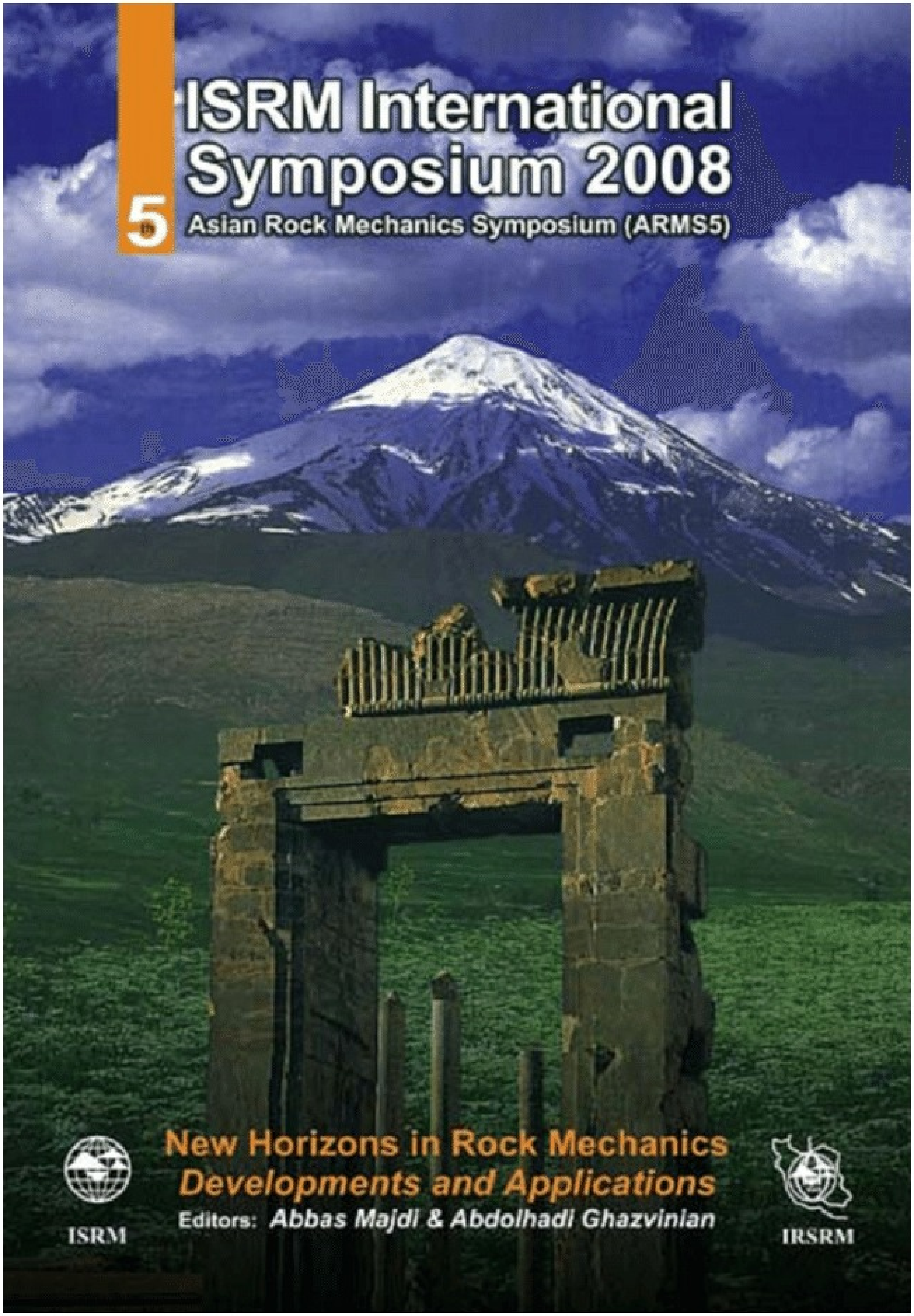
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5th Asian Rock Mechanics Symposium (ARMS5)
New Horizons in Rock Mechanics
Developments and Applications
Editors: Abbas Majdi & Abdolhadi Ghazvinian
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**ISRM International Symposium 2008**
**5th Asian Rock Mechanics Symposium (ARMS5), 24-26 November 2008 Tehran, Iran**

# ESTIMATING ROCK MASS LONG-TERM STRENGTH USING IN-SITU MEASUREMENT AND TESTING RESULTS

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## Abstract

A method for estimating rock mass long-term strength ($\sigma_{LT}$) using the results of *in situ* measurement and test has been developed. It consists of estimation of rock mass strength ($\sigma_{cm}$) using Hoek-Brown criterion, determination of rock mass modulus of deformation ($E_m$) through Goodman's jack test, and construction of rock mass rheological model based on displacement monitoring data, taking into account the stress changes due to stoping activities underneath the test and monitoring locations. The rheological model is used to determine the rock mass long-term modulus of deformation ($E_{LT}$). It is proposed that the long-term strength of rock mass can be estimated by using the rock mass strength and deformation modulus, rock mass long term modulus, and a coefficient that depends on the rock mass characteristics.

*Keywords*: Rock mass; Hoek-Brown criterion; Long-term strength; Rheological model.

## 1. Introduction

Reliable estimate of rock mass strength is required for almost any form of analysis used for the design of underground excavations. Hoek and Brown [1] proposed a method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. This method was modified over the years in order to meet the needs of users who applied it to problems that were not considered when the original criterion was developed. A review of the development of the criterion and of the equations proposed at various stages in this development is given in [2].

Although it is very useful for estimating the rock mass strength, the Hoek-Brown criterion can not be used in estimation of rock mass long-term strength, for which there is no method currently applicable [3, 4, 5]. This work suggests an alternative method for estimating the long-term strength of the rock mass, in particular that in Pongkor underground gold mine, Indonesia. It combined laboratory test, *in situ* test and monitoring, and numerical and rheological modelling.

## 2. Rock and Rock Mass Strengths

### 2.1. *Intact Rock Strength*

Among others, uniaxial and triaxial compression tests seem to be the most frequent tests conducted for design purposes. However, researches have revealed that the uniaxial compressive strength is not an intrinsic material property, as it depends on the specimen geometry (size and shape) and loading rates.

Researches on the geometrical effects have concluded that there is a reduction in strength with increasing sample size. Medhurst and Brown [6] have reported that for coal from Moura mine in Australia, the 'critical' sample size is about one metre, above which the strength remains constant. This argument was further extended by Hoek and Brown [2] who suggested that when dealing with large scale rock masses, the strength will reach a constant value when the size of individual rock pieces is sufficiently small in relation to the overall size of the structure being considered.

A number of studies also reported that the strength decreases as the sample slenderness increases. In addition, it has been observed over the years that higher loading rate leads to higher uniaxial compressive strength. Bieniawski [7], for example, reported this phenomenon after conducting uniaxial compression tests with test duration ranged from 10 minutes to 5 years, as illustrated in Figure 1.

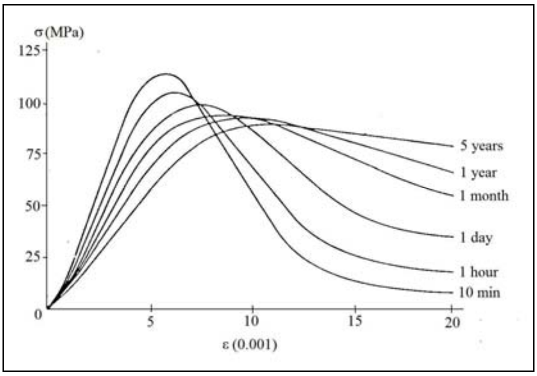


Fig. 1. Effect of test duration on uniaxial compression strength and stress-strain curve [7]

### 2.2. *Rock Mass Strength*

The Hoek-Brown criterion has been widely used to estimate rock mass strength. The generalised Hoek-Brown criterion for jointed rock mass is defined by [2]:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s\right)^a \quad (1)$$

where $\sigma'_1$ and $\sigma'_3$ are the maximum and minimum effective stresses at failure respectively, $m_b$ is the value of the Hoek-Brown constant m for the rock mass, s and a are constants which depend upon the characteristics of the rock mass, and $\sigma_{ci}$ is the uniaxial compressive strength of the intact rock pieces.

To derive Hoek-Brown criterion, laboratory triaxial test is conducted to obtain $m_i$, which is the constant m for the intact rock, and $\sigma_{ci}$. The $m_b$, s, and a are then determined by taking into account the characteristics of the rock mass and applying the following relationships [2]:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \quad (2)$$

where GSI is the Geological Strength Index [8].

For GSI > 25 (reasonable to good quality rock mass), the Hoek-Brown criterion is applicable with

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad (3)$$

and

$$a = 0 \quad (4)$$

For GSI < 25 (very poor quality rock mass), the Hoek-Brown criterion applies with

$$s = 0 \quad (5)$$

and

$$a = 0.65 - \frac{GSI}{200} \quad (6)$$

Moreover, it can be said the Hoek-Brown criterion is a useful tool for relating the intact rock strength to rock mass strength. However, as the triaxial test is carried out with loading rate (0.1 MPa/s or 10 minutes of test duration) that is much higher than that experienced by the rock mass, further investigations are required to apply the criterion for estimating the long-term strength of the rock mass. Similar requirement is also needed to relate the long-term strength of intact rock to that of rock mass.

## 3. Estimating Rock Mass Long-Term Strength

### 3.1. *Approach*

In many cases, back analyses of underground excavations instabilities using numerical modelling have been widely used to estimate the rock mass long-term strength. In this approach the rock mass strength parameters are adjusted until the model gives a similar result with that occurs in the field. It is obvious that this approach requires an instability case which was not available in the work reported in this paper. An alternative approach was then implemented in this work as described in the followings.

Regarding the previous description of Hoek-Brown criterion, the criterion is applicable in the estimation of rock mass strength ($\sigma_{cm}$), based on $\sigma_{ci}$ and GSI. The $\sigma_{cm}$ and rock mass modulus of deformation ($E_m$) obtained from the *in situ* test can then be utilised to obtain the hypothetical rock mass failure strain ($\varepsilon_m$) by utilising the stress-strain curve.

Furthermore, displacement monitoring data can be used to construct the rheological model of the rock mass, which can be used to estimate the rock mass long-term modulus of deformation ($E_{LT}$). Again, by consulting the stress-strain curve, the long-term strength of rock mass ($\sigma_{LT}$) is likely between the stress straining the rock mass to $\varepsilon_m$ and that straining the rock mass to $\varepsilon_{LT}$.

Figure 2 shows how this approach is conducted by utilising the stress-strain curves.

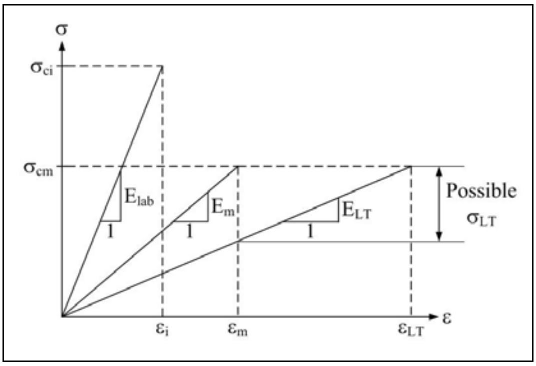


Fig. 2. Approach in estimating rock mass long-term strength

It can be seen in Figure 2, that the following relationship is applicable:

$$\sigma_{LT} = \psi \frac{\sigma_{cm}}{E_m} E_{LT} \quad (7)$$

where,

$\sigma_{LT}$ = rock mass long-term strength.
$\sigma_{cm}$ = rock mass strength.
$E_m$ = rock mass modulus of deformation.
$E_{LT}$ = rock mass long-term modulus of deformation.
$\psi$ = a coefficient.

## 3.2. *Application*

The above approach was implemented in the estimation of long-term strength of rock mass in cross-cut 6A of Pongkor underground gold mine, Indonesia (Figure 3).

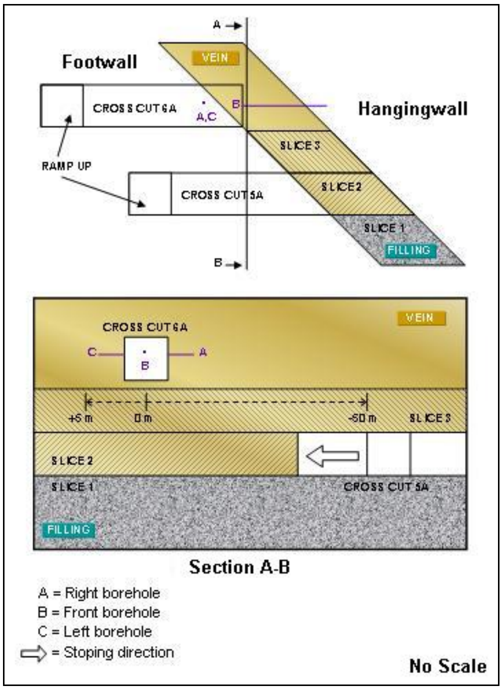


Fig. 3. Research location in Pongkor underground gold mine

### 3.2.1. *Rock mass strength*

An intensive structural geology mapping has been conducted in the research location, including scan-line and window mappings and GSI of the andesitic-breccia rock mass (hangingwall and footwall) has also been calculated and an average value of 59 was obtained. As the average $\sigma_{ci}$ of the intact rock was 88.30 MPa, application of Hoek-Brown criterion then gave a $\sigma_{cm}$ of 22.02 MPa.

### 3.2.2. *Rock mass modulus of deformation*

A number of Goodman's jack tests have been carried out and a detailed description of the tests can be found in [9]. The measured $E_m$ of the rock mass was 6.17 GPa. This value was 47% of the average $E_{lab}$ and this was in accord with the review reported by Mohammad *et al.* [10].

### 3.2.3. *Rock mass rheological model*

Displacement monitoring using multi point borehole extensometer and convergencemeter were conducted in the research location. Figure 4 depicts the installation of the displacement monitoring devices.

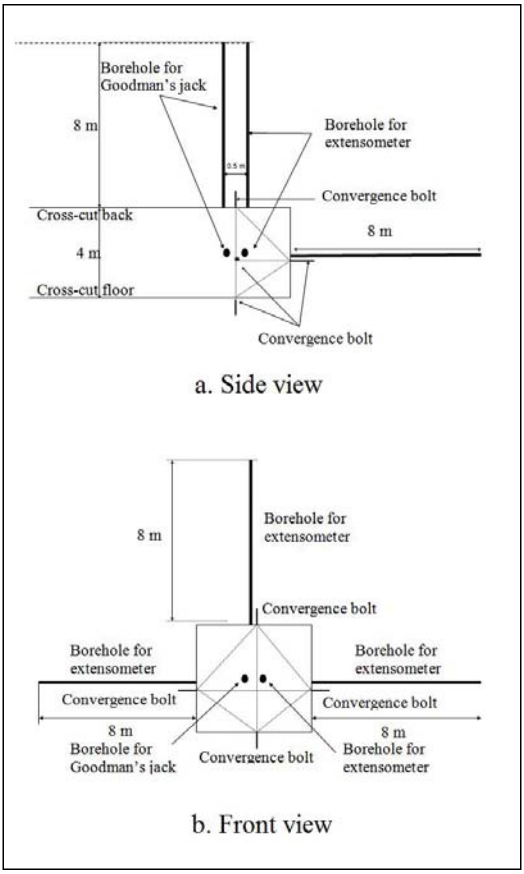


Fig. 4. Installation of displacement monitoring devices

As there were stoping activities underneath the research location, the displacement monitoring were obviously not conducted in a constant stress environment. Consequently, the stress changes had to be quantified and taken into account in the construction of the rheological model of the andesitic-breccia rock mass.

The stress changes quantification was carried out by utilising three dimensional numerical modelling. The modelling revealed that induced stress in the research location increased as the stope advancing toward the location and became constant after the stoping passed the location. Referring to this phenomenon, the appropriate time period for the construction of the rheological model was then decided, as depicted in Figure 5.

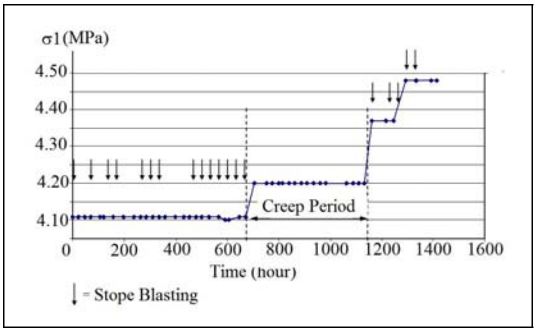


Fig. 5. An example of modelled induced stress changes

Furthermore, the Burger rheological model has been found most suitable for the Pongkor andesitic-breccia. The rheological model for the rock mass is as follows:

$$\varepsilon(t) = \frac{\sigma}{E_2} + \frac{\sigma}{E_1} - \frac{\sigma}{E_1} e^{-(E_1 t/3\eta_1)} + \frac{\sigma}{3\eta_2} t \qquad (8)$$

where all the rheological model parameters are given in Table 1.

Table 1. Rheological model parameters

| Sidewall | $E_1$ GPa | E2 GPa | η1 GPa.hour | η2 GPa.hour |
|---|---|---|---|---|
| Right | 12.70 | 13.90 | 7.29e2 | 8.64e3 |
| Left | 8.92 | 5.39 | 3.88e2 | 7.00e3 |

Equation (8) can be written in the form of E(t) and by using the rheological model parameters in Table 1, the E(t) curves for rock masses in the right and left sidewalls could be constructed. The starting point of the asymptotic part of the curve was then taken as the rock mass long-term modulus of deformation ($E_{LT}$), which in average was 2.53 GPa.

Using this value of $E_{LT}$, the estimated long-term strength of the Pongkor andesitic-breccia is likely between 9.03 MPa and 22.02 MPa, which is 10%-25% of the $\sigma_{ci}$.

## 4. Discussion

The rock mass strength estimated in this work is 25% of the intact rock strength. This fact clearly shows that rock mass strength is controlled by the interlocking of rock blocks and the condition of the surfaces between these blocks, which are represented by the GSI. Extensive laboratory tests and field studies on excellent quality Lac du Bonnet granite [11] suggested that the *in situ* strength of this rock is only about 70% of that measured in the laboratory.

Rock mass modulus of deformation measured in this work is 47% of that obtained in the laboratory. Although it is in line with the review carried out by Mohammad et al. [10], further investigation of factors affected the Goodman's jack test, in particular the induced stresses, is still required.

The long-term strength of Pongkor andesitic-breccia might be only 10%-25% of the intact rock strength. Undoubtedly, if this value is taken into consideration in the design, much conservative design will be recommended and support costs will be much higher. This fact, however, could advise the mining that after some years, the rock mass strength will decrease and some support repair will be required.

## 5. Conclusions

A method of estimating rock mass long-term strength has been developed. It contains laboratory test, *in situ* test and measurement, and numerical modelling.

The method has not been verified, as there is no rock mass failure case history that can be used to back analyse the rock mass strength. Once the failure is available, refinement of the approach used in the development of the method must be conducted.

**Acknowledgments**

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