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A case study for rebar-reinforced shotcrete arches and void filling at the Grasberg Block Cave mine, Indonesia

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Abstract

The Grasberg block cave mine was developed below the Grasberg open pit at PT Freeport Indonesia as part of the transition to underground mining. The initial access drifts were developed in 2004 and production started in 2018, with a projected peak production of 160,000 t per day.

During the development stage of the Drainage Drift 4, it became clear that the geological conditions would be a key parameter in the stability and sequence of these excavations. Where the lithological contact between the Faumai limestone and heavy sulphide zone (HSZ) coincided with the mid Grasberg fault zone, very poor ground conditions and short excavation stand up times lead to tunnel collapses and severe overbreak. Furthermore, a void was created due to the mucking operations.

Void assessment using scan and drilling data was conducted to determine the void boundaries that would be concrete filled. A combination of spiling bars, rebar reinforced shotcrete arches (RRSA), and cable bolts were installed during the excavation process in these poor ground conditions. These ground support types and combinations, particularly the RRSA were designed to provide long-term stability for the excavations.

This paper discusses the development of the various ground support strategies with reference to the RRSA including ground support capacity analysis.

Keywords: *shotcrete arch, poor ground, development strategies, lithological contact, void*

1 Introduction

The Grasberg block cave (GBC) mine is an underground block panel mine that was developed below the Grasberg open pit mine as a part of transition plan from open pit to underground mining. The Grasberg open pit mine was concluded in late 2019. The initial access drift development began in 2004, while the undercutting and draw belling were initiated in 2018. The GBC mine is projected to achieve full production rates of 130,000 to 160,000 t per day in 2025 (Brannon et al. 2020).

Like other block caving mines, the GBC mine utilises a gravity flow mining system that consists of several levels. The undercut, extraction, and haulage levels are the main levels for mining production, while other levels such as service and drainage levels are also important in supporting production during the life of mine.

The drainage level is designed to drain groundwater and water from the production levels. Without this drainage level, the water inflow in the underground mine will not be well maintained, and it could produce several problems, particularly related to safety and productivity, such as flooding and operational delays (Rubio et al. 1998). Considering these flooding impacts, the stability of drainage drifts is important.

Geological condition is one of the key parameters of developing drifts in the GBC due to a wide range of complex lithological units dominated by tertiary dioritic-monzonitic intrusions of Grasberg Igneous Complex and sedimentary country rocks associated with strike-slip faulting (Campbell et al. 2018). This complex lithology condition, combined with geological structures, results in poor and very poor ground conditions in several areas including the Drainage Drift 4 (DD4).

The stability in the DD4, after a tunnel collapse and cavity formations, indicated that a distinct development and ground support strategy will be required. This analysis, including the void assessment and development sequence using rebar reinforced shotcrete arches (RRSA), will be discussed in this paper.

2 Geological condition of collapsed Drainage Drift 4

A ground collapse occurred in March 2018 after mucking activities were completed at the heading of Drainage Drift 4. Based on actual geological mapping (Figure 1), the ground collapse was related to the mid Grasberg fault, a 33–35 m wide fault zone with low rock quality designation (RQD) and intense fractured rock, and a lithology or alteration contact between dalam andesite (Tgda), endoskarn alteration (Tf) Faumai skarn and heavy sulphide zone (HSZ) (Ekaputra 2018). Figure 2 shows the initial observation where the collapsed material originated from the left back of the drift consisting of sandy and low strength rock rubble. Several pieces of rock rubble indicated a slickenside texture as evidence of the fault zone.

Similar geological conditions were also intersected in a geological hole (GBCPA-01-07) that was drilled near the collapsed area (Figure 3). This core sample data indicated an intense fracturing zone that consists of sericite alteration, slickenside texture, a low RQD (0–25), and groundwater (<2.7 m^3/h), associated with mid Grasberg fault zone and alteration contact.

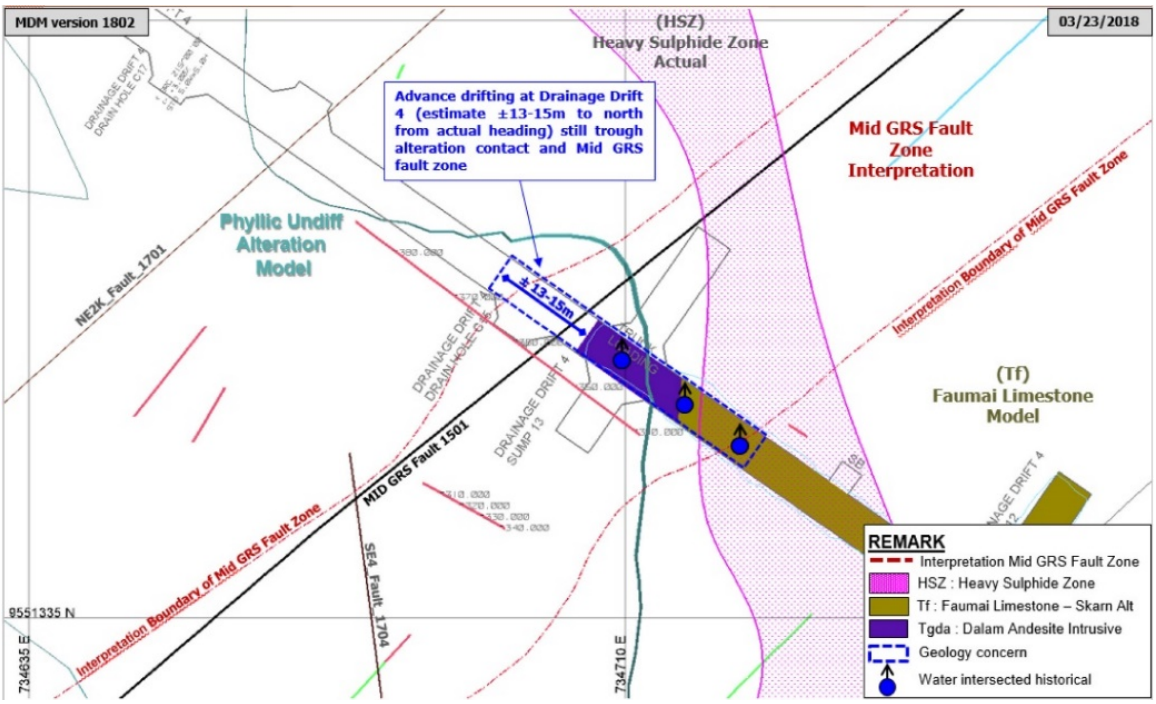


Figure 1 Geological map of GC2710L Drainage Drift 4 (Ekaputra 2018) showing the fault zone and alteration contacts

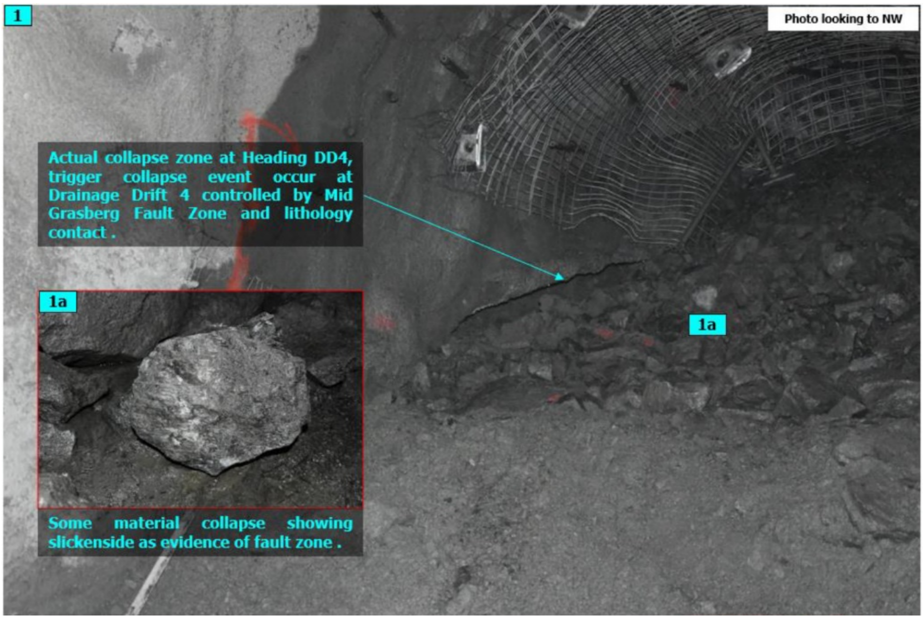


Figure 2 Ground collapse at the heading of GC2710L Drainage Drift 4 (Ekaputra 2018). The inset shows slickenside textures as evidence of the fault zone

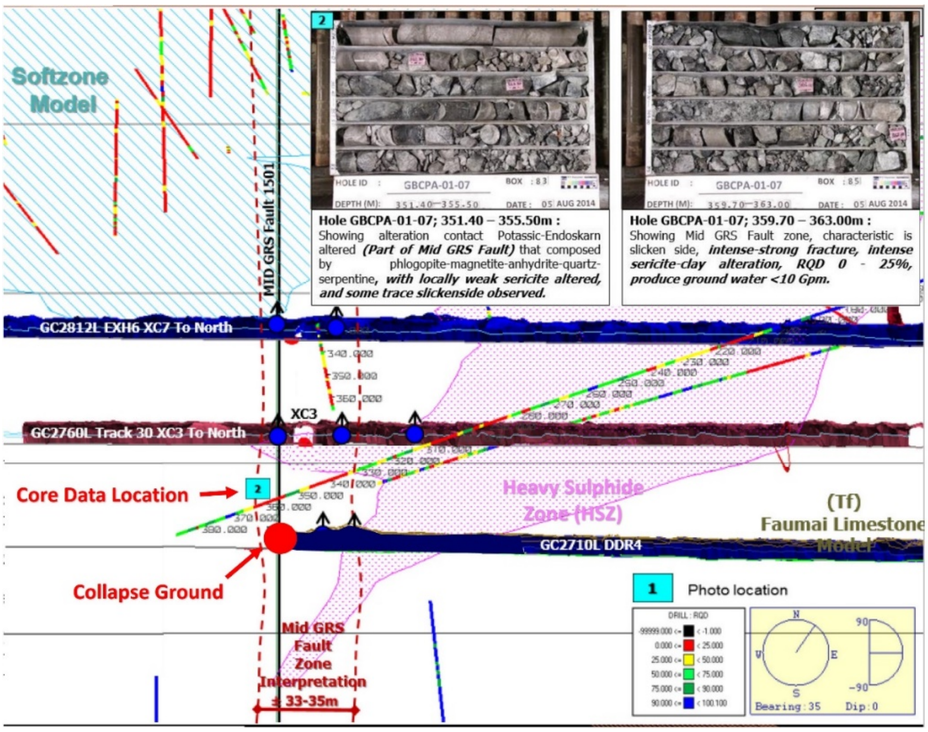


Figure 3 Core sample at hole GBCPA-01-07 near the collapsed area, showing intense fractured rock and low RQD values

3 Void analysis

When the collapsed area was mucked, materials kept coming down and a void was created in the back of DD4. The dimensions of the void were unknown and further investigation was required before detailed ground support could be designed. Various methods, ground penetrating radar (GPR) and cavity monitoring system (CMS) scans were used to determine the size and extend of the void before the decision was made to fill the void.

3.1 Ground penetrating radar survey scan

GPR is a geophysical technique based on radar technology and is primarily used to detect shallow subsurface objects such as pipes, barrels, saturated zones, water or air-filled mine openings, cave, or fault planes (Daniels & Roberts 1994). For this case, the GPR survey scan was conducted to preliminarily identify the broken zone and voids in the concern area.

Since the void was predicted to be located between haulage and drainage levels, the GPR survey was planned to be taken from both levels. From the haulage level, GPR was conducted from the floor, while from the drainage level, it was conducted from the back of the drift. A total of four survey lines (Figure 4) were taken using a 100 MHz frequency. This antenna frequency was chosen to get a wide range of concern area with maximum 20–30 m depth penetration.

Based on the scan results, areas of concern were observed at Line AB and Line DC, as shown in Figure 5. Broken ground or void, as shown by a low reflectivity and discontinuity pattern, indicated the void existence at the pillar between two levels.

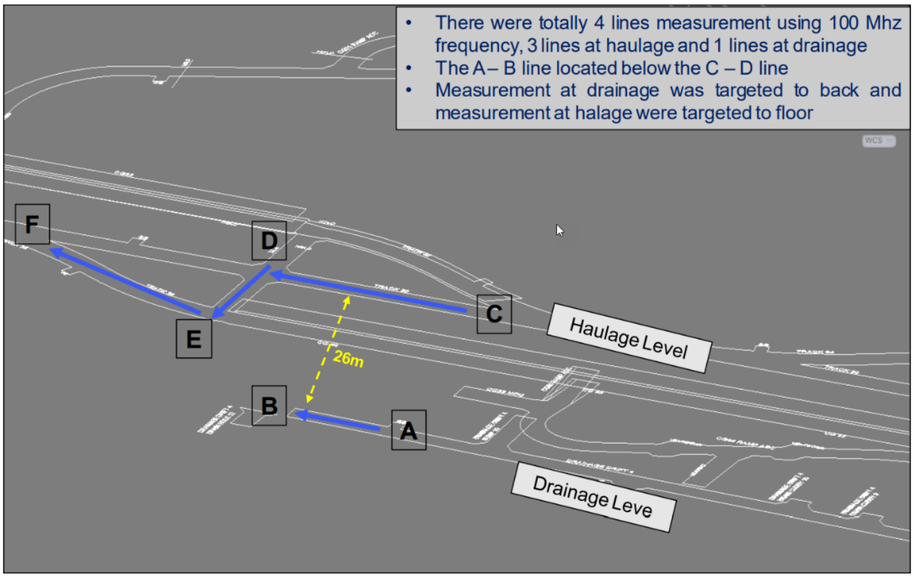
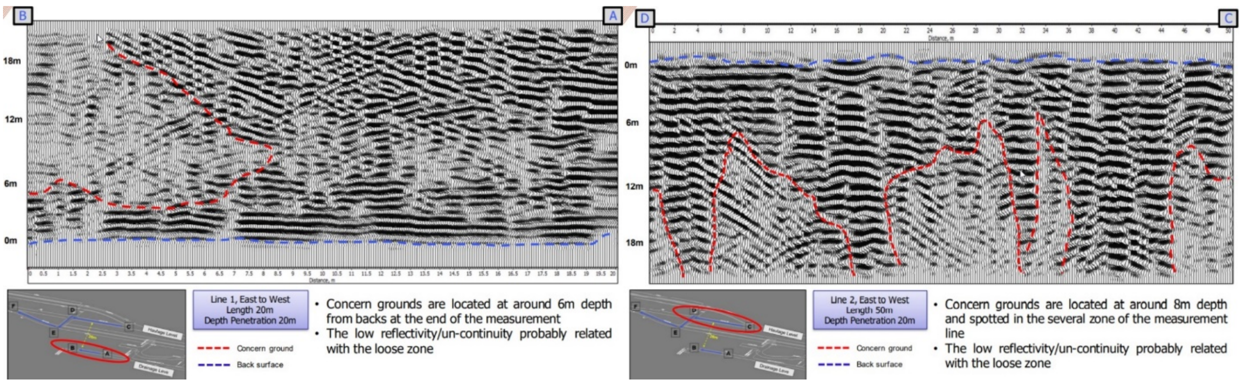


Figure 4 Ground penetrating radar survey line plan from haulage and drainage level to target void and broken ground



(a) (b)

Figure 5 Ground penetrating radar survey result on (a) Section B–A, showing broken ground at 6 m depth from back of drainage level; (b) Section D–C, showing broken ground at 8 m depth from floor of haulage level

3.2 CMS scan and probe holes drilling

Although broken ground and void has been identified by the GPR survey, the actual geometry of the void needed to be determined. A CMS scan was conducted to accurately determine the actual geometry of the void, which will be used to calculate the concrete infilling requirements.

Based on the CMS scan, the dimension of the void was 13 (L) × 10 (W) × 9.5 m (H). The extent of this void towards the lower levels could not be determined due to the collapsed material at the bottom of the projected void. Probe holes were drilled from the drainage level at 1–2 round (4–8 m) before the collapse area to determine the void extension. The void geometry was interpreted using all the data and illustrated in Figure 6.

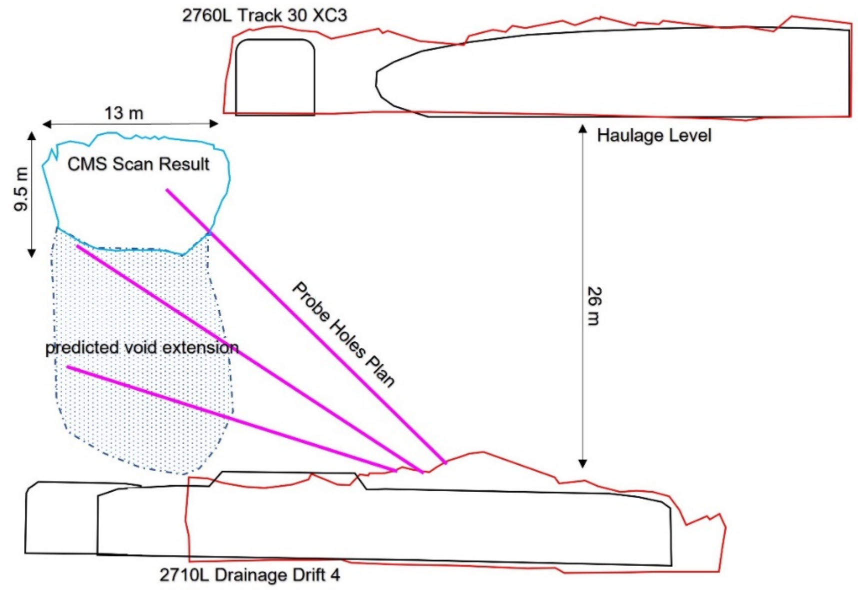


Figure 6 CMS and probe hole results indicating the dimensions of the void

3.3 Void infilling

Considering the GPR and CMS scan results combined with probe holes drilling results, a void infilling plan was designed. The void filling with concrete was executed using four holes with 30 cm diameter drilled from the top level at haulage and service level to ease the pouring process.

A total of 596 m^3 of concrete was used to fill the void. This infilling process was expected to compact and reinforce the broken ground, which would improve the sill pillar stability and be used as an anchoring system for ground support installation beneath the void. On the other hand, the concrete infilling would also give an additional load to the DD4 excavation, which was considered carefully during the ground support design process.

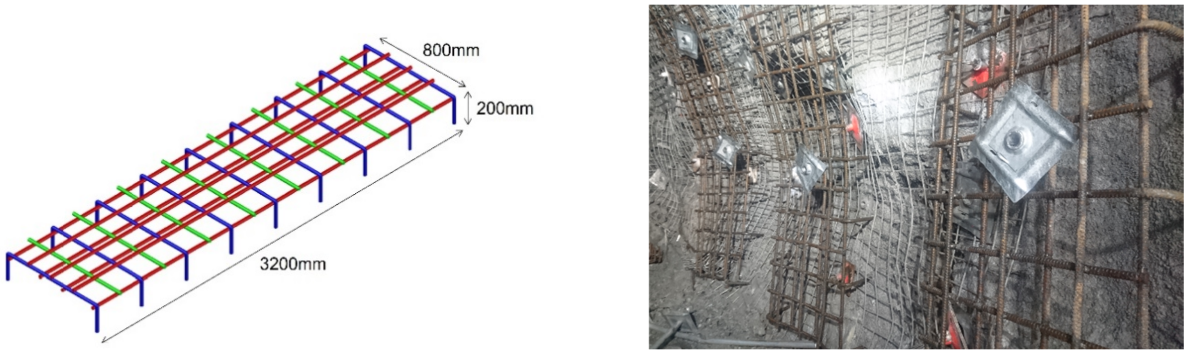
4 Excavation sequence

Prior to the excavation process, the installation of ground support was carried out in several stages, including the installation of spiling bars and RRSA. Spiling bars were used for temporary support during the excavation process, while a combination of cable bolts and RRSA were used as permanent support, to be effective for the planned life of the excavation.

RRSA is commonly used as an option in several conditions where the regular support is insufficient, such as poor to very poor rock conditions, fault zones, or larger tunnel width. The RRSA design may vary, depending on the load condition and tunnel profile.

At PT Freeport Indonesia (PTFI), RRSA are made from 19 mm rebar and welded to each other to form a frame with 3,200 mm length, 800 mm width, and 200 mm depth dimension (Figure 7). This rebar frame is then pinned to the rock using 2.4 m friction bolts (Split Sets®). The arches could be cut shorter, particularly at the shoulder, to ensure that the rebar constructions form an arched shape. Fibre-reinforced shotcrete (FRS) is then applied to cover the rebar arches. Spacing on installation between the arches can be varied depending on the rock and load condition. In poor ground condition, it can range from 1.0 m up to 2.0 m spacing, while in very poor ground condition, double arch system, in which two arches are installed side-by-side, could be applied.

Since the RRSA are installed following the surface contour of the drift, for a single arch installation, it required a set of approximately four frames to be installed in DD4. Total of three RRSA were installed on a 1.0 m spacing to maintain the stability in the sections of the collapsed drift.



(a) (b)

Figure 7 RRSA in Freeport Indonesia. (a) Rebar frame design; (b) Rebar frame pinned to the rock using 2.4 m friction bolt before FRS application

5 Ground support performance analysis

Considering the additional load from the filled concrete void, RRSA are proposed to be installed following the primary ground support consisting of spiling bars, weld mesh, shotcrete, and cable bolts. Detailed analysis was required to determine if the modelled results indicate that the installed ground supports are adequate.

The analysis was conducted with RS2 software to determine the expected displacement around the excavation considering the stress change because of the caving process. An additional analysis was done, using StaadPro software to determine the expected internal force in the RRSA. The load capacity of the shotcrete arches during certain cave mining stages was calculated using empirical equations.

5.1 Numerical modelling

The numerical model (Figure 8) demonstrates the cross-section of the actual drift profile and concrete-filled voids. The material properties used in the RS2 software model are listed in Table 1. Specific properties for actual rock mass are taken from lab testing results, while concrete material properties are derived from equation referred to American Concrete Institute (2019) and Karam & Tabbara (2009).

To determine the changes of displacement during the peak loading period, the analysis was conducted in three stages with different stress values in each stage, as shown in Figure 9. The first stage was the drift condition before the arches were installed, the second stage shows the drift condition after the arches were installed, and the final stage indicates model at peak load condition.

The analysis indicated that the displacement increase is related to the stress increase associated with the advancing cave. Figure 10 shows that during current conditions in June 2023, the expected displacement shown by the model is about 56 mm on the left rib and 54 mm on the right rib. This indicates an increase of 14–20 mm displacement occurred on the drift since the RRSA were installed. At peak loading conditions, which is expected during December 2030, the displacement is expected to reach 60 mm on the left rib and 61 mm on the right rib, indicating an increase of about 19–26 mm since RRSA were installed.

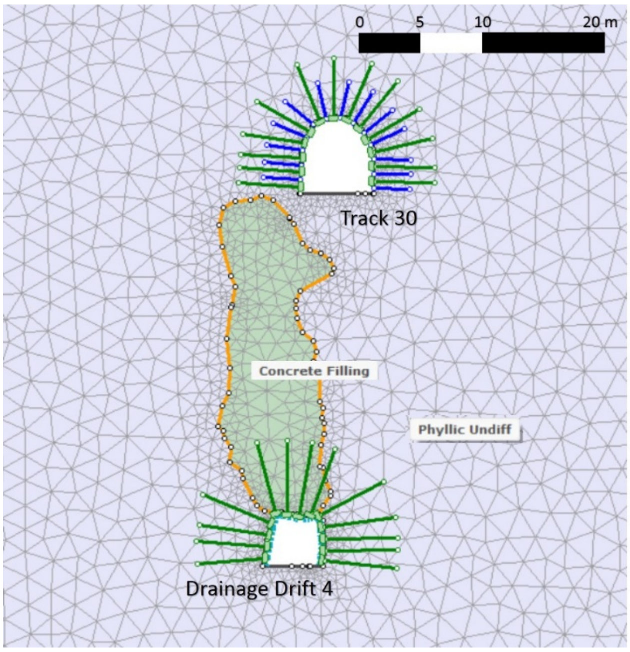


Figure 8 Numerical modelling section showing the position of filled void related to the drainage and haulage level drifts

Table 1 Material properties of rock mass and concrete derived from lab testing and empirical equations

Material	UCS (MPa)	E_{rm} (MPa)	Generalised Hoek–Brown criterion		
			m_b	s	a
Phyllic undiff. alteration (very poor ground)	100	15,208	2.24	0.002	0.5
Concrete filling	47	32,443*	12**	1**	0

*Derived from empirical equations based on American Concrete Institute (2019). **Refer to Karam & Tabbara (2009)

Stress Summary

Model Stage	Frame Number	Date	Sigma 1 (MPa) In-Plane	Sigma 3 (MPa) In-Plane	Theta (°)	Loading Condition
0	061	Dec-15	27	24	10	Pre-peak Loading
1	071	Jun-18	28	23	11	Pre-peak Loading
2	081	Dec-20	34	15	17	Pre-peak Loading
3	091	Jun-23	36	11	17	Pre-peak Loading
4	101	Dec-25	38	11	12	Pre-peak Loading
5	111	Jun-28	38	11	12	Pre-peak Loading
6	121	Dec-30	39	11	10	Peak Loading
7	131	Jun-33	39	12	10	Stress Shadow
8	141	Dec-35	33	12	3	Stress Shadow
9	151	Jun-38	31	12	3	Stress Shadow
10	161	Dec-40	30	11	3	Stress Shadow
11	171	Jun-43	30	11	3	Stress Shadow
12	181	Dec-45	32	11	0	Stress Shadow

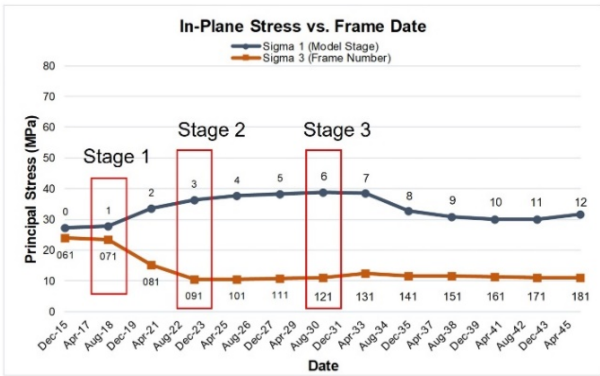


Figure 9 Stress path summary of Drainage Drift 4 collapse area until end of life of mine. Stage 1 of model use stress frame on June 2018, stage 2 on June 2023, and stage 3 on December 2030

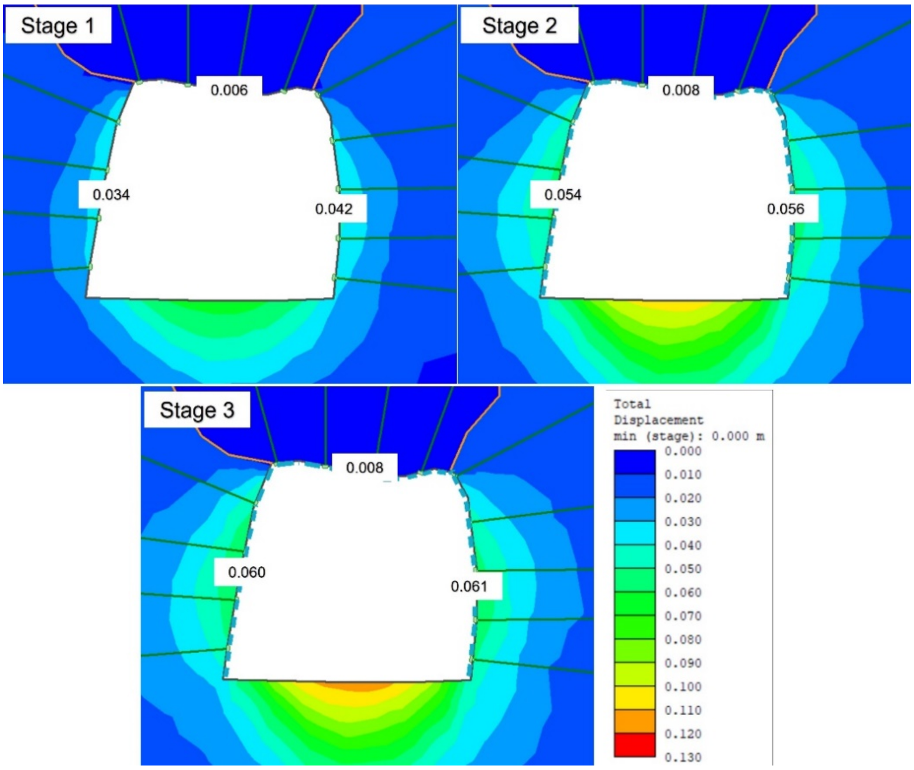


Figure 10 Expected total displacement around the excavations during each stage. Stage 1 is before RRSA were installed in June 2018; Stage 2 is the current condition in June 2023 after RRSA were installed; and Stage 3 is during peak loading around December 2030

5.2 Rebar reinforced shotcrete arches performance

RRSA are an example of passive ground support since they don't directly reinforce the rock mass and only respond to the inward movement from the rock mass. Furthermore, a good understanding of how the rebar arches will react to the movement is an important consideration during the designing process. As the arches are subjected to loading or movement from surrounding rock, some forces are distributed from the rock to the arches. Those internal forces are critical in calculating the support capacity, either related to choosing appropriate size of beam or determining the possibility of additional reinforcement, hence the structure failure can be prevented.

In the analysis for the Drainage Drift 4 study, the RRSA model represents the actual installation shape and using 32 MPa concrete material. These beams were modelled to be subjected to different displacement before and after RRSA installation derived from RS2 numerical analysis. Based on this analysis, the maximum axial force is 156 kNm located on beam 2 and the maximum bending moment is 303 kNm also located on beam 2 (Figure 11).

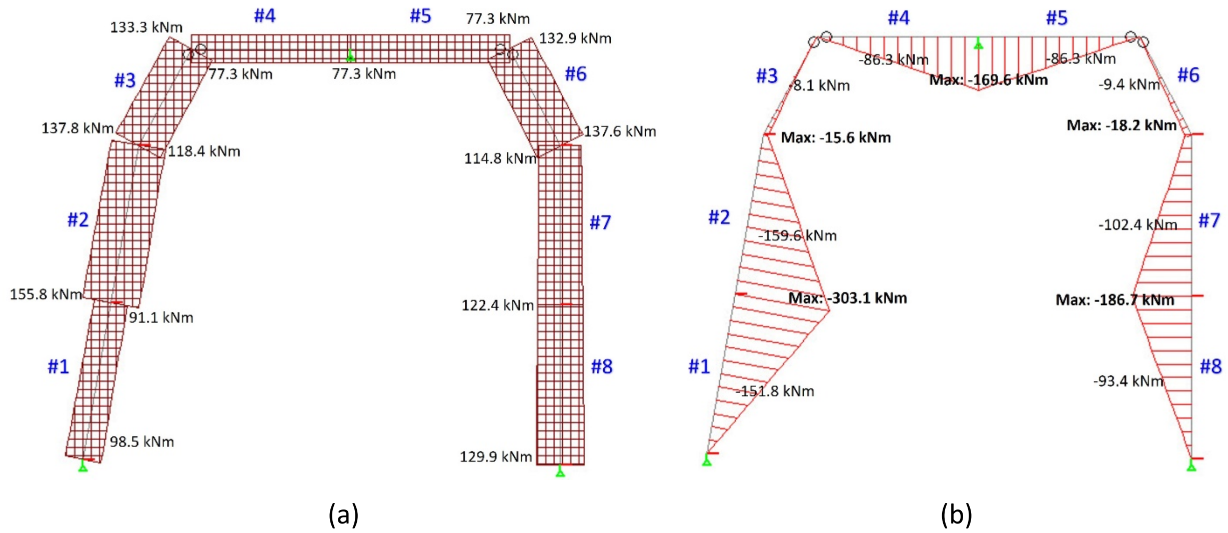


Figure 11 Internal forces from each beam. (a) Axial force; (b) Bending moment

A single rebar frame of RRSA is considered as tied symmetrical column in this analysis. Detailed analysis to determine axial load (P) and moment (M) of an axially loaded column were referred to in the book written by Wight & McGregor (2012), hence the capacity of axial load and moment in each rebar frame can be determined. The correlation between these internal forces can be illustrated as interaction diagram to show the ultimate and effective capacity of rebar frame. Ultimate capacity is the optimum axial load and moment capacity that can be received by the rebar frame, while the effective capacity is the reduced ultimate capacity by reduction factor.

Actual internal forces, axial load and bending moment, which have been given as the result of modelling, were plotted into the interaction diagram. The plotting result can be seen in Figure 12, showing that the current internal forces in rebar frame are still below the ultimate and effective capacity threshold. It is showing that the RRSA are adequate in maintaining the DD4 collapse area.

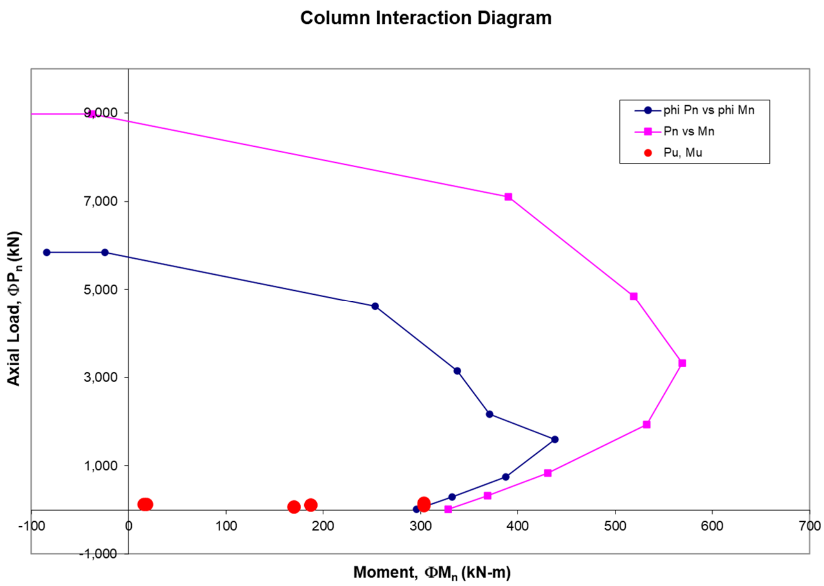


Figure 12 Interaction diagram between axial load and moment of rebar reinforced shotcrete arches. Magenta line shows the ultimate capacity, blue line shows the effective capacity, and the red dots are the actual internal forces occurred in the arches

A sequence of laser scans was conducted to determine if any movement has taken place since the start of the rehabilitation process of DD4. Laser scans were conducted after installation of temporary support, consisting of spiling bars, and then again after installation of the permanent support, consisting of cables and RRSA. Laser monitoring scans and visual observation were taken periodically to monitor the wall and roof displacement in the rehabilitated area. The initial laser scan data was collected in July 2019 and the most recent scan was done in May 2023. The scan result (Figure 13) indicates approximately 24 mm cumulative surface movement on the eastern wall and western wall. This scan result compares well to the RS2 modelling results. The RS2 model in indicates a movement of 20 mm on the eastern wall and 14 mm on the western wall over similar stages. Minimal movements were observed and recorded on the RRSA.

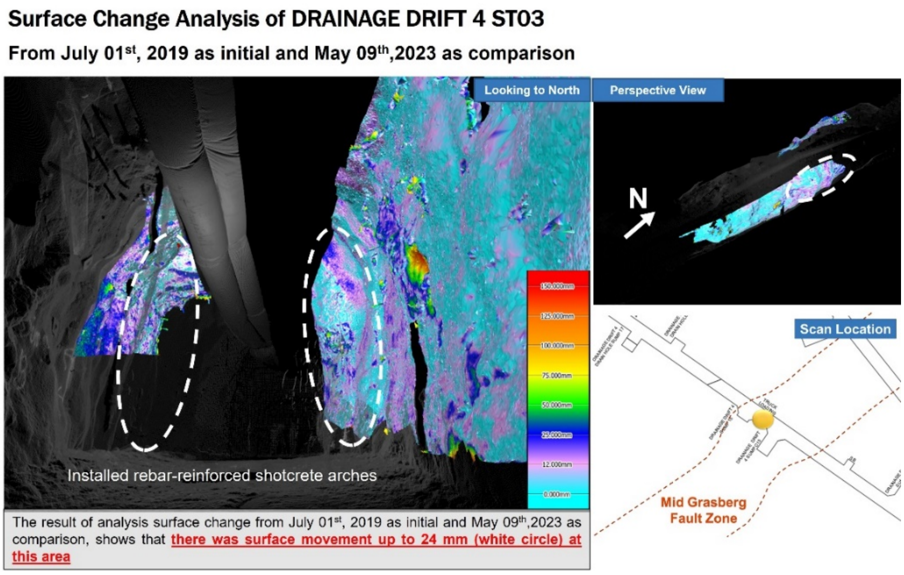


Figure 13 Laser scan result of the Drainage Drift 4. It shows surface displacement since RRSA installation. The white circle indicates the location of the RRSA

6 Conclusion

Despite the in situ stresses and the geometry of the excavation, the geological conditions were the main factors that contributed to drift collapse during the development of Drainage Drift 4. Investigations were done to determine the main cause of the collapse, the extent of the weak geological zone, and potential of void creation to develop appropriate mitigation actions, including the ground support design and future development strategies.

The main aim was to eliminate any stability risks associated with the void. The boundaries of the void were determined through GPR, CMS and probe hole drilling and thereafter filled with concrete. The additional load of the concrete to Drainage Drift 4 was considered during the design of the regular support and rebar reinforced shotcrete arches support.

Rebar reinforced shotcrete arches showed to be a practical option to be considered as a support method for very poor ground conditions and concrete-filled voids.

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