Mitigation of rock slope instabilities impacting Cultural Heritage sites

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ABSTRACT

Cultural heritage sites, such as castles, fortresses and churches are often situated on rock slopes characterised by steep topography and complex ground conditions. These can be attributed to less competent ground, geological formations with problematic behaviour, presence of faults and groundwater related problems. Many sites in Greece are affected by rock slope instabilities, which generally fall in three categories: a) falling rocks impacting monuments, b) monuments founded on rock slopes which have kinematic instabilities c) foundations undermined by rock movements in the form of sliding, overturning or toppling, or sudden loss of support due to collapse of voids. Cultural Heritage sites are characterised by the surrounding setting as a whole and not only the monument itself, therefore it is important to address these problems holistically. The present paper focuses on the main aspects related to rock slope instabilities and their mitigation demonstrated through specific case studies in Greece.

Keywords: rock, slope, instability, mitigation, monument.

INTRODUCTION

Cultural Heritage sites are vulnerable to problems related to rock slope instabilities, such as: a) rock slope failures (planar, wedge, toppling), b) presence of faults – earthquake related instabilities, c) deterioration of slopes due to rock exposure and weathering, d) groundwater and seepage related problems, e) undermining of foundations of structures on rocks, f) undercutting of slopes, g) Collapse of voids (karst, etc), h) rockfalls impacting structures or surroundings of sites. Geotechnical problems related to slope instability and the protection of historical sites in Greece have been addressed by several authors (Marinos & Koukis, 1988, Marinos et al., 2002 and Saroglou et al., 2012). The study of rock slopes entails the understanding of the geological environment, the characterisation of the rock mass, the kinematic stability analysis and finally the design of rock support and improvement as well as monitoring of movements. Rockfalls frequently impact cultural heritage sites, a phenomenon which requires detailed risk assessment and mitigation of risk through detailed trajectory analysis of falling rocks.

ROCK MASS CHARACTERISATION

Introduction

The engineering geological behaviour of rock masses governs the kinematic stability of rock slopes. Depending on the geological origin of the rock and the geological history (stresses, fracturing, faulting, presence of groundwater) it is possible to assess the risk from rock instabilities, which are either acting as the foundation of monuments or are affecting archaeological sites due to detachment of rocks. Characterising intact rock and discontinuities is equally important to determine the strength and deformability of rock masses. In most cases, instabilities are kinematically controlled, as movement of individual blocks occurs along discontinuities and in fewer cases failure takes places through intact rock.

Rock structure and discontinuities - use of photogrammetry

ISRM (1978) introduced the methods for characterisation of discontinuities, still being used today with modifications, which describes orientation, spacing, persistence, roughness, wall strength, opening, presence of infilling, groundwater, number of sets and block volume. The assessment of shear strength of discontinuities is based on the measurement of roughness and wall strength used in the Barton (1976) failure criterion.

3D digital terrain mapping is possible using photogrammetry methods, primarily with LiDAR and UAV methods, resulting in high resolution digital data (point clouds), which can be used for the characterisation of the rock mass and the determination of joint properties. In the last decade, emphasis is given in the determination of joint geometry and orientation, as well as their persistence, spacing and roughness of the surfaces using semi-automatic or automatic techniques. Riquelme et al., (2014) developed a method to determine the joint orientation, persistence and spacing based on photogrammetry. Examples from the application of the method on steep rock slopes are shown in Figure 1, from Corfu Fortress and Church of Koimisi of Theotokos in Petra, Lesvos Island (Saroglou et al., 2025). It is noted that the main joint sets are defined based on the clustering of the point cloud data and are shown with different colour mapping. Kong et al. (2020) developed a methodology to automatically determine joint orientation using 3D point clouds and clustering algorithms. These techniques have been used to determine joint properties and assist in the kinematic analysis of slopes in the case of Corinth Canal failures (Saroglou et al, 2019).

Figure 1. *Digital terrain model and automatic discontinuity mapping. (a) Rock slope at Corfu Fortress, (b) rock slope at Petra, Lesvos Island.*

Based on the manual measurement of discontinuities using geological compass and the automatic discontinuity mapping it is possible to define the orientation of major and minor discontinuity sets. An example from the field data statistical analysis of joints performed for Monemvasia rock slope is shown in Figure 2 (Saroglou et al., 2012).

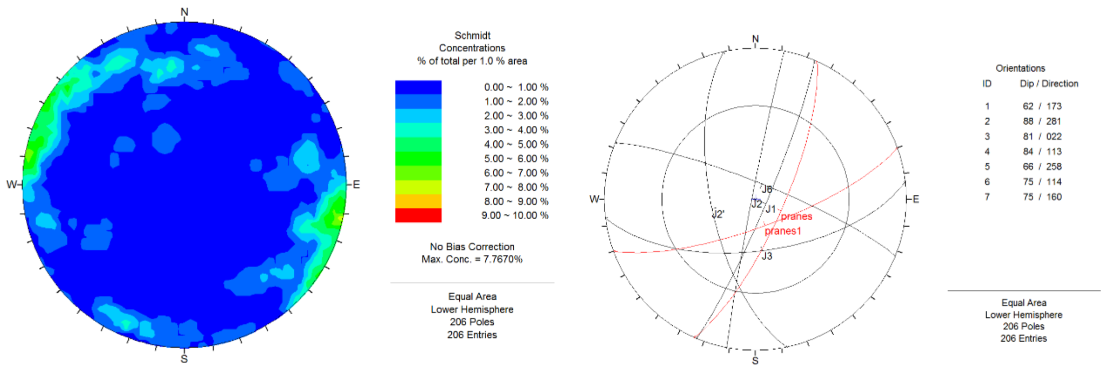

Figure 2. *Stereographic projection of discontinuities in Monemvasia rock slope (a) Schimdt concentration of poles, (b) projection of main discontinuity sets.*

ROCK SLOPE INSTABILITY

Kinematic stability analysis

The main types of kinematic slope instabilities are planar, wedge and toppling of rock blocks, while circular failures are relatively seldom and can be encountered in heavily fractured rock masses. To perform a kinematic stability analysis, it is important to identify the dominant joint sets and their orientation, spacing, persistence and shear strength. As discussed earlier, as new mapping techniques assist in the automatic discontinuities it has become more efficient to perform the kinematic stability analysis of individual blocks formed by different combinations of discontinuities. The analysis is performed under various loading scenarios, under dry conditions, with hydrostatic pressures and with earthquake loading. Based on the kinematic analysis the factor of safety is calculated for each scenario and the requirement of installing active or passive support measures (bolts, anchors) is decided. When designing support, it is important to determine as accurately as possible the bond strength and the required length of the bolts, anchors to resist driving forces during potential movement of blocks.

ROCK MASS IMPROVEMENT AND SUPPORT

Introduction

The rock slope support and improvement measures can be divided in three broad categories: a) those which apply an external force on the slope face e.g. tensioned rock anchors, pattern rock bolts, b) those which offer protection once a rockfall will occur e.g. rock fall barriers, c) other improvement measures, such as grouting of rock joints with associated drainage, construction of buttresses and supports in voided and overhanging areas and d) removal of unstable blocks. Depending on the geometry of the slopes (height, angle) and their accessibility, some of these measures may not be applicable or are very difficult to construct, especially in high and vertical or overhanging rock cliffs.

Rock mass improvement and drainage

Accumulation of groundwater pressure in discontinuities is the primary trigger of rock slope failures, when the overall permeability of the rock mass is low or during extreme rainfall events. One of the most effective methods of improving the stability of rock slopes is the removal of groundwater, especially in rock masses with low permeability. Drainage can be achieved by installing holes perpendicular to the slope face to remove groundwater. Instabilities due to the formation of tensile cracks along joints or faults or loosening of the rock mass are very frequent. In these cases, low pressure grouting can be effective to increase the degree of interlocking and rock mass strength by sealing individual open joints and fault zones with cement/sand mixtures or grouting highly fractured and loose rock masses.
An example of sealing open joints in a basaltic rock mass for the stabilisation of the rock slope at Mythimna Castle in Lesvos is shown in Figure 3. In this case, voids were encountered on the slope either due to the cooling of the basalt or due to previous slope movements, which undermined the stability of the slope. An effective stabilisation was implemented, through filling voids with concrete and constructing a stone wall facing. The concrete filling is bolted to the surrounding rock mass to achieve resistance to sliding (Figure 4).

Figure 3. *Sealing open joints in basaltic rock mass at Mythimna Castle, Lesvos Island.*

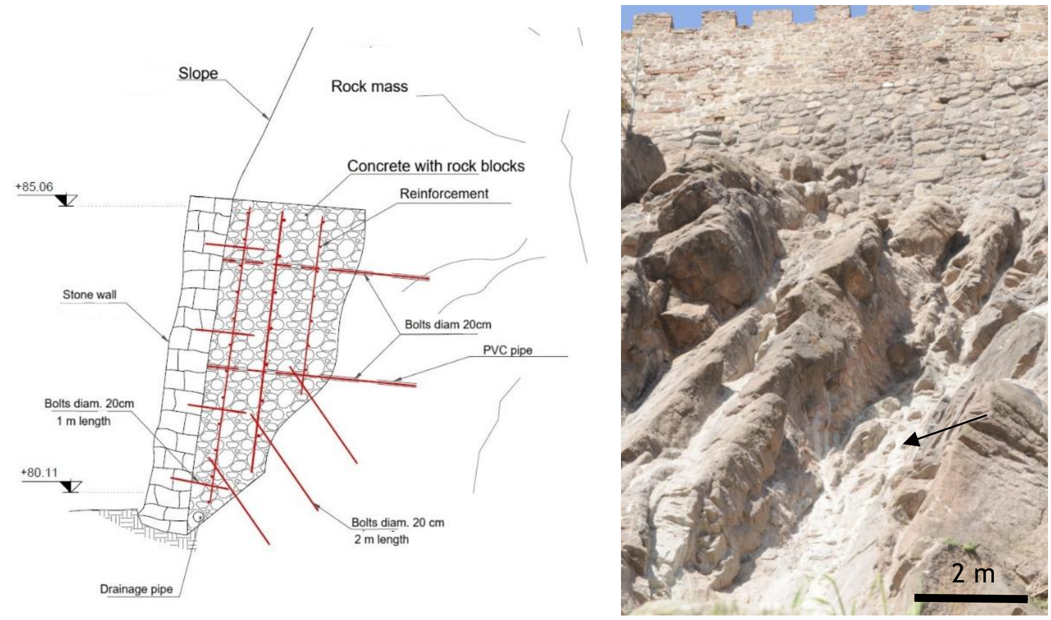

Figure 4. *(a) Drawing detail of void filling with concrete and stone wall facing to support overhanging blocks. (b) constructed filling shown with arrow, example from Mythimna Castle.*

Reinforcement

Stabilisation of rock slopes is performed using passive support measures such as rock bolts, anchored wire mesh or active support measures (e.g. pre-stressed anchors). The selection of the most appropriate technique is governed by many factors, such as the rock mass type, the required support pressure and the groundwater conditions. In most cases, if unstable rock blocks are small (e.g. less than 10 m^3) these can be supported with passive rock bolts either individual or in a bolting pattern. If unstable blocks are larger or movement of significant volumes or rock mass is possible due to presence of weak or fault zones, especially if these dip towards the slope face, then selection of active support measures is required. In the case of very fractured, loose rock mass, in which case individual small blocks can fall and create larger areas of slope instability, it is preferable to install anchored steel wire mesh to strengthen the rock mass and avoid further relaxation. A combination of different methods is also practical depending on the rock mass behaviour and the type of instabilities in various slope areas.

Case study from Petra, Lesvos

An interesting rock slope stability case is that of the volcanic rock cliff in the village of Petra in Lesvos Island, where the Church of the Koimisi of Theotokos is founded (Figure 5). The rock cliff has almost vertical slopes with height up to 40 m and is formed by an andesitic rock mass, fractured by four discontinuity sets. The predominant kinematic instability is wedge or toppling failures of medium-sized rock blocks. The problem of potential rock instability of the volcanic slope in Petra has been investigated in the past (Apostolopoulos & Koukis, 1979) and rock fall episodes have occurred mainly triggered by intense rainfall. Due to the weathering and deterioration of the andesitic rock mass, the risk of rock instability has increased in the last two decades and it was deemed necessary to protect the foundation of the monument as well as the surrounding settlement and visitors from potential rockfalls. Stabilisation measures were proposed to minimize the risk of rockfalls and protect the foundation of the Church (Saroglou et al. 2025).

Figure 5. *Volcanic rock cliff of Church of the Koimisi of Theotokos, Lesvos Island.*

Based on the kinematic analysis for planar, wedge sliding and toppling in each area of the cliff the factor of safety against each type of failure was performed in static conditions, with presence of water pressure in joints and under seismic loading and the stabilisation measures were determined. Typical kinematic analysis for planar and wedge instabilities from this case is presented in Figure 6.

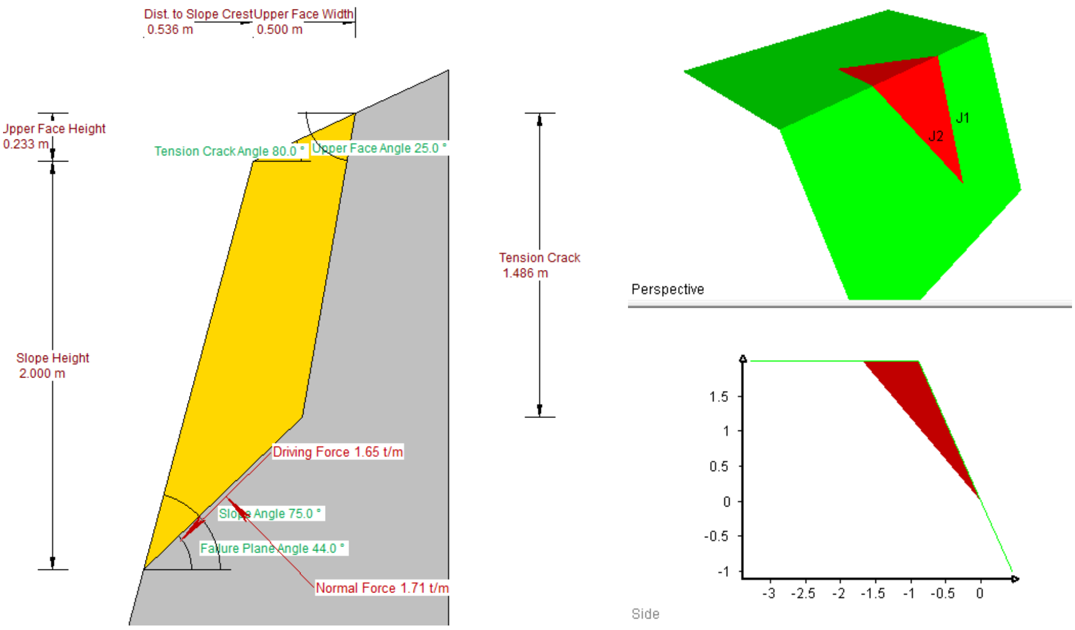

Figure 6. *Kinematic analysis of planar and wedge instabilities in Petra, Lesvos Island.*

The main support measures proposed were:

a) Installation of spot bolts for anchoring individual relatively small rock blocks and pattern bolting for anchoring of larger blocks. The aim of this measure was to support potentially unstable blocks and improve the stability conditions of the upper part of the cliff below the foundation of the Church.

b) Installation of a steel wire rope net, anchored with fully grouted bolts to control further loosening of the rock mass in fractured areas, as presented in Figure 7 and Figure 8a. c) Filling of voids in specific locations with sprayed concrete, depending on their location, geometry and size to minimise the enlargement of voids due to weathering and support the overhanging slope areas (Figure 8).

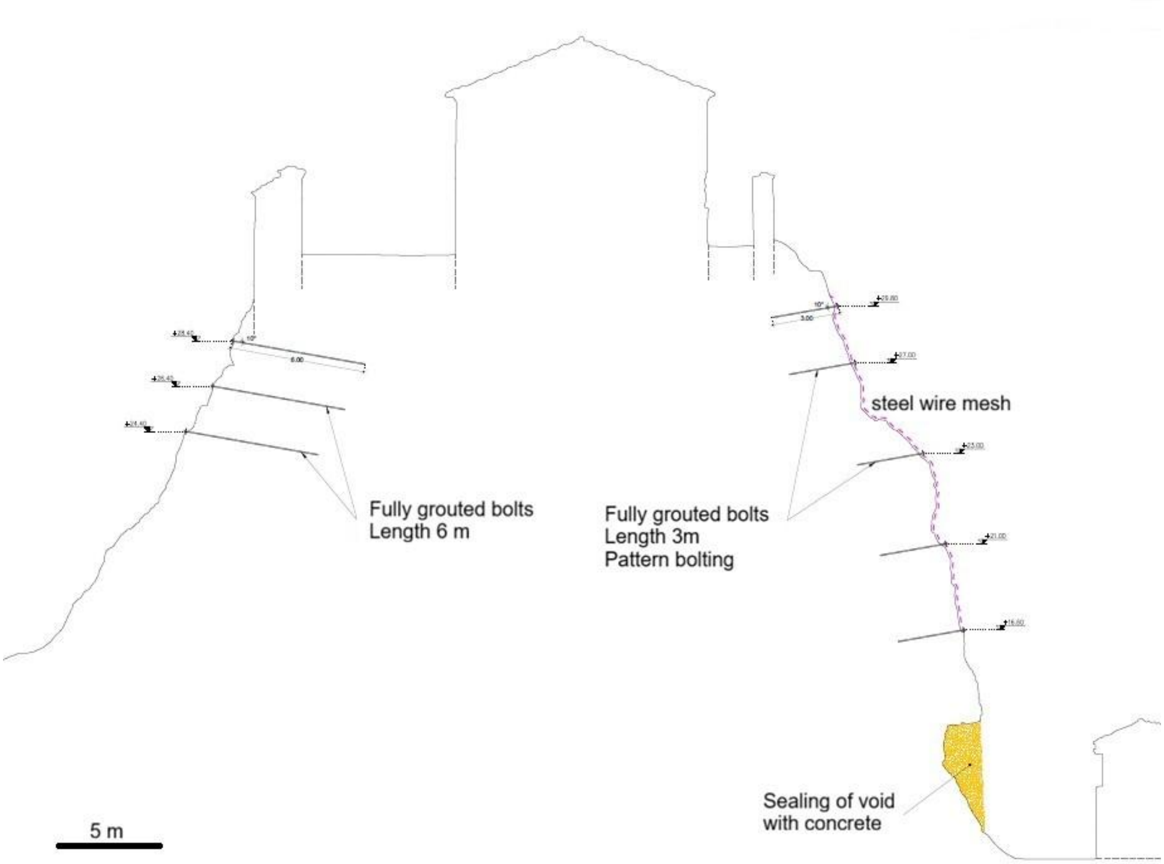

***Figure 7**. Proposed stabilisation measures comprising passive fully grouted rock bolts and anchored steel wire mesh in Petra rock slope.*

***Figure 8.** (a) Installation of anchored wire mesh, (b) filling of voids with cement at the base of slope in overhanging areas.*

Case study from Theoktisti Church, Ikaria Island

An example of an unstable blocks due to instability of individual rock blocks is demonstrated by the case study of Theoktisti Church, which is impacted by the movement of a gneiss rock block, sliding on a gently dipping foliation plane (dip between 16-25^0) as shown in Figure 9. The block is detached from the rock mass as a tensile crack is formed on the backscarp of the block (Figure 10a). A backanalysis was performed to determine the shear strength of the sliding plane. A kinematic analysis under hydrostatic pressure (filled at 65% of the tensile crack height) and earthquake loading (a=0.112g) was performed to determine the required support force (Figure 10b). The force was calculated equal to 262 kN per meter and the proposed anchor length was equal 8 m, inclined at 65^0 from horizontal.

Figure 9. *Gneiss rock mass sliding on foliation plane and impacting a historic church. The church is downslope behind the rock block.*

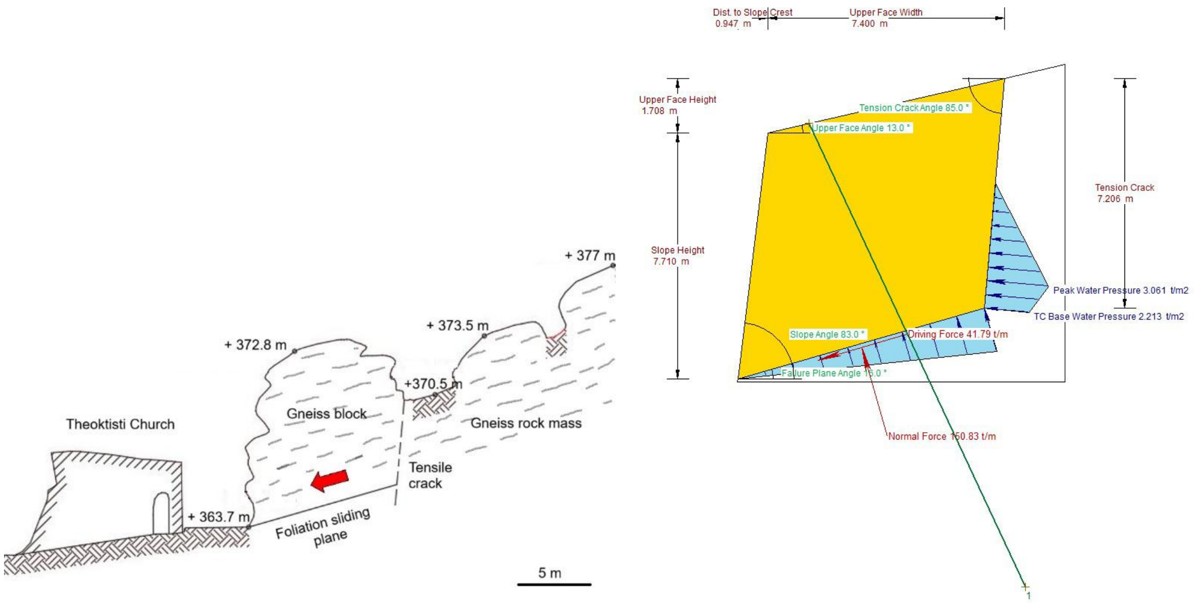

Figure 10. *(a) conceptual kinematic model of sliding block, (b) kinematic analysis of sliding block with hydrostatic pressure and support from rock anchors.*

ROCKFALL RISK

A significant number of Cultural Heritage sites in Greece are affected by rockfalls due to the steep morphology of the sites and the rocky geological setting. The hazard of rockfalls is obviously higher in areas with intense seismic activity, where earthquakes are the principal triggering factor. Characteristic examples are Delfoi (Christaras & Vouvalidis, 2010), Monemvasia Castle (Saroglou et al. 2012), Nafplion and other. A detailed review for rockfall risk in Greece is presented by Saroglou (2019).

Risk assessment

Different risk assessment systems, either qualitative or quantitative, have been proposed to assess the risk from rockfalls. These rely on the rating of various parameters and the classification of rockfall risk for either natural or man-made slopes. Depending on the number of parameters the assessment of risk can be relatively complex. To calculate the probability of rockfalls, one should assess historical data (e.g. frequency and magnitude of rockfall events) as well as investigate the predisposal factors which can trigger future events. In the recent years, the risk assessment is based on the kinetic energy level detailed 2D and 3D modelling to calculate rockfall trajectories and the kinetic energy of falling rocks. A vulnerability assessment of cultural heritage sites requires a detailed analysis of the rockfall hazard and risk as well as the probability of occurrence of the events. A rockfall risk rating system a rockfall risk rating system for natural rock slopes has been developed by Saroglou et al. (2012). The system defines 19 rating parameters, grouped in three major categories on hazard and consequences, which have a different weight in the assessment of the total hazard. The hazard classification of rock slopes based on this system is divided in five categories (between very low to very high risk), as presented in Table 1.

Table 1. *Rockfall risk classes and indicative protection measures*

Risk Class	Total weighted score 1-100	Risk	Indicative protection measures (the choice is site specific)
I	< 20	Very Low	Not necessary. May be sparse spot interventions
II	21 - 40	Low	In limited extent
III	41- 60	Medium	Light measures (such as bolts, nets, removal of unstable blocks, simple light fences)
IV	61 - 100	High	Combination of active (such as bolts, anchors) and passive (such as nets, wire rope cables, buttress walls, fences removal of unstable blocks) measures
V	81 - 100	Very High	Critical state of stability, combination of generalized or/and strong active and passive measures. Residual risk to be accepted.

The case study of Monemvasia archaeological site

The rockfall risk in Monemvasia archaeological site has been assessed based on this system. The risk map in Figure 11 depicts areas having a medium and high risk due to either increased number of existing unstable blocks on the slope or restricted area for their catchment at the slope base or a combination of both factors. The slope foot area between sections C to E presents high risk due to the numerous unstable blocks on the cliff and proximity of structures as well as human activity (stairs to upper city). The area between sections E to Z has medium to high risk, due to the wide catchment zone at the base, which offered ideal conditions for installation of barriers. The area between sections A to B has very restricted catchment zone but as there are no houses in that area the risk is medium. According to this system, stabilisation measures have been proposed and been installed to protect the visitors of the archaeological site and the houses in the surroundings of Monemvasia castle (Saroglou et al. 2012).

Rockfall modelling

To analyse the trajectory of rockfalls it is important to perform detailed digital terrain mapping, geological mapping, identification of release areas of rockfalls, statistical distribution of unstable block volumes, assessment of properties of rock slope and falling rock blocks. Rockfall modelling is either two or three-dimensional, with the latter being more complicated as more data need to be collected on site for the parameters mentioned earlier. If a quick assessment is required, 2D analysis is adequate, but a good understanding of rockfall mechanics is required to assess the trajectories to predict realistic results. 2D rockfall analyses neglects the effect of the topography whilst 3D analyses account for the lateral dispersion of rockfall trajectories (Crosta and Agliardi, 2004), i.e., the ratio of the lateral distance separating the trajectories to the slope length.

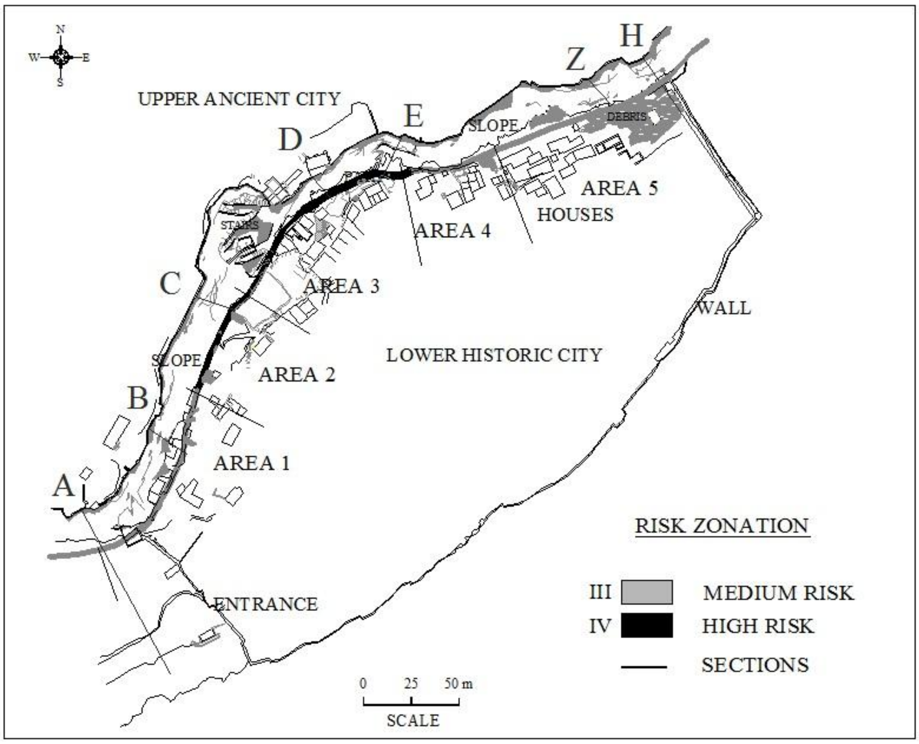

Figure 11. *Risk zonation of rock slope based on proposed rockfall risk rating system*

Case study of Acropolis of Alyki

The case study refers to the Acropolis (4th century BC) situated at the crest of a limestone promontory in Alyki, Voiotia prefecture, central Greece. The Acropolis still preserves a 400 m long fortification that surrounds the north and east side of the promontory. The slopes on which the Acropolis is founded are steep and the area is prone to rockfalls due to the fracturing of the limestone rock mass and the presence of overhanging unstable blocks (Saroglou et al, 2019). In 2019, a rockfall event reaching a total volume of 1000 m^3 occurred after heavy rainfall and impacted the houses located at the base of the slope putting also into risk the integrity of the archaeological site (Figure 12). Mitigation of potential rockfall impacts on visitors of the Acropolis and surrounding houses was deemed necessary and a detailed assessment was commissioned.

Figure 12. *(a) Aerial photograph showing the location of the 2019 rockfall detachment and the area of rock blocks deposition b) Rockfall event impacting houses.*

To assess the rockfall risk, three-dimensional analyses were performed using Rockyfor3D (Dorren, 2016), which simulates trajectories of individual rock blocks using deterministic models. The parameters related to the geometry of the falling rocks and the characteristics of the geological formations should be defined, more specifically: (a) soil type, which is directly linked to the normal coefficient of restitution (R_n), (b) roughness ($rg_{10,20,70}$), which corresponds to the height of a representative obstacle that a falling rock encounters at 10%, 20%, and 70% of the cases during a rebound respectively and (c) dimensions (height, width, length), shape and density of falling rocks. The surface roughness is used to calculate the tangential coefficient of restitution (R_t). The roughness should be determined in the field by identifying homogenous zones in the study area. The volume of a single block was assumed equal to 4 m^3. An initial velocity of the falling rocks was calculated equal to 0.20 m/s as the area is prone to earthquakes (a=0.36g).

The main results from 3D analysis are a) the trajectories of the simulated rock blocks, b) the reach probability (RP), c) the Kinetic Energy (E_{95CI}) defined as the mean kinetic energy (kJ) of the falling rocks in a specific location increased by two standard deviations (95% confidence interval assuming a normal distribution of passing heights). An example from a 3D analysis with kinetic energy distribution of falling rocks from this site is presented in Figure 13.
The kinetic energy of the rocks, ranges between 200-1600 kJ in most areas, but in some places (especially in the central, southern part) they exceed 1600 kJ and in very few cases can reach up to 3200 kJ. To address the potential risk of rockfalls impacting the houses, it was proposed that rockfall barriers are installed at specific locations to intersect the trajectories. The barrier kinetic energy ranged between 500 kJ and 2000 kJ at maximum, dimensioned at the 95% percentile of kinetic energy distribution while their height was calculated equal to 5 m. The appropriate location of the rockfall barriers was calculated based on the 3D analyses as shown in Figure 14. Some outliers may exist (i.e., rock blocks that propagate above the protective barriers), but their impact on the barriers decreases their kinetic energy hence their trajectories will not reach the houses.

Rockfall mitigation

Various techniques are used for the mitigation of rockfalls, such as removal of unstable blocks (scaling), slope reprofiling (benching), installation of ditches, rockfall barriers, reinforced embankments, rockfall galleries and other. In cultural heritage sites, it is very often the case that no mitigation measure should have the least possible impact on the archaeological environment, hence the option of installing rockfall barriers may not be acceptable. Of course, when no other alternative method is feasible due to space constraints and difficulty of installation of bolts and wire rope nets, the installation of barriers is preferred.

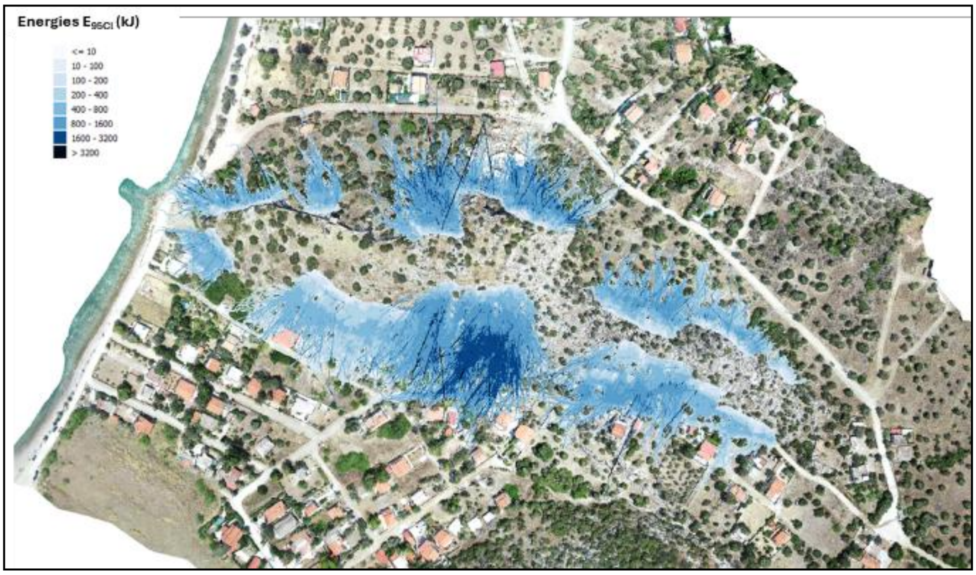

Figure 13. *Distribution of kinetic Energies E_{95CI} in the surrounding area of the Acropolis of Voiotia.*

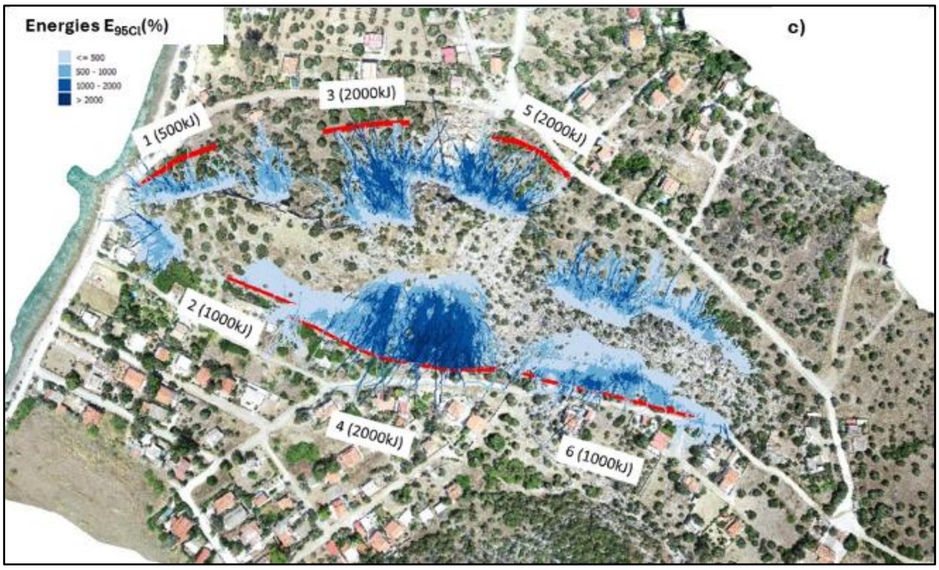

Figure 14. *Distribution of kinetic Energies E_{95CI} after installation of the rockfall barriers (not constructed yet).*

CONCLUSIONS

The selection of appropriate slope stabilisation is dependent on many factors, which are primarily governed by the geological setting, the rock mass conditions and the prevailing mechanism of failure of the rock slope. Preservation of cultural heritage sites requires robust mitigation measures to address the kinematic instabilities and rockfalls. It is essential that different scenarios are considered for different loading conditions. Installation of instrumentation is also necessary to identify potential movements before the onset of failure, although generally movement of rock blocks is sudden and no precursory movement may have occurred before failure. Different rock slope stabilisation techniques were discussed in this paper, which were used for the support slopes and improvement of rock mass conditions to achieve the protection of Cultural Heritage sites in Greece. Specific mention was given to the problem of rockfalls, which is very frequently affecting sites either by impacting visitors or the monuments directly.

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REFERENCES

Andronopoulos B. and Koukis G. (1979). Geological, Microtectonic and Geotechnical Study in the NW part of the Lesbos Island. Unpublished report, Institute of Geological and Mining Research, Engineering Geology Investigations No. 8, 41 p., Athens.

Christaras, B.; Vouvalidis, K. (2010). Rockfalls in the archaeological site of Delphi, Greece. In Proceedings of the IAEG 2010 International Congress, Auckland, New Zealand.

Crosta, G.B., Agliardi, F. (2004). Parametric evaluation of 3D dispersion of rockfall trajectories, Natural Hazards Earth System Sciences, v.4(4), pp.583-598.

Dorren, L.K.A. (2016). Rockyfor3D (v5.2) revealed – Transparent description of the complete 3D rockfall model.

ISRM, (1978). Suggested methods for the quantitative description of discontinuities in rock masses: International Society for Rock Mechanics, Int. Journal of Rock Mechanics and Mining Sciences, Vol:15(6), Pages:319-368.

Kong, D.H., Wu, F.Q., Saroglou, C., (2020). Automatic identification and characterization of discontinuities in rock masses from 3D point clouds, Engineering Geology, Vol: 265, 105442. doi: 10.1016/j.enggeo.2019.105442.

Marinos P, Koukis G (ed.) (1998). The Engineering Geology of Ancient Works, Monuments and Historical Sites. Balkema, 4 volumes. Proc. of Int. Symp. of Greek National Group of IAEG.

Marinos P., Kavvadas M., Tsiambaos G., Saroglou H., (2002). Rock slope stabilization in Mythimna castle, Lesvos Island, Greece. 1st European conference on landslides, Balkema, ed: Rybar Stemberk & Wagner, Prague, p. 635-639.

Riquelme A.J., Abellan A., Tomas R., Jaboyedoff M. (2014). A new approach for semi-automatic rock mass joints recognition from 3D point clouds. *Comput Geosci*.; 68:38–52. https://doi.org/10.1016/j.cageo.2014.03.014.

Saroglou H., Marinos V., Marinos P., Tsiambaos, G. (2012). Rockfall hazard and risk assessment: an example from a high promontory at the historical site of Monemvasia, Greece. Natural Hazards and Earth System Sciences, 12, 1823–1836.

Saroglou C. (2019). GIS-based rockfall susceptibility in Greece. Geosciences, 9, 163.

Saroglou, C., Kallimogiannis, V., Bar, N., Manousakis, G., Zekkos, D., (2019). Analysis of slope instabilities in the Corinth canal using UAV-enabled mapping. Proc. Int. Conf. Nat. Hazards Infrastruct.

Saroglou C., Kong D., Wu F., Kallimogiannis V., Tsirogianni A., Bar N. (2021). Rock mass characterisation for rock slope instabilities using aerial photogrammetry and 3D Point Cloud Model interpretation. Proc. of 3rd European Regional Conference of IAEG, Athens.

Saroglou C., Kong D., Wu F., Kallimogiannis V., Tsirogianni A., Bar N. (2021). Rock mass characterisation for rock slope instabilities using aerial photogrammetry and 3D Point Cloud Model interpretation. Proc. of 3rd European Regional Conference of IAEG, Athens.

.Saroglou C., Kallimogiannis V. (2024). Rockfall risk assessment using 3D analyses, a case study from a cultural heritage site in Greece/ Proc. 4th European Regional Conf. of IAEG, Dubrovnik, Croatia.

Saroglou C., Kallimogiannis V., Tsirogianni A., Klironomou G. (2025). Stabilisation of volcanic rock slopes in traditional settlements. The case of Petra, Lesvos Island. Bulletin of the Geological Society, Special Publication No. 15, 2025, ISBN 978-618-86841-6-4, 1315-1319.