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Preventing pit wall failure through early detection monitoring, geotechnical analysis and execution of a toe-stabilisation buttress

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Abstract

Multi-batter scale instabilities in open pit mining create significant fall of ground hazards with potential for large runout distances of failed material. When such instabilities are identified, one possible geotechnical management approach is to use live radar monitoring of the slope to allow work to continue in the area until progressive deformation exceeds predetermined safe thresholds, at which point workers are withdrawn from the area until 'controlled' failure occurs. However, when the predicted failure and runout zones include a haul ramp or other critical infrastructure, failure prevention is often more economically favourable than accepting a 'controlled' failure.

A common remediation approach for rotational slope instabilities is to add a resisting force to the toe of the slope, usually in the form of a buttress. The construction and scale of the buttress is determined by geotechnical modelling.

This paper presents a case study of a toe-stabilisation buttress used to support an identified slope instability comprised of variable strength detrital materials at a Western Australia Iron Ore open pit mine. This case study covers the early detection of slope movement, subsequent geotechnical finite element analysis of the slope, buttress design and the execution strategy used to ensure a psychologically safe environment for operational personnel.

The integrated solution is discussed, including the four-dimensional slope environment during progressive, transitional and regressive movement stages, as well as considerations for surface water management. Satellite InSAR, ground-based prism systems and ground-based radar systems were used, with real-time deformation alarming, 24/7 monitoring and a trigger action response plan in place.

The geotechnical remediation has allowed continued productive movement in the pit. Batters underneath the buttress are currently being blasted and excavated under an approved geotechnical design, without reactivation of slope movement above alarm thresholds.

Keywords*: slope monitoring, radar monitoring, operational safety, buttress*

1 Introduction

1.1 Background

The BHP Western Australia Iron Ore (WAIO) asset currently includes five open pit mining operational areas in the Pilbara region, with pit depths typically in the range of 100 to 200 m, and deeper in some cases. The pits are comprised of single and double bench configurations with batter heights of 12 and 24 m, respectively.

This case study presents the early detection of initial post-blast movement in a pit wall, the geotechnical back-analysis and remediation design, as well as the execution and monitoring strategies employed to arrest the wall movement. To assess the risk of failure and evaluate proposed remediation measures, a novel and holistic approach to the observed slope behaviour was employed to build a useful geotechnical model (de Bruyn 2021). Engagement and communication with mining teams was critical in implementing the remediation strategy, and a trigger action response plan (TARP) including a movement threshold warning system was used.

1.2 Regional geology

The pit wall in this case study is located in the Marra Mamba Iron Formation of the Proterozoic Hamersley Group (Klein & Gole 1981) in the Pilbara region. It is comprised of a thick detrital sequence consisting of red ochreous material, and a number of tertiary channel deposits (Porter GeoConsultancy n.d.). The predominant lithological units include siltstone (ST3), gravelly siltstone (GS3) and vuggy breccia (VB3) (Figure 1). The detrital sequence is typically associated with rock mass style failure mechanisms due to the soil-controlled nature of the material, learnt through observation, particle size distribution and lack of discrete structural features. The base of the slope shows the exposed West Angelas Member contact units (WA2, WA1) which overlay the Mount Newman member (N).

Figure 1 Case study pit slope, prior to deformation and signs of instability

2 Geotechnical monitoring system set-up

2.1 Monitoring architecture

The WAIO geotechnical team has implemented a suite of monitoring equipment to enable an understanding of the four-dimensional slope stability environment at the operation. The main monitoring types in use are satellite interferometric synthetic aperture radar (InSAR), ground-based prisms and ground-based radar. Each monitoring type has a different range and measurement frequency which allows validation of movement trends across different time and spatial domains (Figure 2). A brief description of each monitoring type in use is provided in Table 1. The set-up of the monitoring units is provided in Figure 3. It is important to note that prisms were only installed on a single berm level; the bearing to absolute values and calibrating to relative values from the radar data will be discussed in Section 3. These units remained deployed throughout the monitoring phase, buttress execution phase and continued excavation below the buttress.

Data from one vibrating wire piezometer was also reviewed. Since the water level measurements were well below the current pit floor (>30 m separation between water level and base of pit), the data available did not suggest pore pressure was an issue in the vicinity of the instability. Considerations for surface water management are discussed in Section 6.

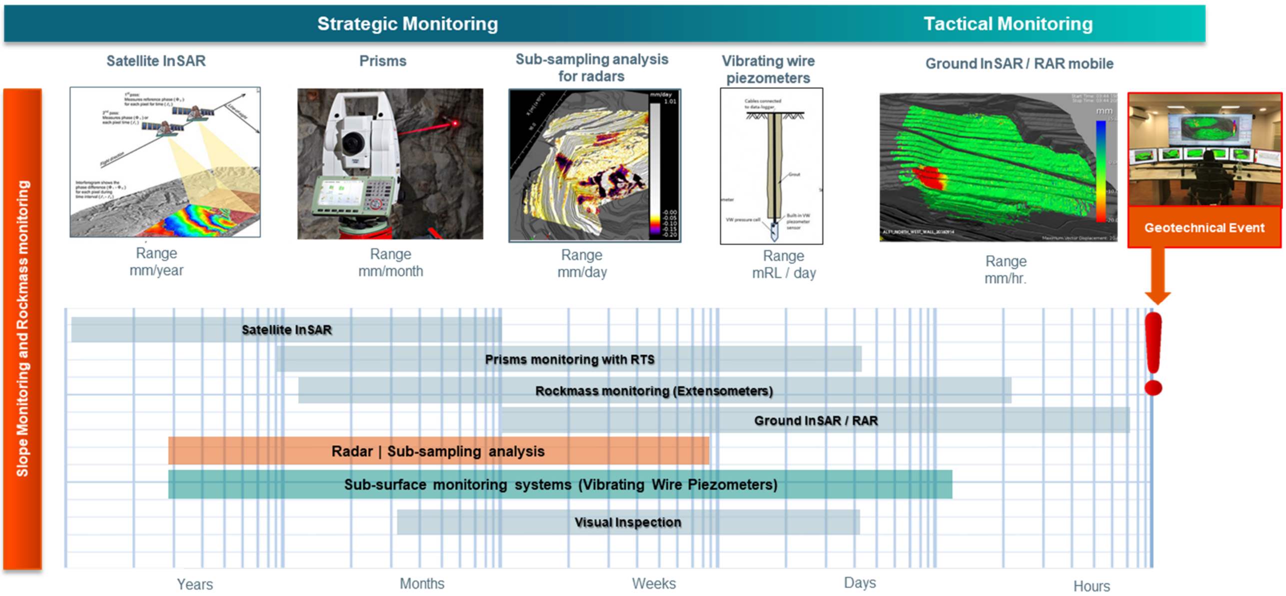


Figure 2 Four-dimensional monitoring strategy

Table 1 Summary of monitoring unit capabilities

Monitoring unit (vendor)	Brief description and application
Satellite (Tre Altamira)	InSAR using Tre Altamira remote sensing satellites (Tre Altamira 2023) to measure amplitude and phase components of the backscattered radar signal onto the Earth's surface (Geoscience Australian 2023). Changes in these readings can be converted into displacement rates. Point data is widespread, with points of interest able to be analysed individually or as a group. TREMaps software features such as area time series plots are used to validate trendlines.
Ground-based prisms (Leica Geosystems)	Leica Geosystems ground-based prisms are used to measure the distance between two points through glass reflectors that return an electromagnetic signal to a total station (static monument) (Leica Geosystems 2023). Prisms are sited at approximate 50 m intervals on various berm levels of the pit wall.
Ground-based radar (Hexagon IDS)	IDS IBIS-Arc-SAR Performance (synthetic aperture radar) (IDS Georadar 2023) are used to provide high-precision real-time movement data through interferometric calculation of the radar 'echo' collected from each ground pixel.

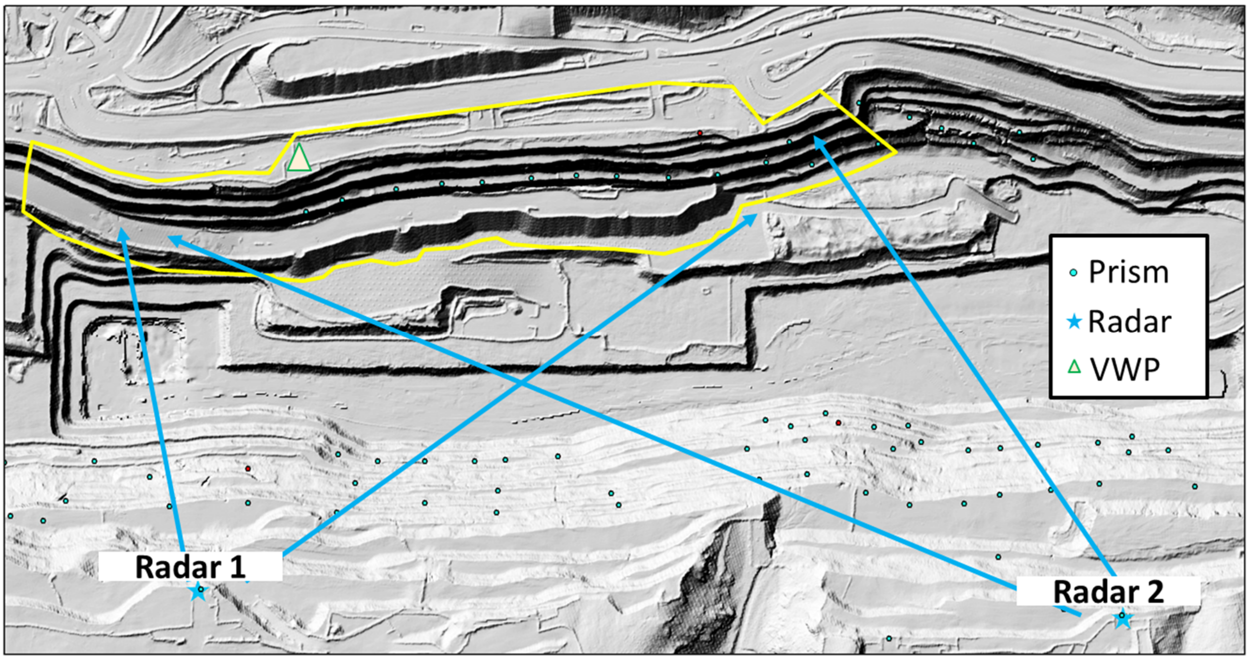


Figure 3 Slope monitoring units and setup (note: InSAR not labelled)

2.2 Slope radar TARP

The IDS IBIS-ArcSAR slope radars have recently been deployed as part of an integrated WAIO-wide network approach. The slope radars are supported by the monitoring service provided by the vendor, which includes 24/7 monitoring coverage with a direct connection to the WAIO Integrated Remote Operating Centre (Dixon et al. 2022). This network agreement allows the site geotechnical engineers to focus on specific ground control duties and minimise the time taken to resolve technology- and database-related faults in the monitoring system.

The actions and responsibilities of individual personnel and teams are defined by a slope radar TARP. The TARP allows for early warning systems to trigger communication alarms to the operational crews if the slope deformation exceeds the movement tolerances set by the site geotechnical engineers.

While the WAIO team employs a number of monitoring units, the slope radar data is used as the basis for the set-up of slope trigger levels due to the fast scanning time (generally less than two minutes). Additionally, blast-induced displacement step changes in the radar data are more discretely visible due to the increased frequency of the scans compared to the prism data (generally every four hours). A number of different alarm settings are typically used in the IDS Guardian monitoring software, including average velocity and average time series alarms. The alarm thresholds are determined based on line of sight (LoS) effects on apparent movement rates detected by the radar, the specific pit geometry and stratigraphy, degree of vegetation and WAIO historical pit failure data.

3 History of slope movement

3.1 Early movement detection

The slope showed an initial linear deformation trend detected immediately after a significant in-pit blast (Blast 1). The blast took the effective wall height from approximately 72–84 m and the closest theoretical geophone recorded a peak particle velocity (PPV) of 173.8 mm/s against the pit wall. It should be noted that the blast directly above this batter recorded a theoretical PPV of 442.4 mm/s. While wall damage cannot be solely explained by PPV measurements (Aldas 2010), the compounding effects of the vibrations should be considered. An initial relative displacement of approximately 10 mm was recorded in radar data on the lowermost exposed batter within the first 24-hour period after Blast 1. The movement then slowed significantly before displaying a regressive trend with approximately 20 mm of relative displacement recorded over a three-week period, equating to a displacement rate of approximately 1 mm/day. It should be mentioned that these measurements represent an average value from an area on the batter face central to the overall identified movement area, at a depth of between 48 and 64 m. All subsequent relative displacements discussed in Section 3 pertain to this area.

Additional alarms with lower thresholds were added to the TARP radar alarm configuration following the initial movement. The sustained regressive movement trend dropped to approximately 0.7 mm/day and mining of the blasted material adjacent to the movement area began. While weak materials exposed in the wall resulted in berm loss, movement continued to be regressive, maintaining a steady movement rate for approximately 12 days. The berm loss area and respective PPV values for the batter blasts is provided in Figure 4.

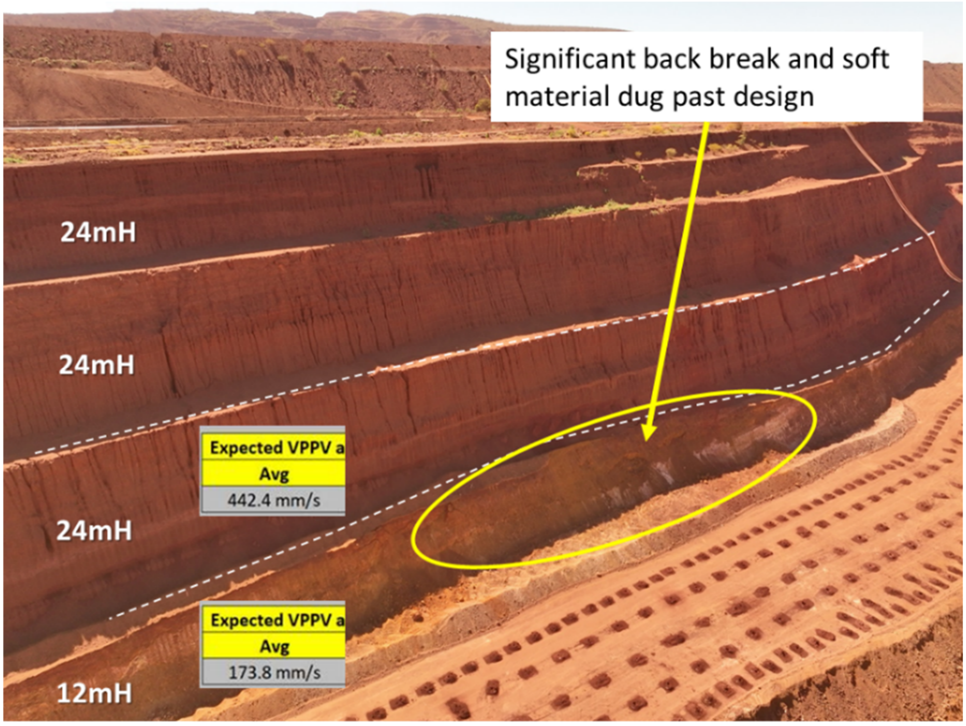


Figure 4 Theoretical peak particle velocity (PPV) and batter condition of the pit wall during identification of instability

Once perpendicular mining cuts progressed past the centre of the movement area, a 15 mm relative displacement was recorded over a 24-hour period. This subsequently led to the cessation of mining activities. Similar to movement encountered after Blast 1, the displacement trend was short-lived. The mining crews re-entered the work area once movement had showed adequate deceleration and was within TARP movement thresholds.

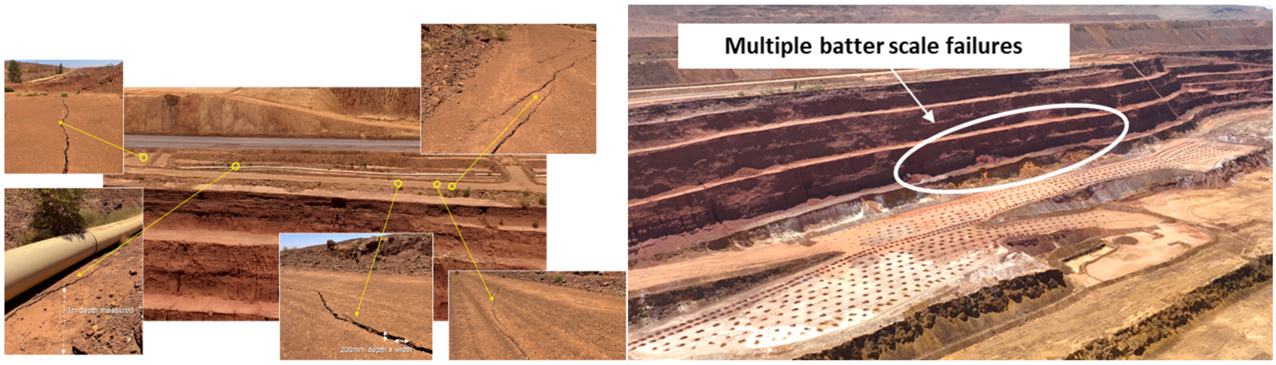
Movement of the wall showed an increased but steady relative displacement rate following completion of mining the batter. A displacement rate of approximately 3 mm/day was sustained for approximately three weeks before becoming slightly regressive and reducing to 2.5 mm/day for a further two weeks. During this time, geotechnical engineers completed regular inspections and reviewed the slope radar data daily.

3.2 Movement reactivation

Approximately three weeks following the completion of mining the batter, production blasting was conducted in the pit (Blast 2). The blast was more than 300 m away from the movement area, with no immediate change in radar or prism movement trends observed. Seven days after the blast, a sudden acceleration in displacement triggered the TARP radar alarms, with approximately 10 mm of relative displacement recorded in a 12-hour period.

The next day extensive yet discontinuous cracking was observed the behind the pit crest, at a distance of up to 50 m behind the crest edge. While small redesign efforts and blast management strategies were previously executed to try and minimise further movement, the sudden movement and significant visual deterioration of the slope and crest triggered closure to the crack-affected crest and the entire pit.

Crest cracking was observed to have a backscarp up to 50 m behind the crest edge, with a dilation of approximately 300 mm in some of the worst-case areas as shown in Figure 5a. A rock drop test was used in several locations to approximate the depth of cracking as 1 m to 5 m in various areas. The cracking showed a circular backscarp tension profile. Some intersecting cracking pathways were evident and inferred as a potential fractured network from the kinematic buckling (Diederichs 2007) that was occurring through the stepped excavation phases. This stepped movement is also reflected in the radar and prism data, which will be discussed below.



(a) (b)

Figure 5 Signs of instability in the pit wall. (a) Crest cracking locations; (b) Multiple batter scale instabilities in the lower part of the slope

A detailed geotechnical review of the area commenced. At this stage it had been 65 days since initial movement post-Blast 1, and approximately 30 days since the completion of mining in front of the moving area. A total relative displacement of approximately 135 mm recorded in the central movement location of the slope radar data (Figure 6). Figure 7 shows the area polygons used to create the slope radar time series data plots. Prism data was also reviewed weekly and demonstrated confidence on the progressive trend, as seen in Figure 8.

The inflections in the displacement graphs demonstrate the velocity changes in the slope movement initiated by the blasting. At each inflection point, however, the velocity recalibrated to a new linear trend, making it difficult to reliably predict a possible time of failure based on the data due to a lack of steady acceleration. Despite the lack of a steady trend in acceleration, the visually observed step changes in the slope displacements, with an increasing steady state velocity associated with each step, demonstrated that slope movement was progressive and moving towards failure. Once the pit had been closed, a detailed slope model assessment commenced.

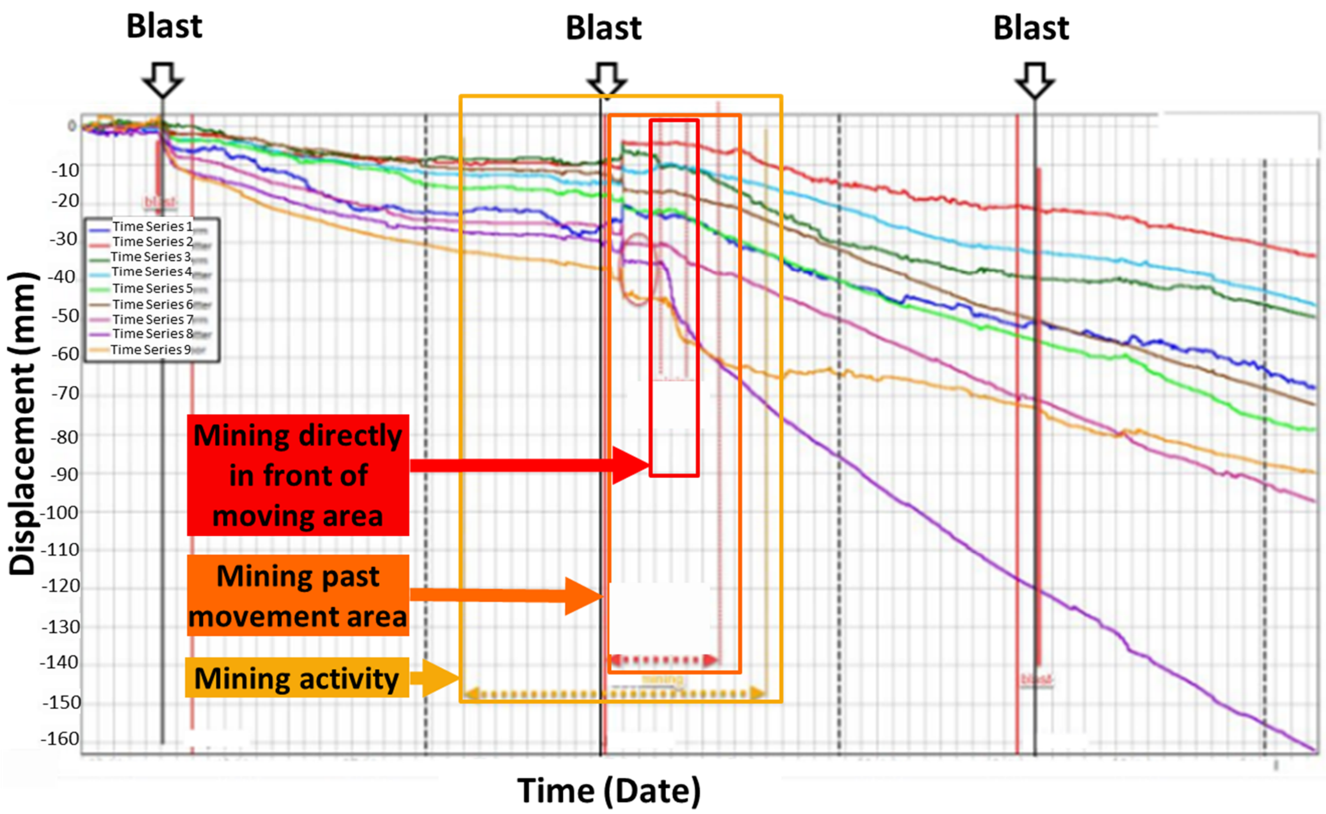


Figure 6 Detailed displacement graphs of ground-based radar area time series plots showing deformation on y-axis over time on x-axis, showing times of significant blasts (Blast 1, Blast 2 and Blast 3)

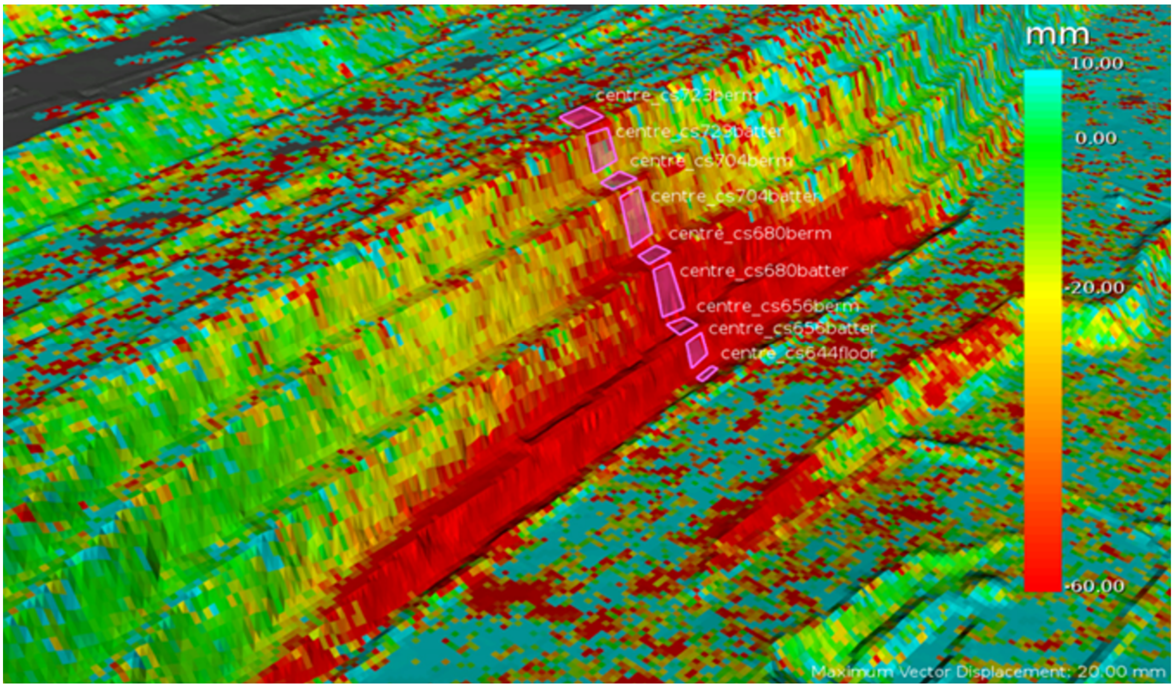


Figure 7 General arrangement of displacement time series areas plotted in Figure 6

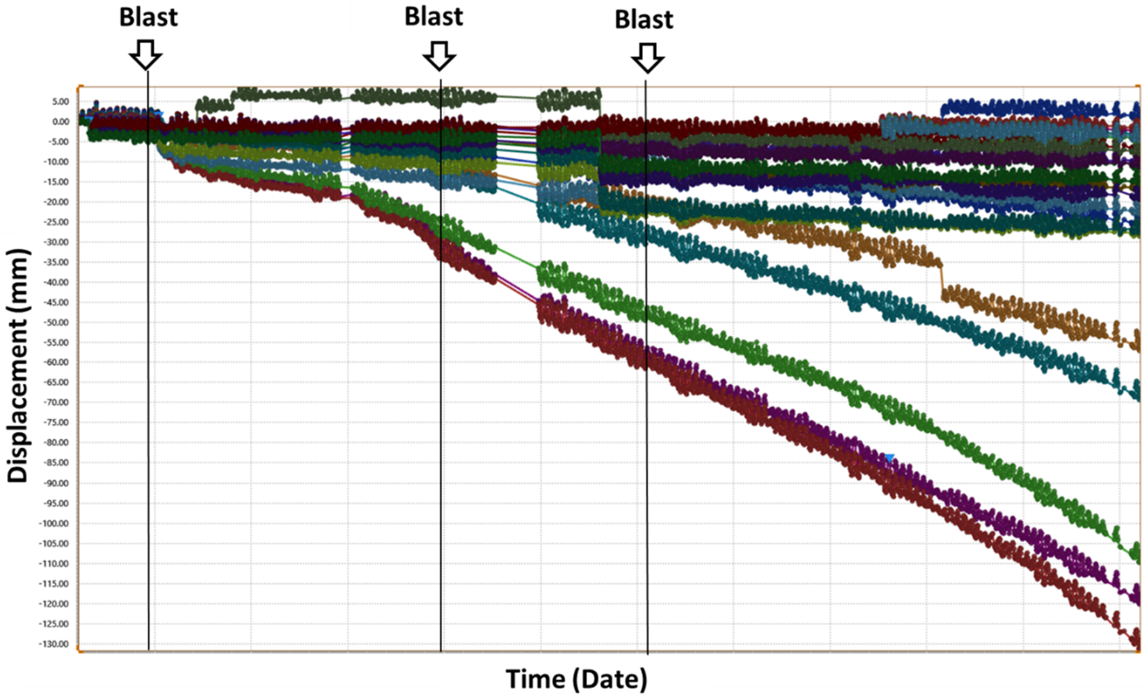


Figure 8 Detailed displacement graphs of ground-based prism plots showing deformation on y-axis over time on x-axis, showing times of significant blasts (Blast 1, Blast 2 and Blast 3)

3.3 Monitoring summary

The increased movement trend was confirmed through verification of the multiple available monitoring methods. The prism, radar and InSAR data are aggregated in Figure 9 to demonstrate the high confidence movement trends evident on all three systems and the onset of each displacement rate change correlating to the blast events. A selection of most representative InSAR points were used in this graph. It should be noted that the InSAR monitoring has been in place for a number of years and explains the initial cumulative displacement. A summary of movement rates at approximate initial detection and peak levels is provided below (Table 2). The difference in presented values can be attributed to calculation methods between the different units where LoS impacts, and will be discussed below.

Table 2 Key movement rates across the different monitoring units

Monitoring unit	Initial detection rate	Peak detected rate
Prisms	0.902 mm/day	1.349 mm/day
Radar	0.06876 mm/hour	0.181 mm/hour
Satellite	7.5 mm/month	12 mm/month

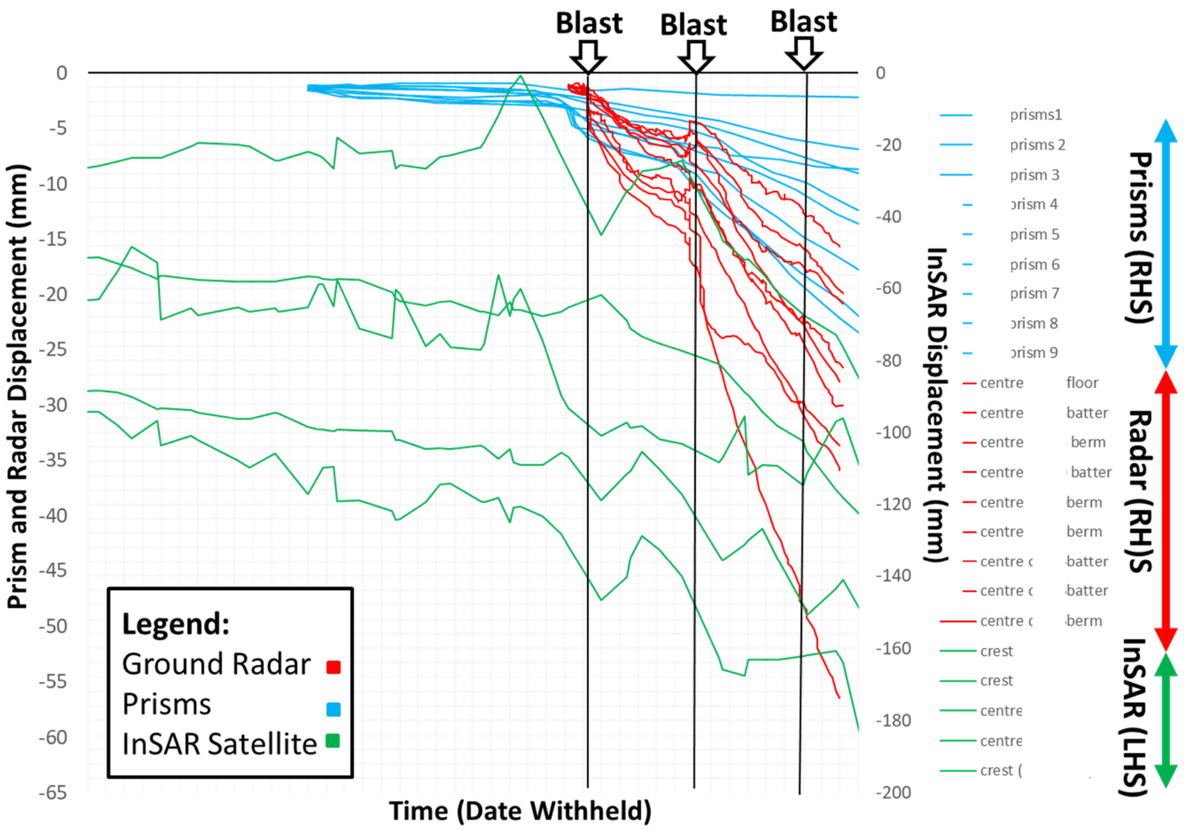


Figure 9 Aggregated displacement graphs showing movement trends across ground-based, ground-based prism and InSAR satellite units, showing times of significant blasts

3.4 Line of sight considerations

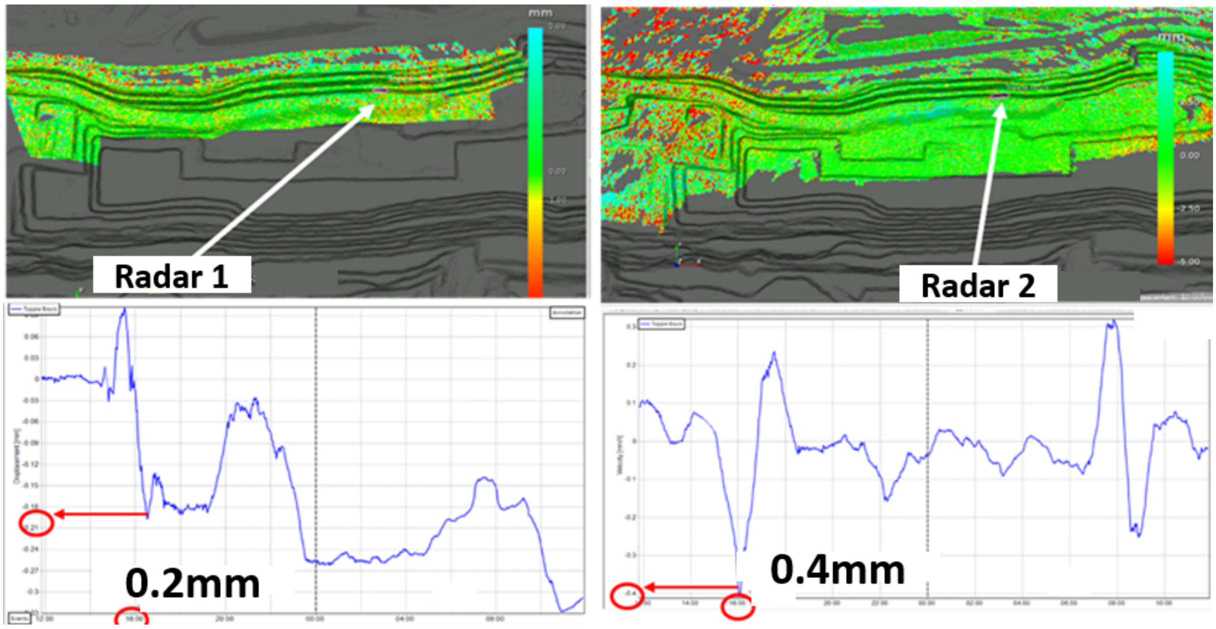
It is important to highlight the relativity to provide an understanding of the difference between the movement values. The radar provides a single vector value for any given point on the surface map as it uses LoS. This also partially explains why each area time series value in Figure 6 is different between the berm levels as the vertical component of the total vector is different at each berm level. If the magnitude and direction of wall movement is known (from prism or other absolute measurement tool), the relative displacement that would be recorded from a given location can be easily calculated. However, without a known (or assumed) three-dimensional vector of movement, the relative displacement cannot be calculated. This is an important consideration for planning radar and prism locations, setting alarm thresholds, comparing relative movements between areas and evaluating the size of monitoring areas. The importance of having different monitoring units that use a range of calculating techniques became well understood through this case study.

During the selection of the radar alarm thresholds, it was important to consider the LoS impacts across the berm levels. As the interferometric radar uses a single LoS displacement (Harries & Jacobsen 2022) it is critical to understand the angle of incidence in both the X-Y and Z axis from the radar to the pit wall relates to the movement vector, and the resultant relative displacement as discussed. The LoS can also impact sensitivity to alarming, and cause activation during rainfall and other atmospheric conditions.

For demonstration purposes, a LoS analysis was completed for the two slope radars used in this case study. Using an imported polygon of an identical area, a time series was created on both radars. The angle of influence for Radar 1 and Radar 2, as well as the measured displacement, is provided in Table 3 and Figure 10. The LoS analysis demonstrates a 50% vector loss in relative displacement in the X-Y axis based on the two points that show a difference of 55°. This example demonstrated the importance of LoS and understanding the nominal displacement values that are built into the velocity trends.

Table 3 Summary of radar line of sight analysis

Radar unit	Angle of incidence	Relative displacement shown
Radar 1	45°	0.2 mm
Radar 2	80°	0.4 mm



(a) (b)

Figure 10 Example of displacement vectors showing vector loss due to line of sight. (a) Radar 1 position; (b) Radar 2 position

4 Development of the slope model

4.1 Understanding the failure mechanism

The case study instability was quickly identified as a deeply-seated problem due to the presence of pervasive crest cracking and increased displacement across multiple batters in the slope face. With no clear structural release plane indicator or material strength deficit from the geological model, a detailed review of the failure mechanism was required.

Site experience with the geology, drone imagery of the movement area and adjacent across-strike walls was harnessed to identify the presence of brittle faults subparallel to the wall and infer the subsurface locations; this was a similar method used in the failure assessment of a nearby deposit (Figure 11). Limited drillhole data was available behind the crest, however, the identification of faults at the face was used to infer potential fault intercepts. When projected to the surface, these positions were more or less coincident with the more pervasive cracking behind, but closer to, the crest.

Figure 11 Generic example of using cross-strike aerial imagery to infer the subsurface location of fault systems

Review of the radar movement data identified anomalous regressive movement at the crest associated with the step change movements in the lower batter, suggesting a potential decoupling of the two locations. Additionally, the locations aligned to tension relief at the crest associated with movement prior to the observation of cracking behind the crest. In reviewing the construction stages of the pit and historical terrestrial and air imagery of the area, it was identified that prior to excavation of the current wall, the natural surface had been heavily trafficked by loaded trucks leaving a layer of imported material overlying the crest. It was hypothesised that the compacted material had resulted in tension cracks forming far behind the likely location of any underlying cracking in the natural ground.

Based on the information gathered, and the seemingly disconnected movement trends between the upper and lower slope, it was concluded that an active passive wedge style failure, driven by the 'brittle' faults interpreted in the wall.

4.2 Building a useful model

A baseline two-dimensional limit equilibrium analysis in *Slide2* (Rocscience 2023a) was created using the existing geological model and geotechnical parameters, with a cross-section created through the centre of the movement area. The initial models yielded an anomalous line of thrust much larger than what could be reasonably expected.

Reviewing this model suggested that the geology model was not truly reflective of the geological complexity between units in the pit wall. Unable to perform additional drilling, a two-dimensional finite element analysis in *RS2* (Rocscience 2023b) was chosen as the way forward for analysing the wall movement. This option provided movement vectors for elements in the model, as well as displacement and maximum shear strain information.

A number of sensitivity analyses were completed by adjusting individual parameters to determine which components of the model were controlling the behaviour and the magnitude of the adjustment. One change included removing a particularly competent welded pisolite unit from the geology model as it was not evident in the slope face. Faults were also added to the model using similar known orientations from other nearby exposures and across-strike interim walls.

Given the relatively limited damage zones observed in the fault structures, they were modelled as weak planes instead of discrete narrow units. They were assigned strength parameters as a coefficient of the material they passed through. This suitably reflected the faults as backscarp release planes, inferred from the observed behaviour and favourable orientations, rather than traditional drivers of instabilities. Sensitivity analyses were performed to ensure relative displacements between fault blocks were commensurate with the variability seen in the monitoring data.

Once geometry and structural elements were added (Figure 12), sensitivity of the slope to the material properties of individual detrital units were tested. This determined both the sensitivity of the slope to the parameters, and the observed behaviour to ensure it was realistic. Due to the movement observed in the wall, the analysis included applying residual material parameters to elements of the model once yielded and utilised to achieve a more realistic geometry on stresses contributing to wall failure.

These parameters were back-analysed for the particular geological conditions and failure mechanism in the slope. They were configured to partially compensate for the known unknowns of variability in the stratigraphic contacts and fault locations behind the wall.

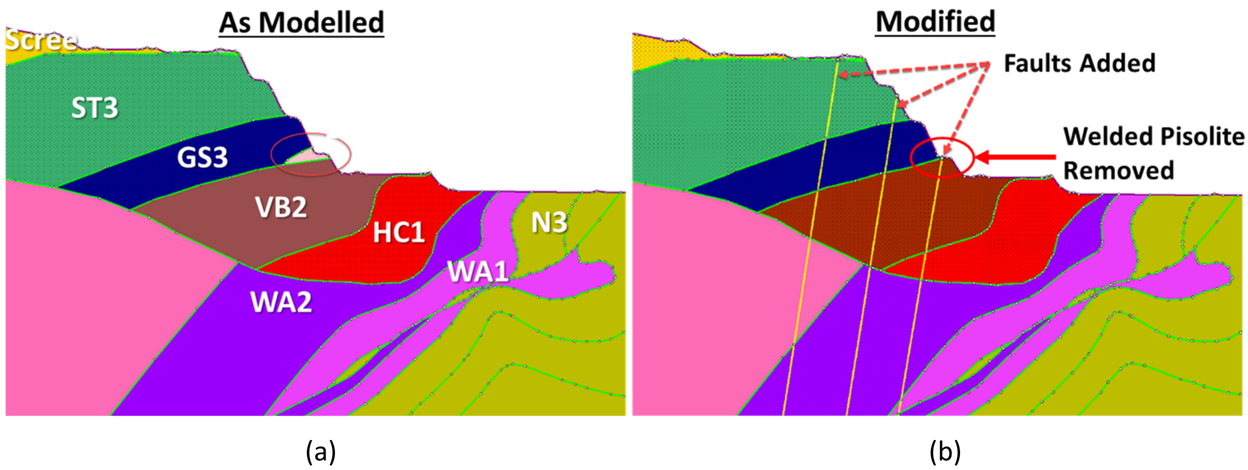


Figure 12 Geological cross-section used in the model set-up: (a) Model cross-section with as-given geology and including the exposed welded pisolite unit; (b) Modified cross-section including additional fault interpretations and the removal of the welded pisolite unit

4.3 Slope failure analysis

Finite element analysis was completed with the modified geology model. The critical strength reduction factor (SRF) design criteria for as-built conditions was 1.1. While this SRF was not manipulated to 1.0, consideration was given to the fact that the wall had not yet catastrophically failed. The model was also considered sufficiently robust to allow assessment of the current and future states of this and adjacent wall sections.

The maximum shear strain function in RS2 was used to demonstrate the impact of the pit wall instability at three stages: existing slope, one additional bench mined and two additional benches mined (Figure 13). The analysis showed a reduced SRF at each stage, with catastrophic failure expected to be triggered by the blasting of the next bench.

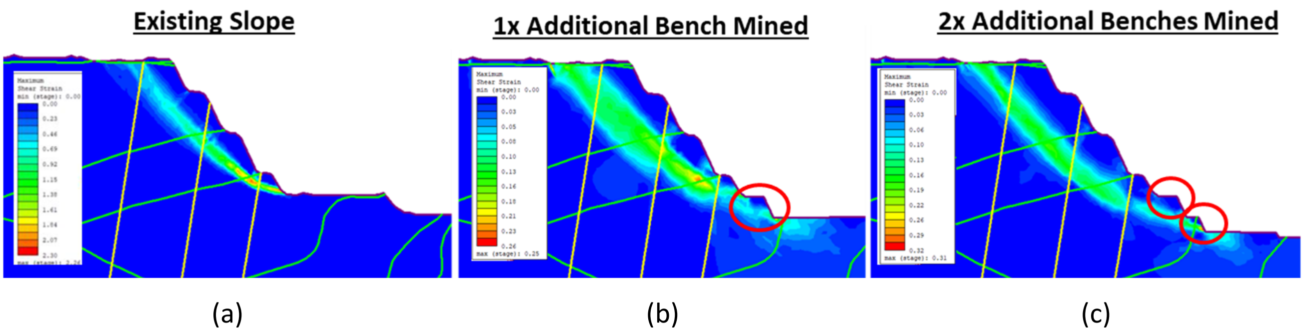


Figure 13 Finite element analysis showing maximum shear strain at three stages. (a) Existing slope; (b) One additional bench mined; (c) Two additional benches mined

The slope prior to blasting and excavating the current bench was also analysed as part of the back-analysis. The results showed that excavating the current bench caused a significant drop in the SRF which correlated well with the change in visual slope conditions and the geological model used in the analysis.

5 Remediation strategy, analysis and design

5.1 Remediation strategy

In order to determine a remediation strategy, all possible options were considered. Broadly, there are three geotechnical options to resolve slope instability (National Resources Conservation Service 2005):

1. Unload the crest, which reduces the active driving forces.
2. Reinforce the toe of the slope, which increases the resisting forces.
3. Allow the failure to (safely) occur and remediate the outcome.

The crest surface was deemed unstable and unsafe to complete unloading activities due to the extensive cracking. The construction of a toe-stabilisation buttress was assessed as the most suitable solution due to equipment suitability and stepped displacement in wall movement. Buttresses have been used at several BHP mine sites by geotechnical engineers to resolve similar issues (Young et al. 2022).

5.2 Remediation analysis and design

Engagement with operations determined that the buttress was to be 20 m wide to satisfy the minimum operational width. The buttress height of 16 m was determined by geotechnical engineers to ensure the exposed fault and associated minor batter scale failures were completely encapsulated and stabilised.

Using the modified geology model, the finite element analysis of the buttress provided an acceptable SRF. The comparison in maximum shear strain between the original design and with the buttess in place is clearly shown in Figure 14. Following design confirmation that the buttress would achieve the design safety criteria, the solution was proposed and accepted for execution. This also triggered a revision of the pit design by mine planning to accommodate the buttress design. Spanning approximately 550 m in planned length with a total waste volume of 689,000 t, the buttress layout is provided in Figure 15.

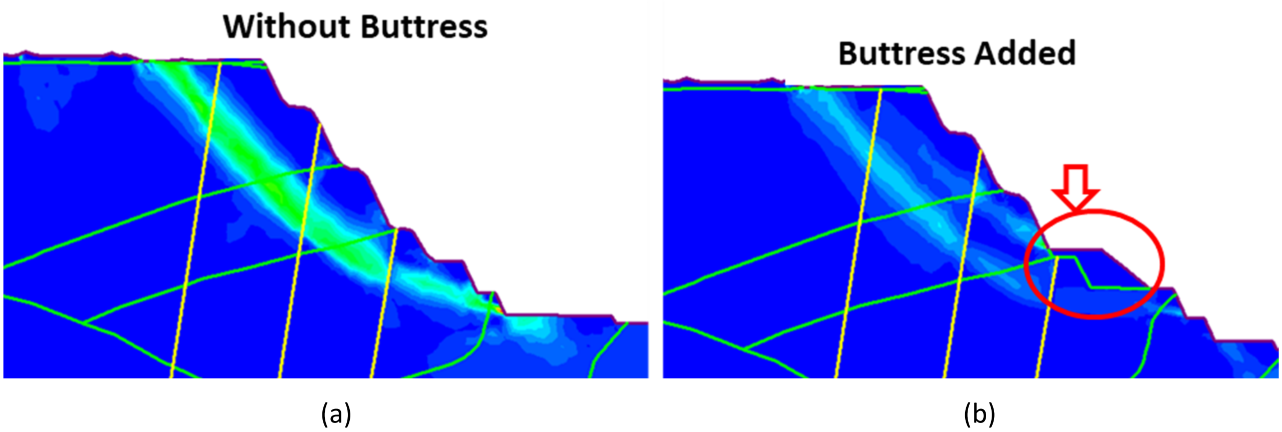
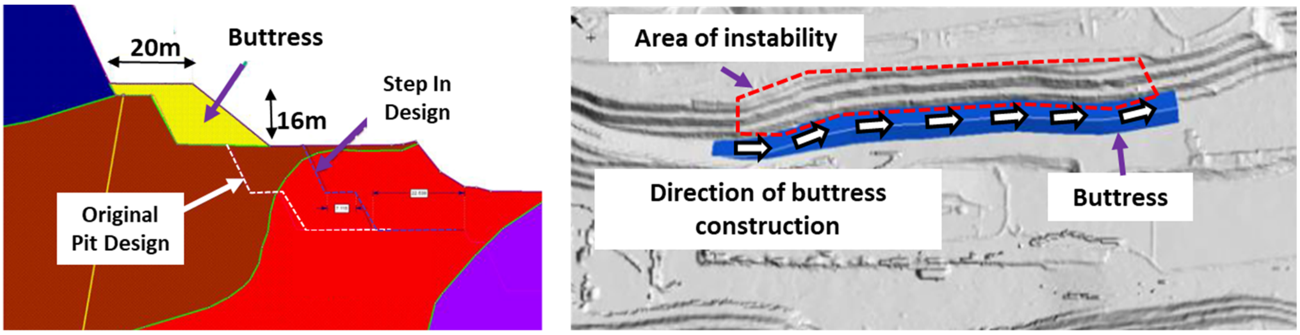


Figure 14 Maximum shear strain displayed for the finite element analysis comparison. (a) Original planned pit design; (b) Remediation configuration of step in with buttress constructed, as circled in red



(a) (b)

Figure 15 Buttress layout. (a) Cross-sectional dimensions; (b) Plan view position and construction direction

6 Execution of remedial design

6.1 Operational engagement and creating a psychologically safe environment

Early engagement with the operational team provided valuable feedback on the buttress design prior to the execution. As a result, the buttress design was widened to allow better truck access, strategically placed turning bays and suitable nightshift lighting tower locations.

The buttress was constructed over a 49-day period. The safe execution of the buttress was underpinned by the use of tactical ground-based radars. Two ground-based radars were used to provide system redundancy. This meant minimal production downtime and allowed for validation of movement data to be performed in real time during construction. A formal job hazard assessment accompanied the use of the slope radar TARP and alarming system in place.

The WAIO geotechnical team employed a number of communication techniques to contribute to the safe execution of the buttress. These techniques helped to form a 'safe to speak up' culture (Foot & Armitage 2022) (Figure 16) and include:

- Formal risk assessment and presentation of the geotechnical analysis with site leadership and supervisor teams. This enabled alignment prior to construction.
- Regular engagement with the scheduling teams to ensure buttress construction was accounted for in weekly plans.
- Face-to-face training with the operational crews on the safety controls in place, material types, accountabilities in the slope radar TARP and the reasoning for the buttress construction.
- Prompt review and assessment of the slope if any visual changes were observed.
- A regular and consistent field leadership regime across the underpinning critical controls.

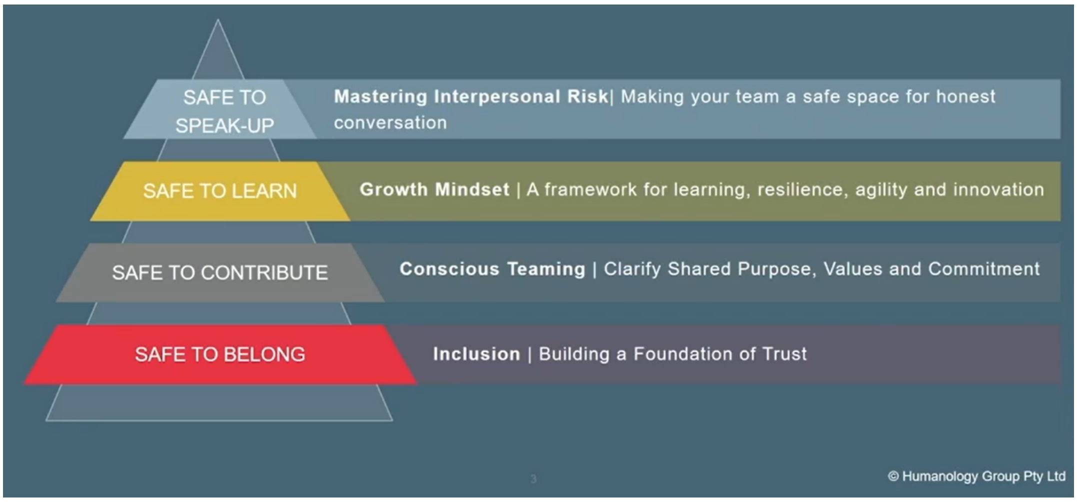


Figure 16 Elements of a psychologically safe environment (Foot & Armitage 2022)

6.2 Monitoring the buttress effect

Buttress construction commenced 135 days after the first blast-induced movement, with a relative cumulative displacement of approximately 260 mm encountered within that time.

It was important to monitor the progressive effect of the buttress to ensure:

- Progress in stabilising the slope was being made, noting the criticality of the time frame and desire to get the buttress in place prior to the Pilbara wet season commencing.
- Proactive identification and control of any residual risk to personnel constructing the buttress for the upper portion of the slope.
- The change in the stress regime across the wall was assessed for any increased movement in the portion of slope that was yet to be buttressed.

As the buttress progressed from west to east (left to right in the images), area time series on the radar were used to track the effect of the buttress on the wall (as shown in Figure 17). Anomalous red areas on the floor show where the buttress footprint progression was not yet reflected in the radar digital elevation model.

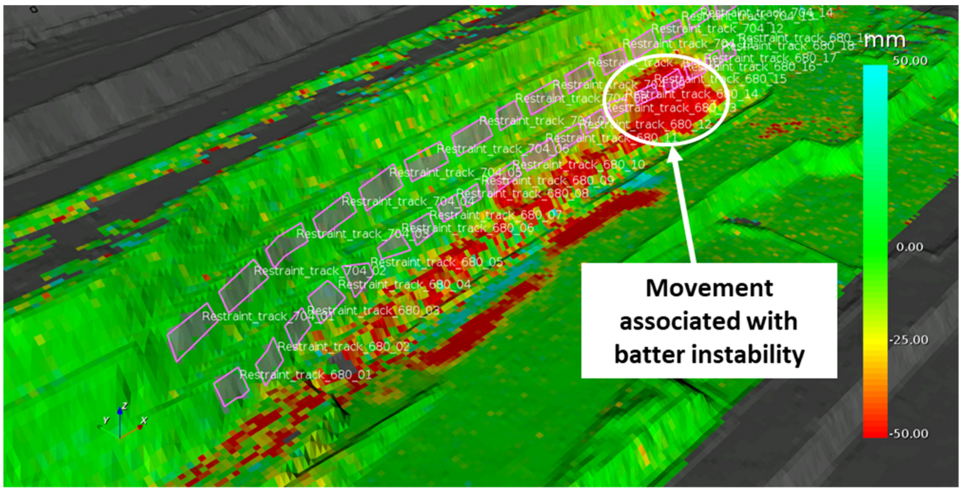


Figure 17 Area polygons for tracking movement restraint along the wall as the buttress progressed

The buttress effect is shown in Figure 18 over a six-day interval. The red circle highlights a batter scale failure acceleration induced by the shifting stress regime. This batter scale failure is also shown in Figure 17 as the red area towards the top right of the image and is discussed further in Section 6.3. It was triggered by the changing stress regime across the pit wall as the buttress progressed. Concurrently, the start of regressive movement due to the advancement of the buttress is shown in the green circle displacement trend of Figure 18. The upper trendlines are from area polygons that have been fully buttressed but which show some interference from passing trucks and dust on the buttress.

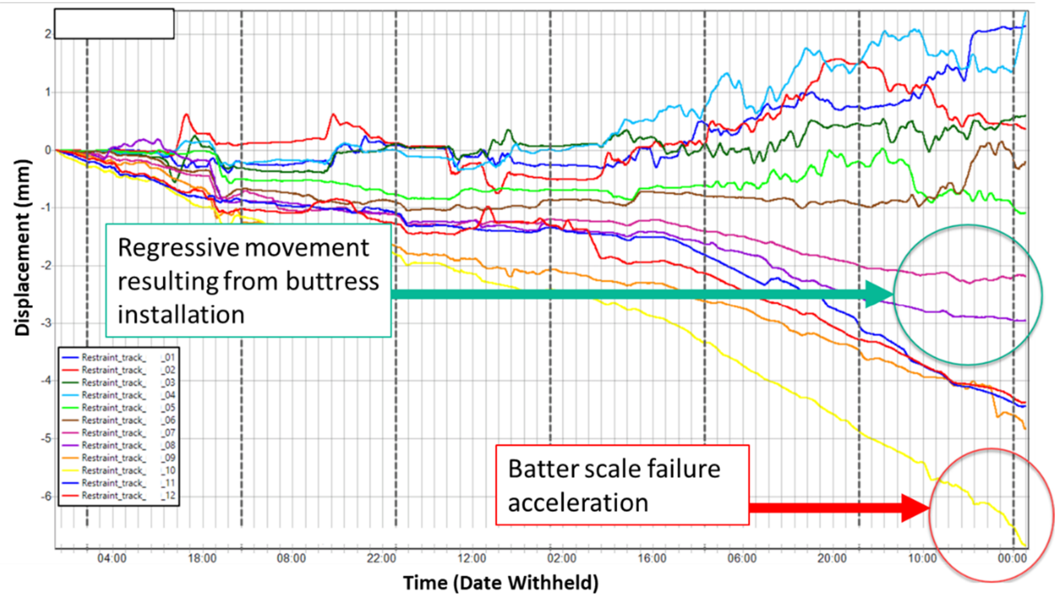


Figure 18 Buttress effect shown using the area time series from Figure 16

6.3 Example response to radar alarming

The TARP and slope alarming system functioned as intended and alerted the team to one instance of instability exceeding alarm thresholds (Figure 19) during the buttress construction. This event involved the operational team being advised by the vendor monitoring team to withdraw from the area several hours before a batter scale instability occurred. The TARP was executed as intended, and the site geotechnical engineers then inspected and added additional controls to manage the batter scale failure so that the buttress construction could re-commence safely. No additional rilling or fall of ground was subsequently encountered.

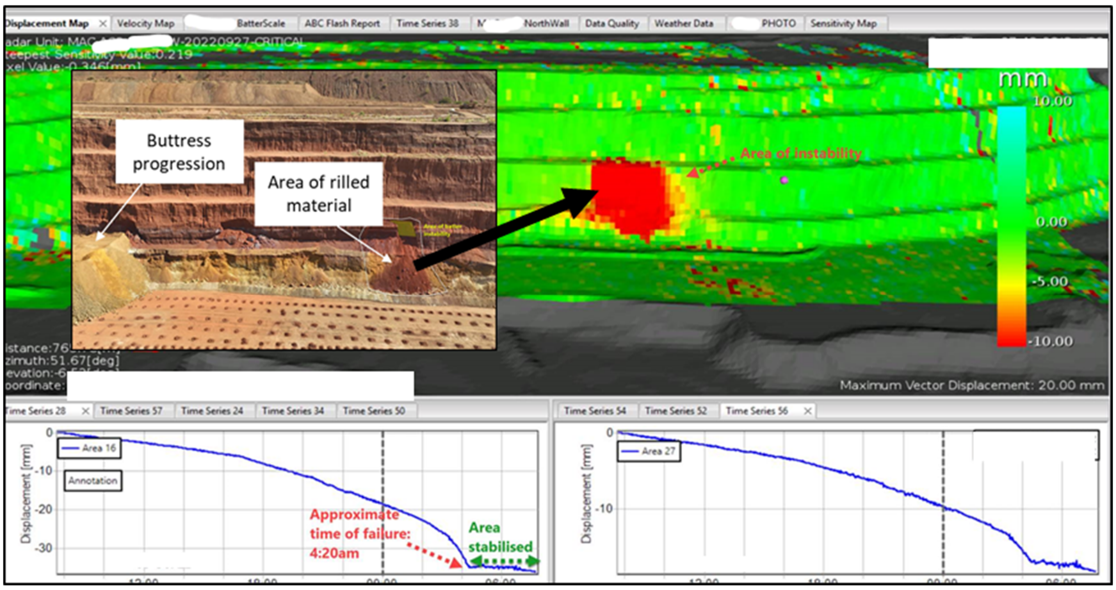


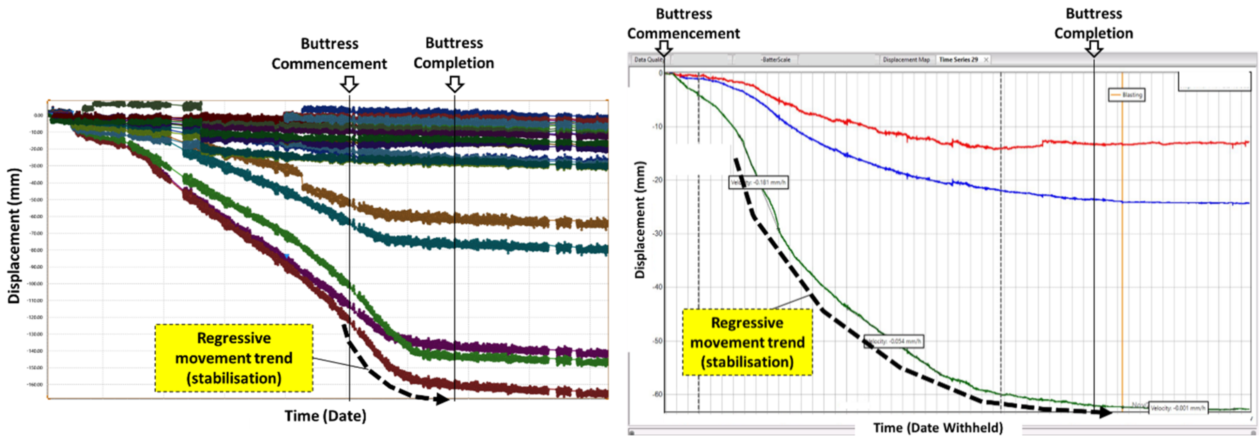
Figure 19 Example of radar data visualisation from a batter scale failure captured by alarming, and safe withdrawal. Inset: inspection of failed area

7 Review of post-buttress movement and complementary considerations for subsequent pit progression

7.1 Review of movement post-buttress completion

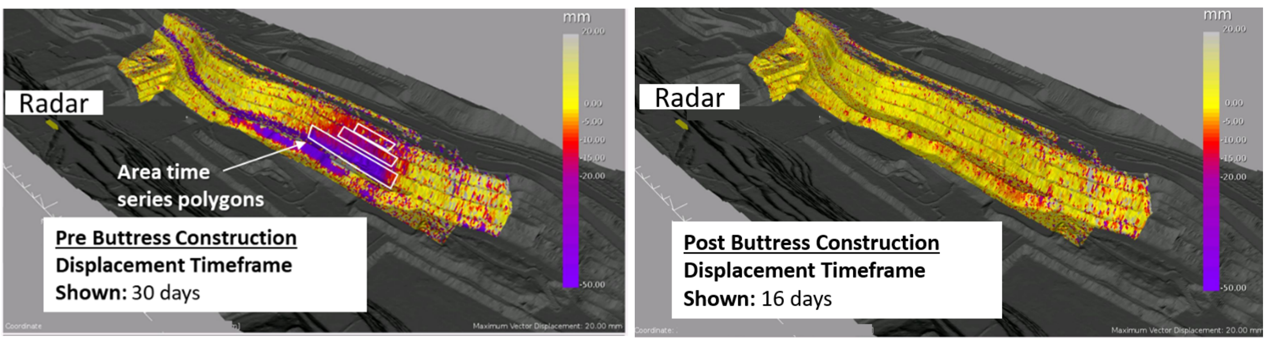
A regressive movement trend was evident across all three primary monitoring systems after the buttress construction was completed. Prism movement rates have subsequently stabilised to <0.01 mm/day at the time of writing the report. These rates are considered within acceptable limits; however, they continue to be reviewed with new data received daily. Radar movement has also shown a similar regressive movement trend since the buttress completion. The area time series displacement charts demonstrate a clear stabilisation since buttress completion, with movement rates currently <0.01 mm/hr (Figure 20). The area time series polygons used are shown in Figure 21a. Three-dimensional visual representation of displacement rates pre- and post-buttress construction is shown in Figure 21.

As the buttress progressed from west to east, the regressive trend was not uniform across the prisms and radar areas. For this reason, further pit progression was put on hold for another month after the buttress completion to ensure the regressive trend had been suitably validated.



(a) (b)

Figure 20 Long-term displacement rates showing slope stabilisation post-buttress construction. (a) Ground-based prisms; (b) Ground-based radar using area time series polygons shown in Figure 21a



(a) (b)

Figure 21 Three-dimensional visualisation of movement from two different time frames: (a) Pre-buttress construction; (b) Post-buttress construction

Due to satellite orbiting frequencies, InSAR data is not provided in real time. However, the data received suggests a decelerating trend has occurred since the buttress construction. The two-dimensional vertical displacement data was selected for detailed analysis of five representative points. Two points were selected on the batter face movement areas and three points selected on the significant crest cracking locations. Figure 22 shows the two most characteristic data points for the movement. The users found that the vertical displacement data is best used for flat surfaces such as the pit crest surface and backscarp, where the InSAR data is more representative and limits orbital vector loss.

Figure 22 Satellite InSAR data collected from representative locations: (a) Batter face location; (b) Crest cracking location

7.2 Surface water management considerations

Consideration of surface water management during the buttress construction, and during subsequent pit progression, was imperative. The WAIO water engineering team assessed the major water flow runoff paths for the pit slope by conducting a watershed analysis (GlobalMapper 2023) seen in Figure 23.

The water management strategy was to primarily minimise water ingress into crest cracks. Controls were added to ensure that any surface water should runoff over the crest instead of being allowed to pool, thus maintaining a free-draining buttress surface without creating significant erosion. Several techniques were employed to achieve this including:

- Substantial windrows constructed at strategic points to minimise water ingress towards the pit crest.
- Suitable cross fall designedon the buttress surface during construction.
- Installed water-diverting drainage windrows with V-drains to direct water off the buttress and minimise water pooling.
- A sump designed in the low point of the pit to capture water.

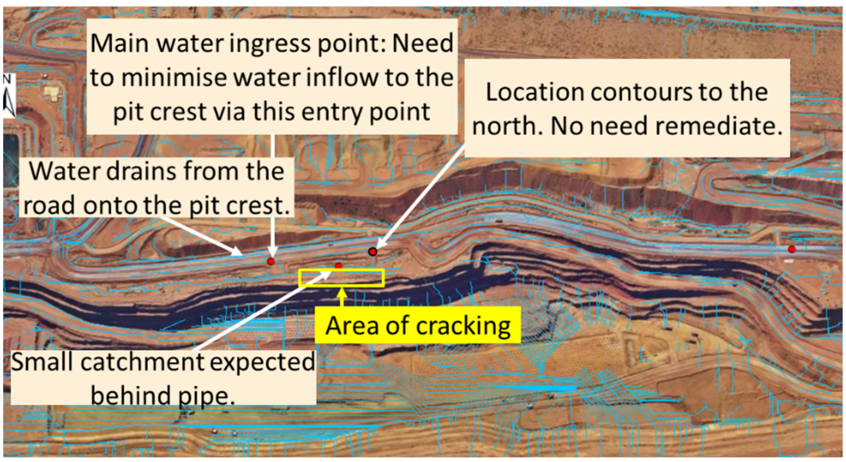


Figure 23 Watershed analysis provided by the WAIO water engineering team

The successful collaboration between the geotechnical and water engineering departments was critical to preparing for the Pilbara wet season. The buttress has now been exposed to several wet weather events, including an event with a maximum of 31 mm rain over a 24-hour period (Bureau of Meterology 2023). The controls were demonstrated to be successful and allow the water to be drained away from the slope face and buttress. A small amount of localised ponding is visible near the pipeline which is 30 m away from the crest edge. Figure 24 shows the crest with minimal water pooling left on the crest after a significant rain event. Importantly, water management controls have reduced saturation of the slope and further erosion through the crest cracks.

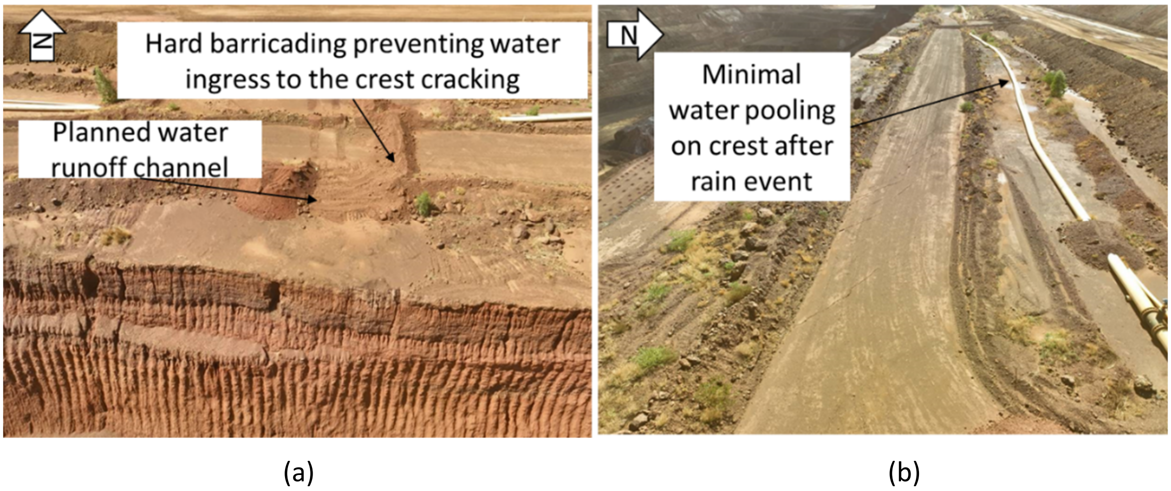


Figure 24 Implemented and verified water management controls. (a) Runoff channel and hard barricading controls installed; (b) Crest condition after a wet weather event showing minimal water pooling at the crest edge

7.3 Blasting considerations

Mining of the pit below the buttress is currently underway, and site geotechnical engineers continue to work closely with drill and blast engineers to align continued safe production. A standard routine has been established to undertake field verification of the drill cone material types and review the drill penetration data with a view to ensuring that suitable blast designs, explosive charges appropriate to actual materials present, and suitable initiation points and blast geometry are agreed to prior to the loading of explosives. Theoretical maximum PPV limits were set based on a review of the theoretical PPV calculations from blasts prior to the identified slope movement. Additionally, free-faced blasting was utilised where possible.

Subsequently, the blast pattern directly below the buttress was fired and safely excavated without any significant wall movement observed in monitoring data. The blast was fired within the recommended PPV limit of 50 mm/s, with a theoretical 90% confidence limit of 44.1 mm/s PPV. No observable back-break or poor digging conditions were encountered. The blasting strategy was executed with an additional six-hour geotechnical stand-off post-blast to ensure any unexpected movement triggered by the blast would not pose a risk to personnel.

Blast and wall performance are reviewed using internal design compliance processes which include the geotechnical engineers comparing the as-built batter to the design batter and issuing any design or operational controls required if non-compliance issues are experienced (Figure 25).

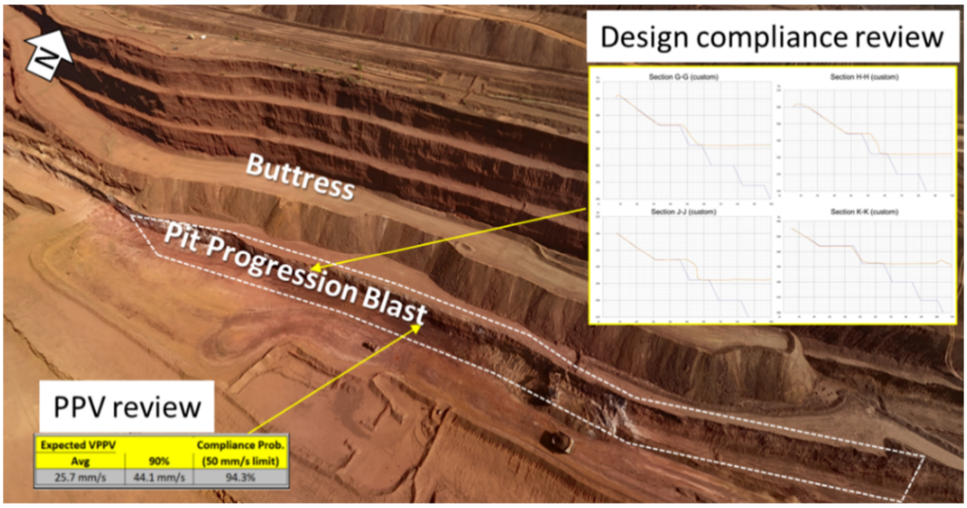


Figure 25 Example peak particle velocity (PPV) and design compliance review of the production blast below the buttress

8 Conclusion

The advancement of slope radar technology in recent years has enabled geotechnical engineers to predict and plan for slope failure. The slope radar network deployed at BHP WAIO operations has enabled the prevention of a significant wall failure through early identification of movement and construction of a toe-stabilisation buttress, allowing continued mining of the pit to depth.

The slope radar TARP enabled the safe execution of the buttress and is a process and set of systems that has been replicated across several of BHP's assets. The success of the slope radar TARP is underpinned by an understanding of the radar theory and alarm setting limitations such as LoS.

The execution of a toe-stabilisation buttress, or any other works in close proximity to potential active wall failures, must be supported through the creation of a psychologically safe environment where operators understand, and are comfortable with, the controls in place. Safety is best enabled through the collaborative effort of multiple teams, with careful consideration for how their work impacts on other teams.

The value of local site knowledge and practical experience to a site geotechnical engineer should not be underestimated. They were useful in building a working model and an informative site photo dataset, and the comprehension of how materials behave over time.

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