Discrete Fracture Network Approach in Ground Support Design Optimisation for Large Span Cavern in Jointed Rock Mass

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ABSTRACT: Ground support design for rock tunnels or caverns often adopts precedents or empirical methods, which were mostly developed in 1960s and 1970s. These methods have undoubtedly contributed to completions of many projects; however, they are unique and closely related to local geology condition at the locations where the methods were developed. Discrete Fracture Network (DFN) model allows inclusion of discontinuities data from site-specific geotechnical investigation stage to be stochastically quantified and used explicitly as design input. Hence, this offers a more quantifiable, verifiable, and reproduceable method to assess rock mass quality & behaviour. Bias in rock mass characterisation and engineering judgement involved in design process therefore can be avoided. This paper demonstrates benefits gained from DFN approach in ground support design for large span rock caverns in urban area. The case study involves a few caverns constructed in jointed volcanic tuff at shallow depth and within a complex bifurcation scheme. Certainly, controlling ground settlement and maintaining ground stability are of importance in this situation. However, balance between maintaining public safety and ground support design should be achieved. The use of DFN and explicit modelling has allowed ground support to be optimized based on more realistic anticipated ground conditions. Coupled with 3D Distinct Element Method, the DFN approach also assisted in assessing pillar condition of the bifurcation area due to elevated stress more realistically rather than relying on conservative method. Overall, it contributed to successful and economical project delivery.

1. INTRODUCTION

Rock mass stability in underground opening is controlled by several factors, including discontinuity conditions & spacing (which control rock mass rating), intact rock strength, and *in situ* stress. These conditions lead to different failure modes; i.e., stress-induced failure, structurally induced failure, or combination of both; as illustrated by Kaiser et al. (2000). Stress-induced failure typically occurs in situations where *in situ* stress is relatively high compared with rock strength. Often, this situation is encountered at great depths. Structure-induced failure is governed by the presence of jointing as the weak element in rock mass where stress condition is relatively low compared with rock strength. This is often the case for shallow underground excavations in hard rock. (Hoek, 2007).

In structure-induced failure, rock wedges are often formed by the intersections of more than 2 structural features and failure occurs when the wedges are loosened and released due to the absence of confinement. In this situation, prediction of rock wedge size is critical for underground support design. Support design approach for this situation is normally undertaken by estimating rock mass pressure based on empirical approach, such as Terzaghi's (1946), Bienawski et al (2007), or Q-system (NGI, 2015). This approach may be suitable for tender or feasibility stage, however, detailed design will certainly require further consideration of rock and defects conditions. Empirical approaches have been mainly derived based on encountered rock mass and tunnelling condition at given sites and ground conditions. Therefore, it may not consider actual discontinuity driven failure conditions. Another setback is that determination of rock mass quality during characterization process is often influenced by subjectivity, bias, and judgement, and one's experience. Therefore, this process becomes convoluted and is often difficult to be verified. Figure 1 below show a real example where logging results are not representative of actual rock conditions. The cores are logged with considerably low RQD values (between 0 and 43), representing heavily fractured rocks, which is not the case. Certainly, this may cause significant commercial implications to the project.

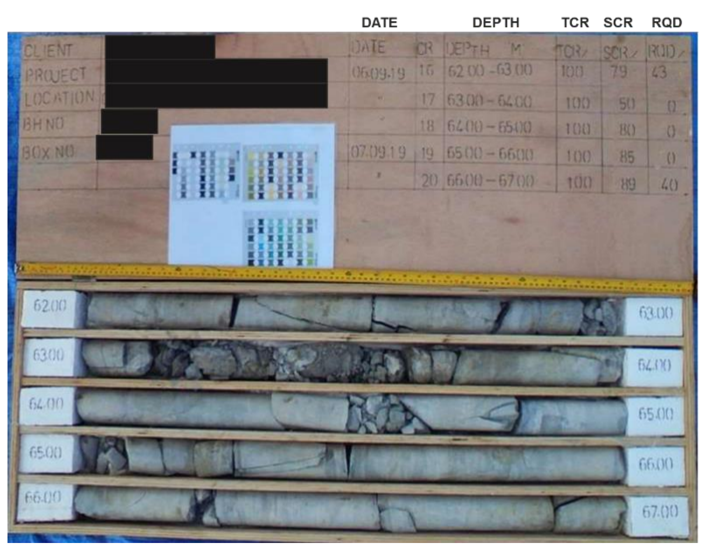

Fig. 1. Example of Incorrect Rock Logging Results.

The other alternative adopted in the industry is analysis based on key-block theory (Goodman & Shi, 1985), which has been implemented in commercially available algorithm such as UnWedge (Carvalho et al., 1991). This method assumes ubiquitous and infinitely long fractures, and as a result, the prediction estimates the largest key block potentially formed around excavation surfaces. While this approach enables engineers to consider project-specific conditions and is considered safe and representative of block instability, many argue that the conservatism can be challenged. The main issue is that it does not allow considerations of natural rock conditions, including complex block geometries, spatially distributed discontinuities, and influence of rock bridges.

Discrete Fracture Network (DFN) approach provides an alternative method that is able to overcome some setbacks described above. DFN model is built upon a set of quantifiable rock mass descriptors; including orientation, trace length, and intensity, to represent equivalent rock mass condition in statistical ways. DFN approach has the capability to provide a clear and reproducible route from site investigation data to modelling because real fracture properties and its heterogeneity nature are preserved through the modelling process (Elmo et al., 2014). Therefore, this approach offers a verifiable process and streamlines convoluted workflow due to subjectivity in engineering judgement typically involved in rock mass characterization process. Some examples of the use of DFN in rock engineering projects have been published and presented, e.g. for fragmentation assessment for block caving (Rogers et al, 2010), rock mass characterization for rock pillar (Elmo et al., 2014), and tunnelling applications (Rogers et al, 2006; Grenon et al, 2015). Since DFN is a stochastically developed model, therefore, it can be further adopted to also predict probability of wedge formation and its likelihood, based on several equiprobable scenarios. This feature provides a robust design method to determine an optimized solutions for rock tunnel support design.

This paper focuses on the application of DFN in optimizing ground support design for large span caverns in jointed rock mass. Several benefits are shown, including material saving and reducing conservatisms in rock mass and rock pillar stability. For the purpose of this paper; fractures, discontinuities, and joints are used interchangeably and they refer to the same meaning.

2. THE PROJECT

2.1. General

The case study used in this paper is based on a completed tunnels project as described by Lagger et al. (2014) and Lagger et al. (2017). The project was completed as part of an expansion of transport network in Queensland, Australia. It required excavation of a few large span caverns and tunnels in jointed volcanic tuff in urban settings.

Figure 2 below shows a bifurcation area as one of the critical locations of the project. The alignment of the project was generally oriented in North-South direction. The Northbound (NB) and Southbound (SB) Caverns spanned between 22 and 26 m wide, with excavated height between 12 and 13 m. The NB Cavern was connected to 2 smaller tunnels, i.e. Tunnel B and Tunnel C (6.5 m wide, and 9.0 m high). These tunnels were separated by 1.0 m wide rock pillar. Another tunnel, Tunnel A, was excavated above NB Cavern. The invert level of Tunnel A was located approximately 7 m above the crown of NB Cavern. These underground openings were excavated in urban areas and at shallow depth. As a reference, NB Cavern was located only 30 m below ground surface. The excavation was carried out using road headers. Other details are withheld due to confidentiality.

2.2. Geological Condition

The site was dominated by jointed volcanic tuff. During the design stage, rock mass classification system had been devised specifically for the project based on the general geological setting, degree of weathering, material strength, and discontinuity conditions. Based on the rock mass characterization results, the caverns and tunnels were expected to be excavated in rock mass with following properties:

- GSI between 50 and 95;
- UCS between 50 and 65 MPa and Intact stiffness between 12 and 22 GPa.

Following in situ stress conditions have been concluded based on several hydraulic fracturing tests, viz:

- Major Principal Stress (σ_1) = $\sigma_{H,NW-SE}$ = σ_v to $2.0\sigma_v$;
- Intermediate Principal Stress (σ_2) = $\sigma_{h,SW-NE}$ = $0.8\sigma_v$ to $1.0\sigma_v$; and
- Minor Principal Stress (σ_3) = σ_v.

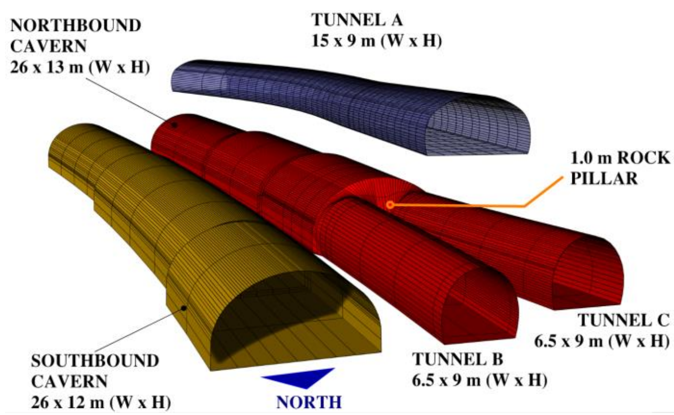

Fig. 2. Southwest View of Caverns and Tunnels.

2.3. Designed Ground Support and Excavation Sequence

To facilitate the construction program, the excavation was designed to be in in central heading-side drifts-and benches fashion, as shown in Figure 3. The maximum excavation advances for the headings and benches were 1.5 m and 4.5 m, respectively. Primary support consisted of shotcrete and rock bolts. It was designed to be installed as temporary support during excavation. Secondary support in form of concrete lining was installed afterwards for permanent support (100 years design life). Primary support was expected to degrade, and its effect would diminish over time.

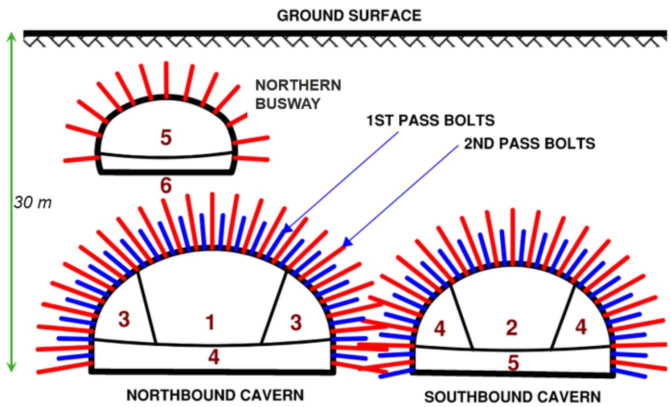

Fig. 3. Proposed Ground Support & Excavation Sequence.

3. DISCRETE FRACTURE NETWORK MODELLING

Instabilities were expected to be controlled by discontinuities in the tuff. Based on the review of existing boreholes and televiewer data, five (5) joint sets had been identified and adopted to establish the DFN model, using FracMan code (Derhowitz et al., 1995; Golder, 2020). FracMan has been successfully implemented in different applications; from fluid flow in rocks, fragmentation modelling for natural rock mass and block caving, to tunnelling (Rogers et al. 2006, Elmo et al. 2010, Elmo et al. 2014, Derhowitz et al. 2017, La Touche & Cottrell, 2017). It has capability in both implicit fragmentation grid algorithm and conventional explicit block search algorithm. The implicit algorithm is useful to investigate the rock mass properties based on underlying DFN, e.g. rock mass quality (GSI), rock mass stiffness. For the project, explicit algorithm has been mainly adopted in the assessment as it was considered more relevant. This process involves generation of realistic wedges defined by intersection of multiple discontinuities for a given excavation surface. The kinematic stability analysis in FracMan adopts similar solution procedure to another key block analysis such as UnWedge.

To establish a realistic DFN model and true discontinuum model for the project, careful consideration and analyses on (1) distribution of fracture size (radius), (2) distribution of fracture orientation, and (3) fracture intensity are required. For distribution of fracture orientation, analyses using available Acoustic Televiewer (ATV) data were carried out for different joint sets. The analyses for each set were then continued to derive fracture size distribution and fracture intensity. In this process, data from the ATV and outcrop mapping were adopted and DFN conditioning was undertaken in determining fracture radius and intensity, until a range of simulated trace length and linear fracture frequency (P10) represented the observed mean values. Depending on geological conditions, fractures can be modelled either as circular or elliptical discs, if it is considered representative. Considering site specific condition, fractures in this model has been assumed as series of circular discs with certain diameter to represent fracture size. The distribution of the fracture size was assumed to follow power law distribution and this method has been widely accepted as representative way to estimate fractures in nature (Priest & Hudson, 1981).

The orientations of all fractures generated in the DFN are shown in a stereoplot in Figure 4. Table 1 and Figure 5 summarize the adopted parameters adopted in the model for each set. Figure 6 below presents the modelled DFN from one realization and a Synthetic Rock Mass (SRM) model.

A comparison was carried out between manual rock mass characterization results and rock mass quality analyzed using the underlying DFN and the implicit algorithm in FracMan. Adopting implicit algorithm, the estimated GSI of the rock mass was 50, which indicates good agreement with manual prediction range as discussed in Section 2.1 above. This result demonstrates that DFN is a suitable rock mass characterization tool. As described above, all parameters adopted in the process are objectively traceable, therefore, we can minimize influence of bias and subjectivity and minimize risks during design process. The established DFN model can also be used to estimate encountered rock face conditions for verification and comparison with mapping results as shown by modelled traces in Figure 7.

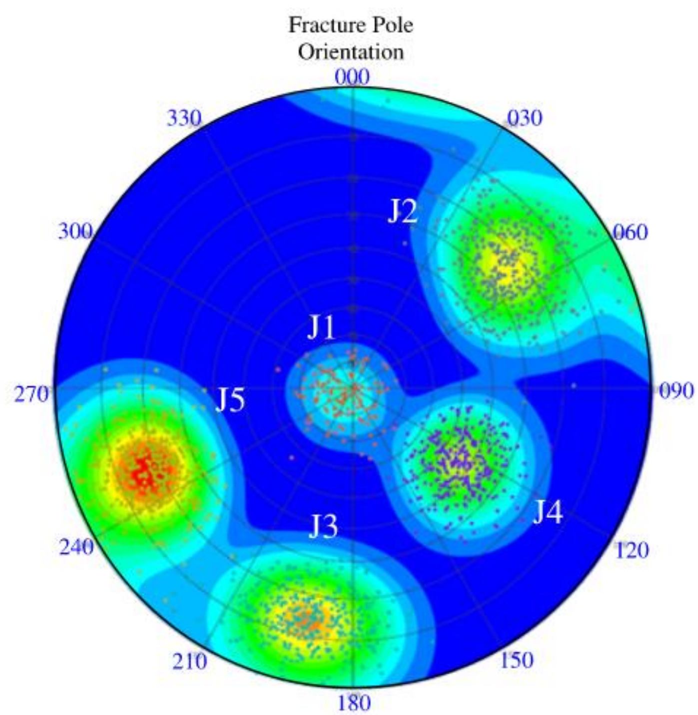

Fig. 4. Stereoplot of the Generated DFN.

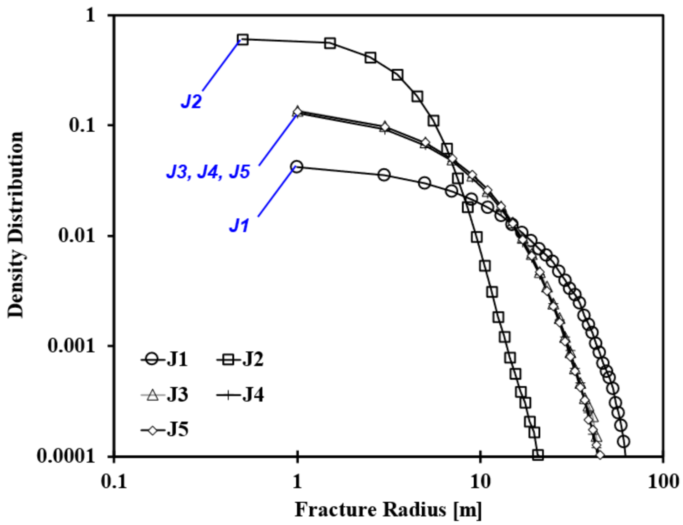

Fig. 5 Adopted Fracture Radius in the DFN model.

Table 1. DFN parameters

Joint Set	Orientation Distribution	Dispersion K	Mean P32 [m^{-1}]
1	Fisher	40	0.295
2	Bivariate Normal	9	0.318
3	Bivariate Normal	6	0.319
4	Fisher	45	0.300
5	Fisher	50	0.316

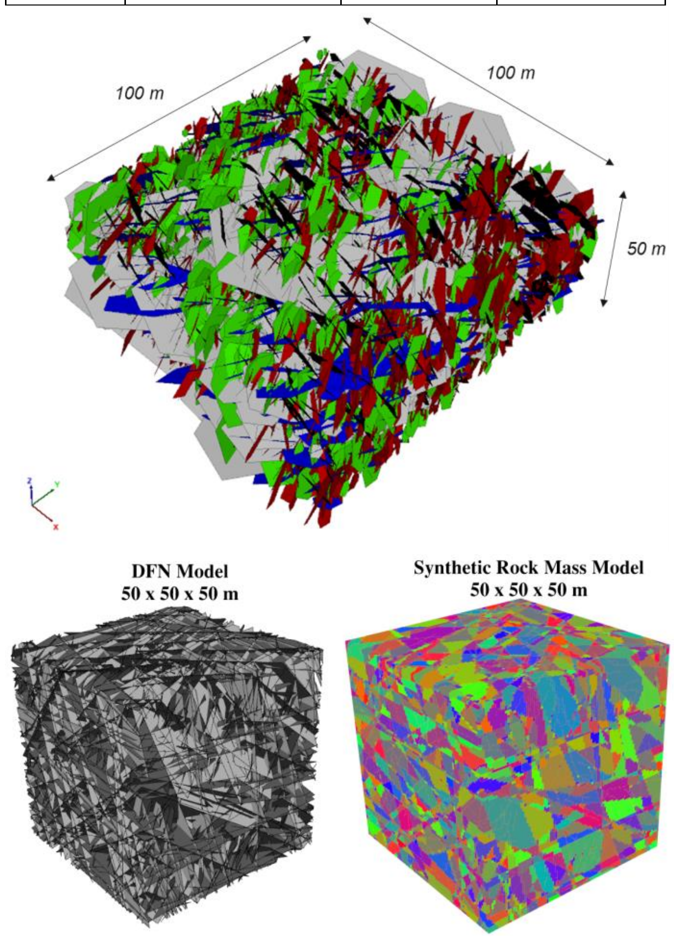

Fig. 6. Generated DFN for the Project (Above), and Synthetic Rock Mass for 50 x 50 x 50 m Rock Mass for Illustration (Lower).

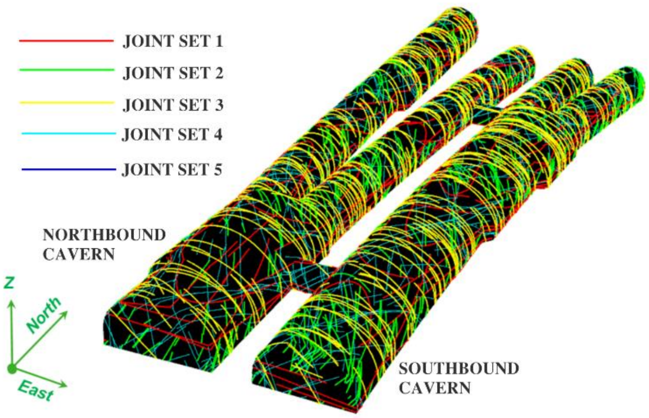

Fig. 7. Estimated Traces of the Underlying DFN on the Excavation Surfaces.

4. GROUND SUPPORT DESIGN OPTIMISATION

The design process involved several critical components for temporary and permanent conditions. For temporary condition, interactions between caverns and tunnel as well as pillar stability were of concern. The utmost importance during excavation was to ensure stability as such that there was no disturbance or damages to public safety and facilities. For permanent condition, there was a drive for optimization in the thickness of permanent lining. Based on initial assessment and precedents, optimization was deemed possible. For these two conditions, the inherent capabilities in DFN and SRM were adopted as alternative design solution to traditional methods.

4.1. Permanent Lining Design Optimization

As explained in Section 2.3 above, permanent concrete lining as secondary support was designed to be installed after excavation was completed and it would need to bear all rock loads in long-term condition without contribution from temporary shotcrete or rock bolts. Typically, lining designer would require information on the rock loads in form of support pressure and the determination of this pressure is largely dependent on the assumptions of the ground behavior. Some comparisons between different methods are therefore presented herein.

Considering the nature of the rock mass in the Project, the main source of the load was expected from wedge load due to kinematic failure. Based on the interpreted joint orientations (Figure 4) and review on the direct shear test results, Figure 8 presents the details of the most critical wedge predicted by UnWedge based on key-block theory, which posed the greatest effect to the lining design. The result indicates that a massive wedge potentially would form at the crown with total volume of 565 m^3 and apex height of 13.4 m. In order to stabilize such a large wedge, minimum support pressure approximately 120 kPa (unfactored) was required.

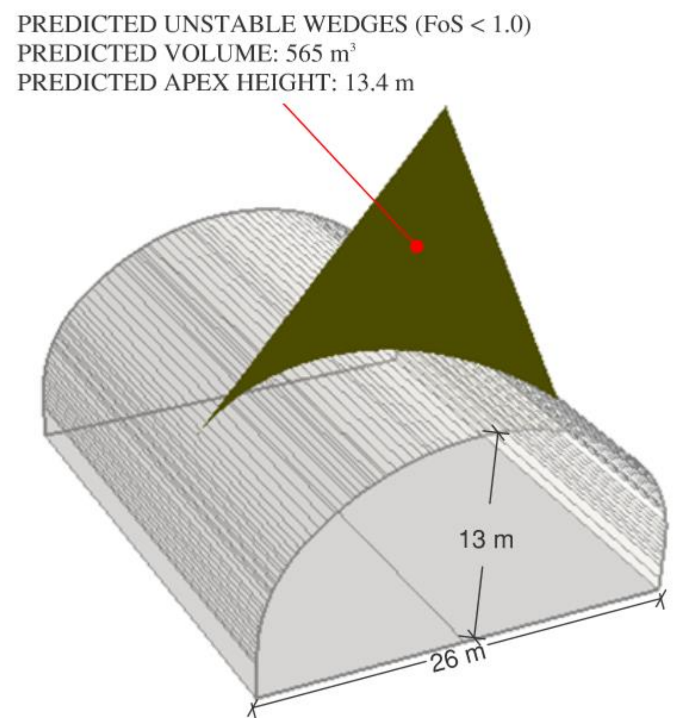

Fig. 8. Rock Wedges Predicted by UnWedge with No Clamping and Friction Angle of 35^0

Such a massive wedge would typically be predicted as a result of fracture ubiquity as the underlying assumption in UnWedge. Size of rock wedges are certainly dependent on the orientation, spacing, and location of fractures. By considering the heterogeneity nature of fractures and its spatial distribution, prediction of unstable rock wedges can be carried out more realistically. For this purpose, kinematic analyses with FracMan adopting the generated DFN were carried out. It should be noted that kinematic stability analysis in FracMan adopts similar procedure to UnWedge. The main difference is in the block search algorithm. FracMan identifies all potential wedges defined by underlying DFN model and excavation surfaces. Wedges are constructed by identifying discontinuities which form two-dimensional blocks in the trace map. This process is then replicated to generate polyhedrons which connects to the excavation surfaces. The wedge volume is calculated using three-dimensional tessellation process considering the unit weight of the rock.

All potential wedges around the cavern (in this case NB cavern), including both unstable (FoS $\leq$ 1.0) and stable wedges (FoS > 1.0) predicted based on the underlying DFN model (5 joint sets) is presented in Figure 9. Compared with Figure 7, the wedges shape and orientation are governed by spatial distributions, fracture size, and heterogeneity of the fractures. To understand the differences with UnWedge result, cumulative distribution functions of the wedge volumes are plotted in Figure 10. Of all the unstable wedges at NB and SB Caverns, data indicate that 96% of the unstable wedge volume would be smaller than 5 m^3, and the indicated median value is approximately 0.01 m^3. The identified maximum size of unstable wedge is 54 m^3 with maximum apex height of 5.7 m, as shown in Figure 11. It should be noted that this size represents only less than 1% of the data and the maximum volume is only less than 10% of that predicted by UnWedge (Figure 8).

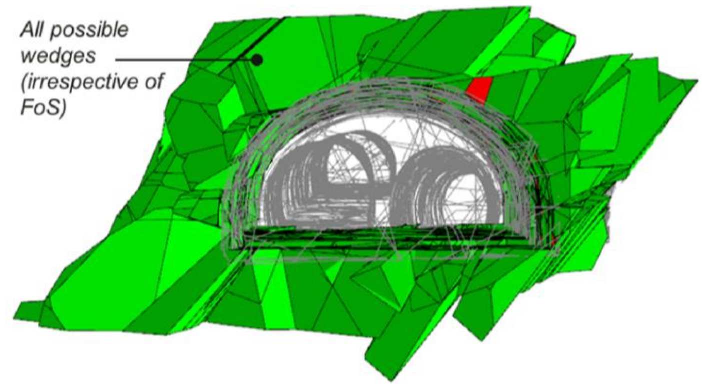

Fig. 9. View Towards Northbound Cavern with Predicted Rock Wedges by FracMan.

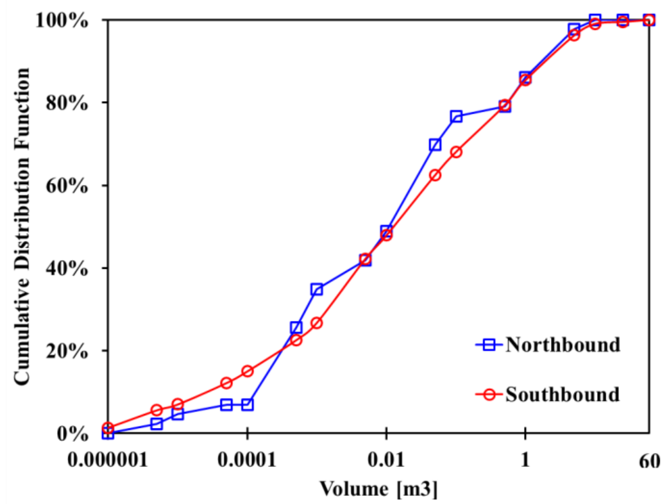

Fig. 10. Probability Function of Wedge Volumes with FoS < 1.0 for Northbound and Southbound Caverns.

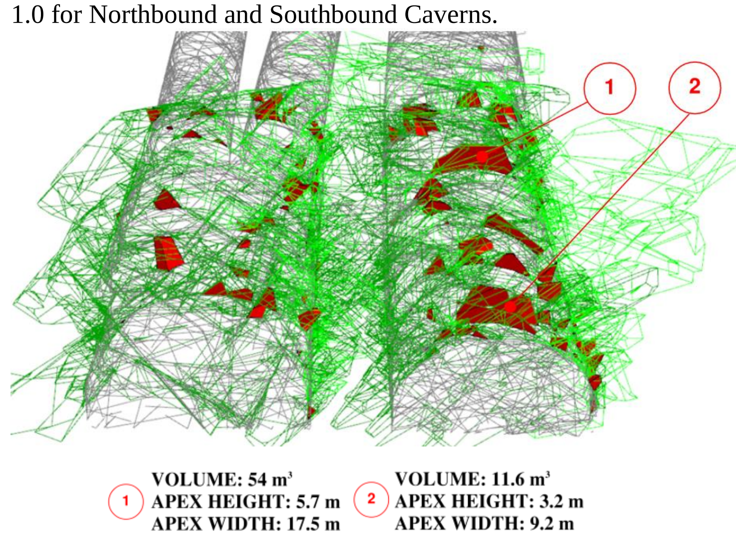

Fig. 11. Predicted Critical Rock Wedges by FracMan.

The kinematic stability analysis results above demonstrates that DFN approach honors the variability in fracture orientations, spacing, and intensity in predicting wedge size. As pointed out by Rogers et al. (2017), typical key block analysis does not consider more complex block geometries, variability in fracture properties and the influence of rock bridges. As such, it only provides a possibility instead of probability. As shown above, the analysis results demonstrate that a true probabilistic analysis has been done. This affords a verifiable process and certainly minimizes bias.

As highlighted above, support pressure is typically an input required by structural designer for permanent lining design. Hence the determination of the pressure becomes critical. Based on the DFN approach, assuming that the maximum wedge size (54 m^3) is adopted, the required support pressure is 38 – 81 kPa or only 32% - 68% of the predicted value by UnWedge (120 kPa). The difference widens if this is compared with value derived from empirical method. Assuming that Q-system is adopted, the estimated support pressure is approximately 74 – 147 kPa is required, assuming Q-value of ~ 2.5 (for GSI = 50), 5 joint sets, and Jr (Joint Roughness Number) of 1.0 to 2.0. Compared to the empirical approach, support pressure predicted by DFN approach is only 51% - 54%.

As lining thickness increases linearly with support pressure, one can deduce that implementation of DFN approach can lead to significant savings in material and cost, and projects can cut carbon footprint significantly and be completed in a more sustainable manner. From this example, it is clear that implementation of DFN has the potential to lead an improved design approach which gives more realistic prediction of rock mass condition and potentially better outcome. Certainly, this should be accompanied by adequate FoS, site verification of rock mass condition, and quality control & monitoring during construction.

4.2. Stability during Excavation – Assessment of Temporary Support and Pillar Stability

Details of challenges during excavation and temporary support design are explained by Lagger et al (2014) and Lagger et al. (2017). Figure 3 illustrates the excavation sequence and the designed temporary ground support system. For the Northbound Cavern, Southbound Cavern, and Tunnel A; the temporary ground support generally consisted of:

- Shotcrete thickness of 75 mm all around;
- 3.7-m long 1st pass bolts installed during first opening; and
- 6.0-m long 2nd pass bolts installed during subsequent excavation.

For Tunnel B and C, only 3.7-m long bolts and similar shotcrete thickness of 75 mm was designed for temporary support. All bolts were designed to be 26.5 mm dia. fully grouted & mechanically anchored bolts, with yield strength of 500 MPa, grout strength of 40 MPa, and installation spacing of 1.75 m.

Ground movement, risk of ground instability and impact to the existing facilities and infrastructures should be minimized and mitigated during construction. Interaction between caverns and tunnels, pillar stability, and adequacy of the temporary supports and excavation sequence were of the main concerns. Lagger et al. (2017) described that these challenges around pillar area were addressed using Boundary Element Method (Examine3D), 2D Finite Element Method (Phase^2), and 2D Distinct Element Method (UDEC), and 3D Finite Difference Method (FLAC3D). This paper presents more refined assessment using full-fledged 3D Distinct Element Model (3DEC) coupled with the established DFN model. This aims to consider more realistic influence of rock joints to the stability, e.g. influence of joint orientations, spatial distribution, and persistence in 3D space. The results were then compared with FLAC3D results. The FLAC3D model was also built based rock mass quality predicted by implicit method in FracMan.

As first assessment, 3D stochastic wedge stability analysis for temporary support was carried out based on established FracMan analysis for permanent lining by incorporating the temporary rock support features, including bolt and shotcrete (Figure 12). In FracMan, the action of the bolt and shotcrete is identical to that implemented in UnWedge. Support pressure from bolts and shotcrete are considered in the analysis, as such, the stability of individual wedge will increase and number of unstable wedges declines, as indicated in Figure 13.

The results of this initial analysis provided an early check and confidence in the capacity of proposed ground support. From here, probability of wedge failure, rather than a single possibility predicted by key-block theory, can be obtained considering that the underlying DFN was adopted.

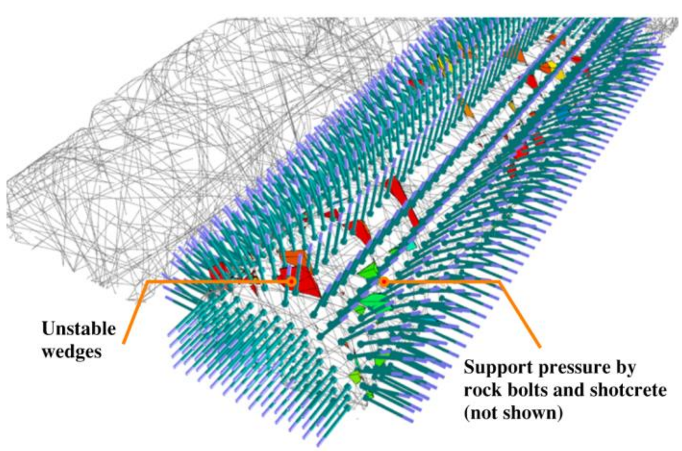

Fig. 12. Adopted Rock Support Model in FracMan (Red wedges represent unstable element).

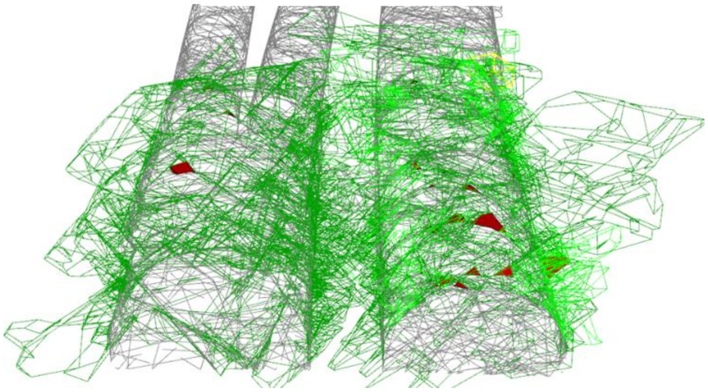

Fig. 13. Diminishing Number of Unstable Wedges due to Support Pressure Provided by Bolts and Shotcrete

Figure 14 below presents the Synthetic Rock Mass model adopted in the 3D DEM (3DEC) for further stability assessment. Both 3DEC and FLAC3D analyses followed the excavation sequence shown in Figure 3. Adopted parameters for both FDM and DEM are presented in Table 2 and 3 below.

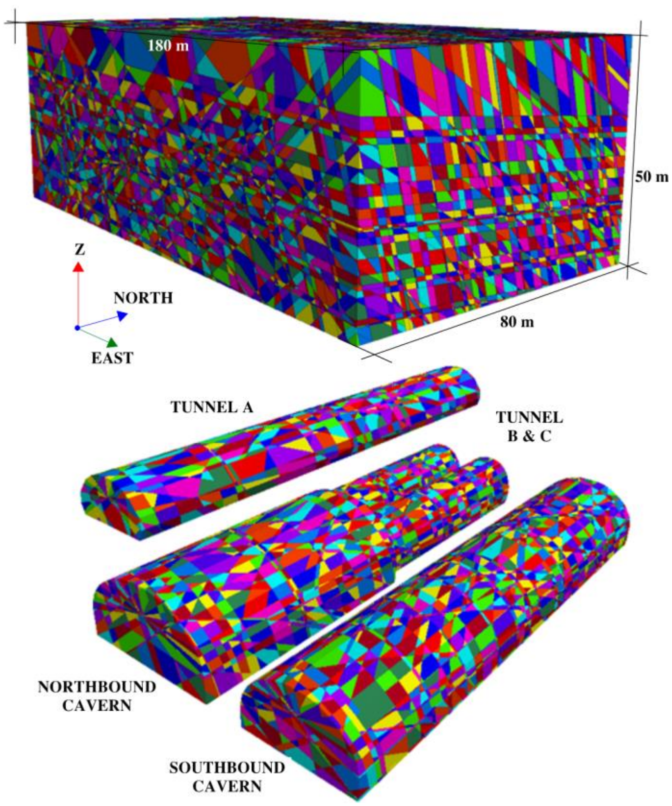

Fig. 14. Synthetic Rock Mass for the Caverns and Tunnels

Continuum analysis result by FLAC3D indicates fairly high risk of instability in the pillar area, where the NB Cavern and Tunnels B & C bifurcate. This can be observed by referring to the contour of strength / stress ratio in Figure 15. Ratio of nearly 1.0 has been identified and typically this number reflects potential for failure in rock mass. As the excavation progresses, deviatoric stress will accumulate and the pillar is easily subject to elevated stress due to the effect of multiple openings. Failure of the pillar is certainly undesirable as it will lead to catastrophic consequence, including damage at the surface level.

Table 2. Rock Mass Properties Parameters

Parameters	Continuum Model (Mass Scale)	Discontinuum Model ($1m^3$ Block Scale)
UCS	50 MPa	50 MPa
GSI	50	75
m_i	10	10
m_b	1.677	4.095
s	0.004	0.062
a	0.506	0.501
$E_{rock\ mass}$	4.6 GPa	12.5 GPa
υ	0.3	0.3

Table 3. Discontinuities Shear Strength

Parameters	Lower Bound	Upper Bound
Cohesion	0 MPa	0 MPa
φ	35 deg	40 deg
Shear stiffness	1,000 MPa/m	2,000 MPa/m
Normal stiffness	10,000 MPa/m	20,000 MPa/m

Continuum model might serve well in modelling shear failure of heavily jointed rock mass condition which is dominated by block rotation (typically GSI < 35). This is certainly not the case where instability is driven by block movement in more sparsely jointed rock, and in this situation, coupled DFN-DEM analysis was considered as better approach to include fractures heterogeneity. When excavation is carried out, elevated stress in each individual block level is inevitable, and in this situation, rock blocks will possibly move and dilation in the interface will interlock the blocks. This may have some positive effect to the stability of rock blocks.

This is demonstrated by the 3DEC results in Figure 16. As observed, there is no significant crushing or excessive movement observed in pillar, even when lower bound parameters were adopted. Based on the pattern, majority of the movements is controlled by individual blocks around the haunches or crown area around the pillar. Mobilized forces of rock bolts installed are also mostly below 100 kN (Figure 16), this is significantly smaller than capacity of the designed bolts (yield capacity of 240 kN for 26.5 mm dia. & steel grade of 600 MPa).

The predicted maximum ground movement was 15 mm at the ground surface level (Figure 18), while during the actual works, the monitored settlement was less. No instability in the bifurcation area was observed.

Based on the above analyses, the use of DFN and 3D DEM as alternative and more realistic methods, has the potential to improve our understanding of jointed rock behaviour. Furthermore, using this approach may enable engineers to increase stakeholders' confidence and reduce conservatism in design process of a project.

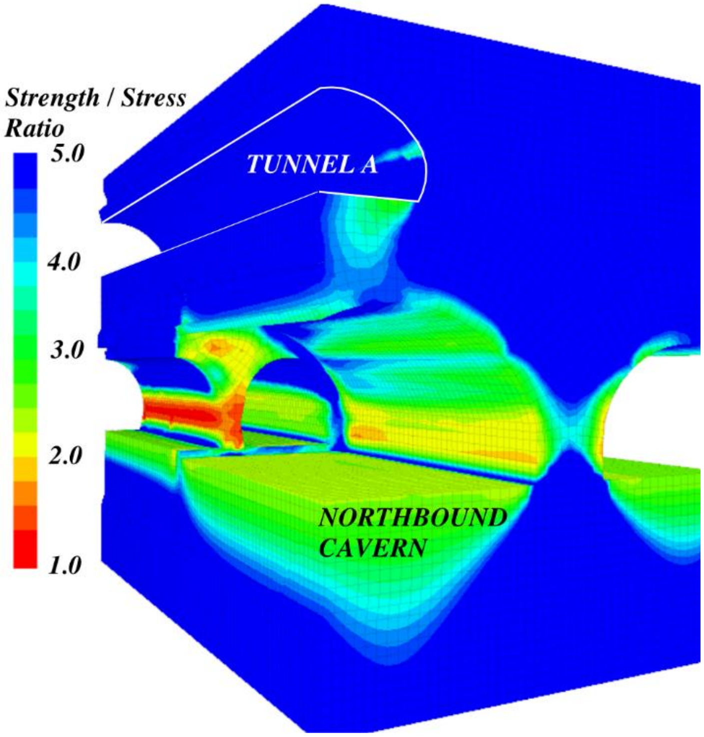

Fig. 15. Calculated Stress / Strength Ratio of the Rock Mass.

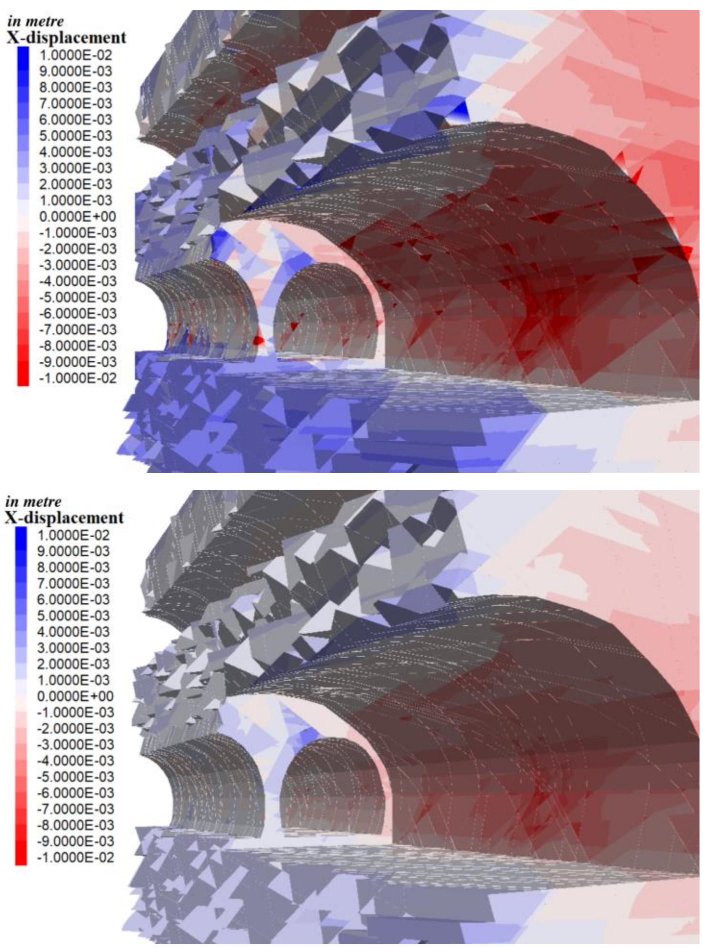

Fig. 16. Predicted Lateral Displacements Using Lower Bound Discontinuities Shear Strength (Upper Figure), and Upper Bound Discontinuities Shear Strength (Lower Figure).

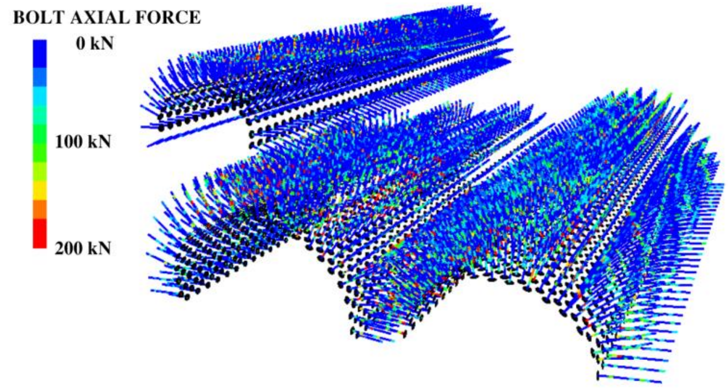

Fig. 17. Bolts Axial Forces – Lower Bound Model

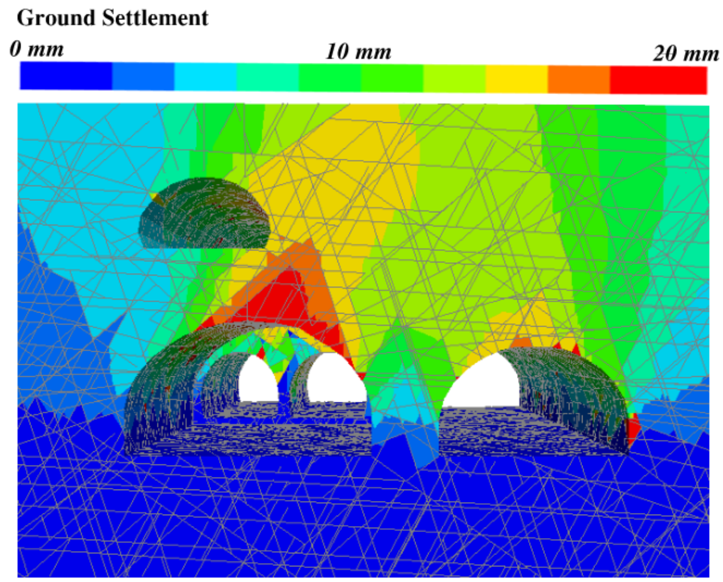

Fig. 18. Predicted ground settlement by 3DEC – Lower Bound Model

5. CONCLUSION

DFN approach has been implemented in decades for many engineering purposes, however its implementation in civil underground projects, is still uncommon. The above study shows several benefits reaped from DFN approach, from rock mass modelling to ground support design.

DFN has the capability in modelling rock mass fabric by describing fractures in a more realistic and objective way than conventional rock mass characterization methods. Using stochastic approach; it captures connectivity of the fracture network, rock bridges and geometry of rock blocks. Fracture description is driven by verifiable data, hence the results are objective and reproduceable. This reduces convoluted workflow typically involved in rock engineering, and certainly offers a breakthrough compared with traditional rock mass characterization methods. Limitations of the traditional systems have been discussed by Palmstrom & Broch (2005), Potvin & Hadjigeorgiu (2016), and Potvin et al (2019); which derived their opinions from experience in civil and mining industries.

Further than rock mass characterization and modelling tool, the use of DFN approach can be enhanced by incorporating it directly in the design process. Some examples above using FracMan coupled with DEM have demonstrated a significant benefit harvested in ground support design process for underground projects. Conservatism can be reduced based on objective data and design optimization was achieved. This led to considerable material savings and increased confidence in handling risks arising from ground conditions.

This approach should be accompanied by proper planning and implementation of instrumentation and monitoring during underground excavation, including detailed site investigation and mapping works. This aims to verify the adopted DFN parameters and if possible, optimize and update the fracture model. Results from remote mapping or photogrammetry (e.g trace planes, scanlines, etc.) can be easily included in the fracture model and subsequently, this can be used as digital engineering tool or library for a project.

With increasing complexity in projects, data driven process such as DFN approach should be encouraged and implemented more often. As organizations create and collate more data, and with increasing computing power, such approach should lead the way rock engineering project is delivered.

6. ACKNOWLEDGMENT

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