The Q-Slope Method for Rock Slope Engineering

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ORIGINAL PAPER

The *Q*-Slope Method for Rock Slope Engineering

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Abstract *Q*-slope is an empirical rock slope engineering method for assessing the stability of excavated rock slopes in the field. Intended for use in reinforcement-free road or railway cuttings or in opencast mines, *Q*-slope allows geotechnical engineers to make potential adjustments to slope angles as rock mass conditions become apparent during construction. Through case studies across Asia, Australia, Central America, and Europe, a simple correlation between *Q*-slope and long-term stable slopes was established. *Q*-slope 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 tunnels under construction for the last 40 years. The Q' parameters (RQD, J_n, J_a, and J_r) remain unchanged in *Q*-slope. However, a new method for applying J_r/J_a ratios to both sides of potential wedges is used, with relative orientation weightings for each side. The term J_w, which is now termed J_{wice}, takes into account long-term exposure to various climatic and environmental conditions such as intense erosive rainfall and ice-wedging effects. Slope-relevant SRF categories for slope surface conditions, stress-strength ratios, and major discontinuities such as faults, weakness zones, or joint swarms have also been incorporated. This paper discusses the applicability of the *Q*-slope method to slopes ranging from less than 5 m to more than 250 m in height in both civil and mining engineering projects.

Keywords *Q*-slope · Rock slope engineering · Slope stability · Rock mass classification · Empirical method

List of symbols

RQD	Rock quality designation
J_n	Joint sets number
J_r	Joint roughness number
J_a	Joint alteration number
J_{wice}	Environmental and geological condition number
SRF_{slope}	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 engineering and mining 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 modeling. Excavation is usually too fast for this. The same limitation usually applies to tunneling, despite numerical modeler's wishes to the contrary. However, rock caverns of larger span are sufficiently 'stationary' for thorough and more necessary analysis, and the same applies to higher rock slopes. The purpose of *Q*-slope is to allow engineering

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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). 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, if not hundreds of thousands of kilometers of such rock cuttings, without support, are made each year. A significant proportion may fail during construction, and shallower slope angles are then chosen.

Application of *Q*-slope can help to reduce maintenance (and bench-width needs) due to all the potential failures. Such are frequently seen when initially constant slope angles are cut through varied structural domains. A series of 'interesting' but troublesome local failures is often the result. Quite often these have been the result of adverse plane failures, wedge failures, or more rarely local toppling. Figure 1 illustrates the very basic classes of potential behavior. These may be accentuated when one of the illustrated joint sets is dominant. Anisotropic behavior is typical, as emphasized by Barton and Quadros (2015). Several combinations of slope angles and jointing in Fig. 1 are expected to be stable (i.e., a2, b3, c3, which are >45°; and d1, d2, d3, d4, which are equal to 45° in the idealized examples illustrated).

Joint or discontinuity shear strength obviously has a significant role in rock slope stability. On a more sophisticated level than in Fig. 1, it may often be a coupled problem where shear-deformation can improve drainage temporarily due to dilation (Fig. 2), until fractures are clogged with run-off fines in future storms. Therefore, the environment in which the slopes are constructed is also critical. The sheared and dilated tension-fracture replicas on the right are from Barton (1973), and the three potential joint behavior modes involving mobilized shear strength, dilation, and increase of permeability are based on coupled M–H (mechanical–hydraulic) Barton–Bandis joint modeling, as described in Barton et al. (1985).

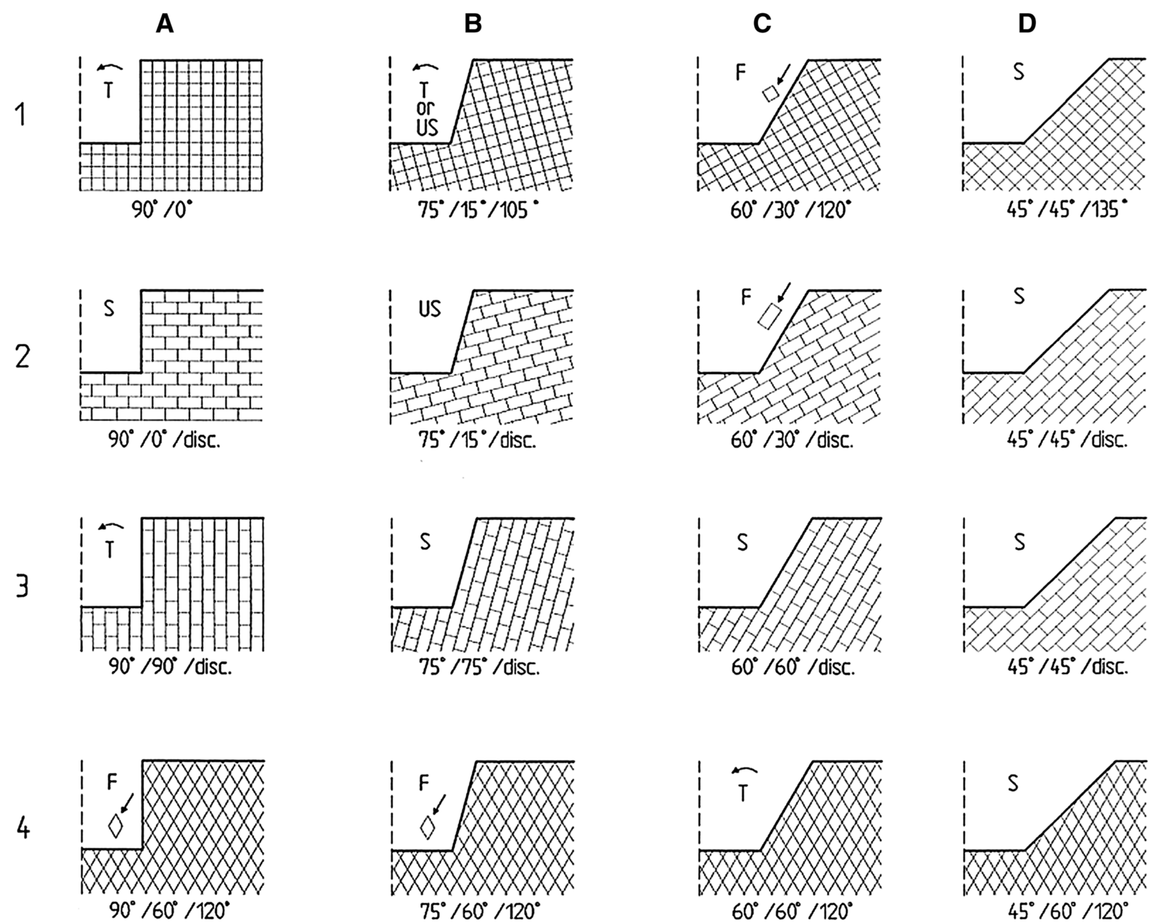

Fig. 1 Common sense solutions to dominant joint orientation problems (*S* stable, *F* sliding failure, *T* toppling, *US* unstable)

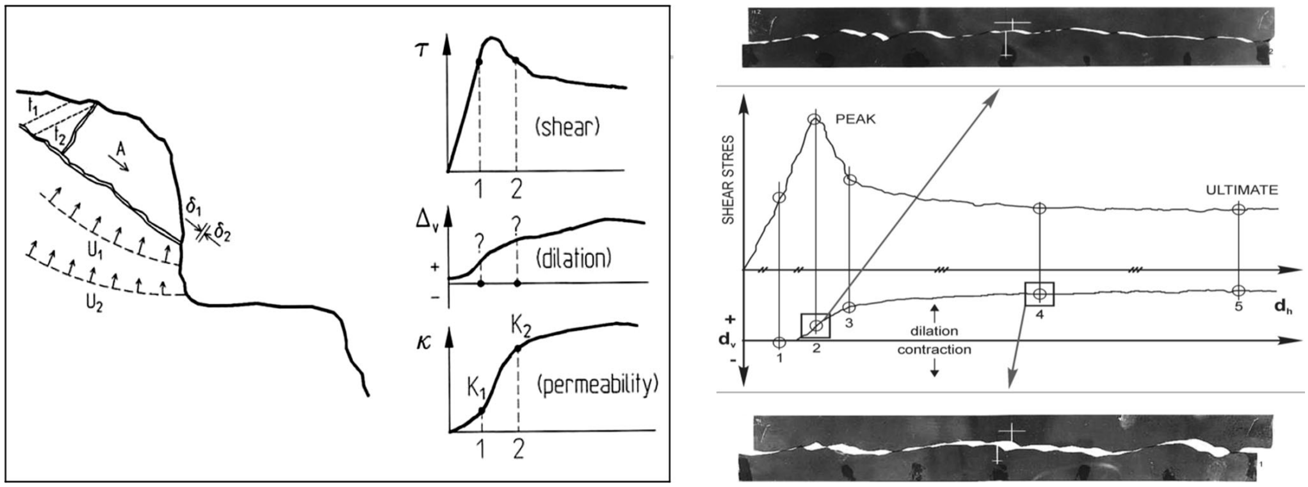

Fig. 2 Example of shear-deformation-dilation effects on permeability (until blocked by run-off fines)

1.1 Empirical Rock Engineering Methods

Several empirical methods for assisting rock engineering design have been developed in the last 50 years and are used for a variety of applications by rock and mining engineers.

In tunneling, 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, 1989),
- MRMR: mining rock mass rating (Laubscher 1977; Laubscher and Jakubec 2001).

For the case of rock slope stability, empirical methods are less frequently used and either simple kinematics, numerical modeling or 'no modeling' may be chosen (i.e., slope angles selected by equipment operators rather than engineers). The following empirical methods were developed to predict the support, reinforcement, and performance of excavated slopes:

- SMR: slope mass rating (Romana 1985, 1995),
- Global slope performance index (Sullivan 2013).

None of the aforementioned empirical rock engineering methods provide guidance in relation to appropriate, long-term stable slope angles in which rock reinforcement and support are*deliberately absent*. Such slopes actually dominate the demand by a huge margin.

1.2 Original Q-System

The Q-system for characterizing rock exposures, drill-core, and tunnels under construction was developed from rock tunneling and rock cavern related case records and has been used by engineers across the world for over 40 years (Barton et al. 1974; Barton and Grimstad 2014).

Single-shell $B + S$(fr) tunnel support and reinforcement design assistance, and open stope design, utilizing Q' (the first four parameters) have been the principal focus of applications in civil and mining engineering. Correlations of Q_c (Q normalized with UCS/100) with stress-dependent *P*-wave velocities and depth-dependent deformation moduli have also proved useful in site characterization and as input to numerical modeling. These approximate correlations remain with the new *Q*-slope value, which may also vary over six orders of magnitude, from approx. 0.001 to 1000. This large numerical range is an important reflection of the large variation of parameters such as deformation moduli and shear strength.

2 *Q*-Slope Development

Q-slope utilizes 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 $(J_r/J_a)_1 \times 0.7$ for set J1 and $(J_r/J_a)_2 \times 0.9$ for set J2, 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.

Q-slope was conceived a decade ago, firstly at a hydropower dam-access road in the Dominican Republic, requiring a precipitous day's journey on a mule, adjacent to very steep planned slopes to reach a dam-site 20 km distant. Despite the steepness of the river-cut slopes, varying from 40° to 60°, the owner did not accept the (presumed) extra cost of suggested rock reinforcement for the mostly steeper and extremely frequent cut slopes.

Q-slope in a more developed form was subsequently applied at a new motorway to be excavated through hilly, forested tropical terrain in Panama. How wide the contractor should clear the forest to make way for twin, appropriately shallow-angle cuttings in relic-jointed saprolite, and successively steeper-angle cuttings in weathered, then fresher jointed rock masses was answered using a combination of seismic refraction and about 1 km of core logging. Planned slope angles were subsequently confirmed with *Q*-slope logging during their construction.

A visual demonstration of the ultra-simple initial objective of *Q*-slope is shown in Fig. 3. The differently inclined slopes of a high cutting for the new Panama Canal extension 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°, 60°–65°, and 80°–85° for respective *Q*-slope values of approximately 0.1, 1.0, and 10.

In 2015, sufficient supporting data from several opencast mine bench slopes and road cuts in Australia, Papua New Guinea, and Laos, and from the motorway and other road cuts in Panama, permitted a correlation between *Q*-slope and long-term stable, reinforcement-free slope angles (Barton and Bar 2015). At the time, limited data were available for larger slopes requiring several stages of excavation such as inter-ramp and overall slopes in opencast mines. As such, the correlation was initially limited to slopes less than 30 m in height.

Additional supporting data and case studies of steep hard rock slopes in Australia and Slovenia demonstrated the practical applications of *Q*-slope (Bar and Barton 2016). It illustrated *Q*-slope's potential to allow engineers to respond to slope-forming rates of many tens of meters per day, stretching to hundreds of meters in the case of some opencast mining operations. Figure 4 illustrates improving geologic structure orientations allowing for steeper bench face angles with depth.

Highly weathered and saprolitic rock slopes were examined across several projects in Far North Queensland (Australia) and demonstrated *Q*-slopes' applicability to softer, weaker rocks (Bar et al. 2016). Further examination of slopes in different regions, geological settings, and additional case studies and supporting data for larger slopes now enables the use of *Q*-slope for these larger slopes. Figure 5 presents the stability chart for all slope heights (note: all case studies

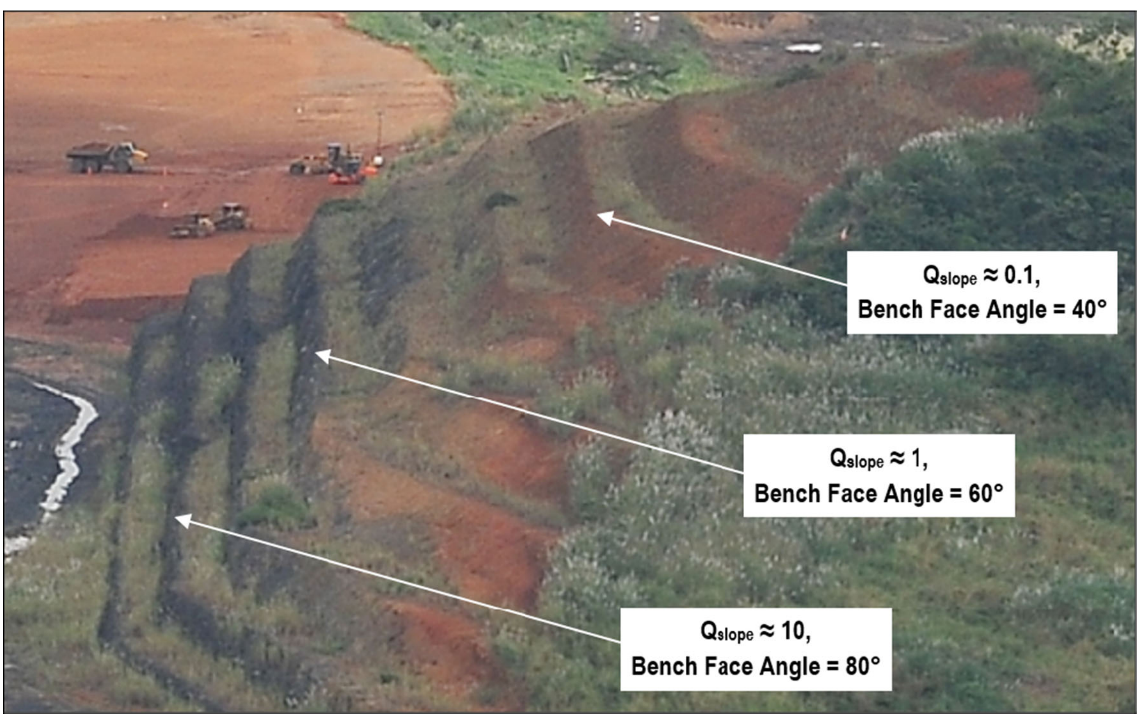

Fig. 3 Convenient example of differentiated saprolite, weathered rock, and fresher rock mass qualities help to explain the appropriately steepened, unreinforced permanent slopes (courtesy of Panama Canal Authority, Barton and Bar 2015)

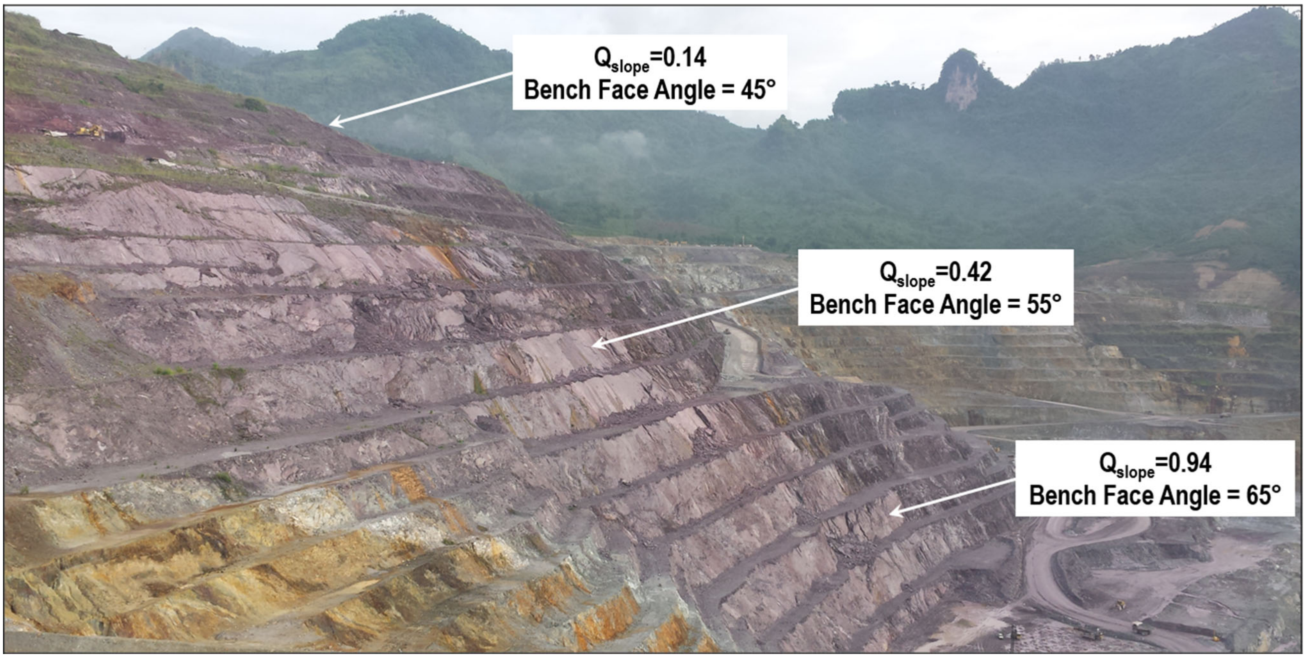

Fig. 4 Changing geologic structure orientations (steepening of bedding planes downslope) help to explain the appropriately steepened bench face angles in a large opencast mine setting, Laos. Improved rock mass quality at greater depth is also expected to allow steeper slopes at greater depth

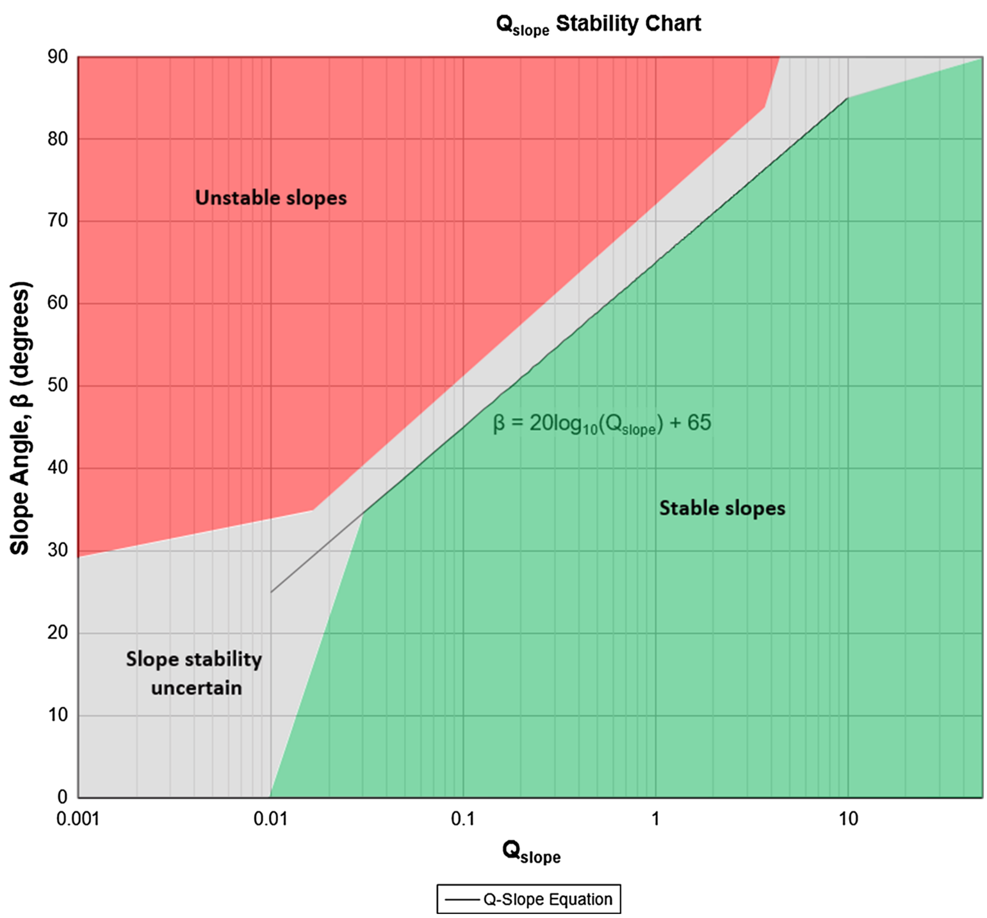

Fig. 5 Q-slope stability chart

involving slopes larger than about 50 m in height were of near-uniform rock mass quality with no changes in lithology—these slopes are quite rare in nature).

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 50 m in height (i.e., which require several stages of excavation), more rigorous analysis is both warranted and advised. Figure 6 illustrates the *Q*-slope dataset for different slope heights and slope angles. It is based on over 400 individual case studies from:

- Australia (New South Wales, Queensland, and Western Australia).
- Asia (Laos, Papua New Guinea, and Turkey),
- Central America (Dominican Republic and Panama).
- Europe (Serbia, Slovenia, and Spain).

Rock types in the case studies:

- Igneous rocks (basalt, diorite, dolerite, granite, granodiorite, monzonite, monzodiorite, porphyry, agglomerate, greywacke, and tuff).
- Sedimentary rocks (sandstone, siltstone, limestone, mudstone, conglomerate, breccias, and banded iron formation).
- Metamorphic rocks (shale, schist, skarns, slates, gneiss, phyllite, marble, metasandstone, and quartzite).
- Saprolites of some of the aforementioned rock types have also been examined.

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

- 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.

3 The *Q*-Slope Method

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 Q-system (Barton et al.

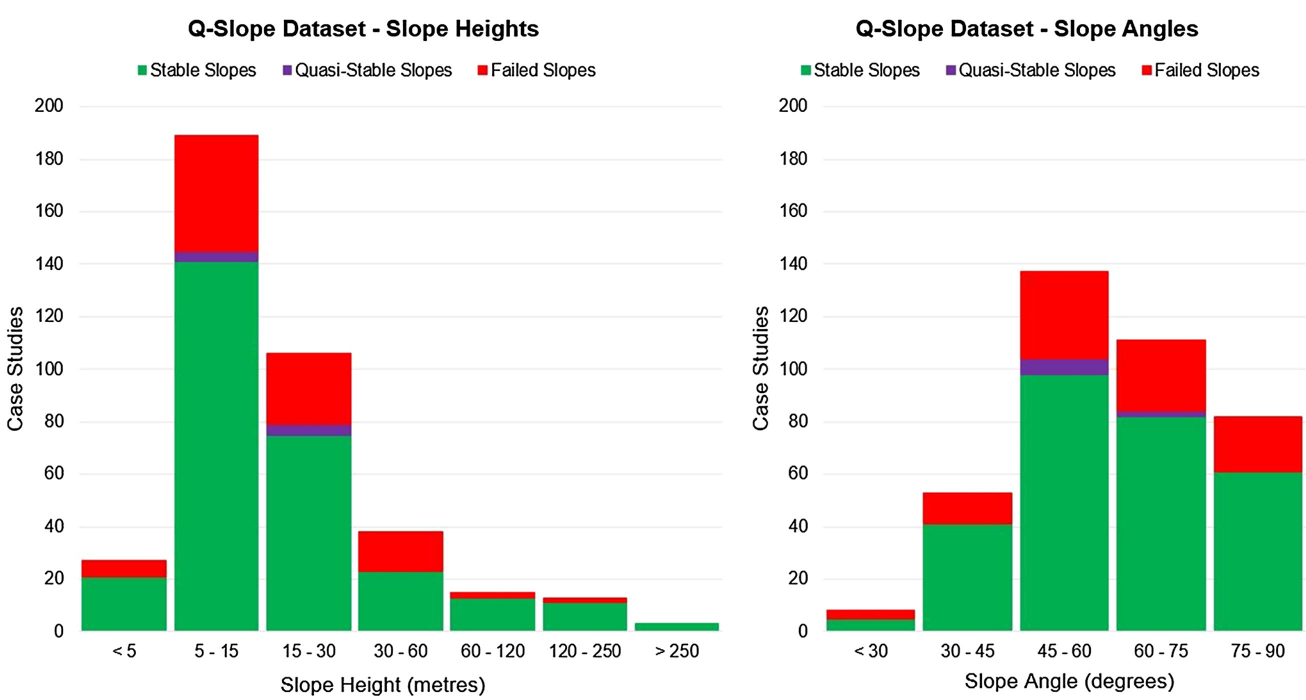

Fig. 6 *Q*-slope dataset (case studies)—slope heights (*left*) and slope angles (*right*)

1974). For Q-system users, the formula for estimating *Q*-slope is mostly familiar:

$$Q_{\text{slope}} = \frac{\text{RQD}}{J_n} \times \left(\frac{J_r}{J_a}\right)_0 \times \frac{J_{\text{wice}}}{\text{SRF}_{\text{slope}}} \qquad (1)$$

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 favorable (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: ($J_{\text{wice}}/\text{SRF}_{\text{slope}}$).

Shear resistance, τ, is approximated using:

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

3.1 The First Four Parameters (RQD, J_n, J_r, and J_a)

The *Q*-slope ratings for rock quality designation, RQD (Deere 1963; Deere et al. 1967), joint set number (J_n), joint roughness number (J_r), and joint alteration number (J_a) remain unchanged from the Q-system (Barton et al. 1974; Barton and Bar 2015). Tables 1, 2, 3 and 4 describe the ratings for RQD, J_n, J_r, and J_a, respectively.

3.2 Discontinuity Orientation Factor: O-Factor

The discontinuity orientation factor (O-factor) described in Table 5 provides orientation adjustments for discontinuities in rock slopes (Barton and Bar 2015). Figure 7 provides photographic examples.

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

3.3 Environmental and Geological Conditions: J_{wice}

The environmental and geological condition number, J_{wice}, is more sophisticated than J_w of the original Q-system

Table 1 Rock quality designation

Rock quality designation description		RQD (%)[a]
A	Very poor	0–25
B	Poor	25–50
C	Fair	50–75
D	Good	75–90
E	Excellent	90–100

[a] 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 2 Joint set number

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, earthlike	20

since slopes are outside and exposed to the elements for a very long time (Barton and Bar 2015).

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.

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 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.

3.4 Strength Reduction Factor: $\text{SRF}_{\text{slope}}$

The strength reduction factor $\text{SRF}_{\text{slope}}$ is obtained by using the most adverse i.e., maximum of SRF_a, SRF_b, and SRF_c described in the subsequent tables.

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 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.

Table 3 Joint roughness number

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

Descriptions refer to small-scale features and intermediate scale features, in that order

Add 1.0 if mean spacing of the relevant joint set is greater than 3 m

$J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength

J_r and J_a classification are applied to the discontinuity set or sets that are least favorable 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 4 Joint alteration number

Joint alteration number description		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	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
(b) Rock-wall contact after some shearing (thin clay fillings, probable thickness ≈ 1–5 mm)		
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 percent of swelling clay-size particles and access to water	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, 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)	10, 13, or 13–20

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. 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).

3.5 Relationship Between *Q*-Slope and Slope Angles

Barton and Bar (2015) derived a simple formula for the steepest slope angle (β) not requiring reinforcement or support for slope heights less than 30 m. This formula is now extended to all slope heights:

$$\beta = 20 \log_{10} Q_{\text{slope}} + 65^\circ \quad (3)$$

Equation 3 matches the central data for stable slope angles greater than 35° and less than 85° as shown in

Table 5 Discontinuity orientation factor—O-factor

O-factor description	Set A	Set B
Very favorably oriented	2.0	1.5
Quite favorable	1.0	1.0
Unfavorable	0.75	0.9
Very unfavorable	0.50	0.8
Causing failure if unsupported	0.25	0.5

Figs. 5 and 8. It has a probability of failure of 1%. 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°.

Figure 8 illustrates the available *Q*-slope data derived from the back-analysis of motorway, railway and road cuts, opencast mine benches, and natural slopes:

- Triangles indicate stable slopes. No visual signs of instability observed at least several weeks, months or years post-excavation.

Very favorably to favorably oriented joints forming cubical blocks: Norway.

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

Unfavorable foliation in phyllite. Steep joints striking into bench slopes are quite favorable: Western Australia.

Very unfavorable bedding planes in siltstone. Vertical joints and joints dipping into slope favorable. Mt. Isa.

Extremely unfavorable joints day-lighting into excavation and causing failure when unsupported. Papua New Guinea.

Extremely unfavorable joints day-lighting and causing failure when unsupported. Papua New Guinea.

Fig. 7 Examples of the need for O-factor (discontinuity orientation factor) application

Table 6 Environmental and geological condition number

J_{wice}^{a}	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

[a] When drainage measures are installed, apply $J_{wice} \times 1.5$, when slope reinforcement measures are installed, apply $J_{wice} \times 1.3$, and when drainage and reinforcement are installed, apply both factors $J_{wice} \times 1.5 \times 1.3$

Table 7 SRF_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 and 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 downslope	20

Table 8 SRF_b stress and strength

	Description	σ_c/σ_1^a	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

[a] σ_c = unconfined compressive strength (UCS), σ_1 = maximum principal stress

- 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-analyzed using known pre-failure geometries and ground conditions.

Figure 8 also includes larger slopes such as inter-ramp slopes in opencast mines where geological units and rock mass quality are uniform or very close to uniform across the height of the slope.

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 6 months and over 50 years. However, users may, if they wish, additionally apply a factor of safety on the steepest slope angle (β) not requiring reinforcement or support.

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 Fig. 9. The authors acknowledge that these iso-potential lines are one possible interpretation of the data and that other similar interpretations are also possible. If certain degrees of failure are accepted, such as percentages of individual benches in opencast mines, then the following equations can be derived:

Table 9 SRF_c major discontinuity

SRF_c		Favorable	Unfavorable	Very unfavorable	Causing failure if unsupported
L	Major discontinuity with little or no clay	1	2	4	8
M	Major discontinuity with $RQD_{100} = 0^a$ due to clay and crushed rock	2	4	8	16
N	Major discontinuity with $RQD_{300} = 0^b$ due to clay and crushed rock	4	8	12	24

[a] RQD_{100} = 1 m perpendicular sample of discontinuity, [b] RQD_{300} = 3 m perpendicular sample of discontinuity

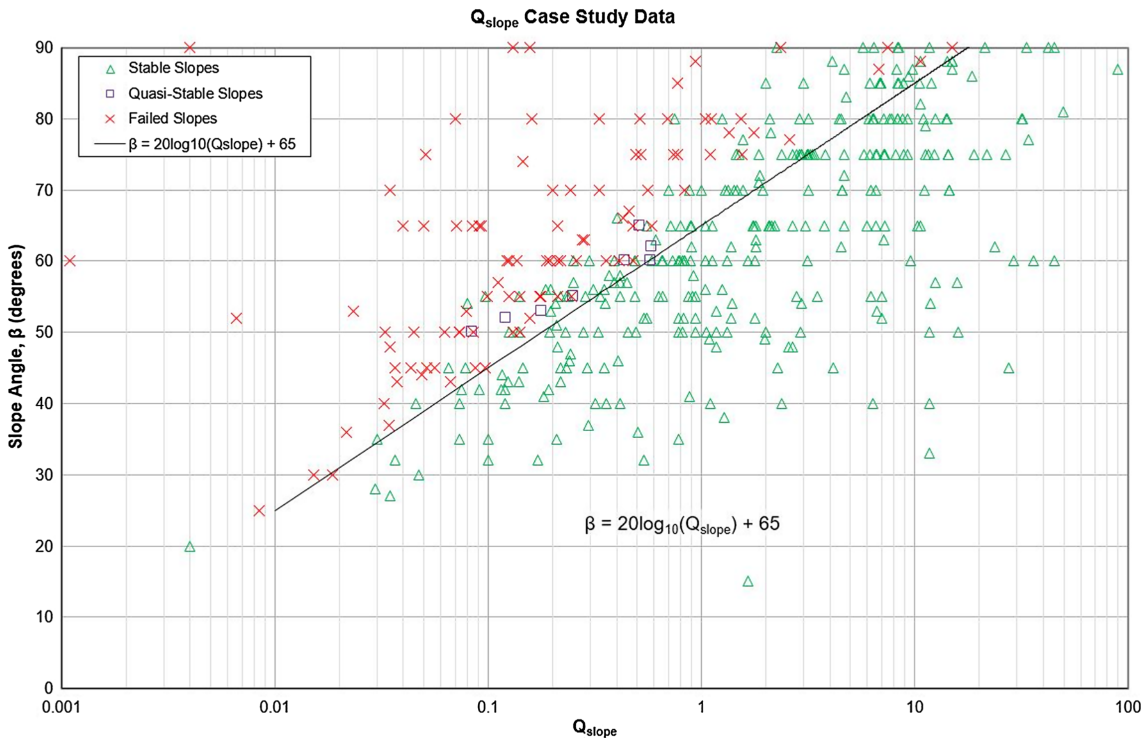

Fig. 8 Q-slope data—412 case studies

$$\text{PoF} = 1\%: \quad \beta = 20\ \log_{10} Q_{\text{slope}} + 65^{\circ} \tag{4}$$

$$\text{PoF} = 15\%: \quad \beta = 20\ \log_{10} Q_{\text{slope}} + 67.5^{\circ} \tag{5}$$

$$\text{PoF} = 30\%: \quad \beta = 20\ \log_{10} Q_{\text{slope}} + 70.5^{\circ} \tag{6}$$

$$\text{PoF} = 50\%: \quad \beta = 20\ \log_{10} Q_{\text{slope}} + 73.5^{\circ} \tag{7}$$

4 Q-Slope Examples

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 (Bar and Barton 2016).

4.1 Planar Sliding

A 12 m high slope was excavated with a slope angle of 55° in very strong quartzite ($\sigma_c > 150$ MPa). The outward dipping bedding (~50°) caused planar failure a few hours after excavation as illustrated in Fig. 10a. The following Q-slope ratings were assigned during back-analysis:

- RQD = 90–100%
- $J_n = 12$
- $J_r = 1$, $J_a = 3$, O-factor = 0.25 (Set A only)
- $J_{wice} = 0.8$ (desert environment, competent rock but unstable structure).
- $\text{SRF}_a = 2.5$, $\text{SRF}_b = 1$, $\text{SRF}_c = 1$.

Based on the assigned ratings, Q-slope and β were estimated as follows:

$$Q_{\text{slope}} = \frac{95}{12} \times \left(\frac{1}{3} \times 0.25\right) \times \frac{0.8}{2.5} = 0.211.$$

$$\beta = 20 \log_{10}(0.211) + 65^{\circ} = 51^{\circ}.$$

Q-slope suggests an angle of 51° would have resulted in a stable slope. Given that the slope failed along a bedding plane dipping at approximately 50°, the back-analysis is considered sensible.

4.2 Wedge Sliding

A 30 m high slope was excavated at an angle of 65° and failed shortly after. The wedge failure occurred in weak, moderately weathered sandstone ($\sigma_c = 35$ MPa) as illustrated in Fig. 10b. The following Q-slope ratings were assigned during the back-analysis:

- RQD = 40–50%
- $J_n = 9$
- Set A: $J_r = 1$, $J_a = 4$, O-factor = 0.5
- Set B: $J_r = 3$, $J_a = 4$, O-factor = 0.9

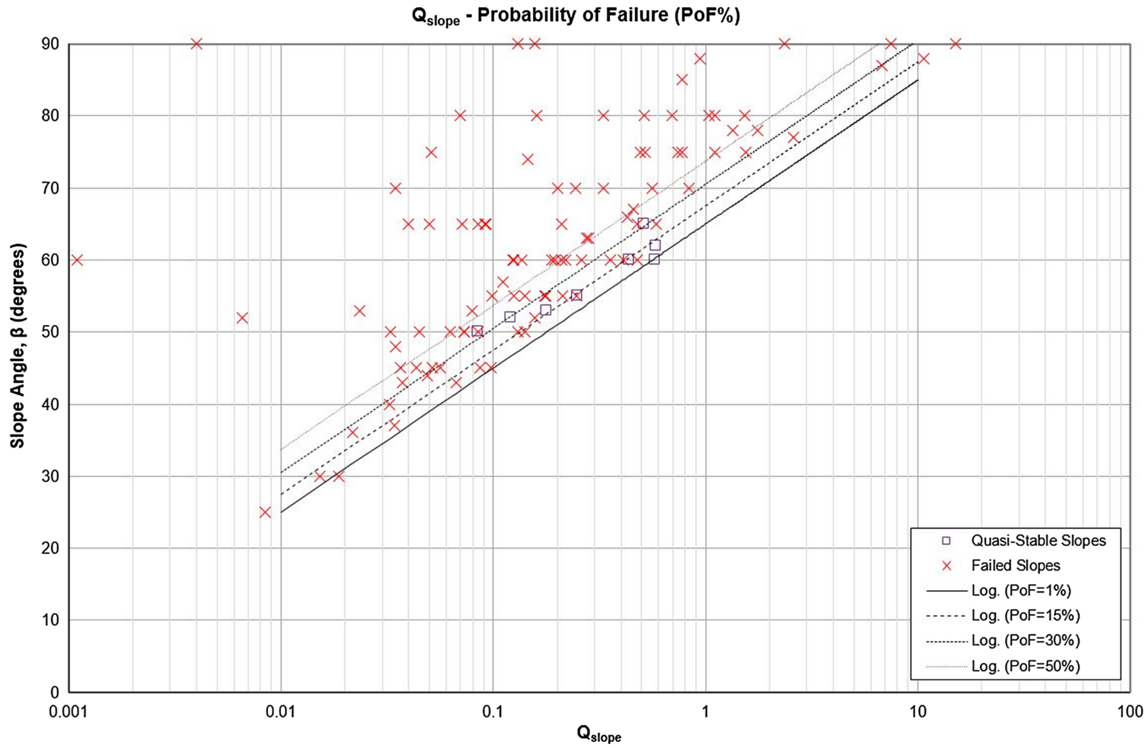

Fig. 9 *Q*-slope—probability of failure based on unwanted events (failed or quasi-stable slopes are both undesirable)

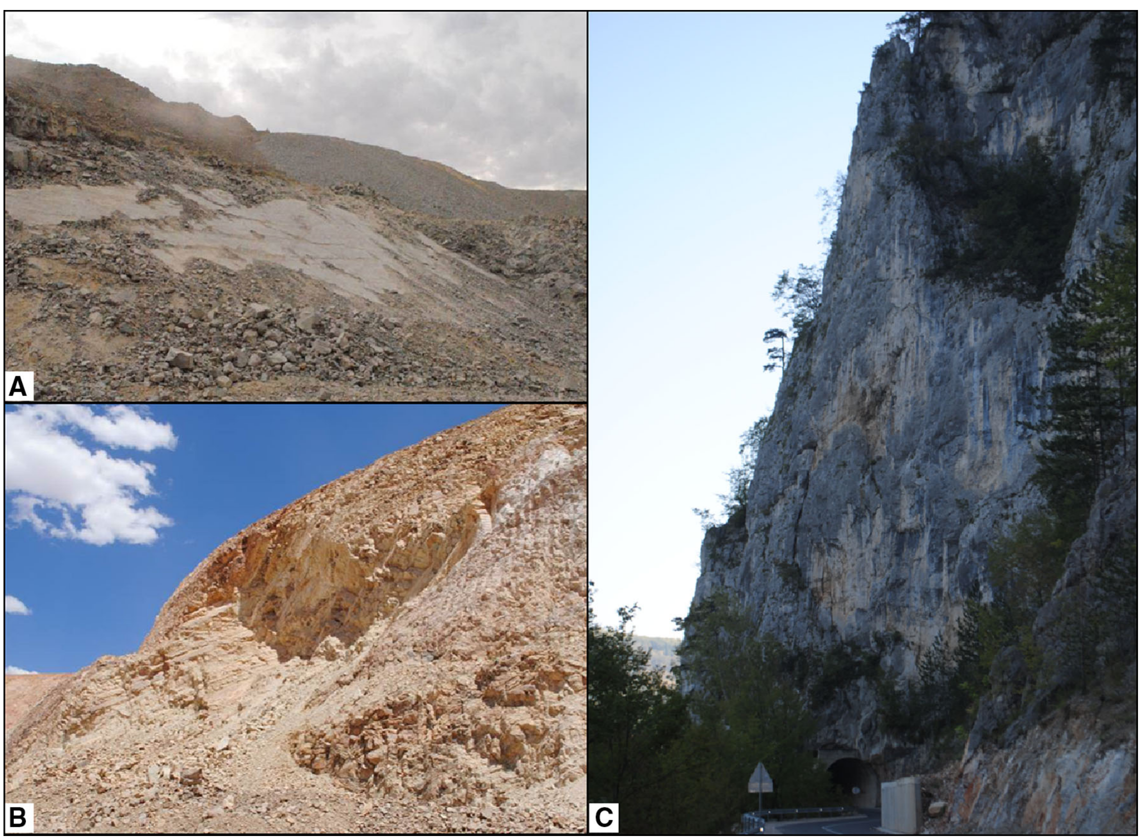

Fig. 10 **a** Plane failure in quartzite bench (slope height = 12 m) from a large opencast mine in Western Australia. **b** Wedge failure in sandstone (slope height = 30 m). **c** Stable limestone slope in Serbia (slope height = 70 m)

- Set C: Release plane or tension crack that did not contribute to the overall shear strength of the wedge.
- $J_{wice} = 1$ (desert environment, competent rock, and generally stable structure where Set B has limited continuity).
- $SRF_a = 2.5$ (slight loosening due to surface location), $SRF_b = 2.5$, $SRF_c = N/A$.

Based on the assigned ratings, Q-slope and β were estimated as follows:

$$Q_{slope} = \frac{55}{9} \times \left[\left(\frac{1}{4} \times 0.5\right)\left(\frac{3}{4} \times 0.9\right)\right] \times \frac{1}{2.5} = 0.206.$$

$$\beta = 20\log_{10}(0.206) + 65^\circ = 51^\circ.$$

Q-slope suggests an angle of 51° would have resulted in a stable slope (i.e., approximately 15° shallower than excavated and consistent with kinematic analysis).

4.3 Steep Stable Slope

A 70 m high slope has been stable for at least 70 years following road construction at an angle of approximately 70° in strong limestone ($\sigma_c = 70$ MPa) as illustrated in Fig. 10c. The following Q-slope ratings were assigned during back-analysis:

- RQD = 90–100% (relatively uniform across entire slope measured at base and crest).
- $J_n = 9$.
- $J_r = 3$, $J_a = 3$, O-factor = 1 (Set A only).
- $J_{wice} = 0.9$ (alpine environment—ice wedging and freeze–thaw in winter, competent rock and stable structure).
- $SRF_a = N/A$, $SRF_b = 3$, $SRF_c = 1$ (minor shears, favorably oriented).

Based on the assigned ratings, Q-slope and β were estimated as follows:

$$Q_{slope} = \frac{95}{9} \times \left(\frac{3}{3} \times 1.0\right) \times \frac{0.9}{3} = 3.167.$$

$$\beta = 20\log_{10}(3.167) + 65^\circ = 75^\circ.$$

Q-slope suggests angles up to 75° would be stable.

4.4 Weak Rocks and Increasing Slope Heights

Residential subdivision earthworks cuttings in Far North Queensland often comprise weak weathered rocks and saprolites since excavations are usually relatively shallow (Bar et al. 2016). Figure 11a is an example of such a slope excavated 5 m high at an angle of 35° without any form of geotechnical investigation or design. The slope comprised topsoil at the surface with the remainder being low strength saprolitic phyllite ($\sigma_c = 1$–2 MPa). Relic geologic structure was clearly visible, even after excavation, and was quite favorably oriented. The following Q-slope ratings were assigned during back-analysis:

- RQD = 10% (minimum value)
- $J_n = 6$
- $J_r = 2$, $J_a = 1$, O-factor = 1 (Set A only)
- $J_{wice} = 0.6$ (wet environment—incompetent rock and stable structure).
- $SRF_a = N/A$, $SRF_b = 7$, $SRF_c = N/A$ (slope height of 5 m results in $SRF_b = 7$).

Based on the assigned ratings, Q-slope and β were estimated as follows:

$$Q_{slope} = \frac{10}{6} \times \left(\frac{2}{3} \times 1.0\right) \times \frac{0.6}{7} = 0.095.$$

$$\beta = 20\log_{10}(0.095) + 65^\circ = 45^\circ.$$

Q-slope suggests angles up to 45° would be stable for a 5 m high slope. In this instance, the slope was steepened to 45° while retaining the crest position. Further proposed earthworks required the slope height to be increased to approximately 20 m. Since, the slope height has changed, Q-slope ratings as before were assigned, assuming ground conditions will remain similar as excavation depths increased:

- $SRF_a = N/A$, $SRF_b = 16$, $SRF_c = N/A$ (slope height of 20 m results in $SRF_b = 16$).

Based on the assigned ratings, Q-slope and β were estimated as follows:

$$Q_{slope} = \frac{10}{6} \times \left(\frac{2}{3} \times 1.0\right) \times \frac{0.6}{16} = 0.042.$$

$$\beta = 20\log_{10}(0.042) + 65^\circ = 37^\circ.$$

Q-slope suggested the overall slope of 20 m in height could have a maximum angle of 37°. This was supported by limit equilibrium analysis and a benched slope with an overall angle of 35° was constructed as illustrated in Fig. 11b.

5 Discussion

Engineering judgment 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. Some points to consider:

- When selecting input parameters for a slope face, local variability is almost certain. Adopting ranges of input parameters, where appropriate, is strongly

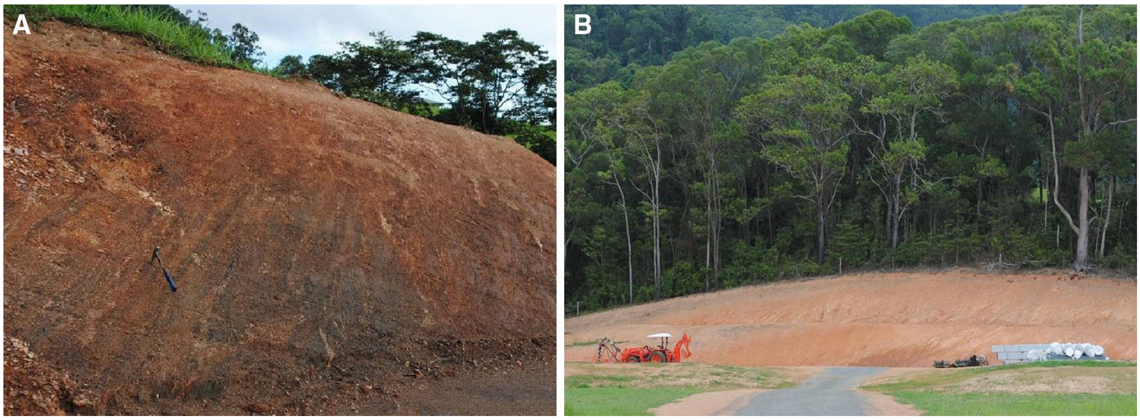

Fig. 11 **a** Preliminary saprolitic phyllite slope (height = 5 m) in a residential subdivision in Far North Queensland, Australia. **b** Final benched slope (height = 15 m)

recommended (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, physically measuring RQD using a rod or tape as described by Hutchinson and Diederichs (1996) is favored over visually 'estimating' or 'guessing.' This provides a similar estimation to core-based RQD.
- The use of strength reduction factors becomes ever more critical in weak materials and as slope heights increase. In both these instances, the stress and strength factor (SRF_b) has a tendency to dominate. Appropriately estimating in situ stress in the slope is vitally important.
- 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.
- As slope heights increase and slope angles are adjusted appropriately, even though the orientation of a geological structure does not physically change, it may become less and less favorable. For example, as slope height is increased, the stress/strength factor (SRF_b) may reduce Q-slope and subsequently, the slope angle (β). This reduction in slope angle may make a geological structure that was initially considered to be unfavorable, now very unfavorable (e.g., a joint set sub-parallel to the steeper slope may become undercut with a shallower slope and therefore less favorable than originally considered).
- A more generally applicable discussion of the effect of applying Q-slope to increasingly high slopes should include the advisory that the stress/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 favorable or unfavorable 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 well 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.
- On occasion, entirely unsupported cut rock slopes appear to defy 'common sense' due to the degree by which they exceed normal expectations of safety. Two cases in this category are illustrated in Fig. 12. The extreme steepness is of course possible principally because of hard rock close to the surface and conveniently steep bedding planes and cross-joints. Nevertheless, 'normal' civil engineering practice would have suggested the fixing of mesh while forming the slopes. Contrary arguments could be defended that loose blocks are best 'flushed down' by heavy rains, when access should be limited.
- Block-clearing by the occupants of successive vehicles, both into and out of a hydropower site in Peru, where 200 km of hazardous roads separated opposite ends of a 5 km-long headrace tunnel, and where several drivers were hospitalized, is fortunately the exception as regards defensible or indefensible access-road rock-slope safety. An important advisory to those cutting wider roads in steep valleys (to aid heavy-equipment transport) is that new slope instabilities are likely to be caused which do not favor local communities, who have been promised 'better roads.' Application of Q-slope could potentially benefit such situations.

Fig. 12 Railway bridge-site access road in Indian Kashmir. An illustration of slope limits pushed by 'favorable-in-the-circumstances' (hard rock) bedding-plane and joint-dip angles. Safety of access during heavy rain is frankly doubted. Slope formed by downward excavation from valley-side footpath 20 to 70 m above this final access road. *Note* scale of tractor in right-side of lower photograph

Our experiences have 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 bench failures during construction.
- Reducing the requirement for ongoing maintenance 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.

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 meters per day, stretching to hundreds of meters in the case of some large opencast 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.

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