Case Studies in the Assessment of Rock Mass Criteria

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Summary Various rock mass failure criteria are being validated against field performance as part of a large research program into the geotechnical risk of concrete gravity dams. The project has sought rock structures which may contribute to providing either upper or lower bounds to various failure criteria. The emphasis therefore has been on finding sites with high quality field and laboratory data. This paper summarises the analysis and results of a large pit slope at Kidston Gold Mine, together with a summary of other previously reported cases. The results from the studies allow the field stresses and strengths to be compared with the commonly used Hoek-Brown empirical rock mass failure criterion. As expected, the bounds on the various failure criteria are very broad and care must be taken when placing any reliance on any particular criterion.

1 INTRODUCTION

The author is currently involved with a research project into the risk assessment of concrete dams. A major component of this project is to attempt to validate various rock mass failure criteria against field performance. An extensive literature review has shown that almost all the data supporting the published criteria are laboratory test results from intact specimens, with, in general, very little and often no field validation. Rock mass strength is clearly scale dependent and thus laboratory testing alone cannot be expected to have captured important field characteristics.

The most commonly used strength criterion is the Hoek-Brown empirical rock mass failure criterion. The most general form of which is given in Equation (1). This criterion was developed by Hoek and Brown (1) due to the lack of any available empirical strength criterion. The equation which has subsequently been updated by Hoek and Brown (2) and Hoek et. al. (3) was based on correlations between brittle fracture of intact rock cores and the original Griffith theory (4). The only 'rock mass' tested and used in the development of the Hoek-Brown criterion was 152mm core samples of Panguna Andesite from Bougainville in Papua New Guinea (1). Brown and Hoek (5) later noted that it was likely this material was in fact 'disturbed'. The validation of the updates of the Hoek-Brown criterion have been based on experience gained whilst using this criterion. It is unfortunate however, that to the authors knowledge the data supporting this experience has not been published.

$$\sigma_1' = \sigma_3' + \sigma_c\left(m_b\frac{\sigma_3'}{\sigma_c} + s\right)^a \qquad (1)$$

Due to the difficulty with assessing the parameters in the Hoek-Brown criterion correlations with rock mass rating parameters were developed. The most current of these is the Geological Strength Index (GSI) (6) which is a variation on the rock mass rating (RMR) of Bieniawski (7 and 8) and the tunneling quality index (Q) of Barton et al. (9). These are shown in Table 1. The parameters m_i and m_b are intact and mass material constants; a and s are constants that depend on the rock mass characteristics; σ_c is the uniaxial compressive strength of the intact rock; and σ_1' and σ_3' are the major and minor principal stresses respectively.

	GSI<25	GSI>25
$\frac{m_b}{m_i}$	$\exp\left(\frac{GSI-100}{28}\right)$	$\exp\left(\frac{GSI-100}{28}\right)$
s	0	$\exp\left(\frac{GSI-100}{9}\right)$
a	$a = 0.65 - \frac{GSI}{200}$	0.5

Table 1: Hoek-Brown GSI Strength Parameters

This paper describes some of the authors work in attempting to correlate field data to the Hoek-Brown criterion. Results from several case studies are shown.

2 KIDSTON GOLD MINE

The footwall of the Wises Hill pit, located at Kidston Gold Mine, Queensland, is approximately 240m high. The average slope angle is approximately 48° with a maximum bench height and slope of approximately 36m and 73° respectively. The pit has been successfully mined with no overall stability problems. The pit was chosen for analysis due to its highly stressed nature. An analysis using the stability program *Slope/W* has been performed and the stresses obtained compared with those output from the Hoek-Brown criterion.

2.1 Rock Parameters

The major rock type in the slope is a very high strength granodiorite breccia. Rock mass parameters were obtained from Coffey Partners International Pty Ltd (10) and line mapping histograms created by Pells Sullivan Meynink Pty Ltd (PSM). Estimates of GSI were derived using RMR_{76}, Q, Table 8.4 in Hoek et. al. (11) and Figure 1 in Hoek (12). The value of m_i for granodiorite was taken from Hoek et. al. (11). The Hoek-Brown parameters are shown in Table 2. Three estimates were made: LB being the lower bound; BE being the best estimate; and UB being the upper bound.

Parameter	LB	BE	UB
GSI (RMR_{76})	77	82	92
GSI (Q)	72	72	73
GSI (Table 8.4)	70	70	85
GSI (Figure 1)	60	65	75
UCS	100	100	120
m_i	30	30	30
a	0.5	0.5	0.5

Table 2: Hoek Parameters for Kidston

As can be seen in Table 2 the value of GSI varies considerably depending on the method used to derive it. The latest approach by Hoek (12) gives the lowest estimate. Indeed, an inspection of this method shows that the highest possible GSI is 80 for a 'blocky structure' with 'very good surface conditions'. Due to this variation it is suggested that those using the criterion make sure they are using the latest update for the estimation of GSI and the Hoek-Brown criterion.

2.3 Slope Stability Analysis

The stability program *Slope/W* was used for the analysis with both Bishop and Morgenstern & Price analyses. The parameters used are shown in Table 3. They were obtained using the Hoek-Brown strength criterion and the solutions derived by Balmer (13) with a linear interpolation of the points in the stress region of interest.

Parameter	GSI=60	GSI=70	GSI=92
m_b	7.2	10.3	22.5
ϕ (°)	67	68	68
c (MPa)	1.2	1.9	6.7

Table 3: Results of Hoek-Brown Analysis (σ_n= 0.8MPa)

It should be noted that the suggested values of minimum principle stress, σ_3, given by Hoek et. al. (11) for use in their spreadsheet are applicable more to highly stressed underground workings than slopes. The high cohesions of rock masses result in negative σ_3 being required to obtain low σ_n on potential failure planes. Users should use stress which yield normal stresses, σ_n, applicable to their problem The values for the cohesion, c, and angle of friction, ϕ, i Table 3 assume a normal stress of 0.8MPa. This norm stress was determined iteratively using the slice base norm forces output from *Slope/W* and adjusting the paramete accordingly.

Both a deep failure surface and a shallower failure surfa incorporating only the steeper section of the pit we analysed. For the deep failure surface the slope was modelle as three material types with the least stressed upper an lower materials given a slightly lower strength (σ_n= 0.8MP used) compared to that of the middle layer (σ_n=1.2MPa).

A further analysis was carried out to assess the effect a joi would have if it occurred at the toe of the slope. This wa justified from the line mapping results by PSM whic showed the presence of long joints (5-10% greater than 50m in the region of the toe. To carry out the analysis a zone material with joint strength properties was added in th location of the toe of the shallow failure slip surface. Th joint strength properties, of zero cohesion and an angle friction, ϕ, of 40°, were taken from Coffey Partne International (10). The weak 'joint' material was stoppe short of the face due to the presence of rock bolting in th face at Kidston.

Tension cracks were added for all analyses at the top of th failure surfaces to limit tension.

2.4 Results

Figures 1, 2 and 3 show the failure surfaces for the dee shallow and joint material analyses respectively. Th analyses through the rock masses gave a FOS approximately 6.5. The analysis where the joint region wa included yielded a FOS of 4.9. Lower factors of safety we obtained for other surfaces but were rejected due to the shallow nature or excessive tensile stresses between slices.

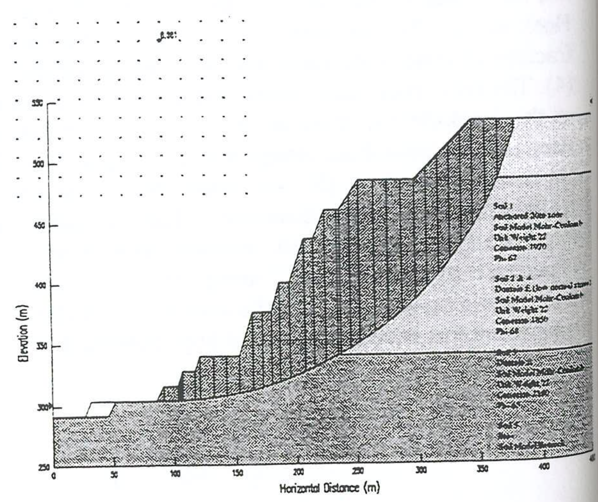

Figure 1: Deep Seated Failure - Kidston Mine

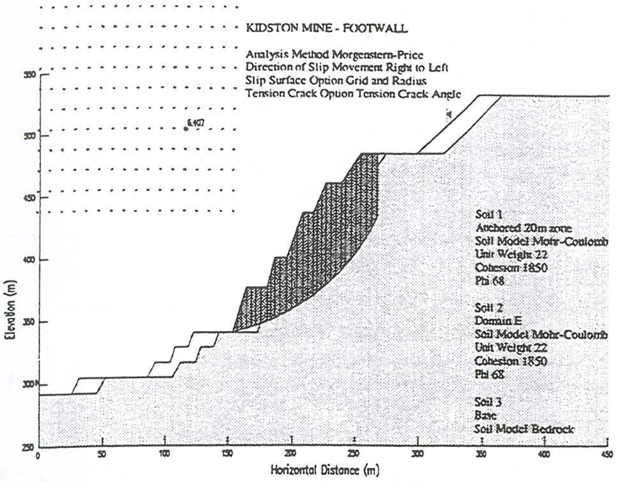

Figure 2: Shallow Failure Surface - Kidston Mine

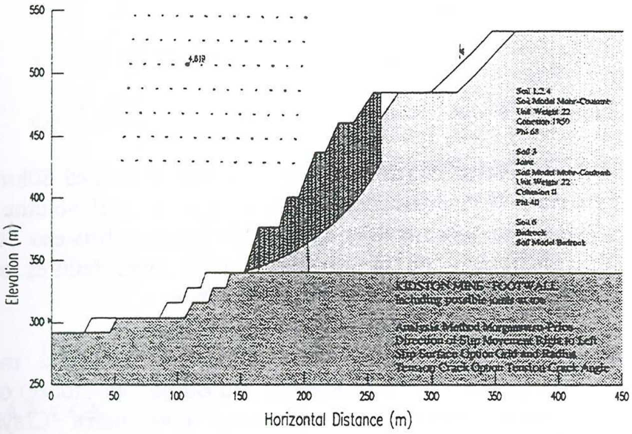

Figure 3: Failure Through Joint Region - Kidston Mine

The slice base normal and shear stresses are plotted on Figure 4 together with the Hoek-Brown criterion curves for the lower bound, best estimate and upper bound GSI values (60, 70 and 92 respectively). Figure 4 shows that although the slope appears highly stressed it does not appear to be a very good bound for rock mass strength.

It is difficult to imagine rock mass behaviour leading to failure for slopes with a high GSI rock mass. Using the best estimate rock mass strengths output by the Hoek-Brown criterion for Kidston a quick analysis with *Slope/W* showed a 70° slope could be over 5000m high without inducing failure.

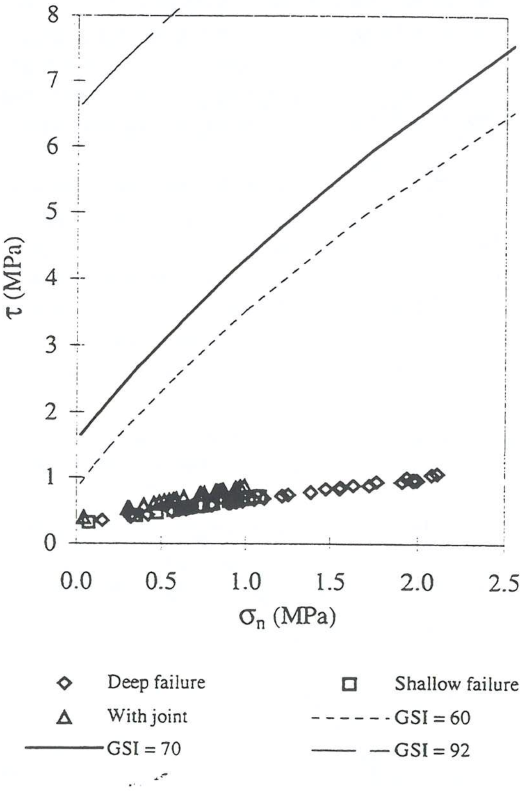

Figure 4: Hoek-Brown vs Stresses for Kidston

3.0 OTHER CASE STUDIES

3.1 Aviemore Dam

Aviemore Dam, a 57m high concrete dam constructed between 1963 and 1968, is located on the Waitaki River, 185km south west of Christchurch, New Zealand. During construction eight large scale in-situ shear tests were carried out on the foundation in order to determine the strength of the concrete/rock interface for stresses relevant to those induced by the proposed dam.

The in-situ tests were carried out on a greywacke rock mass which was closely jointed, veined, and often crushed and sheared. All tests failed through the rock mass, approximately 50-100mm below the rock-concrete contact, involving failure of several defects.

During the tests both the test and reaction blocks failed. Foster and Fairless (14) re-analysed the test results and adjusted the vertical stresses on the test and reaction blocks assuming that vertical forces were transferred from the reaction block to the test block due to differential displacement. The values were adjusted to yield similar τ/σ_n ratios for the reaction and the test blocks. Their results have been adopted as a best estimate of the true failure stresses. By using *UDEC* Helgstedt et. al. (15) found it was possible to validate the assumptions made by Foster and Fairless (14) and thus provide a greater confidence in their results.

The test results are presented in Figure 5 together with the estimated Hoek-Brown envelopes. As can be seen the best estimate Hoek-Brown criterion gave a higher estimate of the rock mass strength compared with the adjusted test values. The failure stresses appear to correspond reasonably to the lower bound estimated Hoek-Brown envelope.

The Hoek-Brown curves derived using the chart in Hoek (6) give a lower estimate of the rock mass strength. The Hoek-Brown curves derived using Barton's Q system give higher estimates than those from Bieniawski's RMR$_{89}$ and the Hoek-Brown chart in Hoek et. al. (11). It appears from the results that the curves derived from the Hoek-Brown chart give the best estimate of the rock mass strength.

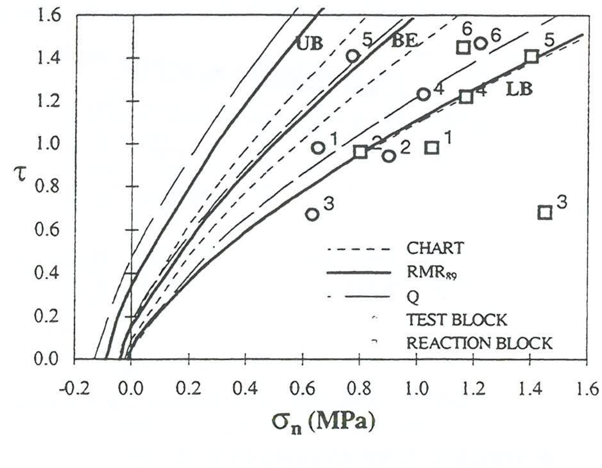

Figure 5: Aviemore In-Situ Shear Test Results

3.2 Chichester Dam

Chichester Dam is a 41m high concrete gravity dam located on the Chichester River, 80km north of Newcastle, NSW. The dam has been heavily investigated with approximately 150 boreholes drilled between 1950 and 1980. As a result of these investigations, the dam was post tensioned to provide additional security against extreme hydrological events.

The parameters used in the analysis are described in detail in Mostyn et. al. (16). The dam foundation was modelled using UDEC. The results from the analysis are shown in Figure 6.

The results from the analysis indicated that a planar wedge failure was the likely failure mode and hence the Hoek-Brown criterion was not applicable. The analysis also showed that the low stresses beneath a dam of this size are not conducive to placing tight bounds on rock mass strength criteria.

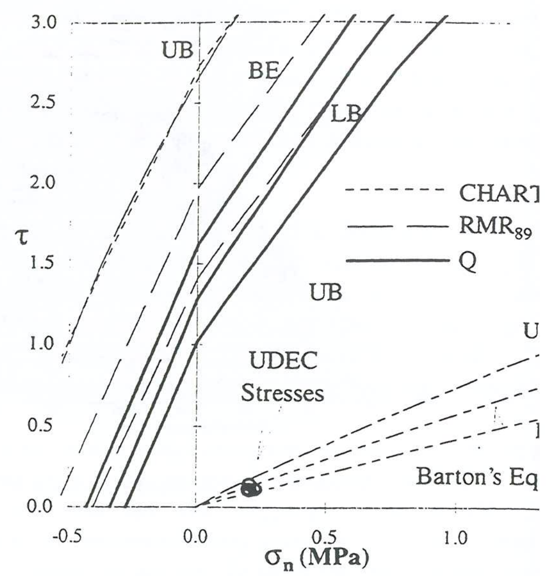

Figure 6: Chichester Dam Results

3.3 Nattai North

The Nattai North escarpment failure is located 80k west of Sydney. The failure, with a total volum million cubic meters and height ranging between 300m, is one of the largest rock mass failures occurred in Australia in modern times.

The conditions at Nattai North are such that competent rock (Scarborough and Bulgo Sandstones) weaker, more fractured strata (Wombarra Cla Together with this, the bedding planes tend to dip slope, resulting in large pillars that are prevente sliding or toppling. These pillars are then supported highly stressed weaker Wombarra Claystone. conditions are believed to be sufficient to cause s through the claystone and failure of the escarpme believed that the failure was induced by mining of coa beneath the escarpment.

The modelling of the failure was performed using Two series of runs were made, with and without mi described in Mostyn et. al. (16). The GSI was es using the two modified Hoek-Brown criterion classi systems, RMR$_{89}$, Q and the Hoek-Brown chart given i (6).

The results of the analysis using *UDEC* correlated w the assumed failure mode. In Figure 7 the stresses p at failure by *UDEC* have been compared to those fr Hoek-Brown criteria. It is seen that the Hoek-Brown c gives a higher estimate of the rock mass strength. The stresses appear to correspond reasonably to the lowe estimated Hoek-Brown envelope.

The shear plane for the mining case showed failure both through the area of fictitious joints in the toe of the slope and through the coal seam. As the coal seam would have a different GSI to that estimated the stresses along both zones have been plotted separately. A Hoek-Brown envelope corresponding to a GSI of 25 for a crushed rock mass with fair to poor joint conditions has been plotted (the lowest curve on Figure 7). This envelope appears to correspond reasonably to the failure stresses obtained along the coal seam.

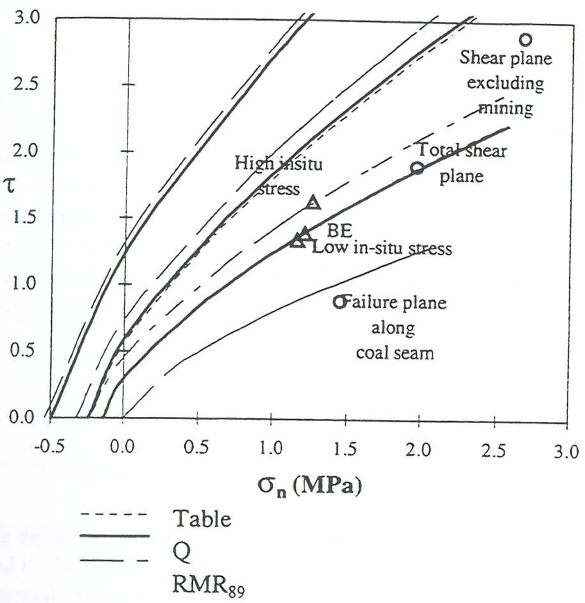

Figure 7: Hoek-Brown vs Estimated Stresses for Nattai

4.0 CONCLUSION

The case studies presented within this paper provide a first step towards a better understanding of the Hoek-Brown failure criterion. Further case studies are currently being analysed and results and conclusions will be presented as they become available. The authors are also involved in investigating the large scale in-situ shear strength of discontinuities. Due to the difficulty of obtaining good case studies on large scale discontinuity failures, only one defect case study has been attempted.

As has been shown in the case of Kidston Mine it is difficult to locate slopes that have failed through rock mass. This is particularly true for rock masses with a reasonably high GSI and/or intact rock strength.

The author would be grateful for any qualitative performance and rock mass characterisation data on rock masses that have failed or are near failure (both mass or defect controlled). If any reader is aware of such information do not hesitate to contact the author.

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3.6 References

1. Hoek, E. and Brown, E.T. "Empirical strength criterion for rock masses", J. of the Geotech. Engng Div., ASCE,, Vol 106, No. GT9, 1980, 1013-1035.
2. Hoek, E. and Brown, E.T. "The Hoek-Brown failure criterion - a 1988 update", Proc. Of the 15th Canadian Rock Mech. Symp., Toronto, 1988, 31-38.
3. Hoek, E., Wood, D. and Shah, S. "A modified Hoek-Brown failure criterion for jointed rock masses", Eurock '92, 1992, 209-214.
4. Griffith, A.A. "Theory of rupture", Proc. 1st Congr. Applied Mechanics, Delft, 1924, 55-63.
5. Brown, E.T. and Hoek, E. "Discussion on paper no. 20431 by R. Ucar, entitled: Determination of shear failure envelope in rock masses", J. of the Geotech. Engng Div., ASCE,, Vol 114 No. 3, 1988, 371-373.
6. Hoek, E. "Strength of rock and rock masses", Int. Soc. for Rock Mech. News Journal, Vol 2 No. 2, 1994, 4-16.
7. Bieniawski, Z.T. "The geomechanics classification in rock engineering applications" Proc. 4th Congr. Int. Soc. Rock Mech., Montreux, Vol 2, 1979, 41-48.
8. Bieniawski, Z.T. Engineering Rock Mass Classifications, New York, Wiley, 1989.
9. Barton, N.R., Lien, R. and Lunde, J. "Engineering classification of rock masses for the design of tunnel support", Rock Mech. Vol 6, No. 4, 189-236.
10. Coffey Partners International Pty Ltd, Wises Hill Pit Geotechnical and Hydrogeological Study - Phase II, Report No. Z105/2-AF, 23 April, 1991.
11. Hoek, E., Kaiser, P.K. and Bawden, W.F. Support of Underground Excavations in Hard Rock, A.A.Balkema, 1995, 84-98.
12. Hoek, E. Technical Note: Reliability of Hoek-Brown estimates of rock mass properties and their impact on

design, submitted to the Int. J. Rock Mech. & Min. Sci, 1997.

13. Balmer, G. A general analytical solution for Mohr's envelope, Am. Soc. Test. Mat., Vol 52, 1952, 1260-1271.

14. Foster, P.F. and Fairless, G.J. Waitaki Dam - Review of Aviemore rock strength. (unpublished) ID: 9W551.BO, 1994, Power Engineering Office Works Consultancy Service Ltd (Wellington, N.Z.).

15. Helgstedt, M.D., Douglas, K.J. and Mostyn, G. "
evaluation of in-situ direct shear tests Aviemore
New Zealand", Australian Geomechanics, No. 37,
1997, 56-65.

16. Mostyn, G., Helgstedt, M.D. and Douglas, K.J. "To
field bounds on rock mass failure criteria", Int. J.
Mech. & Min. Sci., Vol 34, No.3-4, Paper No. 208, 1