Probabilistic assessment of rock loads for tunnel support design

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ABSTRACT: The stability of tunnels through relatively hard rock and low stress environments are typically governed by loosening loads. These are typically caused by unstable blocks bound by discontinuities in the rock mass. There are numerous methods to determine the support pressure required to resist these loads. However, current approaches do not explicitly consider the likelihood of occurrence of such loads. This paper proposes a methodology using binomial probability theory to incorporate the likelihood of a given rock load and calculate the corresponding probability of exceedance. The determination of a probability of exceedance for tunnel support allows for a quantitative assessment of the risk associated with a design. An application of this method is presented with the use of a discrete fracture network (DFN), and the Cross River Rail project in Australia as a case study.

Keywords: Tunnelling, Support design, Rock loads, Probability, Discrete fracture network.

1 INTRODUCTION

There has been a considerable global increase in the number of large-scale tunnel and mining projects. Due to the inherent complexities and variabilities in geological conditions across projects, systematic support systems are often required to safely excavate the tunnels. Given the costs associated with such support structures, there is significant interest from industry in optimization (Langford, Vlachopoulos, & Diederichs, 2016). However, this proves to be a challenging task, as there are no clearly defined rules of acceptability for support and lining design (Hoek, 2001).

Modern structural codes in Australia are largely based on limit state design methods. Limit state design adopts the view that the loads and resistances of a structure are independent probability distributions. The loads are then factored up and the resistances factored down to reduce the overlap between the two distributions such that the probability of failure is acceptably low. The application of load and material factors to tunnel design is not straightforward, especially when designing tunnel support, as the tunnel support may interact with the ground in a complex, non-linear relationship. The limitations of limit state design with regard to the design of tunnel support have been reviewed by numerous experienced geotechnical practitioners (O'Rourke, 1984) (Pells, 2003) (Oliveira, Asche,

& Day, 2017). Nevertheless, there is increasing pressure from the owners, clients and insurers of civil tunneling projects in Australia to adopt limit state design methods for the design of tunnel support (Bertuzzi, 2019).

However, for tunnels in relatively hard rock and low-stress conditions the dominant instability mode is likely to be due to loosening of rock blocks bound by fractures in the rock mass, as opposed to excessive squeezing pressures from convergence following excavation or true rock pressures arising from stress-induced rock mass failure. In such ground conditions, the magnitude of the gravity-driven loosening loads can largely be decoupled from the stiffness of the tunnel support. Martin, Kaiser & Christiansson (2003) provides a generalised method for determining the likely instability mode as a function of the GSI and intact rock strength of the rock mass, and the stress level at the level of the excavation.

Various empirical methods, closed form solutions, and numerical analysis tools can assist with the selection of appropriate support system. However, these approaches are typically used deterministically with respect to the ground conditions, with uncertainty and variability in ground conditions subjectively addressed using conservative design parameters. Often this requires that several key simplifying assumptions are made, and the support system is designed to cater for either a mean, most likely or worst-case scenario depending on the specific situation and tolerance. While such conservative approaches have been successful in reducing the residual risk during construction, they provide limited information on the likelihood of potential hazards and can result in unnecessarily high construction costs.

2 PROPOSED APPROACH

2.1 Basis of Design

The expected frequency of unstable blocks is implicitly considered in rock mass classification methods where defect characteristics such as spacing, persistence, and orientation are an input, and is not considered at all in methods utilizing key block theory.

On the other hand, a discrete fracture network allows for the explicit calculation of the frequency of blocks that are expected to occur over a given length of tunnel. However, the block frequency is typically not explicitly included in the calculation of the design block load. Instead, it is commonly assessed in the form of a cumulative frequency of the support pressures required to stabilize the distribution of unstable blocks calculated from a DFN. A design block is then typically adopted on the basis of an accepted percentile. This method is used by Lagger et al. (2014) for the design of primary support for the Airport Link caverns, "where multiple DFN realisations were used to generate block weight and support pressure cumulative frequency charts." McQueen et al. (2019) also adopted this method for "assisting in the assessment of discrete ground loads on the lining system from key blocks" for the CLEM7 tunnel. This support pressure curve provides a convenient method of quickly analyzing distributive trends in the large datasets that result from multiple realizations of a DFN model. However, while a cumulative frequency gives a useful indication of the upper limits of the expected support pressure, this assessment only considers the severity of the block loads, and omits the likelihood of occurrence.

Therefore, a novel method is proposed to determine a design support pressure in relation to an adopted probability of exceedance using binomial probability theory. Structural design standards typically specify design loads for flood, seismic, or wind events that corresponds to a specific probability of exceedance. Under such standards, structures with higher consequences of failure are designed for rarer events, or a lower probability of exceedance. However, loading from gravity-induced blocks bound by in-situ discontinuities in the rock mass are non-transient. Therefore, the likelihood of occurrence is instead a function of the number of unstable blocks that occurs along the tunnel length. The determination of a probability of exceedance for tunnel support allows for a quantitative assessment of the risk associated with a design.

Mathematically, the probability of an event over a given period can be calculated using the binomial theorem:

$$P(X = x) = \binom{n}{x} p^x (1-p)^{n-x} \quad (1)$$

Where p is the probability of success in a single event, n is the total number of events, and x is the number of successful events desired.

A specific and useful case of this generalized probability distribution is the probability that the design event or greater will occur during the design life, as this represents the summation of all possibilities that exceeds of the structure's limit state:

$$P(X \geq 1) = 1 - (1-p)^n \quad (2)$$

2.2 Application to Tunnelling

Exceedance of the limit state of a single or group of rock bolts that intersects an unstable block will not result in the failure the entire support structure along the tunnel length. Therefore, it is recommended that the number of unstable blocks (n) is considered in relation to a "structural length". For rock bolt design, the structural length could be based on the maximum size of unstable block that can kinematically form. This designates a section of tunnel length that is structurally independent from the adjacent sections for the purposes of discrete block loading. Similarly, a design length should be adopted for when block loading is considered for assessing structural actions on tunnel linings. Contrary to other plane-strain loads that are commonly imposed on a tunnel lining, block loads are discrete and localized. Therefore, a finite length of the lining will exceed limit state for a given block load due to the longitudinal distribution of structural actions. In the absence of three-dimensional structural analyses, it is recommended that the spacing of the construction joints in the lining is adopted as the design length.

The number of unstable blocks that the tunnel will encounter depends on the length of the tunnel. Therefore, the following equation is proposed for calculating the reliability of tunnel support that has been designed for a given support pressure:

$$P = 1 - W^n \quad (3)$$

Where P is the probability of exceedance, n is the calculated number of unstable blocks per structural length, and W is the percentile on the support pressure curve.

This formulation suggests that higher design loads must be considered to achieve a specific probability of exceedance as the n value increases, Figure 1. As the binomial distribution is a discontinuous function, this method is not applicable for massive rock masses where the n value is expected to be less than 1. Conversely, W converges rapidly towards 1 as n increases. This suggests that where the n value is large, the tunnel support must be designed for the worst-case loading, regardless of the adopted exceedance probability.

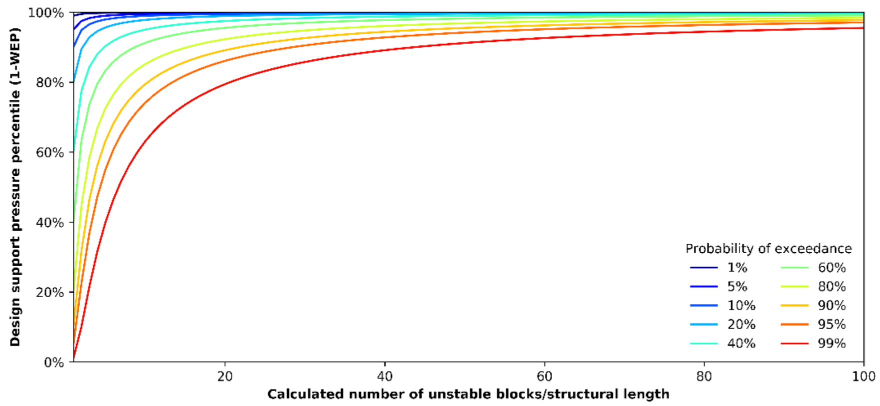


Figure 1. Design support pressure percentile curves for varying block frequencies.

2.3 Acceptable Risk

In Australia, the importance level is defined by the Australian Building Codes Board and reflects the risk of structural failure considered acceptable for a given structure. Similarly, AS1170.0 (2002) defines the importance level based on consequence of failure with regards to loss of human life or economic, social, or environmental impact. An acceptable AEP for wind, snow and earthquake events are stated for a given importance level and design working life. However, there is a lack of consensus in the industry with regards to a quantitative acceptable risk for rock loads in tunnelling.

For meaningful application of the method proposed in this paper, an acceptable and appropriate probability of exceedance must be assessed. This requires a case-by-case assessment of the consequences of block failure, with consideration to things such as the risk to human life, the support element being designed, excavation sequence and methodology, and economic impacts from damage to plant or delays to construction.

3 CASE STUDY

3.1 Project

Cross River Rail is a new 10.2 kilometre rail line from Dutton Park to Bowen Hills, which includes 5.9 kilometres of twin tunnels under the Brisbane River and CBD. The project includes construction of four new underground stations at Boggo Road, Woolloongabba, Albert Street and Roma Street. The station caverns are approximately 20m in span and 14m in height, with primary support typically comprising of rock bolts and a thin shotcrete layer.

3.2 Geology

The northern half of the project, including Albert and Roma Street Stations, are within the Neranleigh-Fernvale Group (NFG). The NFG is a lithologically varied rock mass that largely comprises the basement rock of the Brisbane CBD. It is composed of weakly metamorphosed sandstone (meta-greywacke and arenite), phyllite and subordinate quartzite and meta-basalt.

Foliation moderately dipping to the northeast is present throughout the NFG and is particularly well developed in the phyllite dominated zones. Although generally the foliation occurs as a ‘fabric’, defects do occur along the foliation. These are generally tight, smooth to rough and without infill in fresh to slightly weathered rocks. With weathering, the foliation partings may contain thin clay infill and exhibit smooth surfaces. In addition, the two joint sets have been identified: one set dipping to the southwest at low to moderate angles (i.e., sub-orthogonal to foliation), and the other steeply dipping northwest and southeast.

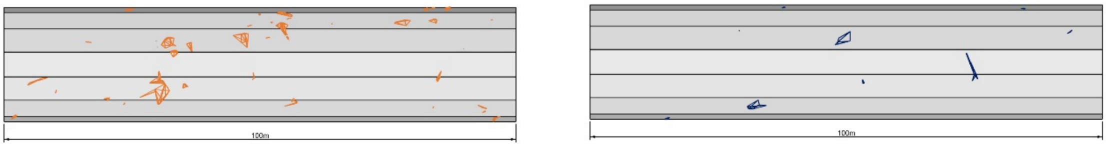
3.3 DFN Modelling

Discrete fracture networks were generated for the 20m span caverns using *FracMan* v7.8 (Dershowitz, et al., 2015). The DFN was also artificially modified by using half of the original fracture intensity, but otherwise using the same input parameters. This was done to represent a less fractured zone within the same rock mass. Rock block analyses were conducted for 50 random generations of the original and modified DFN. As the rock block calculations are based on a stochastic fracture model, the derived block locations along the tunnel cannot be considered as real block positions. Instead, the analysis provides an indication of support pressures that may be expected, which must be related back to the actual length of the excavation.

Identified rock blocks for one realization of the original and modified DFN is presented in Figure 2. Visual comparison indicates a much-lowered incidence of unstable rock blocks for the modified DFN, which represents a better rock mass with substantially reduced defect intensity.

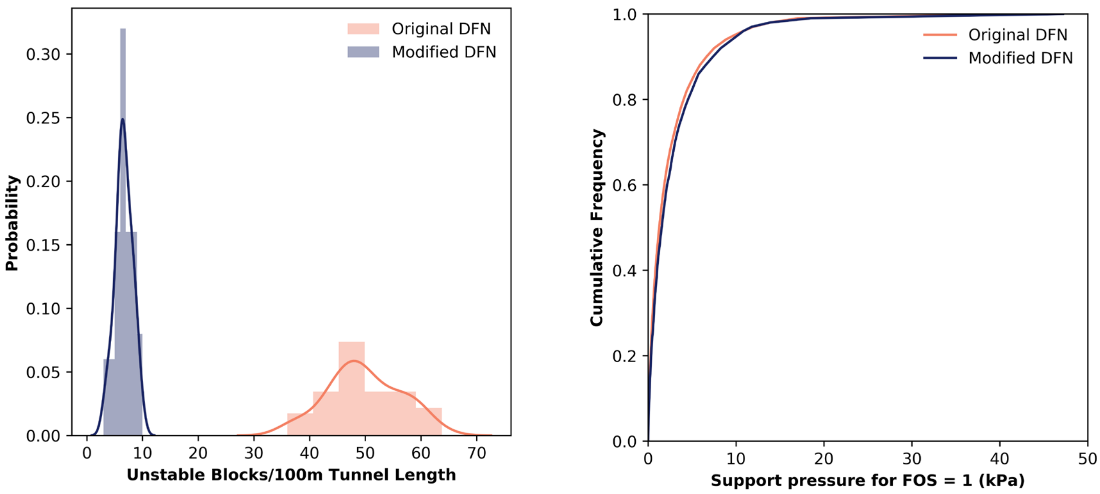
It should be noted that the dataset must be sufficiently large for the support pressure curve to be an accurate and representative of the population distribution, which has been achieved in this example with monte-carlo iterations to generate many realizations of the DFN. The distribution of kinematically unstable blocks and the support pressure curves from all iterations are presented in Figure 3. As the size and shape of unstable blocks are largely a function of fracture orientation and not intensity, the support pressure curves are nearly identical for the original and modified DFN, with a maximum of 47 kPa. Therefore, support design based on the support pressure curves alone will not explicitly consider the reduced level of risk associated with decreasing fracture intensities.

For a given design support pressure, the probability of exceeding that pressure can be determined from the support pressure curve (Figure 3B), which corresponds to the W term in equation 3. For example, 10 kPa support pressure corresponds to the 95th percentile. In addition, the DFN predicts an average of 50 and 7 unstable blocks per 100m of tunnel length for the original and modified DFN respectively, which can be used to calculate the n term of equation 3. Therefore, the probability of exceedance can be calculated using equation 3 for each design support pressure, as shown in Figure 4. Using the proposed probability of exceedance approach, the modified DFN (representing a higher-class rock mass) requires a lower design support pressure than the original DFN to achieve the same probability of exceedance, reflecting the lower risk of instability due to the reduced fracture intensity. Therefore, this method provides a quantitative basis for adopting a lower block load for higher quality rock masses.



(a) Original DFN (b) Modified DFN

Figure 2. Plan view of unstable blocks from DFN modelling.



(a) Unstable block frequency distribution (b) Cumulative support pressure curve

Figure 3. Distribution of kinematically unstable blocks.

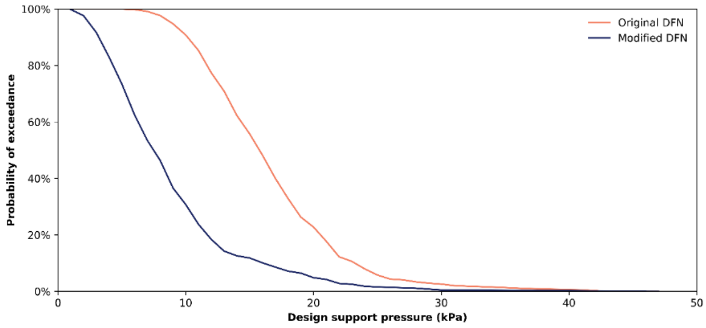


Figure 4. Probability of exceedance of varying design support pressures.

4 CONCLUSIONS

This study reviews the current design approaches to determining a block load for tunnel support design, highlighting limitations. An alternative method is presented, in which a design rock load is calculated in relation to a corresponding probability of exceedance. A practical application of this method is presented with the use of a discrete fracture network. In addition, several design recommendations have been made, and can be summarized as follows:

1. A cumulative frequency distribution of load only considers the consequence of failure, but does not consider the likelihood of occurrence;
2. More rigorous statistical analysis of a discrete fracture network's outputs can be undertaken to account for the spatial distribution of unstable blocks;
3. The design tunnel length is critical to the design support pressure.

REFERENCES

AS/NZS 1170.0. (2002). *Structural design actions General principles.* Standards Australia.

Bertuzzi, R. (2019). *Tunnel Design Handbook* (3rd ed.). Sydney: Pells Sullivan Meynink.

Dershowitz, W. S., Lee, G., Geier, J., Foxford, T., La Pointe, P., & Thomas, A. (2015). *Fracman - Interactive Discrete Fracture Data Analysis, Geometric Modelling, and Exploration Simulation, user documentation, version 7.5.* Golder Associates.

Hoek, E. (2001). *A discussion on acceptable criteria for temporary support and final linings of large span transportation tunnels in poor rock.* Internal Report.

Lagger, H., Purwodihardjo, A., Barrett, S. V., McKenzie, N., & Taylor, E. (2014). Temporary support design for the largest road caverns ever built in Australia - the Lutwyche Caverns. *Proceedings of the 15th Australasian tunnelling conference* (pp. 399 - 410). Melbourne: The Australasian Institute of Mining and Metallurgy.

Langford, J. C., Vlachopoulos, N., & Diederichs, M. S. (2016). Revisting support optimization at the Driskos tunnel using a quantitative risk approach. *Journal of Rock Mechanics and Geotechnical Engineering, 8*(2), 147-163.

Martin, D., Kaiser, P. K., & Christiansson, R. (2003). Stress, instability and design of underground excavations. *International Journal of Rock Mechanics and Mining Sciences*, 40(7-8):1027-1047.

McQueen, L. B., Purwodihardjo, A., & Barrett, S. V. (n.d.). Rock mechanics for design of Brisbane tunnels and implications of recent thinking in relation to rock mass strength.

Oliveira, D., Asche, H., & Day, R. A. (2017). Bridge Standards - Are they appropriate for Tunnel Design. *16th Australasian Tunnelling Conference 2017.* Sydney.

O'Rourke, T. D. (1984). *Guidlines for tunnel lining design / prepared by the Technical Committee on Tunnel Lining Design of the Underground Technology Research Council of the ASCE Technical Council on Research.* New York: American Society of Civil Engineers. Geotechnical Engineering Divisiion.

Pells, P. J. (2003). Against limit state design in rock. *Tunnels & Tunnelling International*, 34-38.