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DEM Modelling Of Rock Masses Affected By Permafrost Degradation

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Abstract **-** An increasing number of rockfalls and rockslides has been observed in high mountains during the last decades. Permafrost degradation, driven by global warming, is considered to be the main triggering factor.
This paper describes a multi-scale approach to the numerical modelling of potentially unstable rock slopes, by means of a Distinct Element model where the presence of ice in joints is taken into consideration.
Starting from previous research at the material scale (ice and frozen soil) and at the joint scale, which allowed to define an original failure criterion for continuous and discontinuous rock joints filled with ice, a large scale DEM model of a selected mountain slope (punta Gnifetti, Monte Rosa massif) is defined and used to reproduce the actual configuration through back-analysis based on geomechanical investigation, and to assess evolutionary scenarios driven by the increase of persistence and of temperature. Relevant numerical issues, regarding boundary conditions and model generation are preliminary discussed, as well.

Keywords**:** Rockmass - Joints - Persistence - Permafrost - Distinct Element Method – Shear strength reduction.

1. Introduction

The warming of permafrost in rock slopes reduces the shear resistance along rock joints by triggering a number of adverse factors, such as the decrease of ice resistance with temperature, water pressure increase and alteration of the creep properties of ice infillings. Moreover, repeated cycles of freezing-thawing are responsible of fracture propagation. In this context, it is equally important to investigate and describe the mechanical behaviour at the scale of materials and joints, and to set-up numerical models at the scale of the rock mass that account for the stress-displacement joint behaviour. Such multi-scale approach has been recently proposed by Wang [1] by using the Distinct Element Method.

1.1. Study case

In this paper, the described approach is applied to the evaluation of the stability of Punta Gnifetti (Signalkuppe) in the Monte Rosa group. In particular, at 4554 m asl Punta Gnifetti is the fourth highest peak in the range. More interestingly, Capanna Margherita hut, the highest one in the Alps, is located just on the top of Punta Gnifetti (Fig. 1a).

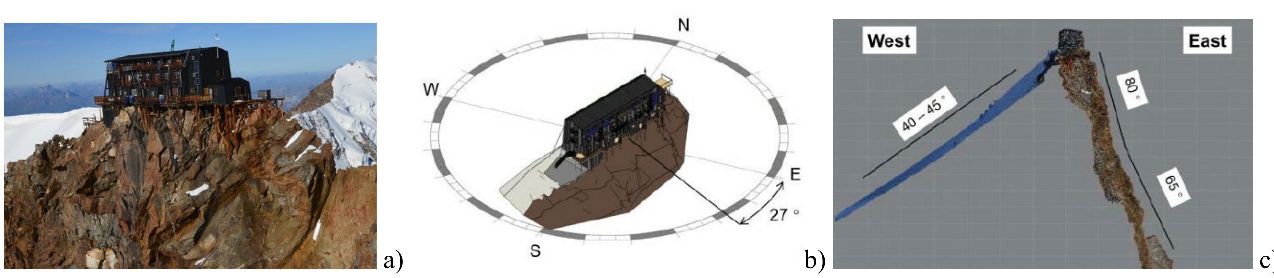


Fig. 1: (a) Punta Gnifetti with Capanna Margherita hut (4554 m asl); (b) sketch of the 3D model; (c) section of the 3D model parallel to the dip direction of the east face.

The procedure adopted in the numerical simulations is based on a geomechanical based back-analysis [2] and the assumption of two different scenarios of rock-mass degradation (fracture propagation and temperature increase). Both back-analysis and forecasting simulations rely on a particular application of the Shear Strength Reduction (SSR) technique, as it will be shown in the following.

A thorough survey allowed to describe the geomechanical features of Punta Gnifetti, by using laser scanner and photogrammetry, georadar and by performing direct measurements of the joint conditions. The reconstructed 3D model of the summit covers an area of some 200x100 m on the main rock face.

The summit is characterized by a main ridge, which runs approximately along the N-S direction, and two very different slopes on either side of it (Fig. 1c). The west side has a moderate slope (40°/45°) and it is covered by a few metre thick layer of snow and ice. The east face (the dip direction is actually 117°) is characterized by a sub-vertical upper part, and a lower part with an inclination of approximately 65°.

The rock mass under the hut is mainly composed of paragneiss, and it is characterized by the presence of eight sets of joints (Tab. 1). The data obtained during the survey and the comparison with literature allowed evaluating the cohesion and friction angle of intact rock (c_r = 30 Mpa; Φ_r = 40°), as described in [1] and [3]. It is worth noting that, at the moment, no information is available regarding the presence of ice in depth. Therefore, both conditions of empty joints and ice filled joints have been considered in the simulations. Although the survey allowed evaluating the trace length for all joints sets, the correlation between this parameter and joint persistence is unreliable. Therefore, persistence will be determined during the back-analysis, as it will be described in the following.

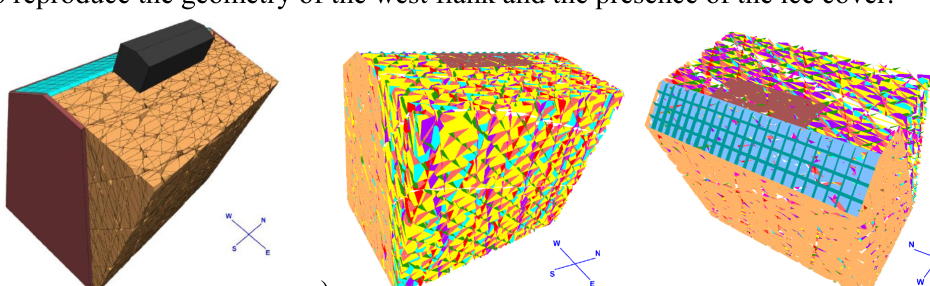
Table 1: Geometry of joint sets.

Joint set	Dip direction (°)	Dip (°)	Spacing (m)
A	99	85	4.1
B	43	76	7.4
C	172	82	8.1
D	309	56	10.8
E	160	48	3.4
F	139	81	4.7
G	77	53	2.9
H	120	62	1.4

2. Numerical simulations

The numerical simulations are performed with the Distinct Element 3DEC code. Considering the low stress level, and for the sake of simplicity, rock blocks and the ice cover are modeled as rigid elements. Therefore the only model parameters are joint stiffness (almost irrelevant in a stability analysis) and resistance (see §2.2).

The numerical simulations are conducted by following the procedure suggested by Calvetti et al. [2], where an initial model with simplified geometry, but complete from the point of view of rock mass structure, is used (Fig. 2). In this case the model is a 50x60x36 m (height x width x depth) parallelepiped. The hut is represented by a rigid element, and the back of the slope is cut to reproduce the geometry of the west flank and the presence of the ice cover.



a) b)

Fig. 2: (a) DEM model of Punta Gnifetti with Capanna Margherita hut; (b) joint sets.

Staring from this model, a back-analysis is performed first, where joint resistance is progressively decreased until the actual configuration of the slope is well reproduced (see §3). Starting from this point, joint resistance is further reduced to simulate evolutionary scenarios under various triggering conditions (see §4). A number of numerical simulations were preliminary performed with the aim of assessing the influence of the joint generation procedure and of boundary conditions [3]. These analyses are not shown here for the sake of brevity.

2.1. DEM model of the rock mass

The joint sets introduced in the model (Fig. 3) are directly based on the data of Tab. 1. It is worth noting that four sets are sub-vertical. Two of them are nearly parallel to the slope face (A and F) while the other two are almost perpendicular to it (B and C). Three joint sets with inclination between 50° and 60° form a sort of fan whose average dip direction is parallel to that of the slope (E, H and G). The remaining set dips in the opposite direction (D). In order to avoid unrealistically regular generation, the orientation of each joint is randomly picked within a ± 2° range from the values of Tab.1.

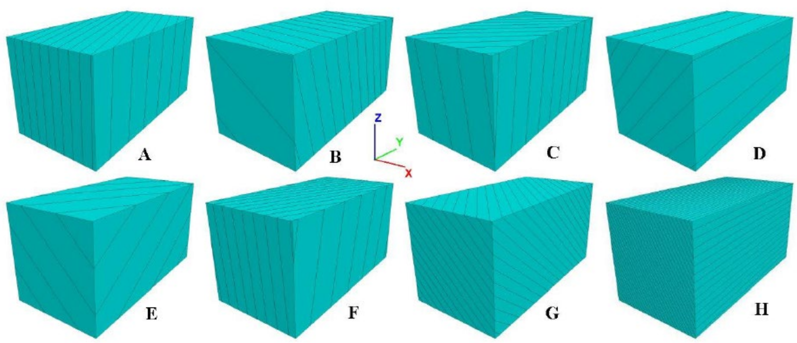


Fig. 3: Schematic representation of the joint sets (note: x-axis is parallel to the dip direction of the slope).

2.2. Joint resistance and joint properties

The Mohr-Coulomb criterion is here used for describing the resistance of rock joints. In particular, in this research we adopt the formula reported by Boffelli [3], who elaborated the results obtained by Wang ([1], [4]), to describe the influence of persistence (k) on cohesion (c) and friction angle (Φ):

$$c = c_f \cdot (k) + c_r \cdot (1 - k) \tag{1}$$

$$\tan \phi = \frac{1}{k + (1 - k) \cdot n} \left[\tan \phi_f \cdot (k) + \tan \phi_r \cdot (1 - k) \cdot n\right] \tag{2}$$

$$n = \frac{\sigma_{nr}}{\sigma_{nf}} \tag{3}$$

where c_f and c_r are the cohesion of fill material and rock bridge, Φ_f and Φ_r are the friction angle of fill material and rock bridge, n is a stress concentration factor, σ_{nf} and σ_{nr} are the normal stress in the fill material and in the rock bridge. It is worth noting that Eq. (2) is an extension of the formula proposed by Jennings [5], who assumed that the normal stress is evenly distributed within the rock bridge and the fill material (i.e. $n = 1$). This is generally not the case: for example, values of n between 10 and 15 are reported in [1] for ice-filled joints on the basis of joint-scale DEM simulations. In the simulations shown in the following, $n = 10$ was assumed for ice filled joints. Eq. (1) is the same formula proposed by Jennings [5], as its validity has been confirmed by [1]. The joint tensile resistance is assumed to be half of the cohesion; furthermore, a fragile behaviour is assumed for the cohesive component [3].

Two conditions are analysed: empty joints (no fill material) and ice filled joints. It is worth to note that joint resistance reduces when the persistence increases (rock bridge fracturing) and/or temperature increases (which reduces the resistance of ice). These two driving factors will be separately considered in the following. As to the influence of temperature on the resistance of ice, on the basis of several (and often conflicting) literature correlations ([6]-[9]) the values of Tab. 2 are retained [3].

Table 2: Ice strength parameters.

T (°C)	c_i (kPa)	Φ_i (°)
-1	250	0
-2	500	0
-3	750	4
-4	1000	8

It is worth noting that the resistance of rock is much higher than that of ice (c_r = 30 Mpa; Φ_r = 40°, see §1.1). However, the contribution of ice cannot be neglected because of the very large k values that characterise the rock mass, as it will be shown in the following.

3. Back-analysis

Starting from the simplified model of Fig. 3, a back-analysis procedure is performed by progressively reducing the joint resistance until the actual configuration of the slope is reproduced [2]. Back-analysis is performed under the two aforementioned conditions: empty joints and ice-filled joints. In this phase, a constant temperature is assumed, and the driving factor for strength reduction is considered to be the increase of persistence; strength parameters are calculated accordingly from Eqs. (1)-(2) and Tab. 2. In the case of ice-filled joints, the back-analysis has to be performed by assuming a value of temperature, first. Four values in the range from -4 to -1 °C have been considered in the simulations. For the sake of simplicity, at the moment a unique value of persistence and temperature is considered for all the joint sets.

3.1. Empty joints

According to Eqs. (1)-(2), in the case of empty joints the increase of persistence involves a progressive reduction of joint cohesion; in particular, joint cohesion is proportional to the rock bridge relative extension, b = (1-k). On the contrary the joint friction angle is unaffected (note that $n = \infty$ in this case).

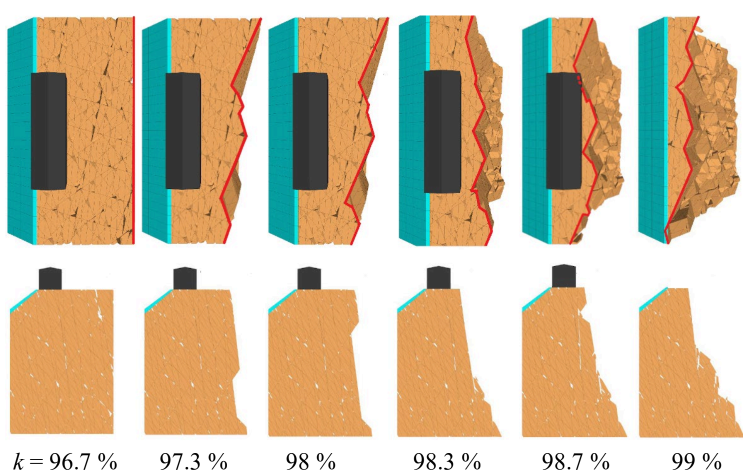


Fig. 4: Back-analysis (empty joints). Stable slope configurations (top view and section).

The results of numerical simulations clearly show that the increase in persistence corresponds to a progressive retrogression of the edge of the slope and to the reduction of the height of the sub-vertical portion of the profile (Fig. 4). By analysing these results and comparing them with the actual slope profile, the realistic value of persistence can be easily selected (see §3.3).

3.2. Ice filled joints

According to Eqs. (1)-(2) and Tab. 2, the increase of persistence involves a progressive reduction of both joint cohesion and friction angle, with the trend being influenced by temperature. As an example, in Fig. 5 the parameters corresponding to T = -4 °C are shown.

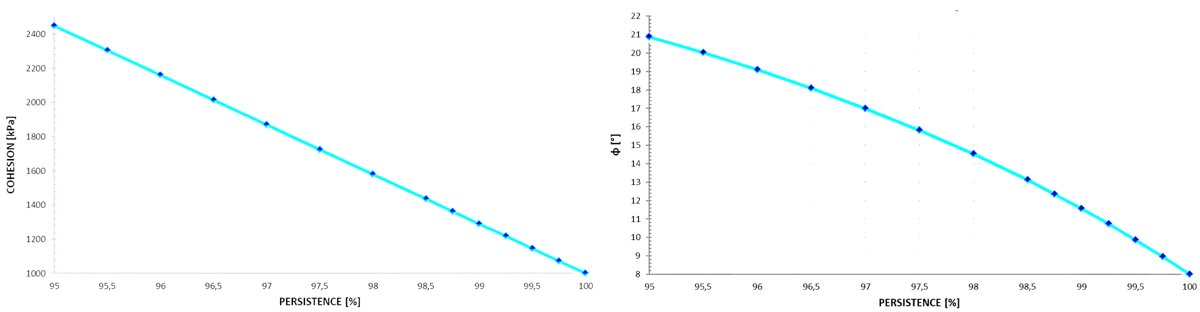
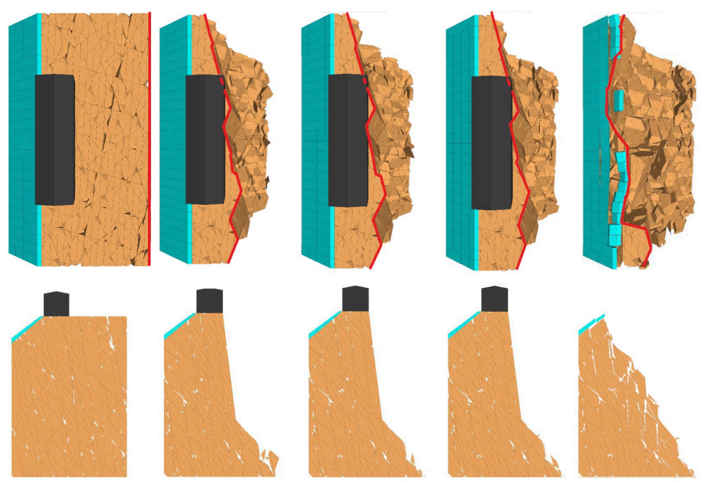


Fig. 5: Back-analysis (ice filled joints, T = -4 °C). Joint parameters vs. persistence, k.

In Fig. 6 the results obtained for T = -3 °C are discussed. The observed trend is similar to that obtained for empty joints. However, the retrogression of the slope is less progressive than in the previous case and it is characterised by a few big steps. The results obtained for all investigated temperatures are compared to the actual slope profile in §3.3.



k = 97.5 % 98 % 98.25 % 98.5 % 98.75 %

Fig. 6: Back-analysis (ice filled joints, T = -3 °C). Stable slope configurations (top view and section).

3.3. Discussion of results

The slope configurations that best reproduce the real slope are reported in Fig. 7 for all the considered conditions. The values of persistence and relative rock bridge extension obtained from the back-analysis, and the corresponding strength joint parameters are reported in Tab. 3.

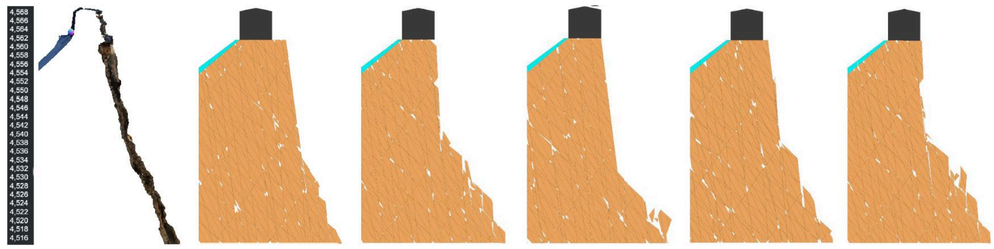


Fig. 7: Back-analysis results and comparison with the actual profile.

Table 3: Joint parameters calibrated from back-analysis (indicated with $_0$ subscript).

Joint Condition	T (°C)	k_0 (%)	b_0 = 1- k_0 (%)	c_0 (kPa)	Φ_0 (°)
Empty	-	98.3	1.67	500	40
Ice filled	-1	96.5	3.5	1291	12.6
Ice filled	-2	97	3	1385	11.2
Ice filled	-3	98	2	1335	11.3
Ice filled	-4	98.75	1.25	1363	12.4

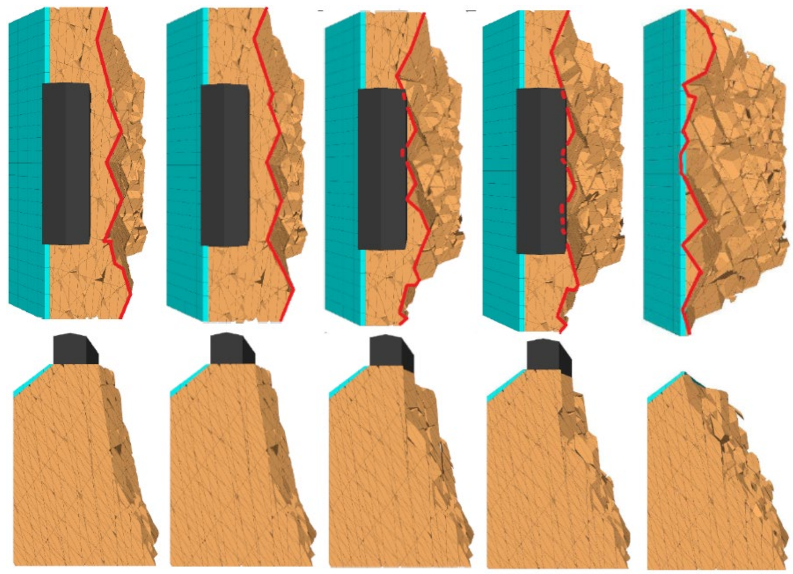
It is worth noting that in all cases it is possible to reproduce the actual slope profile with reasonable accuracy. The calibrated values of persistence are in general very large. The largest values of persistence correspond to the empty joint (when the friction angle is constant) and to the lowest temperature for ice filled joints. This result is in agreement with the decrease of ice resistance with temperature. The empty joint condition is characterised by a "high friction/low cohesion" combination; the opposite occurs for the ice filled joints, with minor adjustments due to temperature variations. This difference is expected to influence the possible evolution of the profile.

4. Scenarios of rock slope evolution

Starting from the reconstruction of the current configuration of Punta Gnifetti and the corresponding values of persistence and resistance parameters (Tab. 3), a strength reduction procedure can be applied in order to evaluate the margin of safety with respect to the future evolution of the slope. In a traditional shear strength reduction method (SSR), resistance parameters are reduced by applying a factor $R > 1$, until a loss of stability occurs. The value of R at failure can be then interpreted as the factor of safety. In our case, a slightly different approach is followed where the reduction in resistance is directly associated with its triggering factor. For empty joints, this factor is identified with the progression of rock bridge damage, i.e. a further increase in persistence (or decrease in rock bridge extension, b). For ice filled joints, the considered triggering factor is temperature increase. Obviously, in all cases the strength parameters at failure can be compared to the calibrated ones and an overall factor of safety, F_s, could be calculated. However, this value is just the outcome of the relationship among the set of factors that determine shear resistance of joints (persistence, rock resistance, temperature).

4.1. Empty joints: fracture propagation

In this case, the resistance is progressively reduced by progressively increasing the persistence, *i.e.* reducing the rock bridge extension and joint cohesion. The corresponding evolution of the slope (Fig. 8) is similar to the one described in §3.1. In particular, a relatively progressive retrogression of the edge of the slope is observed, until the configuration corresponding to the complete collapse of the peak is attained. This occurs for a value of relative rock bridge extension $b_f \approx b_0/2$. In traditional terms, this would correspond to an overall factor of safety $F_s \approx 2$.

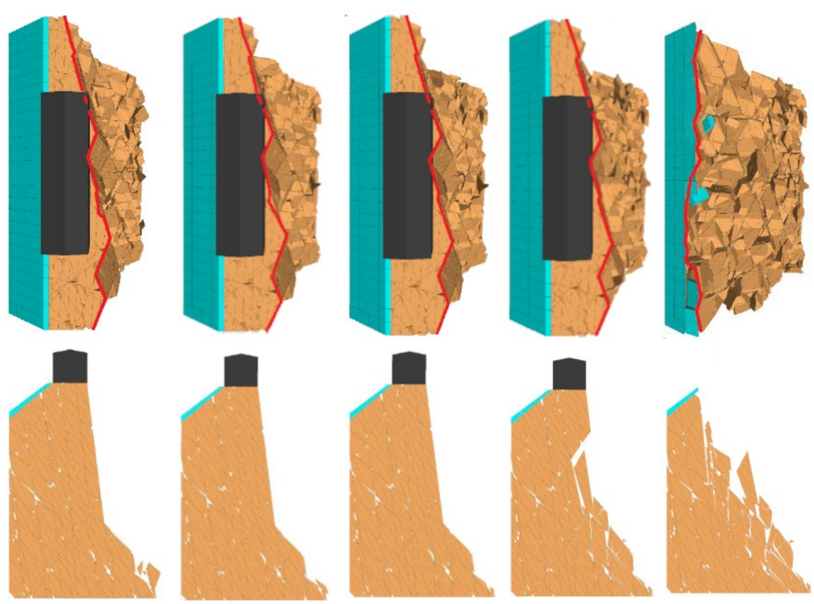


$b = b_0 = 1.67$ % $b = 1.47$ % $b = 1.27$ % $b = 1.0$ % $b = b_f = 0.87$ %

Fig. 8: Slope evolution triggered by increase in persistence.

4.2. Ice filled joints: temperature increase

In this case both cohesion and friction angle are gradually reduced as a consequence of increasing temperature. The corresponding evolution of the slope is shown in Fig. 9 for T_0 = -3°. Similar results are obtained for the other investigated initial temperature values. The results are characterized by a stable initial phase, followed by a sudden failure. This trend is similar to what observed during the back-analysis, where a slope retrogression much less progressive than in the case of empty joints occurred.



$T = T_0 = -3$ °C $T = -2.75$ °C $T = -2.5$ °C $T = -2.25$ °C $T = T_f = -2.0$ °C

Fig. 9: Slope evolution triggered by increase in temperature.

5. Conclusion

The results presented in this paper represent the application to a real case of the multi-scale approach proposed by the Authors; information from the material scale and joint scale ([10], [4]) is here used to calculate the resistance of joints in a large scale DEM model. Given the lack of complete information about site conditions, a series of analyses are conducted under different hypotheses regarding joint conditions and temperature. For this reason, the presented results are to be considered indicative and more about the applicability of the approach than the reliable assessment of the stability conditions of the investigated site.

Despite current shortcomings and lack of information, the results of the simulations show that the numerical model can reproduce the current configuration of the considered rock face. In addition, the model was used to study the possible evolution of the rock slope, and in particular the instability that would arise in case of resistance degradation. In this respect, two external triggering factors, which are considered to be responsible of most real-world collapses, were simulated: the increase of joint persistence (which can be caused by repeated cycles of freeze/thaw); the increase in temperature. In both cases, the effects on the slope stability have been analysed and the trend of the evolution has been detected and qualitatively discussed. As a general comment, the obtained results indicate that rock bridges have a critical influence on the stability and evolution of rock slope, in particular due to the fact that persistence is close to 100%.

A new in-situ campaign is underway, in order to complete missing information about the ice/water presence in the joints, temperature, water pressure and in-depth geomechanical configuration of the rock mass.

Acknowledgements

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