of α = 0.74, β = 0.85, and k_T = 10.2 MPa is:

$$d_c = w - \left[0.002279Hh^{0.85}C^2\right]^{0.365} \qquad [13]$$

The pillar life index, PLI, is:

$$\text{PLI} = \left[\frac{d_c}{mh^x}\right]^{\left(1/_{1-x}\right)} \qquad [14]$$

Where *m* = constant, 0.1799

x= constant, 0.7549.

4. RESULTS OF THE STASBILITY ANALYSIS

4.1. Pillar Stability

The pillar deterioration using the as design pillar dimansions (as provided by the mine in cad file format) Pillar size of 11.5m and bord width of 6.5m and an average mining height of 3.5m were calculated for 10, 20, 30 50 and 100 years.

Table 2 indicate that the pillar scaling over time is not influenced by the mining depth as the change in size over time is constant. Where the depth below surface does play a role is with the calculation of the minimum safe pillar size (dc). For the shallower piles the dc value is larger. For example, the minimum safe pillar size where the overburden is just 10m thick is 8.44m and the safe pillar size for a pillar at 60m is only 5.61m.

The pillar life index indicate that the pillars will have a support lifespan of in the order of 200 years (range from 184 years to 204 years).

Table 2: The time dependent deterioration of the pillars over time

	Mining Height	Pillar Width	Center to centre distance	Mining depth below surface	Pillar size over time					Critical scaling distance	Pillar Life Index
	m	m	m	m							centuries
	h	w	C	H	wT10	wT20	wT30	wT50	wT100	dc	
Central 2Smin	3.5	11.5	18	20	10.69	10.53	10.43	10.29	10.07	7.56	1.98
Central 2Save	3.5	11.5	18	30	10.69	10.53	10.43	10.29	10.07	6.93	1.94
Central 2Smax	3.5	11.5	18	40	10.69	10.53	10.43	10.29	10.07	6.42	1.91
Central 4S min	3.5	11.5	18	10	10.69	10.53	10.43	10.29	10.07	8.44	2.04
Central 4S ave	3.5	11.5	18	15	10.69	10.53	10.43	10.29	10.07	7.95	2.01
Central 4S max	3.5	11.5	18	15	10.69	10.53	10.43	10.29	10.07	7.95	2.01
East 2S min	3.5	11.5	18	40	10.69	10.53	10.43	10.29	10.07	6.42	1.91
East 2S ave	3.5	11.5	18	50	10.69	10.53	10.43	10.29	10.07	5.99	1.87
East 2S max	3.5	11.5	18	60	10.69	10.53	10.43	10.29	10.07	5.61	1.84
East 4S min	3.5	11.5	18	15	10.69	10.53	10.43	10.29	10.07	7.95	2.01
East 4S ave	3.5	11.5	18	20	10.69	10.53	10.43	10.29	10.07	7.56	1.98
East 4S max	3.5	11.5	18	30	10.69	10.53	10.43	10.29	10.07	6.93	1.94

4.1.1. Overburden and roof stability

The immediate hangingwall of seam 4 and seam 2 are shale. Shale is generally fine grained and thinly bedded with a low permeability and low porosity that will form a natural aquitard for any groundwater seepage or in this case percolation from the unlined diversion in the central block.

Considering stable span theory for a span between two supports the thickness of the roof should be at least three times the span. The design span width along the room and pillar area is 6.5m, thus the minimum safe roof thickness is 20m. This assume no defects or discontinuities in the roof and that the roof is properly supported according to the support standards defined during mining.

In the wetland area of the eastern block the low permeability and porosity conditions will be more prevalent due to the natural decomposition of the rock mass and the settlement of the fine particles in the wetland system.

4.1.2. Shallow Mining Conditions

Considering the relative shallow middling from surface to the coal seams in the areas under consideration, stable span theory indicate that the stability of the roof between pillars will not be stable over the long term where the mining is shallower than 20m below surface with a significant risk of fracturing and erosion (self-mining) and the ultimate collapse of the roof between pillars and at intersections of travel ways. This will result in ingress of water into the old workings and downstream starvation of the aquatic system.

4.1.3. Water ingress risk

The stream diversion channel only receives water from the upstream catchment during above normal rainfall events, therefore the exposure to water ingress is limited to periods of heavy rainfall.

Down-stream from the bridge the risk of artificial recharge of the diverted river increase as stormwater run-ff from the haul road is allowed to enter the system (photo 2).

Photo 1: Westward view of the stream diversion channel over the central underground block.

4.1.4. Groundwater levels and decanting

Over time it is likely that the workings will flood due to the groundwater level returning to a natural state. Ingress of surface water as a result of fracturing of the overburden where the roof is stressed will increase the rate of flooding of the workings. Communication with the groundwater consultant and the mine environmental team, it is likely that the planned workings will decant at the eastern end of the Central Area.

The central and eastern mining areas is located close or at the current topographical low areas. Water ingress from the disturbed ground both north and south of the central area will also increase the groundwater recharge and the rise of the groundwater to pre- mining levels after mine closure.

Photo 2: Area where stormwater run-off from the road is allowed to enter the drainage.

5. CONCLUSIONS

The pillar dimensions proposed for the mine plan of the with a pillar size of 11.5m skin to skin and a bord width of 6.5m and a maximum mining height of 3.5m is stable and will be able to support the roof for the duration of the mining and for approximately 200 years thereafter.

Localized unforeseen geotechnical conditions such as discontinuities and localized minor faults and geological displacements may occur, but roof mapping and regular inspections will identify such areas and roof support will be installed as per the mine's support standards.

After mine closure roof slabbing can occur, this result either in doming or self-stabilization within 3x the span. When doming occurs, the roof usually self stabilizes. Where fractures close to the pillars edge occur, slabbing can occur and the self-mining of the roof in shallow areas can continue to surface. Prediction of ground behaviour in shallow conditions < 20m from surface is less predictable and it is likely that sinkholes and or surface subsidence can occur.

The river diversion area only flows after periods of heavy rainfall therefore the exposure of potentially hazardous conditions when water ingress can occur is limited to that time and a period thereafter.

The middling between the proposed workings and wetland area in the eastern underground block is a little thicker and their risk of seepage and water ingress is less due to the nature of the soil. Doline formation rather than sinkhole formation is likely to occur in areas where mining is between 20 and 40m below surface. For areas where mining will be shallower than 20m subsidence and inrush is likely to occur over time.

6. RECOMMENDATIONS

Monitoring of rainfall in the catchment area of the stream diversion is important to determine the rainfall level that report run-off. The flow at the entry point, west of the workings and at the eastern extent should be monitored and recorded.

The water level and the discharge from the wetland just downstream wetland area must be monitored and recorded to define a base level flow. Any reduction in the ratio between rainfall and flow will indicate stream capture and underground ingress up stream.

Any seepages and especially seepages from vertical to near vertical discontinuities in the roof must be mapped and the areas monitored, and the inflow estimated. Periods of inflow and dry periods must be noted. The wet areas must be plotted on a plan and the diversion channel indicated. In future these would be the likely areas where sinkholes or surface subsidence can occur if the features align with the channel.

Based on the recorded data a management plan for mining under the stream diversion and wetland areas must be developed. The areas should be handled similarly to rockfall hazard management.

Due to the potential for unstable roof conditions and the formation of sinkholes it is recommended that no mining be conducted where the roof of the excavation is shallower than 20m below surface

7. FINAL PROPOSED LAYOUT

After consultation and considering the geotechnical and hydrogeological risks and impacts with respect to mining shallower than 20m below the ricer and wetland the proposed mine plan was revised. The revised mineplan will only mine deeper than 20m below the river diversion. Therefore no mining of the 4 seam will occur under the river diversion and the wetland (Figure 7), therefore the risk of roof collapse between the pillars will be low. This wil result in an low risk of water inrush after mine closure and the subsequent contamination of surface and groundwater downstream.

The 2 seam is deeper than 20m and all the mining blocks as defined will be mineable (Figure8).

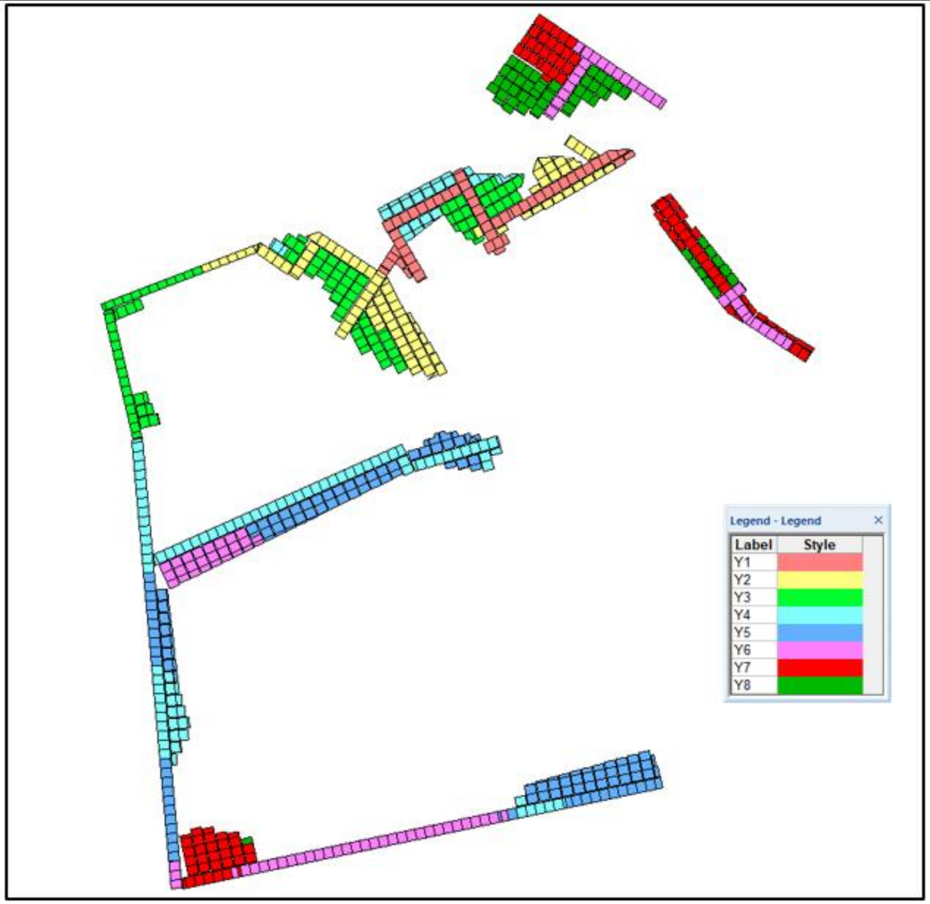
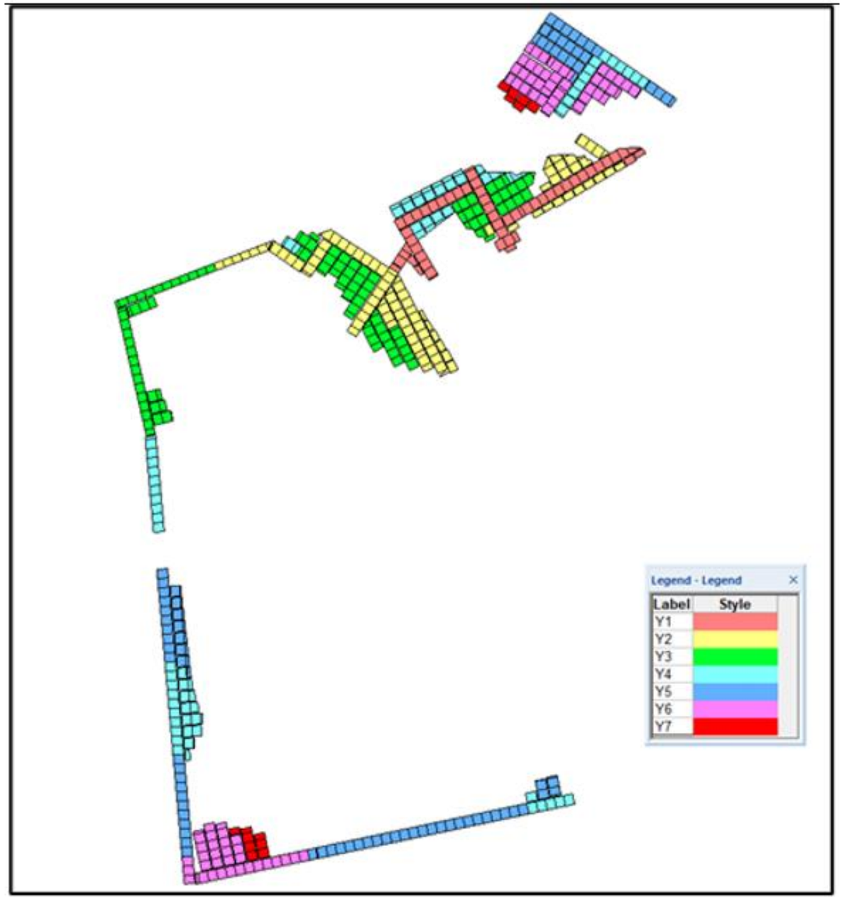
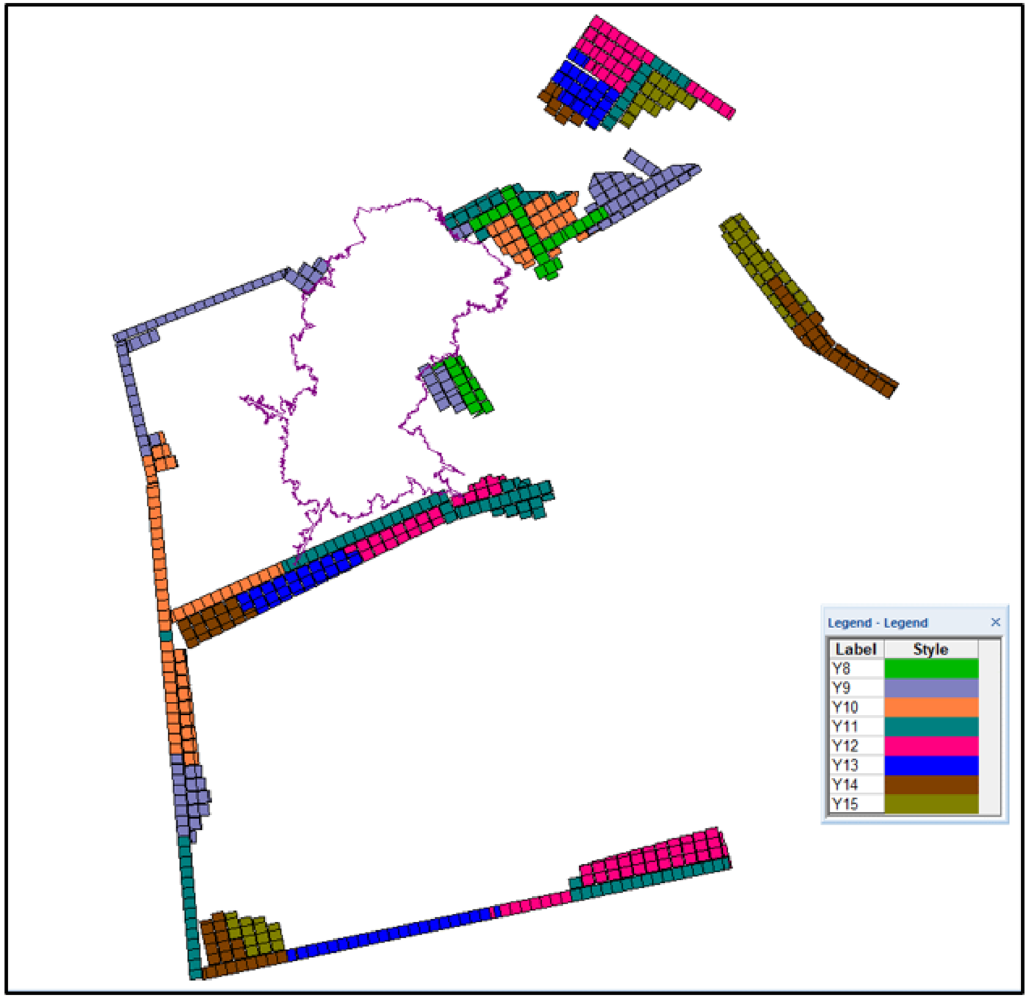
	
4 Seam original proposed layout	4 Seam Revised layout

Figure 7: Original and revised layout of 2 Seam mining blocks



2 Seam Unchanged Layout

Figure 8: 2 Seam layout remain unchanged

8. REFERENCES

8.2.2. J.N. van der Merwe, J.N. 2019. Coal pillar strength analysis based on size at the time of failure Journal of the Southern African Institute of Mining and Metallurgy *ref.On-line version* ISSN 2411-9717

8.2.3. Van der Merwe, J.N. 2016. Review of coal pillar lifespan prediction for the Witbank and Highveld coal seams. *Journal of the Southern African Institute of Mining and Metallurgy,* vol. 116, no. 11. http://dx.doi.org/10.17159/2411-9717/2016/v116n11a11