# BACK-ANALYSIS OF ROCK MASS MODULUS AND HORIZONTAL STRESS RATIO AT ROMA STREET STATION, BRISBANE, AUSTRALIA

**Harry Buchanan**

*Pells Sullivan Meynink, Australia*

*Abstract*

This paper presents a back-analysis of rock-mass modulus and horizontal stress ratio using convergence monitoring data from the Roma Street cavern excavation within the Neranleigh-Fernvale strata of the Brisbane CBD. With sparse existing information in the literature, the results provide important information for the design of tunnels and basements in this growing urban area. The results compare well with the limited existing data. The benefits of using the adopted back-analysis method introduced by Pells (1981) to provide confidence in design parameters are discussed.

## 1 Introduction

Optimum design of underground excavations relies heavily on understanding existing stress state and rock mass response to changes in stress, especially when considering excavation shape, advance rate and ground support (E. Hoek, 2007). Horizontal stress ratio, K ($\sigma_v/\sigma_h$), and elastic rock mass modulus, $E_m$, are especially crucial in accurately predicting stress distribution, tunnel deformation and settlement in both basic calculations and complex numerical modelling. These parameters are even more relevant in highly urbanised areas such as the Brisbane CBD, where serviceability deformation limits of surrounding structures often govern design decisions.

Reliable data for insitu stress is limited across the Brisbane region due to the high cost of field measurements and scarcity of large tunnelling projects. Insitu stress can vary geographically, between geological units and weathering profiles (E. Hoek, 2007). Pells (1981) introduces a back-analysis method for evaluating field stress ratio and rock mass modulus from tunnel convergence monitoring data. This serves as a cost-effective solution to provide more confidence in the rock mass modulus and insitu stress used in tunnel design. It also has the significant advantage that it estimates the rock-mass stress as opposed to the stress within small intact rock samples (Bertuzzi, 2019).

This paper provides the results from the back-analysis of monitoring data collected during excavation of Cross River Rail's Roma Street cavern within the Neranleigh-Fernvale Group (NFG) rock mass in Brisbane, Australia.

### 1.1 Project background

Cross River Rail (CRR) is Brisbane's first underground rail line, including four new underground stations and 5.9km of twin tunnels running under the Brisbane River and CBD. The new underground Roma Street station will become Brisbane's 'Grand Central' ("Roma Street Station Precinct," 2021), the location of which is shown in Figure 1. The cavern excavation is 280m long and the excavation profile spans approximately 22m wide and 15m high. The cavern was constructed using the sequential excavation method.


Figure 1 Cross River Rail overview. Adapted from "Planning and Environment", 2021.

## 1.2 Geological context

The Roma Street station tunnel has been excavated wholly within the NFG rock mass, predominantly in slightly weathered (SW) to fresh (FR), high strength, interbedded phyllite and metagreywacke, as shown in Figure 2. The key geotechnical features along the cavern identified from site investigation and construction records include:

- Foliation fabric typically oriented 50 to 60° towards north-east
- Continuous shearing sub-parallel to foliation
- 7 to 12m rock cover across the site
- Fault crushed and fault disturbed material associated with a wide structure of the Normanby Fault Zone (NFZ) running through the middle of the station
- Localised changes in foliation orientation and defect condition within the fault margin

The NFZ is a major geological feature extending through the Brisbane CBD, trending NW-SE. The "structural zone" of the NFZ includes a series of well defined major faults separating blocks of predominantly intact and undisturbed rock (Bennett & Norbert, 2014). The influence of this structure on horizontal stress ratio within the NFG has not been well researched.

Metamorphic rocks with well-defined foliation, such as the NFG, typically exhibit anisotropic conditions (Amadei, 1996; Amadei & Savage, 1991). Baczynski (2001) noted that this is most likely the case in the NFG, however, available data on rock mass strength, behaviour and stress anisotropy is limited.

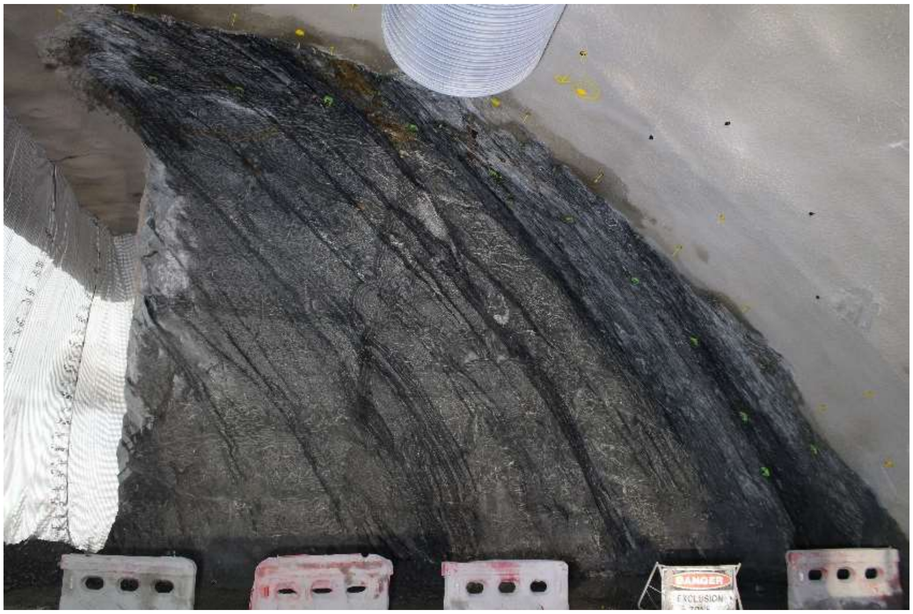



Figure 2 Heading excavation in NFG

## 2 Background

### 2.1 Literature review

Published data for insitu stress and Young's modulus of major geological units within Brisbane is limited, especially in relation to differing weathering profiles. No data has been published regarding insitu horizontal stress ratio of the NFG rock mass to the knowledge of the author. McQueen, Purwodihardjo, and Barrett (2019) noted that hydraulic fracture and overcoring measurements on the Clem7 and Airport Link projects indicate the major horizontal stress is equal to approximately twice the overburden stress. However, there is no published basis for this conclusion.

Look and Griffiths (2001) presented a summary of the geotechnical investigations in the Brisbane region from the South East Transit Project, Inner Northern Busway and the S1 Sewer Tunnel. The results indicated that for SW to FR, high strength NFG, with a UCS of 20 to 60MPa, the intact Young's modulus ($E_i$) ranges from 6,500 to 23,400MPa.

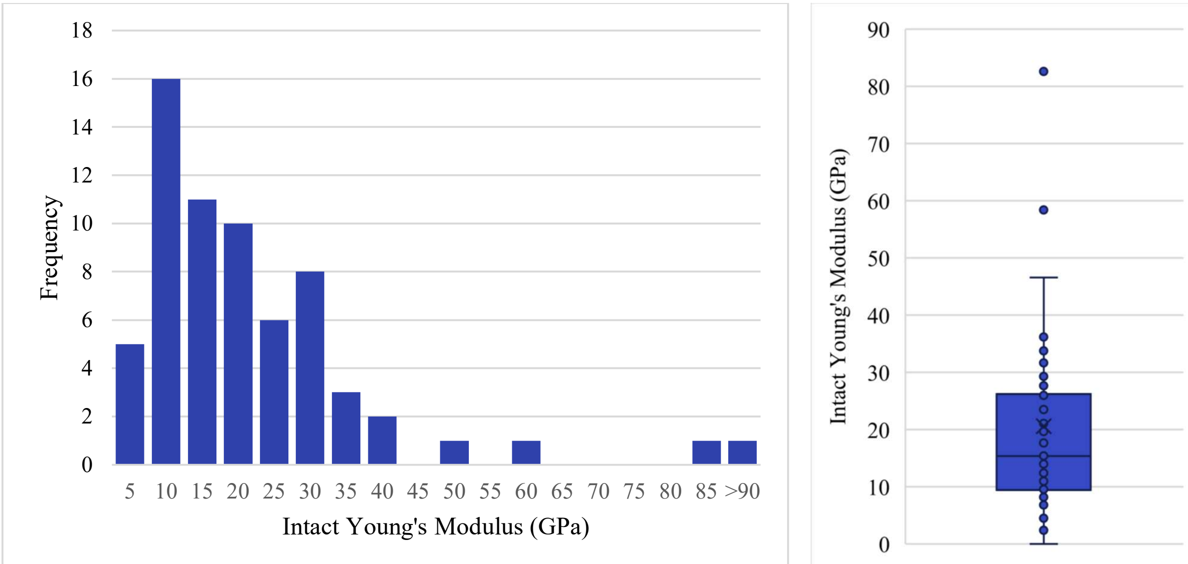
Grubb (1989) presents results from five excavations within the Brisbane CBD. A wide scatter is noted in this data set for SW to FR NFG resulting from the variability in clay infilled defects and weaker zones across sites (Grubb, 1989). The range of intact Young's modulus for SW to FR NFG is recorded as 700 to 6,500 MPa, averaging 1,500MPa and 3,000MPa for SW and FR rock respectively.

### 2.2 Unpublished data

Existing data collated from CRR, Bus and Train (BaT) Tunnel, Airport Link and Clem7 Tunnel site investigation projects using hydrofracture and overcore test methods indicates that the horizontal stress ratio, K, is between 1.0 and 4.0 for NFG. With few large tunnelling projects in Brisbane, the data set is limited and spread geographically and across weathering profiles.

### 2.3 Cross River Rail site investigation data

Results from intact Young’s modulus tests carried out as part of the CRR site investigation for SW to FR, high strength NFG are shown in Figure 3 and a summary is presented in Table 1.




(a) Frequency histogram

(b) Box plot

Figure 3 Intact Young’s modulus test results

Table 1 Summary of intact Young’s modulus

| | Minimum | 25th Percentile | Mean | 75th Percentile | Maximum |
|---|---|---|---|---|---|
| $E_i$ (GPa) | 0.0 | 9.5 | 20.6 | 26.3 | 169.0 |

### 2.4 Equivalent continuum

Rock mass modulus can be derived from intact Young’s modulus following the equivalent continuum method, introduced by Duncan and Goodman (1968), which equates the deformation of the intact rock and the deformation of the joints to the deformation of an equivalent continuum as in Eq. (1).

$$\frac{1}{E_{mp}} = \frac{1}{E_i} + \frac{1}{k_{np} s_p} \quad (1)$$

Where:

$E_m$ = rock mass modulus

$E_i$ = intact Young’s modulus

$k_{np}$ = joint normal stiffness

$s_p$ = joint spacing

Table 2 summarises the rock mass modulus for SW to FR, high strength NFG derived from the equivalent continuum method and assuming isotropic conditions. A joint spacing and joint normal stiffness of 0.5 to 2.0m and 10,000MPa/m respectively was adopted.

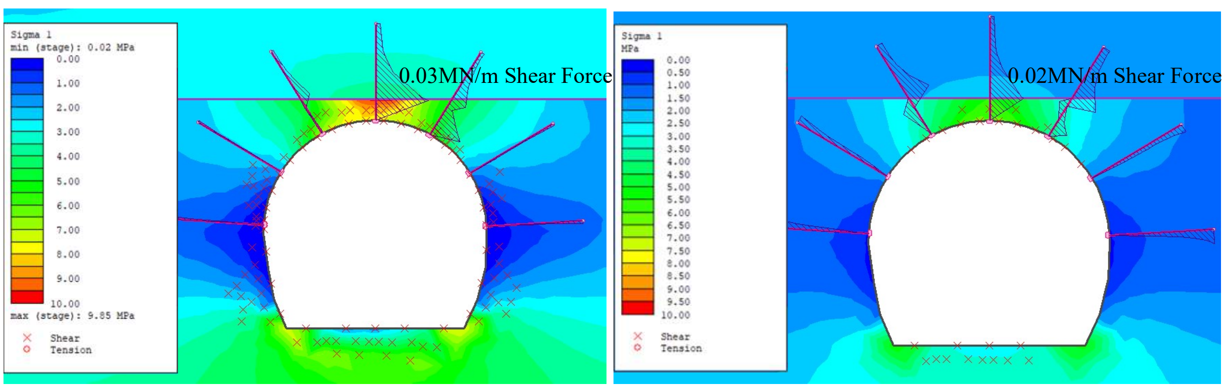
Table 2 Rock mass modulus using equivalent continuum method

| | Look and Griffiths (2001) | Grubb (1989) | CRR Data |
|---|---|---|---|
| Intact Young's modulus, $E_i$ (MPa) | 6,500 to 23,400 | 700 to 6,500 | 9,500 to 26,300[1] |
| Rock mass modulus, $E_m$ (MPa) | 2,800 to 10,800 | 600 to 4,900 | 3,300 to 11,400 |

[1] Interquartile range from data set used for comparison.

## 2.5 Importance of insitu stress

To illustrate the potential design impacts of these results, a typical rail mined tunnel in SW, high strength NFG at 30m depth was assessed, adopting a 3m advance with a single sub-horizontal shear 700mm above the crown. Typical design parameters were used, and a finite element model was run in Rocscience's RS2 finite element modelling software adopting the upper bound value for K, 4.0, from the literature and unpublished data. For comparison, a second model was run using K equal to 2.5. The results of the two analyses are shown in Figure 4.




(a) Upper bound value from available data, K=4.0 (b) K=2.5

Figure 4 Stress distribution and bolt loads for a typical mined tunnel excavation in NFG

The results of the analyses show that when increasing the K value from 2.5 to 4.0 there is a significant increase in maximum stress above the crown and bolt shear force load, 78% and 50% respectively. This results in the potential for the rock mass or bolts to yield in regions of high horizontal stress under the chosen conditions. This demonstrates that obtaining accurate and reliable data on the appropriate horizontal stress can have significant design implications.

Without the appropriate precedent such as that provided within this paper, two adverse scenarios could arise: poorly optimised design or unsafe design. Adoption of stress ranges which are unnecessarily large results in a design which could have otherwise been more economic with a decrease in ground support and increase in cut advances during construction. Conversely, adoption of narrow stress ranges could result in designs being unsafe and real deformations and loads being greater than catered for in the design.

## 3 Analysis Method

Closed form solutions for stress and displacement surrounding a circular excavation in a homogeneous, isotropic and linearly elastic medium were published by Kirsch in 1898, as presented in Eq. (2) (Pells, 1981).

$$u_r = \sigma_v d \, \frac{(1-\vartheta^2)}{2E}[(1-2\cos 2\theta)+(1+2\cos 2\theta)K] \tag{2}$$

Where:

$u_r$ = radial displacement at a point on the tunnel boundary
$\sigma_v$ = vertical field stress (overburden stress) at tunnel depth
$d$ = tunnel diameter
$E$ = Young's modulus
$\vartheta$ = Poisson's ratio

Following the method introduced by Pells (1981) and developed by Bertuzzi (2019), two shape influence factors, $I_1$ and $I_2$, are substituted into Eq. 2 to obtain a solution for non-circular tunnels, where vertical stress is equal to overburden stress. Rock mass modulus, $E_m$, is used as the rock mass is assumed to be isotropic and homogeneous for the purposes of this analysis.

$$u = \frac{K\sigma_v}{E_m} I_1 + \frac{\sigma_v}{E_m} I_2 \tag{3}$$

Where:

$u$ = displacement at a point in the rock mass

Using convergence measurements from tunnel monitoring, Eq. (3) can be re-arranged to define a linear relationship between $E_m$ and K for any data point:

$$E_m = K\frac{\sigma_v}{u} I_1 + \frac{\sigma_v}{u} I_2 \tag{4}$$

Influence factors can be defined by numerical analysis. Following the procedure outlined by Bertuzzi (2019) for an isotropic rock mass, the back-analysis is carried out as follows:

1. Two finite element models are set up in an isotropic, linear elastic, homogeneous rock mass, each with different values for $E_m$ and K. Displacement in the same direction of the monitoring instrumentation at each data point is taken from both models and substituted into Eq. (4), allowing influence factors $I_1$ and $I_2$ to be solved simultaneously.
2. For each point, displacements measured by monitoring equipment, $u$, is substituted into Eq. (4) and the resulting linear relationship for each data point is plotted on a graph of $E_m$ vs $K$.
3. The zone of the graph that encompasses the highest concentration of $E_m$ vs $K$ line intersections gives the back-analysed values for rock mass modulus and insitu stress ratio, as shown in Figure 5.

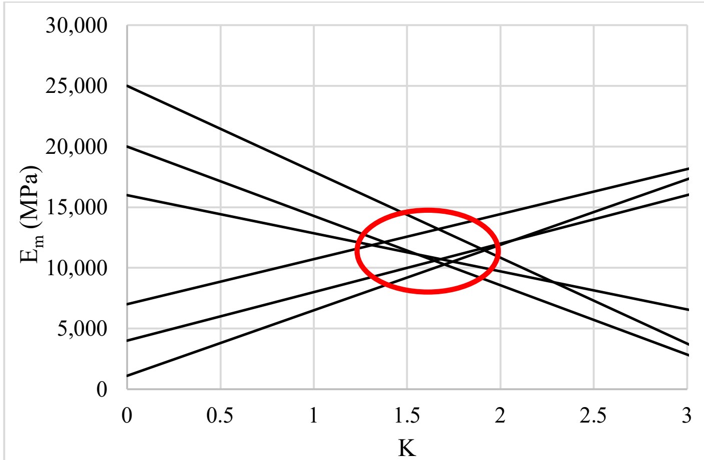



Figure 5 Example of $E_m$ vs K graph

### 3.1 Model

Following the back-analysis method presented by Bertuzzi (2019), a two-dimensional finite element model was set up in Rocscience's RS2 software with a homogeneous, isotropic, linear elastic rock mass without support, as shown in Figure 6. Typical design values for high strength NFG were used, with Poisson's ratio equal to 0.2 and a unit weight of 27kN/m$^3$. Sensitivity analyses were conducted to confirm the impact of omitting ground support in the model was negligible (less than 5%) for displacement surrounding the excavation.

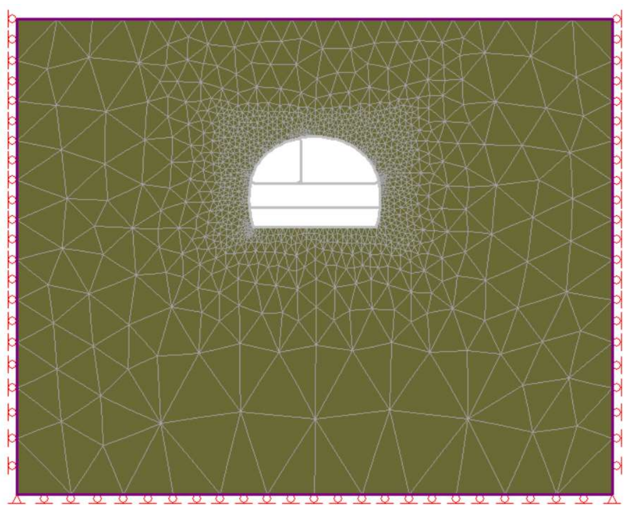


Figure 6 RS2 Model

### 3.2 Convergence monitoring during construction

Tunnel convergence was measured using multi-rod extensometers, borehole inclinometers and optical survey arrays. Borehole inclinometers were drilled from the surface and extend 5m below the tunnel invert, where the ground was considered stable. Inclinometers were read with a probe at 0.5m intervals, taking readings in opposite direction for both the A and B axis. Multi-rod extensometers were anchored and grouted into the rock mass from the surface, or the tunnel as required, and read electronically. Extensometer and inclinometer data was considered most reliable given the higher accuracy. Optical survey targets can typically introduce up to ±2mm error, which was considered unreliable for the purposes of this back-analysis where less than 5mm convergence was expected between each heading excavation.

### *3.2.1 Challenges of monitoring during construction*

The largest challenge of implementing this method is the availability of accurate and reliable monitoring data. A large spread of results can be expected using this method resulting from random, systematic and human error in monitoring data collection. This creates the need for redundant data. The most reliable data is obtained using accurate instrumentation that minimises human error, for example, multi-rod extensometers and inclinometers installed prior to excavation. Instrumentation installed within a tunnel is best installed as close to the excavation profile as possible, where the largest deformations are expected, without risking obstruction during excavation. The results of monitoring instrumentation should be reviewed regularly, and any obstructions or suspected errors should be recorded. This aids in applying data correction where required, for example inclinometer depth correction or re-defining the baseline reading if an instrument is obstructed. To ensure reliable data is selected, expected results and construction records, including geological mapping and endoscopes, should be closely reviewed to recognise and account for localised irregularities.

## 4 Results

Tunnel convergence at each stage of excavation was assessed by recording a baseline reading prior to heading relaxation and displacement reading at full heading relaxation, approximately two tunnel diameters behind and ahead of the monitoring instrument location respectively (Evert Hoek, Carranza-Torres, & Diederichs, 2008). Monitoring locations were compared to construction records to limit any localised geological effects that might cause atypical results, such as sub-horizontal shearing in proximity to the crown.

The reliability of each data point was assessed based on stability of readings during periods of excavation outside of the excavation influence zone. Data outside the limits listed below was considered unreliable.

1. Stable readings (+/-) 15% of mean value for one week pre and post excavation
2. Stable readings (+/-) 30% of mean value for one week pre and post excavation
3. Stable readings (+/-) 40% of mean value for one week pre and post excavation

A typical extensometer data example, located five metres above the crown, is shown in Figure 7.

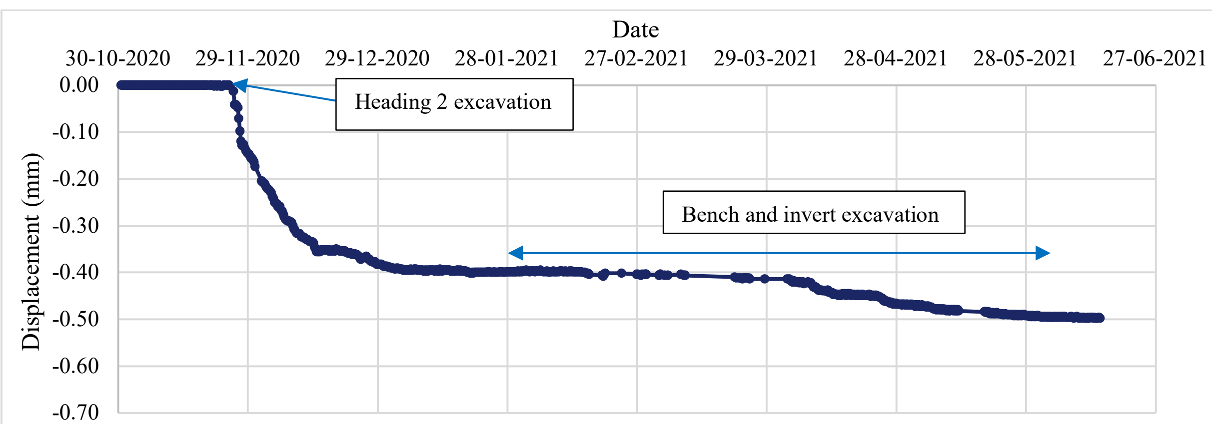



Figure 7 Displacement vs time graph

Inclinometer data was corrected for bias using correction procedures outlined by Mikkelsen (2003). To account for the heterogeneous effects of foliation parallel structures, the maximum displacement for each inclinometer reading was compared to the maximum displacement from the model. A typical inclinometer example is shown in Figure 8.

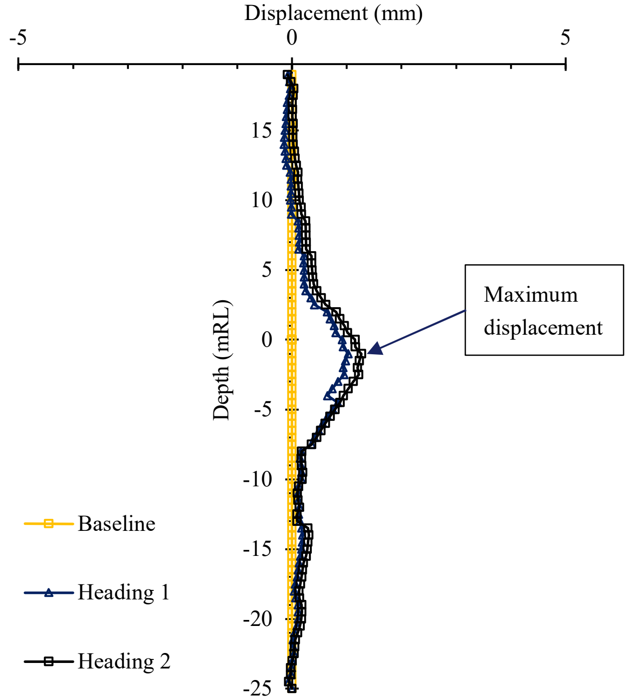
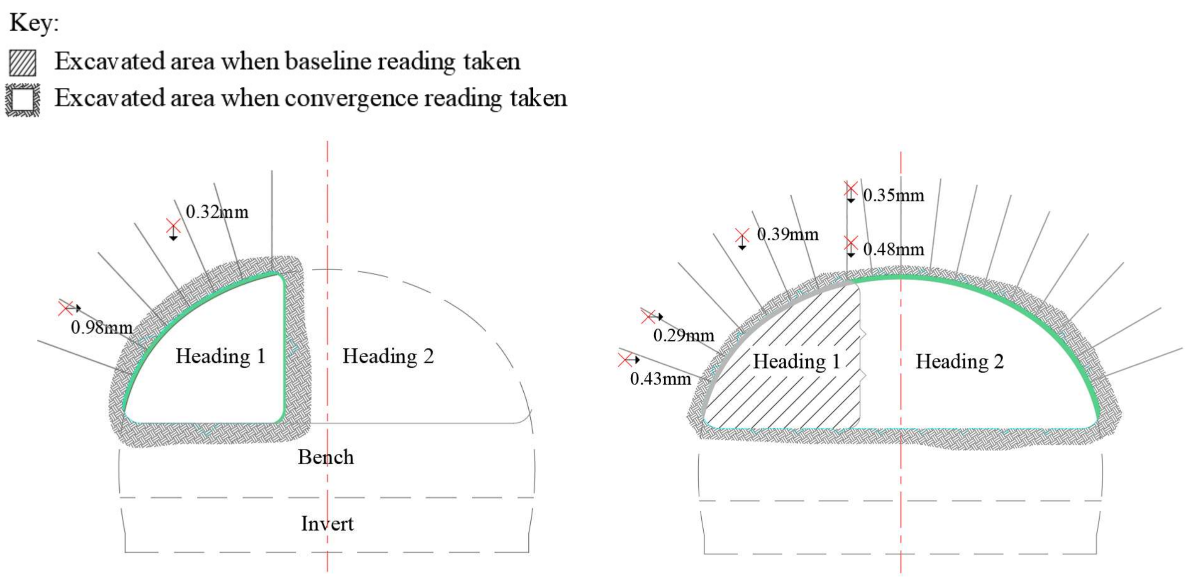
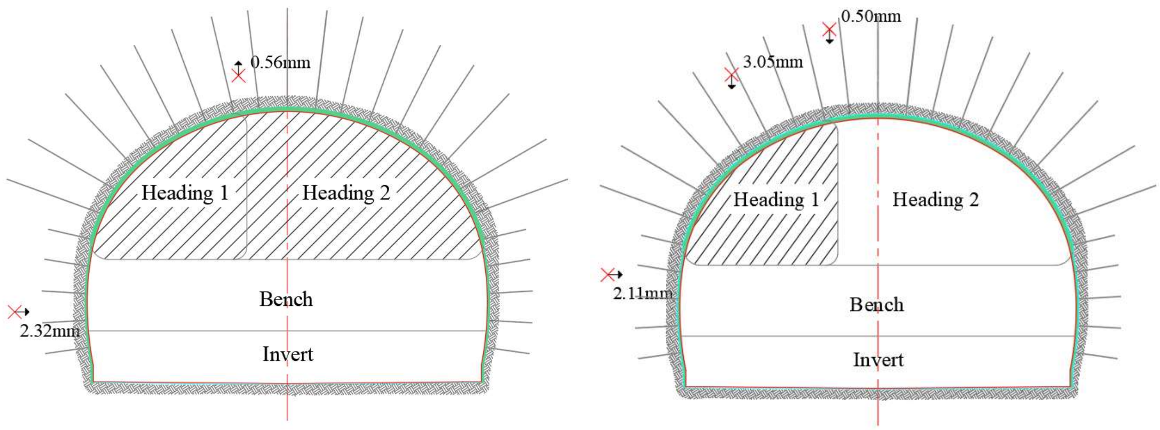



Figure 8 Inclinometer displacement vs depth graph

The cross-section summary of convergence monitoring data from the Roma Street cavern excavation is presented in Figure 9.



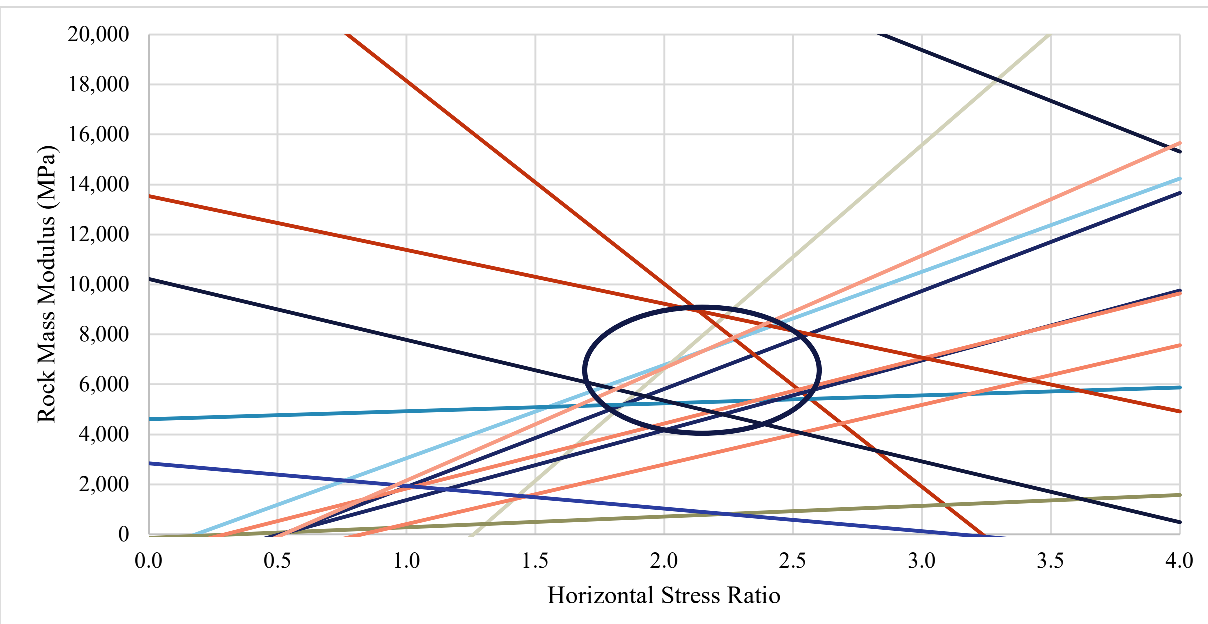

(a) Heading 1 excavation (b) Heading 2 excavation



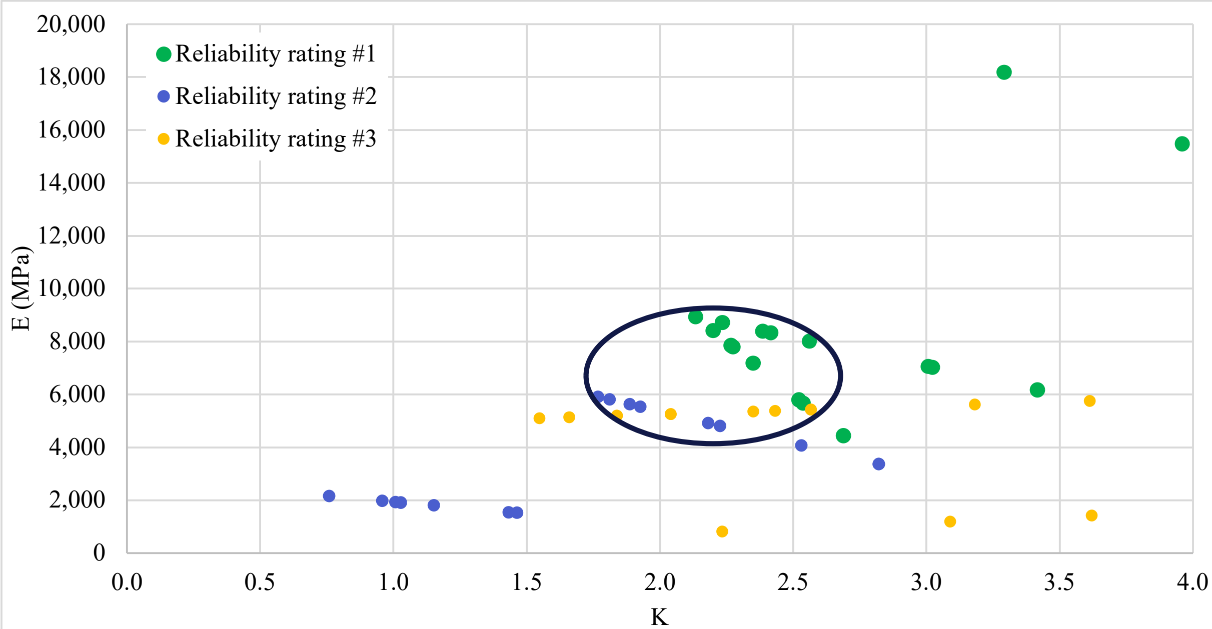

(c) Bench and invert excavation (d) Heading 2, bench and invert excavation

Figure 9 Convergence monitoring data summary

The back-analysed results for $E_m$ and K for the Roma Street cavern are summarised in Figure 10. Figure 10 (a) plots the $E_m$ vs K relationship for each data point. Figure 10 (b) plots the intercept points of these lines. Only data points from intersecting direct and inverse relationships were considered, given the high sensitivity of results obtained from lines with similar slope to small changes in measured displacement. The reliability rating is governed by the minimum of the two intersecting data points. The circled area encompasses an increased concentration of reliable results, and thus represents the results from the back-analyses.




(a) Plot of $E_m$ vs K linear equations




(b) Plot of $E_m$ vs K linear equation intercepts

Figure 10 Back-analysis results

The results from the back-analysis indicate that within the SW to FR, high strength NFG at Roma Street, rock mass modulus is between 4,000MPa to 9,000 MPa, and the horizontal stress ratio is between 1.7 to 2.7. The results from the model are compared with published data and unpublished data from previous site investigations in Table 3. The value for rock mass modulus compares well to available data. Similarly, the results for horizontal stress ratio are within the range of expected values. The results indicate that K values as high as 4.0 or as low as 1.0 are unlikely in this area. Designers of future excavations and tunnels in NFG should consider this range of insitu stresses.

Table 3 Back-analysis results compared to previous data

| Parameter | Back-analysis | Published data | CRR data | Unpublished data |
|---|---|---|---|---|
| $E_m$ (MPa) | 4,000 to 9,000 | 600 to 10,800 | 3,300 to 11,400 | Not applicable |
| K | 1.7 to 2.7 | Not applicable | Not applicable | 1.0 to 4.0 |

## 5 Conclusion

This paper presents a comparison of available data, published and unpublished, to the results of back-analyses following the simple method outlined by Bertuzzi (2019) for SW to FR, high strength NFG at Roma Street cavern. The results of this back-analysis indicate that rock mass modulus and horizontal stress ratio are between 4,000 to 9,000MPa and 1.7 to 2.7 respectively. This compares well to available data and provides a rationale to decrease the unnecessarily large range of values that may be considered in design when limited data is available.

Designers of future tunnels, basements and other sub-surface structures in NFG rock should consider the interpretation of rock mass modulus and insitu stress presented. For future tunnelling and underground excavation projects in NFG, more data is required to obtain a better understanding of insitu stress state and rock mass parameters across Brisbane. Additionally, further testing should focus on the relationship between NFG rock mass parameters, lithofacies, stress state and foliation orientation.

## References


Amadei, B. (1996). Importance of anisotropy when estimating and measuring in situ stresses in rock. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 33*(3), 293-325. doi:https://doi.org/10.1016/0148-9062(95)00062-3

Amadei, B., & Savage, W. Z. (1991). Analysis of borehole expansion and gallery tests in anisotropic rock masses. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 28*(5), 383-396. doi:https://doi.org/10.1016/0148-9062(91)90077-Y

Baczynski, N. (2001). Intact rock strength of Neranleigh-Fernvale strata. *36*, 9-15.

Bennett, C., & Norbert, M. (2014). Tunnelling within the Bunya Phyllite of Legacy Way, Brisbane, Queensland. In *In: 15th Australasian Tunnelling Conference 2014: Underground Space - Solutions for the Future. Barton, ACT: Engineers Australia and Australasian Institute of Mining and Metallurgy, 2014: 131-140.*

Bertuzzi, R. (2019). *Estimating Rock Mass Properties*. PSM, Unit G3 56 Delhi Rd North Ryde New South Wales Australia: Robert Bertuzzi.

Duncan, J. M., & Goodman, R. E. (1968). Finite element analysis of slopes in jointed rock.

Grubb, K. B. (1989). *Engineering geology of the central business district of Brisbane.* (Masters by Research Thesis Masters by Research Thesis). Queensland University of Technology, quteprints. Retrieved from https://eprints.qut.edu.au/35961/

Hoek, E. (2007). *Practical Rock Engineering*.

Hoek, E., Carranza-Torres, C., & Diederichs, M. (2008). The 2008 Kersten Lecture Integration of geotechnical and structural design in tunneling.

Look, B., & Griffiths, S. G. (2001). An engineering assessment of the strength and deformation properties of Brisbane rocks. *Australian Geomechanics, 36*, 17-30.

McQueen, L. B., Purwodihardjo, A., & Barrett, S. V. L. (2019). Rock mechanics for design of Brisbane tunnels and implications of recent thinking in relation to rock mass strength. *Journal of Rock Mechanics and Geotechnical Engineering, 11*(3), 676-683. doi:https://doi.org/10.1016/j.jrmge.2019.02.001

Mikkelsen, P. (2003). *Advances in inclinometer data analysis*.

Pells, P. J. N. M., B. K.; Redman, P. G. (1981). *Interpretation of Field Stresses and Deformation Moduli from Extensometer Measurements in Rock Tunnels*. Paper presented at the Fourth Australian Tunnelling Conference, Melbourne, Australia.

Planning and Environment. (2021). *Cross River Rail.* Retrieved from https://crossriverrail.qld.gov.au/planning-environment/

Roma Street Station Precinct. (2021). *Cross River Rail.* Retrieved from https://crossriverrail.qld.gov.au/precincts/roma-street-station-precinct/