Q-SLOPE: AN EMPIRICAL ROCK SLOPE ENGINEERING APPROACH IN AUSTRALIA

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ABSTRACT

The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of excavated rock slopes in the field. Q-slope allows geotechnical engineers and engineering geologists to make potential adjustments to slope angles as rock mass conditions become apparent during the construction of reinforcement-free road or railway cuttings and in open pit mines. Through case studies across Australia, the Americas, Asia and Europe, a simple correlation between Q-slope and long-term stable slopes was established. The Q-slope method is designed such that it suggests stable, maintenance-free, bench face slope angles of, for instance, 40-45°, 60-65° and 80-85° with respective Q-slope values of approximately 0.1, 1.0 and 10.

Q-slope was developed by supplementing the Q-system which has been extensively used for characterizing rock exposures, drill core and underground mines and tunnels under construction for the last 40 years. The Q' parameters (RQD, J_n, J_r and J_a) have remained unchanged in Q-slope, although a new method for applying J_r/J_a ratios to both sides of a potential wedge is used, with relative orientation weightings for each side. The term J_w has been replaced with the more comprehensive term J_{wice}, which takes into account long-term exposures to various climates and environments. SRF categories have been developed for slope surface conditions, stress-strength ratios and major discontinuities such as faults, weakness zones or joint swarms.

This paper discusses civil and mining engineering applications of the Q-slope method in Australia for a variety of ground conditions from very weak to strong rocks, blocky to massive, isotropic rock masses to laminated, heterogeneous, highly anisotropic rock masses. A case study is also presented to illustrate the compatibility of Q-slope with P-wave velocity and acoustic and optical televiewer data obtained from borehole geophysical surveys to determine appropriate rock slope angles.

NOMENCLATURE

RQD – rock quality designation

J_n – joint sets number

J_r – joint roughness number

J_a – joint alteration number

J_{wice} – environmental & geological condition number

SRF_{slope} – largest of three strength reduction factors: a, b and c

SRF_a – physical condition number

SRF_b – stress and strength number

SRF_c – major discontinuity number

O-factor – orientation factor for the ratio J_r/J_a

1 INTRODUCTION

In both civil and mining engineering projects, it is practically impossible to assess the stability of rock slope cuttings and benches in real-time using analytical approaches such as kinematics, limit equilibrium or finite and distinct element modelling. Excavation is usually too fast for this. Furthermore, in Australia, the cost of engineering services and labour are too high to facilitate such detailed slope design guidance and reconciliation during excavation. The same limitation usually applies to tunnelling, although caverns and large underground openings are sufficiently stationary for thorough and more necessary analysis, and the same applies to high rock slopes.

The purpose of Q-slope is to allow engineering geologists and geotechnical engineers to assess the stability of excavated rock slopes in the field, and make potential adjustments to slope angles as rock mass conditions become visible during construction (Barton & Bar, 2015). Key areas of Q-slope application are from the surface and downwards: bench face angle decisions in open pit mines, and for numerous slope cuttings to reach remote project sites in mountainous terrain through varying geological conditions. In many rock slope problems, the engineer needs to quickly decide whether the slope will be excavated at angles of 45 to 90° or shallower than 45°. The use of Q-slope during excavation can help to reduce maintenance and bench-width needs due to all potential failures. Such are frequently seen when initially 'constant' slope angles are excavated through different structural domains. A series of troublesome yet interesting local failures is usually the result. In many cases, these have been the result of adverse plane failures, wedge failures, or more rarely, local toppling.

A range of empirical methods for describing and characterizing rock masses have been developed in different countries over the last 50 years and are described by Duran & Douglas (2000).

In underground mining and tunnelling, empirical methods including the Q-system (Barton et al. 1974; Barton & Grimstad, 2014), rock mass rating (Bieniawski, 1976; Bieniawski, 1989) and mining rock mass rating (Laubscher, 1977; Laubscher & Jakubec, 2001) are commonly used to derive appropriate support and reinforcement for specific excavation spans. The Q-system is also an integral component of open stope design (Potvin et al. 1988; Mawdesley et al. 2003) and forms the basis of ground support designs for many if not most underground mines in Australia (Potvin & Hadjigeorgiou, 2015).

Empirical methods are less frequently used for assessing the stability of rock slopes in favour of kinematics, numerical modelling or "no modelling", where slope angles are decided by equipment operators rather than geotechnical engineers or engineering geologists. Slope mass rating (Romana, 1985; Romana, 1995) and global slope performance index (Sullivan, 2013) are examples of empirical methods for slopes; although neither these, nor any other methods known to the authors, were developed specifically to give guidance or advice in relation to appropriate, long-term stable slope angles in which reinforcement is purposely absent. Such slopes (reinforcement-free) are by far the most commonly excavated around the world, in both civil and mining engineering projects.

Q-slope utilizes the same six parameters as the Q-system: RQD, J_n, J_r, J_a, J_w and SRF (Barton & Bar, 2015). However, the frictional resistance pair J_r and J_a can apply, when needed, to the individual sides of potentially unstable wedges. Simply applied orientation factors, like $(Jr/Ja)_1$ x 0.7 for set J_1 and $(J_r/J_a)_2$ x 0.9 for set J_2, provide estimates of overall whole-wedge frictional resistance reduction, if appropriate. The term J_w, which is now termed J_{wice} (one of two symbol-modifications), takes into account an appropriately wider range of environmental conditions appropriate to rock slopes, which obviously stand in the open for a very long time. These conditions include the extremes of intense erosive rainfall and ice wedging, as may seasonally occur at opposite ends of the rock-type and regional spectrum. There are also slope-relevant SRF categories for slope surface conditions, stress-strength conditions and the presence of major discontinuities. The formula for estimating Q-slope is:

$$Q_{slope} = \frac{RQD}{J_n} x \left(\frac{J_r}{J_a} \right)_0 x \frac{J_{wice}}{SRF_{slope}} \quad (1)$$

Tables A1 to A7 and Figure A1 have been appended to help describe the parameters in Equation 1. Bar & Barton (2017) provide additional guidance and background. As with the Q-system, the rock mass quality in Q-slope can be considered a function of three parameters, which are crude measures of:

1. Block size: *(RQD/J_n)*.
2. Shear strength: least favourable *(J_r/J_a)* or average shear strength in the case of wedges *$(J_r/J_a)_1$ x $(J_r/J_a)_2$*.
3. External factors and stress: *(J_{wice}/SRF_{slope})*.

Shear resistance, τ, is approximated using:

$$\tau \approx \sigma_n \tan^{-1} \left(\frac{J_r}{J_a} \right) \quad (2)$$

Barton & Bar (2015) derived a simple relationship for the steepest slope angle (β) not requiring reinforcement or support for slope heights less than 30 metres. From the Q-slope data, the following correlations are simple and easy to remember:

- Q-slope = 10 - slope angle 85°.
- Q-slope = 1 - slope angle 65°.
- Q-slope = 0.1 - slope angle 45°.
- Q-slope = 0.01 - slope angle 25°.

This relationship was extended to all slope heights following the collation of supporting data from across Australia, Asia, the Americas and in Europe and can be described with the following formula (Bar & Barton, 2017):

$$\beta = 20 \log_{10} Q_{slope} + 65° \tag{3}$$

Equation 3 matches the central data for stable slope angles greater than 35° and less than 85° as shown in Figure 1 and has a probability of failure of 1%. Equation 3 does not represent a specific factor of safety as would be obtained by undertaking numerical analyses. Rather it represents the boundary of long-term stable slopes based on observed performance, normally between six months and over 50 years. Users may, if they wish, additionally apply a factor of safety on the steepest slope angle (β) not requiring reinforcement or support. In order to apply Q-slope to larger slope heights, one needs to adequately consider the uniformity of the lithological units and rock mass quality across the height of the slope. Q-slope may not be applicable if the slope is a combination of poor rock mass quality zones mixed with good quality zones. In these instances, and in general for slopes larger than say 50m in height (i.e. which require several stages of excavation), more rigorous analysis is both warranted and advised.

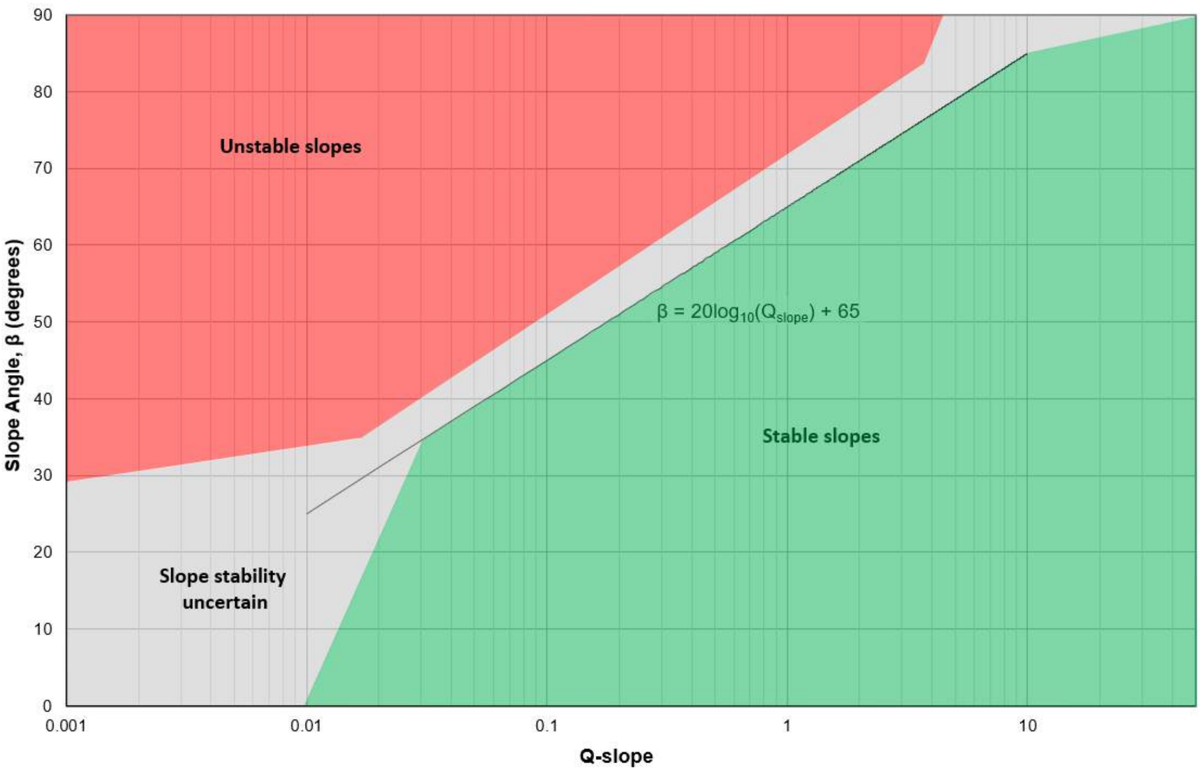

Figure 1: Q-slope stability chart (Bar & Barton, 2017).

A visual demonstration of the objective of Q-slope is shown in Figure 2 where improving rock mass quality as a result of reduced weathering grades and higher strength materials at depth allow for steeper bench face angles with depth. Barton & Bar (2015) and Bar & Barton (2016, 2017) provide several examples of individual case studies with detailed calculation steps for determining Q-slope.

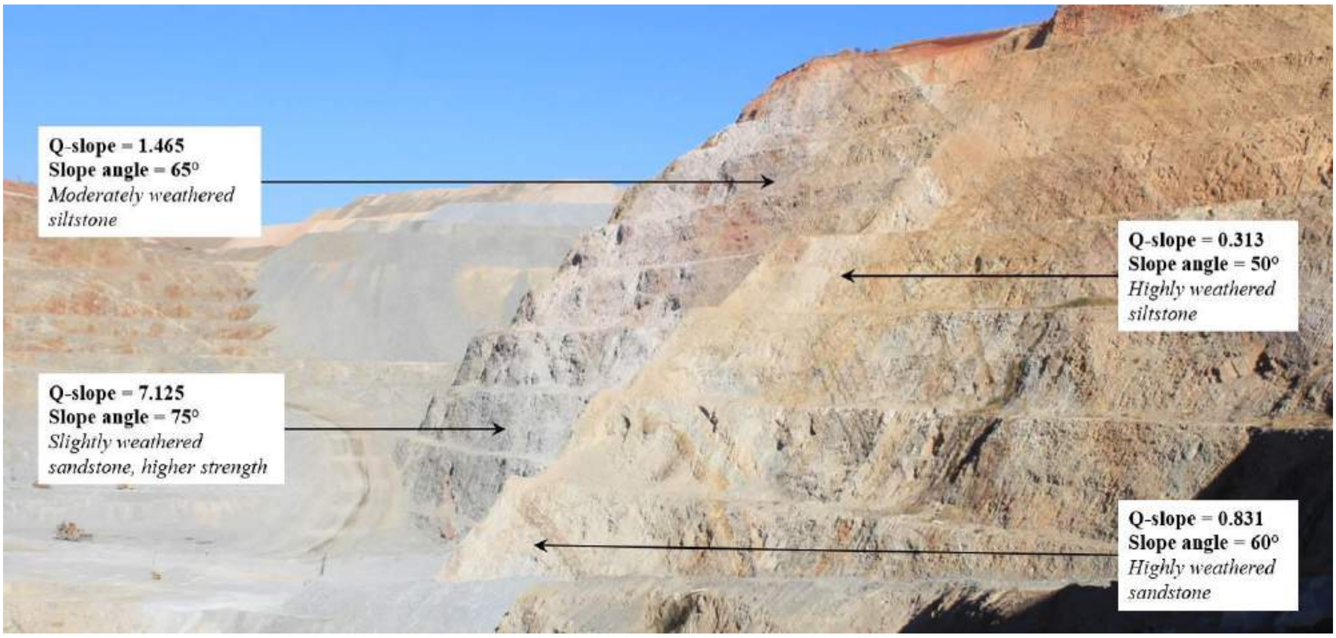

Figure 2: A convenient example of differently weathered rocks on two cutbacks (separated by a fault zone) and improving rock mass quality with depth, help explain the appropriately steepened, unreinforced bench slopes in an open cut mine in Western Australia.

Considering only the failed and quasi-stable slopes, both of which are undesirable or unwanted events, Bar & Barton (2017) estimated the probability of failure (PoF) using iso-potential lines. If certain degrees of failure were to be accepted, such as percentages of individual benches in open cut mines, then the following equations were also derived:

PoF=1%: $$\beta = 20\log_{10} Q_{slope} + 65° \quad (4)$$

PoF=15%: $$\beta = 20\log_{10} Q_{slope} + 67.5° \quad (5)$$

PoF=30%: $$\beta = 20\log_{10} Q_{slope} + 70.5° \quad (6)$$

PoF=50%: $$\beta = 20\log_{10} Q_{slope} + 73.5° \quad (7)$$

2 AUSTRALIAN Q-SLOPE CASE STUDIES

The Q-slope method has been applied to a variety of igneous, sedimentary and metamorphic rock masses at the locations across Australia shown in Figure 3. It has also been successfully applied to several highly weathered and saprolitic slopes in Far North Queensland (Bar et al. 2016) and the Goldfields Region between Perth and Kalgoorlie.

Figure 4 illustrates the Australian Q-slope dataset for different slope heights and slope angles. It is based on over 250 individual case studies from:

- New South Wales (Bathurst, Coffs Harbour, Kempsey, Orange, Sydney and Terrigal regions).
- Queensland (Airlie Beach, Atherton, Brisbane, Cairns, Cape Tribulation, Cloncurry, Cooktown, Gympie, Kuranda, Lakeland, Port Douglas, Malanda, Mareeba, Mount Isa, and Townsville regions).
- Western Australia (Great Sandy Desert, Kalgoorlie, Newman, Pannawonica, Perth and Tom Price regions).

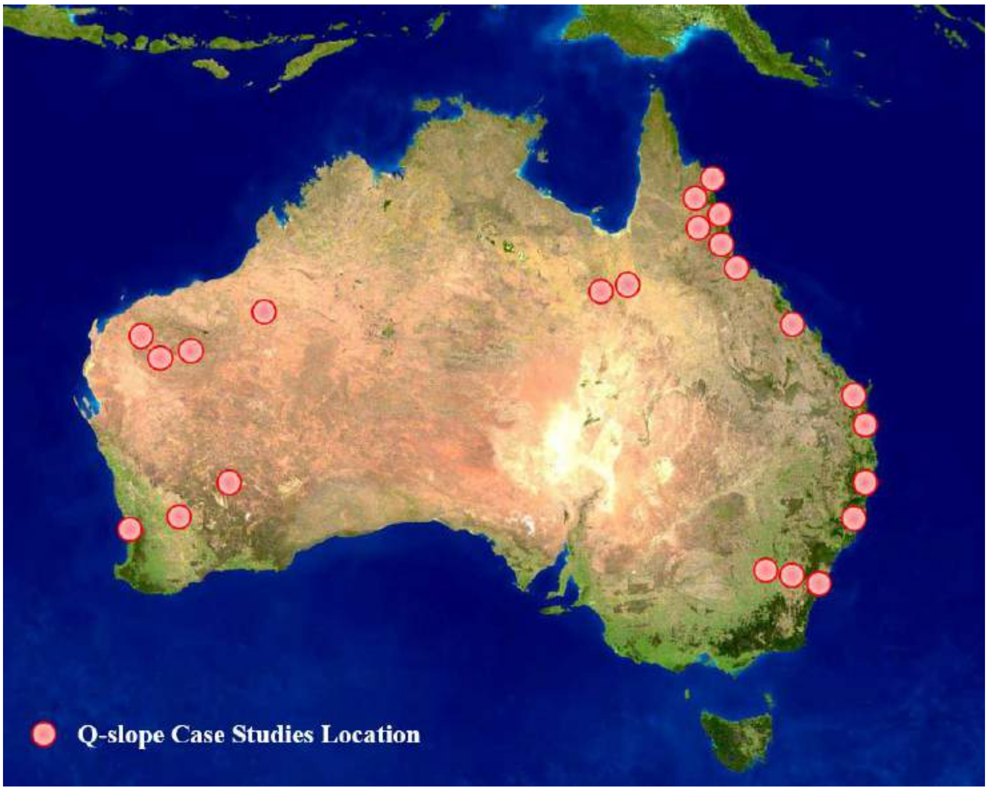

Figure 3: Location of Q-slope case studies in Australia

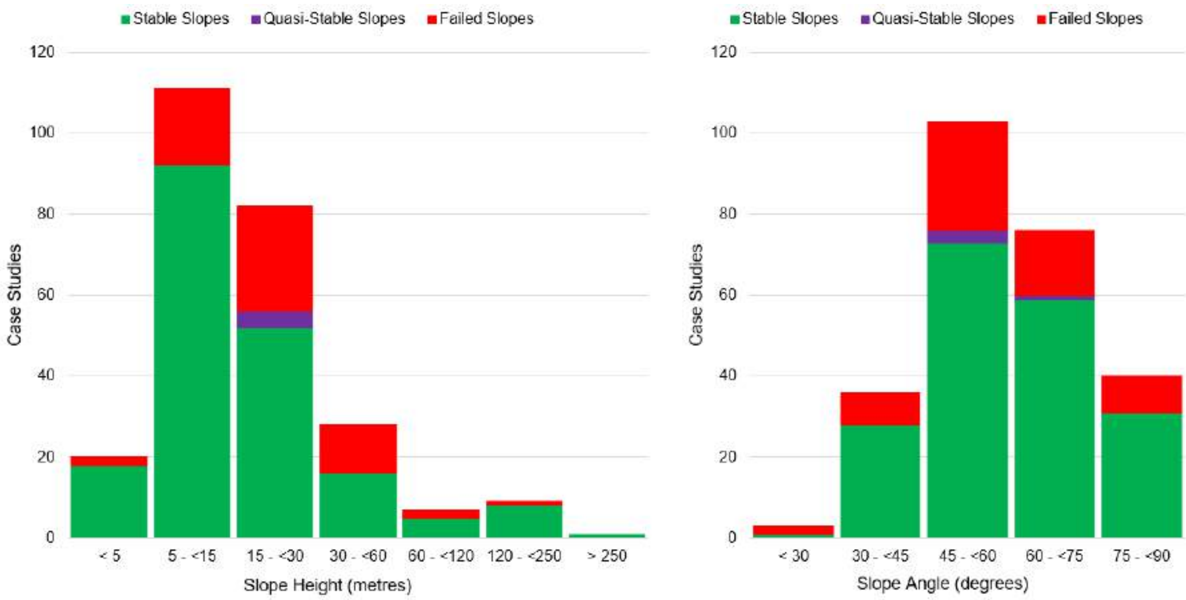

Figure 4: Australian Q-slope dataset (case studies) – slope heights (left); slope angles (right).

Figure 5 illustrates the available Q-slope data derived from the back-analysis of slopes cut along roads and highways, benches and inter-ramp slopes from open cut mines and natural slopes across Australia:

- Green triangles indicate stable slopes with no visual signs of instability observed for at least several weeks, months or years post-excavation.
- Purple squares indicate quasi-stable slopes (more than likely to collapse in the near future with rainfall or weathering effects). These have visible signs of slope instability such as tension cracks, dislocation or monitored deformation.
- Red crosses indicate failed or collapsed slopes that have been back-analysed with an understanding of pre-failure geometries and ground conditions.

Several case studies and rock slope field assessment examples have been presented in previous Q-slope publications (Barton & Bar 2015; Bar & Barton 2016; Bar et al. 2016; Bar & Barton 2017) and comprise stable slopes as well as a range of simple and complex failure mechanisms, including:

- Planar sliding on a single discontinuity.
- Planar sliding on a discontinuity with a second discontinuity acting as a release plane (e.g. a sub-vertical joint set).
- Wedge failures comprising two intersecting discontinuities.
- Complex wedge failures comprising two or more intersecting discontinuities (often with at least one acting as a release plane).
- Toppling failures (localized).
- Rotational failure as a result of shearing weak rock masses.
- Complex rotational failures including both sliding along discontinuities and shearing through intact rock bridges in strong rock masses.

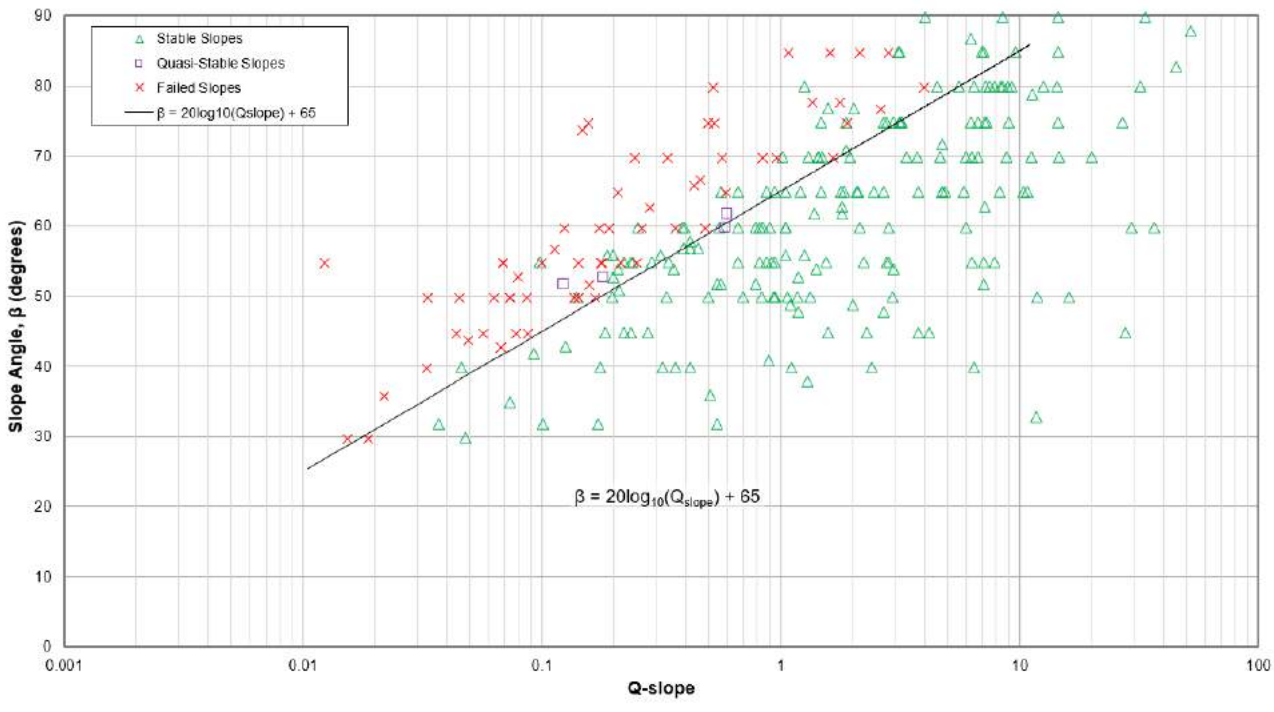

Figure 5: Australian Q-slope data – 258 case studies.

2.1 EXAMPLE FIELD ASSESSMENT OF ROCK SLOPE USING Q-SLOPE

A 25m high slope was excavated in the 1990's in a moderately weathered, closely bedded siltstone and remains stable at an angle of 65° (Figure 6). The unconfined compressive strength (σ_c) ranges from 25 to 50 MPa and bedding, although spectacularly folded, strikes very favourably into the slope. The syncline is also host to a sub-vertical fault striking favourably into the slope.

Field-assessed Q-slope parameters and Q-slope calculation steps are shown in Table 1 and Equation 8, respectively. Q-slope suggests slope angles up to 72° would be stable (Equation 9).

Table 1: Field assessed Q-slope parameters

RQD	J_n	Set	J_r	J_a	O-factor	J_{wice}	SRF_a	SRF_b	SRF_c
40	6	A	2	4	2	1	-	3	1
		B	-	-	-				

$$Q_{slope} = \frac{RQD}{J_n} x \left(\frac{J_r}{J_a}\right)_0 x \frac{J_{wice}}{SRF_{slope}} = \frac{40}{6} x \left(\frac{2}{4} x2\right) x \frac{1}{3} = 2.22 \tag{8}$$

$$\beta = 20 \log_{10} Q_{slope} + 65° = 20 \log_{10} (2.22) + 65° = 72° \tag{9}$$

Figure 6: Stable, beautifully folded, moderately weathered siltstone slope (height = 25m; slope angle = 65°) from a large open pit mine in Western Australia.

3 ROCK SLOPE DESIGN WITH BOREHOLE GEOPHYSICS DATA AND Q-SLOPE

Barton (2006) derived a general relation between the Q-value and P-wave velocity by normalizing the Q-value with Equation 10 where unconfined compressive strength (UCS or σ_c) is in megapascals (MPa) and Q_c is the normalized Q-value. P-wave velocity (V_p) in kilometres per second (km/s) can be then be estimated using Equation 11.

$$Q_c = \frac{\sigma_c}{100} xQ \tag{10}$$

$$V_p \approx 3.5 + \log Q_c \tag{11}$$

Rearranging Equation 9 to estimate the normalized Q-value, results in Equation 12.

$$Q_c \approx 10^{(V_p - 3.5)} \tag{12}$$

The Q-value, and therefore, the normalized Q-value (Q_c) does not consider the orientation of geological structures relative to the proposed rock slope design and the environmental conditions in which the slope will be constructed. That is, the discontinuity orientation factor (O-factor) and environmental and geological conditions number (J_{wice}) have not yet been considered. SRF_{slope} in most cases should be equal to one as stress reduction factors were already considered in the Q-value relationship with V_p. Equation 13 approximates Q-slope by relating it to the normalized Q-value:

$$Q_{slope} \approx (Q_c)_0 \, x \, \frac{J_{wice}}{SRF_{slope}} \tag{13}$$

3.1 EXAMPLE BENCH FACE ANGLE DESIGN FROM BOREHOLE GEOPHYSICS

P-wave (V_p) and S-wave (V_s) velocities and several other geophysical attributes can be derived from full waveform acoustic logging of boreholes (Cheng & Toksoz, 1981). Similarly, acoustic (ATV) and optical (OTV) televiewer can be used to identify and measure the orientation of geological structures from vertical or inclined boreholes (Thomas et al. 2015). Figure 7 presents samples from a case study from an open cut mine in Western Australia associated with below the water table siltstones and sandstones where borehole geophysics was practicable.

Differences in V_p are observed between the different weathering grades of siltstone and the sandstone and V_p increases with depth (range 100-250 metres below natural surface). Based on the V_p data, a decreasing degree of fracturing with depth is expected (and was verified through drill core logging). From Figure 7 only (a typical sample of data), differences between the materials are evident with V_p values listed in order from closest to the surface to deepest:

- MW Siltstone - $V_p \approx 3.40$ km/s.
- SW Siltstone - $V_p \approx 3.80$ km/s
- SW Sandstone - $V_p \approx 4.25$ km/s.

S-wave velocity (V_s) appears to display a distinct difference between siltstone and sandstone, irrespective of the degree of weathering. Poisson's Ratio (ν) generally appears to be similar in both siltstone and sandstone. It should be noted that only very limited geophysics data was available from the moderately weathered siltstone due to its close proximity to the top of the groundwater table. As a result, in the stereographic projections obtained from ATV (acoustic televiewer) only, moderately and slightly weathered siltstone ground types are combined. As illustrated in Figure 7, the orientation of pervasive geological structures varies between the siltstone and the sandstone. These are interpreted against the proposed bench scale (12-24m high) slope angle and orientation to derive the O-factor(s) and J_{wice}.

Table 2 presents data obtained from borehole geophysics data for the estimation of Q-slope and β using Equations 1 and 3, respectively. Bench face slope angles (β) derived from geophysics and Q-slope increased with higher P-wave velocity and intact rock strength in the different ground types. The orientation of geological structure also contributed, particularly in the stronger material.

Table 2: Q-slope estimation from borehole geophysics data

Ground Type	V_p (km/s)	σ_c (MPa)*	Approximate Q	O-factor		J_{wice}	Approximate Q-slope^	β (°) (PoF=1%)
				Set A	Set B			
MW Siltstone	3.40	35	0.60	1	1	1	0.60	61
SW Siltstone	3.80	50	1.50	1	1	1	1.50	69
SW Sandstone	4.25	75	4.22	0.75	N/A	1	3.16	75

* *σ_c was derived from laboratory testing and not geophysics.*

^ *SRF_{slope} was equal to one in this instance and not included in the table.*

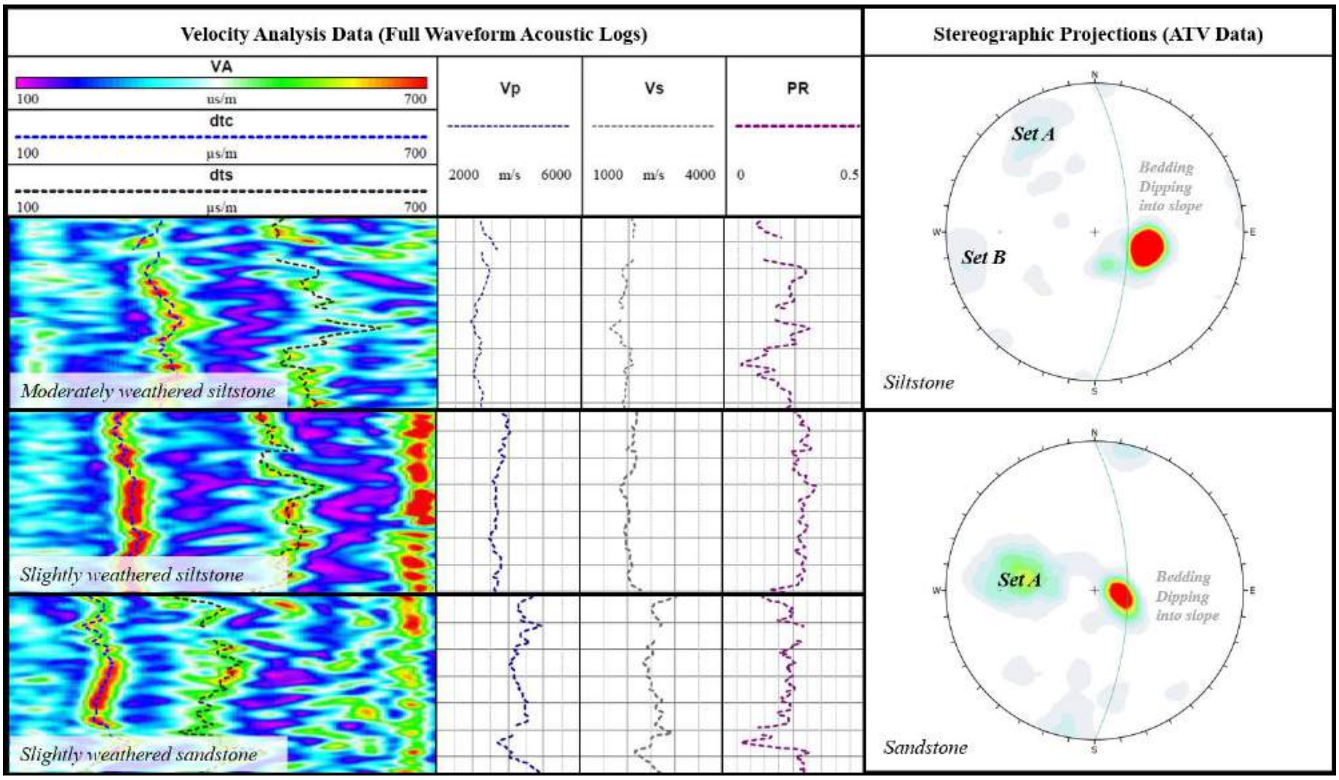

Figure 7: Samples of borehole geophysics data. Left: Full waveform acoustic downhole V_p (P-Wave velocity) V_s (S-Wave velocity & PR (Poisson's Ratio) logs in moderately & slightly weathered siltstone and slightly weathered sandstone. Right: Stereographic projections for geological structures obtained from acoustic televiewer.

4 DISCUSSION

The initial development of the Q-slope method was stimulated by the need to suggest 'width of forest clearing' for a future motorway with numerous planned cuttings and embankments in hilly terrain in Panama. The only information available was about 1 km of shallow drill-core, and numerous seismic refraction profiles with P-wave velocities (usually with three depth intervals). There were old road cuttings in the neighbourhood, and the condition of these old slopes (somewhat variable) was of course an advantage in formulating a potential Q-slope versus slope-angle. The general principal for estimating forest-clearing width, which would also apply to a 'green field' opencast development was that the slopes would get successively steeper when progressing from saprolite, through weathered rock, to sound rock in the approx. 20-50m high cuttings. As we have seen in Table 2, velocities follow suit as Q and Q-slope values are (generally) increasing with depth. It is interesting in this connection, to compare Q-histograms as depth is increased in the same lithology. Nearly all the Q-parameters are seen to 'move to the right' – in a more convincing (and 'scientific') way than simply moving a 'cross-hatched circle' in a GSI diagram. Figure 8 shows an example from Panama motorway logging (Barton, 2011).

Q-slope can be applied to rock slope engineering problems irrespective of rock strength, degree of fracturing, degree of weathering, etc. It also remains unchanged whether it is being used as a predictive or retrospective analysis tool. However, Q-slope cannot be applied to soil masses, rock fill, or landslide debris.

Our experiences have continuously shown that Q-slope enables geotechnical engineers and engineering geologists to rapidly and effectively assess the stability of rock slopes in the field, both during, and after excavation. Q-slope has been applied in both mining and civil engineering projects where it has been beneficial in:

- Reducing problematic and costly (both in financial and time losses) bench failures during construction.
- Reducing ongoing maintenance requirements as potentially problematic areas are identified and dealt with early.

- Identifying opportunities for steepening slope angles, reducing overburden excavation costs, and yielding additional revenue in the form of ore recovery in mining.

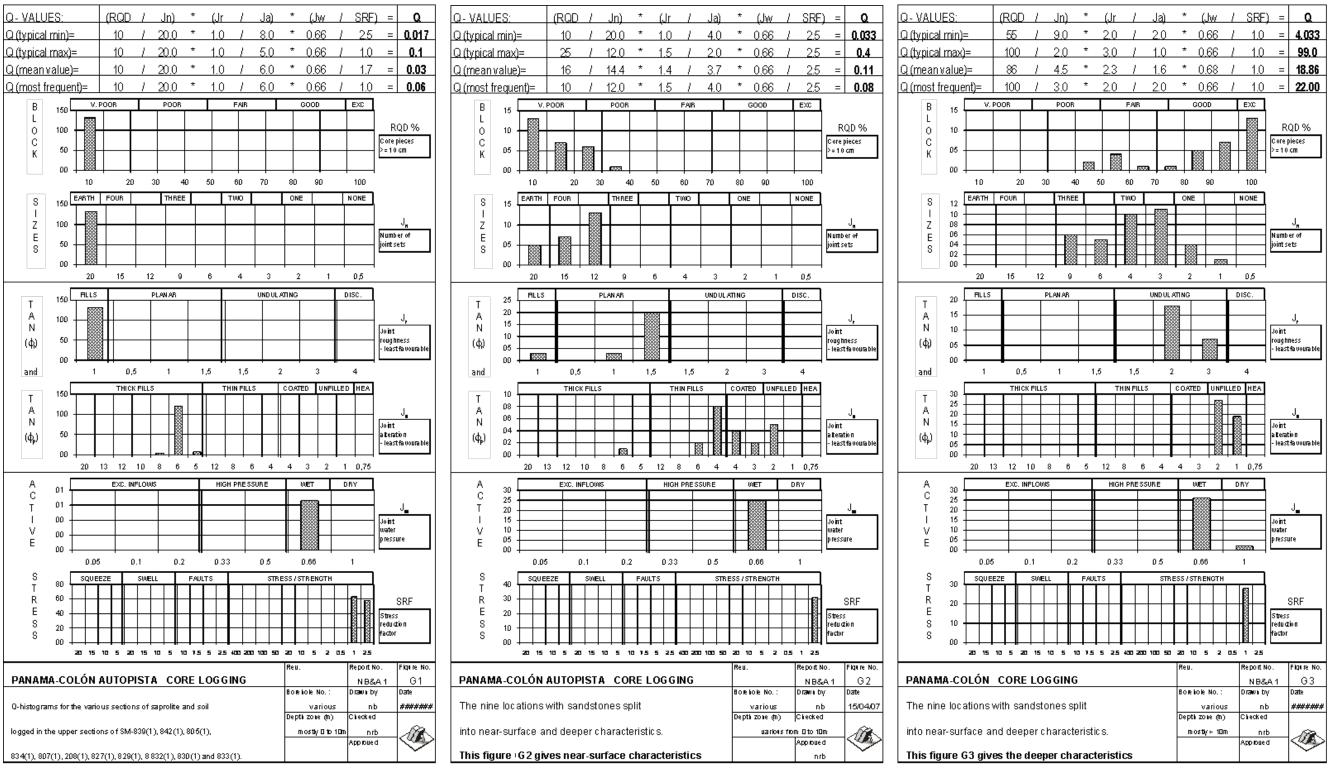

Figure 8: One method of collecting Q-value statistics when core-logging is the Q-histogram method. This was used when first applying Q-slope at a motorway project in Panama. Note the shift-to-the-right of most Q-parameters when progressing down a bore-hole from core-box to core-box in saprolite, weathered sandstone, sounder sandstone.

It is not the intention to promote Q-slope as a substitute for more rigorous analyses of slope stability. Where such is warranted, and where time permits, more rigorous analyses would always be preferred. For example, when dealing with larger slopes (heights in excess of 50m, or when several stages of excavation are required), the increased excavation time should permit more rigorous analyses to be made. However, engineers may sometimes need to respond at slope-construction rates of many tens of metres per day, stretching to hundreds of meters in the case of some large open cut mines or multiple-pit operations. In such cases, some quantifiable estimates, with significant *a posteriori* case record supporting evidence, may prove valuable because Q-slope is applicable at low cost and is rather fast.

Bar & Barton (2017) provide further insight and background to the Q-slope method for rock slope engineering.

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REFERENCES

Bar N, Barton N (2016) Empirical slope design for hard and soft rocks using Q-slope. *Proc. 50th US Rock Mechanics / Geomechanics Symp.* ARMA 2016, Houston, 26-29 June 2016. ARMA16-384.

Bar N, Barton N (2017) The Q-slope Method for Rock Slope Engineering. *International Journal of Rock Mechanics & Rock Engineering,* December 2017, Volume 50, Issue 12, Springer-Verlag: 3307-3322.

Bar N, Barton N, Ryan CA (2016) Application of the Q-slope method to highly weathered and saprolitic rocks in Far North Queensland. Ulusay et al. (eds.), *Rock Mechanics and Rock Engineering: From the Past to the Future; Proc. ISRM Int. Symp. Eurock 2016*, Cappadocia, 29-31 August 2016: 585-590.

Barton N (2006) *Rock Quality, Seismic Velocity, Attenuation and Anisotropy*, Taylor & Francis Group, London UK: 756p.

Barton N, Bar N (2015) Introducing the Q-slope method and its intended use within civil and mining engineering projects. Schubert & Kluckner (eds.), *Future Development of Rock Mechanics; Proc. ISRM Reg. Symp. Eurock 2015* & 64th Geomechanics Colloquium, Salzburg, 7-10 October 2015: 157-162.

Barton N, Grimstad E (2014) Forty years with the Q-system in Norway and abroad. Fjellsprengningsteknikk, Bergmekanikk, *Geoteknikk, NFF,* Oslo, 4.1-4.25, 25p.

Barton N, Lien R, Lunde J (1974) Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics* 6: 189-236.

Bieniawski Z.T (1976) Rock mass classification in rock engineering. Bieniawski (ed.), *Exploration for rock engineering*; Proc. Of the Symp., Cape Town, Balkema, 97-106.

Bieniawski Z.T (1989) *Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering*, New York, Wiley, pp272.

Cheng C.H, Toksoz MN (1981) Elastic wave propagation in a fluid-filled borehole and synthetic acoustic logs. *Geophysics Journal*, Volume 46-7: 1042-1053.

Duran A, Douglas K (2000) Experience with empirical rock slope design, *Proc. ISRM Int. Symp. Melbourne*, November 2000: 7p.

Laubscher D.H (1977) Geomechanics classification of jointed rock masses - mining applications. *Transactions of Institute of Mining and Metallurgy:* Section A Mining Industry, Volume 93, A1-A8.

Laubscher D.H, Jakubec J (2001) *The MRMR rock mass classification for jointed rock masses*. Hustrulid & Bullock (ed.), Underground Mining Methods: Engineering Fundamentals and International Case Studies, New York, AIME, 474-481.

Mawdesley C, Trueman R, Whiten WJ (2003*) Extending the Mathews stability graph for open stope design*. EBSCO Publishing: A27-A39.

Potvin Y, Hadjigeorgiou J (2015) Empirical ground support design of mine drives. *Proc. Underground Design Methods 2015, Perth,* ACG: 419-430.

Potvin Y, Hudyma M, Miller HDS (1988) The Stability Graph Method for Open Stope Design. *90th CIM AGM, Edmonton, Canada*, No.44: 1-28.

Romana M (1985) New adjustment ratings for application of Bieniawski classification to slopes. *Proc. ISRM Int. Symp. Role of Rock Mechanics in Excavations for Mining and Civil Works*, Zacatecas: 49-53.

Romana M (1995) The geomechanical classification SMR for slope correction. *Proc. 8th ISRM congress on Rock Mechanics 3, Tokyo,* September 1995: 1085-1092.

Sullivan T.D (2013) Global slope performance index. In Dight (ed.), *Proc. of Slope Stability 2013, Brisbane*, September 2013: 55-80.

Thomas R.D.H, Neilsen J.M, Wilson H.F, Lamb P (2015) Structural interpretation from Televiewer survey. Dight (ed.) *Proc. of FMGM 2015, Sydney*, September 2015: 729-742.

APPENDIX – Q-SLOPE INPUT PARAMETERS

Table A1: Rock quality designation (RQD)

Rock quality designation description		RQD (%)*
A	Very poor.	0-25
B	Poor.	25-50
C	Fair.	50-75
D	Good.	75-90
E	Excellent.	90-100

** Where RQD is reported or measured as ≤ 10 (including zero), a nominal value of 10 is used to evaluate Q-slope. RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.*

Table A2: Joint set number (J_n)

Joint set number description		J_n
A	Massive, no or few joints.	0.5-1
B	One joint set.	2
C	One joint set plus random joints.	3
D	Two joint sets.	4
E	Two joint sets plus random joints.	6
F	Three joint sets.	9
G	Three joint sets plus random joints.	12
H	Four or more joint sets, random, heavily jointed.	15
J	Crushed rock, earth like.	20

Table A3: Joint roughness number (J_r)

Joint roughness number description		J_r
a) Rock-wall contact, b) contact after shearing		
A	Discontinuous joints.	4
B	Rough or irregular, undulating.	3
C	Smooth, undulating.	2
D	Slickensided, undulating.	1.5
E	Rough or irregular, planar.	1.5
F	Smooth, planar.	1.0
G	Slickensided, planar.	0.5
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact.	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact.	1.0

i) Descriptions refer to small-scale features and intermediate scale features, in that order.
ii) Add 1.0 if mean spacing of the relevant joint set is greater than 3m.
iii) J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength.
iv) J_r and J_a classifications are applied to the discontinuity set or sets that are least favourable for stability both from the point of view of orientation and shear resistance τ, where $\tau \approx \sigma_n \tan^{-1} (J_r/J_a)$.

Table A4: Discontinuity orientation factor (O-factor)

O-factor Description	Set A	Set B
Very favourably oriented.	2.0	1.5
Quite favourable.	1.0	1.0
Unfavourable.	0.75	0.9
Very unfavourable.	0.50	0.8
Causing failure if unsupported.	0.25	0.5

TableA5: Environmental and geological conditions number (J_{wice})

J_{wice}*	Desert Environment	Wet Environment	Tropical Storms	Ice Wedging
Stable structure; competent rock.	1.0	0.7	0.5	0.9
Stable structure; incompetent rock.	0.7	0.6	0.3	0.5
Unstable structure; competent rock.	0.8	0.5	0.1	0.3
Unstable structure; incompetent rock.	0.5	0.3	0.05	0.2

**Note: When drainage measures are installed, apply J_{wice} x1.5.*
When slope reinforcement measures are installed, apply J_{wice} x1.3.
When drainage and reinforcement are installed, apply both factors J_{wice} x1.5 x 1.3.

Very favourably to favourably oriented joints forming columns and near-cubical blocks: Orange region, NSW.

Favourably oriented, (inconspicuous) relic foliations striking into slope of weak saprolite of phyllite: Cairns region, QLD.

Unfavourable foliation in phyllite, but if steep and striking along bench slope: quite favourable: Kalgoorlie region WA.

Very unfavourable bedding planes in siltstone. Vertical joints & joints dipping into slope favourable. Mount Isa region QLD.

Very unfavourable bedding planes causing failure in shale. Joints dipping into slope favourable. Newman region WA.

Extremely unfavourable quartzite bedding day-lighting and causing failure when unsupported. Great Sandy Desert WA.

Figure A1: Australian examples of discontinuity orientation factor application.

Table A6: Joint alteration number (J_a)

Joint alteration number description		Φ_r approx. (degrees)	J_a
a) Rock-wall contact (no clay fillings, only coatings)			
A	Tightly healed, hard non-softening, impermeable filling, i.e. quartz or epidote.	-	0.75
B	Unaltered joint walls, surface staining only.	25-35	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25-30	2.0
D	Silty- or sandy-clay coatings, small clay disintegrated rock, etc.	20-25	3.0
E	Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.	8-16	4.0
b) Rock-wall contact after some shearing (thin clay fillings, probable thickness ≈ 1-5mm)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30	4.0
G	Strongly over-consolidated non-softening clay mineral fillings.	16-24	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings.	12-16	8.0
J	Swelling-clay fillings, i.e. montmorillonite. Value of Ja depends on per cent of swelling clay-size particles, and access to water.	6-12	8-12
c) No rock-wall contact when sheared (thick clay/crushed rock fillings)			
M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition).	6-24	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening).	-	5.0
OPR	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition).	6-24	10, 13, or 13-20

Table A7: Strength reduction factors (maximum of SRF_a, SRF_b & SRF_c becomes SRF_{slope})

Strength reduction factor A: physical condition description					SRF$_a$
A	Slight loosening due to surface location, disturbance from blasting or excavation.				2.5
B	Loose blocks, signs of tension cracks & joint shearing, susceptibility to weathering, severe disturbance from blasting.				5
C	As B, but strong susceptibility to weathering.				10
D	Slope is in advanced stage of erosion and loosening due to periodic erosion by water and/or ice-wedging effects.				15
E	Residual slope with significant transport of material down-slope.				20
Strength reduction factor B: stress and strength description			σ_c / σ_1*		SRF$_b$
F	Moderate stress-strength range.		50-200		2.5-1
G	High stress-strength range.		10-50		5-2.5
H	Localized intact rock failure.		5-10		10-5
J	Crushing or plastic yield.		2.5-5		15-10
K	Plastic flow of strain softened material.		1-2.5		20-15
Strength reduction factor C: major discontinuity description		Favourable	Unfavourable	Very unfavourable	Causing failure if unsupported
L	Major discontinuity with little or no clay.	1	2	4	8
M	Major discontinuity with RQD_{100} = 0^ due to clay and crushed rock.	2	4	8	16
N	Major discontinuity with RQD_{300} = 0^ due to clay and crushed rock.	4	8	12	24

* *σ_c = unconfined compressive strength of intact rock (UCS); σ_1= maximum principal stress.*
^ *RQD_{100} = 1 metre perpendicular sample of discontinuity; RQD_{300} = 3 metres perpendicular sample of discontinuity.*