Evaluation of Spalling Fallout on Excavation Disturbed Zone under Deep Hard Rock Tunnel

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Abstract. The prediction of compressive stress-induced failures is of concern when designing and constructing facilities in rock for deep underground excavation. The purpose of this study is to model compressive stress-induced failure and fallouts with appropriate material models and strength parameters for deep hard rock tunnel excavation. Three method of numerical modelling are used, which are Generalised Hoek-Brown; Mohr-Coulomb; and Mohr-Coulomb with Cohesion Softening Friction Hardening (CSFH) material models for capturing the observed rock behaviour. A parametric study was also carried out to verify that the peak friction angle of 10° used in CSFH model. The results show that numerical models used only Generalised Hoek-Brown and Mohr Coulomb strength parameters does not show a good agreement with the observed fallout. The comparison revealed that the numerical models using the Mohr-Coulomb with CSFH provides most realistic to the observation fallout length. This model is valid for prediction of failure and fallouts in hard rock masses with high quality (GSI >65 MPa; intact rock compressive strength >70MPa).

1. Introduction

The excavation of deep underground opening disturbs the primary stress state, leading to stress redistribution and deformation of rock mass around the excavation boundary. The design concept applied to underground excavation relies on two major mechanisms, which are the in-situ stresses and the deformation. However, the mechanical behavior and deformation mechanisms of the rock under high in-situ stress are different from those in low stress conditions. Overstressing is usually happened due to the overburden induced stress. The spalling fallout due overstressed rock can significantly influence the overall performance of an underground excavation. Hence, research on excavation disturbed zones (EDZ) in deep underground excavation is becoming increasingly important for understanding the mechanical behavior of rock masses in underground design and construction.

The EDZ is defined as the zone immediately around the excavation boundary where these damage processes strongly affect physical, mechanical and hydraulic properties of the rock. Extensive studies have been performed to understand and predict the extent of EDZ, and in recent years advances have been made in the understanding of the formation mechanism of EDZ [1]–[3]. It is generally accepted that in high in situ stress conditions the excavation induced stress redistribution is the main cause for
the formation of EDZ, which plays a more important role on the extent of EDZ than that of the excavation method.

Fallout or rock burst is the explosive failure of the rock surface which occurs when there is very large induced stress around the excavation opening [4], [5]. It is occurs due to the instability of brittle and massive rock, caused by the continuously overstressing [6], [7]. The sudden rock burst will impose large dynamic loading towards the tunnel which is subjected by significant overburden stress [8], [9]. For instance, the rock burst was encountered in Pahang-Selangor tunnel at 1040 m of overburden, with followed by spalling of rock from the tunnel walls [10]. Hence, the excavation in brittle rock at great depth may encounter problems such as non-violent spalling or intense rock burst.

Brittleness is a characteristic of many geomaterials in which the pre-existing heterogeneities among the mechanical and geometrical properties of the constituent materials. It is one of the most important concerns for the pre and post construction of tunnel, as it might lead to rock burst or formation of break out zone if no proper supports are used to reduce the rock’s brittleness [11]. This study is carried out to understand more about the fallout in brittle rock when the excavation is located at deep underground and the effect of the EDZ to the surrounding opening by using finite element method.

2. Background of the study area
The Pahang–Selangor Raw Water Transfer Project includes the excavation of a 44.6 km-long with 5.2 m diameter circular tunnel. The water transfer tunnel project is located in the central area of Peninsular Malaysia (see Fig. 1.). The tunnel crosses the Titiwangsa Range, the main range of Peninsular Malaysia. The highest peak of this range is 2,183 m, and the tunnel route is approximately 1,350 m above sea level. The tunnel excavation used three TBMs (TBM-1, TBM-2, and TBM-3) for about 35 km of the whole tunnel length with a maximum depth of approximately 1,200 m.

![Fig. 1. Tunnel structure of Pahang Selangor Raw Water Transfer](image)

There are three main types of granitic rocks along the transfer tunnel consist of Kuala Lumpur Granite, Genting Sempah Microgranite and Bukit Tinggi Granite. TBM-1 was employed to completely excavate Bukit Tinggi granite, which is porphyritic and coarse grained, mostly fresh to slightly decomposed, and cut by the Krau Fault. TBM-2 was employed for all three granite types, with 75% of the drive being in Genting Sempah micro-granite and passing through two major faults: the Bukit
Tinggi Fault and Lepoh Fault. TBM-3 was employed for Kuala Lumpur granite, with small sections of Hawthorndon schist and several faults, such as the Tekali Fault and Kongkoi Fault [12].

3. Methods
This section describes the application of laboratory test which use to indicate the mechanical properties of the intact rock core specimens. The rock mass that use for coring is from the construction site of Pahang-Selangor raw water transfer project. The right method of sample preparation is important because it will affect all test results. Therefore, the specimen preparation must be done consistently and in the correct manner to assure quality data. The method of drilling out the intact rock core specimens from the rock mass is by following the procedure of the raw cored rock sample. Based on the experiment data for the mechanical properties of rock, conditions of the underground excavation will be simulated in the FEM software for the further analysis.

3.1. Laboratory test
Laboratory tests have provide mechanical properties of the intact rock core specimens. The rock mass that use for coring is from the construction site of Pahang-Selangor raw water transfer project. The right method of sample preparation is important because it will affect all test results. All of the experiments have been done by fully complying ASTM standard. The mechanical properties of those rock samples include density test, ultrasonic pulse velocity test, unconfined compression test and Poisson ratio test.

The major purpose of performing the density test is to obtain the dry density of the rock samples then for the ultrasonic pulse velocity test is to determine the Elastic Modulus of the rock samples. For unconfined compression test, this is to determine the ultimate stress that can be withstand by the rock sample. Poisson ratio test is to determine the coefficient of deformation from the rock samples.

3.2. Rock mass strength estimation
Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for the design of underground excavations. The most common ways of determining the rock mass strength is by failure criteria.

3.2.1 The Generalized Hoek-Brown criterion. The failure criterion which establishes the strength of rock in terms of major and minor principal stresses was predicts strength envelopes that agree well with values determined from laboratory triaxial tests of intact rock. However, the intact rock core specimen does not represent the rock mass which generally contains joints, discontinuities and quality of rock. Thus, Hoek has done research about the failure criterion of intact rock through lab experiment while Brown studied the jointed rock mass changes in model analysis. The failure criterion of rock started from the physical properties of rock by introducing aspects to reduce these intact rock properties into data that can represent the whole actual rock mass characteristics and condition (Hoek et al., 2002).

The Generalized Hoek-Brown criterion is non-linear and relates the major and minor effective principal stresses as equation below:

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

where $\sigma'_1$ is a major effective principal stress at the failure; $\sigma'_3$ is minor effective principal stress at failure; $\sigma_{ci}$ is uniaxial compressive strength of the intact rock material; $m_b$ is a reduced value of the material constant; and $m$, $s$ and $a$ are constants which depend upon the characteristics of the rock mass.

In most cases it is practically impossible to carry out triaxial tests on rock masses at a scale which is necessary to obtain direct values of the parameters in the Generalized Hoek-Brown equation.
Therefore some practical means of estimating the material constant $m$, $b$, $s$ and $a$ are required (Hoek et al., 2002).

3.2.2. Mohr-Coulomb Failure Criterion. The Mohr–Coulomb failure criterion represents the linear envelope that is obtained from a plot of the shear strength of a material versus the applied normal stress. This relation is expressed as:

$$\tau = \sigma \tan(\theta) + c$$

where $\tau$ is shear strength; $\sigma$ is normal stress; $\theta$ is friction angle; and $C$ is cohesion. The equivalent Mohr–Coulomb parameters, rock mass cohesion ($c$), and friction angle ($\theta$), can be obtained based on the Hoek–Brown envelope and a chosen a range of $\sigma_3$. In $\sigma_1 - \sigma_3$ space, the Mohr–Coulomb failure criterion is expressed as equation below:

$$\sigma_1 = \frac{2c \cos \theta}{1 - \sin \theta} + \frac{4 + \sin \theta}{1 - \sin \theta} \sigma_3$$

where $(2c \cos \theta)/(1 - \sin \theta)$ is the unconfined compressive strength of the rock mass and $(1 - \sin \theta)/(1 - \sin \theta)$ is the slope of the failure envelope.

For deep tunnels, the maximum confining level ($\sigma_3_{max}$) from the following equation:

$$\frac{\sigma_3}{\sigma_{cm}} = 0.47 \left( \frac{\sigma_{cm}}{\gamma H} \right)^{-0.94}$$

Where $\sigma_{cm}$ is the rock mass strength, $\gamma$ is the unit of weight of the rock mass, and $H$ is the depth of the tunnel below the surface.

3.2.3. Cohesion-Softening Friction-Hardening (CSFH). The CSFH, a peak friction angle ($\theta_{peak}$) of $10^\circ$ was used (Edelbro, 2009) and the residual cohesion ($c_{res}$) was set equal to 30% of the rock mass cohesion ($c_m$), while the residual friction angle ($\theta_r$) was set equal to the friction angle of the rock mass ($\theta_m$). The peak cohesion ($c_{peak}$) was determined using the Mohr-Coulomb criterion

$$c_{peak} = \frac{\sigma_{cl}(1 - \sin \theta_m)}{2 \cos \theta_m}$$

Where $c_{peak}$ is peak cohesion; $\theta_m$ is friction angle of the rock mass; and $\sigma_{cl}$ is uniaxial compressive strength of the intact rock material.

3.3. Numerical modelling

For numerical modelling method, a "circle domain" Fig. 2 was used in order to decrease the number of elements compared to a rectangular shaped domain and thus enable a finer discretization in the region closest to the boundary. The extent of the modelled domain was defined by an expansion factor of four in relation to the excavation dimension, to eliminate boundary effects. The domain was discretized with a finite element mesh of six-noded, triangular elements. A mesh gradation factor of 0.1 was used. The mesh setup in Phase2 is separated concerning element size on the excavation boundary.
(discretization density) and element sizes within the model domain (mesh density). The selection of regions with finer discretization was in this work based on results of elastic models where the extent and depth of regions with a strength factor (safety factor) less than one was identified. To avoid an abrupt change of element sizes in regions near the finer discretization region, a smooth change of element size was applied. The element size at the tunnel boundary (in the region of the predicted failure) was set to 0.01 m. In regions outside the predicted failed zone, the element side length was set to 0.2 m (except for close to the predicted failure, where it was set to 0.02 m). Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for

![Fig. 2. Circle domain used for the modelling analysis](image)

4. Result and discussion
The laboratory test results describe the mechanical properties of those rock samples. The result of physical properties of rock which have been tested through experiments for further analysis procedure are being compiled as Table 1 below:

| Sample | UCS (MPa) | Modulus of Deformation (MPa) | Unit Weight (kN/m³) | Poisson Ratio |
|--------|-----------|-------------------------------|---------------------|--------------|
| 1      | 118       | 54450                         | 2650                | 0.2          |
| 2      | 124       |                               |                     |              |
| 3      | 126       |                               |                     |              |

With the data obtained from laboratory test results of the intact rock samples, the analysis for strength properties had been carried out using RocData (Rocscience Inc. 2016). The material strength properties determined by RocData have been used as input for numerical analysis. The results are as shown in Table 2 below:
Table 2: Material strength properties

| Sample | Generalized Hoek-Brown | Mohr Coulomb | Cohesion Softening Friction Hardening |
|--------|-------------------------|--------------|--------------------------------------|
|        | mb | s | a | Cohesion C (Mpa) | Friction Angle ∅ | Peak Friction Angle ∅_{peak} (°) | Residual Friction Angle ∅_{res} (°) | Peak Cohesion, C_{peak} (Mpa) | Residual Cohesion, C_{res} (Mpa) |
| 1      | 10.57 | 0.03 | 0.50 | 6.85 | 53.68 | 10 | 53.68 | 23.12 | 6.85 |
| 2      | 10.96 | 0.03 | 0.50 | 6.38 | 52.46 | 10 | 52.53 | 22.12 | 5.35 |
| 3      | 7.40 | 0.01 | 0.50 | 5.38 | 50.47 | 10 | 45.11 | 19.41 | 5.09 |

4.1. Numerical modelling

A comparison of fallout depth with difference material models from the numerical analysis using programs RS² (Phase² 9.0) are shown in Fig. 3. The results obtained amongst all the modelling method used, CSFH shows the closest fallout length when compared with the observed value. Generalised Hoek-Brown Criterion and Mohr-Coulomb Failure Criterion are over-estimate the fallout length because it assumes that rock behaves perfectly elastic once its’ strain exceeded the yield point.

![Fig. 3. Result of fallout of Sample 1 using: a) Generalised Hoek-Brown Criterion, b) Mohr-Coulomb Failure Criterion and c) CSFH material method.](image)

We can deduce that Generalised Hoek-Brown Criterion and Mohr Coulomb Failure Criterion don’t show a good agreement result with the observed fallout but CSFH method has better agreement with the observed fallout. A CSFH material model can be used for prediction of compressive stress-induced fallouts. The results obtained from three different methods of modelling are compared to the observed fallout as shown Table 3 below:

Table 3: Comparing Modelling Result with Observed Fallout

| Sample | Generalised Hoek-Brown (m) | Mohr-Coulomb (m) | CSFH (m) | Observed Fallout (m) |
|--------|---------------------------|------------------|----------|----------------------|
| 1      | 0.518                     | 0.518            | 0.313    | 0.310                |
| 2      | 0.477                     | 0.475            | 0.312    | 0.300                |
| 3      | 0.470                     | 0.467            | 0.285    | 0.295                |

The result of the parametric study of the peak friction angle (∅_{peak}) used for Cohesion Softening Friction Hardening (CSFH) is also presented in Fig. 4.
Fig 4. Result of analysed and observed fallout of Sample 1, 2 and 3 on different peak cohesion
Referring to the graphs above, we can conclude that for all the three samples, at the peak friction angle (\(\phi_{\text{peak}}\)) of 10°, they showed good agreement with field observations as suggested by Hajiabdolmajid, 2003. A lower angle resulted in a too wide and too deep failed zone while a higher angle underestimated the fallout.

5. Conclusions
The intact rock parameters which were determined from the lab experiments have proven that both the rock types are strong rock with consisted high compressive strength and generally fresh, slightly weathered. The Poisson ratio of 0.2 has proven that the rock experience low strain changed when load was applied. Hence, the properties of the both rock had been successfully achieved. In the analysis process, A CSFH material model can be used for prediction of compressive stress-induced failouts. This model is valid for prediction of failure and failouts in hard rock masses with high quality (GSI>65; intact rock compressive strength>70 MPa). Generalised Hoek-Brown Criterion and Mohr Coulomb don’t show a good agreement result with the observed failouts. A peak cohesion representative of the intact rock strength in combination with the brittle-plastic CSFH model resulted in the best agreement with observed failouts. Moreover a peak friction angle (\(\phi_{\text{peak}}\)) of 10° showed good agreement with field observations. A lower angle resulted in a too wide and too deep failed zone while a higher angle underestimated the failout.

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