

Q-slope – rock slope engineering 10 years on

Neil Bar^{1,2}

1 Gecko Geotechnics LLC
PO Box 1574, Kingstown VC0100, Saint Vincent and the Grenadines
e: neil@geckogeotech.com
t: +7 701 029 6851

2 Budapest University of Technology and Economics
Department of Engineering Geology and Geotechnics
Műegyetem rkp. 3, 1111, Budapest, Hungary

Nick Barton³

3 Nick Barton & Associates
Fjordveien 65c, 1363 Høvik, Oslo, Norway
e: nickrbarton@hotmail.com

Abstract

The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of rock slopes in the field. It enables geotechnical engineers and engineering geologists to make adjustments to slope angles as ground conditions become apparent during the excavation of reinforcement-free slopes in civil engineering and mining projects.

Q-slope was developed by supplementing the Q-system which has been extensively used for 50 years for characterizing rock exposures, drill core and tunnels under construction. 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 (O-factor) for each side. The term J_w has been replaced with the more comprehensive term J_{wice} , which takes into account long-term exposure to various climatic and environmental conditions such as intense erosive rainfall and ice-wedging effects. SRF (strength reduction factor) categories have been developed for slope surface conditions, stress-strength ratios and major discontinuities such as faults, weakness zones or joint swarms.

Through over 600 case studies in 36 rock types across five continents, a simple relationship between Q-slope and long-term stable slope angles has been established. It includes several failure modes and applies to slopes ranging from less than 5 m to more than 250 m in height. This paper discusses Q-slope application and use for the last 10 years. It presents updated Q-slope stability charts and discusses the time dependent behaviour of rock slopes.

Highlights

- Q-slope method now supported by a database of over 600 case studies, which is provided to the reader for their use and future research.
- Updated slope stability charts for empirically assessing rock slope stability and selecting appropriate slope angles.
- The Q-slope method for rock slope engineering has been adopted in over 45 countries around the world over the last 10 years.

Keywords

- Q-slope
- Slope stability
- Civil engineering
- Mining engineering

- Rock mass classification
- Empirical method

Abbreviations

- Q_{slope} – Q-slope value for assessing rock slope stability
- Q_{slope}' – Modified Q-slope value that does not consider orientation factors, external factors and stress
- β – Long term stable slope angle in degrees
- RQD – rock quality designation
- J_n – joint sets number
- J_r – joint roughness number
- J_a – joint alteration number
- O-factor – orientation factor for the ratio J_r/J_a
- J_{wice} – environmental and geological condition number
- $\text{SRF}_{\text{slope}}$ maximum 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
- σ_c – uniaxial compressive strength of intact rock in megapascals

1.0 Introduction

Rock slope failures on man-made and natural slopes include slope instability, rock falls and landslides, as well as debris flows and shallow landslips in weathered rock (Wyllie 2017). Socioeconomic consequences of rock slope failures include direct costs such as removing the failed rock debris and stabilizing the slope, and a wide range of indirect costs. Indirect costs may include:

- Civil engineering: potential losses of life, damage to vehicles and injury to passengers on highways and railways, traffic delays, business disruptions, loss of tax revenue due to decreased land values, and flooding and disruption of water supplies where rivers are blocked by slides
- Mining engineering: potential losses of life, damage to mining equipment and injury to mine personnel, mine production delays, impairment or sterilization of mineral resources, reputational damage, and mine or mining company closure.

Documented examples of socioeconomic consequences of rock slope failures include:

- The Vajont slide in Italy in 1963 inundated a reservoir, sending a wave over the crest of the dam that destroyed the town of Langarone and resulted in over 2,000 fatalities (Kiersch 1965; Ghirotti et al 2013)
- The Manefay Slide at the Bingham Canyon copper mine in 2013 was estimated to be over 70,000,000 m³ in volume and cost over \$1 billion to clean up. In Salt Lake County, Utah, sales-tax revenues reduced by 3.4% by the end of 2013 and were attributed to the slope failure (Ward 2015)
- In Medellin, Colombia, where bedrock is highly weathered and intense rainfall events occur, 74% of deaths resulting from natural hazards are due to slope failures. A single slope failure in 1974 resulted in over 770 fatalities (Carvajal et al. 2012).

Between 2004 and 2016, a total of 55,997 fatalities were recorded globally from 4,862 individual, non-seismic slope failures including landslides (Froude and Petley 2018). Slope failure occurrence triggered by human activity such as construction, illegal mining and hill cutting is increasing. Froude and Petley (2018) identified that the majority of landslides that were not initiated by rainfall or earthquakes, were triggered by human activity such as:

- Mining (232 multi-fatality events; 67 single fatality events)
- Construction (170 multi-fatality events; 140 single fatality events)

- Illegal hill cutting (60 multi-fatality events; 27 single fatality events).

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 FEM/DEM (numerical) modelling. The rate of excavation advance is usually too fast for this (Bar and Barton 2017).

Several empirical methods for assisting rock engineering design have been developed in the last 50-60 years and are used for a variety of applications by rock and mining engineers. By way of example in tunnelling, underground mining and other underground excavations, the following empirical methods are commonly used to derive appropriate support and reinforcement for specific excavation spans:

- Q-system (Barton et al. 1974)
- RMR: rock mass rating (Bieniawski 1976; Bieniawski 1989)
- MRMR: mining rock mass rating (Laubscher 1977; Laubscher and Jakubec 2001).

For the case of rock slope stability, empirical methods were less frequently used and either simple kinematics, numerical modelling or ‘no modelling’ may often be chosen (i.e. slope angles selected by equipment operators or foremen rather than engineers). SMR (slope mass rating) and C-SMR (continuous slope mass rating) were developed to predict the support, reinforcement and performance of excavated slopes (Romana 1985; Romana 1995; Tomás et al. 2007, 2012). However, no empirical rock engineering methods provide guidance in relation to appropriate, long-term stable slope angles in which rock reinforcement and support is *deliberately absent*. Such slopes actually dominate the demand by a huge margin.

The Q-slope method for rock slope engineering was introduced in 2015 with the purpose of enabling engineering geologists and rock 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 and Bar 2015; Bar and Barton, 2017). Prime areas of application are ‘from-surface-and-downwards’ bench angle decisions in open pits, and for the numerous slope cuttings needed to reach remote hydropower projects, tunnel and dam sites, often through strongly varying structural geologies.

Tens to hundreds of thousands of kilometres of rock slopes, without support, are made each year, for roads and railways, in remote hydropower projects, accesses for tunnel and dam sites, and for benches in surface mines (open pits) and quarries. In most cases, slopes are excavated through variable geological conditions with changing lithologies, weathering profiles and geological structures or discontinuities. A series of ‘interesting’ but troublesome and potentially unsafe local failures is often the result, during or after excavation. Quite often these are the result of adverse plane failures, wedge failures, or more rarely local toppling. Complex combinations of these mechanisms can occur, and can also coincide with shearing through weak rock masses.

Application of Q-slope can help to reduce maintenance (and bench-width needs) due to all the potential failures and can help to improve safety by reducing slope failure occurrences due to human activity.

2.0 The Q-slope Method for Rock Slope Engineering

Q-slope development from the Q-system utilized the same six parameters RQD, J_n , J_r , J_a , J_w and SRF. 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 $(J_r / J_a)_{J1} \times 0.7$ for set J1 and $(J_r / J_a)_{J2} \times 0.9$ for set J2, provide estimates of overall whole-wedge frictional resistance reduction, when appropriate. The term J_w , which is now termed J_{wice} (one of two symbol-modifications), takes into account a wider range of environmental conditions appropriate to rock slopes, which are constructed in the open and exposed to the elements 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.

The Q-slope method requires the assignment of ratings for rock quality designation (RQD), joint set number (J_n), joint roughness number (J_r) and joint alteration number (J_a), which remain unchanged from the original Q-system (Barton et al. 1974). For Q-system users, Equation 1 for estimating Q-slope is mostly familiar (Bar & Barton 2017, 2018a, 2018b; Barton & Bar, 2015, 2019):

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

As with the Q-system, which comprises of three quotients that provide crude measures of block size, inter-block shear strength and active stress, the quotients in Q-slope also provide indicative estimates of:

1. Relative block size: (RQD / J_n) .
2. Inter-block shear strength: least favourable (J_r / J_a) or average shear strength in the case of wedges: $(J_r / J_a)_1 \times (J_r / J_a)_2$.
3. External factors and stress: (J_{wice} / SRF_{slope}) .

Inter-block shear resistance, τ , is approximated using Equation 2:

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

Modified Q-slope, or Q-slope' is defined by Equation 3 in which orientation factors, external factors and stress are removed in a similar manner to Q'.

$$Modified\ Q_{slope} = Q_{slope}' = \frac{RQD}{J_n} \times \left(\frac{J_r}{J_a} \right) \quad (3)$$

A visual demonstration of the ultra-simple initial objective of Q-slope, is shown in Figure 1. It shows the differently inclined slopes of a high cutting for the new Panama Canal extension that were geotechnically designed by the Panama Canal Authority geotechnical engineers. Q-slope is designed such that it suggests similar safe, maintenance-free, reinforcement-free bench-face slope angles of, for instance, 40-45° in saprolite, 60-65° in weathered rock and 80-85° in fresher rock for respective Q-slope values of approximately 0.1, 1.0 and 10 (Barton and Bar 2015).

Figure 2 presents another visual demonstration of Q-slope application in a large surface mining (open pit) operation in Laos. In this case, dominant geological structure orientations (i.e. bedding planes) become more favourable with depth, allowing for steeper bench face angles (Bar and Barton 2015). Such anisotropic behaviour is typical in many rock masses, as emphasized by Barton and Quadros (2015).

The following sections describe the Q-slope input parameters and provide examples to guide engineering geologists and geotechnical engineers.

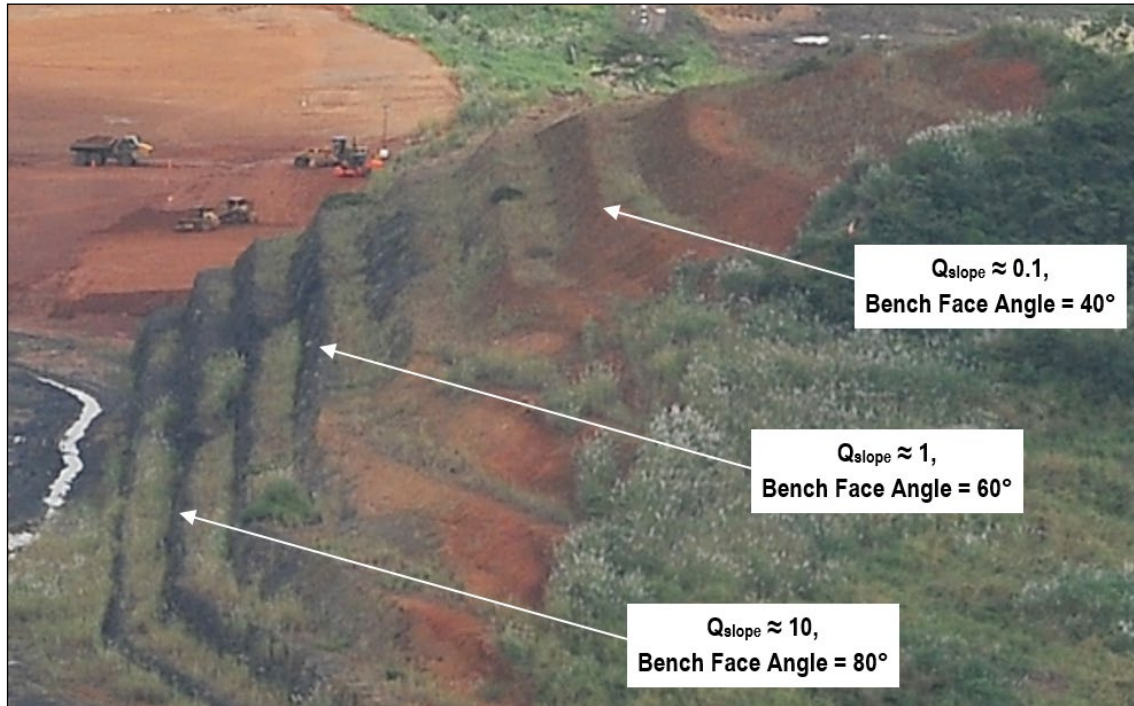


Figure 1. A convenient example of differentiated saprolite, weathered rock and fresher rock mass qualities, help to explain the appropriately steepened, unreinforced permanent slopes (courtesy of Panama Canal Authority, Barton and Bar 2015).

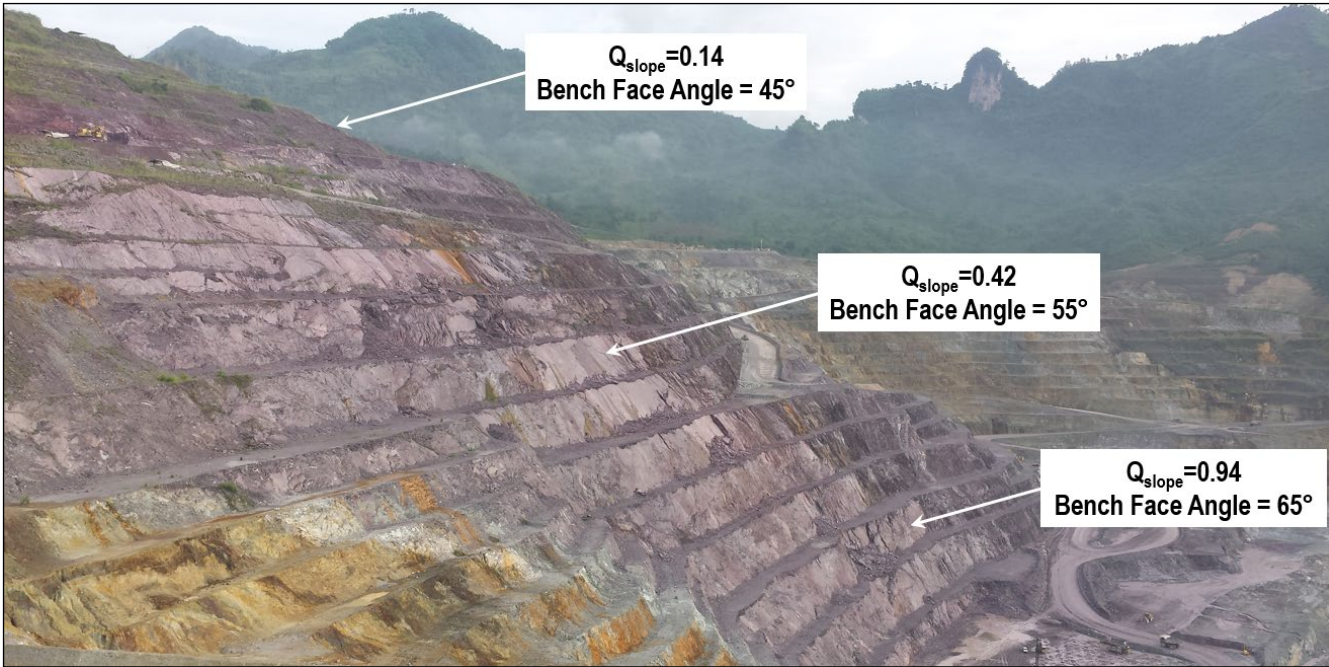


Figure 2. Changing geologic structure orientations (steepening of bedding planes downslope) help to explain the appropriately steepened bench face angles in a large open cast mine setting, Laos (Bar and Barton 2017). However, generally improved rock mass quality at depth may also influence this, as in Figure 1.

2.1 RQD

The rock quality designation index, RQD, is adopted to provide a quantitative estimate of in-situ rock mass quality from drill core logs (Deere 1963; Deere et al. 1967).

Table 1 describes the ratings for RQD for use in Q-slope. In cases where RQD is estimated or measured as being equal to or less than 10 (or even zero), a nominal value of 10 is used to evaluate Q-slope. In other cases, RQD intervals of 5 (i.e. 100, 95, 90, etc.) are sufficiently accurate.

When selecting any input parameters for a slope face, local variability is almost certain. Adopting ranges of input parameters, where appropriate, is strongly recommended, particularly for RQD (e.g. saying RQD ranges between 25% and 35% is encouraged rather than simply noting RQD=30%).

In the specific case of determining RQD from slope faces or other rock outcrops, physically measuring RQD using a rod or tape as described by Hutchinson & Diederichs (1996) is favoured over visually ‘estimating’ or ‘guessing’. This provides a similar estimation to core-based RQD.

Table 1: Q-slope input parameter: 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

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

2.2 J_n

The joint set number, J_n, is the rating for the number of joint (discontinuity) sets in the same domain as described in Table 2. The term 'joint' is adopted from the original Q-system but refers to any type of discontinuity or geological structure when estimating J_n.

When estimating J_n, all discontinuity or geological structure sets need to be accounted for (i.e. not just joints). This includes bedding and foliation in sedimentary and metamorphic rocks, respectively.

A joint or discontinuity can occur singly as a random joint, or appear more frequently as joint sets (Mandl 2005). A joint set (or discontinuity set) is a family of near-parallel, similarly spaced discontinuities with similar physical or geomechanical properties. Figure 3 provides a series of photographs showing different rock masses with corresponding J_n and RQD values as a point of general reference.

Table 2: Q-slope input parameter: 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

Note: All discontinuity or geological structure sets need to be accounted for (i.e. not just joints)

The first quotient in Equation 1 (RQD / J_n) represents the overall geological structure of the rock mass, or the overall degree of fracturing. The quotient gives a very crude measure of the relative block size.

The combined use of RQD and J_n (relative block size) in Q-slope, significantly reduces the RQD limitations described by Palmström (2005). Figure 3 demonstrates this effect by adapting Figure 2.7 to distinguish between different types of rock masses with similar RQD values.

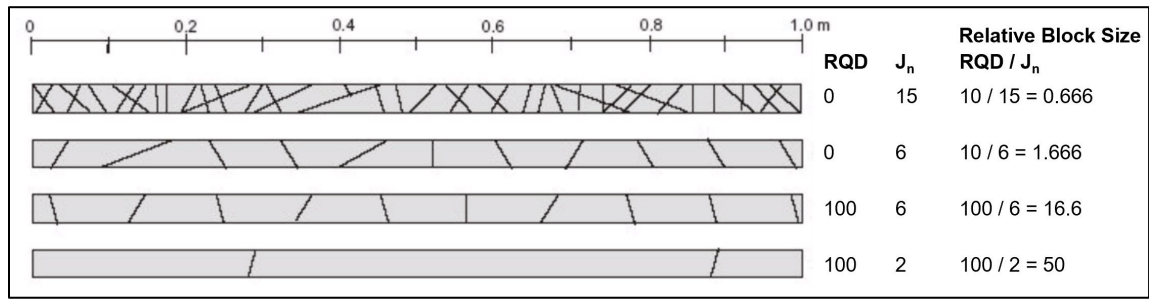


Figure 3: Example of relative block size (RQD / J_n) use for distinguishing between different rock masses with similar RQD values; Note: Where RQD is reported or measured as ≤ 10 (including zero), a nominal value of 10 is used to evaluate Q-slope



A. $J_n = 1$. Occasional random joints in road cutting in tuff (note: slope is 8m high), Türkiye.



B. $J_n = 2$. One joint set – bedding in a coal mine (note: slope is 60m high), very low RQD, Colombia.



E. $J_n = 6$. Two joint sets (foliation sub-parallel to slope and vertical joints striking into slope) and random joints in phyllite, Australia.



F. $J_n = 9$. Three joint sets forming cubical blocks in granite, RQD=100%, Norway.



G. $J_n = 12$. RQD: 75-90%. Three joint sets and random joints in sub-horizontally bedded sandstone, Australia



H. $J_n = 15$. RQD=0%. Heavily jointed granodiorite adjacent to regional scale fault zone (note: pen for scale), Papua New Guinea.

Figure 4: Photographs of joint set number examples (J_n) with corresponding rock mass RQD values

2.3 J_r

The joint roughness number, J_r, is the rating for the roughness of the least favourably oriented joint (discontinuity) set in the same domain as described in Table 3. The least favourably oriented discontinuity set is referred to as Set A hereafter.

If required, Set B can be incorporated in cases of potentially unstable wedge formations. In such cases, J_r ratings should be recorded individually for discontinuity sets A and B.

Sketched profiles of J_r ratings and respective JRC (joint roughness coefficient) values are displayed in Figure 5. It should be noted that the sketched profiles (I to IX) are at least 100 cm in length (Barton 1987).

Table 3: Q-slope input parameter: J_r

Joint roughness number description		J _r
<i>a) Rock wall contact, b) contact after shearing</i>		
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
<i>c) No rock-wall contact when sheared</i>		
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

Notes: i) Descriptions refer to small-scale features and intermediate scale features, in that order.

ii) Add 1.0 to J_r rating if mean spacing of the relevant joint set is greater than 3 m.

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 classification 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 $\tau \approx \sigma_n \tan^{-1} (J_r/J_a)$.

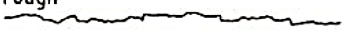

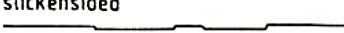
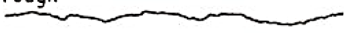


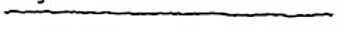
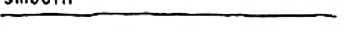

Relation between J_r and JRC_n Subscripts refer to block size (cm)		J_r	JRC_{20}	JRC_{100}
I	rough 	4	20	11
II	smooth 	3	14	9
III	slickensided 	2	11	8
Stepped				
IV	rough 	3	14	9
V	smooth 	2	11	8
VI	slickensided 	1.5	7	6
Undulating				
VII	rough 	1.5	2.5	2.3
VII	smooth 	1.0	1.5	0.9
IX	slickensided 	0.5	0.5	0.6
Planar				

Figure 5: Suggested methods for the quantitative description of discontinuity classes using J_r and JRC_n (subscripts refer to block size in centimetres). Note: the sketched profiles (I to IX) are at least 100 cm in length (Barton, 1987)

2.4 J_a

The joint alteration number, J_a , is the rating for the surface alteration and infilling of the least favourably oriented joint (discontinuity) set described in Table 4.

In cases of potentially unstable wedge formations, individual J_a ratings should be recorded for discontinuity sets A and B alongside their respective J_r ratings.

Table 4: Q-slope input parameter: J_a

Joint alteration number description		J _a
<i>a) Rock wall contact (no clay fillings, only coatings)</i>		
A	Tightly healed, hard non-softening, impermeable filling, i.e. quartz or epidote.	0.75
B	Unaltered joint walls, surface staining only.	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0
D	Silty- or sandy-clay coatings, small clay disintegrated rock, etc.	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.	4.0
<i>b) Rock-wall contact after some shearing (thin clay fillings, probable thickness \approx 1-5mm)</i>		
F	Sandy particles, clay-free disintegrated rock, etc.	4.0
G	Strongly over-consolidated non-softening clay mineral fillings.	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings.	8.0
J	Swelling-clay fillings, i.e. montmorillonite. Value of J _a depends on percentage of swelling clay-size particles, and access to water.	8-12
<i>c) No rock-wall contact when sheared (thick clay/crushed rock fillings)</i>		
M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition).	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening).	5.0
O P R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition).	10, 13, or 13-20

2.5 O-factor

The discontinuity orientation factor (O-factor) described in Table 5 provides orientation adjustments for discontinuities in rock slopes. Engineering judgement and a degree of subjectivity is required to select appropriate O-factor values. Figure 6 provides photographic examples of favourably and unfavourably oriented discontinuities.

The Set A orientation factor is applied to the most unfavourable discontinuity set. If required, the Set B orientation factor is applied to the secondary discontinuity set (e.g. in case of potentially unstable wedge formations).

Use of the O-factor in Equation 1 are described the examples in Table 10 and Figure 8.

Table 5: Q-slope input parameter: O-factor

O-factor Description	Set A	Set B (where applicable)
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



Very favourably to quite favourably oriented joints in columnar basalt, Australia.



Favourably oriented, (inconspicuous) relic foliation striking into slope of weak saprolite of phyllite, Australia.



Incipient, individually quite unfavorable relic joints forming wedge in weak saprolite, Panama.



Unfavourable foliation; but if steep and striking parallel to bench slope: quite favourable, Australia.



Very unfavorable bedding planes in siltstone. Vertical joints and joints dipping into slope favorable, Australia.



Very unfavorable bedding planes in siltstone. Joints dipping into slope favorable, Australia.



Extremely unfavorable joints daylighting into excavation and causing failure when unsupported, Papua New Guinea.

Extremely unfavourable quartzite foliation daylighting and causing failure when unsupported, Australia.

Figure 6: Photographs of O-factor examples

2.6 J_{wice}

The environmental and geological condition number, J_{wice} , is more sophisticated than J_w of the Q-system since slopes are outside and exposed to the elements for a very long time, arguably forever. Described in Table 6, J_{wice} has a new structure for slopes, including tropical rainfall erosion-effects and ice-wedging effects. Adjustment factors in case of slope reinforcement or drainage measures are also included.

Table 6: Q-slope input parameter: J_{wice}

Environmental and geological condition number, J_{wice} matrix					
J_{wice}^*		Desert Environment	Wet Environment	Tropical Storms	Ice Wedging
Geological Structure	Intact Rock				
Stable	Competent	1.0	0.7	0.5	0.9
Stable	Incompetent	0.7	0.6	0.3	0.5
Unstable	Competent	0.8	0.5	0.1	0.3
Unstable	Incompetent	0.5	0.3	0.05	0.2

**Note: When drainage measures are installed, apply $J_{wice} \times 1.5$.*

When slope reinforcement measures are installed, apply $J_{wice} \times 1.3$.

When drainage and reinforcement are installed, apply both factors $J_{wice} \times 1.5 \times 1.3$.

Competent rocks are generally durable, resistant to erosion and deformation, and not susceptible to slaking. Often these have a relatively high unconfined compressive strength, say 50 MPa and above. The estimate of J_{wice} should take into consideration the environmental conditions in which the slope is constructed, which will include the competence or otherwise of the rock, and therefore the likely long-term stability of possibly adverse geological structures. The most hostile or dynamic environmental conditions experienced by the slope should be adopted. For example, if a slope is constructed in a desert environment that regularly experiences extremely cold winters, freezing and thawing, it would be appropriate to adopt ice wedging as the most adverse environmental condition.

2.7 SRF_{slope}

The strength reduction factor (SRF_{slope}) is obtained by selecting the most adverse (i.e. maximum) of SRF_a , SRF_b and SRF_c , which consider physical condition, stress and strength and major discontinuities within the slope.

Table 7 describes strength reduction factors (SRF_a) for the physical condition of the slope surface (now or expected) due to susceptibility to weathering and erosion.

Table 7: Q-slope input parameter: SRF_a

Strength reduction factor: Physical Condition		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

Table 8 describes strength reduction factors (SRF_b) for adverse stress-strength ranges in the slope. SRF_b becomes more critical for weak, low strength materials such as highly weathered and saprolitic rocks, and also becomes more critical with increasing slope height, and therefore, with increasing stress.

Maximum principal stress (σ_1) may be estimated by considering in-situ stresses, material density and slope geometry. However, in the unlikely event that slope-related in-situ stress measurements are available, these may be preferred. This might apply in the case of high slopes.

Another fast and simple means of estimating σ_1 is by substituting it for the vertical stress, σ_v , expected in the slope failure mechanism. This is most applicable for small slopes.

Table 8: Q-slope input parameter: SRF_b

Strength reduction factor: Stress and Strength		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

*Note: σ_c = unconfined compressive strength (UCS).

σ_1 = maximum principal stress.

Table 9 describes strength reduction factors (SRF_c) for major discontinuities such as faults, weakness zones and joint swarms which may also contain clay filling that adversely affects slope stability. Figure 7 displays photographic examples of major discontinuities.

Major discontinuities may or may not have a similar orientation to a discontinuity set, such as a joint set or bedding plane. However, major discontinuities are typically single features with considerably different geomechanical properties (i.e. lower shear strength due to soft, plastic infilling).

Table 9: Q-slope input parameter: SRF_c

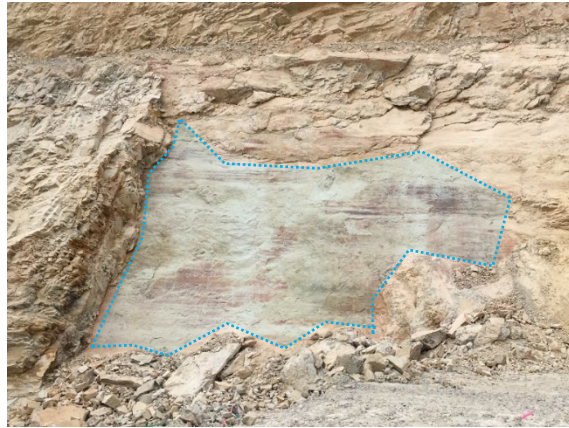
Strength reduction factor: Major Discontinuity, SRF_c matrix					
SRF_c^*		Favourable	Unfavourable	Very unfavourable	Causing failure if unsupported
L	Major discontinuity with little or no clay	1	2	4	8
M	Major discontinuity with $RQD_{100} = 0$ due to clay and crushed rock	2	4	8	16
N	Major discontinuity with $RQD_{300} = 0$ due to clay and crushed rock	4	8	12	24

*Note: $RQD_{100} = 1$ metre perpendicular sample of discontinuity.

$RQD_{300} = 3$ metres perpendicular sample of discontinuity.



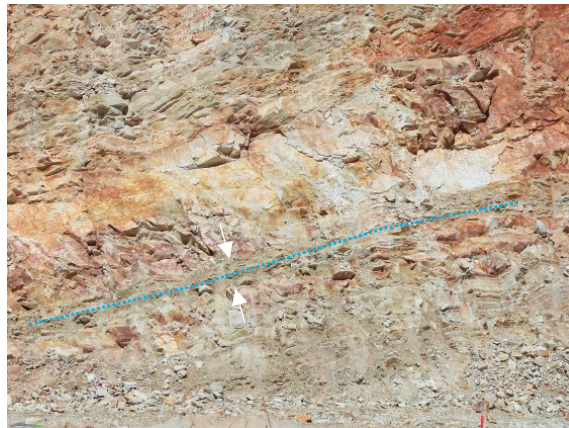
L. Favourable fault striking into slope with no clay (width <20 cm) in quartzite, Australia.



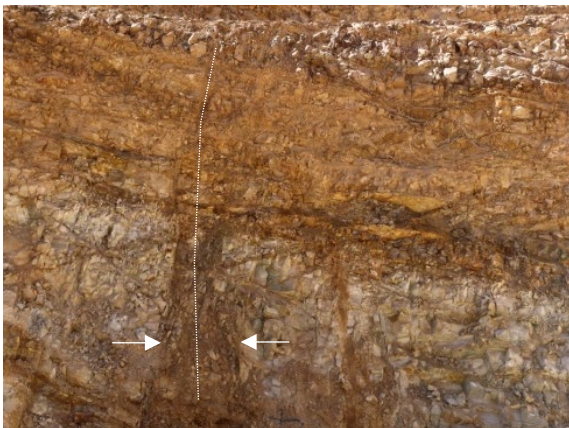
L. Very unfavourable fault sub-parallel to slope with no clay (width <10 cm) in siltstone, Australia.



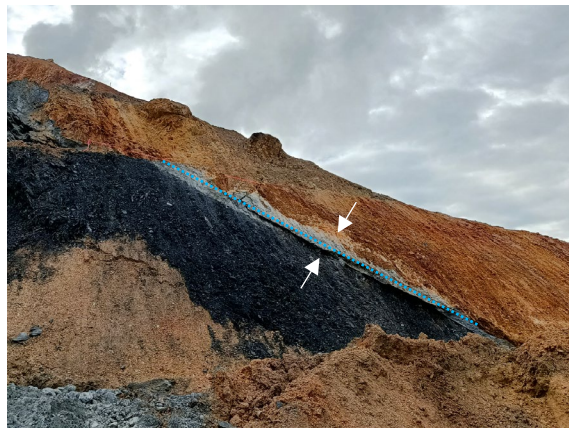
M. Favourable fault with $RQD_{100} = 0$ due to crushed rock (width 1 m) striking into slope in quartzite, Australia.



M. Unfavourable thrust fault with $RQD_{100} = 0$ due to crushed rock and some clay (width 1 m) in sandstone, Australia.



N. Favourable fault with $RQD_{300} = 0$ due to crushed rock (width 3.5 m) striking into slope in sandstone, Australia.



N. Very unfavourable fault with $RQD_{300} = 0$ due to clay (width 2-4 m) causing failure if unsupported, Dominican Republic.

Figure 7: Photograph examples of Major Discontinuities

3.0 Q-slope application examples

Table 10 and Figure 8 provide two examples of Q-slope application assist with use of Equation 1.

The plane failure was a 20m high slope was excavated with a slope angle of 75° in strong monzonite ($\sigma_c = 90$ MPa). An outward dipping joint set (~60°) caused planar failure a few days after excavation as illustrated in Figure 8 (left). The following Q-slope ratings were assigned during back-analysis:

- RQD = 75-90%
- $J_n = 6$
- $J_r = 1.5$, $J_a = 2$, O-factor = 0.25 (Set A only)
- $J_{wice} = 0.7$ (wet environment, competent rock and generally stable structure).
- $SRF_a = 2.5$, $SRF_b = 1$, $SRF_c = N/A$.

Based on the assigned ratings, Q-slope was estimated as follows (i.e. with the application of a single discontinuity set and O-factor):

$$Q_{slope} = \frac{82.5}{6} \times \left(\frac{1.5}{2} \times 0.25 \right) \times \frac{0.7}{2.5} = 0.722$$

Table 10: Q-slope application examples

Example	RQD (%)	J_n	Set A			Set B			J_{wice}	SRF_a	SRF_b	SRF_c	Q-slope
			J_r	J_a	O	J_r	J_a	O					
Plane Failure	75-90	6	1.5	2	0.25	-	-	-	0.7	2.5	1	-	0.722
Wedge Failure B	80	9	1	3	0.25	1.5	2	0.9	0.5 x 1.5	2.5	1	1	0.150

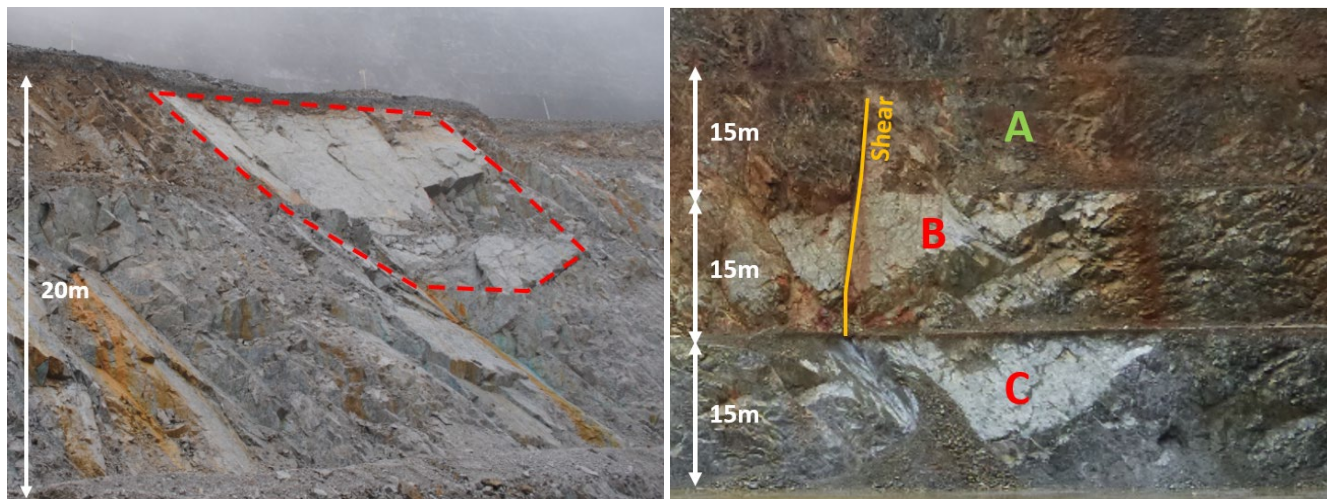


Figure 8: Q-slope application examples relating to Table 10. Left: Example for plane failure. Right: Example for wedge failure 'B'

Wedge failures occurred on successive 15m high benches that were excavated at angles of 65°. Similar wedge failures occurred on both benches in strong monzodiorite ($\sigma_c = 70$ MPa) as illustrated in Figure 8 (right). The following Q-slope ratings were assigned during the back-analysis of the first wedge failure 'B':

- RQD = 80%
- $J_n = 9$

- Set A: $J_r = 1$, $J_a = 3$, O-factor = 0.25
- Set B: $J_r = 1.5$, $J_a = 2$, O-factor = 0.9
- $J_{\text{wice}} = 0.5 \times 1.5$ (wet environment, competent rock but unstable structure. Horizontal weep holes installed for drainage).
- $\text{SRF}_a = 2.5$ (slight loosening due to surface location), $\text{SRF}_b = 1$, $\text{SRF}_c = 1$ (shear striking into slope).

Based on the assigned ratings, Q-slope was estimated as follows (i.e. with the application of a two discontinuity sets and O-factors):

$$Q_{\text{slope}} = \frac{80}{9} \times \left[\left(\frac{1.5}{3} \times 0.25 \right) \left(\frac{1.5}{2} \times 0.9 \right) \right] \times \frac{0.5 \times 1.5}{2.5} = 0.150$$

Several working examples of Q-slope application have been presented in previous publications by Bar and Barton (2017, 2018a, 2018b).

4.0 Q-slope database

In rock slope engineering, two key service levels exist for rock slopes: ultimate limit state (slope failure or collapse) and the serviceability limit state (onset of excessive deformation or unacceptable deterioration).

In the Q-slope method for rock slope engineering case studies, the status of rock slopes has been assessed in the field using the following design criteria:

- Failed – slope has failed or collapsed, in which the assessment focuses on the collapsed portion of the slope. Such slopes have been back-analysed using known pre-failure geometries, ground and environmental conditions.
- Quasi-stable – slope has not yet collapsed; however, either the onset of excessive deformation or unacceptable deterioration is evident. This includes visible signs of instability such as tension cracks, dislocation or accelerating deformation trends identified by means of monitoring instrumentations (e.g. survey prisms, surface extensometers, etc.). Such slopes are more than likely to collapse soon with rainfall or weathering effects.
- Stable - slope is stable and no visual signs of instability are observed at least several weeks, months or years post-excavation.

For the purpose of designing long-term, stable or maintenance-free slope angles, both failed and quasi-stable conditions are considered unwanted or undesirable outcomes.

A total of 602 case records were collected between 2012 and 2023, which includes 49% more cases than presented by Bar and Barton (2017). Each of the case records was a real slope that was retrospectively assessed in the field using the Q-slope method in a variety of conditions, including different:

- Stability status: 419 cases were stable (70%), 12 were quasi-stable (2%) and 171 were failed (28%)
- Slope geometries
- Industries and engineering application; civil (42% of cases) and mining (58%)
- Geographic locations; including 17 countries across 5 continents
- Variable environmental and climatic settings
- Geological settings; including 36 rock types: 172 igneous case records (28%), 100 metamorphic (17%), 284 sedimentary (47%) and 46 with no rock type recorded (8%)
- Failure mechanisms
- Slope related time horizons; including stable time and time before failure.

The case records in the Q-slope database comprised 30% unwanted or undesirable outcomes (failed and quasi-stable slopes) and 70% stable slopes as shown in Figure 9.

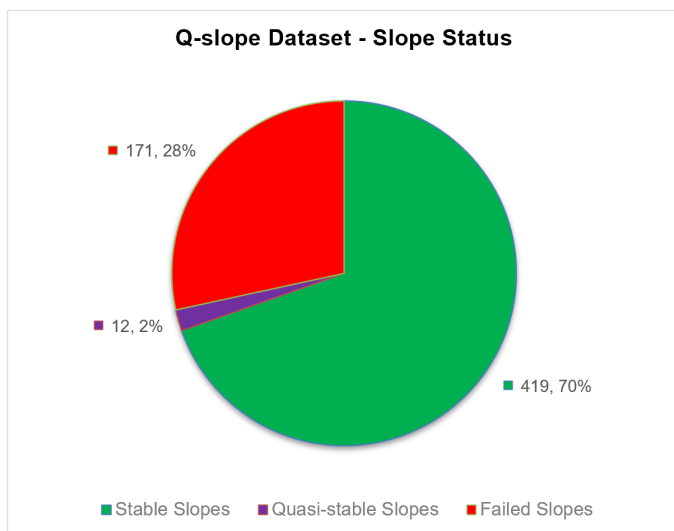


Figure 9: Q-slope database – stability status (30% unwanted events: failed and quasi-stable)

The case records in the Q-slope database include various slope heights and slope angles as shown in Figure 10.

Greater than 90% of case records were obtained from slopes with heights smaller than 60 m. Such slopes, and particularly those smaller than 30 m in height (i.e. >75% of case records) are often excavated in a very short period of time. Whilst it is not the intention to promote Q-slope as a substitute for more rigorous analyses of slope stability, conventional analytical and numerical methods for block and wedge stability although ‘fast’, cannot reasonably be done while accompanying new rock slope excavations on an hour-by-hour basis while stationed in the field. For larger slopes, the increased time-scale should permit more rigorous analyses to be made, which are both warranted and advised.

Greater than 85% of case records were obtained from slopes with angles greater than 45°. Shallower rock slopes are less common and are typically saprolitic to highly weathered, or heavily influenced by unfavourable geological structure.

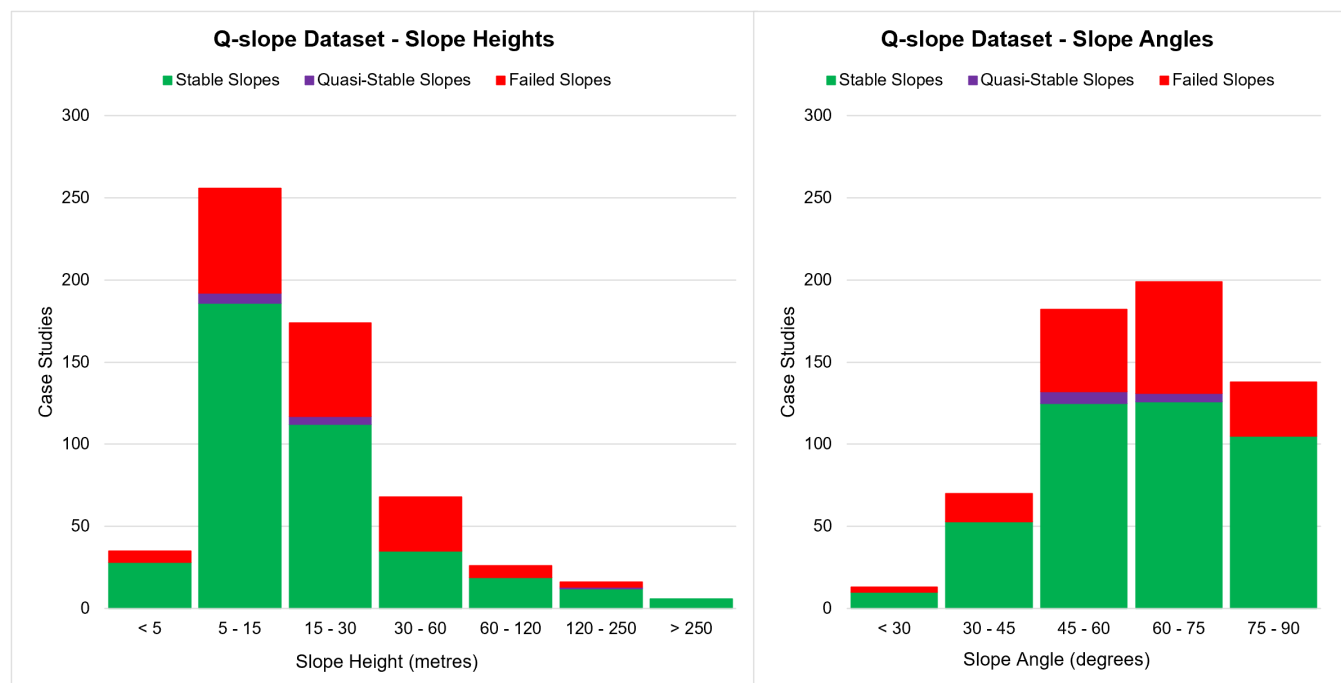


Figure 10: Q-slope database – slope heights and slope angles

The case records in the Q-slope database are relatively evenly spread across the civil engineering (42%) and mining (58%) industries.

Application within civil engineering has been focused on roads and highways (>90% of case records) due to availability and safe access as shown in Figure 11. Consequently, few case records are available from railways and dam access roads. Approximately 5% of case records are from residential and light industrial property development in hilly terrain.

As shown in Figure 11, mining industry application is spread across various mining commodities and scales of mine production. Over 25% of case records are from high production iron ore and coal mines. Large open pit gold, diamond, and base metal (copper, nickel, etc.) mines comprise 65% of the case records. Smaller scale mining applications including aggregate and marble quarries comprise less than 10% of the case records. It should be noted that the marble (stone) quarry case records use no blasting in their operations.

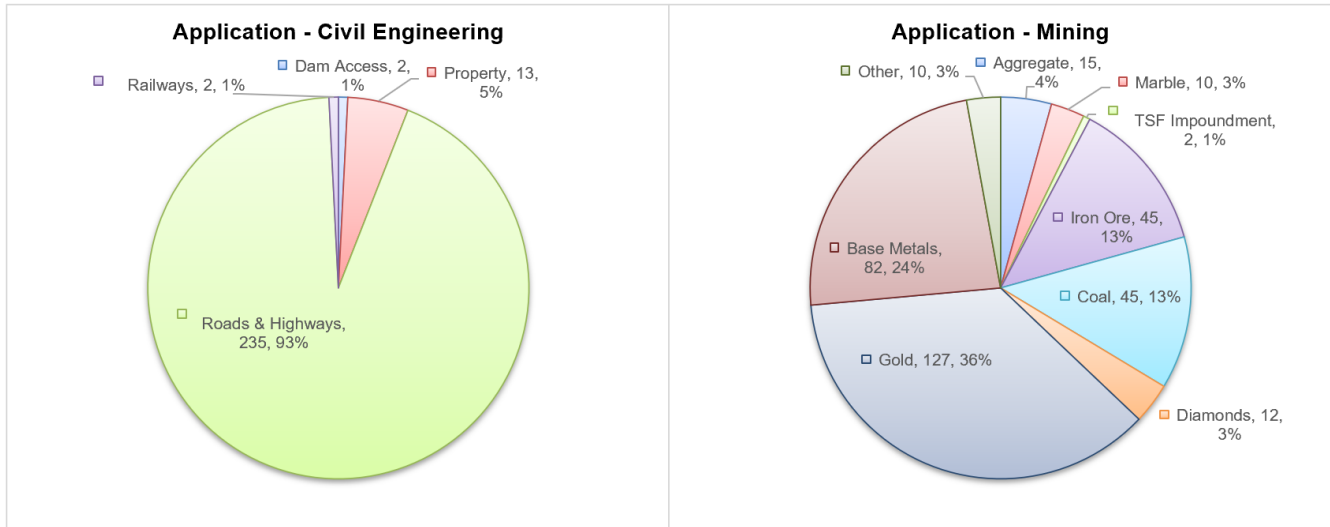


Figure 11: Q-slope database – engineering applications

The case records in the Q-slope database are geographically spread across five continents and grouped into four regions as shown in Figure 12. Approximately 50% of the case records are from Australia, with the rest spread across Europe, Asia and the Americas. Similarly, the case records are from different environments and climatic settings.

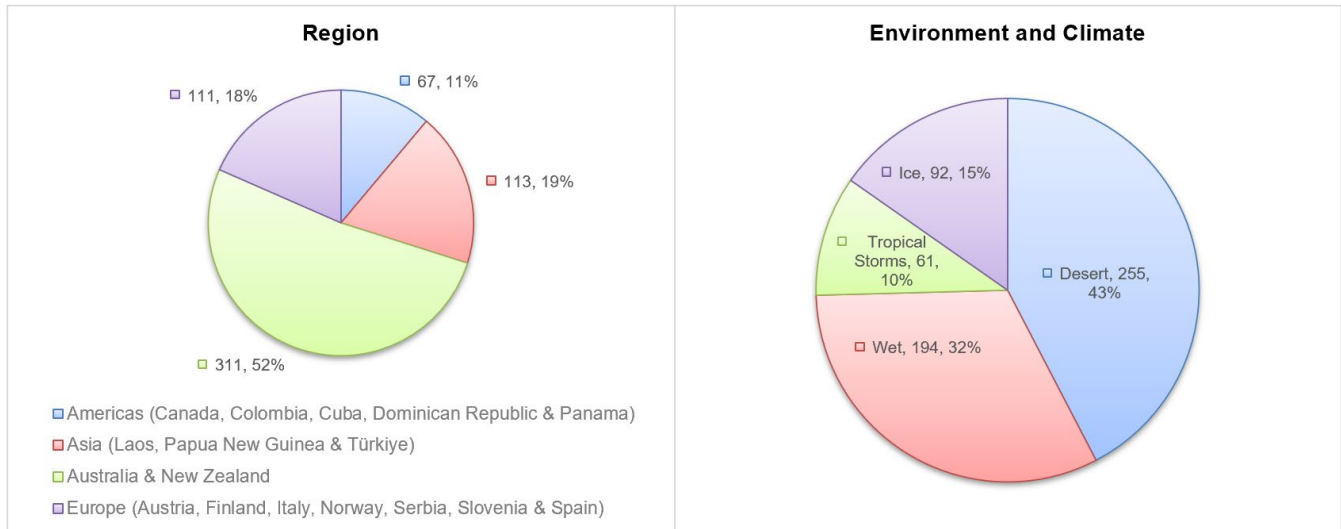


Figure 12: Q-slope database – geographic regions, environment and climatic settings

The case records in the Q-slope database are from various geological settings and comprise different rock types as follows:

- Igneous rocks (15 types)
- Metamorphic rocks (10 types)

- Sedimentary rocks (11 types).

Table 11 lists the frequency of occurrence of different rock types. Sedimentary rocks account for almost 50% of the case records.

Approximately 6% of the case records were comprised of saprolites of the original rock types. Rock types were not recorded for 8% of the case records.

Table 11: Q-slope database – frequency of occurrence of rock types

Igneous		Metamorphic		Sedimentary	
Agglomerate	7	Gabbro	1	Banded Iron Formation	28
Amphibolite	8	Gneiss	14	Breccia	2
Andesite	22	Greywacke	7	Chert	2
Basalt	25	Marble	13	Claystone	6
Carbonatite	4	Metasandstone	11	Conglomerate	3
Diorite	24	Phyllite	18	Limestone	69
Dolerite	7	Quartzite	10	Mudstone	14
Granite	24	Schist	13	Sandstone	62
Granodiorite	1	Skarn	9	Shale	26
Kimberlite	5	Slate	4	Siltstone	53
Monzonite	16			Interbedded Sandstone and Siltstone	19
Porphyry	7				
Rhyolite	6				
Trachyte	3				
Tuff	13				

Intact rock strengths were estimated in the field based on ISRM (1981) and range from saprolites and weak, highly weathered rocks (UCS < 5 MPa) to fresh rocks (UCS > 100 MPa). Case records in the Q-slope database represent a variety of rock strengths as shown in Figure 13.

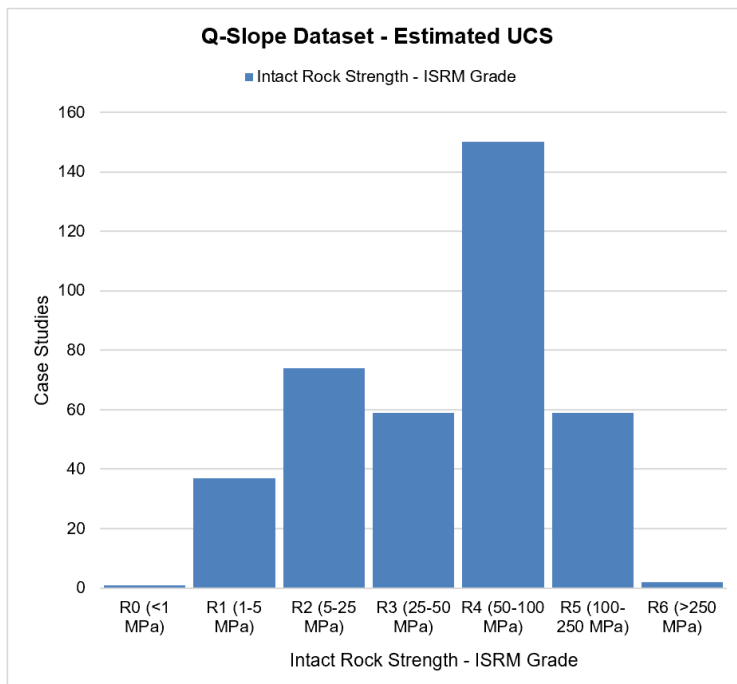


Figure 13: Q-slope database – intact rock strength

The ‘failed slope’ and ‘quasi-stable slope’ case records in the Q-slope database can be categorized into simple rock slope failure mechanisms as shown in Figure 14.

The back-analysed failed and quasi-stable slopes in the case studies were based on pre-failure slope geometry and contained a range of simple and complex failure mechanisms:

- Plane failure
 - 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 failure
 - 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
 - Block toppling
 - Flexural toppling (local)
- Circular
 - 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. However, where possible these were categorized into the failure mode driving the instability (e.g. plane, wedge, and less commonly, toppling).

Plane and wedge failure are the most common failure mechanisms observed in rock slopes accounting for over 80% of case records. Toppling is less frequent (11%) and circular failure is rare (3%) in rock slopes.

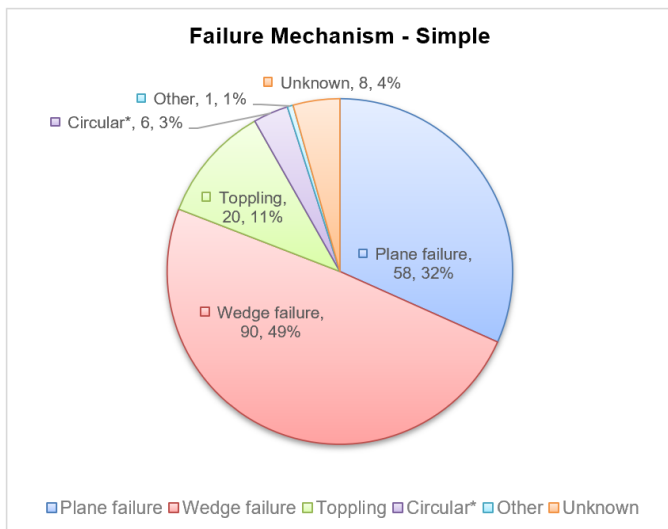


Figure 14: Q-slope database – simple rock slope failure mechanisms

In the case records where data was available (failed slopes only), the time before failure (or stand-up time) was recorded and categorized as shown in Figure 15.

Greater than 60% of slopes that failed, failed within one month after excavation and greater than 90% of all slopes that failed, failed within six months.

Similarly, where data was available (stable slopes only), the stable time or stable period of slopes was recorded and categorized as shown in Figure 14.

Over 75% of slopes have been stable for over six months, and almost 60%, stable for at least two years.

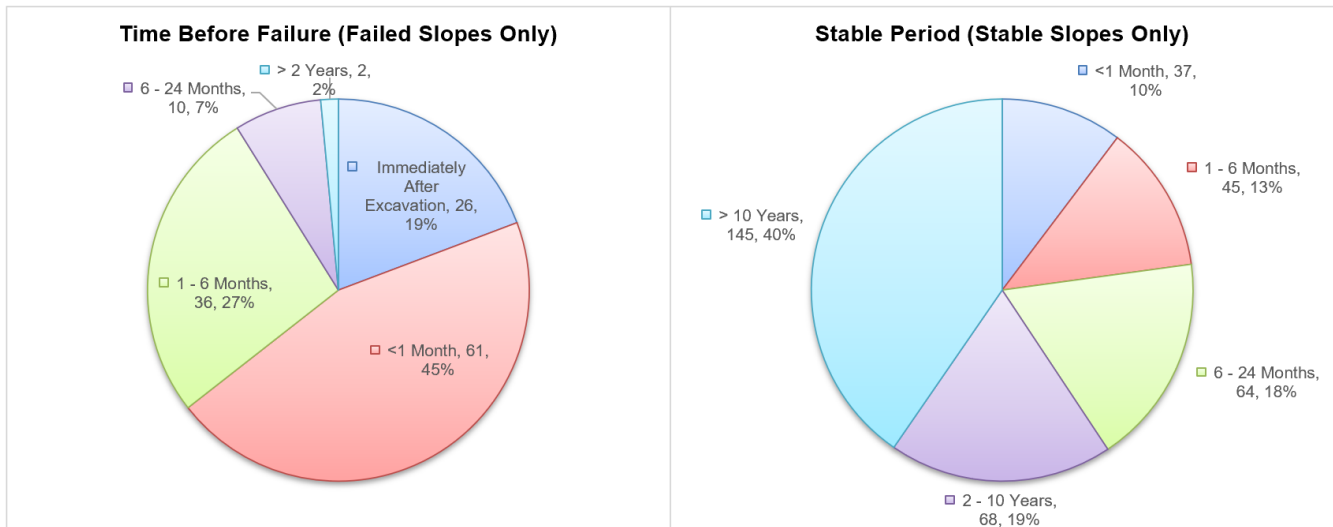


Figure 15: Q-slope database – time horizons: time before failure or stand-up time for failed slopes only; and stable period for stable slopes only

5.0 Q-slope relationship with slope angles

A simple trend or relationship between Q-slope and slope angles is visible in Figure 16, which illustrates the available Q-slope data derived from the back-analysis of rock slopes:

- Triangles indicate stable slopes. No visual signs of instability observed during at least several weeks, months or years post-excavation.
- Squares indicate quasi-stable slopes (more than likely to collapse soon with rainfall or weathering effects). Visible signs of slope instability such as tension cracks, dislocation or deformation by means of monitoring (survey prisms or surface extensometers) are being continuously observed.
- Crosses indicate failed or collapsed slopes that have been back-analysed using known pre-failure geometries (i.e. pre-failure slope angles) and ground conditions.

Equation 4 matches the lower boundary of stable slope angles (bounded by failed slopes) greater than 25° and less than 85° as shown in Figures 15 and 16. Long-term stable slope angles are represented by β .

$$\beta = 20 \log_{10} Q_{slope} + 65^\circ \quad (4)$$

Equation 4 does not represent a specific factor of safety as would be obtained by undertaking limit equilibrium or numerical analyses. Rather it represents the boundary of long-term stable slopes based on observed performance; normally between six months and over 10 years (Figure 15). The case records for failed slopes give extra confidence to long-term stability since only 7% of ‘failed slopes’ collapsed six months or longer after being excavated (i.e. the highest risk period is shortly after excavation).

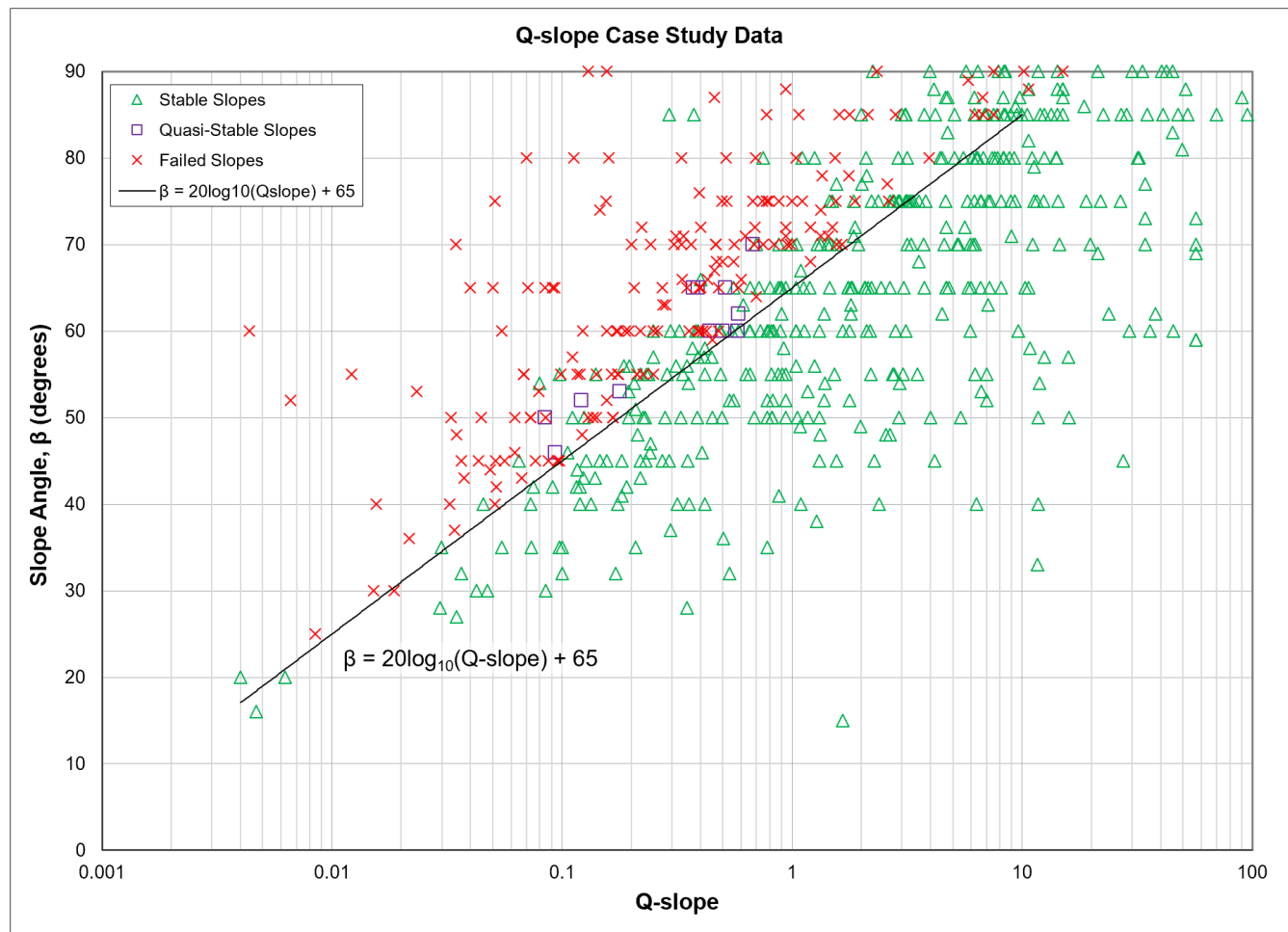


Figure 16: Q-slope database – 602 case records

Equation 4 is graphically displayed in Figure 16 and has only two instances in which failed slopes have pre-failure angles equal to β . There are no instances in which pre-failure slope angles of failed case records are lower than β . Based on this, a probability of failure (PoF) of less than 1% is associated with Equation 4.

Q-slope users may, if they wish, additionally apply a factor of safety on the steepest slope angle (β) not requiring reinforcement or support.

Barton and Bar (2015) suggested the following correlations, which 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° .

6.0 Probability of failure based on unwanted events

Considering only the failed and quasi-stable slopes, both of which are undesirable or unwanted events, the probability of failure (PoF) was calculated and is displayed using iso-potential lines in Figure 17.

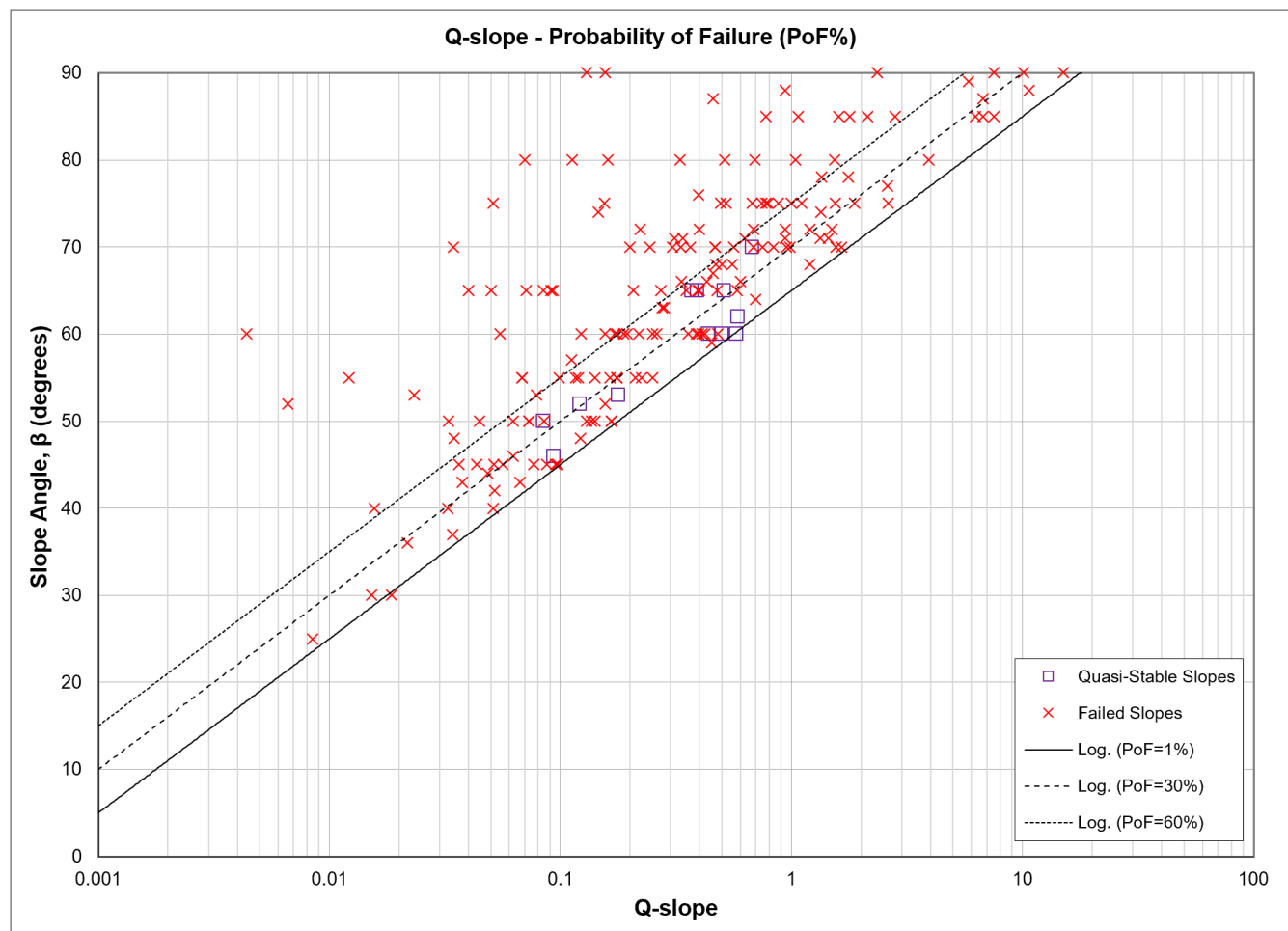


Figure 17: Q-slope probability of failure based on unwanted events (171 failed and 12 quasi-stable slopes) using parallel iso-potential lines representing Equations 5 to 7

In projects where certain degrees of failure are accepted, such as percentages of individual benches in surface mines, then the following equations can be derived:

$$PoF = 1\%: \quad \beta = 20 \log_{10} Q_{slope} + 65^\circ \quad (5)$$

$$PoF = 30\%: \quad \beta = 20 \log_{10} Q_{slope} + 70^\circ \quad (6)$$

$$PoF = 60\%: \quad \beta = 20 \log_{10} Q_{slope} + 75^\circ \quad (7)$$

It is acknowledged that these iso-potential lines are one possible interpretation of the data, and that other similar interpretations are also possible. In fact, initial interpretations using iso-potential lines based only on two-thirds of the current case records yielded slightly different PoF relationships (Bar and Barton 2017). However, Equations 5 and 6 for PoF=1% and PoF=30%, respectively remain effectively unchanged.

7.0 Q-slope stability chart

The Q-slope database was used to develop an updated slope stability chart that provides guidance on expected slope stability conditions. The Q-slope stability charts in Figures 17 and 18 have been provided to assist geotechnical engineers and engineering geologists using Q-slope and Equations 4 and 5 to differentiate between the following stability conditions:

- Stable slopes
- Unstable slopes
- Slope stability uncertain.

Figure 18 shows the development of the Q-slope stability chart, which is based on:

- Green area for stable slopes bounded by:
 - PoF = 1% or Equation 4 as the upper limit
 - A minimum Q-slope value of 0.004 since no case records of stable slopes had lower Q-slope values
 - A maximum β of 85° since sub-vertical slopes are highly susceptible to localized crest failures and rock falls, particularly where blasting is used during the excavation process
- Red area for unstable slopes bounded by:
 - PoF = 30% or Equation 6 as the lower limit. A PoF of 30% is often considered as the maximum acceptable probability of failure for individual benches in many surface mines where safety risks are managed well and such failures have no significant impact on costs or delays to mine production (Karzulovic, 2004)
 - A minimum β of 22° which corresponds to a Q-slope value of 0.004 (minimum Q-slope value in case records) in Equation 1.6
- Grey area for slope stability uncertain between the green and red areas.

Figure 19 presents the updated Q-slope stability chart without the case records. This has been updated 10 years after its inception by Barton and Bar (2015).

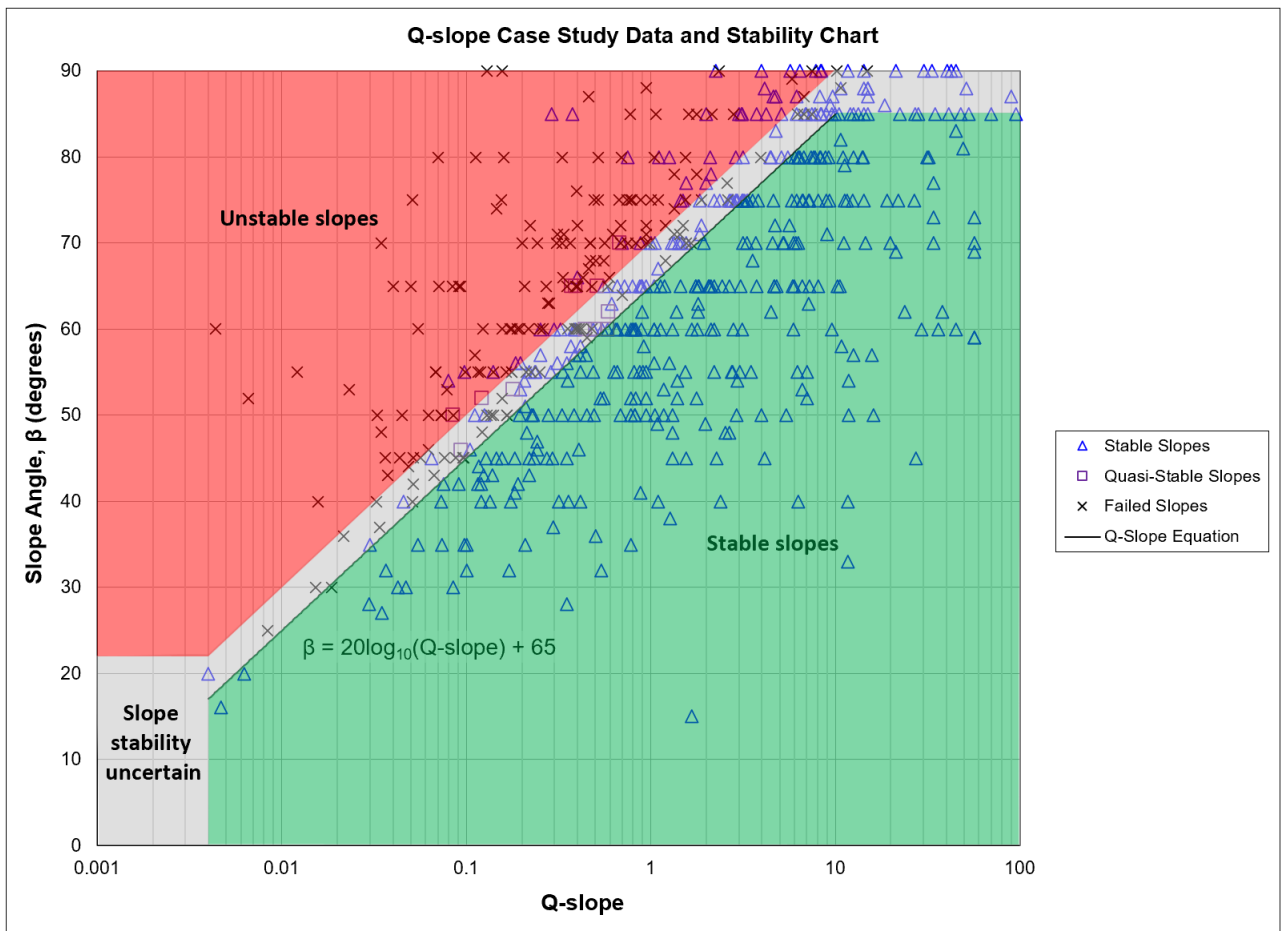


Figure 18: Q-slope database – 602 case record and basis of stability chart

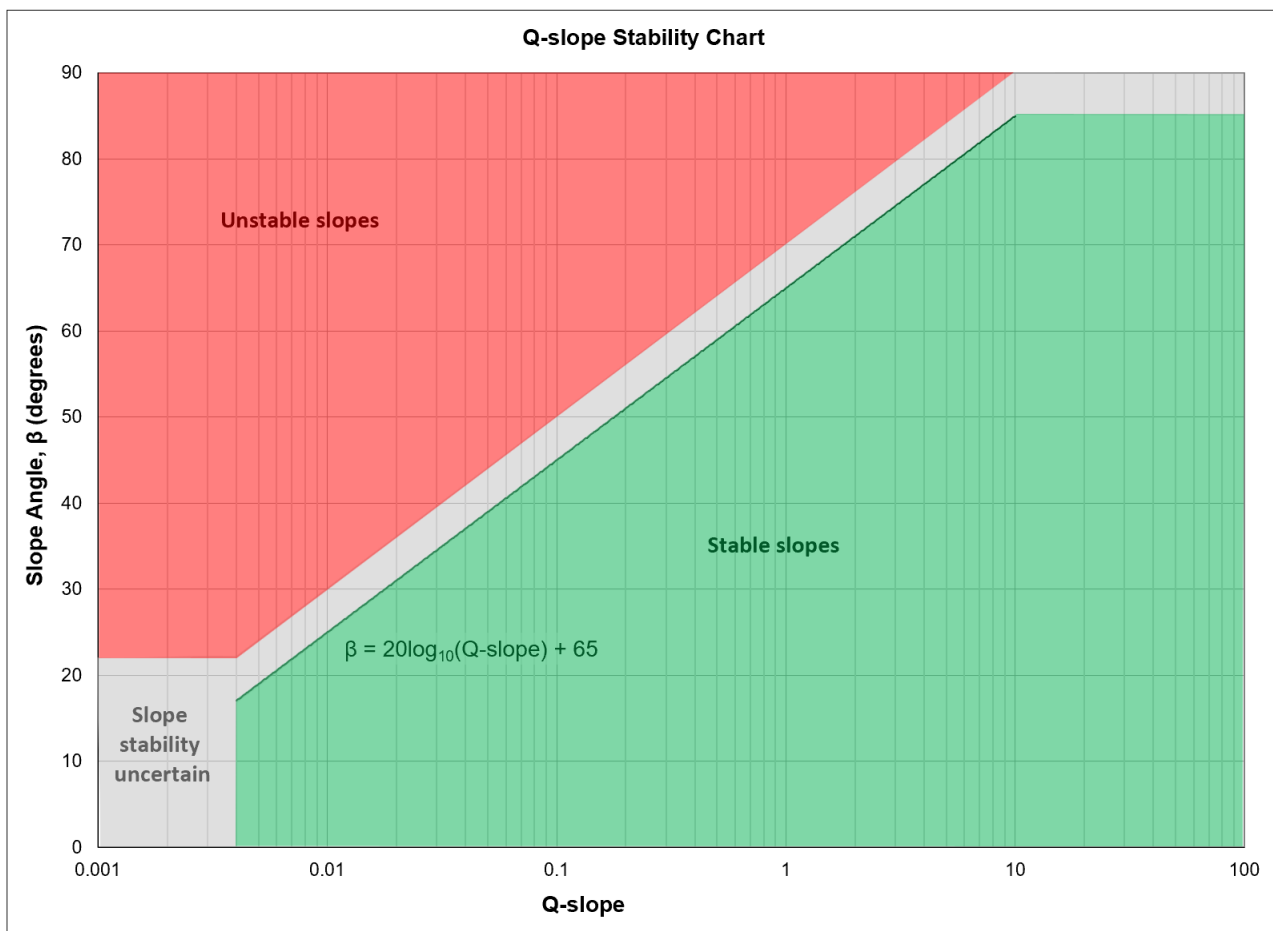


Figure 19: Q-slope stability chart – updated 10 years after inception

8.0 Discussion and limitations

Q-slope can be applied 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.

Q-slope cannot be applied to soil masses, rock fill or landslide debris.

When using Q-slope to determine the steepest slope angle (β), it is important to check the results from Equations 4 or 5 against the Q-slope stability chart in Figure 19 for guidance on expected slope stability conditions. This particularly refers to the ‘slope stability uncertain’ areas where supporting Q-slope data is limited.

Engineering judgement is used in all empirical methods. This is also true for Q-slope where the input parameters may be, in some instances, open to interpretation in the field.

When selecting input parameters for a slope face, local variability is almost certain. Adopting ranges of input parameters, where appropriate, is strongly recommended (e.g. recording RQD ranges between 25% and 35% is encouraged rather than simply noting RQD=30%).

Where uncertainty remains about an individual, or multiple parameters, an engineer should not be tempted to accept the results from a single Q-slope evaluation. Rather, the engineer should undertake a set of parametric studies in which that parameter or those parameters are varied over a credible range of values to obtain an understanding of the sensitivity of a design to these variations.

In the specific case of determining RQD from slope faces, physically measuring RQD using a rod or tape is favoured over visually ‘estimating’ or ‘guessing’. This provides a similar estimation to core-based RQD.

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 approximately 50 m in height (i.e. which require several stages of excavation), more rigorous analysis is both warranted and advised. Bar and McQuillan (2021) review several coal mine case studies where different lithological units (interbedded sandstone and siltstone) with quite similar rock mass quality using Q-slope.

The use of strength reduction factors becomes ever more critical in weak materials and as slope heights increase. In both these instances, SRF_b , has a tendency to dominate and as such, appropriately estimating in-situ stress (σ_1) in the slope is vitally important.

Even though in most cases intact rock strength (σ_c) increases with depth, as slope heights increase, the stress and strength factor (SRF_b) may increase ‘faster’ and as a result, reduce Q-slope and subsequently the slope angle (β).

A more generally applicable discussion of the effect of applying Q-slope to increasingly high slopes should include the advisory that the stress and strength factor (SRF_b) may not be the only parameter affected by the increased slope height. It is logical to expect that RQD and J_n could vary in either favourable or unfavourable directions, depending on improved conditions in the lower slope (normal case) or worsened conditions in the case of a more jointed rock type at depth. The J_r/J_a ratio (and potential wedge formation) might be more adverse for the higher slope since larger features may now be ‘sampled’, possibly involving one or more intersecting faults that did not influence the bench-scale stability. A fifth and sometimes vitally important factor is J_{wice} , because the larger slope height may be affected by a less fully drawn down phreatic surface than the usually fully drained bench-scale slopes. Nevertheless, weather-related J_{wice} conditions will apply quite strongly to the near-surface benches.

In contrast, for stronger materials or slopes of smaller heights, stability is often dictated by individual or sets of geological structures, or the presence of major faults (SRF_c). Their mechanical characteristics and orientations are likely to dictate stability. Barton and Bar (2019) review several cases where major faults and fault zones influence stability.

When slope angles are adjusted, even though the orientation of a geological structure does not physically change, its orientation (and O-factor) may become more or less favourable. An increase in slope angle may make a geological structure that was initially considered to be quite favourable, now unfavourable (e.g. a joint set sub-parallel to the shallower slope may become undercut with a steeper slope and therefore less favourable than originally considered).

Q-slope has been integrated with seismic velocity (V_p) and normalized rock mass quality (Q_c) in a similar manner to the Q-system. Practical applications to date have included the use of geophysical data from seismic refractivity, seismic tomography and borehole geophysics comprising full waveform sonic (Bar and Barton 2018a, 2018b; Espin and Araujo 2022). Acoustic and optical televiewer has also been used to identify and measure the orientation of geological structures from boreholes, which can be used to estimate O-factors with respect to planned slope orientations.

Remote sensing techniques such as terrestrial laser scanning and digital, terrestrial and aerial photogrammetry for geological interpretation and rock mass characterization have become commonly used in the last decade. Bar et al (2021) demonstrated that remote sensing data can be combined with Q-slope to derive geological structure information in areas that are not safely accessible by people (i.e. estimate J_n and O-factors, the orientation component of SRF_c and assist with the estimation of J_{wice}). Where very high resolution digital terrain models are developed, RQD can also be estimated. Similar engineering judgement applies to Q-slope parameter estimation using aerial reconnaissance and digital interpretation as with in-field investigation.

Limitations of remote sensing techniques for rock mass characterization are mainly associated with limited ability to estimate geomechanical properties of intact rock (e.g. σ_c) and discontinuities (J_r and J_a). Supplementary information from boreholes or in-field investigations are still needed to estimate Q-slope effectively.

9.0 Global application and future development

Since its inception in 2015, the Q-slope method for rock slope engineering has been adopted by practitioners and researchers in over 45 countries around the world.

Figure 20 shows the countries of known application based on the current Q-slope database, a rock mass classification system usage survey undertaken by Erhardter et al (2023), and a review of available literature.

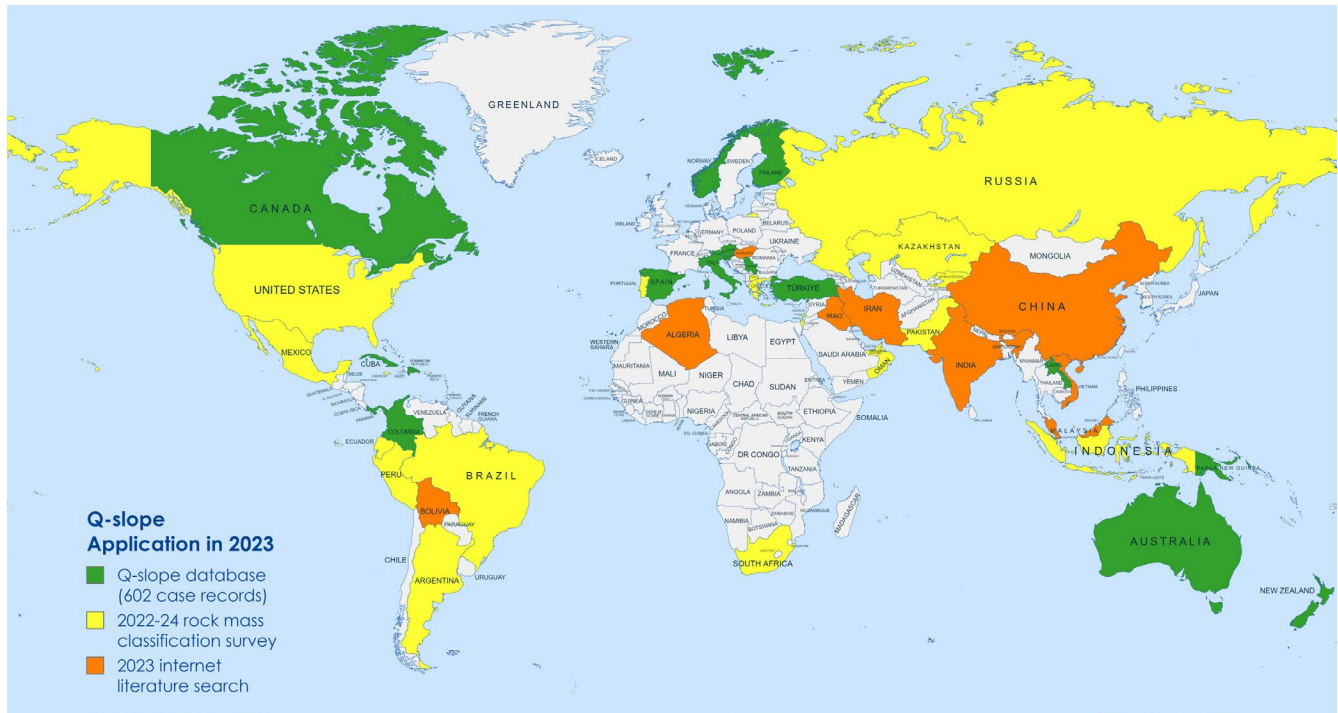


Figure 20: Q-slope application world map – 46 countries

Q-slope and Modified Q-slope (Q-slope') have been correlated with other rock mass classification systems including BQ method (Song et al 2019); continuous slope mass rating, CSMR (Siddique et al 2020); geological strength index, GSI (Narimani et al 2023); rock fall hazard rating system, RHRS (Saito de Paula et al 2019); SMR (Jordá-Bordehore et al 2018; Vinicius 2019; Azarafza et al 2022); and slope stability assessment methodology, SSAM (Bar and McQuillan 2021b).

Several authors have proposed design chart modifications based on local regions in Bolivia (Bernal et al 2023), Iran (Azarafza et al 2020) and Spain (Jordá-Bordehore 2017). The development of local correlations and further development of the Q-slope method for rock slope engineering is encouraged. However, it is essential that such research is adequately reviewed to ensure the Q-slope method is applied correctly. By way of example, Komurlu (2022) applied the Q-slope method without including the O-factor and concluded 'the method can cause misleading estimations', which is completely true if the method is not used as intended. It also remains critical that newly formed empirical relationships have a physical meaning that can be used by engineering geologists and geotechnical engineers in the field for making decisions, and not just 'lines-of-best-fit' to new or local datasets. Attempts have also been made to apply artificial intelligence to geohazard analysis using Q-slope (Mao et al. 2023). Whilst this research is novel, its practical application remains to be seen.

The writers encourage the continued development of the Q-slope and make available the database of over 600 records for future use.

10.0 Conclusion

To reliably assess rock slopes in terms of design, performance and potential for failure, as well as make adjustments to slope angles, engineers must select appropriate slope stability analysis techniques.

Q-slope provides an empirical rock engineering method to rapidly and effectively assess the stability of rock slopes in the field, and make potential adjustments to slope angles based on observed ground conditions as they become visible during and after excavation.

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 50 m, or when several stages of excavation are required), the increased time-scale should permit more rigorous analyses to be made, which are both warranted and advised. However, engineers may sometimes need to respond at slope-construction rates of many tens of metres per day, stretching to hundreds of metres in the case of some large surface mining 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. More conventional analytical and numerical methods for block and wedge stability though ‘fast’, cannot reasonably be done while accompanying new rock slope excavations on an hour-by-hour basis, while stationed in the field. This is the actual purpose of Q-slope and its relative simplicity.

Q-slope has been applied across a range of civil engineering and mining projects where it has been beneficial in:

- Reducing the number of problematic and costly bench failures during excavation.
- Reducing the requirement for ongoing maintenance as potentially problematic areas are identified and dealt with early.
- Identifying opportunities for safely steepening slope angles, reducing overburden excavation costs and environmental disturbance footprints, and yielding additional revenue in the form of extra ore recovery in mining.

Acknowledgements

The authors sincerely thank Dr Ákos Török, Dr Balázs Vásárhelyi, Dr Luis Jordá-Bordetore, Dr Alison McQuillan and Ms Yirith Morales for their material contributions. The authors are also grateful to Ms Rebecca Leckie, Mr Paul Sewell, Mr Terence Millward, Mr Simon Thomas, Mr Cameron Ryan, Ms Tessa Ormerod, Mr Sam Nicoll, Mr Chris Alickson, Dr Senaka Welideniya, Mr Sachum Wapa, Ms Marisa Torombe, Ms Katie Goguen, Ms Sarah Cain, Mr Marc Rougier, Mr Phil de Graaf and Mr Juan Carlos Cobián for their kind assistance during various slope stability field investigations between 2012 and 2023.

Declarations

The authors declare that they have no conflict of interest. No external funding was received for undertaking this study.

References

- Azarafza M, Nanekaran YA, Rajabion L, Akgün H, Rahnamarad J, Derakhshani R, Raoof A (2020) Application of the modified Q-slope classification system for sedimentary rock slope stability assessment in Iran. *Engineering Geology* 264(105349). <https://doi.org/10.1016/j.enggeo.2019.105349>
- Azarafza M, Koçkar MK, Zhu HH (2022) Correlations of SMR-Qslope Data in Stability Classification of Discontinuous Rock Slope: A Modified Relationship Considering the Iranian Data. *Geotechnical and Geological Engineering* 40: 1751-1764. <https://doi.org/10.1007/s10706-021-01991-w>
- Bar N, Barton N (2017) The Q-Slope Method for Rock Slope Engineering. *Rock Mechanics and Rock Engineering* 50: 3307-3322. <https://doi.org/10.1007/s00603-017-1305-0>
- Bar N, Barton N (2018a) Rock Slope Design using Q-slope and Geophysical Survey Data. *Periodica Polytechnica Civil Engineering* 62(4): 893–900. <https://doi.org/10.3311/PPci.12287>
- Bar N, Barton N (2018b) Q-slope: An Empirical Rock Slope Engineering Approach in Australia. *Australian Geomechanics* 53(4): 73-86.
- Bar N, Borgatti L, Donati D, Francioni M, Salvini M, Ghirotti M (2021) Classification of natural and engineered rock slopes using UAV photogrammetry for assessing stability. *IOP Conf. Ser.: Earth Environ. Sci.* 833(012046): 8p. <https://doi.org/10.1088/1755-1315/833/1/012046>

- Bar N, McQuillan A (2021a) Q-slope application to coal mine stability. IOP Conf. Ser.: Earth Environ. Sci. 833(012043): 8p. <https://doi.org/10.1088/1755-1315/833/1/012043>
- Bar N, McQuillan A (2021b) Q-slope and SSAM applied to excavated coal mine slopes. MethodsX 8(101191): 6p. <https://doi.org/10.1016/j.mex.2020.101191>
- Barton N (1987) Predicting the Behaviour of Underground Openings in Rock. 4th Manual Rocha Memorial Lecture, Lisbon: 21p.
- Barton N, Bar N (2015) Introducing the Q-slope method and its intended use within civil and mining engineering projects. In Schubert & Kluckner (ed.), Future Development of Rock Mechanics; Proc. ISRM Reg. Symp. Eurock 2015 & 64th Geomechanics Colloquium, Salzburg, 7-10 October 2015: 157-162.
- Barton N, Bar N (2019) The Q-slope Method for Rock Slope Engineering in Faulted Rocks and Fault Zones. In Fontura et al. (eds.), Rock Mechanics for Natural Resources and Infrastructure Development, Proc. 14th Int. Congress on Rock Mechanics and Rock Engineering, Foz do Iguassu, 13-18 September 2019: 3424-3432.
- Barton N, Lien R, Lunde J (1974) Engineering classification of rock masses for the design of tunnel support. Rock Mechanics 6: 189-236. <https://doi.org/10.1007/BF01239496>
- Barton N, Quadros E (2015) Anisotropy is everywhere, to see, to measure and to model. Rock Mechanics and Rock Engineering 48: 1323-1339. <https://doi.org/10.1007/s00603-014-0632-7>
- Bernal CB, Lain R, Jordá L, Cano M, Riquelme A, Tomás R (2023) Stability Assessment of Rock Slopes Using the Q-Slope Classification System: A Reliability Analysis Employing Case Studies in Ecuador. Applied Science 13(13): 18p. <https://doi.org/10.3390/app13137399>
- Bieniawski ZT (1976) Rock mass classification in rock engineering. In Bieniawski (ed.), Exploration for rock engineering; Proc. of the symp., Cape Town, Balkema: 97-106.
- Bieniawski ZT (1989) Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering, New York, Wiley: 272p.
- Carvajal HEM, Restrepo PAI, Azevedo GF (2012) Landslide risk management in Medellin, Colombia. In Lacerda, Palmeira, Netto & Ehrich. (eds.), Extreme Rainfall Induced Landslides. Brazil, Oficina de Textos: 299-323.
- Deere DU (1963) Technical description of rock cores for engineering purposes. Felsmechanik und Ingenieurgeologie 1: 16-22.
- Deere DU, Hendron AJ, Patton F, Cording EJ (1967) Design of surface and near surface excavations in rock. In Fairhurst (ed), Proc. 8th U.S. Symp. Rock Mechanics: Failure and Breakage of Rock, AIME, New York: 237-302.
- Erhart G, Hansen TF, Qi S, Bar N, Marcher T (2023) A 2023 perspective on Rock Mass Classification Systems. In Schubert & Kluckner (ed.), Proc. 15th ISRM Congress 2023 & 72nd Geomechanics Colloquium, Salzburg, 9-14 October 2023: 758-763.
- Espin JR, Araujo S (2022) Stability Assessment of a Bedding Rock Slope Using Q-slope and Seismic Tomography: A Case Study in the Ecuadorian Amazon. Periodica Polytechnica Civil Engineering 66(1): 220-227. <https://doi.org/10.3311/PPci.19005>
- Froude MJ, Petley DN (2018) Global fatal landslide occurrence from 2004 to 2016. Natural Hazards and Earth System Sciences 18(8): 2161–2181. <https://doi.org/10.5194/nhess-18-2161-2018>
- Ghirotti M, Masetti D, Massironi M, Oddone E, Sapigni M, Zampieri D, Wolter A (2013) The 1963 Vajont Landslide (Northeast Alps, Italy) Post-Conference Field Trip (October 10th, 2013). Italian Journal of Engineering Geology and Environment 6: 635-646.
- Hutchinson DJ, Diederichs MS (1996) Cable bolting in underground mines. Vancouver, Bitech: 406p.
- Jordá-Bordehore L (2017) Application of Q_{slope} to Assess the Stability of Rock Slopes in Madrid Province, Spain. Rock Mechanics and Rock Engineering 50: 1947-1957. <https://doi.org/10.1007/s00603-017-1211-5>

- Jordá-Bordehore L, Bar N, Cano M, Riquelme A, Tomás R (2018) Stability assessment of rock slopes using empirical approaches: comparison between Slope Mass Rating and Q-slope. In Proceedings of XIV Congreso Internacional de Energía y Recursos Minerales and Slope Stability 2018, 10-13 April 2018, Seville, Spain, 5p.
- Karzulovic A (2004) The importance of rock slope engineering in open pit mining business optimization. *Landslides: Evaluation and Stabilization*. London, Taylor & Francis Group: 443-453.
- Kiersch GA (1965) The Vajont Reservoir Disaster. *Mineral Information Service* 18(7): 129-138.
- Komurlu E (2022) Case studies on Q-slope method use for slope stability analyses. *Studia Geotechnica et Mechanica* 44(3): 190-197. <https://doi.org/10.2478/sgem-2022-0010>
- Laubscher DH (1977) Geomechanics classification of jointed rock masses – mining applications. In *Transactions of Institute of Mining and Metallurgy: Section A Mining Industry*, Vol.93: A1-A8.
- Laubscher DH, Jakubec J (2001) The MRMR rock mass classification for jointed rock masses. In Hustrulid & Bullock (ed.), *Underground Mining Methods: Engineering Fundamentals and International Case Studies*, New York, AIME: 474-481.
- Mandl G (2005) *Rock Joints: The Mechanical Genesis*. Berlin, Springer-Verlag: 222p.
- Mao Y, Chen L, Nanekaran YA, Azarafza M, Derakhshani R (2023) Fuzzy-Based Intelligent Model for Rapid Rock Slope Stability Analysis Using Q_{slope} . *Water* 15(16). <https://doi.org/10.3390/w15162949>
- Narimani S, Davarpanah SM, Bar N, Török A, Vászrhelyi B (2023) Geological Strength Index Relationships with the Q-system and Q-slope. *Sustainability* 15(14): 16p. <https://doi.org/10.3390/su151411233>
- Palmstrøm A (2005) Measurements of and correlations between block size and rock quality designation (RQD). *Tunnelling & Underground Space Technology* 20(4): 362-377. <https://doi.org/10.1016/j.tust.2005.01.005>
- Romana M (1985) New adjustment ratings for application of Bieniawski classification to slopes. In *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. In *Proc. 8th ISRM congress on Rock Mechanics* 3, Tokyo, 25-29 September 1995: 1085-1092.
- Saito de Paula M, Maion AV, Campanha GAC, Castilho LMN, Cunha MA (2019). Q-Slope and RHRS for the evaluation of highway rock slopes – Serra do Mar, Brazil. In Fontura et al. (eds.), *Rock Mechanics for Natural Resources and Infrastructure Development*, *Proc. 14th Int. Congress on Rock Mechanics & Rock Engineering*, Foz do Iguassu, 13-18 September 2019: 8p.
- Siddique T, Pradhan SP, Vishal V, Singh TN (2020) Applicability of Q-slope Method in the Himalayan Road Cut Rock Slopes and Its Comparison with CSMR. *Rock Mechanics and Rock Engineering* 53: 4509-4522. <https://doi.org/10.1007/s00603-020-02176-2>
- Song Y, Xue H, Meng X (2019) Evaluation method of slope stability based on the Q_{slope} system and BQ method. *Bulletin of Engineering Geology and the Environment* 78: 4865-4873. <https://doi.org/10.1007/s10064-019-01459-5>
- Tomás R, Cuenca A, Cano M, García-Barba J. (2012) A graphical approach for slope mass rating (SMR). *Engineering Geology* 124: 67-76. <https://doi.org/10.1016/j.enggeo.2011.10.004>
- Tomás R, Delgado J, Serón JB (2007) Modification of slope mass rating (SMR) by continuous functions. *International Journal of Rock Mechanics & Mining Sciences* 44(7): 1062-1069. <https://doi.org/10.1016/j.ijrmms.2007.02.004>
- Vinicius A (2019) Proposta de correlação entre os índices SMR e Q-slope, Dissertation. Universidade de São Paulo, Brasil: 123p.
- Ward JT (2015) Bingham Canyon Landslide: Analysis and Mitigation. Dissertation. University of Nevada, USA: 150p.
- Wyllie DC (2017) *Rock Slope Engineering: Civil Applications*. 5th. Boca Raton, CRC Press: 620p.