https://doi.org/10.48047/AFJBS.6.13.2024.7326-7338

AFJBS African Journal of Biological Sciences

Journal homepage: http://www.afjbs.com

Research Paper Open Access

ANALYSIS OF TUNNEL EXCAVATION IN THE FLYSCH FORMATION USING HOEK-BROWN STRENGTH CRITERION

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Volume 6, Issue 13, July 2024

Received: 15 June 2024

Accepted: 05 July 2024

Published: 25 July 2024

doi: 10.48047/AFJBS.6.13.2024.7326-7338

ABSTRACT

The aim of this paper is to determine the geotechnical properties of clastic sedimentary rocks and especially sandstone which constitute a great part of the flysch formation. Laboratory tests were conducted in samples collected from a tunnel site (near portal, depth up to 30-40m) in Manipur state, India. Physical and mechanical properties like, dry density (ρ_d), sound velocity (V_p), Brazilian tensile strength (σ_t), point loading strength (I_{s50}) and uniaxial compressive strength (σ_{ci}), were determined as per the relevant Indian standards. Additionally, the material constant (m_i) and uniaxial compressive strength (σ_{ci}) of intact rock were determined by fitting method in the RocDatav5.0 software. Further, the effect of geological strength index (*GSI*) and disturbance factor (*D*) on rock mass strength or minor principal stress(σ_1) for particular given value confining stress (σ_3) were investigated. To illustrate the advantage of Hoek-brown strength criterion, a tunnel is analysis is involved with numerical analysis with Phase^{2}v9.0 software Hoek-Brown (HB) strength criterion followed the non-linear failure envelope, which is nearly applicable for all types of rock mass with the link of *GSI*. Therefore, HB strength criterion can predict the accurate deformation and plastic zone formation of surrounding rock mass for the tunnel excavation.

Keywords: Sandstone, Hoek-Brown Criterion, Geological Strength Index, Disturbance Factor, Tunnel Excavation

1. INTRODUCTION

Analysis of a variety of problems in rock engineering requires determination of rock mass strength parameters. There are various empirical failure models available in literature, which estimate the failure envelope of rock and rock masses (Hoek and Brown, 1980; Yudhbir et al., 1983; Shorey et al., 1989; Yoshida et al., 1990; Ramamurthy, 2001). Among these the Hoek-Brown (HB) strength criterion is the most well-known, and the most frequently used to a wide range of rock engineering applications (Cai, 2010, Sari, 2012). The HB strength criterion especially used for determination of parameters of equivalent linear and non-linear strength envelope for intact rock material and rock masses (Hoek and Brown, 1997).

Therefore, in this paper, HB strength envelope (defined by σ_{ci} and m_i) has been developed using multiple data such as uniaxial tensile strength, compressive and triaxial compressive strength values. The original method of fitting the HB failure criterion using the spread sheet for linear. regression (Hoek and Brown, 1997) and the fitting method (Levenberg-Marquardt) utilized in the RocData v5.0 software (Rocscience Inc., 2014), were used to determine the intact rock parameters (σ_{ci} and m_i). Further, the parametric effect on rock mass strength using new HB strength criterion is defined by Hoek et al., 2002, popularly known as Generalised Hoek-Brown (GHB) failure criterion for isotropic rock mass; are studied. The three HB constants used in this criterion (mb, s, a) are function of geological strength index (*GSI*) classification system (Hoek and Morinos, 2007), the D factor and intact Hoek-Brown curve (defined by σ_{ci} and m_i). Finally, the application of HB failure criterion for the numerical analysis with Phase^2 v9.0 software (Rocscience Inc. 2014) of deep buried rock tunnel was carried out. The total deformation and radius of plastic zone formation around the tunnel has been studied for different rock mass geological condition. The first step in developing a HB rock mass strength envelope is to determine the intact strength envelope based on laboratory test results. There are two intact rock parameters requires for the HB strength criterion for intact rock including, constant m_i and σ_{ci}. These two intact rock parameters can be derived from laboratory testing data including σ_t, σ_{ci} and conventional triaxial strength ($\sigma_1 > \sigma_3 = \sigma_2$). The HB strength criterion initially developed by using large numbers of triaxial data on intact rock. samples using range of triaxial data $0 < \sigma_3 < 0.5\sigma_{ci}$. Hoek and Brown (1980) suggested that at least five well-spaced data points should be included in the analysis. The HB strength failure envelope mainly containing the yield value of strength in tension and compression (Sari, 2012). Hence, it is very difficult to fitting multiple data over whole range of loading conditions, including tensile, could signal that the nature of the failure mechanism change as the minimum principal stress moves from tension to high compression (Cai, 2010). Therefore, one must first develop an intact envelope based on tensile (direct or Brazilian tensile strength test), uniaxial compressive and triaxial laboratory test results. Lade (1993) state that it may advantageous to include the tensile strength values in estimation of material parameters as such that data gives good control on the failure envelope over low stress range. A comprehensive analysis by Douglas (2002) of large database of test results highlights the fact that there is inadequacy in determination of HB empirical failure parameters as currently proposed for intact rock. Hence, there are various regressions or curve fittings technique is frequently used in rock engineering applications to determine the mean intact HB strength envelope (Douglas, 2002). Shah and Hoek (1992) have found that the simplex reflection technique is better for fitting laboratory strength data to non-linear HB failure criterion than ordinary least squares regression. Sari (2012) has been proposed the best fit with inclusion of uniaxial tensile and compressive strength combining with triaxial data for low confining range and found that the modified non-linear curve fitting (used in his paper) to be the most suitable procedure for estimating the HB parameters for the Ankara andesite. Langford and Diederichs (2015) have proposed a new set of linear and non-linear regression approaches for the mean intact strength envelope and quantify the error due to obtained uncertainty from test data.

2. METHODOLOGY

2.1 HB STRENGTH CRITERION

The Hoek-Brown (HB) criterion (Eq. 1), which was initially proposed by Hoek and Brown (1980) for estimating the intact rock strength. Hoek and Brown (1980, updated 1997) proposed a failure criterion applicable to isotropic and homogenous intact rocks, confirming to the non-linear response of strength with confining pressure and is represented as (Eq. 1):

$$\left(\sigma_1-\sigma_3\right)=(m_i\sigma_{ci}\sigma_3+s\sigma_{ci}{}^2)^{0.5}$$

(1)

where, m_i and s are the rock material constant; σ_1 and σ_3 are major and minor principal stresses at failure; σ_{ci} is the *UCS* of intact rock. Two components of m_i and s are known as material constants which are varied corresponding to rock nature and rock mass quality. Consequently, variable of s for intact rock material would be determined (Ramamurthy, 2001, Hoek et al., 2002). It should be noted that component of material constant m_i and UCS can be estimated based upon analysis of a laboratory results on intact cylinder specimens assuming the s=1 for intact rocks.

Further, it was extended estimate the rock mass strength by using geological strength index (GSI) and a disturbance factor D to reduce the intact rock properties (Hoek et al., 2002). This empirical failure criterion describes a non-linear relationship of rock mass strength in terms of effective principal stress (σ_1, σ_3) as indicated through the following equation:

$$\left(\sigma_1-\sigma_3\right)=(m_b\frac{\sigma_3}{\sigma_{ci}}+s\sigma_{ci}{}^2)^a \qquad (2)$$

where, σ_1 and σ_3 are the major and minor principal stresses at failure, σ_{ci} is the UCS of the intact rock.

The various parameters derived from the laboratory tests on intact rock and the strength envelope of this criterion. A set of empirical expressions, associated with a series of observed rock mass failure in field constitutes the “GHB failure criterion parameters” (m_b, s and a). Moreover, the specific parameters obtained from analysis of laboratory test results include σ_{ci}, m_i, *GSI* and *D* (Hoek et al., 2002). The variable of m_b lowers m_i value by a reducing factor to indicate field condition influences such that shown in equation below (Eq. 3). The value of m_b as per equation 3a, whereas; s and a are fixed constants for particular rock mass, which can be calculate according to equation (3b) and (3b) respectively.

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{3a}$$

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

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{3c}$$

The disturbance Factor (*D*) is depending upon the degree of disturbance during construction to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses i.e. intact rocks to 1 for very disturbed rock masses i.e. highly fissured rock masses (Hoek et al. 2002).

2.2 PHYSICAL AND MECHANICAL PROPERTIES OF SANDSTONE

The laboratory tests were carried out on relatively homogeneous and isotropic sandstone rock specimens. Tests were conducted in dry condition to avoid the water content variation effects which further provide better comparison of the results. The laboratory tests include: dry density (ρ_d), sound velocity (V_p), Brazilian tensile strength (σ_t), point loading strength (I_{s50}), uniaxial compressive strength (σ_{ci}) and conventional triaxial strengths for m_i determination. All laboratory tests cited were carried out according to Internal Society for Rock Mechanics (ISRM) suggested methods (Brown, 1993). The test results are summarised in Table 1, including for each parameter the range of values, the mean value, standard deviation and number of specimens tested. The sandstone rock classified as CL based on laboratory results (Deere and Miller, 1966).

Table 1 Summary of laboratory test results of sandstone sample

Parameters	Minimum value (Min)	Maximum value (Max)	Mean value (Mean)	Standard deviation	Number of samples (N)
σ_{ci} (MPa)	44.11	84.89	64.50	42.49	8

σ_t (MPa)	5.11	7.66	6.21	2.05	5
I_{s50} (MPa)	2.37	2.91	2.64	0.44	5
V_p (m/s)	2350	3520	2915	1165	8
Dry density (g/cc)	2.48	2.55	2.52	0.072	8
E_i (MPa)	6281	12072	8724	5804	8

3. INTACT HB STRENGTH ENVELOPE

The first step in developing a rock mass strength envelope is to determine the intact strength envelope based on laboratory test results. To get the strength envelope for the intact rock major and minor principal stresses at failure, obtained from different tests (uniaxial tensile (Brazilian), uniaxial compressive and triaxial laboratory test) were plotted as shown in Fig. 1. Strength envelope as shown in Eq. (1), the relationship between the principal stresses is defined by the σ_{ci} and the m_i. The single strength envelope for intact rock material has been defined using σ_{ci} and m_i. The strength envelopes are shown in Fig. 1 and results of regression analysis or best fit are summarized in Table 2.

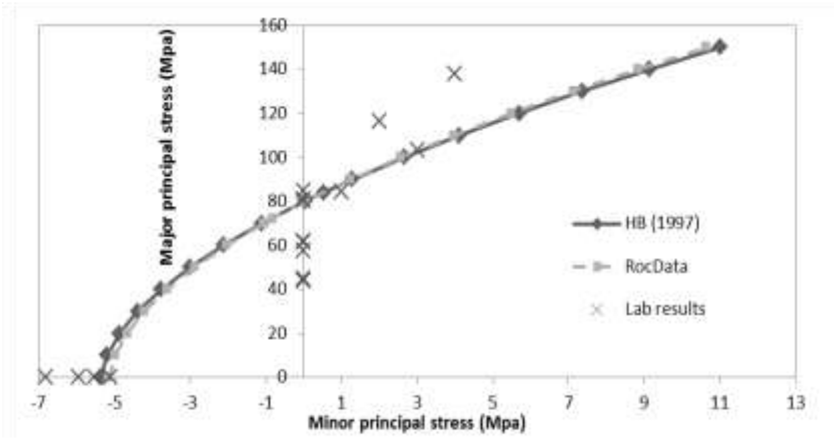

Figure 1 HB intact strength envelopes of sandstone where σ_3=σ_t and σ_1=0 for tensile strength data

The values of m_i for intact rock, varies from 4 for very fine clastic rocks like clay stone 4±2 to 21±3 for coarse conglomerates. Similarly, the m_i of the medium sandstone is equal to 17±4 (Hoek and Brown, 1997, Marinos and Hoek, 2000). In this m_i values as per Hoek and Brown (1997) is considered for numerical modelling with least sum of squared residual (SSR) values i.e. 1.0037.

Table 2 Summary of Hoek-Brown parameters of regression analysis where $\sigma_3 = \sigma_t$ and $\sigma_1 = 0$ for tensile data

Parameters	Hoek-Brown criteria (1997) *	RocData Software**	Morinos and Hoek (2000)
σ_{ci} (Mpa)	79.734	79.692	-
m_i	14.775	15.411	17±4
σ_t (Mpa)	-5.35	-5.15	-
SSR	1.0037	1.095	-

*Linear regression method
**Levenberg-Marquardt method

4. ROCK MASS STRENGTH

The rock mass strength has been determined using the generalised Hoek-Brown (GHB) (Hoek et al. 2002) strength criterion, which is applicable for isotropic rock mass and the strength envelope, depends upon on four independent quantities: *GSI*, *D*, m_i and σ_{ci} as showed in Eq. 2 and 3. In these quantities, the geological condition of rock masses in terms of geological strength index (GSI) classification system (Morinos, 2007; Morinos et al., 2007), is determined by analysing outcrops and borehole data and assigning a value based on qualitative descriptions of mass block and joint condition (Fig. 2).

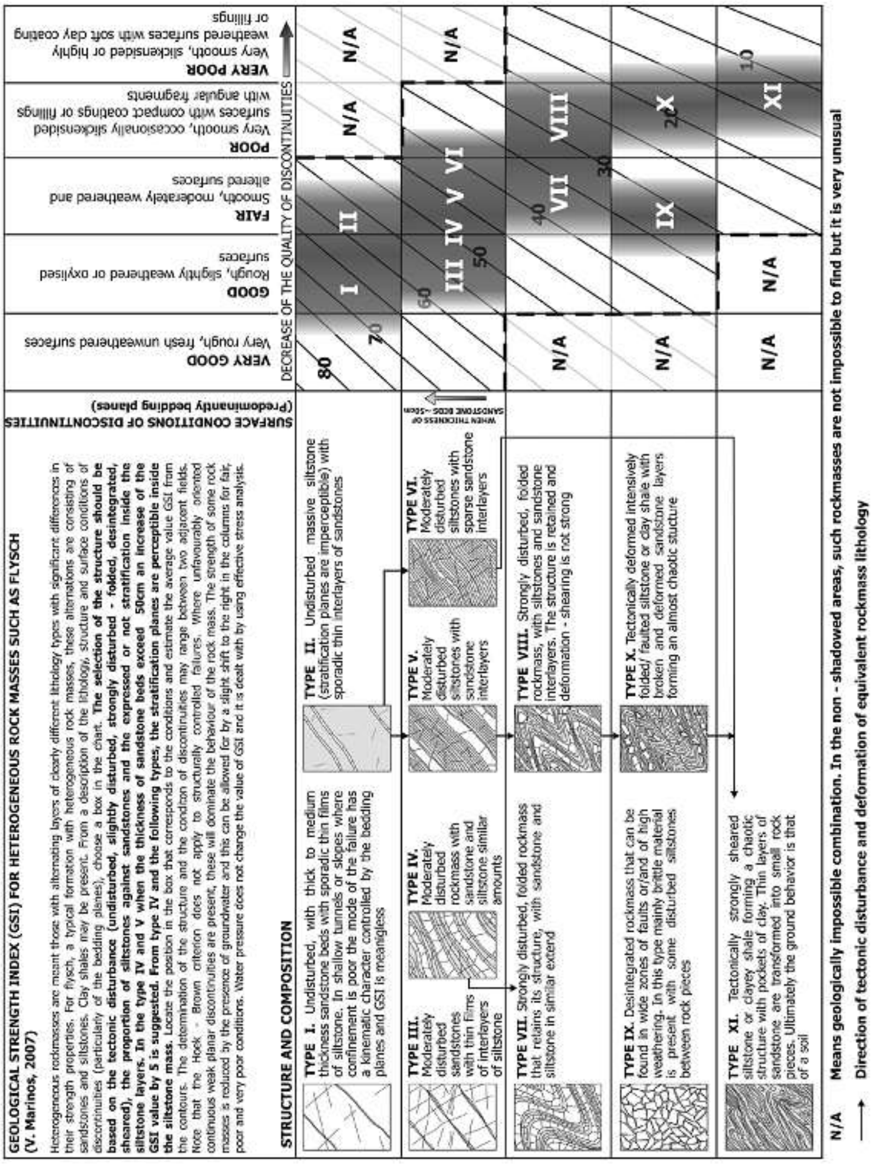

Figure 2 The new GSI chart for heterogonous rock mass such as flysch (Morinos et al., 2007)

Furthermore, the parameters m_i and σ_{ci} have been used for this study 14.8 and 79.8MPa respectively as Hoek and Brown (1980) and the effect of *D* on rock mass strength or minor principal stress (σ_1) with varying *GSI* (say; 20, 40, 60, 80) are studied. It is found that the effect of *D* is much greater when *GSI* is relatively small for worse quality rock mass then when *GSI* is big for better quality rock mass. The parameters *D* affect the halving value of the rock mass strength (σ_1) when the GSI is 20 as shown in Fig. 3. It is reminded that determining the value of *D* for fractured rock mass is key parameters the application of the HB strength criterion (Sah and Krishna, 2013). The reduced value of material constant m_b is directly affect the rock mass strength (σ_1) and also control the failure envelope of given rock mass.

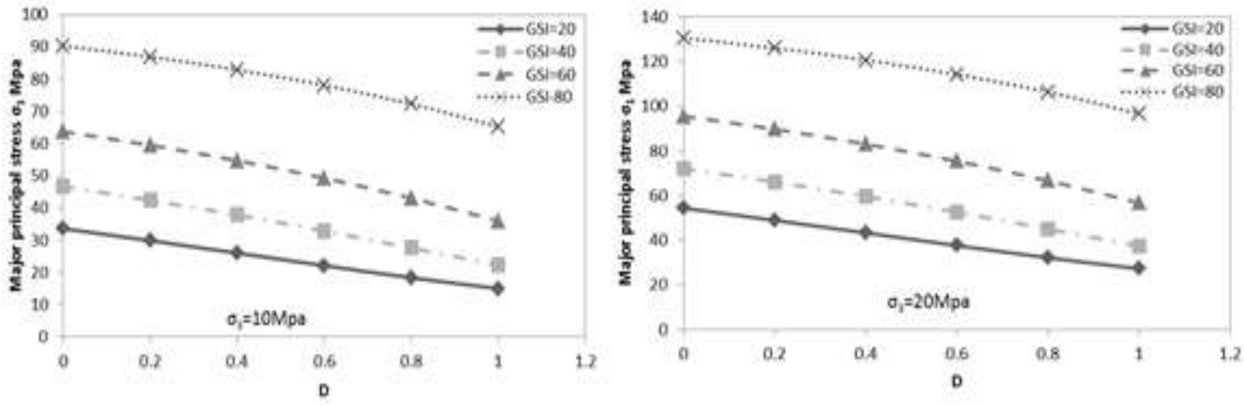

Figure 3 The effect of disturbance factor (D) on rock mass strength or major principal stress for σ_3=10MPa (left side) and σ_3=20MPa (right side)

5. APPLICATION OF HB STRENGTH CRITERION ON ROCK TUNNEL

The Phase2 6.0 software (Rocscience Inc. 2014) was selected for the numerical analysis on rock tunnel, very appealing reason that permits the direct usage of the HB strength criterion. It also features the option of using Mohr-Coulomb failure criterion, strength reduction after failure parameters, dilatancy in the failure zone, different excavation layouts, uniform or non-uniform field stress conditions and isotropic or non-isotropic properties. The summary of required input used in software including failure criterion, rock mass parameters, tunnel layout and excavation types etc. summarised in Table 3.

Table 3 Modelling Parameters used in numerical analysis

Failure criterion	Generalised HB strength criterion
Post-failure behaviour	Elastically-perfectly plastic
Elastic properties	Isotropic
GSI values	20, 40, 60, 80

σ_{ci}, m_i and E_i	79.8MPa, 14.8 and 8724 MPa
Rock mass deformation (E_{cm})	Generalised Hoek and Diederichs (2006)
Disturbance factor (D)	0-1
Depth of tunnel	150m
Loading type	Constant
Excavation section	Horse-shoe
Excavation	Full face

In the present study, *D* factor has been created around the tunnel boundary, which extends up to 3m in the rock mass from the boundary of the tunnel (Hoek, 2012; Singh et al., 2014). The result of numerical analysis has shown in Fig. 4 for rock mass condition GSI=40. The Fig, 4 demonstrated that average total displacement (*u*, in m) and radius of plastic zone (R_p, in m) are 0.0158 m and 4.87 m respectively.

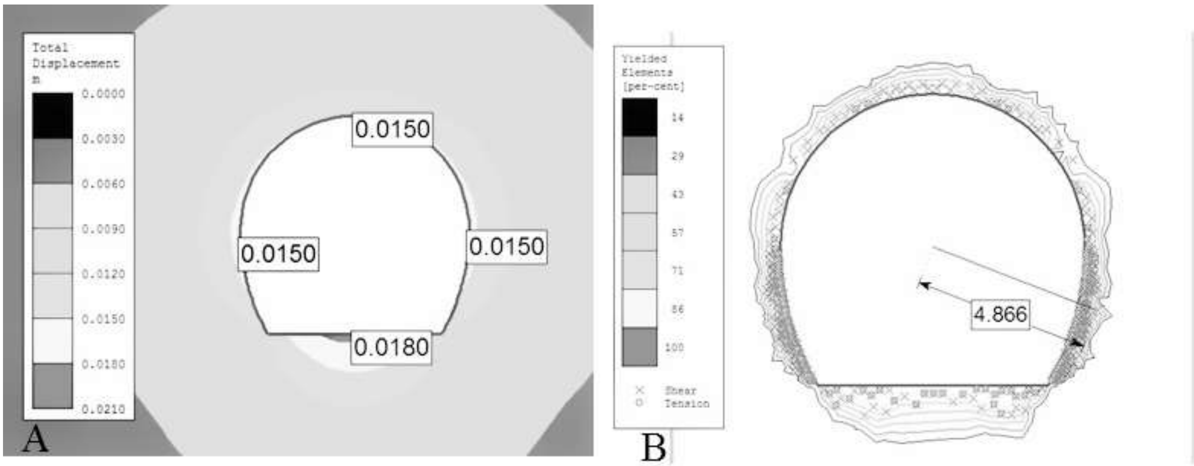

Figure 4 Typical analysis result of tunnel excavation in rock mass having *GSI*=40

Furthermore, the study has been explored to quantifying the effect of D on tunnelling in the various rock mass conditions. According to results presented in Fig, 5; the total displacement and radius of plastic zone decreases with increasing the D values. It is also found that the effect of *D* is much greater when *GSI* is relatively small for worse quality rock mass then when *GSI* is big for better quality rock mass (shown in Fig. 5).

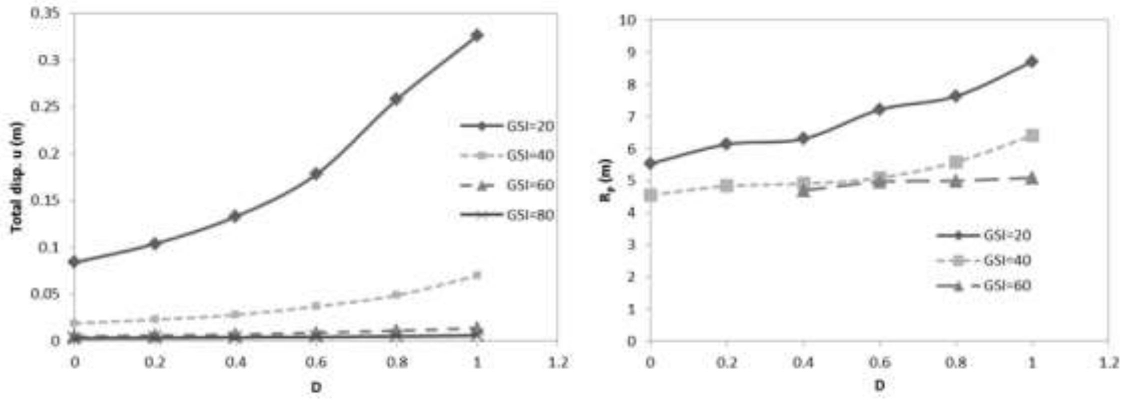

Figure 5 The variation of disturbance factor (*D*) versus total deformation (left side) and radius of plastic zone (right side) for different *GSI* values

6. CONCLUSIONS

The following conclusions from the paper were illustrated as follows:

(1) The HB strength criterion is more fitting for the material characteristics of rock and can also the geological condition for rock mass. With the non-linear expression, HB strength criterion response that the failure of rock with material is relative with the confining conditions. Accordingly, m_i and σ_{ci} were determined using two different curve fitting methods.

(2) The effects of parameters including *GSI* and *D* factor have been studied on rock mass strength or minor principal stress (σ_1). It is found that the effect of *D* is much greater when *GSI* is relatively small for worse quality rock mass then when *GSI* is big for better quality rock mass.

(3) Finally, the application of HB strength criterion was studied for rock tunnel. The numerical analysis is carried out using Phase^2 software, it found that, total displacement and radius of plastic zone decreases with increasing the D values. It is also found that the effect of *D* is much greater when *GSI* is relatively small for worse quality rock mass then when *GSI* is big for better quality rock mass.

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