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Norwegian Group for Rock Mechanics
ISBN: 978-82-8208-072-9
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ISRM International Symposium
Eurock 2020 – Hard Rock Engineering
Trondheim, Norway, 14-19 June

Q-Slope addressing ice wedging and freeze-thaw effects in Arctic and Alpine environments

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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 open cast mines, Q-slope allows rock engineers to make potential adjustments to slope angles as rock mass conditions become apparent during excavation.

Q-slope was developed over the last decade by modifying some of the Q-parameters so that rock-cuttings and bench faces could be characterized. Drill core and seismic velocity can still be used as supportive input. The original Q-value has traditionally been used for estimating single-shell support and reinforcement needs in tunnels, caverns and mine roadways and access ramps. A simple correlation between Q-slope and long-term stable slopes was suggested five years ago. Through over 500 additional case studies from Asia, Australia, the Americas and in Europe, Q-slope has been confirmed as giving stable, maintenance-free rock-cuttings and bench-face slope angles of for instance 40-45º, 60-65º and 80-85º with respective Q-slope values of approximately 0.1, 1.0 and 10.

Assessing rock slope stability in arctic and alpine environments brings its own challenges both during the peak of winter when ice building in joints can result in wedging or jacking, and in the pre- and post-winter seasons when cyclic freeze-thaw effects often degrade the quality of the rock mass. Modelling such processes using numerical techniques is possible to some extent; however, it is impractical as a routine application.

This paper discusses the use of the Q-slope method as a means of appraising rock slope stability in environments susceptible to ice wedging and freeze-thaw effects.

Keywords

Q-Slope, rock slope stability, empirical method, arctic environment, alpine environment

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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 numerical modelling. Excavation is usually too fast for this. The same limitation usually applies to tunnelling, despite numerical modeller'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 enable 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).

Q-slope utilizes the same six parameters as the Q-system: RQD, J_n, J_r, J_a, J_w and SRF (Barton et al. 1974; Barton and Grimstad 2014). 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 (0-factor), like $(J_r/J_a)1$ x 0.75 for set J1 and $(J_r/J_a)2$ x 0.9 for set J2, provide estimates of overall whole-wedge frictional resistance reduction, if appropriate. Bar and Barton (2017; 2018) provide additional guidance on the practical use of orientation factors. The term J_w, which is now termed J_{wice}, takes into account an appropriately wider range of environmental conditions for 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. Eq. 1 is the formula for estimating Q-slope:

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

Where *RQD* Rock quality designation
J_n Joint sets number
J_r Joint roughness number
J_a Joint alteration number
0-factor Discontinuity orientation factor for the ratio J_r/J_a
J_{wice} Environmental and geological condition number
SRF_{slope} Maximum of three strength reduction factors: SRF_a, SRF_b and SRF_c
SRF_a Physical condition number
SRF_b Stress and strength number
SRF_c Major discontinuity number

Tables 1 to 9 present the Q-slope ratings used in Eq. 1. The strength reduction factor SRF_{slope} is obtained by using the most adverse, or maximum, of SRF_a, SRF_b and SRF_c. Bar and Barton (2017; 2018) provide further insight and guidance on the Q-slope method and ratings; and provided a simple formula for long-term stable, steepest slope angle (β) not requiring reinforcement or support (Eq. 2).

$$\beta = 20log_{10}Q_{slope} + 65 \tag{2}$$

Table 1 Rock quality designation (Deere 1963)

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

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

Table 2 Joint sets number

Joint sets 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

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

i) Descriptions refer to small-scale features and intermediate scale features, in that order.
ii) Add 1.0 if mean spacing of the relevant joint set is greater than 3m.
iii) J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength.
iv) J_r and J_a 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 $\tau \approx \sigma_n \tan^{-1} (J_r/J_a)$.

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 ≈ 1-5mm)		
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 Ja depends on per cent 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 5 Discontinuity orientation factor – 0-factor

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

Table 6 Environmental and geological condition number

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

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

Table 7 Physical condition number - 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 & 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 Stress and strength number - SRF_b

Stress and strength description		σ_c / σ_1 *	SRF_b
F	Moderate stress-strength range	50 - 200	2.5 - 1
G	High stress-strength range	10 - 50	5 - 2.5
H	Localized intact rock failure	5 - 10	10 - 5
J	Crushing or plastic yield	2.5 - 5	15 - 10
K	Plastic flow of strain softened material	1 - 2.5	20 - 15

* *σ_c = unconfined compressive strength (UCS).*
σ_1= maximum principal stress.

Table 9 Major discontinuity number - SRF_c

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

* *RQD_{100} = 1 metre perpendicular sample of discontinuity.*
RQD_{300} = 3 metres perpendicular sample of discontinuity.

2 Rock Engineering Challenges in Arctic and Alpine Environments

The Arctic is the northernmost polar region of Earth, typically north of the Arctic Circle (66° 33'N), where the average summer (July) temperature is below 10°C and the northernmost tree line roughly follows the isotherm at the boundary of this region (Smithson et al. 2008).

Alpine environments have many descriptions including 'regions above the tree line', 'mountain climate' and 'highland climate'. McKnight and Hess (2000) combine alpine and mountain climates in the same group or category as polar climates (including arctic) in the Köppen climate classification. Arctic and alpine environments are similar in that they are cold with temperatures routinely well below freezing (0°C). In the spring, summer and autumn months, temperatures may frequently oscillate between above and below freezing in some areas – enabling cyclical freezing and thawing.

The effects of freeze-thaw cycles on intact rocks and soils have been investigated in several laboratory experienced, which call generally concluded that some rocks and soils are more susceptible to degradation than others, but that all experience some form of degradation over time (Chamberlain and Gow 1979; Yu et al. 2015) Degradation typically manifests in a reduction of strength and increased permeability and is a physical, or mechanical, weathering process commonly observed in arctic and alpine environments. This weathering process is called frost weathering, cyrofracturing or ice wedging.

Recent research has indicated that saturated, porous rocks, have markedly higher intact rock strength (compressive, tensile and shear) when frozen compared to both unfrozen-saturated and even when dry (Qi et al. 2016; Davarpanah and Vásárhelyi 2018; Török et al. 2019; Wang et al. 2019).

In rock masses rather than intact rock, freeze-thaw cycles also have the potential to cause ice-jacking, whereby blocks of rock in a slope are displaced or destabilized as water infilling open joints progressively freezes and expands. Block dislodgement or failure can be experienced during freezing or the subsequent thawing cycle when additional shear resistance from freezing dissipates.

The complex and time-dependent mechanical processes involving ice wedging experienced in arctic and alpine environments are currently near-impossible to replicate in numerical simulations on a slope scale. However, the use of empirical or observational approaches remains practicable.

3 Q-Slope addressing ice wedging and freeze-thaw phenomena

A total of 90 case studies within the Q-slope dataset are from Arctic or Alpine environments from various locations including Canada, New Zealand, Serbia and Slovenia. These case studies, shown in Fig. 1, conform quite well with Q-slope data from other environments and Eq. 2.

Three case studies are examined in this paper to demonstrate some of the complexity around rock slopes in Arctic and Alpine environments; Table 10 presents a summary of the results.

Table 10 Case Studies – Q-slope addressing ice wedging and freeze-thaw effects

	Case Study 1		Case Study 2		Case Study 3	
RQD	100		25 - 50		50 – 60	
J_n	3		9		6	
Set	*A*	*B*	*A*	*B*	*A*	*B*
J_r	3	-	1	4	1.5 - 3	-
J_a	3	-	2	1.5	2 - 3	-
O-factor	0.5	-	1	1	1	-
J_{wice}	0.9		0.5		0.9 x 1.5	
SRF_a	-		5		-	
SRF_b	-		2		2	
SRFc	1		-		-	
SRF_{slope}	1		5		2	
Q_{slope}	15		0.37 - 0.74		2.81 - 15.2	
β (°)	89		56 - 62		74 - 89	

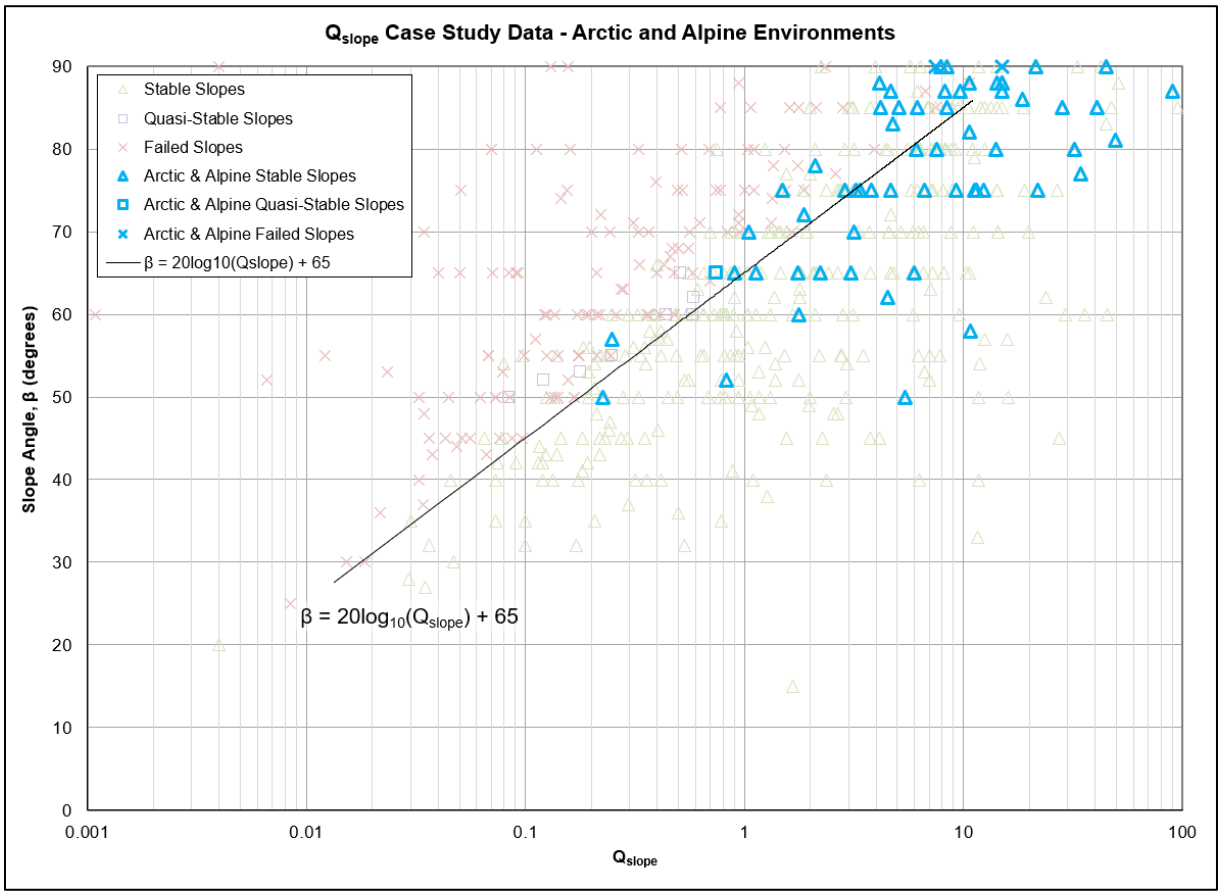

Fig. 1 Q-slope dataset: 90 of 500+ case studies from Arctic and Alpine Environments

3.1 Case Study One – Ice Jacking in Stone Quarry

A dominant, sub-vertical but slightly outward dipping joint set initiated a localized block fall in a stone quarry in Slovenia as shown in Fig. 1.1. Unconfined compressive strength of the limestone and marble typically exceeds 200 MPa.

It is understood that ice jacking is a prominent challenge working in this environment and that Alpine Engineers manually remove loose blocks and destabilizing areas on an annually basis by accessing the slope faces using ropes and rappelling down.

Slope faces are excavated at 90° without using explosives to avoid fracturing the stone. RQD is almost exclusively 100%, except near localized shears found at approximately 5 – 10 metre spacing. The slope failed back along the joint, with the back scarp being approximately 86 - 88°, i.e. very narrow sliver failure was actually induced by the joint.

3.2 Case Study Two – Frost Weathering on Alpine Road

Frost weathering is progressively degrading both the intact rock properties and joint condition on a ski field road cutting on the Southern Island of New Zealand (Fig. 1.2). The exposed schist has an unconfined compressive strength of R2-R3, i.e. in the order of 25 MPa.

The 15 metre high slope was excavated at approximately 65° and is located at an altitude of 1600 metres above sea level, in an alpine environment. The slope appears to be quasi-stable with wedge being progressively undercut through the weathering process. Localized (small) block falls and unravelling have occurred, and cracks are evident in the slope face.

RQD is poor (25-50%) and likely to reduce with time due to progressive frost wreathing. A distinct planar-smooth joint set, quite favourably oriented forms part of the wedge with the second half being made up of discontinuous joints that appear to be becoming more connected with time, i.e. with frost weathering cycles. It also appears that erosion from surface water runoff is impacting stability.

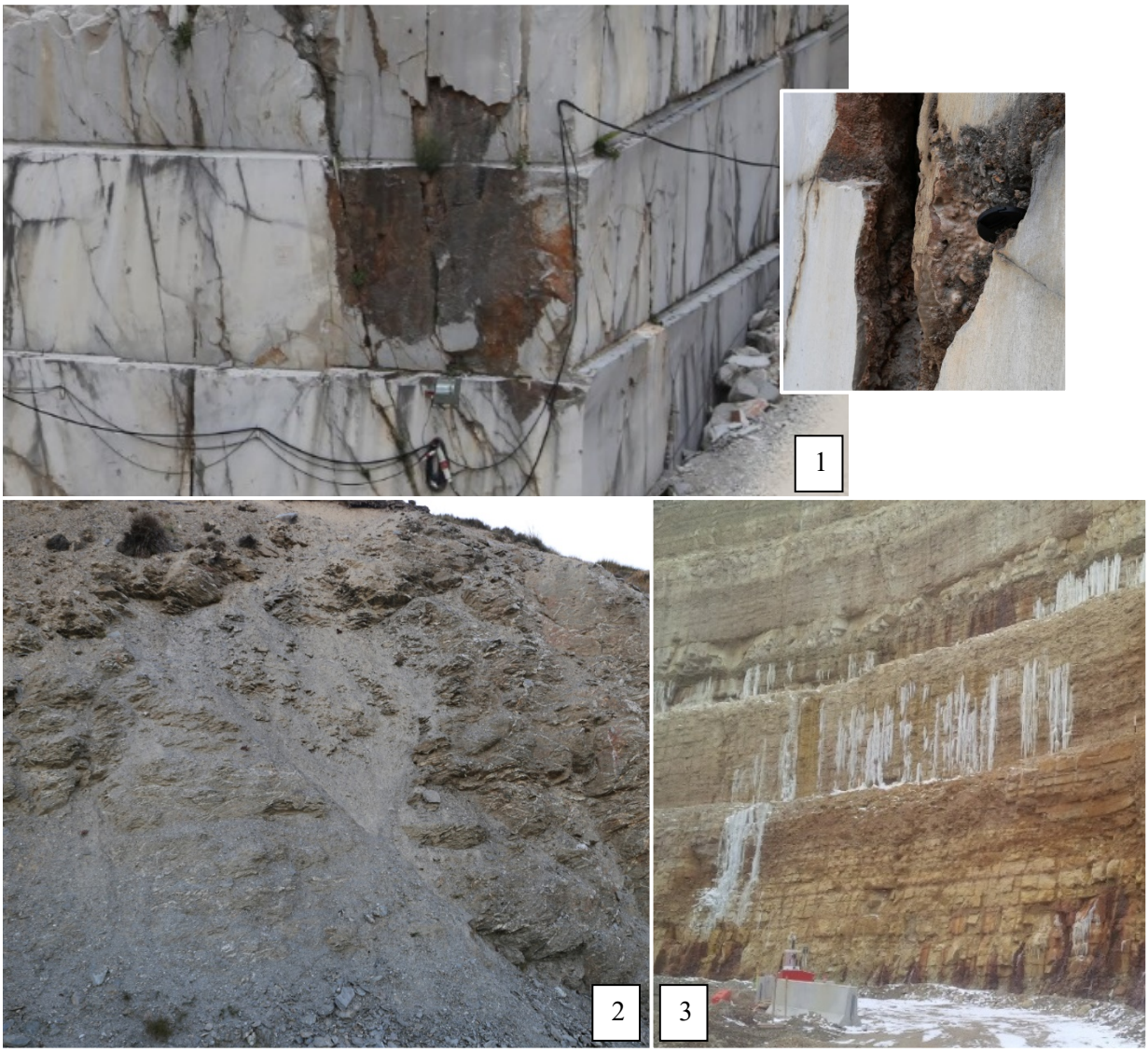

Fig. 2 [1]: Ice jacking block fall in limestone from Case Study 1 (inset shows progressive ice wedging damage in an open joint); [2] frost weathering from Case Study 2; [3] Freeze-Thaw on mine pit slopes from Case Study 3.

3.3 Case Study Three – Freeze-Thaw in Arctic Mine

Water ingress combined with freeze-thaw effects has significant potential to destabilize mine pit slopes in an Arctic region of Canada. The pit slopes are constructed in altered limestone in close proximity to a geological contact with kimberlite (Fig. 1.3).

Unconfined compressive strength is approximately 45 MPa. Joint orientations are favourable, and the joints are generally not pervasive when compared to the slope height. Joint roughness and alteration are quite variable and horizontal drains are used to depressurize the slope.

The bench slopes are 20 metres high and are excavated at approximately 85°. Q-slope estimates suggest the slope should generally be stable long-term stability when excavated up to 75°. The 'stable lifetime' of the slope is for now, unknown.

Discussion

Ice wedging and freeze-thaw effects in Arctic and Alpine environments can have a profound impact on rock slope stability and cannot be practically predicted using numerical simulations. The Q-slope method has been effectively used to assess the stability of 90 case studies involving rock slopes in Arctic and Alpine environments.

A challenge of understanding rock slope stability, and therefore, using the method, is the 'time-dependency' aspect which is very relevant in these environments. For designing slopes using Q-slope, users are advised to consider appropriately using strength reduction factors, particularly for physical condition (SRF_a).

Future work relating to the Q-slope method should consider, in more detail, the time-dependent effects of different environments or climates on various rock slope. A key constraint of the future study is expected to be data availability and as such, researchers and practitioners are encouraged to contribute.

Acknowledgements

The authors sincerely thank Phil de Graaf, Marc Rougier, Katie Goguen and Sarah Cain for case study contributions from Canada.

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