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# Back-analysing rock mass modulus from monitoring data of two tunnels in Sydney, Australia

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A B S T R A C T

This paper presents two case studies where the rock mass modulus and in situ stress are estimated from the monitoring data obtained during the construction of underground excavations in Sydney, Australia. The case studies comprise the widening of existing twin road tunnels within Hawkesbury sandstone and the excavation of a large cavern within Ashfield shale. While back-analysis from detailed systematic monitoring has been previously published, this paper presents a relatively simple methodology to derive rock mass modulus and in situ stress from the relatively simple displacement data routinely recorded during tunnelling.



## 1. Introduction

Together with its strength, the modulus of a rock mass is almost the key used for the design of slopes, foundations and underground excavations in rocks (Brown, 2008). Rock mass modulus is often estimated from correlations to rock mass classification systems. Sometimes it is estimated by assuming an analytical or numerical model. Occasionally it is estimated by in situ measurement and/or back-analysis of displacement data measured during excavation.

This paper gives a brief overview of the methods before presenting two case studies to show a relatively simple methodology for deriving rock mass modulus and in situ stress from the displacement data routinely recorded during tunnelling. The case studies are from two tunnelling projects in Sydney, Australia: the widening of existing twin road tunnels within Hawkesbury sandstone and the excavation of a large cavern within Ashfield shale.

## 2. Overview

### 2.1. In situ measurements

Many researchers have expressed the difficulties of in situ testing, its interpretation and the high variability of results. Bieniawski (1978) noted that the modulus as interpreted from the best understood test, the plate bearing test, can still differ by a factor of 2–3. Hoek and Diederichs (2006) found that the least reliable in situ measurements are those from various down-hole jacks and borehole pressuremeters, particularly in hard jointed rock mass, an observation supported by the data in Chun et al. (2009). Vibert and Ianos (2015) compiled in situ test results from different methods and concluded that: "*As a common feature of these tests, it can be said that they are relatively difficult to implement, and interpretation and calculation of moduli necessitate assumptions which cannot be directly verified. Due to the natural dispersion of results, they are obviously to be performed in large number for allowing assessing an average behaviour*".

A comparison between the in situ modulus derived from plate bearing tests carried out within a carefully excavated adit and a drill-and-blast adit was made by Palmstrom and Singh (2001). They concluded that the modulus of the rock mass in the drill-and-blast adit could typically be a third of that obtained in the carefully excavated adit.

Anisotropy further complicates the in situ tests' interpretation. Tziallas et al. (2009), for example, quoted that the ratio of modulus to the unconfined compressive strength (UCS) for schist varies between 250 and 1100, depending on the relative orientation of testing to foliation, compared to the typical range of 300–500 for most rocks.

Reasonable conclusions were also drawn from Tziallas et al. (2009) that the in situ test methodology needs to be considered when assessing rock mass modulus ($E_m$) values and that the resultant $E_m$ values will be highly variable.

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### 2.2. Classification systems

Paraphrasing Brown (2008), there is now a 40-year history of attempts to correlate rock mass modulus with rock mass classifications. However, the inherent scatter in measured $E_m$ values means that any curve fitting approach is expected to be a poor predictor.

The most widely used classification systems for tunnel design include rock mass quality ($Q$) (Barton et al., 1974; Grimstad and Barton, 1993; Barton and Grimstad, 2014) and geological strength index ($GSI$) (Hoek, 1994; Hoek and Brown, 1997; Marinos and Hoek, 2000), as well as antecedence rock mass rating ($RMR$) (Bieniawski, 1973, 1976, 1989) and rock quality designation ($RQD$) (Deere, 1968). The correlations between $E_m$ and these systems are discussed in the following sections. In addition, for massive or slightly jointed rock mass, $E_m$ (in GPa) ≈ 0.2$UCS$ (in MPa) (Palmstrom and Singh, 2001).

#### 2.2.1. RQD system

Zhang and Einstein (2004) suggested that the following correlation between $RQD$ and modulus provides useful bound values:

$$E_m/E_i = 10^{0.0186RQD-1.91}A \tag{1}$$

where $E_m$ and $E_i$ are the Young's moduli of rock mass and intact rock, respectively; $A = 0.2$, 1 or 1.8 for lower bound, mean or upper bound, respectively.

However, the resulting large scatter from these bounds suggests that the correlation is probably not sufficiently accurate for many design purposes.

#### 2.2.2. RMR system

Bieniawski (1978) presented the $RMR$ classification with rock mass modulus data derived from plate bearing, flat jack, borehole dilatometer and geophysical tests. He obtained the following equation by curve fitting and claimed an accuracy to predict in situ rock mass modulus within 20%:

$$E_m = 2RMR_{76} - 100 \tag{2}$$

Serafim and Pereira (1983) brought attention to the evident issue with Eq. (2) that a negative in situ modulus is predicted for $RMR_{76} \leq 50$. They added data to those of Bieniawski and proposed an exponential relationship to predict in situ modulus:

$$E_m = 10^{(RMR_{76}-10)/40} \tag{3}$$

Various other correlations with $RMR$ have been derived and modified by subsequent researchers (e.g. Galera et al., 2007; Mohammadi and Rahmannejad, 2010; Nejati et al., 2014).

#### 2.2.3. Q system

A correlation between $Q$ value and rock mass modulus was provided by Grimstad and Barton (1993) as $E_m = x\log_{10}Q$. They gave the typical value for $x$ as 25 but noted that it could range between 10 and 40. Barton (1995) updated the correlation as

$$M = 10Q^{1/3} \tag{4}$$

where $M$ is the rock mass deformation modulus. A similar relationship was derived by Palmstrom and Singh (2001), i.e. $E_m = 8Q^{0.4}$.

#### 2.2.4. GSI system

Hoek and Diederichs (2006) collated a large database of in situ rock mass modulus measurements and derived a sigmoidal relationship by curve fitting as follows:

$$E_m = 100000\frac{1-D/2}{1+e^{(75+25D-GSI)/11}} \tag{5}$$

This relationship includes a subjective term called the disturbance factor, $D$, which ranges from 0 for an undisturbed rock mass, to 1 for a fully disturbed rock mass.

#### 2.2.5. Collated data

Fig. 1 plots the rock mass modulus database collated from the published data presented in the preceding sections as well as from Stephens and Banks (1989), Douglas (2002), Kayabasi et al. (2003), and Kallu et al. (2015). As the published sources did not always include the raw data, there may be some duplication in the data presented in the figure. Borehole test data were excluded from the dataset to select the most reliable rock mass modulus values. The data which are likely to represent disturbed rock mass are also excluded. As can be seen in Fig. 1, most of the undisturbed rock mass data fall within a range about the value predicted by Eq. (5) for $D = 0$. However, the range shown in Fig. 1 by the dashed lines is generated by replacing the constant 75 in the numerator of the exponential in Eq. (5) with 70 and 80, respectively, i.e.

$$E_m(\text{in GPa}) = 100\frac{1-D/2}{1+e^{(B+25D-GSI)/11}} \tag{6}$$

where $B = 70$, 75 or 80 for the upper bound, mean or lower bound, respectively.

Fig. 2 plots the data that are likely to represent disturbed rock mass, which come from Palmstrom and Singh (2001) and Nejati et al. (2014). Data from other sources are not shown as the extent to which the rock mass from these sources is disturbed is not known. The lower bound, mean and upper bound curves generated from Eq. (6) with $D = 0.5$ are also shown. This suggests that $D = 0.5$ may be appropriate for rock masses disturbed by drill-and-blast and stress relief.

The data in Figs. 1 and 2 show that, while the equation proposed by Hoek and Diederichs (2006) is a good fit to the mean, it is better to acknowledge the uncertainty in the prediction by quoting the range and using Eq. (6).

### 2.3. Analytical models

The model shown in Fig. 3 is an example of a relatively simple, theoretical model for a sedimentary rock mass. The model relates the deformation characteristics of a jointed rock mass to the deformations of intact rock and joints. The rock mass can be considered as blocks of intact rock separated by evenly spaced, parallel bedding planes and orthogonal joints. For the particular model shown in Fig. 3, Kulhawy (1978) quoted the earlier works of Goodman et al. (1968) and Duncan and Goodman (1968) to calculate the rock mass modulus ($E_m$) as a function of defect spacing for loading perpendicular to the defects as

$$\left.\begin{aligned} \frac{1}{E_{m,p}} &= \frac{1}{E_i} + \frac{1}{k_{np}s_p} \\ \frac{1}{G_{pq}} &= \frac{1}{G_i} + \frac{1}{k_{np}s_p} + \frac{1}{k_{nq}s_q} \\ (p &= x, y;\ q = y, z, x) \end{aligned}\right\} \tag{7}$$

Complexity can be built into this type of model to include orthorhombic layers (Gerrard, 1982), joint slip (Adhikary and Dyskin, 1997, 1998), and/or defect shear stiffness (Zhang, 2010) as follows: