Discrete fracture network combined with discontinuum based design for deep shafts – quantifiable risk assessment and design method

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ABSTRACT: Deep shafts provide critical support services for underground mines and therefore robust engineering assessment and design processes are vital for their successful, cost-effective construction and reliable operation. With the trend towards increasingly challenging and complex shaft projects in terms of diameter, depth, ground conditions and demands for optimisation, there is an ever-increasing need for improvements to the engineering processes. Conventionally, geotechnical assessments for shafts have widely employed empirical methods such as the Raise Bore Rock Quality (QR) system proposed by McCracken & Stacey (1989). While these methods have provided a useful tool to aid design, they have typically been based on a limited database of project experience and so their results need to be interpreted in the context of local experience and engineering judgement. To advance the engineering process and achieve optimised designs, a more quantified approach that considers the site-specific conditions is required. This paper describes an approach based on Discrete Fracture Network (DFN) modelling integrated with discontinuum-based numerical modelling. In this approach, features observed in drill core and televiewer logs are used to derive a stochastically representative ‘synthetic rock mass’ model, which then can be exploited to assess potential rock mass behavior. The DFN model developed was incorporated into a full-fledged 3D discontinuum model to assess the rock mass instability, joint-controlled wedges and stress-induced spalling potential. This combined approach allows us to quantify the likelihood of instability in different rock strata, potential rock wedge sizes, factor of safety of different wedges, potential rock mass deformation and potential major failure mechanisms which are a combination of major instability factors. Ultimately, this approach allows engineers to optimise possible solutions for shaft excavation and support design.

1 INTRODUCTION

For underground mines, shafts provide ventilation, access, and other critical services. The high capital cost for construction, and the even higher opportunity cost of any unexpected interruptions to shaft operation, mean that robust and rigorous engineering assessment and design processes are needed to help support the successful completion of shaft projects. In the mining industry, there is an increasing trend towards larger diameter and deeper shafts. Also, shaft excavations rely on rapid excavation methods (i.e. less than 6 months) to accommodate production activities. For this purpose, raise and blind boring techniques are typically
preferred compared to drill & blast techniques, as they offer better control in rock mass stability during excavation.

Conventionally, geotechnical assessment and design processes for shafts adopt empirical methods, such as the Raise Bore Rock Quality (Q_R) system proposed by McCracken & Stacey (1989) which is primarily based on the Q-system (Barton et al, 1974). Certainly, such methods have contributed to successful delivery of numerous projects in the past few decades, and their use will undoubtedly continue in the future. However, it should be acknowledged that these methods are only as good as the available data, as highlighted by Palmstrom & Broch (2005), Potvin & Hadjigeorgiu (2016), Penney et al. (2018), and Potvin et al (2019). It should also be noted that assessment outcomes based on empirical methods can be biased by personal experience and understanding. Penney et al. (2018) highlighted common errors consistently observed related to the use of the (Q_R) system.

Considering those limitations, a more quantified, objective and verifiable approach that considers site-specific conditions to achieve optimised shaft designs is desirable. The Discrete Fracture Network (DFN) approach provides an alternative method that can overcome some of the drawbacks described above. It is built upon a set of quantifiable rock mass descriptors; including defect orientation, trace length, and intensity; to represent equivalent rock mass conditions in a statistical way. Combined with 3D Distinct Element Method (DEM), this approach provides for a more comprehensive assessment of a wide range of parameters; such as internal and external ground water pressure, block instability, progressive failure, etc. Also, it allows engineers to assess shaft stability based on site-specific ground conditions and to design a tailored solution accordingly. Examples of the use of DFN approach in various rock engineering projects have been presented by, among others, Rogers et al. (2006), Panton et al. (2015), Rogers et al. (2017), and Haryono & Purwodihardjo (2021).

This paper describes a geotechnical stability assessment for a mine shaft which was excavated through jointed and fractured rocks. It demonstrates the workflow of the process, from applying the empirical approach through to the advanced numerical modelling approach, i.e. DFN and 3D DEM based approach. It highlights the benefits of the numerical modelling approach in optimising geotechnical stability assessments and designs of the shaft, thereby helping to mitigate geotechnical risks and at the same time minimise the conservatism and subjectivity inherent in the empirical approach.

2 CASE STUDY – STABILITY ASSESSMENT OF A DEEP MINE SHAFT

2.1 Details of the shaft

The case study described here is focused on a coal mine ventilation shaft with a planned depth of 280 m and a final internal diameter of 5.0 m. The selected construction methodology for the shaft was blind boring.

2.2 Geological setting

The Bowen Basin in Central Queensland covers an area of over 60,000 km² and hosts the largest coal reserves in Australia. The geology in the vicinity of the mine includes a veneer of Palaeogene & Neogene (Tertiary) sediments comprising semi-consolidated sandstone, mudstone and other minor sediments, overlying the Permian coal measures. The Tertiary and Permian sediments are superficially weathered, resulting in the formation of a thin, heterogeneous layer of residual soils and weathered clays. Ground investigation works undertaken for the shaft revealed that the most critical horizon geotechnically was the first 36 m below ground level (mbgl). This section of the shaft was assessed as being the most susceptible to instability during blind-boring due to the presence of the fractured and slake-prone strata of the Permian Fairhill Formation, comprising thinly interbedded carbonaceous mudstone, coal and tuff. It should be noted that the Fairhill Seam and Fairhill Formation are local names applied to the geographical unit recognised by the Australian Stratigraphic Units.
database as the “Fair Hill”. The following discussion is therefore focused on the first 36 mbgl of the shaft.

2.3 Ground investigations and interpreted ground conditions

Ground investigations undertaken for the planned shaft included geotechnical core drilling and logging, downhole geophysical logging (including acoustic teviewer), and laboratory testing. Laboratory testing included Unconfined Compressive Strength (UCS) tests with modulus measurement, Direct Shear (DS) tests of rock joints, Slake Durability Index (SDI) tests, and Cerchar Abrasivity Index (CAI) tests. A groundwater monitoring well was installed adjacent to the proposed shaft location, with the measured water level being at 16.8 mbgl. The encountered ground conditions within the first 36 mbgl are summarised in Table 1 and Figure 1 below.

The results of SDI tests indicate that the sandstone and siltstone units have a low susceptibility to slaking, while the carbonaceous mudstones/tuffs (14.6 – 25.4 mbgl) of the Fairhill Formation have a high susceptibility to slaking. The risk of spalling failure was assessed to be low to moderate based on the available information on intact rock strength and in situ stress.

Table 1. Inferred Ground Conditions at the shaft location.

<table>
<thead>
<tr>
<th>Depth [mbgl]</th>
<th>Lithology</th>
<th>Weathering</th>
<th>Field Inferred Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 – 2.2</td>
<td>Residual soil</td>
<td>Completely weathered</td>
<td>-</td>
</tr>
<tr>
<td>2.2 – 8.3</td>
<td>Sandstone</td>
<td>Moderately to highly weathered</td>
<td>Very low to medium</td>
</tr>
<tr>
<td>8.3 – 14.6</td>
<td>Interbedded sandstone &amp; siltstone</td>
<td>Slightly weathered</td>
<td>Low to high</td>
</tr>
<tr>
<td>14.6 – 25.4</td>
<td>Interbedded coal, Carbonaceous mudstone &amp; Tuff <em>(Fairhill Formation)</em></td>
<td>Moderately weathered to fresh</td>
<td>Very low to medium</td>
</tr>
<tr>
<td>25.4 – 28.0</td>
<td>Siltstone</td>
<td>Fresh to slightly weathered</td>
<td>Medium to high</td>
</tr>
<tr>
<td>28.0 – 36.0</td>
<td>Sandstone</td>
<td>Fresh</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure 1. Summary of Ground Conditions.
2.4 Proposed construction methods and sequence

Blind boring was adopted as the main excavation method for the shaft. For the critical interval (0 – 36 mbgl), some modifications were designed to aid stability. The first 9 mbgl of excavation was designed to be open-cut (sloped), with a pre-sink liner (5.1 m dia.) to be installed into the open cut excavation and then backfilled with concrete around the outside. Following this, the first stage of blind boring would commence from inside the pre-sink liner and progress to 28.4 mbgl, with internal fluid support being provided to assist in maintaining rock mass stability prior to liner installation. The Stage 1 steel liner (4.6 m dia.) is then to be installed to 28.4 mbgl and the liner annulus grouted prior to resumption of blind boring and installation of subsequent steel liner stages. Below 28.4 mbgl, blind-boring and liner installation will continue in stages (not discussed in this paper).

3 STABILITY ASSESSMENT

Stability assessments followed a staged approach. A preliminary assessment of the shaft stability in rock was undertaken using the empirical approach described by McCracken and Stacey (1989). The empirical assessment was intended to be a first pass approach to identify geotechnical zones which may warrant more detailed assessment. Based on this, further assessment focused on detailed stochastic kinematic analysis using a DFN model, and detailed 3D DEM analysis.

3.1 Q-Raisebore (Q_R) Approach (McCracken & Stacey, 1989)

McCracken and Stacey (1989) proposed a modification to the Q-system (Barton et al, 1974) to derive Raise Bore Quality Index (Q_R). Q_R applies three adjustment factors to Q values; including wall adjustment factor, orientation adjustment factor, and weathering adjustment factor; to account for raise boring conditions.

McCracken and Stacey (1989) used the concept of Excavation Support Ratio (ESR) to develop a relationship between Q_R and the maximum stable unsupported raise bore diameter. They considered ESR for safe excavation was 1.3 for a 5% probability of failure. Based on this, for a bored shaft with a drilled diameter of 5.0 m, a minimum Q_R value of approximately 5.1 is required. It can be observed from Figure 2 that a significant proportion of the upper 28 m of the Shaft does not achieve this minimum threshold value of Q_R. Therefore, the risk of instability was assessed to be high. This is mainly due to a poorer quality of the rock mass between 14.60 to 25.4 mbgl, where the interbedded coal, carbonaceous mudstone and tuff beds (Fairhill Formation) were encountered. It should be noted that the portion above 9 mbgl will be supported by the pre-sink liner prior to blind boring and therefore any potential instability in this area can be mitigated.

3.2 Estimated stand-up time (Bienawski, 1989)

Maximum stand-up time was assessed using the approach described by Bienawski (1989). Initial assessment results indicated a stand-up time of less than 24 hours for the unsupported condition in the Fairhill Formation. This was considered insufficient for the planned blind boring approach, since an estimated 33 hours were required to excavate through the extent of Fairhill Formation. Further assessment was therefore undertaken to consider the beneficial effects of the drilling fluid in supporting the shaft during blind boring, by adopting improved ratings for Joint Water Reduction Factor (Jw) and Stress Reduction Factor (SRF). In this situation, Q-values were improved by a factor of 2 for the purpose of the stand-up time assessment, resulting in an estimated stand-up time ranging from 24 to 70 hours.

From a construction perspective, this timeframe was still considered to be tight, and so careful planning was required to verify that the assessed stand-up time was not exceeded. For this reason, further detailed analysis using the DFN and 3D DEM approach were also required.
3.3 *Discrete fracture network modelling & stochastic kinematic analysis (fracman)*

The $Q_R$ approach was developed to assess stability of unsupported raise bore shafts. As such, the results may be overly conservative as they ignore the effect of internal fluid pressure during blind boring. To better understand the interaction between rock mass strength and structure, internal fluid pressure, and in-situ groundwater pressure; detailed numerical analyses using DFN modelling, stochastic 3D kinematic analyses, and 3D DEM analyses; were carried out.

The DFN model was developed using fracture data from geotechnical logging, televiewer logs and core photos. Fracture data were analysed to establish defect orientation and intensity for each set. The adopted parameters are presented in Table 2. Figure 2 compares the ATV data, $Q_R$ values, and the DFN model. This process was completed using WSP-Golder proprietary software, FracMan (Golder, 2020). The code has been successfully implemented in a wide range of applications, including fluid flow in rocks, fragmentation modelling, and tunnelling applications (Rogers et al., 2006; Derhowitz et al., 2017; Haryono & Purwodihardjo, 2021).

Joint shear strengths adopted in the analyses were estimated based on logged joint conditions (Jr and Ja) and where available, back analysis of Direct Shear tests. Table 3 summarises the adopted joint shear strengths. Residual shear strengths have been adopted in the 3D kinematic analyses. The kinematic analyses also considered clamping effect ($k_H/k_V = k_h/k_v = 0.8$), presence of internal fluid pressure applied from ground surface ($\gamma = 10 \text{ kN/m}^3$), and presence of the ground mass within the pre-sink zone as vertical pressure. The piezometric pressure was assumed at the top of the Fairhill Formation. Both the DFN and kinematic analyses were modelled in multiple realisations to capture equiprobable scenarios.

In contrast to the empirical approach, the effect of internal fluid pressure to improve the shaft stability can be analysed directly in the 3D kinematic analysis, as shown in Figure 3.

<table>
<thead>
<tr>
<th>Bedding Partings</th>
<th>Mean Dip/Dip Direction</th>
<th>Fisher K</th>
<th>P10</th>
<th>Fracture Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fairhill Formation</td>
<td>08/045</td>
<td>50</td>
<td>4.3</td>
<td>10 m</td>
</tr>
<tr>
<td>Other interbedded Sandstone &amp; Siltstone</td>
<td>08/045</td>
<td>50</td>
<td>2.3</td>
<td>10 m</td>
</tr>
<tr>
<td>Subvertical joints</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fairhill Formation</td>
<td>69/218</td>
<td>50</td>
<td>1.4</td>
<td>5 m (±1 m)</td>
</tr>
<tr>
<td>Other interbedded Sandstone &amp; Siltstone</td>
<td>69/218</td>
<td>50</td>
<td>1.4</td>
<td>5 m (±1 m)</td>
</tr>
</tbody>
</table>

Figure 2. Interpreted $Q_R$ and Unsupported Span Values, and Established DFN Model.
(only a few critical realisations are presented). It is evident that the internal fluid pressure assists in maintaining stability of the shaft, with the number of unstable wedges (with FoS ≤ 1.3) decreasing significantly as internal fluid pressure is applied.

### Table 3. Adopted Joint Shear Strengths.

<table>
<thead>
<tr>
<th>Considerations</th>
<th>( c_{\text{peak}} )</th>
<th>( \phi_{\text{peak}} )</th>
<th>( c_{\text{residual}} )</th>
<th>( \phi_{\text{residual}} )</th>
<th>Ja</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fairhill Formation Bedding and subvertical joints</td>
<td>0</td>
<td>22</td>
<td>0</td>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>Other interbedded Sandstone and Siltstone Beddings and subvertical joints</td>
<td>0</td>
<td>25</td>
<td>0</td>
<td>25</td>
<td>1-2</td>
</tr>
</tbody>
</table>

*Note: \( c \) = cohesion (kPa) and \( \phi \) = internal friction angle (degree)*

Figure 3. 3D Kinematic Analysis Results – with and without Presence of Internal Fluid Pressure.

### 3.4 3D distinct element method analysis – 3DEC

The synthetic rock mass model in the 3D DEM analysis (Figure 4) utilised the previously established DFN model, with some minor modifications in the Fairhill Formation zone. For the interbedded Sandstone & Siltstone layers, the fracture network from FracMan was adopted without modification in the 3DEC (Itasca, 2016) model. The Voronoi tessellation technique was employed to model the blockiness of the Fairhill Formation using Neper (Quey et al., 2011).

The 3DEC model aimed to assess different scenarios, including: (1) excavation without internal support, (2) excavation with internal fluid pressure, and (3) excavation with internal fluid pressure considering slaking condition in the Fairhill Formation. Excavations were modelled in stages. Groundwater level outside the shaft was maintained at the top of Fairhill, whilst groundwater level inside the shaft was adjusted according to the advancing base level. The adopted joint shear strengths are presented in Table 3. The main difference with the
Kinematic analysis was that a strain-softening joint model was adopted in the 3DEC model. This is also the case for the rock block shear strength, as summarised in Table 4 below.

<table>
<thead>
<tr>
<th></th>
<th>c,peak</th>
<th>$\phi$, peak</th>
<th>c,residual</th>
<th>$\phi$,residual</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fairhill Formation</td>
<td>177</td>
<td>15</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>Other interbedded Sandstone and Siltstone</td>
<td>178</td>
<td>46</td>
<td>0</td>
<td>25</td>
</tr>
</tbody>
</table>

Note: $c$ = cohesion (kPa) and $\phi$ = internal friction angle (degree)

Figure 5 presents displacement contours of the rock mass within the shaft after excavation past the Fairhill Formation. Without any internal fluid pressure, block failure was predicted (displacement greater than 100 mm). The predicted Factor of Safety (FoS) in this case is $< 1.0$. Similar to the kinematic analyses, application of internal fluid pressure assisted in maintaining stability during excavation, even considering slaking condition. Predicted maximum displacement and FoS in the case with internal pressure and without slaking was 20 mm with FoS $> 1.3$, and 50 mm with FoS $> 1.2$ for the case with internal pressure and slaking residual shear strength. These results show an improved stability compared to the empirical approach, and essentially strengthen the conclusion from the kinematic analyses.

4 RISK MITIGATION MEASURES

The results of the 3D DEM analyses were used to inform the development of a Trigger Action Response Plan (TARP) for construction of the upper portion of the shaft, incorporating appropriate risk mitigation measures. The TARP was developed in consultation between the mine operator, the shaft construction contractor, and the geotechnical consultant (WSP-Golder). This allowed an informed decision to be made with input from different perspectives. The governing geotechnical factor for construction of the upper portion of the shaft was the balance between the assessed stand-up time of the Fairhill Formation strata and the time required to excavate through these strata and then install the steel liner. This was the basis of the trigger levels defined in the TARP. Based on the preliminary assessment using the empirical approach, the predicted stand-up time was between 24 and 72 hours. In contrast, considering...
the combined results from the kinematic and 3D DEM analyses, the assessed stand-up time was increased to 96 hours, which was still considered to be a conservative prediction. This increase was sufficient to allow the contractor to confidently proceed with planning of the blind boring, on the condition that appropriate risk mitigation measures were developed and incorporated into the TARP. The mitigation measures included monitoring of a range of key drilling performance parameters to detect signs of potential instability, combined with a contingency plan for dealing with major instability if it were to occur. The parameters to be monitored included BHA weight (i.e. weight on the drilling head), drilling torque, quantity of soil/rock materials discharged while the drill is not being advanced, and the drilling fluid level. In the event that signs of deterioration in the shaft stability are observed, or if it becomes apparent that the 96-hour time limit for drilling and liner installation will be exceeded, the contingency plan within the TARP required that the blind boring head be removed, and the hole be backfilled with mass concrete to stabilise the rock mass. Subsequently, an assessment would be made of whether the shaft could be re-drilled at the same location through the concrete backfill, or if it would need to be redrilled in another location.

5 CONCLUSION

While empirical methods have undoubtedly provided a useful comparative tool for first-pass assessments of shaft stability, they have a number of shortcomings that may yield overly conservative engineering outcomes. In the above case study, it has been demonstrated how the combined DFN and 3D DEM approach advances the engineering process by providing a quantitative method that considers the site-specific ground conditions. This approach allows engineers to quantify the likelihood of instability, the size distribution of potential rock wedge failures, the factor of safety of different rock wedges, the potential rock mass deformation and the potential major failure mechanisms that can form by a combination of major instability factors. Ultimately, this approach allows engineers to optimise possible solutions for shaft excavation and support design.

At the time of writing this paper, the excavation to the proposed final depth and lining of the shaft has been successfully completed. During the excavation, there were no trigger levels exceeded and no mitigation actions required. The proposed combined DFN and 3D DEM approach directly benefited the project by providing a quantitative stability assessment method based on site-specific factual data. This allowed for an increased estimate of stand-up

Figure 5. 3DEC Analysis Results for Different Case Studies.
time to be adopted as the basis of the TARP trigger levels, which provided additional flexibil-
ity to the construction program.

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