Application of the Conventional and a New Failure Criteria in the Stability Analysis of Underground Structures

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ABSTRACT

There are three major failure criteria: the Mohr-Coulomb, the Bieniawski and the Hoek and Brown criteria which are widely used in the stability analysis of structures in rocks. The subject of special interest in this paper is to determine parameters attributing in each of the criteria based on the results obtained from laboratory triaxial tests, to discuss some problems associated with the application of these criteria and to suggest some approaches to determine the in-situ strength parameters using these criteria in finite element analysis (FEA) of underground structures. The research was carried out in two distinct parts. In the first part, a comprehensive laboratory testing was carried out to determine the mechanical properties of rocks in the strata units. The second part included FEA of an underground roadway using the results obtained from laboratory testings.

1.0 CONVENTIONAL FAILURE CRITERIA

The Mohr-Coulomb criterion defined in Eq. 1 postulates a linear relationship between confining pressure and compressive strength of rocks. To date, many investigations have proved that the internal angle of friction decreases by increasing the confining pressure and as such resulting in a reduction in the rock strength. Thus, the Mohr-Coulomb criterion leads to an over-estimate of rock strength. However, constants parameters C and N_ϕ, of this criterion can be easily determined from simple regression analysis of triaxial tests carried out under different confining pressures.

$$\sigma_1 = C + N_\phi . \sigma_3 \tag{1}$$

where,
σ_1 = major principal stress,
σ_3 = minor principal stress, and
C & $N\phi$ = constant parameters in the Mohr-Coulomb criterion.

The Bieniawski criterion is an empirical failure criterion expressed in Eq. 2.

$$\sigma_{1n} = 1 + n.(\sigma_{3n})^m \tag{2}$$

where,
σ_{1n} = normalised major stress,
σ_{3n} = normalised minor stress, and
n & m = constant parameters in the Bieniawski criterion.

Since m is usually less than 1.0, the term $(\sigma_{3n})^m$ will not have a real value if $\sigma_{3n} < 0$. Therefore, the application of this criterion to the stability analysis of underground structures is limited to the conditions where $\sigma_{3n} > 0$. Moreover, calculation of n and m parameters based on the triaxial test data requires an advance statistical program due to the power form of the equation.

The Hoek and Brown criterion is another empirical failure criterion which is widely used for rock engineering design purposes. This criterion as defined in Eq. 3 predicts a parabolic Mohr envelope for the rock strength.

$$\sigma_{1n} = \sigma_{3n} + \sqrt{m.\sigma_{3n} + s} \tag{3}$$

where,
σ_{1n} = normalised major stress = σ_1/σ_c ,
σ_{3n} = normalised minor stress = σ_3/σ_c ,
m & s = constant parameters in the Hoek and Brown criterion, and
σ_c = average uniaxial compressive strength of rock material.

Although a complete discussion on the derivation of this failure criterion has been given by Hoek and Brown (1980), there are still uncertainties about estimation of m and s parameters. Suggestions for m and s values for different rock groups by Hoek and Brown allocates many rocks in one group, i.e. all argillaceous rocks such as siltstone, mudstone, shale, etc are in a single group having same values for m and s. This means that these rocks have the same strength according to this classification. Above all, there is also a problem with the multi-regression analysis of fitting the curve by using this criterion which requires an advance statistical program.

2.0 THE NEW EXPERIMENTAL FAILURE CRITERION

Because of the problems involved in the conventional failure criteria, an attempt has been made during the course of this research to establish a new criterion. This new experimental criterion is expressed in Eq. 4.

$$\sigma_{1n} = K + (P + T \times \sigma_{3n})^{0.5} \tag{4}$$

where,

σ_{1n} = normalised major stress,

σ_{3n} = normalised minor stress, and

K, P & T = constant parameters in the Experimental criterion to be determined for each type of rocks.

The procedure for determining K, P & T parameters in the Experimental criterion is as follows:

- obtain the uniaxial compressive and uniaxial tensile strengths of the rock
- carry out at least two triaxial compressive tests under various confining pressures
- plot the above results in a coordinate system as shown in Fig. 1.

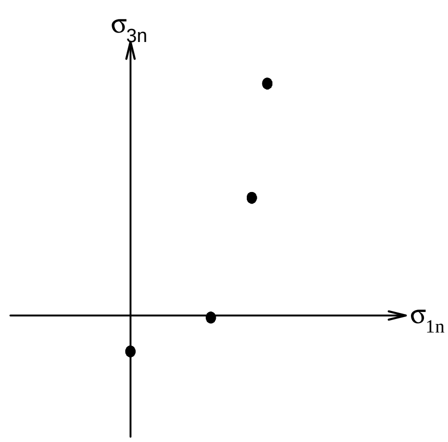


Fig. 1 - Plot of triaxial test results for the Experimental failure criterion.

- fit a polynomial equation like [$Y = a x^2 + b x + c$] to the obtained data, and determine a, b and c parameters. This can be done by using very simple programs such as Cricket Graph on PC or MAC computers.
- calculate K, P and T parameters as follows:

$$K = -\frac{b}{2a} \qquad P = K^2 - \frac{c}{a} \qquad T = \frac{1}{a}$$

Sensitivity analysis on the Experimental criterion showed that this criterion is sensitive to the K value and not much to P and T values. The author believes that this criterion is useful for the laboratory studies as well as full scale stability analysis of underground structures because of the following advantages:

(i) it presents a mathematical equation which considers non-linear relationship between triaxial strength and confining pressure,
(ii) it is applicable for all ranges of the confining pressures (σ_3),
(iii) its constant parameters can be easily determined by using a simple program, and
(iv) it gives a good agreement with measured values as it is indicated by the mean square values given in the corresponding Tables.

3.0 COMPARISON OF FAILURE CRITERIA DURING STABILITY ANALYSIS OF UNDERGROUND STRUCTURES

During the present research, six major rock types were encountered in the stability analysis of roadways and intersections at an underground coal mine in NSW, Australia. In order to determine various parameters of described failure criteria, appreciable number of samples were prepared and tested under various confining pressures. In this research, number of samples, preparation and testing procedure, and interpretation of the results conformed the ISRM standards. The statistical code, SAS, was used for regression analyses of fitting the Bieniawski, and the Hoek and Brown equations to the measured values. Results of this investigation are summarised in Tables 1 to 4.

Table 1 - Constant parameters after the Mohr-Coulomb criterion
($\sigma_1 = C + N_\phi . \sigma_3$)

Rock Type	UTS_i (MPa)	UCS_i (MPa)	N_ϕ	Mean Square (MPa)
Sandstone (m.g.)	4.66	65.5	3.179	0.01
sand f.g. + mud	5.40	44.3	4.115	1.66
sand c.g. + shale	3.60	53.0	4.295	1.11
Coal	1.27	23.0	4.766	3.55
Mudstone	6.80	30.0	5.371	3.30
Sandstone (c.g.)	4.66	75.0	4.228	0.34
Sandstone (m.g.)	4.56	65.5	3.179	0.01

The "mean of square value", which is a statistical index parameter showing the accuracy of fitting curve, was reasonably close to zero when fitting the Bieniawski and the proposed Experimental criteria curves to the measured values.

Table 2 - Constant parameters after the Bieniawski criterion
$(\sigma_{1n} = 1 + n.(\sigma_{3n})^{m})$

Rock Type	n	m	Mean Square (MPa)
Sandstone (m.g.)	3.278	1.018	0.00
sand f.g. + mud	3.360	0.766	0.12
sand c.g. + shale	2.631	0.591	0.01
Coal	4.058	0.585	0.13
Mudstone	4.112	0.591	0.23
Sandstone (c.g.)	1.932	0.571	0.00
Sandstone (m.g.)	3.278	1.018	0.00

Table 3 - Constant parameters after the Hoek and Brown criterion
$(\sigma_{1n} = \sigma_{3n} + \sqrt{m.\sigma_{3n} + s})$

Rock Type	m	s	Mean of Square (MPa)
Sandstone (m.g.)	5.25	1.0	0.20
sand f.g. + mud	11.81	1.0	0.36
sand c.g. + shale	10.52	1.0	0.34
Coal	17.63	1.0	0.60
Mudstone	19.15	1.0	0.70
Sandstone (c.g.)	8.33	1.0	0.28
Sandstone (m.g.)	5.25	1.0	0.20

Table 4 - Constant parameters after the Experimental criterion
$(\sigma_{1n} = K + (P + T \times \sigma_{3n})^{0.5})$

Rock Type	K	P	T	Mean Square (MPa)
Sandstone (m.g.)	0.190	0.577	7.610	0.07
sand f.g. + mud	-0.137	1.786	18.316	0.48
sand c.g. + shale	0.089	0.938	16.258	0.16
Coal	0.128	1.109	23.256	0.28
Mudstone	-0.530	3.069	30.303	0.65
Sandstone (c.g.)	0.185	0.766	9.259	0.11
Sandstone (m.g.)	0.190	0.577	7.610	0.07

The results obtained from different criteria were used in the stability analysis of a 2-D model of roadway. The strata column and dimension of the model are shown in Fig. 2 and mechanical properties of rocks are tabulated in Table 5. The model was analysed under 10.0 MPa vertical stress with the ratio of horizontal to vertical stress equal to 2.5. The major and minor principal stresses, σ_1 and σ_3, were obtained for all elements and a safety factor (SF) was then calculated based on each of the failure criteria as expressed in Eqs. 5 to 8:

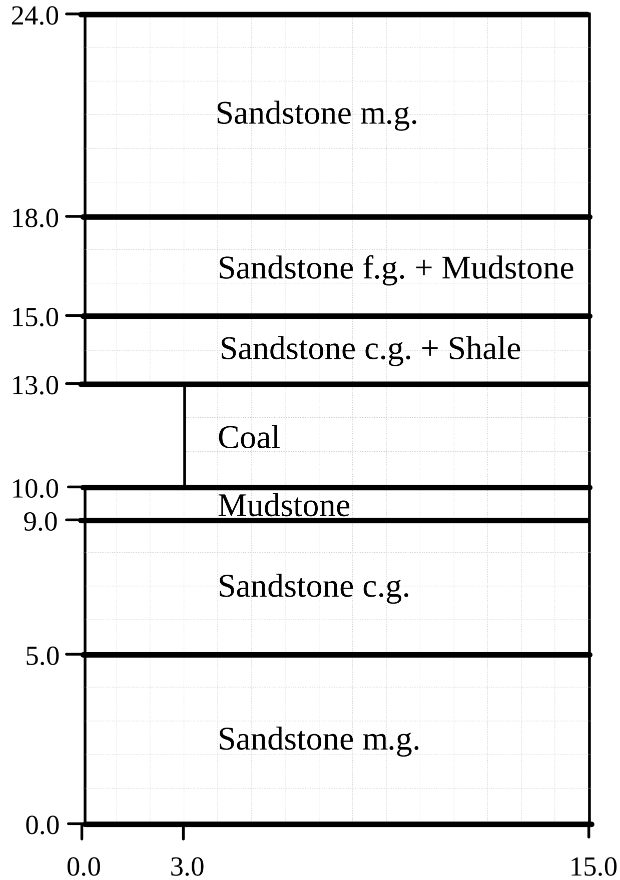


Fig. 2 - Strata column and dimension of the 2-D model of the roadway

Mohr-Coulomb

If $\sigma_3 > \tau_o$ Then $SF = \dfrac{C + N_\phi . \sigma_3}{\sigma_1}$ (5)

If $\sigma_3 < \tau_o$ Then $SF = 0$

Bieniawski

If $\sigma_3 > 0.0$ Then $SF = \dfrac{1 + n.(\sigma_{3n})^m}{\sigma_{1n}}$ (6)

If $\sigma_3 < 0.0$ Then $SF = 0$

Hoek and Brown

If $\sigma_3 > \frac{1}{2} C.(m - \sqrt{m^2 + 4s})$ Then $SF = \dfrac{\sigma_{3n} + \sqrt{m.\sigma_{3n} + s}}{\sigma_{1n}}$ (7)

If $\sigma_3 \leq \frac{1}{2} C.(m - \sqrt{m^2 + 4s})$ $SF = 0$

Proposed Experimental

If $\sigma 3 > To$ Then $SF = \dfrac{K + \sqrt{P + T \cdot \sigma_{3n}}}{\sigma_{1n}}$ (8)

If $\sigma_3 < T_o$ Then $SF = 0$

Table 5 - Mechanical properties of strata units used in stability analysis of a roadway

Strata units	Elastic Modulus (GPa)	Poisson's Ratio (--)
sandstone (m.g)	10.0	0.20
sand. f.g. + mudstone	7.0	0.25
sand. c.g. + shale	5.0	0.20
coal	3.5	0.30
mudstone	8.0	0.25
sandstone (c.g)	12.5	0.20
sandstone (m.g)	10.0	0.20

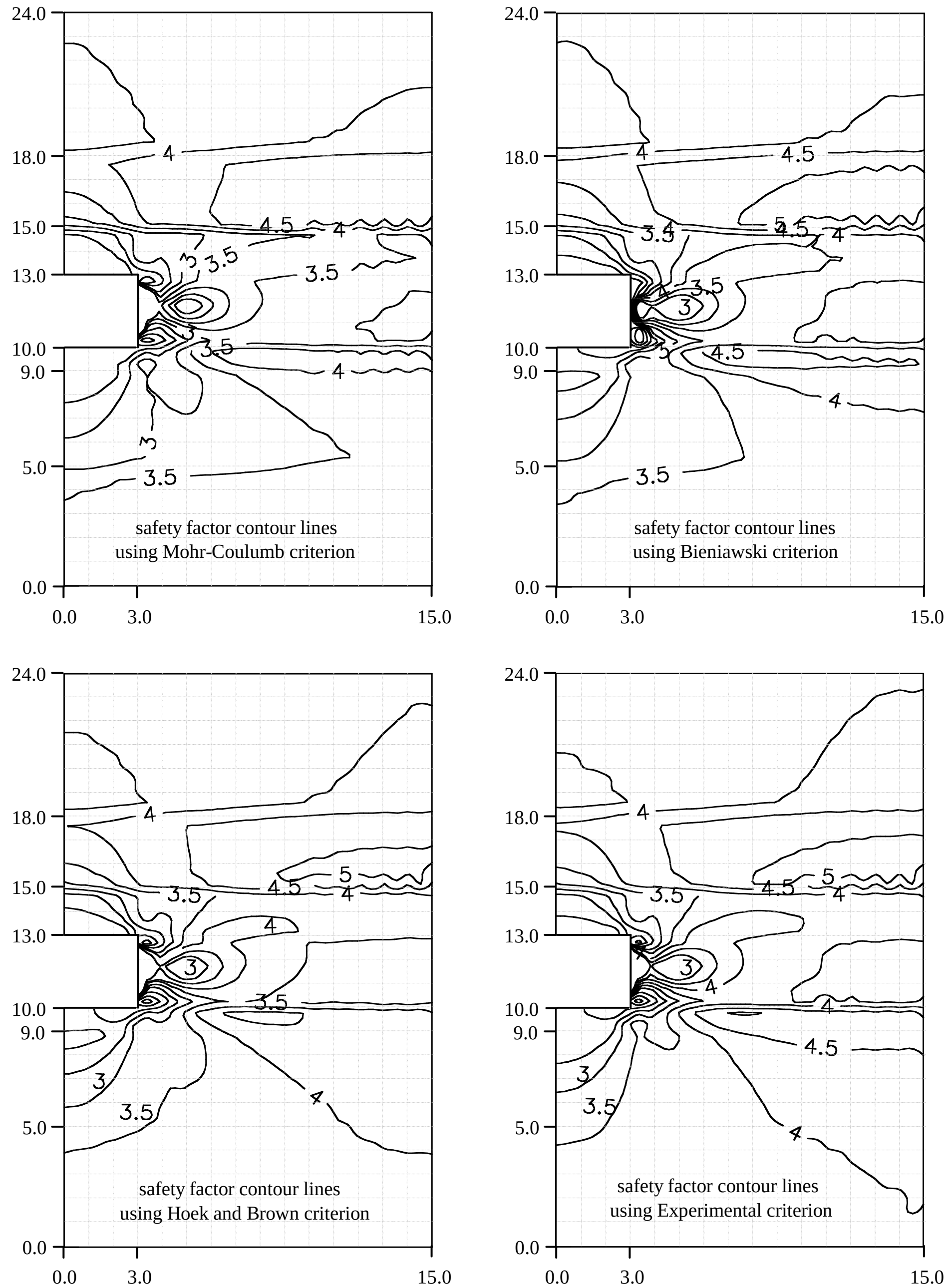
Results of this study are presented in Figs. 3 to 6. In these figures, the safety contour lines calculated after different failure criteria are plotted around the roadway.

Comparing the safety contour lines in Figs. 3 to 6, it can be seen that there is not much difference between the results obtained by applying different failure criteria as far as the laboratory tests results, intact parameters, are used. However, there are minor variations for locations very close to the opening.

4.0 SCALE EFFECT ON THE STRENGTH PROPERTIES OF LABORATORY RESULTS

There is always some concern that to what extent the test on small size samples is representative of large in-situ rock masses. Accordingly, this aspect of Rock Mechanic is reviewed here briefly to help making some correlations between the laboratory measured strength parameters and the in-situ values.

Literatures on this issue indicate that, although the size effect and in-situ properties have been the centre of the interest among mining engineers for last 30 years, there is not a standard approach to the key-function of scale effect. It is believed that the engineering judgement and local experiences are still basic considerations in selecting scale factors. There are a number of investigations on this issue. A summary of them is given below.



Figs. 3 to 6 - Safety factor contour lines calculated based on the Mohr-coulomb, Bieniawski, Hoek and Brown and Experimental criteria, respectively

The size and scale effect is generally considered by applying a reduction factor to the measured parameters such as uniaxial compression strength, cohesion, angle of internal friction, Young's Modulus and Poisson's ratio. The strength reduction from laboratory measured data to insitu values has received considerable attention in the past, particularly as related to mine pillar designs (Bieniawski, 1984). Many attempts have been made to explain this relationship by means of empirical relationships. Four of these works are pointed out here.

Protodyakonov (1964) expressed the relationship between the strength of specimen and the strength of in-situ rock mass in Equation 9.

$$\frac{\sigma_d}{\sigma_m} = \frac{\frac{d}{b} + m}{\frac{d}{b} + 1} \qquad (9)$$

where,

σ_d = strength of cubic specimen with side length of d,
σ_m = in-situ strength of the rock mass,
b = distance between discontinuities in the rock mass,
d = side length of the specimen, and
m = strength reduction factor, given in Table 6.

Table 6 - Strength reduction factor, m, in Eq. 9

Rock Strength(MPa)	Stress state Compression	Tension
> 75	2 < m < 5	5 < m < 15
< 75	5 < m < 10	15 < m < 30

Wilson (1980) suggested that to obtain the in-situ strength of rock mass, the unconfined laboratory strength C_o be divided by a factor f where

f = 1 for strong massive unjointed rock (including concrete),
= 2 for widely-spaced joints or bedding planes in strong rocks,
= 3 for more jointed, but still massive rocks,
= 4 for well-jointed and weaker rocks,
= 5 for unstable seat earth and closely-cleated rock such as coal, and
= 6 and 7 for weak rock in the neighbourhood of fault zone.

Hoek and Brown (1980) expressed the correlation between laboratory strength measured, σ_c, and in-situ strength in the form of empirical failure criterion which was presented in Eq. 3 earlier in this paper. The parameters m and s in Eq. 3 are constants which are depending on the properties of the rock and the extent to which the rock mass has been fractured. A range of values for m and s has been given for different rock types as well as for the insitu situations (Hoek and Brown, 1980). Priest and Brown (1983) proposed the following equations for estimation of insitu parameters m and s;

$$m = m_i . \exp\left(\frac{RMR - 95}{13.4}\right) \qquad (10)$$

$$s = \exp\left(\frac{RMR - 100}{6.3}\right) \qquad (11)$$

For the intact rock m = m_i is determined from a fit of Eq. 3 to triaxial test data from laboratory specimens, taking s =1 for rock material.

Weakening Coefficient is a classification system for coal measure formations proposed by Singh (1986). This system was developed to define a reduction factor, WC (weakening coefficient), to be applied to the intact sample values for acquisition of the rock mass properties. This system includes following parameters;

- Rock quality designation, RQD,
- Joint spacing index, K_1,
- Joint surface index, K_2,
- Joint filling index, K_3, and
- Joint aperture index, K_4.

The overall joint coefficient K and weakening coefficient WC are calculated by Eqs. 12 and 13.

$$K = K_1 \times K_2 \times K_3 \times K_4 \quad (12)$$

$$WC = K \times RQD \quad (13)$$

Where WC is weakening coefficient of rock mass and RQD (%) is rock quality designation. A correlation was made between RMR rating values and corresponding weakening coefficient by Gahrooee (1989) as expressed in Eq. 14.

$$WC = 0.018 \times e^{(0.039\ RMR)} \quad (14)$$

The weakening coefficient is suggested to be used to determine the in-situ constants m and s in the Hoek and Brown criterion by the following empirical equations:

$$\log(K1) = 0.118 + 1.827 \times \log(WC) \quad (15)$$

$$\log(K2) = 0.047 + 4.052 \times \log(WC) \quad (16)$$

$$m = m_i \times K1 \quad (17)$$

$$\sigma = \sigma_i \times K2 \quad (18)$$

where,

WC = weakening coefficient of the rock mass,

m & s = constant parameters for rock mass in the Hoek and Brown criterion, and

m_i & s_i = constant parameters for intact rock in the Hoek and Brown criterion.

5.0 APPLICATION OF ROCK MASS CLASSIFICATION TO DETERMINE THE IN-SITU STRENGTH OF ROCKS

For the purpose of this study, the South African Council for Scientific and Industrial Research (CSIR) classification system, RMR rating system, was used to evaluate the strata units behaviour. Assessment of the geotechnical parameters for different strata units including: uniaxial strength of intact samples (UCS), rock quality designation (RQD), joint spacing, joint condition, ground water condition and the effect of joint strike and dip orientations in the roadways were carried out according to the following descriptions and results are summarised in Table 7.

Table 7 - Geotechnical parameters used to determine RMR rating of the strata units

Strata Unit	UCS (MPa)	RQD (%)	spacing of joints (mm)	condition of joints	water condition	RMR	RMR (adjusted)
sandstone (m.g)	65 (7)	55.3 (13)	103 (10)	class II (20)	class I (10)	60	50
sand. f.g. + mudstone	40 (7)	78.5 (17)	165 (10)	class II (20)	class II (7)	61	51
sand. c.g. + shale	31 (4)	44.1 (8)	107 (10)	class III (12)	class II (7)	41	31
coal	23 (2)	25-50 (8)	50-120 (10)	class III (12)	class I (10)	42	32
mudstone	30 (4)	< 25 (3)	60-100 (10)	class II (20)	class I (10)	47	37
sandstone (c.g)	75 (7)	78 (17)	175 (10)	class II (20)	class I (10)	64	54
sandstone (m.g)	65 (7)	55 (13)	120 (10)	class II (20)	class I (10)	60	50

The uniaxial compressive strength of intact rocks were taken from Table 1. The RQD and joint spacing were calculated from underground borehole drillings at the site of investigation. The ground water condition was surveyed in the gate entries. The general water condition was dry to wet, and it was not a serious problem in the stability of structures; therefore, class I and class II were taken into account for different strata units in this regard. Since all roadways were driven within the coal seam (parallel to the bedding planes), the effect of joint strike and dip orientation in roadways fell in the third category: "Dip 0^{o} - 20^{o} irrespective of strike" which is "unfavourable" condition. As a result, an adjustment factor of -10 was considered on the overall RMR rating.

In addition to CSIR classification system (RMR index), the Norwegian Geotechnical Institute (NGI) system (Q index) and the Central Mining Research Station (CMRS) system (R index) were also examined. The NGI tunnelling quality index (Q) developed by Barton et al. (1974) is based on the evaluation of a large number of case histories of underground excavation stability, particularly civil engineering cases. There are three suggestions to correlate RMR and Q indexes denoted in Equations 19 and 21.

$$RMR = 19 \ Ln\ (Q) + 26 \qquad \text{(Singh, 1986)} \qquad (19)$$
$$RMR = 9 \ Ln\ (Q) + 44 \qquad \text{(Bieniawski, 1989)} \qquad (20)$$
$$RMR = 18.79 \ Ln\ (Q) + 13.48 \qquad \text{(Sunu, 1988)} \qquad (21)$$

The CMRI classification is a new system developed by Venkateshwarlu and Raju (1987). This system is proposed for coal measure rocks and is based on the evaluation of 52 collieries in India, some of those included more than one case study. The method has five basic parameters as follows:

- Layer thickness (approximately equivalent to RQD),
- Structural features (faults, slips, slickensides, joint sets, structural irregularities),
- Slake durability,
- Intact rock strength, and
- Ground water.

Besides the above factors, the stress state (including depth of cover and horizontal stresses) and proximity of other excavations are taken into account in the CMRS for calculation of the R index. Based on 44 case studies by Sheorey (1991), the following correlation has been made between R (CMRI system) and Q (NGI system);

$$R = 46 + 12 \times \log (Q) \qquad (22)$$

In the present research, the Q and R indices were also determined by using RMR values and Eqs. 20 and 22. The Q and R values determined for various strata units encountered in this investigation are in the range of those given by Sheorey (1991). The results for the strata units are tabulated in Table 8. These values can be used to estimate rock load or mean support load density (MLD) in roadways and intersections.

Table 8 - Rock mass classification index values for strata units, RMR, Q and R

Strata units	RMR (CSIR)	Q (NGI)	R (CMRI)
Sandstone (m.g)	50	1.948	49.47
sand. f.g. + mudstone	51	2.177	50.05
sand. C.g. + shale	31	0.236	38.47
coal	32	0.264	39.05
mudstone	37	0.459	41.95
sandstone (c.g)	54	3.038	51.79
sandstone (m.g)	50	1.948	49.47

The weakening coefficient, WC, and constant parameters of strata units, m and s, in the Hoek and Brown criterion are determined by applying the estimated RMR values into Eqs.14 to 18. These values were compared with those calculated by using Eqs.10 and 11 proposed by Priest and Brown (1983). Although both methods gave very low values for m and s, the weakening coefficient method seemed to be more conservative. Results of the study are tabulated in Table 9.

Table 9 - Constants m and s calculated based on the weakening coefficient and the Priest and Brown suggested equations (UCS = [(s) ^ 0.5] x UCSi)

Strata	Index		WC		Priest and Brown		
	RMR	WC	m	s	m	s	UCS (MPa)
sandstone (m.g)	50	0.127	0.158	0.000256	0.183	0.000357	1.24
sand. f.g. + mudstone	51	0.132	0.381	0.000300	0.449	0.000419	0.91
sand. c.g. + shale	31	0.060	0.082	0.000013	0.089	0.000018	0.22
coal	32	0.063	0.147	0.000015	0.160	0.000021	0.10
mudstone	37	0.076	0.228	0.000033	0.253	0.000045	0.20
sandstone (c.g)	54	0.148	0.333	0.000482	0.391	0.000674	1.95

The in-situ values for m and s in Hoek and Brown failure criteria given in Table 9, were used to calculate the in-situ safety factor around the roadway. Figure 7 shows the safety factor contour lines around the roadway based on the in-situ values of m and s. It can be seen that the results are extremely far from reality. The author believes that the purposed approaches for estimation of in-situ values of m and s by Priest and Brown and by weakening coefficient method are an appreciable underestimate of in-situ strength of rock mass. These values should not be used for stability analysis of underground structures in the FE method.

An attempt was made to solve the foregoing problem of finding the insitu values of rock mass strength. The reduction coefficients for m and s in Equations 17 and 18, as $r_m = \exp\left(\frac{RMR - 95}{13.4}\right)$ and $r_s = \exp\left(\frac{RMR - 100}{6.3}\right)$, were plotted against the RMR (Fig. 7). It can be seen that the reduction factor is significant when RMR > 50. Many different equations were tested but when the results were utilised to calculate the in-situ safety factor of the elements around the roadway, unrealistic results were obtained. It was then decided to develop a new method in which the reduction factor is applied to the safety factor rather than to the individual constants m and s. The following is a new approach for estimation of the in-situ strength of strata units based on the RMR index. In the Experimental failure criterion, the safety factor is calculated using parameters obtained from laboratory tests. The overall reduction coefficient of the safety factor, r_{sf}, is then calculated based on the RMR values for various rock types as shown in Fig. 7.

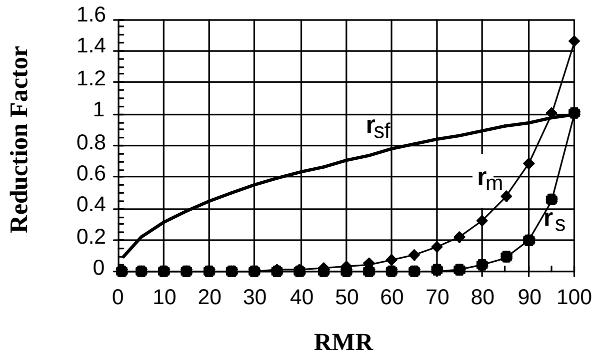


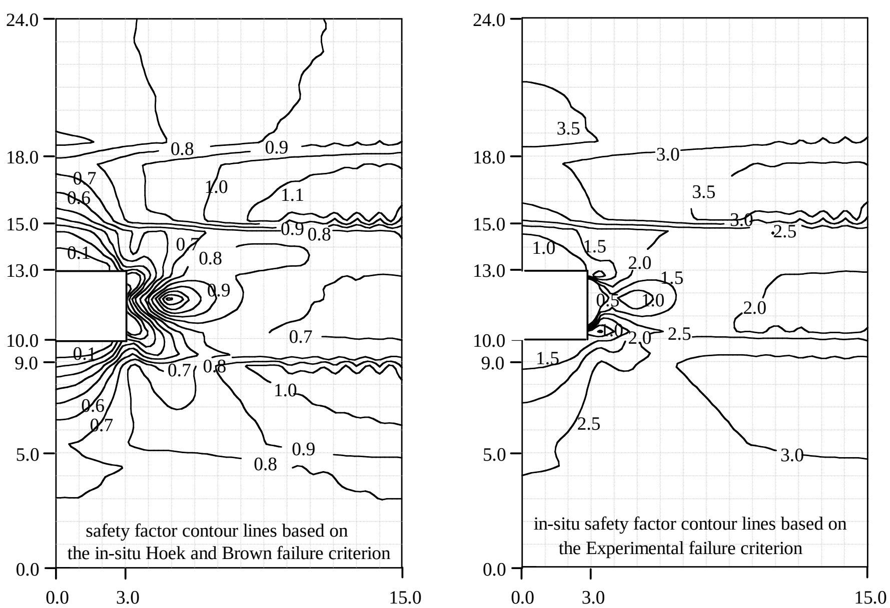
Fig. 7 - Reduction factor of constants m and s, r_m and r_s, based on the Priest and Brown equations, and the safety factor reduction coefficient, r_{sf}, in the Experimental criterion.

$$r_{sf} = 0.1\sqrt{RMR} \tag{23}$$

$$SF_{insitu} = r_{sf} \times SF_{intact} \tag{24}$$

where,

r_{sf} = reduction coefficient to safety factor,
$SF_{in\text{-}situ}$ = safety factor calculated for the in-situ condition, and
SF_{intact} = safety factor calculated based of the laboratory testing results.



Figs. 8 and 9 - Safety factor contour lines around the roadway based on: the in-situ values of m and s after the Hoek and Brown criterion, the in-situ values of safety factor after the Experimental criterion

The result of this approach is presented in Fig. 8. In Fig. 8, the in-situ safety factor contour lines around the roadway are calculated based on the new approach expressed in Eqs. 23 and 24.

It can be seen in Fig. 8 that the Experimental criterion gives more realistic prediction of failure zone around the roadway while some modification to the reduction coefficient on the safety factor, presented in Eq. 24, may make this approach more realistic.

6.0 CONCLUSION

An extensive laboratory testing was carried out on six rocks type. Mechanical properties of these rocks including complete stress-strain curve, tangent Young' Modulus, Poisson's ratio, compressive and tensile strength were obtained. In addition, the theoretical and empirical failure criteria were examined against the triaxial compressive test results. Some problems encountering with the conventional criteria were discussed and as a result a new experimental criterion was established. Fitting different failure criteria equations to the measured data, it was shown that the Bieniawski and the Experimental criteria were more closely matched to the data.

Attention was also drawn on the scale factor (reduction factor) for mechanical properties of rock masses (strata units). An attempt has been made to use available techniques and methods to predict the in-situ properties of strata units from laboratory results. It was shown that previous methods for determining the in-situ parameters of the Hoek and Brown criterion were too conservative to be applied to stability analysis of underground structures. Therefore, a new approach was proposed to estimate the in-situ safety factor of rocks around an excavation based on the RMR index. The results indicated that the new approach was more realistic.

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