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ROCK ENGINEERING AND ITS OBSESSION WITH ROCK BRIDGES: WHY EVERYTHING WE CALL REAL CANNOT BE REGARDED AS REAL

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EXTENDED ABSTRACT

È pratica comune considerare nella progettazione di versanti in roccia la presenza di ponti in roccia (*rock bridges*) come componente resistente che contribuisce ad aumentare la coesione apparente di un ammasso roccioso potenzialmente instabile. Analogamente, alla rottura dei ponti in roccia viene spesso attribuito il raggiungimento delle condizioni di rottura di un versante in roccia. Leggendo articoli scientifici su questo argomento, si ci potrebbe convincere che i ponti di roccia siano oggetti tangibili e misurabili e che quindi possano essere rappresentati nei calcoli di progettazione da un parametro (o da un insieme di parametri). Questa assunzione, lungi dall'essere corretta, deriva dall'utilizzo di metodologie imperfette sviluppate negli ultimi 50 anni ed è in qualche modo rafforzata dall'idea che esperimenti alla scala di laboratorio su campioni con fratture predefinite siano rappresentativi dei ponti di roccia alla scala del versante. Terzaghi definì i ponti in roccia come il rapporto tra l'area dei giunti nel piano della sezione e l'area totale del piano di sezione. Questa definizione si riferisce ai ponti in roccia nel piano e non va oltre un'interpretazione teorica del problema. E' difatti impossibile determinare l'esatta estensione dei ponti in roccia. Partendo dal presupposto che la rottura possa essere descritta utilizzando il criterio di Mohr-Coulomb, si potrebbe tentare di calcolare l'area di roccia intatta necessaria per garantire la stabilità apparente del pendio. Tuttavia, i risultati dipendono dai valori assunti di coesione della matrice rocciosa, di angolo d'attrito interno per i ponti di roccia, e di coesione apparente e angolo d'attrito per le discontinuità che contengono i ponti. Altro fattore, ancora più importante, l'analisi deve presupporre il numero di ponti di roccia e la loro posizione. Diventa evidente che anche nel caso di un caso relativamente semplice come quello di un solo blocco su un piano inclinato, sarebbe impossibile convalidare queste ipotesi senza eseguire test distruttivi. L'industria e il mondo accademico continuano a ignorare questa fondamentale ambiguità. Ad oggi, si ritiene ancora che i ponti di roccia possano essere definiti (*a priori*) come semplici entità geometriche secondo le definizioni e le equazioni di resistenza proposte da Jennings negli anni sessanta e settanta. Ciò non sorprende, dal momento che la comunità dell'ingegneria meccanica è lenta nell' accettare cambiamenti e continua a giustificare l'uso di metodi di progettazione sulla base di abitudini empiriche invece di adeguati processi scientifici. In questo articolo proponiamo una sfida a questa interpretazione tradizionale e un nuovo approccio che potrebbe avere un impatto significativo sul modo in cui comprendiamo e analizziamo i ponti in roccia. Per comprendere i ponti di roccia, è necessario sfidare l'idea di ciò che costituisce la realtà e accettare l'idea che non è possibile definire e misurare qualcosa che non esiste e che si manifesta nel momento in cui che il versante diventa completamente instabile.

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ABSTRACT

Rock bridges are critical in determining the stability of rock slopes and underground excavations. However, measuring them is impossible since their definition extends beyond a mere geometrical problem. Rock bridges are understood primarily in the context of rock mass strength, representing known unknowns akin to the principle of complementarity in physics. The current understanding suggests that while we can measure rock bridges post-failure, their pre-failure definition and measurement elude us. Historically, researchers have focused on the geometric perspective of rock bridges, with limited attempts to investigate actual field evidence. This has led to a disconnect between the theoretical problem and practical measurement. Since Terzaghi first highlighted this issue in 1962, more progress has yet to be made in addressing the fundamental limitations of our understanding of rock bridges. This paper argues for a paradigm shift towards analyzing rock bridges through the lens of rock mass damage and recognizing that rock bridge strength is directionally dependent.

Keywords: *rock bridges, natural slope, engineered slopes, rock mass damage.*

INTRODUCTION

It is common practice to design rock slopes by invoking the presence of rock bridges as a resisting component (*e.g.*, Call & Nicholas, 1978; Read & Lye, 1984; Baczynski, 2008; Valerio *et alii*, 2020). Similarly, engineers often explain observed slope failures regarding rock bridge failure. When reading papers on this subject, one may be excused if concluding that rock bridges are tangible and measurable objects that can be represented by a given parameter (or a set of parameters) in design calculations.

As the title of this paper suggests, the question of rock bridges becomes a question of what we call reality. As Elmo (2023) discussed, the interpretation of rock bridges in the literature needs to be corrected. It results from flawed methodologies developed in the last 50 years. It is somehow reinforced by the idea that laboratory-scale experiments of samples with predefined cracks are representative of rock bridges in the field.

Terzaghi (1962) defined rock bridges as the ratio between the area of the joints in a section plane and the total area of the section plane (Fig. 1a). This definition is limited to in-plane rock bridges and does not extend beyond a theoretical interpretation of the problem. Indeed, it would be impracticable to determine the extent of the intact rock portions (rock bridges) that contribute to the stability of the tabular blocks shown in Figure 1(b). Under the assumption that failure can be described using a Mohr-Coulomb criterion, one may attempt to back-calculate the area of intact rock responsible for the stability of the tabular blocks. However, the results depend on the assumed values of internal rock cohesion and internal friction for the portion of intact rock holding the blocks in place and the apparent cohesion and friction angle for the continuous portion of the failure surface. More importantly, the back-analysis must assume several rock bridges and their locations. It becomes apparent that even in a relatively simple case like the one shown in Figure 1(b), it is only possible to validate these assumptions by performing destructive testing.

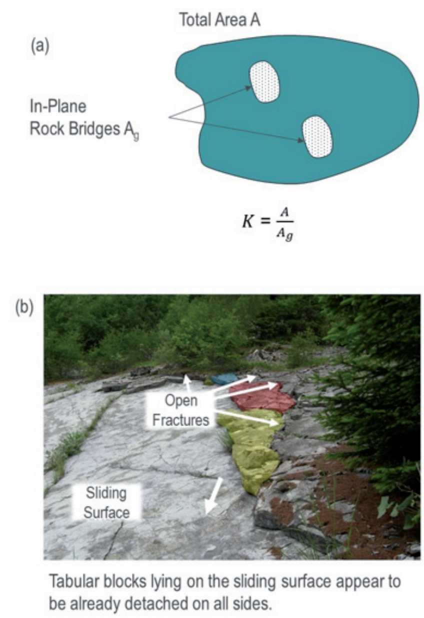

Fig. 1 - (a) Definition of rock bridge by Terzaghi (1962) and (b) example of tabular blocks that appear to be already detached on all sides and yet not sliding (adapted from a photograph by Andrea Wolter, in Donati et alii (2023))

Industry and academia continue to ignore this fundamental ambiguity. They still trust that rock bridges can be defined (*a priori*) as simple deterministic geometrical entities according to the definitions and strength equations proposed by Jennings (1967) and Jennings & Steffen (1972). This should not be a surprise since the rock engineering community is known to resist changes (Elmo & Stead, 2021) and justify using design methods based on empirical habits instead of proper scientific processes. In reality, engineers rely on those dated formulae as a convenient excuse to justify using limit equilibrium methods or highly simplified finite element and discrete element methods. We understand the gravity of such accusations, but these are supported by a list of significant limitations, including:

i) Rock bridge failure in Jennings's method has no temporal dimension (*i.e.*, all rock bridges fail simultaneously).

ii) Rock bridge strength in Jennings's method is independent of the location and number of rock bridges.

iii) Rock bridge strength in Jennings's method is based on accurately measuring the number of rock bridges and their dimensions at the design stage.

iv) Rock bridge strength is expressed in terms of the strength of an equivalent rock mass material. Internal cohesion (intact rock) and apparent cohesion (fracture surface) are combined into an equivalent rock bridge cohesion, ignoring the fact that apparent cohesion, by definition, is not a true cohesive force.

v) Rock bridges only exist in a fractured rock mass with non-persistent fractures (*i.e.*, it ignores rock mass damage due to interlocking).

Using Jenning's and step-path methods that calculate rock bridge strength using similar formulations can lead to conclusions that may refer to nonrealistic failure mechanisms and potentially non-conservative design calculations. Our research challenges this traditional interpretation and proposes a novel approach that could significantly impact how we understand and analyze rock bridges, potentially leading to more accurate and effective design calculations. To understand rock bridges, one must challenge what constitutes reality and accept that it is impossible to define and measure something that does not exist until failure has occurred.

ROCK MECHANICS *vs* ROCK ENGINEERING

Understanding the distinction between rock mechanics and rock engineering is extremely important when studying rock bridges since they constitute known unknown problems that engineers seek to transform into known known ones (Fig. 2).

Rock mechanics is the scientific study of the mechanical behaviour of rock materials and rock masses. It involves understanding the physical properties, responses to stress, and failure mechanisms of intact rock (lab scale) and rock masses (field scale). Rock engineering applies rock mechanics principles to design, construct, and maintain structures interacting with rock masses.

Rock engineering combines knowledge of measurable quantities (lab scale) with qualitative assessment (field scale). While field tests are possible, they are rare and, when one considers the scale of problems to be analyzed, still not truly representative of the rock mass as a whole. The definition of rock engineering is further complicated by the difference between constructing and excavating a structure. This semantic problem results in technical differences between what we define as an active design process (building – controlled by known knowns and known unknowns) and a reactive design process (excavating – controlled by unknown conditions).

Rock engineering should inform rock mechanics of the loading conditions that need replicating in a laboratory or a numerical setting. If we agree that rock mass damage is stress path dependent, then laboratory testing and numerical simulations must replicate stress path conditions equivalent to those encountered in the field.

For instance, when considering hard-rock pillars, a large-scale sample (*e.g.*, 1 to 2 m wide) loaded uniaxially would not be equivalent to a pillar excavated in a mine development. The spatial location of the pillar as part of an array of many pillars within a development panel impacts the stress redistribution within the pillar. At the same time, the four faces of the pillar are never excavated in one single pass.

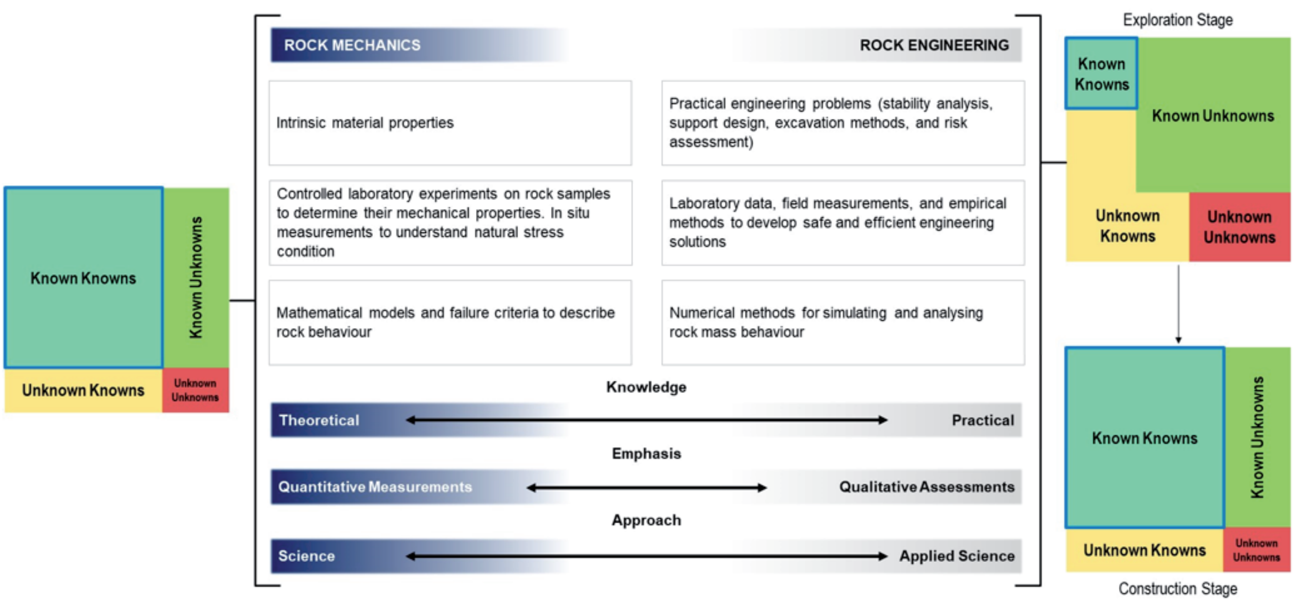

Fig. 2 - Different characteristics of rock mechanics versus rock engineering in relation to knows and unknowns

The period during which studies of rock bridge problems began to get traction (the 1960s and 1970s) was when the leading school of thought favoured the idea of rock masses as equivalent continuum media. This paradigm still dominates rock engineering design today, as demonstrated by the wide adoption of continuum methods and limit equilibrium methods to simulate rock engineering problems independently of their size and the relationship between problem size and structural geological conditions (Bewick & Elmo, 2024).

Numerical results can generally be calibrated and validated using deformation targets. However, under these circumstances, the numerical models depend on the empirically assumed rock mass deformation moduli. A numerical model must also match stress paths to further constrain the results since it controls the degree of rock mass damage. In Fig. 3 and Fig. 4, we present the results of numerical models for a large 800 m deep open pit. The model includes two different fracture networks (Model A, shallow dipping basal plane; Model B, steeply dipping basal plane). The rock mass strength component is represented by assuming a Hoek Brown failure envelope and decreasing rock mass quality from a massive GSI 90 to a blocky GSI 70. The results include fracturing indicators (Fig. 3) and vertical deformations (Fig. 4). Further details about the modelling conditions and material properties are included in Elmo *et alii*, (2009). When comparing the portion of the slope undergoing a deformation larger than 0.2 m and the corresponding damage zone (Fig. 5), Model A shows a much larger discrepancy than Model B, highlighting the critical and mutual role of embedded fracture networks and the strength of the rock matrix in controlling slope behaviour. It is also worth mentioning that neither Model A nor Model B shows large deformations at the toe of the slope despite the large amount of rock mass damage concentrated there. The contrast between simulated rock mass damage and deformation is a good indicator of the problem of using equivalent continuum properties to account for the role of fractures that could not be included directly in the model due to mesh size requirements to reduce computational times. Furthermore, the results show the importance of using monitoring data from deep inside the slope for calibration and validation purposes; indeed, in Model A, a

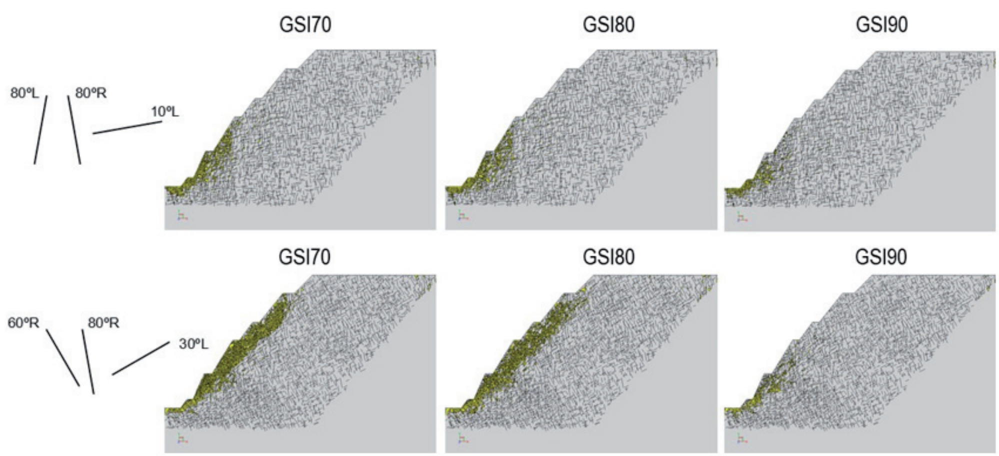

Fig. 3 - Numerical simulations of an 800 m slope based on geometry, stress and material properties were initially discussed in Elmo et alii, 2009. The figure shows slope areas impacted by rock mass fracturing (yellow areas) for varying rock mass properties and jointing conditions

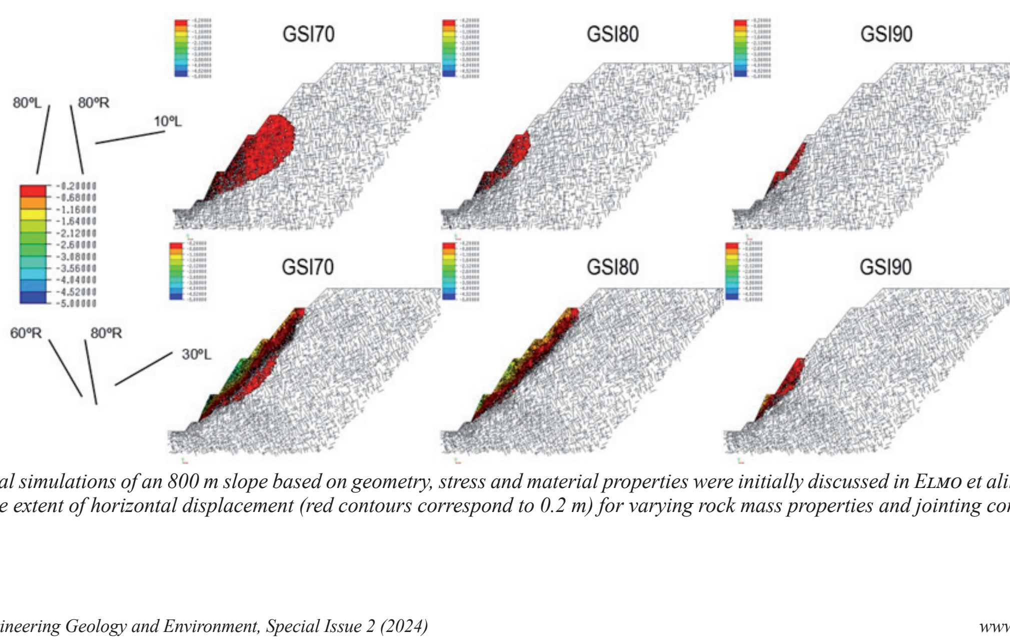

Fig. 4 - Numerical simulations of an 800 m slope based on geometry, stress and material properties were initially discussed in Elmo et alii, 2009. The figure shows the extent of horizontal displacement (red contours correspond to 0.2 m) for varying rock mass properties and jointing conditions

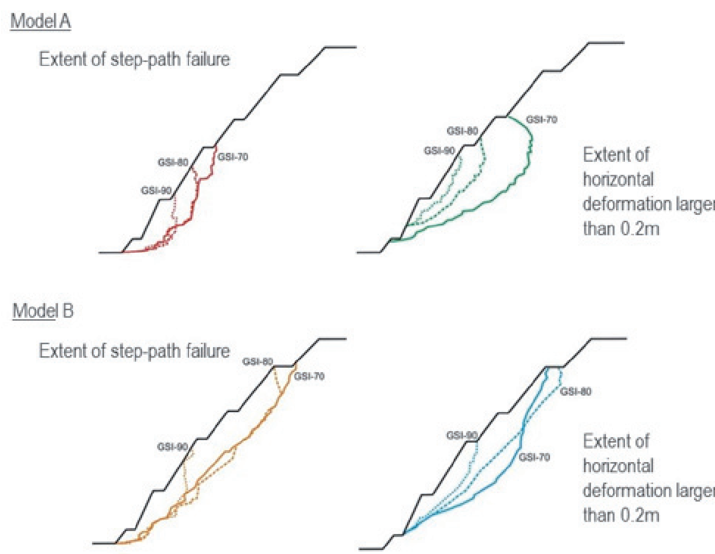

Fig. 5 - Numerical simulations of an 800 m slope based on geometry, stress, and material properties were initially discussed in Elmo et alii, 2009. The figure compares the extent of rock mass damage and horizontal deformation larger than 0.2 m

monitoring point located in the second bench close to the bottom of the pit would yield the same deformation independently of the assumed rock matrix properties.

The discussion above demonstrates that to effectively simulate rock bridge failure; numerical models should transition to the explicit simulation of rock mass damage and include a sufficiently large number of pre-existing discontinuities. Suppose there were no modelling constraints (*i.e*., no limits to discretization size, ability to consider fracture network, GPU processing). In this case, there is no reason we would need to rely on Jenning's approach to incorporate rock bridges in the design process since the model response would directly capture intact rock failure, interlocking and rock mass damage.

THE PHILOPSICAL LESSONS WE CAN LEARN BY STUDYING NATURAL SLOPES

Nature offers excellent examples of slopes whose stability can only explained in terms of rock bridge potential (Fig. 6 to Fig. 8). We have used the term "potential" in accordance with the Bologna Interpretation by Elmo (2023), who stated that rock bridges' location, geometry, and intensity can only be fully defined post-failure. We know rock bridges must exist for these natural structures to be stable. However, we cannot physically observe them. Therefore, we can neither define nor measure a rock bridge percentage as Jennings's method suggests. It is worth mentioning that describing these natural structures as "stable" is a rather qualitative attempt to explain their factor of safety (FoS). It appears paradoxical that we cannot accurately measure the factor of safety (FoS) of natural rock structures until they fail and the failure surface (or a combination of failure surfaces) is revealed. It is a binary model in which a slope is either stable (FoS greater than one) or unstable (FoS less than one). Numerical simulations of slope stability using shear strength reduction can provide the strength factor for the slope, which is analogue to a FoS. However, the strength factor must be calculated concerning an initial condition, and any numerical model must rely on assumptions when representing so-called initial conditions. Therefore, the strength factor is not an absolute measurement of the stability of a slope but rather a relative indicator that depends on the assumed failure envelope.

The corollary is conventional numerical methods cannot accurately predict the progressive failure of rock slopes like the one shown in Fig. 6 to Fig. 8. Furthermore, the classical rock bridge analysis (Jennings' method) ignores the presence of in-plane rock bridges, which are critical to the behaviour of the slopes as shown in Fig. 7 and Fig. 8. Even if one were to monitor a slope using conventional tools (*e.g*., inclinometers, GPS positioning systems, change detection), it would not be possible to translate the monitoring data into a detailed characterization of rock bridges (location, geometry, strength), except for back-calculating the overall (global) strength that they may provide (*i.e.*, the slope's rock bridge potential). The model output is either controlled by a deformation-driven calibration, or it inevitably assumes that it is inconsequential from an engineering perspective whether a failure occurs in stages. The latter condition is typical of limit equilibrium models.

Natural and engineered slopes adhere to a principle analogous to energy conservation. A rock mass potential exists that transforms into various kinematically controlled mechanisms, including block sliding and rotation, elastic deformation, plastic yielding, and intact rock fracturing. At the same time, it is accepted that using rock mass properties alone is not acceptable when discrete structures control failure, the question of when it is reasonable to use elastoplastic constitutive criteria has no simple answer. As discussed by STEAD *et alii*, (2007), the deformation of a rock slope can be considered plastic at the global scale. However, it primarily originates from tensile failure at stress levels significantly lower than the rock mass's plastic yield stress. The main distinction between slope failure due to plastic deformation and the one caused by the formation of tensile micro-fractures is that the latter results in dynamic changes in the kinematics of the rock slope mass as deformation progresses.

The San Leo rock slope (BORGATTI *et alii*, 2015) is a classic example of the latter. It is not a matter of knowing the actual FoS for the San Leo rock slope but of understanding the onset of potentially large kinematic mechanisms. To make matters more complex, the level of risk tolerated for San Leo is significantly lower than the one accepted when designing open pit slopes. This impacts the level and accuracy of the monitoring instruments required to corroborate models' predictions.

The discussion leads to a potentially provocative question: if we cannot measure the FoS of natural rock structures (*e.g.*, Fig. 6 to Fig. 8), why are engineers convinced that it is possible to design and excavate rock slopes on the basis of an FoS approach (*e.g.*, READ & STACEY, 2009, open pit slope guidelines)? MACCIOTTA *et alii*, (2020) reported that the ideal slope design must satisfy design acceptance criteria (DAC) while considering geotechnical uncertainties and the potential consequences of slope failure. DACs are an assumed FoS that can be adjusted to accommodate geotechnical uncertainty. The assumed FoS depends on the problem scale and the consequences of failure. In other words, the design process is not controlled by geotechnical aspects but by economic factors. Under these circumstances, failure is no longer an engineering condition but the outcome of financial risk assessments. Indeed, would engineers risk steeping a slope if there were no financial drawbacks? The same conclusions apply if using a probability of failure (PoF) approach instead of FoS, with the interesting condition that a slope designed with a FoS of –let us say– 1.3 may still yield a non-zero PoF. Somehow, like the proverbial Schrodinger's cat, the design of a slope can be imagined to be stable and non-stable at the same time, depending on the degree of geotechnical knowledge of the person conducting the study and the risk of failure they tolerate.

The quantum dimensionality of slope design results from using models that simplify reality. The FoS calculated by the models represents a simulated reality and not the actual reality. The FoS of an engineered structure exists only concerning a design function, and failure is defined as not meeting the assumed design function. For natural slopes, the design function only concerns human requirements (*e.g.*, a rock fall potentially impacting a highway). In other words, the concepts of FoS and stability become an anthropic hypothesis arbitrarily assigned to natural processes.

Human activity appears not to have directly impacted the natural structures depicted in Fig. 6 through Fig. 8. However, determining the exact point at which human activity contributes to the failure of natural rock slopes is complex. Two significant and mutually acceptable arguments can be provided:

- Regardless of their preservation or design quality, natural and engineered structures cannot withstand geological timescales indefinitely. They will ultimately succumb to natural erosion and other geophysical processes.

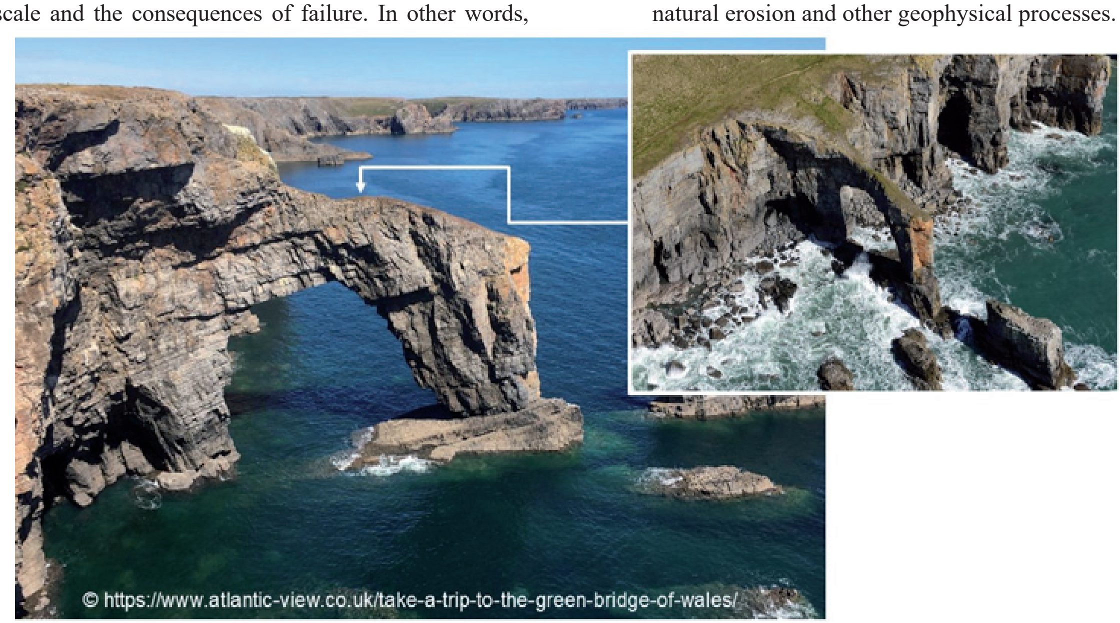

Fig. 6 - The Green Bridge of Wales (Carboniferous Limestone), located west of Flimston Bay in Wales (U.K.). This natural structure was damaged following heavy storms in mid-October 2017, which led to the failure of a large portion of the buttress on the southern end

- Human activities contribute to climate change, accelerating the frequency and intensity of extreme weather events and erosion processes. This perspective indicates that human actions are not without consequence and can significantly impact the stability of rock slopes.

While the first argument absolves humans by attributing slope failures to natural processes over long periods, the second argument is a stark reminder of the detrimental effects of human activities on the environment. Although we cannot prevent these failures from occurring in the distant future, we must take

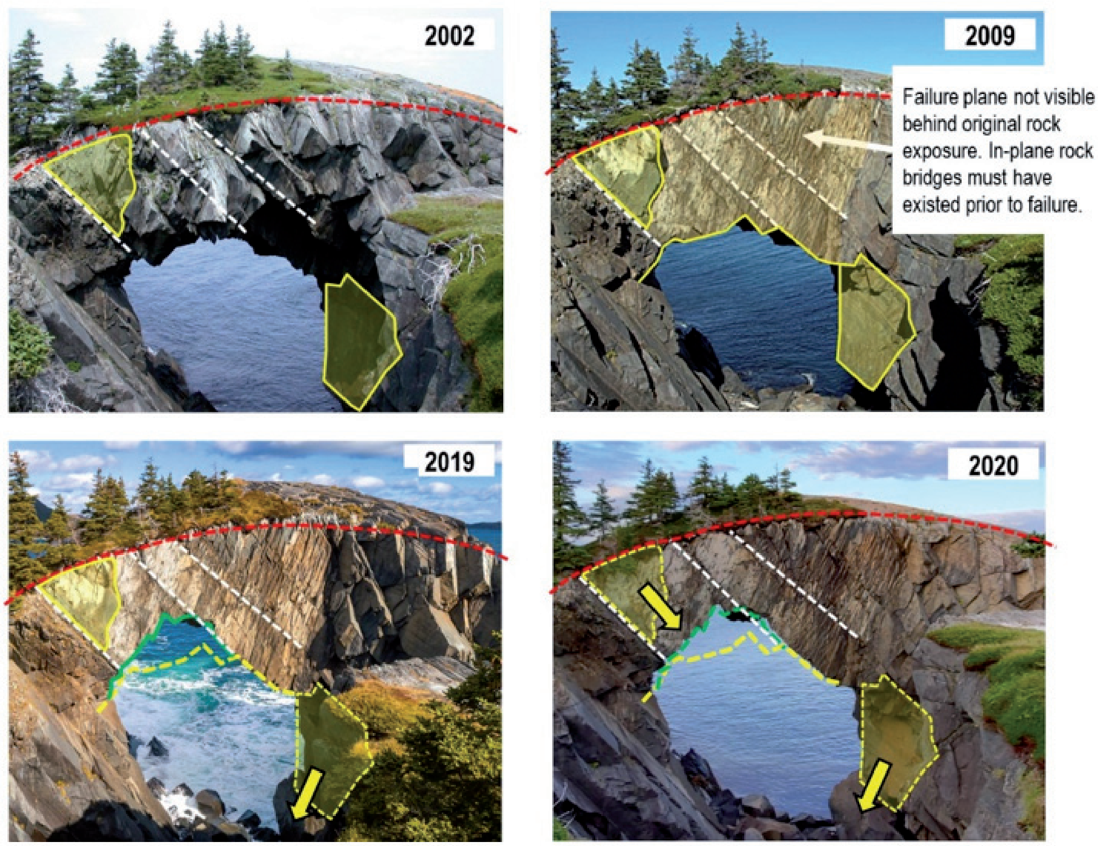

Fig. 7 - Evolution of Berry Head Arch (Newfoundland, Canada) from 2002 to 2020 (Elmo, 2023). Photos sourced from Google Images under a creative common license CC2.5

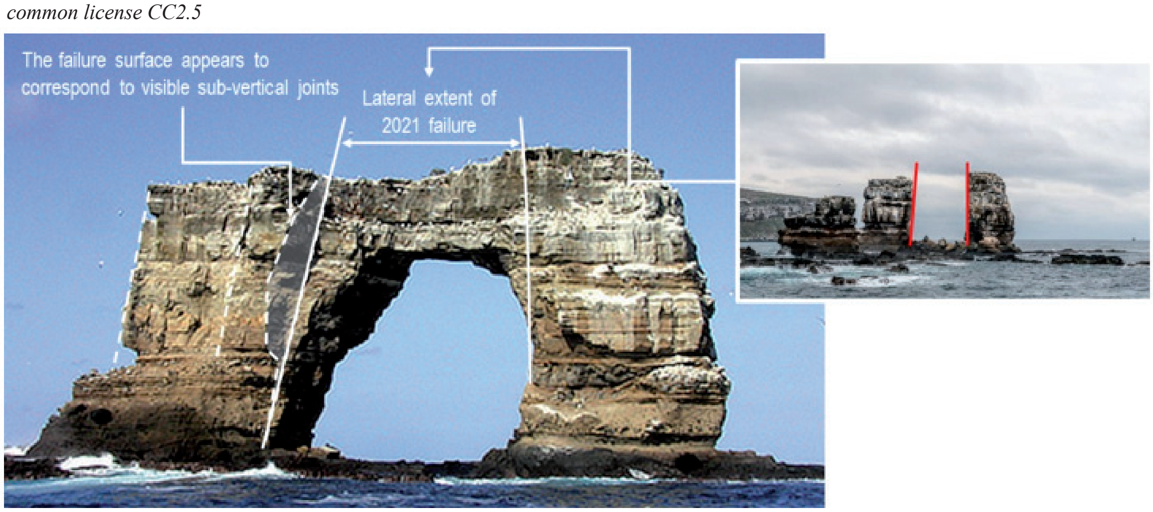

Fig. 8 - Darwin's Arch, Galapagos Island. This natural arch collapsed on May 17, 2021, leaving two pillars, aptly renamed Pillars of Evolution. Photos shared under a creative common license CC2.5

proactive measures to ensure that this distant future does not manifest prematurely due to our actions.

FINAL REMARKS

What are rock bridges? They exist, and yet they are not real. Because we cannot measure rock bridges, the challenge is to infer their impact on slope stability from other observable data. Unfortunately, many engineers chose to ignore the complexity of rock bridges' epistemology and continue to simplify rock bridges to geometric distances between non-persistent fractures. This simplistic approach has significant limitations and can lead to unrealistic failure predictions since strength equations based on so-called rock bridge percentages may overestimate the equivalent properties of the rock mass.

The study of rock bridges requires looking at design problems through the lens of the novel concept of rock mass potential, which depends on a combination of essential parameters, including:

- Intact rock strength.
- Loading conditions (magnitude and direction).
- Rock mass connectivity.
- Rock mass interlocking.

The concept of rock bridge potential implies adopting complex geomechanical models. However, it would be ill-advised to cite the complexity of geomechanical models as a justification for continuing to adopt traditional methods based on a flawed definition of rock bridge percentage based on being practical. The term "practical" pertains to applying a technique rather than the theoretical basis behind it. Seeking practical alternatives (e.g., limit equilibrium methods and rock bridge percentages) could lead to misinterpreting the risk of failure.

In our discussion, we have emphasized the uncertainties in determining the intensity and location of rock bridges before failure. Since rock bridges effectively materialize upon failure, accurately calibrating and validating forward analyses becomes practically impossible. Therefore, the design process should pivot towards the numerical modelling of fractured rock masses using discrete element models and principles of fracture mechanics.

Additionally, there is a pressing need to deepen our understanding of damage-related processes, including time-dependent damage. Enhanced focus on these areas will lead to more accurate predictions and safer engineering designs.

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