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The Bologna Interpretation of Rock Bridges

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Abstract: One can only know where a rock bridge is once one measures it. In addition, to measure it, you need the rock mass to fail. This critical problem is ignored by many, and engineers continue to refer to rock bridges as geometrical distances between non-persistent fractures. This paper argues that this rather simplistic approach can lead to non-realistic failure mechanisms. We also raise the critical question of whether the inappropriate functioning of strength equations centred on the measurement of rock bridge percentages could result in misinterpreting the risk of failure. We propose a new interpterion, aptly called the Bologna Interpretation, as an analogy to the Copenhagen Interpretation of quantum mechanics, to highlight the indeterministic nature of rock bridges and to honour the oldest university in Europe (Bologna University). The Bologna Interpretation does not negate the existence of rock bridges. What rock bridges look like, how many there are, and where they are, we do not know; we can assume their existence and account for their contribution to rock mass strength using a potential analogue.

Keywords: rock ridges; Bologna interpretation of rock bridges; rock bridge potential; rock mass stability; rock mass damage

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1. Introduction

Engineers agree that rock bridges control the stability of rock slopes and underground excavations. What eludes us is the ability to measure the extent of entities—rock bridges—that lack an accurate definition and only become real in the context of characterising rock mass strength. Rock bridges are indeed a problem of what constitutes reality, and for all intents and purposes, rock bridges represent known unknowns. Analogous to physics problems, rock bridges follow the principle of complementarity, and accordingly, we can postulate the following (adapted from [1]):

> *"We can measure the extent of rock bridges post-failure, but we cannot define—and therefore measure—what rock bridges look like pre-failure."*

Knowing the intensity of rock bridges pre-failure requires a clear and unique definition of what they look like; more importantly, it implies that we have dedicated tools to measure them. Even post-failure, there remains the challenge of translating the results of a back analysis into a unique measurement of rock bridge intensity, since to do so would require knowledge of the rock bridges' position pre-failure. In addition, if we knew their position pre-failure, then it would mean we would have the resources to identify what a rock bridge looks like, and therefore, we would be able to measure their intensity. Despite the paradox, engineers speak of rock bridges as real physical entities. They are not; only their impact in terms of stability is real.

Many researchers have looked at the problem of rock bridges from a geometric perspective, but few have attempted to look at actual field evidence of rock bridges. The result is a disconnect between what cannot be measured since it represents an undefined problem (rock bridge) and what engineers believe they are measuring (rock bridge percentage). Sixty years have passed since Terzaghi first looked at this problem in 1962 [2]. In this paper, we look at the issue of rock bridges through a critical lens; more importantly, we challenge the interpretation of rock bridge percentage attributed initially to [3]. There is a need to embrace a paradigm shift and analyse the problem of rock bridges from the point of view of rock mass damage, and, more importantly, to recognise that rock bridge strength is directionally dependent. In this paper, the authors explain how the problem of rock bridges can be better analysed using the principles of fracture network connectivity and a novel concept of rock bridge potential.

Not only does the topic of rock bridge analysis represent a niche field amongst the community of rock engineers, but over time, the lack of critical teaching and learning has contributed to the propensity to use what we argue are flawed methodologies. At the same time, the literature abounds with examples in which the results of laboratory-scale experiments with samples containing a limited number of pre-defined cracks are misconstrued as examples of rock bridge analysis, neglecting essential differences between simple pre-defined geometries and complex fracture networks in the field.

2. History of Rock Bridges and Rock Bridge Percentage

2.1. Terzaghi and Jennings's Interpretation of Rock Bridges

The problem of rock bridges has attracted the attention of many researchers since the early 1960s. Terzaghi [2] referred to rock bridges as gaps. He defined the ratio between the area of the joints in a section plane and the total area of the section plane as the effective joint area of the rock mass along the section (Figure 1a). Note that the definition of rock bridges in Figure 1a applies solely to in-plane rock bridges.

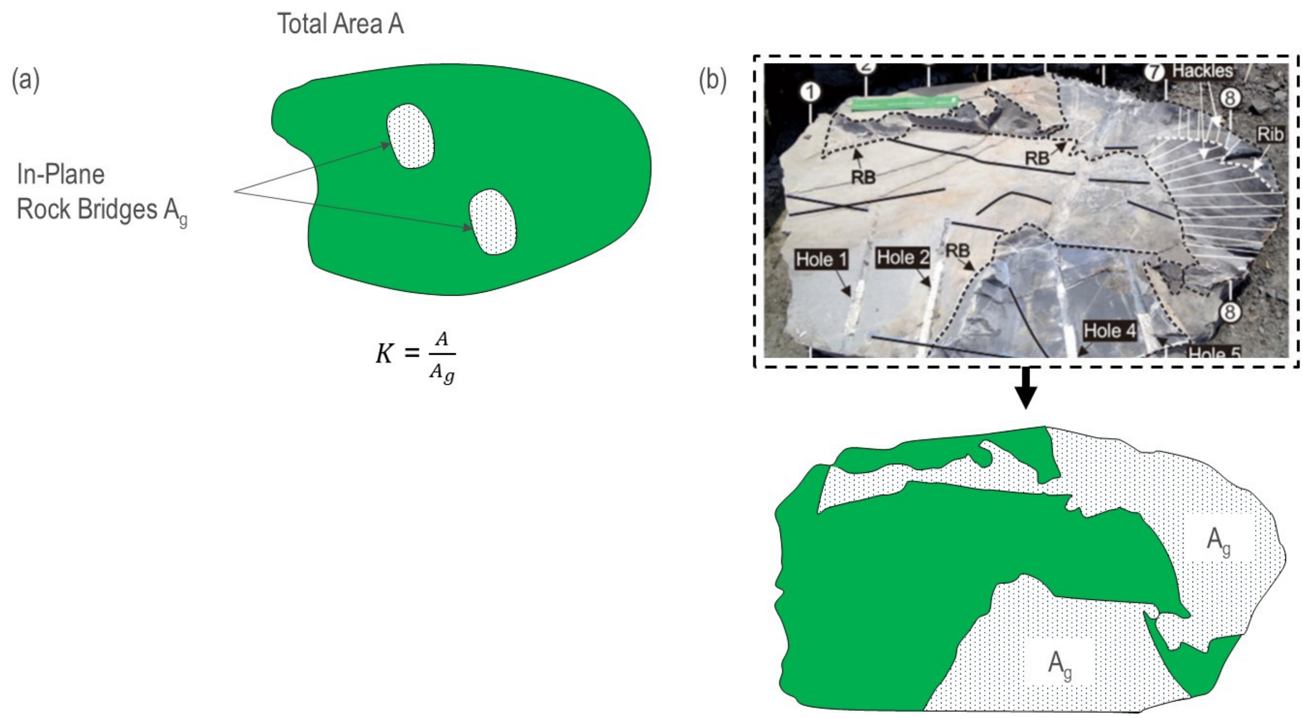

Figure 1. (**a**) Definition of rock bridge by [2], and (**b**) interpretation of rock bridges using an image from [4].

Shear failure across the section plane is resisted by both frictional strength (joint) and the cohesive intact rock strength. Terzaghi called the latter effective cohesion, defined as follows:

$$K = \frac{A}{\sum A_g} \tag{1}$$

According to [2], Equation (1) provides a simple theoretical contribution since it is impracticable to determine the value of the intact rock portions (rock bridges) for a given section plane through the rock mass. Shang et al. [4] demonstrated the existence of these gaps across natural joint surfaces using a forensic excavation technique (Figure 1b). Because it is impossible to validate assumptions about rock bridges' extent without performing destructive testing, the work by [4] has somewhat confirmed Terzaghi's hypothesis that rock bridge measurement is impossible before failure. Nonetheless, Terzaghi recognised that effective cohesion might be lost due to external forces, for example, the removal of lateral support and significant daily and seasonal temperature variations. Recent failures (Figure 2) of sheeting joint structures on steep slopes demonstrate Terzaghi's hypothesis. It is also reasonable to assume that blasting and excavation processes would significantly contribute to the loss of effective cohesion.

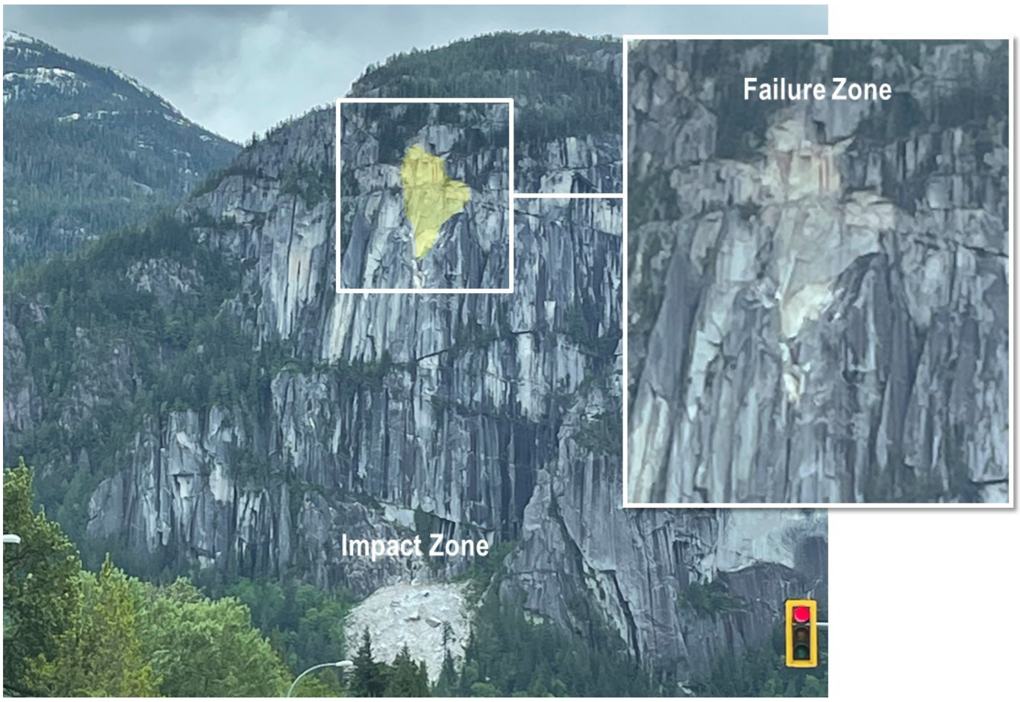

Figure 2. Example of failure of sheeting joint structures on steep slopes (Squamish, Canada).

While recognizing their role in controlling slope stability, Terzaghi declined to define ways to measure rock bridges. Subsequently, [3] and [5,6] built on Terzaghi's ideas and focused on the stability of rock slopes with discontinuous joints. The concept of rock bridge percentage first appeared in [3], which stated that the average or apparent cohesion for the whole surface would be a function of a factor (k), defined based on the length of the failure surface relative to the length of the portion of intact material (percentage of intact material intersected), as shown in Figure 3 and Equation (2).

$$c_a = kc_u \tag{2}$$

where c_a is the equivalent cohesion and c_u represents the shear strength of the rock material. In [3], the authors mentioned that the portion across the failure surface of the shear strength would be almost entirely cohesive for intact rock. Therefore, the intact rock frictional angle (ϕ_r) would be zero. The factor k in Equation (3) represents the ratio of the length of intact material to the total length of the potential failure surface. The same authors suggested that engineers could use a drawing of the slope showing the jointing pattern within the slope. Once they had delineated the potential failure surface, they could estimate the factor k according to the following:

$$k = \frac{\sum RB_i}{\sum L_i + \sum RB_i} \tag{3}$$

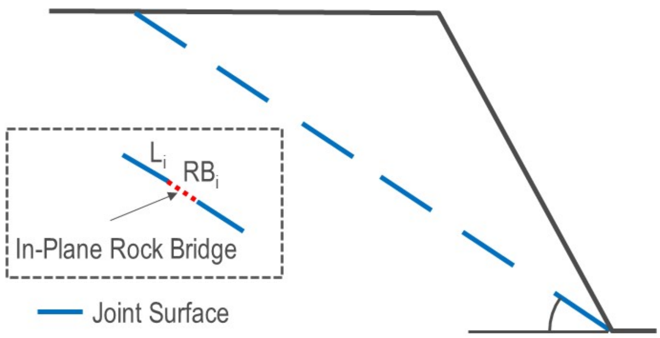

Figure 3. Simple slope problem introducing the problem of rock bridges according to the definitions by [3,5,6]. L_i and RB_i indicate the pre-existing joint surfaces and in-plane rock bridges, respectively.

In Equation (3), L_i and RB_i indicate the pre-existing joint surfaces and in-plane rock bridges, respectively. Several important questions arise. Are the rock bridges shown in Figure 3 to be interpreted as in-plane rock bridges according to the Terzaghi hypothesis [2]? If so, the 2D nature of Jennings and Steffen's interpretation [3] is such that, for a given joint surface, there would exist different 2D traces depending on the location of the in-plane rock bridges relative to the assumed section plane. As shown in Figure 4, the problem becomes undefined (we do not know where the in-plane rock bridges are located) and probabilistic at the same time (if we knew their location, we would need to consider multiple 2D realisations). If, on the contrary, the rock bridges shown in Figure 3 are not in-plane rock bridges but gaps between intermittent joints, is the assumption of co-planar irregular joints geologically valid? It becomes clear that [3] have ignored the comment made five years earlier by Terzaghi, and their work reflects a somewhat optimistic idea that rock bridges could be defined (a priori) as simple deterministic geometrical entities.

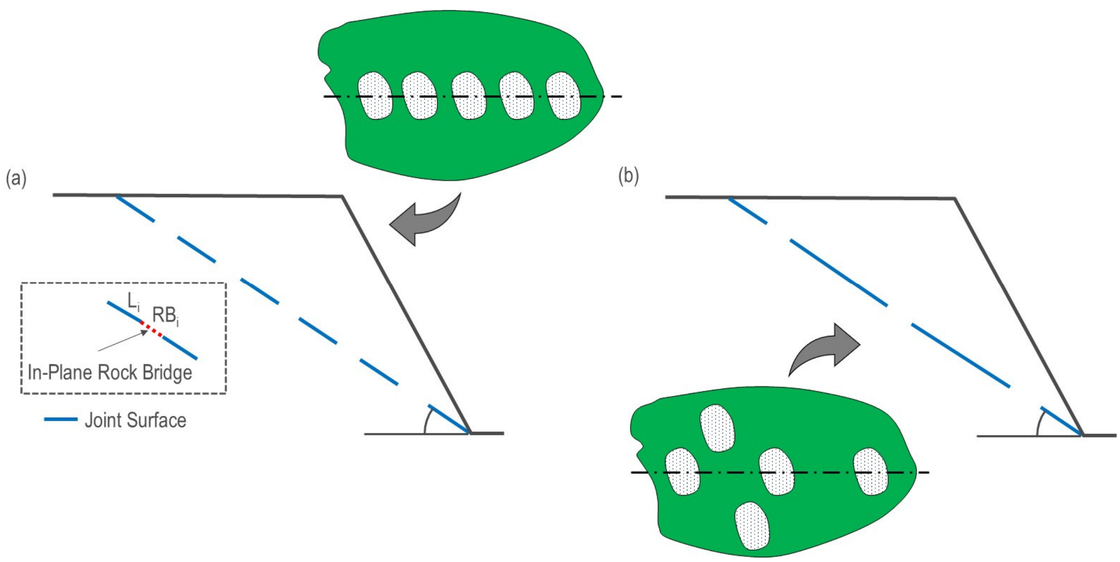

Figure 4. Simple slope problem describing the challenges of representing in-plane rock bridges using the method proposed by [3,5,6]. Even though (**a**,**b**) represent equivalent in-plane rock bridge conditions, a 2D cross section would result in different rock bridge lengths between (**a**,**b**).

Nonetheless, from [3], the interpretation of rock bridge percentage has come to influence the work of many researchers that followed. In this context, we could open a short epistemological parenthesis. Paraphrasing the words of the famous physicist Max Planck, we conclude that the arguments brought forward by [3] regarding their proposed geometrical definition of rock bridges did not triumph by convincing their predecessors (Terzaghi). Still, it triumphed because of the absence of opponents (indeed, Terzaghi had passed away four years earlier, and therefore he did not have the opportunity to challenge Jennings and Steffen's arguments).

While [2] only referred to frictional strength in addition to effective cohesion, [3] introduced the concept of the equivalent friction angle $\varnothing_a$ relative to the continuous portion of the joint surface.

$$tan\varnothing_a = (1-k)tan\varnothing_j \tag{4}$$

Similar arguments and equations were later presented by [5], who suggested that the behaviour of the slope shown in Figure 5 below would be equivalent to that of a rock slope composed of a hypothetical material with an apparent cohesion c_a and an apparent friction angle $\varnothing_a$. However, the same author provided a different definition of k. Therefore, readers should be careful when comparing the equations presented in the 1962 and 1967 papers (Equations (2) and (3)) with those published after 1970 (Equations (5)–(7) below).

$$k = \frac{\sum L_i}{\sum L_i + \sum RB_i} \tag{5}$$

$$c_a = (1-k)s_m + kc_j \tag{6}$$

$$tan\varnothing_a = ktan\varnothing_j \tag{7}$$

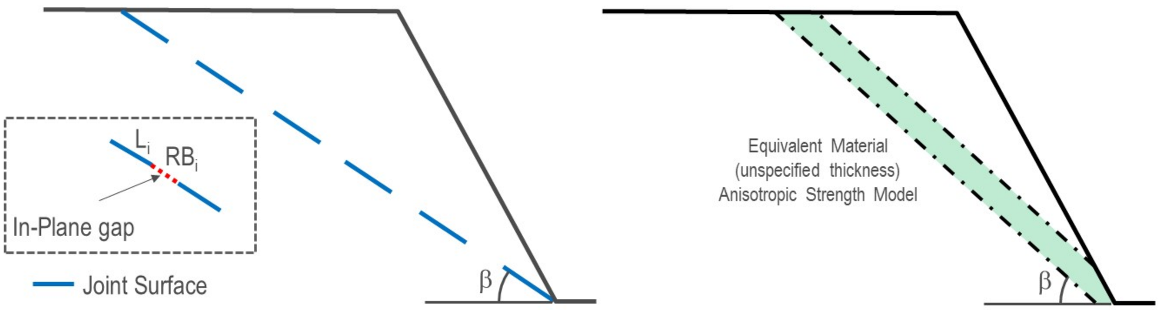

Figure 5. Simple slope problem showing how the presence of rock bridges is modelled using equivalent continuum methods.

It would be incorrect to use c_a and $\varnothing_a$ as the cohesion and friction of an equivalent discontinuity plane, since the cohesion parameter for joint surfaces is an apparent resistance mobilised as a function of the degree of roughness and applied normal and not true cohesive strength. In [6], Equation (7) above was changed to include the frictional contribution of the intact rock portion, though they did not clearly explain why the change was indeed required.

$$tan\varnothing_a = (1-k)tan\varnothing_m + ktan\varnothing_j \tag{8}$$

Jennings's method [5,6] assumes that the intact rock cohesion and the intact rock friction angle are mobilised simultaneously, and therefore, does not conform with more recent work by [7,8] about brittle failure processes. Arguably, this limitation applies independently of whether we use Equation (6) in combination with Equations (7) and (8). It is a natural consequence of using a Mohr–Coulomb failure criterion to simulate rock mass behaviour. The period during which the concept of rock bridge percentage was developed (the 1960s, 1970s) indeed betrays the rock bridge percentage approach as belonging to a school of thought that favoured the idea of rock masses as equivalent continuum media.

There is no doubt that the work by [5,6] has inspired subsequent studies on step-path analysis. These include the development of several probabilistic limit equilibrium methods (e.g., [9–13]). More recently, [14,15] described a technique for estimating rock bridge percentage that identifies potential failure pathways through trace maps. This approach is used in combination with Equations (6) and (8) to derive input properties for limit equilibrium models.

A literature review on step-path analysis reveals that, over the past 30–40 years, rock bridge percentage has somehow established itself as the industry standard. To understand this aspect, we need to look for an epistemological explanation. According to [16], many of the methods used by the rock engineering community have been elevated to the rank of an industry standard for no reason other than the personal stances of individual engineers, thus reflecting preferential attachment bias. Furthermore, epistemology teaches us that group work may not confer an advantage in contrasting orthodoxy, since members of the same group are more likely to have a similar knowledge imprint [17] and therefore accept practices without questions.

In Section 2.2 below, we present arguments to support our view that the definition of rock bridge percentage shown in the literature needs to be revised. Subsequently, Section 3 will introduce a new interpretation of rock bridges that focuses on characterizing damage-related processes and time-dependent damage.

2.2. Important Limitations of the Definition of Rock Bridge Percentage

The limitations of rock bridge percentage go beyond the nature of rock bridges—we should not think of rock bridges as deterministic and measurable physical entities. We trace the origin of these limitations to an imperfect geometrical interpretation that is still widespread among engineers today. In particular:

- The concept of rock bridge percentage was initially developed for co-planar intermittent joints (Figure 5 above) and later applied to simple step-path problems. However, we cannot postulate that it would apply to more complex fracture networks.
- The concept of rock bridge percentage assumes that rock bridge strength is independent of the location of the rock bridges. The calculation of rock bridge percentage does not account for the location of the rock bridges, and it inevitably leads to averaging the contribution of the intact rock portions across the assumed failure surface.
- The concept of rock bridge percentage assumes that rock bridge strength is scale-independent. Accordingly, the same measurement of rock bridge percentage could be associated with either a single (larger) rock bridge or many (smaller) rock bridges. Using simple numerical models, [18] showed that the loading at which rock bridges fail is not constant, even though the models assume the same rock bridge percentage. The results illustrated in Figure 6 show that the strength of a rock bridge is not only a function of its size but its location too.
- The rock bridge percentage concept assumes that rock bridges' failure can be modelled using limit equilibrium (LE) methods. By definition, LE analysis ignores kinematics (sliding, rotations, and moments) and brittle failure mechanisms that may potentially develop within rock masses. This raises the critical question of whether the inappropriate functioning of Equations (6)–(8) could result in underestimating the risk of failure. This important aspect is raised later in Section 3.
- The definition of rock bridges found in the literature assumes that only "positive" steps contribute to the kinematic characteristics of the resulting rock mass wedge (Figure 7). Therefore, a bias is introduced if a condition is imposed on the dip angle of joint surfaces ($0 \leq$ dip angle $\leq 90°$) to avoid cases in which failure of intact rock bridges would result in a negative gap and the formation of a tapered rock mass wedge. Despite having the same rock bridge percentage, the slopes shown in Figure 7 would behave very differently. Therefore, it would be incorrect to simulate their behaviour using equivalent (anisotropic) properties defined according to Equations (6)–(8). According to [19], ignoring the formation of negative rock bridges increases the risk of not considering the full spectrum of possible failure surfaces/modes. Negative rock bridges could contribute to increasing shear resistance, in which case, failure would not occur along a well-defined failure surface but through a zone of rock mass damage (Figure 7).
- The concept of rock bridge percentage is founded on the belief that gaps must exist in the fracture network for intact rock failure to occur. Therefore, it ignores the possibility

of intra-blocks fracturing and the combined role of network connectivity and block interlocking. When determining a rock bridge percentage, engineers need to impose an a priori failure condition defined by the existence of an equivalent discontinuity plane across which or along which failure will occur. However, intact rock failure can also happen for a fully connected fracture network [19]. This portion of intact rock failure cannot be accounted for by Equations (6)–(8).

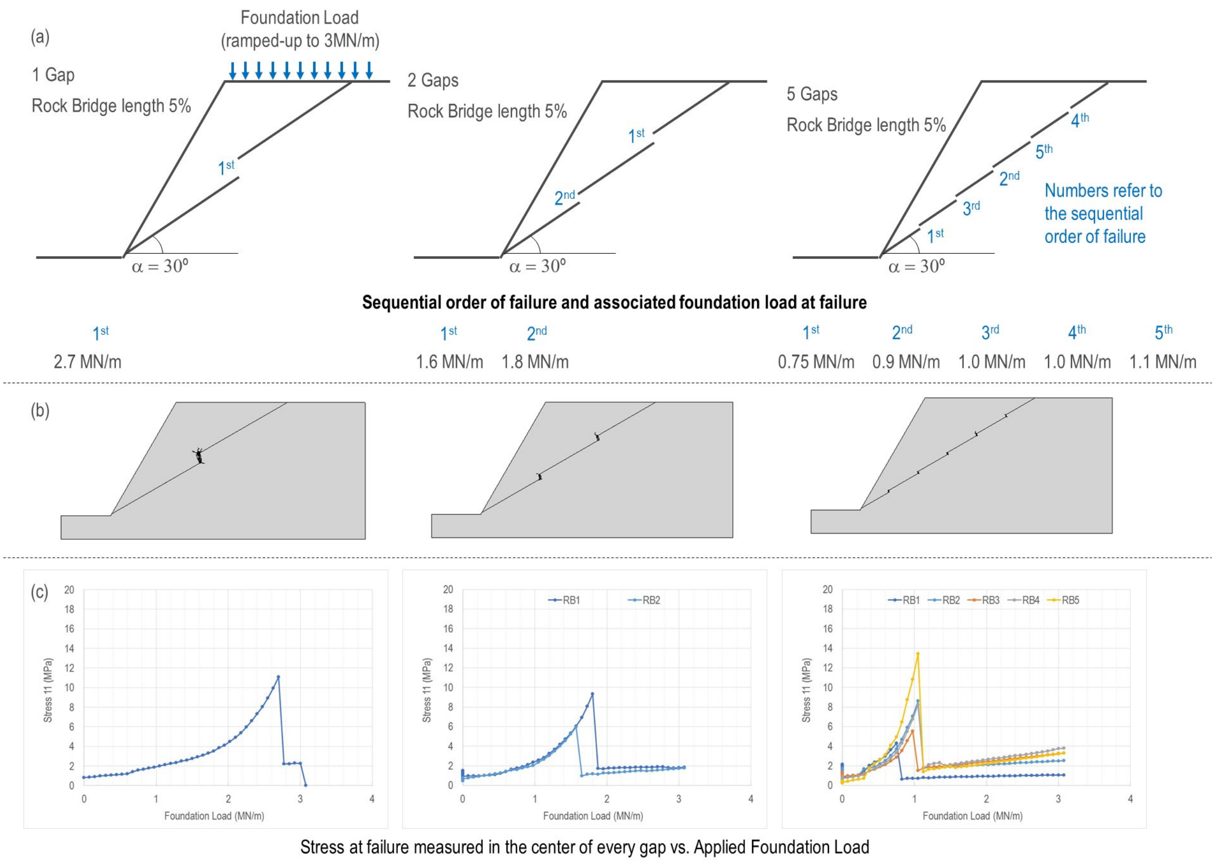

Figure 6. Results for the numerical simulations with 1, 2, and 5 rock bridges, respectively (modified from [18]). (**a**) Geometries and loading conditions; (**b**) fracturing results; and (**c**) principal stress sigma 11 measured at the midpoint of every rock bridge.

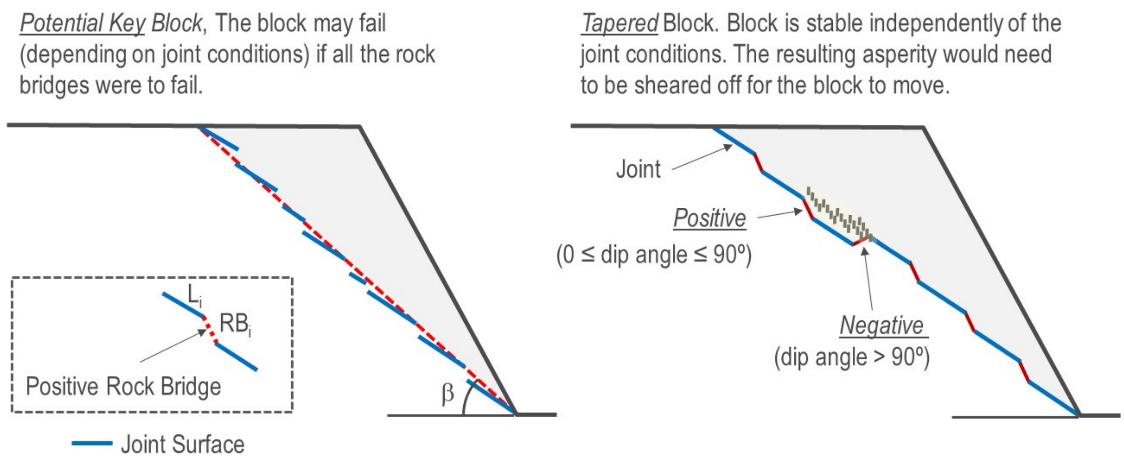

Figure 7. Influence of negative rock bridges concerning block forming potential and block theory definitions of potential key and tapered blocks, raising the importance of rock mass damage over the simple concept of rock bridge percentage.

In [19], the authors commented that block-forming potential and kinematics are instrumental in defining what constitutes a rock bridge. By doing so, the authors raised the need for a new interpretation of rock bridges. These new conditions add a directionality constraint to the definition of rock bridges and link the rock bridges' existence to variations in the applied loading conditions (magnitude and direction) and rock mass damage.

3. The Bologna Interpretation of Rock Bridges

The approach described by [3,5,6] incorrectly treats rock bridges as measurable gaps between intermittent joints. We propose a very different and thought-provoking interpretation of rock bridges, one that is genuinely interdisciplinary and borrows ideas and concepts familiar to philosophy and quantum mechanics. The quote attributed to the physicist Nils Bohr "Everything we call real is made of something we cannot call real" aptly describes the nature of rock bridges. Furthermore, we need to recognize the limits to how much information we can gather about rock masses, and that—paraphrasing Prof. Al.-Kalili—rock exposures are like curtains, behind which there is no reality, only the potential for reality. There exist two fundamental restrictions to our knowledge of rock bridges:

- A rock bridge exists only the moment it fails, and therefore we can neither see nor measure a rock bridge until it has failed;
- Because of their conditional existence, we can only describe the contribution of rock bridges to rock mass strength as a potential.

To understand these restrictions, we can reinterpret an analogy proposed by Prof. Al.-Kalili to explain quantum entanglement: "as the coin spins, one cannot tell whether the coin is tail or head. Only when the coin stops the result (head or tail) is revealed." In our version of the analogy, the spinning coin represents a stable rock mass; we know rock bridges exist in it, but to see and measure them, we need the rock mass to fail (i.e., the coin stops spinning). We call this paradigm the Bologna Interpretation of rock bridges as an analogy to the Copenhagen Interpretation of quantum mechanics, to highlight the indeterministic nature of rock bridges and to honour the oldest university in Europe (Bologna University). The interpretation is illustrated and explained in Figure 8.

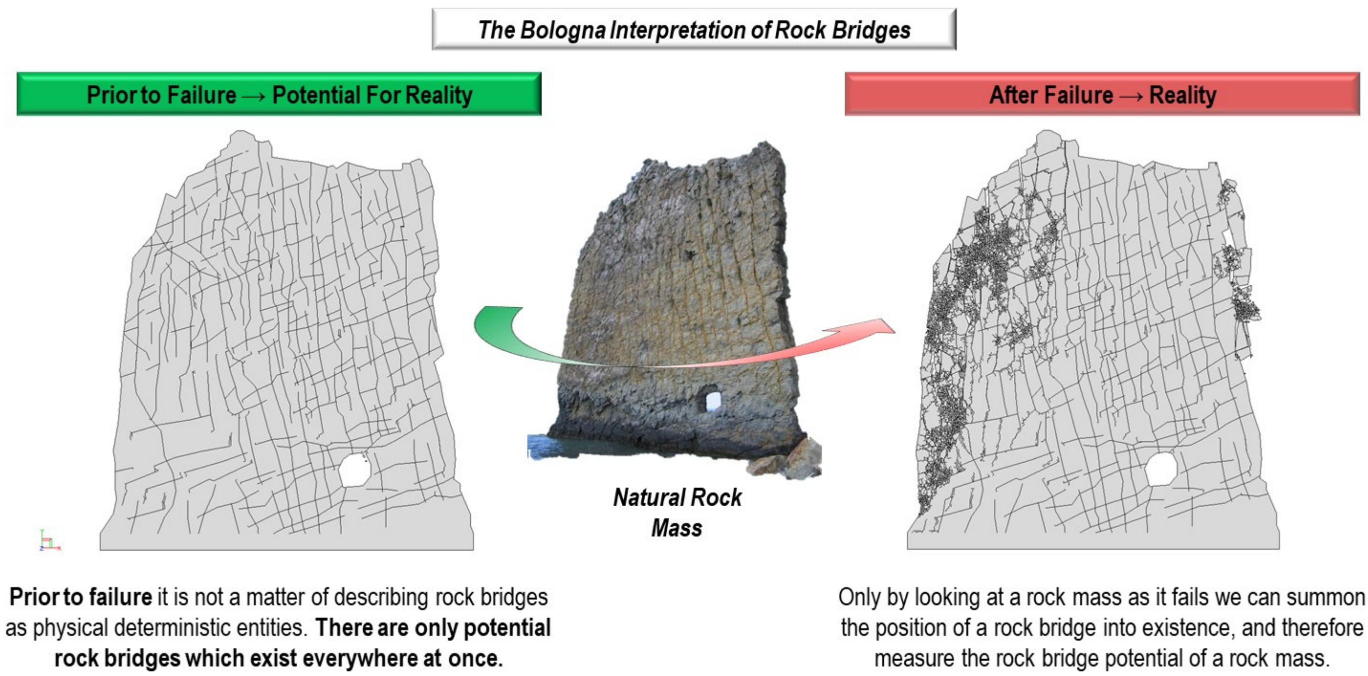

Figure 8. Illustration of the Bologna Interpretation of rock bridges, showing that rock bridges could only be defined and therefore measured post-failure.

The Bologna Interpretation does not negate the existence of rock bridges; on the contrary, it agrees with observations made by [4,20,21] which showed that rock bridges could only be observed, and their extent measured, post-failure. What rock bridges look like, how many there are, and where they are, we do not know; we can assume their existence and account for their contribution to rock mass strength using a potential analogue.

The Concept of Rock Bridge Potential

The Bologna Interpretation is strongly associated with the concept of rock mass potential, which depends on a combination of different parameters, including the following:

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

During the failure process, the rock mass potential is transformed into kinematically controlled mechanisms, elastic deformation, and plastic yielding, which manifests in the form of intact rock fracturing and sliding and rotation of blocks. To define rock mass connectivity and degree of interlocking, [22,23] have developed a connectivity-based characterisation tool (NCI, network connectivity index). The NCI system combines information about fracture size (fracture intensity), fracture intersections, and the number of fractures per area or volume (fracture density). The definition of NCI for 2D and 3D problems are, respectively, as follows:

$$NCI_{2D} = \frac{P_{21}}{P_{20}} I_{20}\; I_{20} = \frac{X_{INT} + H\frac{X_T}{W} + H\frac{X_B}{W} + W\frac{X_R}{H} + W\frac{X_L}{H}}{\text{Area}} \tag{9}$$

$$NCI_{3D} = \frac{P_{32}}{P_{30}} I_{30} \tag{10}$$

In Equation (9), P_{21} represents the areal fracture intensity (ratio of the sum of fracture length to sampling area), P_{20} is the fracture density (number of fractures per sampling area), and I_{20} is the intersection density (number of intersections per sampling area), corrected to account for the shape of the 2D exposure (expressed as a width-to-height ratio, W: H) and the assumed relative loading direction. In Equation (10), P_{32} represents the volumetric fracture intensity (ratio of the sum of fracture area to sampling volume), P_{30} is the volumetric fracture density (number of fractures per sampling volume), and I_{30} is the volumetric intersection density (number of intersections per sampling volume). NCI_{2D} and NCI_{3D} are measurements of connectivity intensity; therefore, Equation (10) has been modified from the initial definition introduced in [1] to correct the resulting dimensions of the NCI parameter (m^{-1}). Currently, NCI is limited to jointed rock masses (open structural features). While the presence of veins may give the impression of an apparently blocky and fully connected system, the author recognizes the challenge of using traditional classification methods to group veined rock masses into conventional massive, blocky, very blocky, etc., categories.

Notwithstanding, it is possible to take advantage of numerical simulations of fracturing processes to define a rock bridge potential [23] defined as the ratio of induced fracturing (NCI_{rb}) to the summation of natural (NCI) and induced fracturing (NCI_{rb}):

$$\text{Rock Bridge Potential} = \frac{NCI_{rb}}{NCI + NCI_{rb}} \tag{11}$$

In the literature, rock bridge strength is often considered independent of rock mass strength. The rock bridge potential introduced in Equation (11) breaks away from this notion and demonstrates that rock bridge strength is just a manifestation of rock mass strength. Indeed, the rock bridge potential describes whether stress-induced phenomena primarily control rock mass behaviour (e.g., spalling) or vice versa, and failure occurs due to structurally controlled mechanisms or as a combination of both. In [1], the authors used the results of several synthetic rock mass models to show the relationship between rock bridge potential with the excavation behaviour matrix by [8], which differentiates potential rock mass behaviour based on rock mass structures (defined by the Geological Strength Index—GSI—[24]) and induced stress relative to crack initiation threshold. This is illustrated in Figure 9. The boundaries between the different excavation behaviours are represented by white dashed lines to remind engineers that we cannot truly quantify rock mass behaviour using qualitative schemes such as GSI [25]. Furthermore, the rock mass connectivity and degree of interlocking are stochastic; therefore, assigning a unique NCI to a rock mass is not meaningful. In [26], the author carried out an extensive sensitivity analysis to study the impact of the parameters used to calculate NCI, and he recommended using NCI ± 1 to constrain NCI for a given rock exposure.

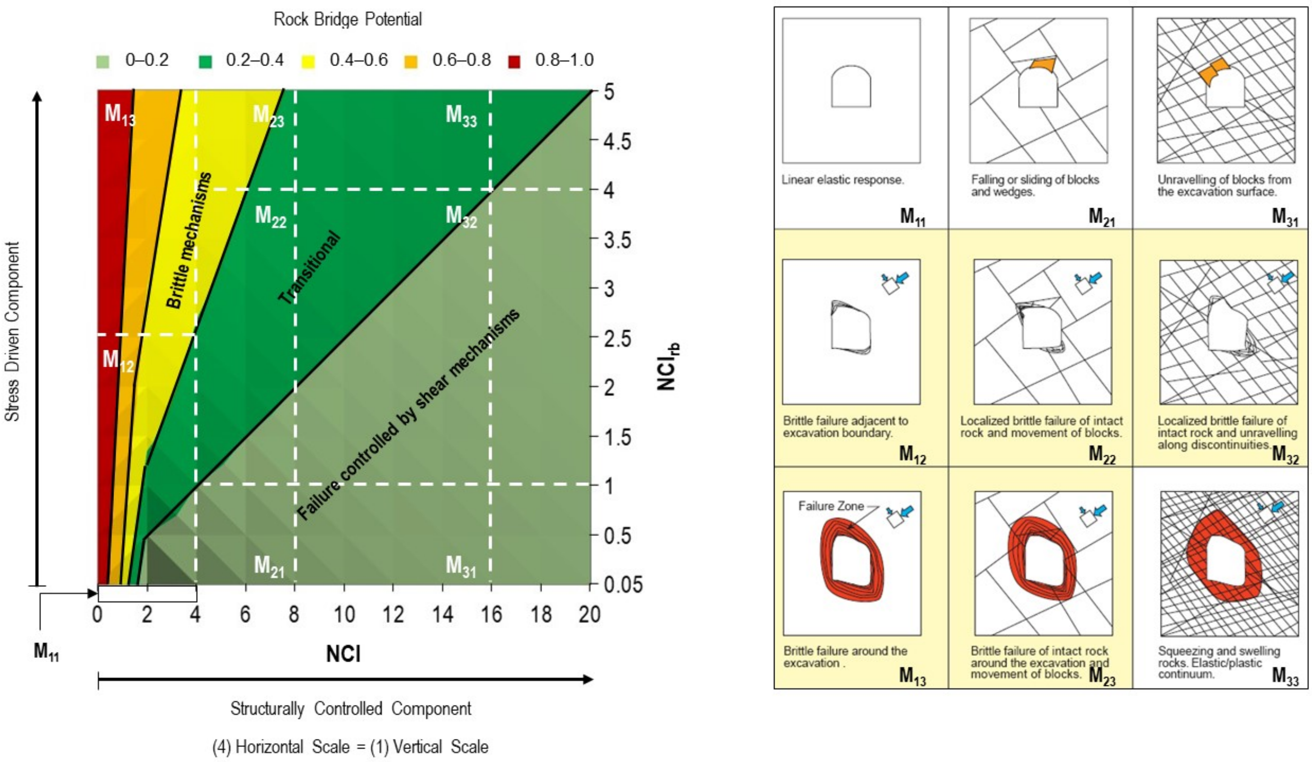

Figure 9. Relationship between rock bridge potential (**left**) and the rock mass behaviour matrix by [8] (**right**).

Using modelling results from [23], Figure 10 shows how we could use NCI and NCI_{rb} to determine the rock mass potential for two direct shear test models, defined by the proportional contribution of stress-driven failure (represented by NCI_{rb}) and structurally controlled failure (represented by NCI). Models A1 and A2 behave very differently, and almost oppositely, despite having comparable NCI values before failure.

The concept of rock bridge potential is not limited to slope problems, and it can be used to characterize the failure mechanisms for a broader set of problems. Figure 11 shows three pillar models (after [27]). P1 and P2 have the same initial NCI, but the fracture network has been rotated 40 degrees counter-clockwise. Pillar P3 represents the same network as Pillar P2 but with more extended fractures. The calculated NCI_{rb} is 1.3, 0.4, and 0.1 for models P1, P2, and P3, respectively. The results shown in Figures 10 and 11 demonstrate that it is impossible to assign unique descriptions of rock mass behaviour solely based on the structural characteristics of a rock mass, defined using, for example, GSI, volumetric fracture intensity (P_{32}), volumetric fracture count, and NCI.

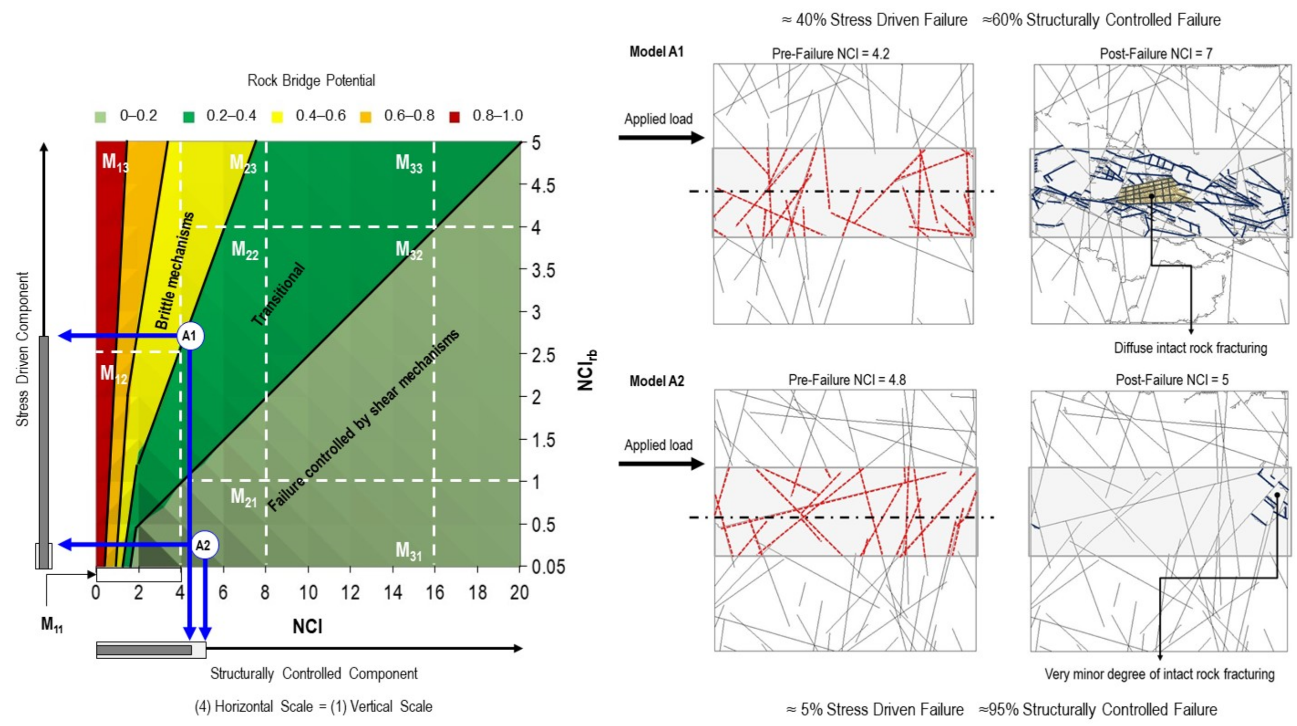

Figure 10. Example of using the concept of rock bridge potential to characterise the rock mass behaviour, defined by the proportional contribution of stress-driven failure (represented by NCIrb) and structurally controlled failure (represented by NCI). Direct shear tests models from [23]. Percentages of rock mass behaviour are determined using the relative lengths across the horizontal and vertical scale (corrected for scale).

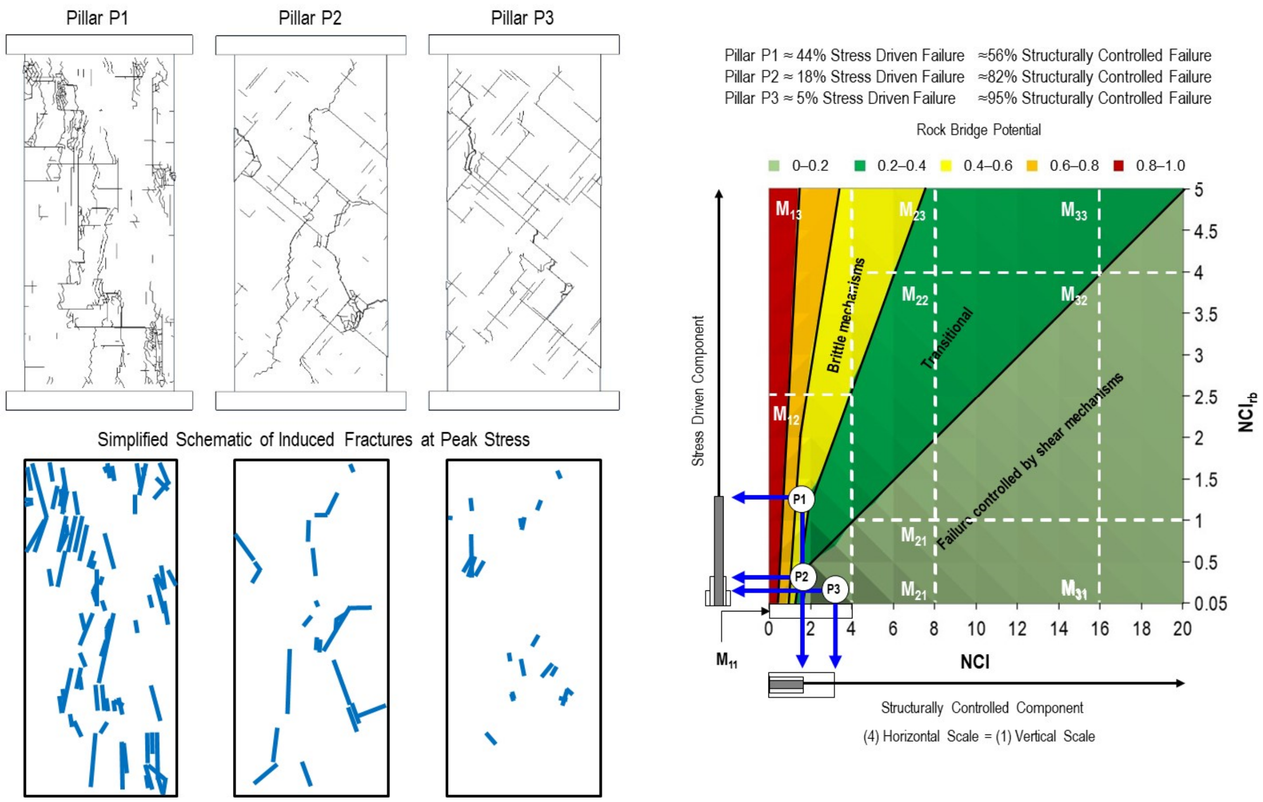

Figure 11. Concept of rock bridge potential applied to study the behaviour of jointed pillars (8 m high, 4 m wide). Modelling results were published in [27].

Two important corollaries follow from the discussion above:

- The degree of interlocking of a rock mass is an emerging property that cannot be quantitatively measured or qualitatively assessed independently of the applied loading conditions (magnitude and directions) and degree of rock mass connectivity.
- Pure structural controlled failure only occurs when NCI_{rb} is zero. Therefore, it confirms our hypothesis that, before failure, it is not a matter of describing rock bridges as deterministic physical entities. There are only potential rock bridges which exist everywhere at once, and their impact (defined by NCI_{rb}) can only be measured post-failure.

While the structure of Equation (11) may appear similar to that of the continuity factor defined by [3], the rock bridge potential and the Bologna Interpretation acknowledge that we cannot measure rock bridges pre-failure. The design process should therefore focus on the numerical modelling of fractured rock masses using discrete element simulations (e.g., [23,28]). We understand that the concept of rock bridge potential implies adopting potentially complex analysis. Still, we argue that seeking easy and faster alternatives (e.g., limit equilibrium methods) could lead to misinterpreting the risk of failure.

Compared to rock bridge percentage and Equations (6) and (8), the rock bridge potential approach accounts for scale effects explicitly. To calculate the rock bridge potential, one has to know the parameter NCI_{rb}. To determine $NCI_{rb,}$ one has to carry out synthetic rock mass modelling (SRM models, i.e., discrete modelling in which fracturing is simulated explicitly) for representative rock mass volumes at a scale typically larger than 10 m^3 (10 m^2 for 2D models). Because of their size, SRM models meet the criteria of representative elementary volumes [27]. Figure 12 shows the relationship between rock bridge potential and normalized strength. The results include eight synthetic direct shear models (rock mass scale) described in [23] and the three pillar models described above. The normalised strength is taken as the ratio of the modelled rock mass cohesion to the intact rock cohesion and the ratio of the modelled uniaxial rock mass strength to the uniaxial intact rock strength for the synthetic direct shear models and pillar models, respectively. Even when considering a rock bridge potential of 1.0, the resulting normalised strength would not exceed 0.65 and 0.55 for the synthetic direct shear and pillar models, respectively.

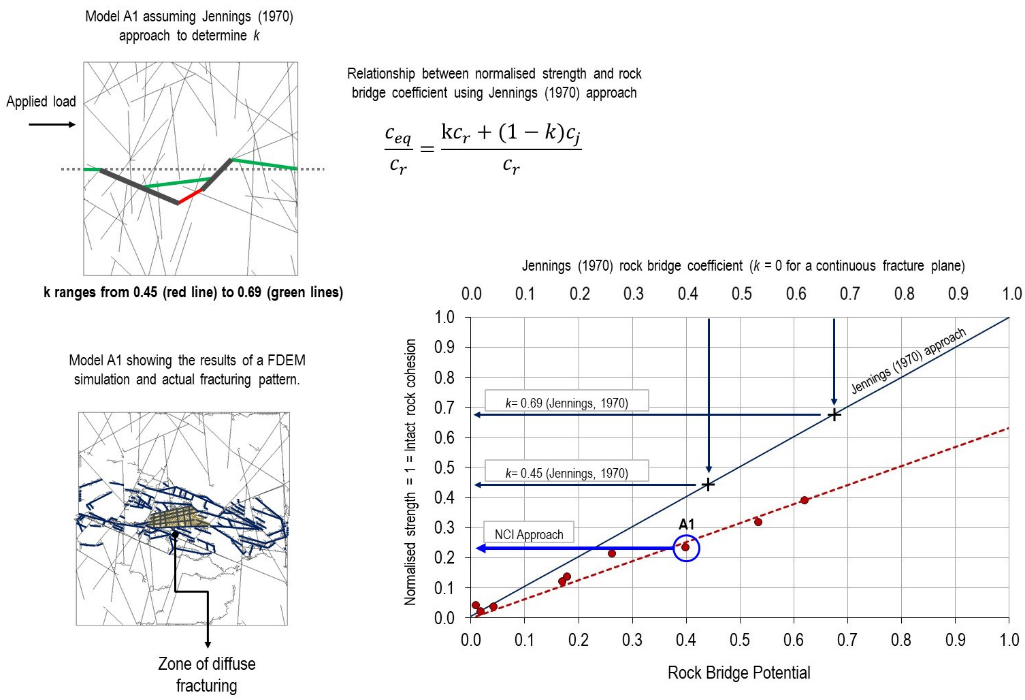

Figure 12. Normalised strength as a function of the calculated rock bridge potential for the direct shear test models is shown in Figure 10 (material properties from [23]) and Figure 11 (using material properties listed in [27]).

Using one of the synthetic direct shear models by [23] as a reference (Model A1 shown in Figure 10), it is possible to draw a comparison between the normalised strength calculated using the concept of rock bridge percentage and intact rock strength values, and the normalized strength corresponding to the modelled value of rock bridge potential, which is automatically corrected for scale effects (Figure 13). The results show the critical problem with the rock bridge percentage approach. In the presence of 100% rock bridges, the mobilised strength would be equivalent to intact rock strength. Therefore, using intact rock cohesion as an input parameter in Equation (6) could lead to non-conservative assumptions.

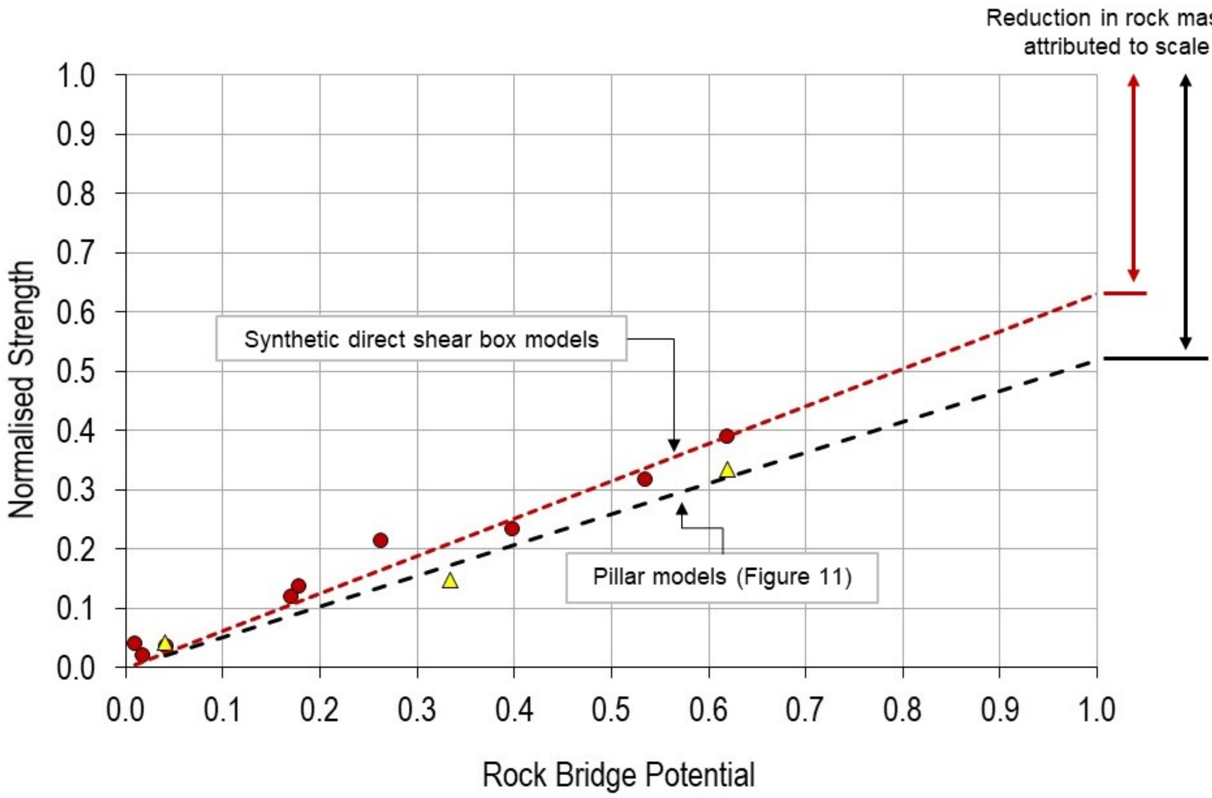

Figure 13. Comparison between the normalised strength calculated using the concept of rock bridge percentage and the normalized strength corresponding to the modelled value of rock bridge potential. The results show the critical problem with the rock bridge percentage approach and the risk of using non-conservative assumptions when adopting the method proposed by [3,5,6].

In [19], the authors suggested a modified version of Equation (6), scaling the value of intact cohesion for every identified rock bridge. However, we agree with Terzaghi (1962) that Equation (12) below would only have a purely theoretical value, since there remains the problem that we need to identify rock bridges before failure. Therefore, we cannot measure the scaling factors required for using Equation (12).

$$c_{eq} = [(RB_1)c_{1,\, scaled} + \ldots + (RB_n)c_{n,\, scaled}] + Kc_j \tag{12}$$

4. The Bologna Interpretation of Rock Bridges and the Problem of In-Plane Rock Bridges

Examples of rock slopes whose stability is controlled by the inferred presence of intact rock bridges are discussed in [28–31]. We have used the term inferred, since, according to the Bologna Interpretation, rock bridges' location, geometry, and intensity can only be fully defined post-failure. There is a geological exception to this interpretation. Indeed, [32] used thermography to remotely detect intact in-plane rock bridges for the surfaces of granitic exfoliation sheets at two locations on the El Capitan in Yosemite Valley (CA, USA). Nonetheless, we argue that the work by [32] confirms the Bologna Interpretation since it shows that we would only be able to demonstrate the measurements if the two rock blocks under consideration were to fail. Furthermore, the approach appears to be limited to specific geological conditions (sheeting joints), and it has yet to be tested for more complex geological conditions, as those shown in Figures 14 and 15 below. We know rock bridges must exist for Parus Rock (Figure 14) and Berry Head Arch (Figure 15) to remain standing. However, we cannot physically observe them. Therefore, we can neither define nor measure a rock bridge percentage for these structures (except for the site shown in Figure 15, where failure has already occurred).

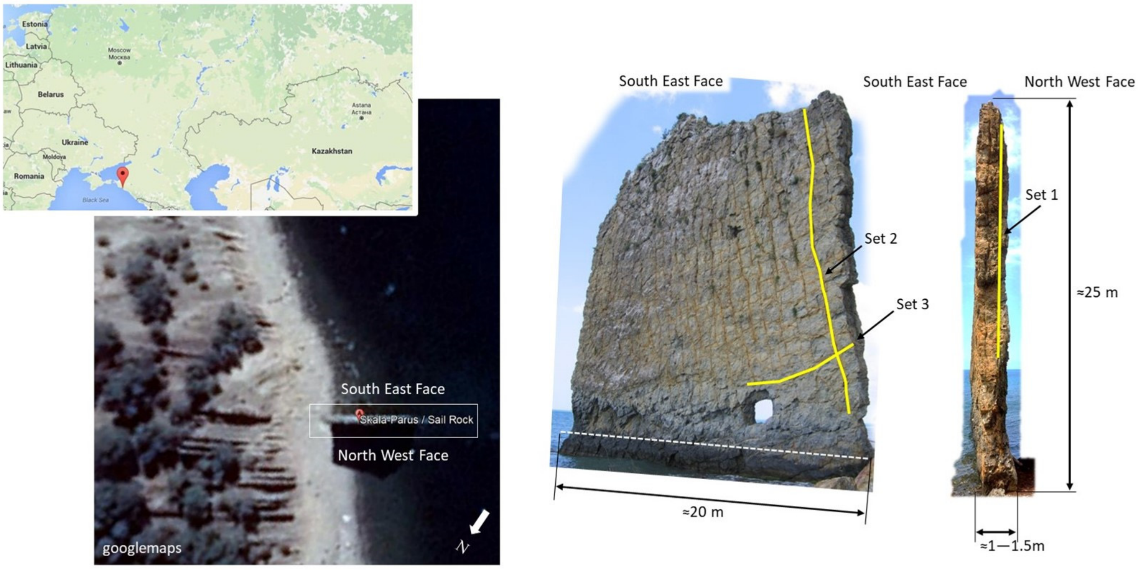

Figure 14. Parus Rock. Data from GoogleMaps and photos shared under a creative common license CC2.5.

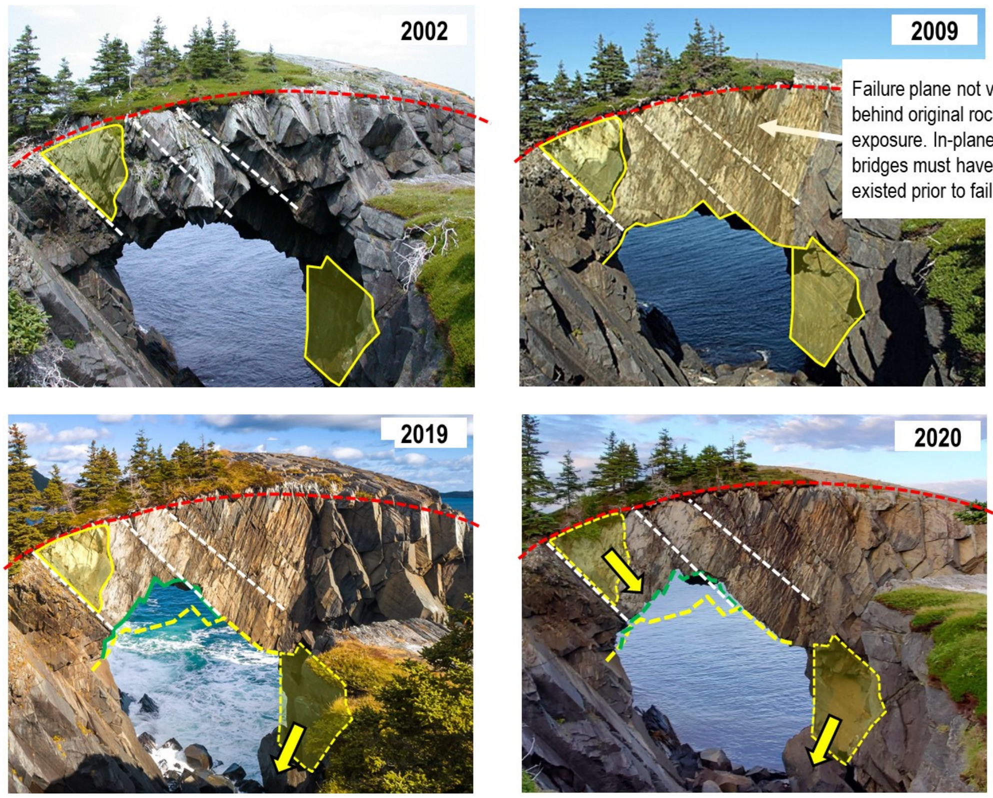

Figure 15. Evolution of Berry Head Arch (Newfoundland, Canada) from 2002 to 2020. Photos sourced from Google Images under a creative common license CC2.5.

Parus rock (also known as sail rock, Krasnodar Krai, Gelendzhik, Russian Federation) is an isolated remnant of one of the sandstone butters or plateaus that eroded along parallel cracks (sandstone layers are approximately 1 to 1.5 m thick). The dimensions of Parus rock, about 20 m long and 25 m high, are impressive considering its limited width. The geology of the area includes vertical layers of alternating clay shales and fine-grained yellow-grey dense sandstone of the late Cretaceous age. Close-up analysis of photos shows that Parus Rock is transected by two parallel, albeit not continuous, planes striking SW-NE (herein referred to as Set 1). Two conjugate Sets are visible on the SE and NW sides. Set 2 generally dips towards NE at approximately 70 to 80 degrees. Set 3 dips towards SW at about 20 to 30 degrees, although the orientation of Set 3 on the NW face appears to be inverted to NE (20 to 30 degrees dip angle). The different joint Sets contribute to a pretty blocky appearance, further highlighting the incredible equilibrium of this geological structure.

The Berry Head Arch (Newfoundland, Canada) is a monolithic arch made of sedimentary rocks. The arch's opening is approximately 18 m high (from sea level). The composite photos in Figure 15 show the evolution of Berry Head Arch over a period of 18 years. The arch's thickness has been gradually reduced over the years (compare images between 2002 and 2020). A sub-vertical joint set striking longitudinally is apparent from the photo dated 2009, with another set dip at approximately 45 degrees striking across the arch deck.

Parus Rock and Berry Head Arch are good examples of the role that in-plane rock bridges must play in the stability of natural structures. As discussed earlier in Section 2, many researchers who have investigated the problem of rock bridges and step-path failure have treated rock bridges as geometrical entities between intermittent joints. The classical step-path analysis ignores the presence of in-plane rock bridges. We argue that assuming full in-plane fracture continuity on the premise of adopting a conservative approach to design could lead to misinterpreting mechanisms and stability performance by focusing attention on other strength parameters. Indeed, we would be unable to simulate Parus Rock and Berry Head Arch's mechanical behaviour if we ignored in-plane rock bridges.

Using data from [1], Figure 16 shows the results of a simple modelling experiment. The results demonstrate that assuming zero tensile strength for the predefined fracture network representing the South-East face of Parus rock would yield a non-stable rock mass, even when using stronger properties for the intact rock matrix. The model shows that a nominal tensile strength (0.05 MPa) is required for the in-plane rock bridges to simulate stable conditions. The problem would still exist if considering a 3D model, since photographic evidence suggests the two failing blocks shown in Figure 16c externally appear to be fully formed. Therefore, non-visible in-plane rock bridges must exist for the blocks to be stable. In line with the principles defined in [1], the objective of the numerical experiment presented in Figure 16 is not to back analyse and determine accurate material properties but to explain the governing failure mechanisms and their association with the concept of in-plane rock bridges. These preliminary results should not be used as a basis for making decisions concerning engineering design or any policy relating to access to Parus Rock.

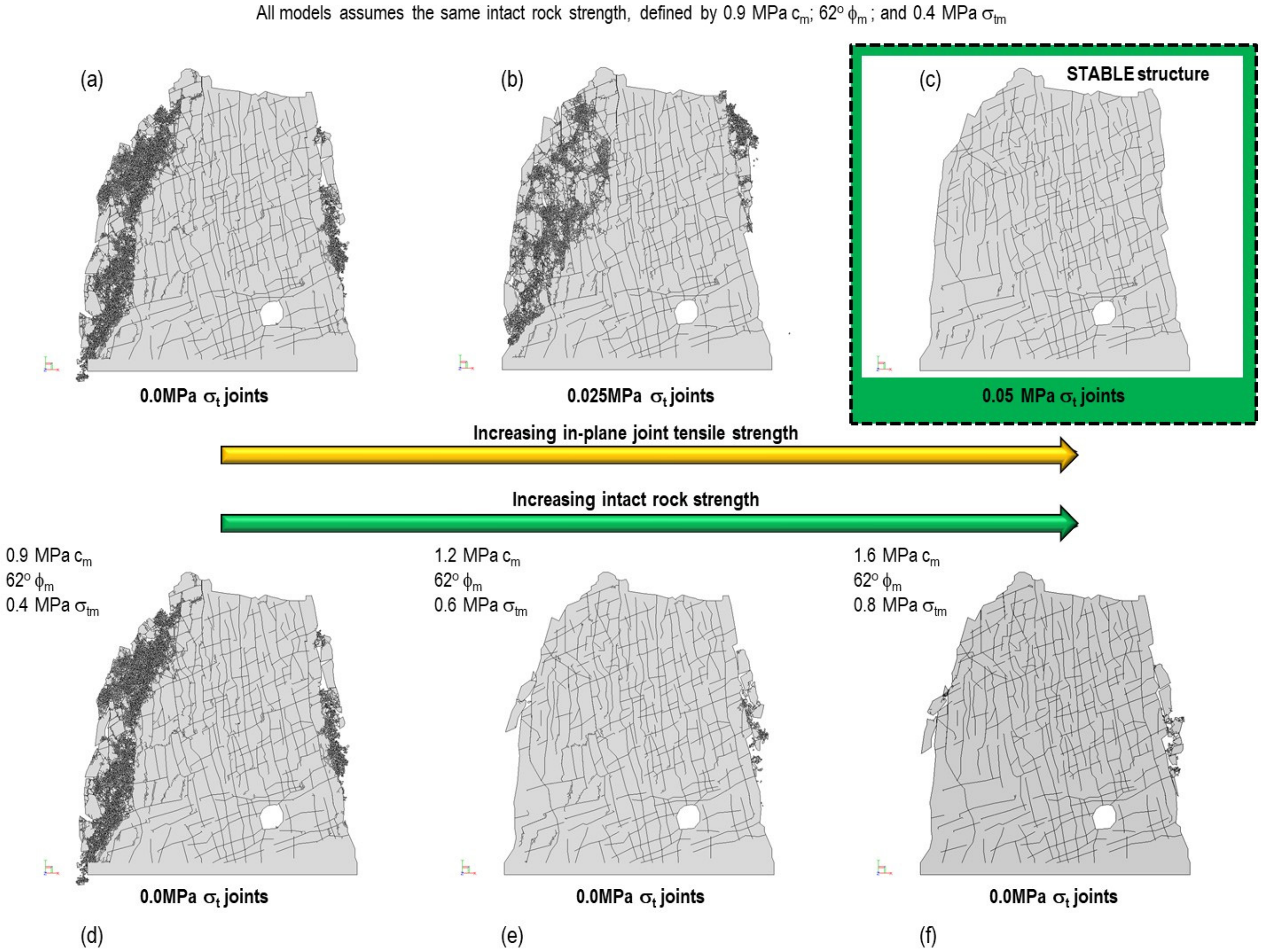

Figure 16. Simple modelling experiment using data from [1] that demonstrates the importance of determining accurate strength values for in-plane rock bridges. Models (**a**–**c**) have the same intact rock strength but increasing tensile strength for the fracture traces. Models (**d**–**f**) show that increasing intact rock strength under the assumption of 0 MPa tensile for the fracture traces is not sufficient to yield stable conditions for the rock mass.

5. Conclusions

There is no doubt that rock bridges exist, but their existence and definition are conditional to the specific rock engineering problem and failure mechanism being considered. Our job as engineers is to use what we see to represent what we do not. This critical problem is ignored by many engineers, who continue to refer to rock bridges as geometrical distances between non-persistent fractures. This rather simplistic approach has many limitations and can lead to non-realistic failure mechanisms. We have presented several arguments to demonstrate that the concept of rock bridges in the literature needs to be revised. We also raised the critical question of whether using strength equations based on measuring a rock bridge percentage could lead to overestimating the assumed (equivalent) rock mass properties.

To address the concerns about the rock bridge percentage, we have introduced a new interpretation of rock bridges (Bologna Interpretation) and presented applications of rock bridge potential. The proposed Bologna Interpretation of rock bridges highlights the challenges posed by uncertainties related to the knowledge of both the intensity and the location of rock bridges before failure. Because rock bridges would only come into existence upon failure, in principle, it becomes impossible to truly calibrate and (validate) the results of a continuum-based forward analysis because of the impossibility of accurately determining a key input parameter for equivalent rock mass strength (rock bridge intensity). The design process should therefore focus on the numerical modelling of fractured rock masses using discrete element models and fracture mechanics principles. More attention should also be given to better understanding damage-related processes, including time-dependent damage.

It is relatively easy to determine NCI_{3D} using 3D discrete fracture network (DFN) models. The challenge is to simulate rock mass behaviour in 3D using discrete modelling in which fracturing is simulated explicitly. There is a need to overcome issues concerning the following: (i) the meshing of complex fracture systems, (ii) the calibration of contact modelling properties between discrete elements (often mislabelled in the literature as micro-properties, thus ignoring the fact that contact properties are not scale-independent), (iii) long computational times, and (iv) the need to run multiple realisations to account for the stochastic nature of the embedded fracture system. Nonetheless, work is ongoing to study applications of rock bridge potential using 3D discrete geomechanical models. The work includes learning whether extending NCI to veined rock masses is possible.

Finally, any stability analysis (whether performed in 2D or 3D) that addresses the problem of rock bridges must account for in-plane rock bridges. This critical remark could be extended to any natural or engineered structure. While adopting a conservative approach is understandable, we should question the practice of ignoring what natural structures can teach us regarding bridge characterization.

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