APSSIM 2016 – PM Dight (ed.)

doi:10.36487/ACG_rep/1604_05_Sainsbury

# Three-dimensional numerical modelling of potential structurally controlled failure mechanisms at the Kanmantoo open pit

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## Abstract

*The Kanmantoo Copper Mine is located 55 km southeast of Adelaide, South Australia, and is owned and operated by Hillgrove Copper Pty Ltd (HGO). The largest of three currently operating pits at Kanmantoo is the Giant Pit, which is operating over HGO's recently completed Kavanagh Pit which was mined to a depth of 240 m. The Giant Pit, in which mining is currently taking place in the upper benches, will extend to a maximum depth of 380 m with steeper overall slope angles.*

*Mining of the Kavanagh Pit provided a valuable understanding of the geotechnical controls on slope stability and the efficiencies that could be achieved in the design of the Giant Pit. The high-strength intact rock and four distinct structure sets impose specific structural controls on the stability of all pit walls, which are well understood and well controlled from the experience of mining the Kavanagh Pit. HGO recognised that if small-scale failures could be controlled, better crest retention and improved berm capacity would be achieved, thereby enabling steeper overall slope angles.*

*Although no deep seated failures have occurred to date, the steeper overall slope angles, and the greater depth of the Giant Pit prompted concerns about the possibility of larger-scale, deeper-seated failures that may include a combination of sliding on structures and shearing through rock. The deep pit and steep walls also prompted concerns about the potential impact of mining induced stresses.*

*Numerical modelling was conducted using the three-dimensional distinct element code 3DEC, which represents the rock mass as a combination of deformable finite difference zones and discrete discontinuities. To adequately represent the jointed rock mass and the structural controls on stability of each pit wall, structures needed to be represented explicitly as discontinuities in the 3DEC models. Discrete fracture networks were generated to include discontinuities of an intermediate scale with relative frequencies, lengths and spacing appropriate for each structure set.*

*This work has demonstrated that complex jointed rock masses can be simulated successfully in numerical models by the use of a combination of appropriately spaced discrete structure and rock mass blocks, in a way that represents the key structural controls observed in the mined pit slopes. Besides representing the overall rock mass behaviour more realistically, such models can also better capture the influence of excavation sequences and both in situ and mining induced stresses, therefore, providing useful insight on the behaviour of the excavation that can be used for both mine planning and design.*

## 1 Introduction

Hillgrove Copper Pty Ltd (HGO) owns and operates the Kanmantoo Copper Mine, located approximately 55 km southeast of Adelaide, South Australia (Figure 1).


**Figure 1 Project location**

The mine operates in several open pits. Giant Pit, which is the largest and deepest, operates over HGO's recently completed Kavanagh Pit, older open-cut mine voids that were mined in the 1970s by a previous owner, and underground workings that date from the 19th Century and the 1970s.

Kavanagh Pit was, at its completion, the largest and deepest pit at Kanmantoo to date with the floor at 978 m RL, and was 240 m deep. Mining of the Kavanagh Pit provided HGO with a valuable understanding of the geotechnical controls on slope stability, and what changes and improvements could be made in the design of the Giant Pit. The high-strength rock and four distinct structure sets impose specific structural controls on pit wall stability, which are well understood and well controlled from the experience of mining in the Kavanagh Pit.

The Giant Pit will extend and deepen the Kavanagh Pit to form a larger pit up to 380 m deep, and 580 m in diameter (Figure 2). Mining has commenced in the upper northwestern benches of the Giant Pit, which will be excavated in the same rock types as the Kavanagh Pit, with similar geotechnical controls on stability.

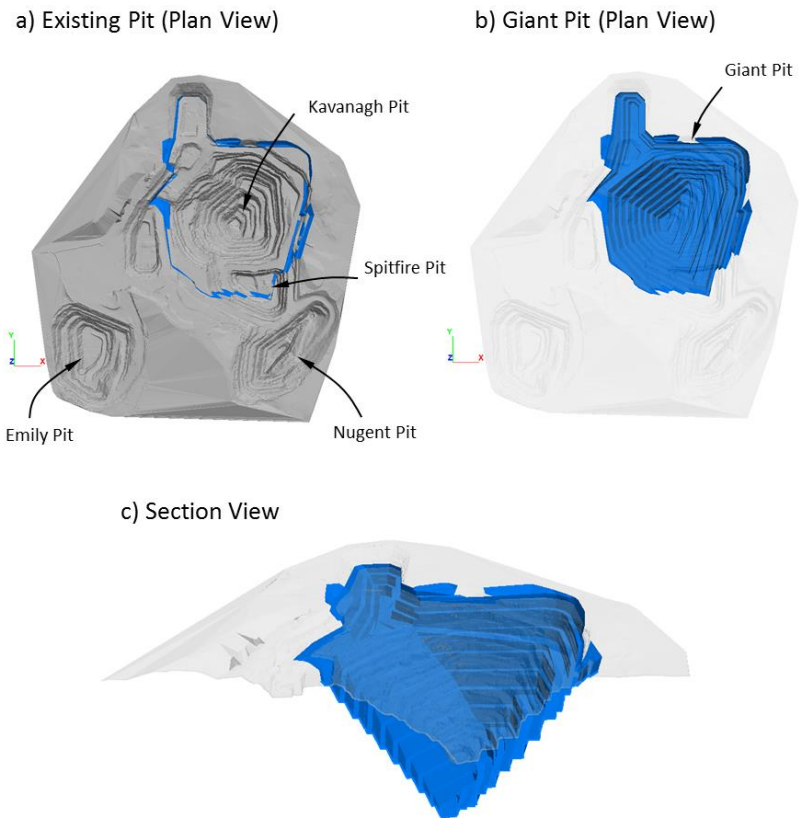


**Figure 2 Existing Kavanagh Pit and proposed Giant Pit design**

The Giant Pit is proposed to be mined at steeper wall angles than the existing Kavanagh Pit. Steeper overall wall angles are proposed in the west, northwest and southwest walls of Giant Pit, to be achieved through the use of shear pins for improved batter stability and berm crest controls.

In the southwest, west and northwest walls, the opportunity was identified for a significant increase in slope angles. The overall slope angle of the west wall of Kavanagh was approximately 49°, and for the Giant design, it is 58°.

Deep seated failures through fresh rock have not occurred, and to date have been considered unlikely to occur. However, the considerable depth of the Giant Pit, coupled with the steeper proposed overall wall angles, raised two concerns for larger scale stability:

- Whether mining induced stresses at depth may cause rock mass failure.
- Whether structurally controlled failures may occur at a larger scale by a combination of sliding on structures, and rupture through the rock mass.

# 2 Geotechnical setting

## 2.1 Pit design

Figure 3 shows a cross-section of the existing and proposed pits, and highlights the significantly steeper proposed slope angles which will be achieved through the use of ground support to improve batter stability and retain berm crests.

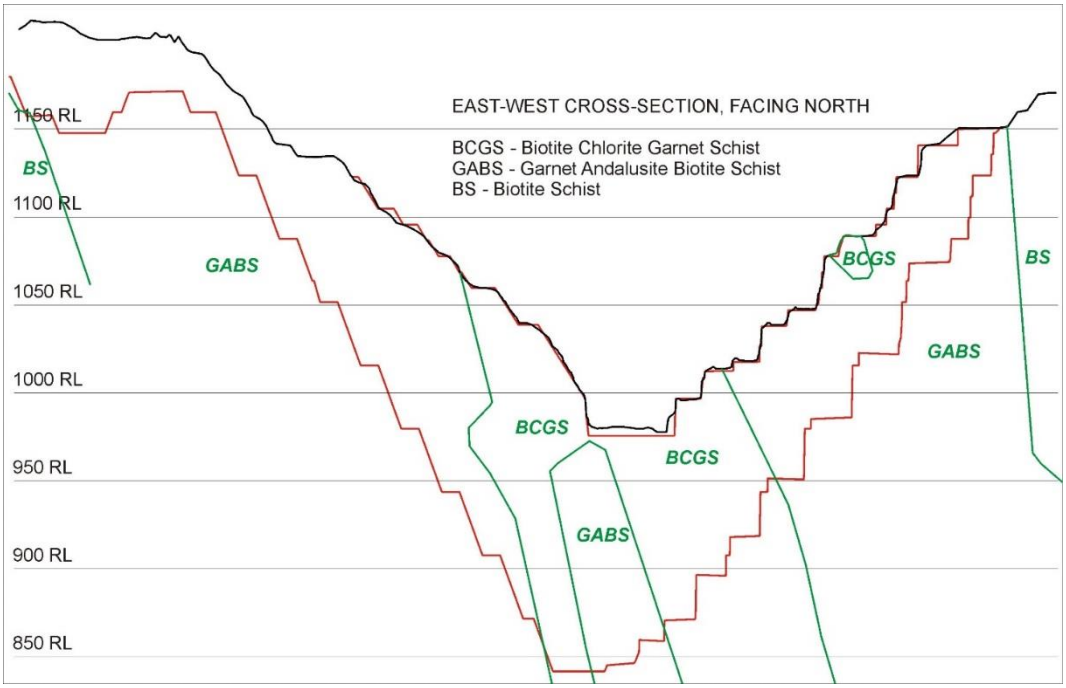


**Figure 3 Cross-section through existing Kavanagh Pit and proposed Giant Pit design, and geology**

## 2.2 Geology

The mine lies within an approximately north-south striking garnet-rich alteration zone near the centre of the Kanmantoo Syncline, on the southeastern edge of the Adelaide Fold Belt.

Regional faults including the Bremer and Nairne Faults trend approximately north-south and lie several kilometres to the east and west of the mine.

No major regional faults occur within or near the deposit. Only one fault is known to have been mapped, and is currently under water in the Spitfire Pit. The region has a complex structural history with several

phases of deformation, and thrust faulting is known to have occurred in the region. The lack of regional marker beds has resulted in a poor understanding of the detailed faulting structure of the Kanmantoo Group.

The local geology comprises schists that conform to the regional north-south trend, with three main rock types, shown on Figure 3:

- Biotite Schist (BS) at the western and eastern boundaries, is light grey, fine grained and poorly foliated, and is weaker than the other units.
- Garnet Andalusite Biotite Schist (GABS), grey-green and foliated, and contains 5 to 10 mm porphyroblastic crystals of andalusite in a coarse grained schistose matrix of biotite, quartz, almandine garnet and chlorite.
- Biotite Chlorite Garnet Schist (BCGS), which is associated with the mineralisation. The BCGS is poorly foliated and fine grained, and is gradational with the GABS described above. There are no currently observable differences in rock mass quality or strength within the current Kavanagh Pit, most of which lies within the BCGS and GABS.

Thus, broadly, most of the Giant Pit lies within the stronger, better quality rock mass of the GABS and BCGS, with weaker BS on the western and eastern margins. The western margin of the GABS transitions to BS over a zone, represented by a region of intermediate rock mass strength.

## 2.3 Structures

Almost 15,000 structural measurements have been collected at the mine, and they reveal four distinct structural sets, with orientations shown on a stereonet, shown as Figure 4, and described in Table 1.

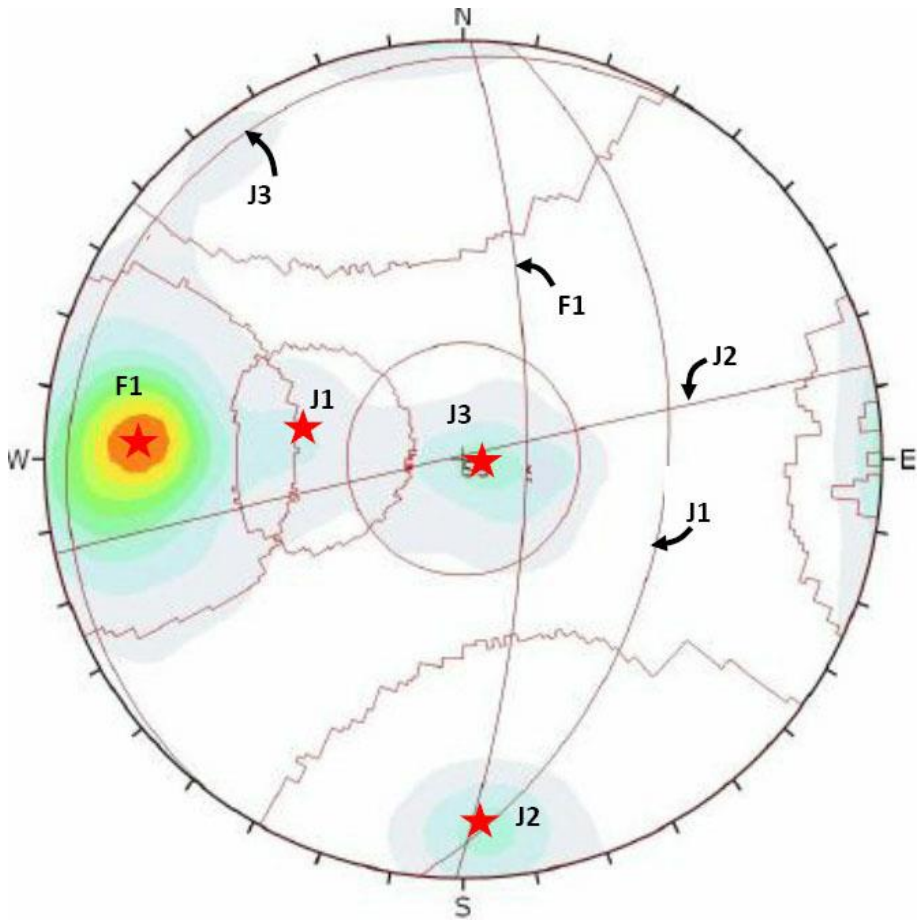


**Figure 4 Structure set orientations**

**Table 1 Structure sets, summary and description**

| Set | Mean dip/dip direction (deg.) | Comments |
|---|---|---|
| F1 (foliation) | 73 / 091 | Foliation is prevalent, but joints parallel to the foliation are less so. F1 joints are more common in the upper west wall of Giant Pit. |
| J1 (joints) | 38 / 096 | J1 joints are very wavy, rough and persistent, with some longer than 20 m. J1 surfaces have a cross-cutting schistosity with banded texture that creates a small to medium scale roughness. J1 joints are more common in the western half of Giant Pit, gradually becoming less common in the east. |
| J2 (joints) | 88 / 347 | J2 joints contain quartz veins, and are planar, slightly rough and persistent, typically at least 10–15 mm long, and some longer than 60 m. J2 joints are usually tightly healed as the quartz provides some tensile strength, but they open up due to blasting. |
| J3 (joints) | 04 / 306 | J3 joints are very shallow and generally only 2–3 m long. They are occasionally seen as upper release surfaces for some small wedge failures, resulting in small overhangs. |

To simulate the jointing within the various rock mass domains a discrete fracture network (DFN) was developed using the stochastic DFN generation module within the 3DEC program (Itasca 2013). Figure 5 illustrates the distribution of structures within one DFN realisation, while Table 2 provides a summary of the DFN statistics. Exposure mapping from the Kavanagh Pit provides good confidence in the DFN generation. The DFN generation and validation process was based on a methodology described by Vakili et al. (2014).

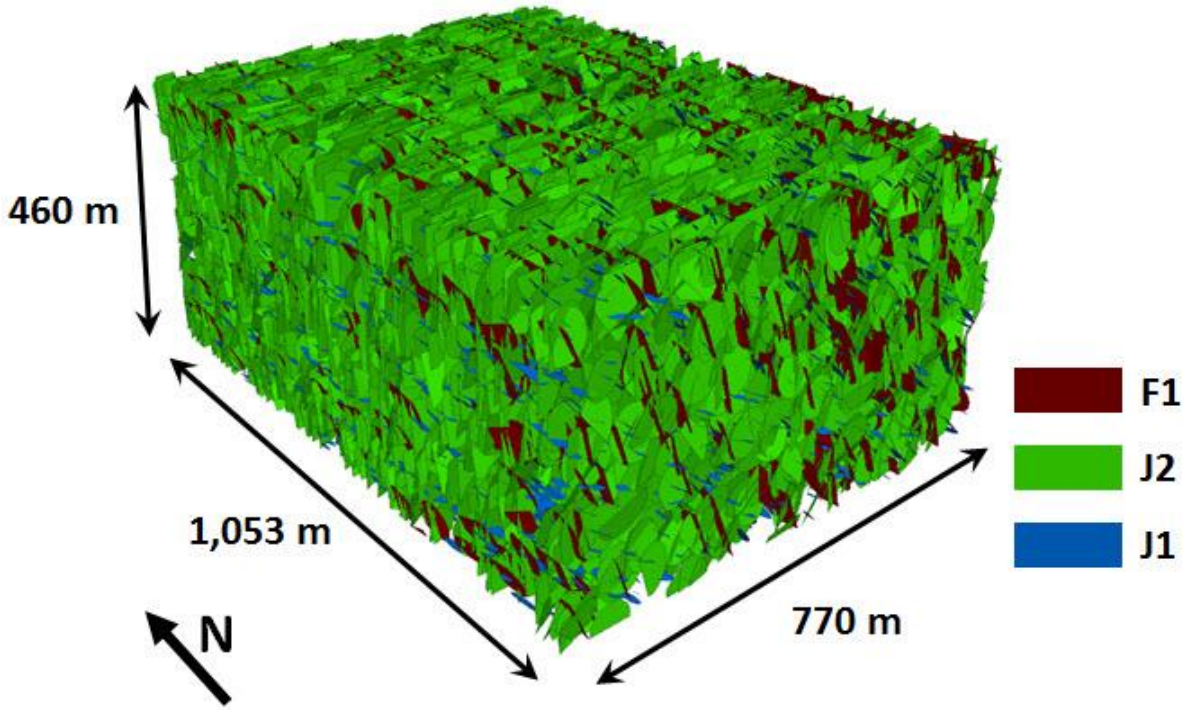


**Figure 5 Joint traces with DFN**

**Table 2 DFN Characteristics of intermediate scale structures within 3DEC models**

| Joint set | Average dip (std. dev.) | Average dip direction (std. dev.) | Average spacing | Persistence |
|---|---|---|---|---|
| J1 | 38° (±10°) | 96° (±10°) | 10 m | 20–60 m |
| J2 | 88° (±10°) | 347° (±10°) | 6 m | 20–80 m |
| F1 | 73° (±10°) | 91° (±10°) | 10 m | 20–60 m |

## 2.4 Slope stability controls

Pit wall stability in the completed Kavanagh Pit has been dominated by structures, with each wall having particular interactions with structures at a batter scale. Multi batter or larger scale failures have not occurred in the completed Kavanagh Pit, or to date in the Giant Pit.

Stability controls are summarised in Table 3. Figure 6 shows a view of the northwest, north, and west walls of the partly completed Kavanagh Pit in April 2014. The influence of J1 joints in the west and northwest walls can clearly be seen.

**Table 3 Stability controls**

| Wall | Stability controls |
|---|---|
| East | Structures are favourably oriented. Occasional rock falls and crest losses occur on random structures. Set F1 dips steeply into the wall, but has not produced toppling failures.<br>Steep walls have been achieved, and rock falls are the predominant hazard. |
| South | Set J2 dips out of the wall, with dips ranging from 70 to 90°. Planar sliding may occur on batters steeper than 70°. |
| Southwest | Set J1 forms wedges with Set J2. Set F1 joints form steeper back release planes where F1 joints occur, which are more common in the upper west wall.<br>Wedge failures tend to be less common than in the northwest wall for reasons explained in the northwest wall discussion below. |
| West | Set J1 dips to the east and causes planar sliding failures. Set F1 joints occasionally form steeper back release planes.<br>Set J1 will have a significant impact on the success of west wall batter designs in Giant. |
| Northwest | Set J1 dips towards the east, forming wedges with Set J2. Set F1 joints occasionally form steeper back release planes.<br>Wedges form between J1 and J2 structures in northwest and southwest walls. Wedge failures tend to be more common in northwest walls where failure involves tensile rupture of quartz healed J2 joints, i.e. blocks pull away and leave an undercut; whereas in the southwest wall, failure involves shearing on the quartz healed J2 joints. J2 joints are weaker in tension than in shear, resulting in more wedge failures in the northwest wall. |
| North | Set J2 dips into the wall, with dips ranging from 70 to 90°.<br>Blocks fall away from J2 (fail in tension) causing undermining of batters in north walls. This has been observed to be considerably more common where poor batter control practices were in use by a previous mining contractor. |


**Figure 6 Kavanagh Pit, April 2014**

HGO employ shear pins as a method of ground support, which are 12 m long vertical steel bars cemented in holes, spaced 2 m apart and 1 m from crests. Shear pins were trialled in the nearby Nugent Pit, and proved successful in preventing sliding on J1 joints and retaining crests in the west, northwest and southwest walls. After their successful trial in Nugent Pit, they have begun to be employed in the upper walls in the west of Giant Pit, and have proven successful in retaining crests and maintaining effective berm widths.

The west, northwest and southwest walls of Giant Pit are designed with 36 m high benches, compared to 18 m as mined in Kavanagh Pit, and with steeper overall slope angles. The potential for larger scale failures to occur on a combination of J1 joints and rock mass was raised as a concern for larger scale stability, and led to the three-dimensional distinct element numerical analysis that is the subject of this paper.

# 3 Numerical analysis of the Giant Pit

## 3.1 Modelling methodology

Naturally, discrete modelling techniques provide the most accurate description of discontinuous materials such as jointed rock masses (Riahi & Curran 2009). However, historically it has not been practical to explicitly simulate the joint fabric of an entire rock mass for routine analyses of large-scale three-dimensional excavations. Application of the distinct element method (DEM) to the analysis of three-dimensional slope stability problems has traditionally been limited to the discrete simulation of only the pit-scale major structures such as faults and shear zones, while the bench-scale joint fabric is simulated with an equivalent continuum material (Sainsbury et al. 2007; Severin et al. 2013; de Bruyn et al. 2013).

To investigate the structurally controlled failure mechanisms within the Giant Pit, the DFN of intermediate-scale structures described in Section 2.3 was implemented within a three-dimensional mine-scale 3DEC model. The 3DEC code has been developed specifically to study complex failure mechanisms involving large numbers of explicit structures that divide a rock mass into blocks. Slip, separation and rotation along explicit structures can occur, while the individual blocks can deform and yield.

A bi-linear Mohr–Coulomb strain-softening constitutive model was used to represent the behaviour of the equivalent continuum rock mass between the intermediate-scale structures. Because the Mohr–Coulomb criterion was used to define the strength of the rock mass, values for cohesion and friction angle were obtained by a least-squares fit to the Hoek–Brown curve. A bi-linear fit was obtained over a range of confining stress from 0 to 1 MPa.

## 3.2 Rock mass material properties

Bench-scale exposure mapping together with laboratory testing was conducted to derive the equivalent continuum rock mass strength properties used throughout the analysis. Table 4 presents the Hoek–Brown parameters used to represent the fresh domains (see Section 3.6). A constant dilation angle of 10° was used for all domains.

**Table 4 Rock mass material properties used for each domain**

| Domain | Density (kg/m³) | GSI | $\sigma_{ci}$ (MPa) | $m_i$ | D | $E_{rm}$ (GPa) | $v$ |
|---|---|---|---|---|---|---|---|
| GDM1a | 3,400 | 80 | 47.5 | 10 | 0 | 61.2 | 0.20 |
| GDM1b | 2,900 | 70 | 47.5 | 10 | 0 | 38.8 | 0.22 |
| GDM2 | 2,600 | 45 | 18 | 10 | 0 | 6.1 | 0.25 |
| GDM3 | 2,800 | 50 | 18 | 10 | 0 | 9.3 | 0.25 |

Little blasting induced disturbance of the pit walls was evident at the site. In addition, the stress relaxation effect or disturbance caused by mining induced stresses were accounted explicitly in the model by using a strain-softening material model. Therefore no disturbance factor (D) was used in the models.

## 3.3 Joint properties

The in situ shear strength of large-scale geological structures is difficult to determine with good confidence. To investigate the waviness of exposed discrete structures, high resolution three-dimensional laser scanning of exposed J1 joints was conducted with a Maptek I-Site 8820 laser scanner (Hutchinson &

Howarth 2015). These laser scans have been used for input to an experimental numerical simulation of a large-scale direct shear environment, and to calculate the unevenness amplitude as input for the empirical Barton–Bandis joint shear strength estimation (Barton & Bandis 1982).

Figure 7 presents the shear strength derived from both the experimental large-scale numerical direct shear environment and the empirical Barton–Bandis joint shear strength estimation. The results of the numerical direct shear environment provide an upper-bound estimate of joint shear strength, which is the subject of ongoing research and testing. The empirically derived shear strength values were used to simulate the shear strength of the intermediate structures.

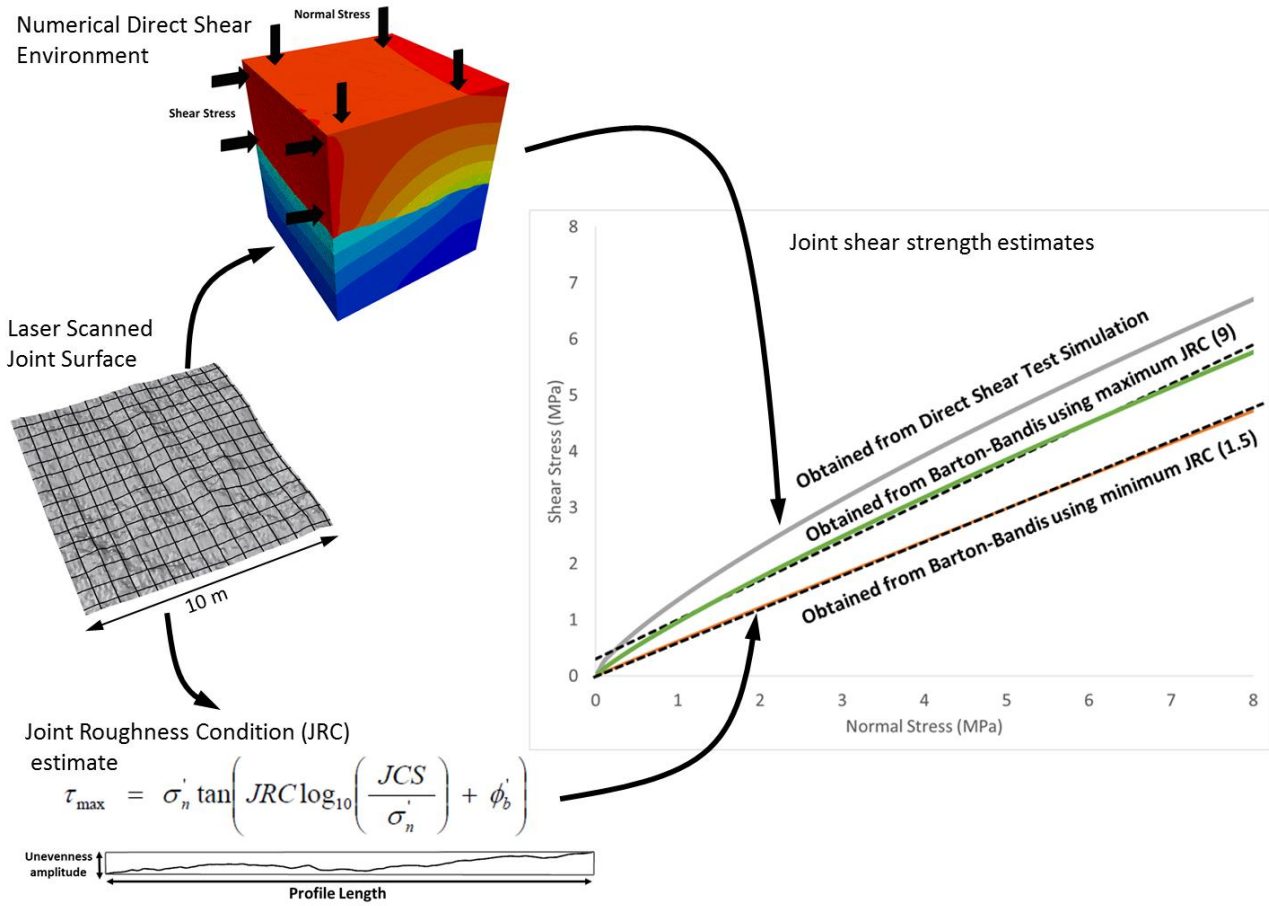


**Figure 7 Joint shear strength estimates used in analyses**

The 3DEC built-in Coulomb-slip contact model was used to represent the behaviour of the joints. The cohesion and friction angle were obtained from the best-fitting Mohr–Coulomb envelopes shown in Figure 7. A dilation angle of 5° was used for all joints. This dilation angle was obtained from the large-scale numerical direct shear testing. A Barton–Bandis representation has been used, and it was necessary to represent the shear strength using a Mohr–Coulomb model in the modelling because 3DEC does not include Barton–Bandis. The stress range was chosen to represent deeper failure, which was the key objective of the modelling.

## 3.4 Pore pressures

Groundwater has been assessed by HGO with a total of 26 groundwater investigation and monitoring wells within and adjacent to the site. Groundwater is contained within a low-yielding fractured rock aquifer. The phreatic surface elevation ranges from approximately 1,200 m RL in the northwest of the site, to approximately 1,130 m RL in the southeast. For modelling purposes, a phreatic surface was assumed to be 20 m behind the pit walls.

Considering the potential uncertainty associated with the position of the phreatic surface with respect to the open pit wall, an additional model run was completed using an alternative phreatic surface location for the final stage of the model. In this model the phreatic surface was assumed to be closer to the pit wall at deeper elevations as it approaches the toe of the pit. Apart from slight increase in slope deformation and localised bench-scale instability, no major difference was evident between the two models, suggesting that the pore pressure profile has little impact on the overall stability of the pit walls.

## 3.5 In situ stresses

There is no site specific in situ stress data available for the Kanmantoo Mine. Based upon a summary of principal stress trends for the Gawler-Curnamona stress province in southeastern South Australia provided by Lee et al. (2010), it was assumed that the vertical in situ stress is lithostatic (based on the weight of the rock above) and the major principal stress is oriented west-northwest–east-southeast with a magnitude 1.8 times the vertical stress.

## 3.6 Model geometry

Figure 8 illustrates the 3DEC model constructed to simulate the proposed Giant Pit design. The distance to the model boundaries was chosen based on previous experience of pit modelling, noting that only the Giant Pit stability is the subject of investigation. Roller side boundaries were used, and no boundary effects were evident in the models.

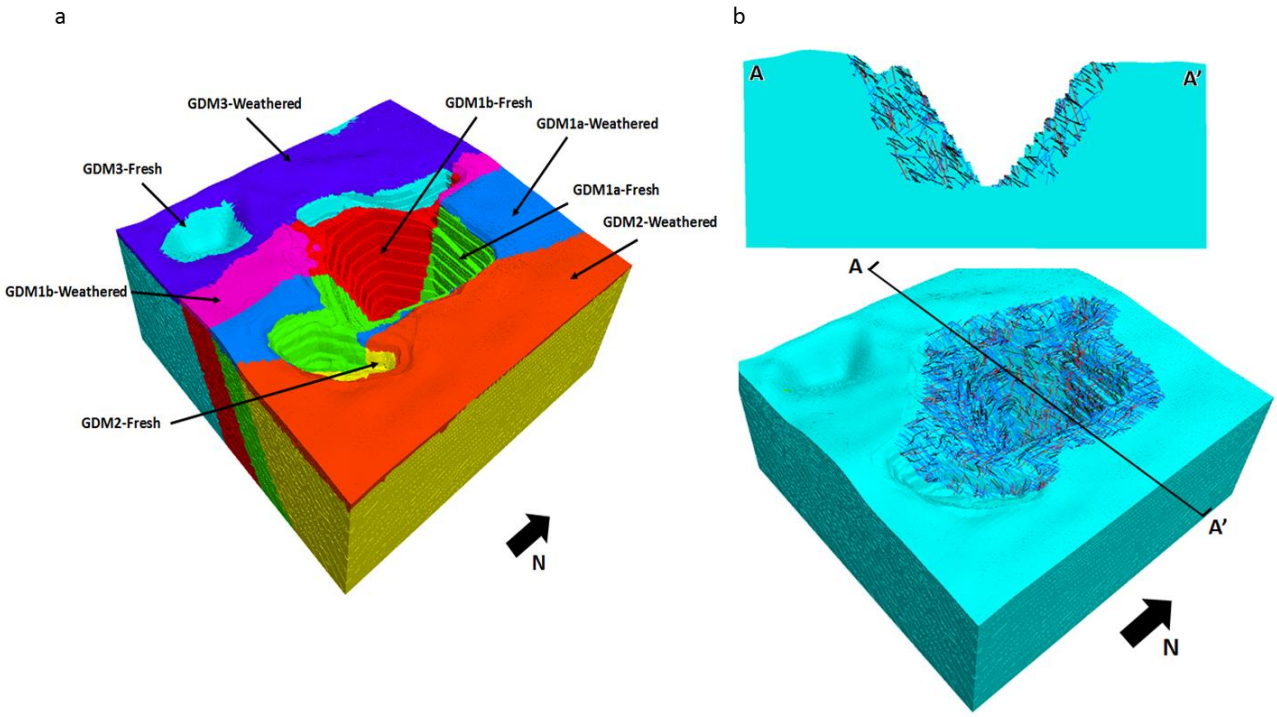


**Figure 8 (a) Rock mass domains simulated within model; (b) explicit joints included within the model**

To accurately simulate the stress path of the final pit slope, the model was excavated in multiple stages that reflect a simplified excavation sequence of the current Kavanagh, Spitfire and Nugent Pits, followed by excavation of the Giant Pit in 20 m horizontal benches.

The geotechnical domains incorporated in the models were based on actual lithological boundaries supplied by HGO. Figure 8(a) illustrates the geotechnical domains simulated within the 3DEC model.

Figure 8(b) illustrates the distribution of the discrete structures simulated in the model based upon the DFN described in Section 2.3. The discrete structures are intended to simulate the actual in situ joint spacing within 20 m of the pit wall and a coarser joint spacing with 20–100 m of the pit face.

# 4 Modelling outcomes

## 4.1 Assessment of stability

The performance of the final pit surface is presented with a simple displacement criterion. Areas within the model with a total displacement greater than 1 m (light grey areas) and with a velocity of 1e-5 metres per modelling step (light grey areas) are assumed to be in a state of active failure. This criterion was adopted as a result of several numerical back-analysis at other mine sites.

Figure 9 illustrates the total displacement and velocity predicted within the final Giant Pit. The final overall pit slope is predicted to remain stable. However, localised bench-scale instability is predicted primarily within the final northwest and east walls. Note that the illustrated total displacement reflect the displacement induced by the Giant Pit excavation only.

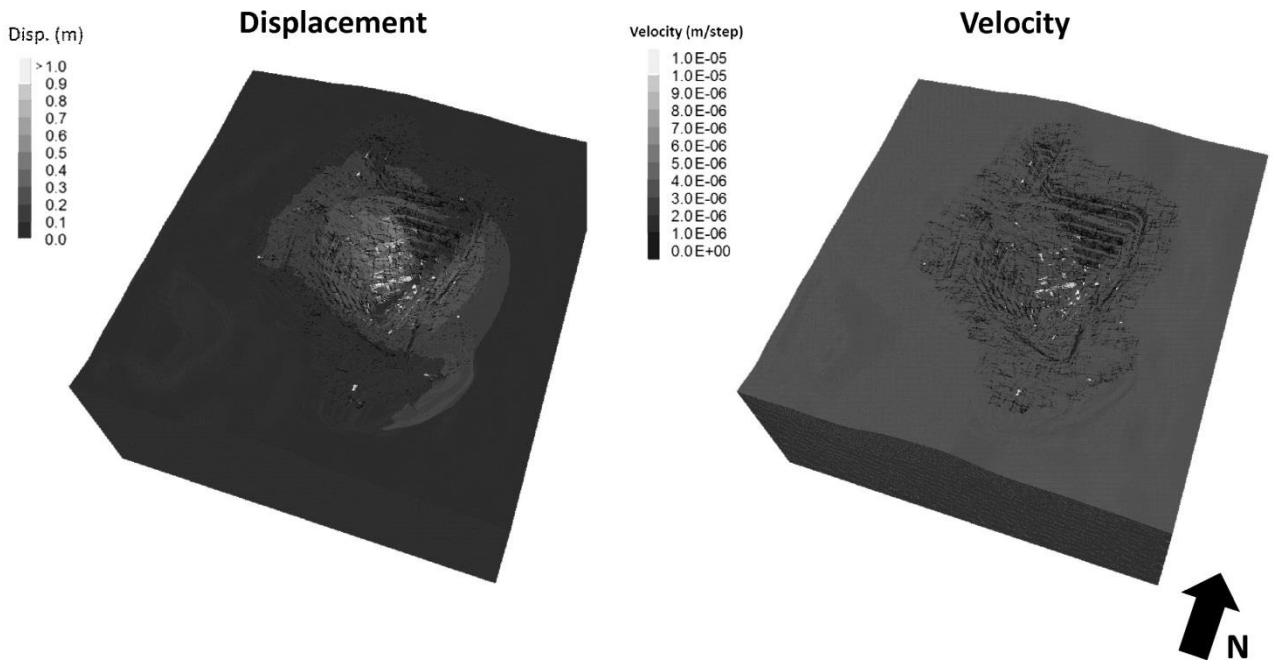


**Figure 9 Simulated pit slope total displacement and velocity**

The total displacement within the final northwest and east walls is illustrated in Figure 10. The modelling has shown that no large-scale instabilities or stress-related concerns are predicted. Within the northwest wall, bench-scale instability is predicted to manifest as wedge-type failures associated with the J1, J2 and F1 structures. Within the east wall, bench-scale instability is predicted to manifest as crest loss associated with the near vertical batter faces and F1 structures. The number and extent of the small-scale failures are realistic when compared with observed pit conditions. This can be seen by comparing the model results with the northwest wall of the Kavanagh Pit in shown in Figure 11. Lighter grey shades are areas of greater movement, and failures are shown in white.

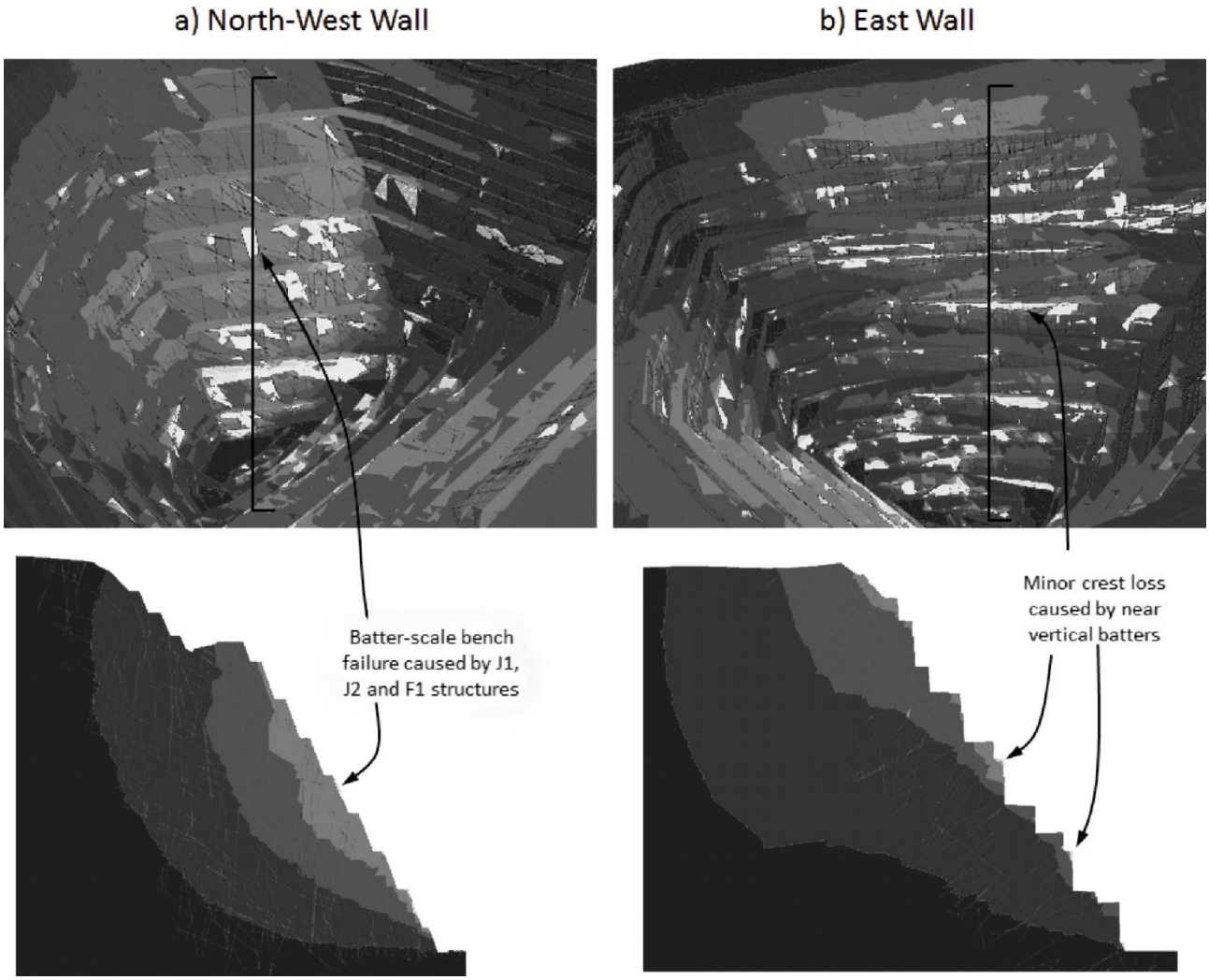


**Figure 10 Pit slope displacement within the northwest and east walls of the Giant Pit**


**Figure 11 Northwest wall of Kavanagh Pit showing crest losses on J1 joints**

## 4.2 Assessment of slope Factor of Safety

In order to assess the Factor of Safety (FS) of the final Giant Pit against overall slope failure, shear strength reduction (SSR) analyses were conducted. In this approach, shear strengths of rock mass and structures were divided by incrementally increasing factors until failure occurs, as indicated by displacement and velocity criteria described in Section 4.1. The FS is greater than the highest SSR at which the slope is stable, and is less than the lowest SSR that results in failure. FS is contoured on pit walls and cross-sections for cases where smaller failures have lower FS, and larger failures have higher FS.

Figure 12 illustrates the areas of instability predicted after SSR reductions of 1.0, 1.3, 1.45 and 1.6. The lightest grey contour indicates areas with a FS less than 1.0. The successively darker grey contours indicate additional areas that become unstable at a FS of 1.3, 1.45, and 1.6, respectively. The darkest grey regions indicate areas that remain stable at a FS of 1.6.

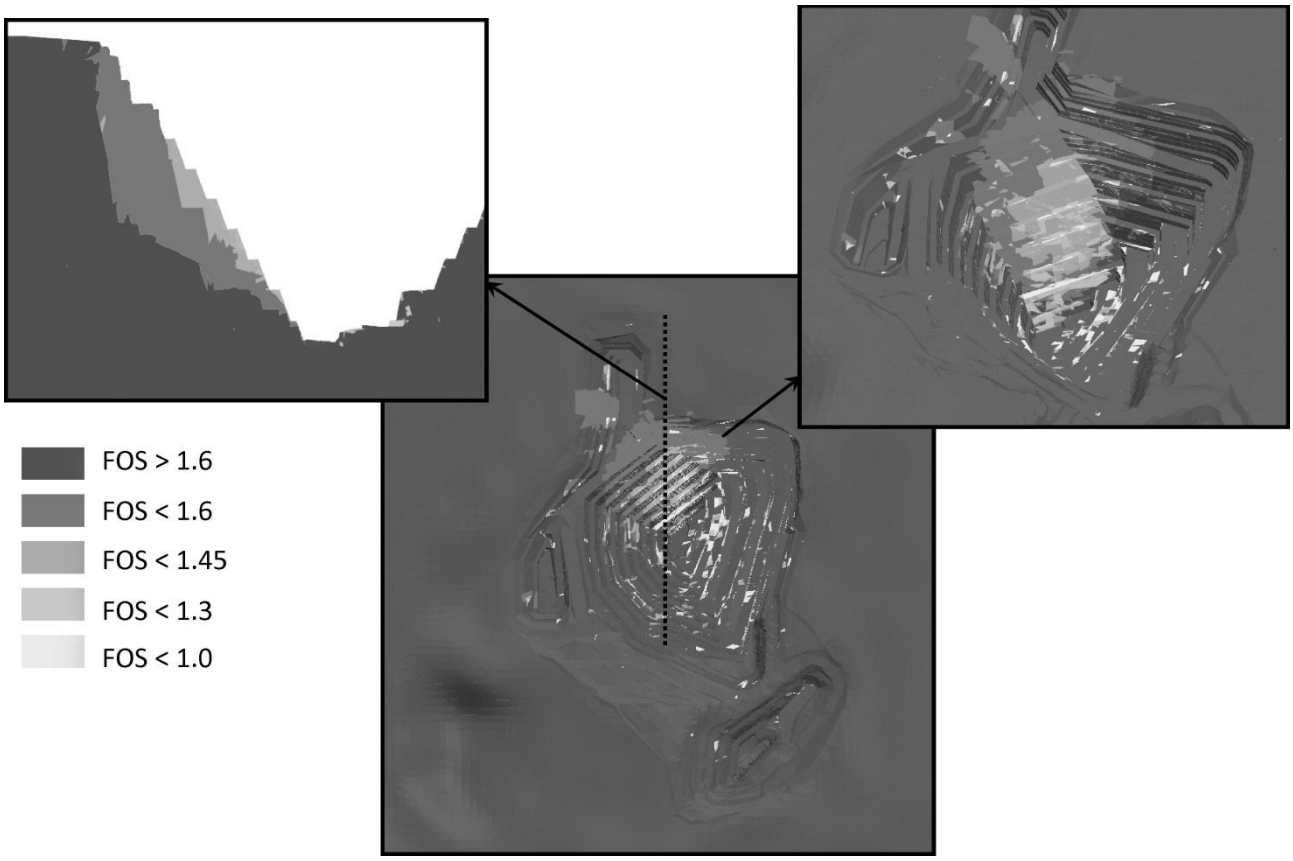


**Figure 12 Factor of Safety contours for final pit slope**

# 5 Conclusion

Three-dimensional numerical analyses have been conducted with the 3DEC program to investigate the stability of the proposed Giant Pit at the Kanmantoo Copper Mine. The proposed Giant Pit is an aggressive design that is proposed to mine significantly steeper than the existing Kavanagh Pit.

For all cases analysed, the overall final slopes of the Giant Pit are predicted to remain stable. The modelling also predicts no stress-related concerns.

Minor bench-scale instability is predicted primarily in the northwest and east walls. Within the northwest wall, bench-scale instability is predicted to manifest as wedge-type failures associated with the J2 and J1 structures, which corresponds well with observed conditions. Fewer failures are predicted on J1 structures in the west wall, which appears to be due to their random distribution in the model resulting in few structures intersecting crests in adverse locations. Observed west wall performance has indicated that

planar sliding failures on J1 structures are common in the Kavanagh Pit and will continue to be a risk in the west wall of Giant Pit. Shear pinning should continue to be done to address the wedge and planar crest failures to retain rockfall capacity on the northwest, west and possibly the southwest walls.

Within the east wall, the modelling predicts that bench-scale instability may manifest as crest loss associated with the F1 structures and the near vertical batter faces. This is considered to be an overly conservative prediction, as the F1 structures are distributed throughout the jointed region of the models but few F1 structures are seen in the east walls of the pit.

The number and extents of small-scale failures on the North-West Wall are realistic when compared with observed pit conditions, providing confidence in the modelled representation of the rock mass and structures. The East Wall has fewer F1 joint structures than in the model and the small scale failures in that region are therefore over represented in the model. The 3D models do not include shear pins to support crests against failure on the J1 joints, and are therefore more representative of the conditions seen in the Kavanagh Pit.

SSR indicates that the Giant Pit has a FS greater between 1.45 and 1.6 against large-scale multi-bench slope failure. Based on these results, and provided that anticipated minor bench-scale instabilities and crest losses can continue to be managed, the proposed design is expected to be achievable.

This project has demonstrated that discrete modelling of intermediate structures in a mine-scale model can be achieved using three-dimensional DEM techniques, resulting in realistic outcomes that represent the observed pit slope response to mining.

## Acknowledgement

The authors wish to thank the management of Hillgrove Resources Limited for permission to publish this paper.

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