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Analysis of foundation problems using discontinuum and equivalent continuum approaches with embedded discrete fractures

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ABSTRACT: Numerical methods and computing techniques are now integrated components in rock mechanics and rock engineering design, providing an opportunity to increase our fundamental understanding of the factors governing rock mass behavior. It is increasingly evident that models of rock mass behavior should incorporate realistic representation of fracture networks as well as should constitute an effective aid for the evaluation of scale-effects for those engineering problem where performing field tests at different scales is not technically or economically viable. This paper provides a discussion on proposed theoretical approach broadly adopted to study the stability of slopes that include intermittent joints. Limitations of the approach are demonstrated by showing the results of numerical analyses carried out with discontinuum (ELFEN) and continuum (PLAXIS) codes applied to the study of conceptual slope and foundation problems in fractured rock masses. The paper highlights the importance in rock engineering design of applying numerical modeling for rock bridge related problems, and emphasis is given to methods to account for rock bridge strength at the desired engineering scale.

1. INTRODUCTION

The importance of scale effects in rock engineering design is well recognized. It is possible to directly study scale-effects associated with randomly distributed flaws in an otherwise intact rock specimen at the laboratory scale. For instance, Bieniawski and Van Heerden (1975) indicated that the unconfined compressive strength of different rock materials such as iron ore, quartz diorite and coal decreases with size, reaching constant values for samples of approximately 1.5 m edge length. This behaviour was initially explained in terms of the larger sample containing more flaws in so-called critical locations (Goodman, 1980). Cundall (2008) summarised the effect of size on strength by referring to studies by Bažant and Chen (1997) and mentioning examples of basic theories of scaling that may be responsible for the observed size effects.

However, the problem of scale-effects become quite a complex problem when dealing with larger rock mass volumes containing natural discontinuities, since it is not possible (or at least not economical) to perform field tests at different scales. In this context, numerical models provide a useful alternative to test the variation of rock mass strength with increasing sample size, as demonstrated by Elmo (2012) and Elmo et al. (2016) for compressive, indirect tension and shear loading conditions. However, numerical models may introduce an indirect form of scale effects due to the simplification required when modelling fractured rock masses, independently of whether a continuum or discontinuum modelling approach is used.

The simplification process has a significant impact when considering rock engineering problems that rely on estimates of intact rock bridges, i.e. the size of the intact rock portion in between natural fractures which, if failed, may contribute to the formation of a continuous failure surface, Figure 1. Jennings (1970) defined the overall failure surface as an Equivalent Discontinuity and provided a measurement of its continuity (K) as a function of fracture persistence, with K is equal to 1 for a fully continuous surface.

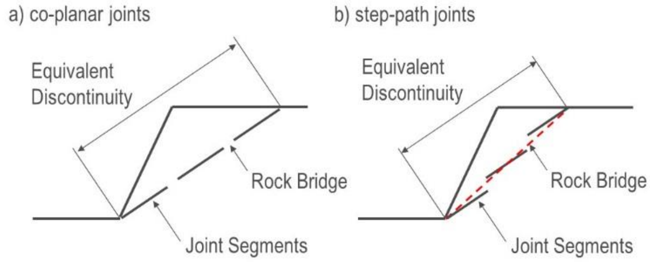


Fig. 1: Example of co-planar and step-path rock bridge problems.

The coefficient K is used to estimate equivalent cohesion, (c_{eq}) and friction angles (ϕ_{eq}) of the Equivalent Discontinuity (Equations [2] and [3] below, in which c and ϕ are the cohesion and friction of the intact rock, c_j and ϕ_j are the cohesion and friction of the joint surface).

$$c_{eq} = (1-K)c + Kc_j \quad [1]$$

$$Tan\phi_{eq} = (1-K)Tan\phi + K\, Tan\phi_j \quad [2]$$

Note that In Jennings (1970), Equations [1] and [2] were developed on the hypothesis that the intermittent joints are co-planar. Nonetheless, researchers and engineers have continued over the years to apply Equations [1] and [2] to the study of slope stability and step-path problems despite the fact those Equations pertain to a very simple case, which is seldom encountered in the field (co-planarity of joint surfaces). Furthermore, the approach neglects two fundamental scale effects problems:

- Type 1. The same measurement of rock bridges (1-K) could be obtained for different problems. In other words, according to Equation [1] and [2], a single (larger) rock bridge and many (smaller) rock bridges may yield the same equivalent cohesion and friction angles if the total length of rock bridges is the same for the two problems (Equivalent K).
- Type 2. Because the fracture intensity parameter would determine what portion of the naturally occurring fractures could be explicitly modelled, any simplification process would necessarily introduce some form of implicit rock bridges, the size of which would depend on the model resolution or size of the representative elementary volume (REV) for the rock mass.

As shown in Elmo et al. (2018), the definition of K can be modified to include the height of the slope H and the dip of the joint segments. Therefore, the strength of a rock bridge would be a function of its size and whether the load applied is mostly compressive, shear, tensile, or a combination of those. For the Type 1 problem, it is reasonable to assume that larger intact rock bridges in the field would contain micro flaws, therefore he same basic scale effects theories (e.g. Bažant and Chen, 1997) could be applied to determine the intact cohesion and friction angle of each specific rock bridge. Those parameters would also change with the location of the rock bridge, since the degree of confinement would be different.

Type 2 problems are relatively more complex since there would be the need to scale up rock mass properties rather than intact rock properties. Using synthetic shear box texts at 2, 5, 10 and 20 m scale, Elmo et al. (2012) demonstrated that synthetic rock mass models could be used to obtain equivalent GSI ratings scaled to the problem under consideration. More recently, Fadakar and Elmo (2018) have developed a new method to address Type 2 problems and quantify the effect of rock bridges in terms of rock mass quality using Graph Theory.

This paper use discontinuum and continuum codes to: i) demonstrate the role that scale effects play in Type 1 problems; and ii) to discuss the limitations of current step-path analyses. To do so, failure of rock bridges is simulated by considering a slope problem (55 m high slope) with intermittent joints, and a foundation load applied to the crest of the slope to generate enough induced stresses to overcome intact rock strength since gravitational induced stresses alone within the slope would not be sufficiently large to overcome intact rock strength.

2. NUMERICAL ANALYSIS OF STEP-PATH AND FOUNDATION STABILITY PROBLEMS

Rock masses are typical inhomogeneous and discontinuous media. The strength and deformability of a rock mass can be viewed as a combination of the mechanical properties of the intact rock material and weaknesses in the form of discontinuities; the associated reduction in strength and deformability is ultimately dependent on the frequency and mechanical properties of the discontinuities. Although the rock mass strength can be much less than that of the intact rock, load bearing capacity of rock masses is typically greater than that of soil material. This paper does not comprise a review on settlement and bearing capacity failure of foundations in rock; principal literature sources on the subject include, amongst the others, Goodman (1980), Kulhawy and Goodman (1980), Wyllie (1992), Serrano and Olalla (1996), Merifield et al. (2006) and Prakoso and Kulhawy (2006). The presence of a single discontinuity oriented in a given direction may critically affect the apparently favorable stability of a structure founded on rock. Intersecting discontinuities can also form blocks whose movement can ultimately result in the failure of the foundation, hence jeopardizing the stability of the entire structure. In particular circumstances, however, the stability analysis should be extended to include the strength of the intact rock. Whereas the rock strength may be sufficient to support the load imposed onto it by the structure, fracturing of the intact rock may favour rock bridging between discontinuities, with the creation of rock blocks/wedges, which in turn may become unstable.

2.1. Model Set Up

The model consisted of three different geometries, as shown in Figure 2. Intact rock and joint material properties (Table 1) were consistent with those used in rock bridge analyses by Vyazmensky et al. (2009), whilst geometrical parameters were consistent with those used in step-path simulation in rock slopes by Yan et al. (2007). A foundation load of 3 MN/m (slightly extending past the intersection of the top joint segment with the slope crest) was gradually applied in the model after the equilibrium stage.

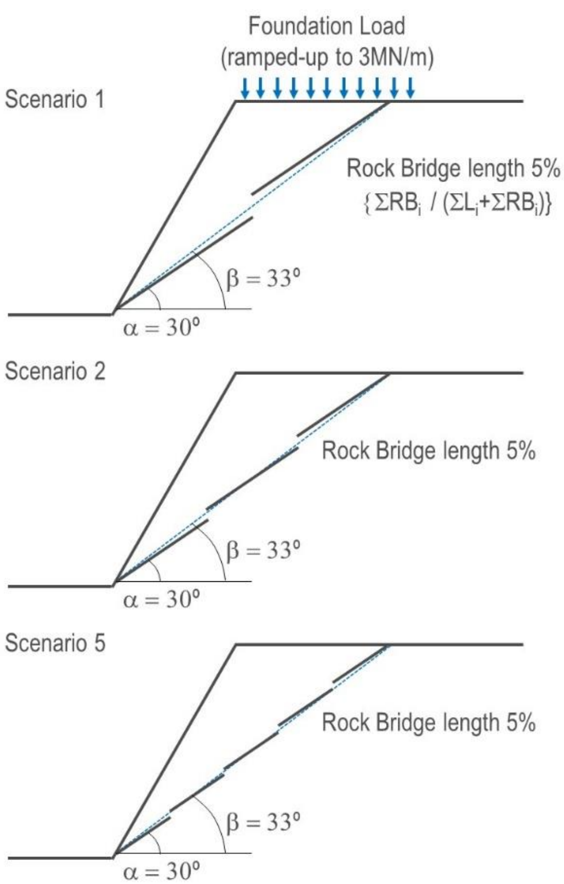
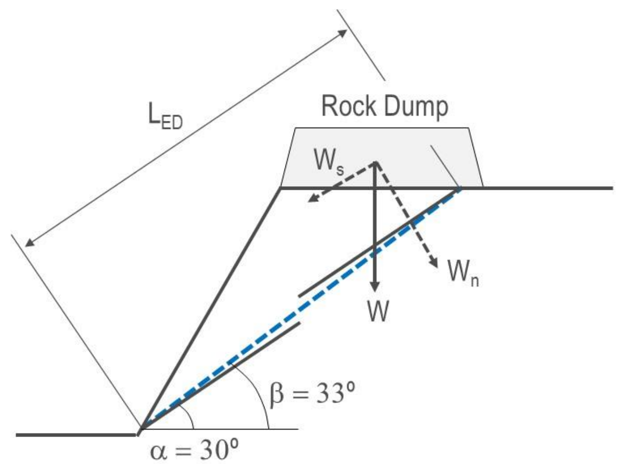


Fig. 2: Model geometries and loading conditions.

Table 1. Material Properties (After Vyazmensky et al., 2009)

Property	**Unit**	**Value**
Fracture energy, G_f	J m^{-2}	60
Tensile strength, σ_t	MPa	10
Young's modulus, E	GPa	60
Poisson's ratio, ν	-	0.25
Density, ρ	kN/m^3	26
Internal cohesion, c_i	MPa	20
Internal friction, ϕ_i	Degrees (°)	50
Fracture cohesion, c_f	MPa	0
Fracture friction, ϕ_f	Degrees (°)	30
Normal stiffness, k_n	GPa/m	5
Tangential stiffness, k_t	GPa/m	0.5

The analysis focused on simple fracture geometries and a scenario in which the Factor of safety (FoS) of the slope would still be greater than 1 even after failure of all rock bridges. This scenario was chosen as to introduce controlled conditions that could be easily back-analysed using simple limit equilibrium equations and approximating the applied foundation load to an overburden weight, Figure 3.



$$FoS = \frac{c_{eq}L_{ED} + W_n cos\beta Tan\phi}{W_n sin\beta}$$

Fig. 3: Simplification considered in the analysis to consider the problem from a limit equilibrium perspective and calculating a factor of safety for the slope.

For the geometries shown in Figure 2, it could be easily demonstrated that the length of the Equivalent Discontinuity (L_{ED}) and the forces involved (summation of the weight of the block and the foundation) would not change. Therefore, for friction angle less than 30 degrees, the cohesion required for the slope to yield a factor of safety (FoS) of 1 would be the same for all the 3 different scenarios, independently of the size of the rock bridge. Similarly, since the rock bridge percentage does not change, the same c_{eq} and ϕ_{eq} would apply to all modelled scenarios, even if in the presence of 1, 2 and 5 rock bridges. These points raise some important limitations of both using a limit equilibrium approach to study the stability of a slope that includes intermittent joints and using equivalent strength parameters that do not account for the actual size of the rock bridges. The authors argue that the number, location and size of rock bridges should not be described using a single scale independent parameter like the term K in Equation [1] and [2].

Those limitations above could be further demonstrated using numerical analysis, as discussed in the following Section.

2.2. FEM-DEM Approach to Analyze Rock Bridge Problems

The hybrid Finite Element/Discrete Element FEM-DEM code ELFEN (Rockfield, 2017) has been successfully used for step-path simulation in rock slopes by Stead et al. (2004), Eberhardt et al. (2004) and Yan et al. (2007). The numerical capability of the code ELFEN to simulate fracture initiation, extension and coalescence was applied to the case where a rock-bridge exists in between pre-inserted discontinuities. The code ELFEN can simulate crack formation under tensile (i.e. Mode I) conditions. A Rankine rotating crack material model is implemented in the code and fracturing is controlled by tensile strength and fracture energy parameters. A new fracture is introduced when a limiting tensile stress is reached. For tension/compression stress states, the Rankine model is complemented with a capped Mohr-Coulomb criterion in which the softening response is coupled to the tensile model. Fracturing due to dilation is accommodated by introducing an explicit coupling between the inelastic strain accrued by the Mohr-Coulomb yield surface and anisotropic degradation of mutually orthogonal tensile yield surfaces of the Rankine rotating crack model. Detailed descriptions of these constitutive material models can be found in Klerck (2000), Klerck et al. (2004) and Owen et al. (2004).

2.3. FEM Approach to Analyze Rock Bridge Problems

As previously described, the purpose of this study is to show how the FEM and DEM could be used for the assessment of step-path failure for slope stability problems. The FEM numerical analyses are carried out using the code PLAXIS (Plaxis, 2017). The input for the geometrical conditions is based on the description provided in Section 2.1, and 3 scenarios (one, two and five rock bridges) are developed for intermittent joints dipping at 30°

The model is defined by "clusters", areas fully enclosed by lines, in which the intact rock and joint material properties consistent with the material parameters (Table 1) as well as the initial state of stress conditions. As far as the discontinuities are concerned, they are modeled by using interface elements available in PLAXIS (van Langen and Vermeer, 1991), which simulate the behavior of a thin intensely shearing material.

The geometry is thus populated with a 2D finite element mesh, using 15-node triangular elements, refined along the pre-inserted discontinuities and in correspondence of the insertion of the rock bridges in between the discontinuities. The proper local refinement is obtained by specifying a local "*Coarseness factor*" for each selected geometry entity and by using the "*Enhanced mesh refinement*" option available in PLAXIS. The stability of the slope is then assessed by applying a foundation load on the crest of the slope, gradually increased up to a value that induce a loss of integrity of the rock bridges.

2.4. Modelling Results and Discussion

Figure 4 shows the results of the FEM-DEM simulations. The results are presented in terms of the maximum foundation load that cause failure of the rock bridges; additionally, Figure 4 presents the changes in the induced σ_1 within the rock bridge area (RB1 to RB5, from toe to crest) as the foundation loading is progressively increased. The rock bridges not necessarily fail at the same time and for the same applied stress level.

The results obtained in the FEM models are shown in Figure 5. Note that to evaluate the integrity of the rock bridges in PLAXIS, the principal stresses are investigated during the loading procedure. Failure of the intact rock bridges is triggered when a limiting tensile stress (tension cut-off) is reached and, in turn, a clear progressive damage zone is established in correspondence of the existing rock bridge. It is thus reasonably assumed that the rock bridge is completely damaged when it is inferred that the minimum principal stress σ_3 within the rock bridges is equal to the tensile strength and thus tensile yield is occurred - by plotting plastic points.

Tables 2 and 3 presents a summary of the loads at failure and induced σ_1 at failure for the FEM-DEM and FEM models, respectively.

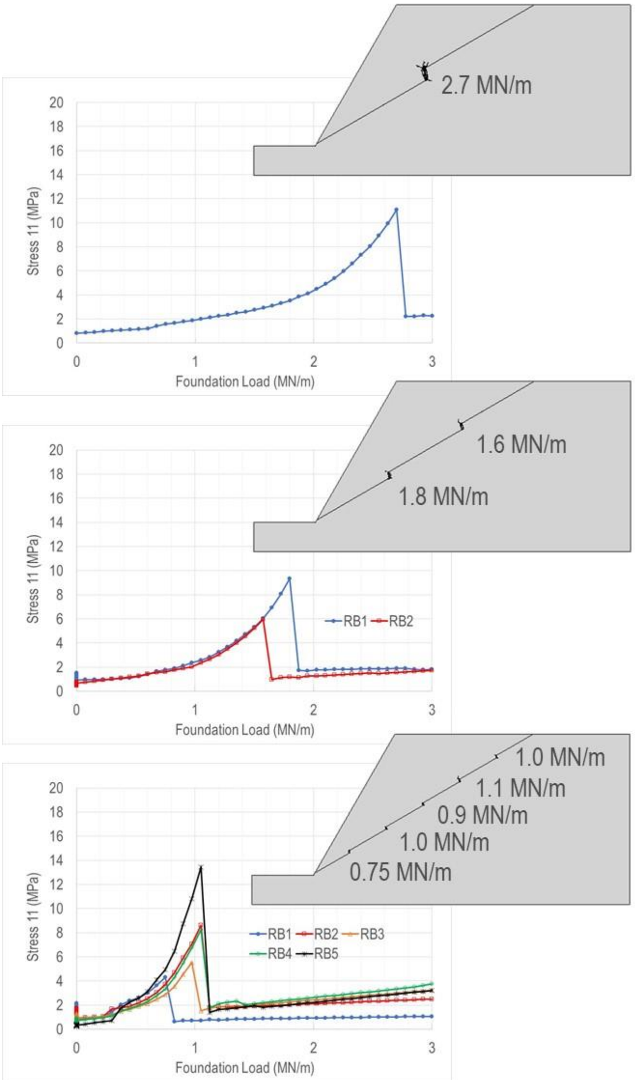


Fig. 4: Results for the FEM-DEM simulations; models with 1, 2 and 5 rock bridges, respectively.

Table 2. summary of the loads at failure and induced σ_1 at failure for the FEM-DEM (RB1 to RB5).

Model	Load (MN/m)	Average (MN/m)	σ_1 (MPa)
1 Rock Bridge	2.7	2.7	11.08
2 Rock Bridges	1.8	1.7	9.34
	1.6		5.96
5 Rock Bridges	0.75	0.95	4.31
	1.0		8.24
	0.9		5.51
	1.1		8.20
	1.0		13.41

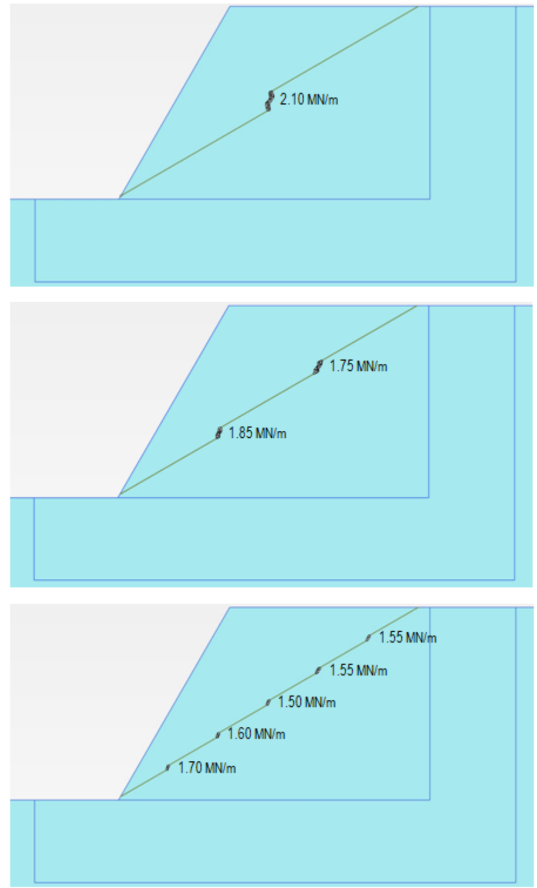


Fig. 5. Results for the FEM simulations; models with 1, 2 and 5 rock bridges, respectively

Table 3. summary of the loads at failure and induced σ_1 at failure for the FEM model (RB1 to RB5).

Model	Load (MN/m)	Average (MN/m)	σ_1 (MPa)
1 Rock Bridge	2.10	2.10	7.30
2 Rock Bridges	1.85	1.80	7.50
	1.75		14.50
5 Rock Bridges	1.70	1.60	8.80
	1.60		8.00
	1.50		15.30
	1.55		22.10
	1.55		27.50

The results indicate that the model with one single rock bridge (5 m long) would be able to withstand a much large foundation load than the model with five rock bridges (each one 1 m long). Interestingly, the relationships between the size of the rock bridge and the applied foundation load at failure would have an opposite trend to the ones provided by basic scale effects theories (e.g. Bažant and Chen, 1997). In the models, the largest rock bridge withstands the highest load, which contradict the wisdom that intact rock strength decreases with increasing sample size. However, the contradiction is only apparent, and the results could be explained considering the assumptions that are made in the models: the models do not contain (nor indirectly account for) micro-flaws in the rock bridge region. In other words, the rock bridge material in the models has no defects, thus basic rock scale effects theories would not apply.

This has clear implications for modeling of rock bridge problems. Additional research required to carry out a set of parametric analyses enabling a comprehensive comparison of the two numerical techniques with the final aim to establish guidelines for the numerical assessment of slope stability and step-path problems.

Differences between the results obtained with the two numerical methods are imputable to the intrinsic differences in the approaches of the two techniques. However, similar trend is obtained and this premises the good conditions for the definition of a proper reduction factor to be applied to FEM results through the comparison with FEM-DEM. The assessment of damaging rock bridges for many scenarios by means of FEM-DEM, capable to simulate crack formation and propagation under tensile conditions but time consuming, along with FEM ones would provide a sufficient amount of data for the reliable definition of reduction factors for FEM by preserving the proper degree of conservativeness obtained with FEM-DEM. This would thus allow the use of the more expeditious FEM analyses only for engineering practice (at least at preliminary stages of design).

3. CONCLUSIONS

In this paper the study of slope stability and step-path problems is assessed through different two numerical codes (discontinuum and continuum) which analyze a slope with embedded intermittent fractures dipping at 30° and a foundation load applied to the crest of the slope. Three different geometries are simulated in order to demonstrate the limitation of well-known equations (Jennings, 1970), commonly applied in research and engineering practice, for step-path failure problems.

Emphasis is placed on scale effects and to methods to account for rock bridge strength at the desired engineering scale. The approach proposed by Jennings (1970) neglects fundamental scale effects problems. Moreover, for this kind of engineering problems the evaluation of scale-effects through field tests at different scales is not technically or economically viable. The above consideration along with the results reported highlight the usage of numerical modeling for rock bridge related problems is paramount of importance in rock engineering design.

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