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Geotechnical assessment of rock slopes at the medieval fortification of Mileševac

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Abstract

This Mileševac Fortress is a medieval stronghold located in southwestern Serbia, in the vicinity of the town of Prijepolje. The fortress is positioned on a steep rock cliff, overlooking the Mileševka River, the Mileševa Monastery, and the adjacent settlement. For the purpose of its reconstruction, comprehensive geotechnical investigations were undertaken to assess slope stability and to identify potential hazards that may compromise visitor safety. The fieldwork included detailed engineering–geological mapping and photogrammetric surveying of the slope using an unmanned aerial vehicle (UAV). Rock samples were collected from several locations for laboratory testing and petrographic analysis. Point cloud data were analyzed to define the rock mass structure, followed by a kinematic stability assessment. The results indicated a potential for block detachment on the eastern side of the cliff along the access path, with estimated volumes ranging between 0.02 and 0.05 m³. Consequently, a three-dimensional rockfall analysis was performed to establish the necessary design parameters for the implementation of a rockfall protection barrier.

Keywords

Rock slope, Photogrammetry, Kinematic analysis, Rockfall analysis.

1 Introduction

The medieval town of Mileševac represents an important cultural and historical monument located in the southwestern part of Serbia, in the immediate vicinity of the Mileševa Monastery and about 7 km east of Prijepolje. It was first mentioned in historical sources in 1444, although it is assumed that it was built in the first half of the 13th century, during the construction of the Mileševa Monastery. It is believed that its main function was the protection of the monastery, as well as the control of a section of the so-called Bosnian Road, one of the most important medieval transport routes connecting medieval Serbia with Bosnia and further with Dubrovnik. The Ottomans permanently captured the fortress in 1465 and held it until the end of Ottoman rule in this area (Deroko 1950).

The fortress was built on a steep hill and is naturally protected by cliffs on the eastern, northern, and western sides (Fig. 1). For the purposes of the reconstruction of the fortress it was necessary to perform rock mass characterization and slope stability assessment, both for the proper planning of reconstruction works and for ensuring safe movement of visitors within the site.

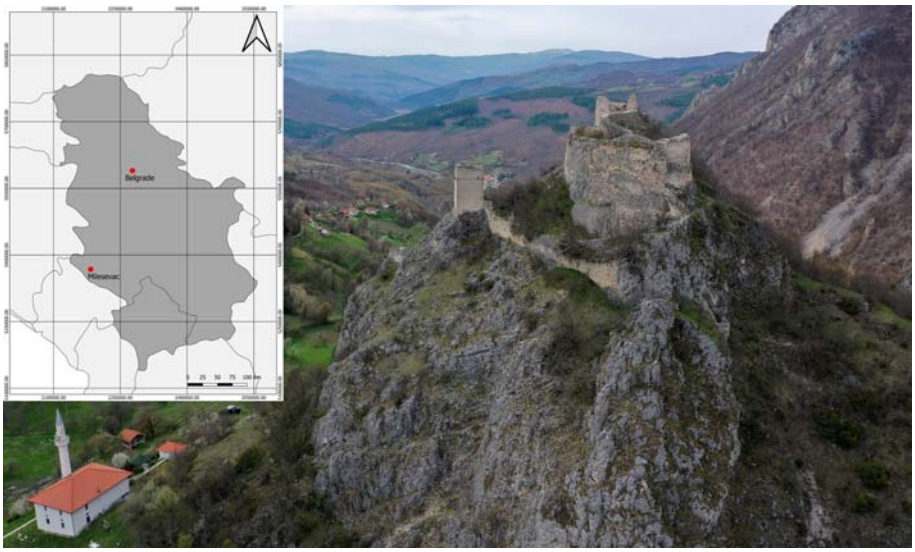


Fig. 1 Drone view of the medieval Mileševac fortress located on steep limestone cliffs above the Mileševka valley with the site location in southwestern Serbia.

The collected field data enabled kinematic analyses through which the dominant types of instability were identified.Subsequent stability analyses included the determination of the factor of safety of potentially unstable wedges and three-dimensional rockfall analysis.

The aim of this paper is to present the results of geotechnical investigations conducted in the area of the medieval fortress of Mileševac, with particular emphasis on the application of photogrammetry for identifying the structural characteristics of the rock mass and 3D rockfall analysis.

2 Research methodology

The investigations included engineering geological mapping of the terrain, laboratory testing of the physical and mechanical properties of the rock, as well as mineralogical and petrographic analyses of samples. Due to the limited accessibility of certain parts of the terrain, photogrammetric survey using an unmanned aerial vehicle (UAV) was applied.

2.1 Engineering geological mapping

Engineering geological mapping of representative rock mass outcrops was conducted at five locations along the steep slopes of the hill within the fortress area.

The mapping included the identification of discontinuity sets and the measurement of their orientation elements using a geological compass, measurement of roughness profiles using a Barton profilometer, as well as testing the strength of discontinuity walls with a Schmidt hammer (Fig. 2). The condition of discontinuities was assessed in accordance with the RMR classification.

During field mapping, 20 measurements of discontinuity orientations, 20 roughness profiles, and more than 100 Schmidt hammer rebound measurements on discontinuities and the surrounding rock mass were collected (both on fresh and altered rock).

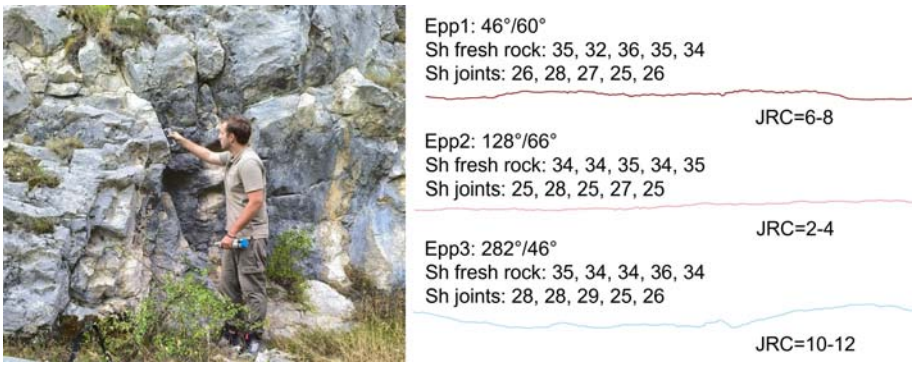


Fig. 2 Measurement of discontinuity roughness using a Barton profilometer in the field (left) and digitized roughness profiles with corresponding JRC values for identified discontinuity families. (Sh denotes Schmidt hammer rebound value).

During field mapping, representative rock block samples were also collected for laboratory testing.

2.2 Photogrammetric survey of slopes

In order to analyze the rock mass structure in more detail, the slope in the area of the medieval town of Mileševac was surveyed using a DJI Air 2S unmanned aerial vehicle equipped with a 20 MP camera with a 1-inch CMOS sensor. The survey was performed through multiple manual flights in horizontal and vertical directions relative to the slope face, ensuring adequate overlap between photographs.

In order to fully capture the area around the fortress, the survey was conducted from several directions. From the western and northwestern side 268 photographs were collected (resolution 4.32 mm/pix), from the eastern side 135 photographs (resolution 3.03 mm/pix), from the southeastern side 213 photographs (resolution 2.75 mm/pix), while from the southern side 155 photographs were collected (resolution 6.01 mm/pix). Based on the collected photographs, four three-dimensional terrain models were generated.

2.2.1 Process of generating 3D model

The three-dimensional slope model was generated using the Structure from Motion (SfM) photogrammetry method. Unlike the classical stereoscopic approach, the SfM method does not require prior knowledge of camera position and parameters, as they are determined during image processing through the application of appropriate algorithms.

The algorithm identifies characteristic points in photographs and determines camera positions based on their matching (so-called camera alignment). For reliable reconstruction it is necessary that the photographs have sufficient longitudinal and lateral overlap, usually in the range of 60–80%. Image processing and generation of the three-dimensional model were performed in the Agisoft Metashape software package. In the first step of the process key features are extracted from the photographs, after which the external and internal camera parameters are determined and a sparse point cloud is generated. After filtering unreliable points the model can be positioned in space using GCP (Ground Control Points). For the purposes of this research a local coordinate system was used, with the model scaled based on known distances between markers placed on the slope face. The orientation of the point cloud was verified by comparing the orientation elements of characteristic discontinuities measured in the field and on the reconstructed model.

In the final stage of the process a dense point cloud was generated (Fig. 3). The accuracy of the model primarily depends on the distance from the surveyed object, the number of photographs, their overlap and their resolution.

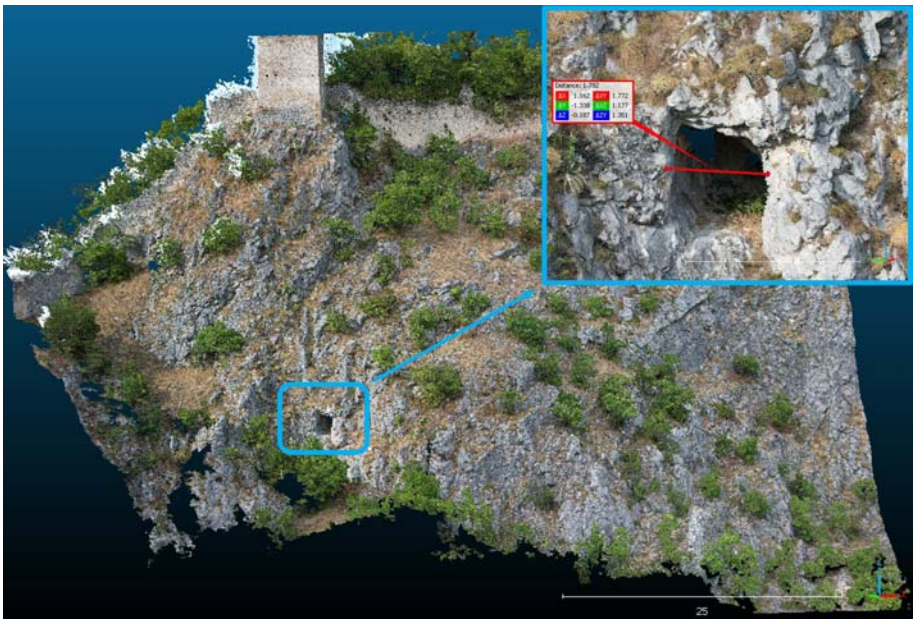


Fig. 3 Dense point cloud of the slope (view from the southern side) with a detail of the cave.

2.2.2 Principle of collecting the structural data on the point cloud

After generating the three-dimensional model, the dense point cloud was exported in .xyz format into the CloudCompare software, which enables detailed analysis of rock surface geometry and measurement of structural elements.

For the structural analysis, the Compass plugin was used, which allows manual determination of discontinuity orientation elements (azimuth and dip angle) directly on the point cloud. Measurements are performed by selecting planar surfaces corresponding to discontinuities, where a plane and a normal vector are automatically defined on the selected surface (Fig. 4). This manual mapping option was chosen in favour of semi-automatic and automatic techniques due to the presence of vegetation on the slope face, which prevented their reliable application.



Fig. 4 Principle of orientation measurements (planes with normals) and representation of measured values in dip angle/azimuth format.

In this way, a large number of discontinuity orientation measurements could be obtained from parts of the slope that were inaccessible for conventional field measurements. Based on a detailed analysis of the point cloud, critical zones characterized by pronounced discontinuities and potentially unstable rock blocks were also identified, with particular attention given to the area directly below the fortress walls and towers. In addition to determining orientation elements, measurements of spacing between discontinuities were also performed on the point cloud, as well as estimation of the dimensions of potentially unstable rock blocks.

2.3 Laboratory tesitng

Laboratory tests were conducted on representative rock mass samples extracted from blocks with approximate dimensions of 20 × 20 × 30 cm. A total of 10 cylindrical specimens were prepared from the rock blocks for laboratory testing. The samples were prepared and tested in accordance with ISRM and ASTM standards in the Rock Mechanics Laboratory of the Mining Faculty in Bor. Uniaxial

compressive strength was determined on 6 samples, while tensile strength was determined by the Brazilian test on 4 samples. The Point Load Test (PLT) was performed on a total of 11 samples.

The basic friction angle (φ_b) was determined using the tilt test according to ISRM recommendations (Alejano et al., 2018), applying the Stimpson method with three cylindrical samples. The basic friction angle was later used for the determination of Barton-Bandis strength criterion.

3 Results

3.1 Engineering geological characteristics of the terrain

The terrain is composed of grey to bluish limestones, unaltered, with a massive texture and locally slightly karstified. The rock mass is occasionally marly or dolomitic, and locally exhibits visible calcarenite structures. It is commonly intersected by calcite veins. At least three discontinuity sets are present, dividing the rock mass into blocks of decimetre to metre dimensions. Discontinuity surfaces are mostly fresh to slightly weathered, smooth to moderately rough, most commonly with apertures up to 5 mm and hard infilling, while more rarely they occur without infilling. Laboratory tests performed on limestone samples yielded the parameters shown in Table 1.

Table 1. Results of laboratory tests

Parameter	Range	Average
Unit weight γ_z (kN/m^3)	26.20 - 26.30	26.25
UCS σ_p (MPa)	33.70 - 65.30	47.90
UCS_{PLT} σ_{c50} (MPa)	13.69 - 85.31	44.35
Indirect tensile strength σ_z (MPa)	3.10 - 7.80	5.08

3.2 Kinematic analyses

Kinematic analyses were performed using a standardized procedure, in which the orientations of all identified discontinuities were first plotted on a stereonet. Depending on the geometric relationships between individual discontinuities and their orientation relative to the slope face, the potential for different types of instability was evaluated.

The results of the kinematic analyses for the dominant type of instability on each side of the cliff are presented on stereographic projections (Fig. 5).

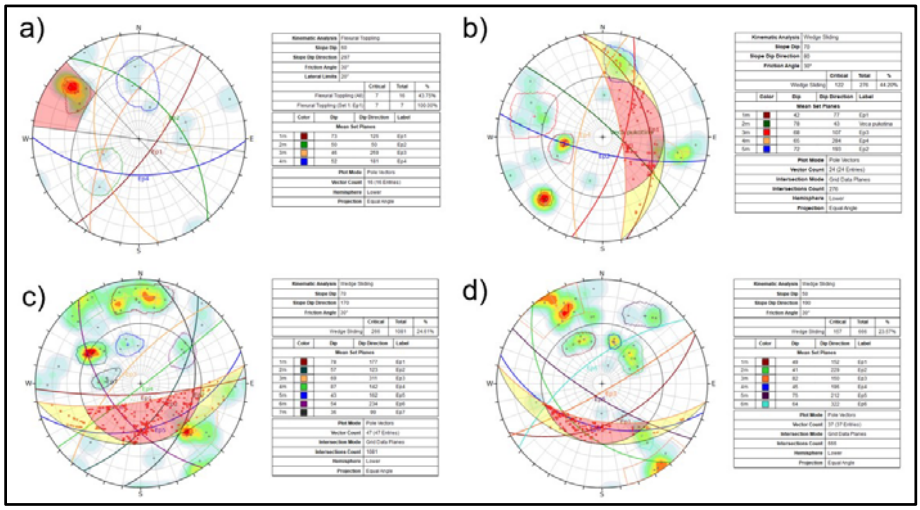


Fig. 5 Results of the kinematic stability analysis: a) northwest side, b) east side, c) southeast side and d) south side.

3.3 Rock mass geotechnical parameters

Geotechnical parameters were determined by combining field and laboratory data using classification systems. The GSI value was determined using the quantified GSI chart (Hoek et al., 2013), where RQD and the J_{Cond89} parameter from the RMR classification (sum of scores for continuity, aperture, roughness, weathering and infilling of discontinuities) were used as input parameters. This approach is

based on descriptive but measurable indicators and reduces the level of subjectivity in determining the GSI value. RQD was estimated from the volumetric joint count (J_v) according to Palmstrom (2005).

The adopted representative value was GSI = 60. The parameter m_i was adopted with a value of 11, which is consistent with recommendations for limestone. The uniaxial compressive strength was determined as the average value obtained from laboratory testing, also taking into account values estimated from point load tests.

Along with the strength parameters, the Hoek–Brown failure criterion was used to estimate the deformation modulus of the rock mass. Its value was determined using the equation proposed by Hoek and Diederichs (2006). For this purpose, a modulus ratio (MR) value of 400 was adopted. Based on the obtained Hoek-Brown parameters, equivalent Mohr-Coulomb parameters were determined for the corresponding stress levels.

All adopted geotechnical parameters are presented in Table 2.

Table 2. Adopted geotechnical parameters for limestone rock mass and discontinuities

γ kN/m^3	c MPa	φ (^{o})	σ_i MPa	m_i	GSI	JRC	JCS MPa	φ_r (^{o})	E_{rm} MPa	ν
26.2	0.662	57	45	11	60	9	35	29	9360	0.25

3.4 Wedge stability analyses

Based on the results of the kinematic analysis, a detailed stability analysis of potential wedges was performed using the Swedge software (Rocscience Inc). The eastern slope was identified as critical, where the highest probability of wedge formation was observed and visitor exposure is greatest. A total of 24 discontinuities identified using the photogrammetric method were included in the model (Fig. 6).

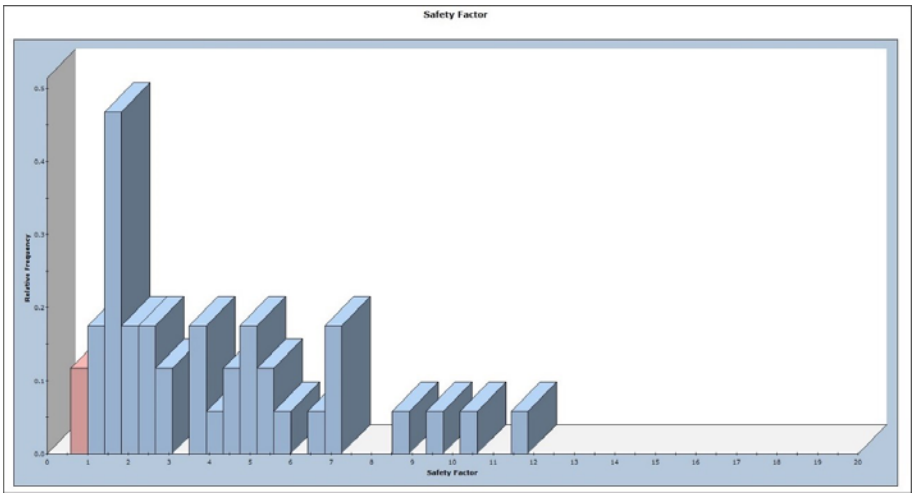


Fig. 6 Histogram of the factor of safety distribution for formed wedges.

The combination analysis resulted in the formation of 41 potential wedges, of which only one shows a factor of safety lower than 1. The critical wedge has a volume of approximately 0.05 m³. A larger number of blocks with similar volumes show factors of safety close to the limiting value, indicating the possibility of local block detachment, especially after intense rainfall and under seismic conditions.

3.5 3D rockfall analysis

The rockfall analysis was performed using the RocFall3 software (Rocscience Inc.). The simulation was performed for blocks whose volumes were defined based on detailed inspection of the point cloud as well as the results of the wedge stability analysis. The largest percentage of potentially unstable blocks falls within the interval of 0.02 to 0.05 m³. Based on the known volume and the unit weight of the rock mass, block masses used in the analysis were calculated.

The determination of microlocations (source areas) from which rockfall could initiate was performed through a combination of photogrammetric analysis and, to a lesser extent, field mapping. In order to

consider the most unfavorable scenario, rockfalls were initiated from the highest elevations within the identified unstable zone, thus assuming the most critical case.

In addition, characteristic input parameters were defined, including the properties of the slope material and blocks, as well as the initial movement conditions. The slope was modeled as limestone, with a normal coefficient of restitution Rn = 0.5, a tangential coefficient of restitution Rt = 0.9 and a friction angle φ = 35°. The blocks were divided into two groups – large and small – in order to examine their behavior depending on mass and dimensions. Large blocks were defined with a mass of approximately 130 kg, while small blocks had a mass of about 50 kg. For both groups of blocks, an initial translational velocity of 1.5 m/s was adopted, while the initial rotational velocity was neglected (0°/s).

In order to ensure representative results, a larger number of blocks per group was used in the analysis, which enabled obtaining a statistically reliable distribution of trajectories, energies and runout zones.

The following figure (Fig. 7) presents the results of the analyses obtained for smaller blocks with masses up to 50 kg.

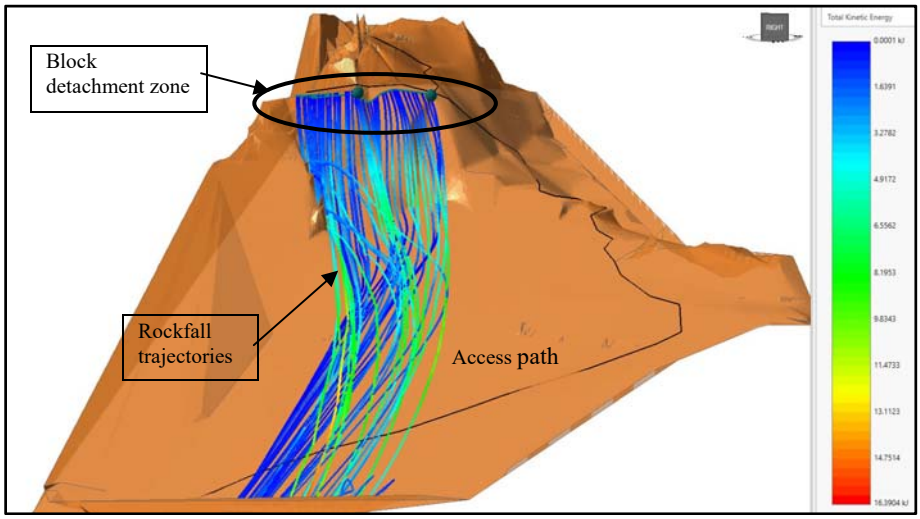


Fig. 7 Trajectories of detached blocks on the eastern slope with distribution of kinetic energy.

Most blocks with masses up to 50 kg reaching the access path exhibit relatively low kinetic energy values, typically below 3 kJ. However, a limited number of blocks attain significantly higher energies, with some cases exceeding 10 kJ.

An additional rockfall simulation was performed for larger blocks with masses up to 130 kg. The obtained results indicate significantly higher maximum kinetic energy values exceeding 20 kJ, while individual cases reach up to 30 kJ. Higher block velocities were also recorded, indicating a significantly greater potential destructive effect.

Based on the previously conducted analyses, a final simulation was performed in which the installation of a protective barrier was modeled along the section of the pedestrian path that could be reached by detached blocks. The barrier was dimensioned according to the characteristics of potentially released blocks and their movement trajectories in three-dimensional space. It was installed along a length of approximately 21 m (Fig. 8), with a height of 2.5 m and a designed energy absorption capacity of 35 kJ, consistent with the energy levels obtained from the previous analyses.

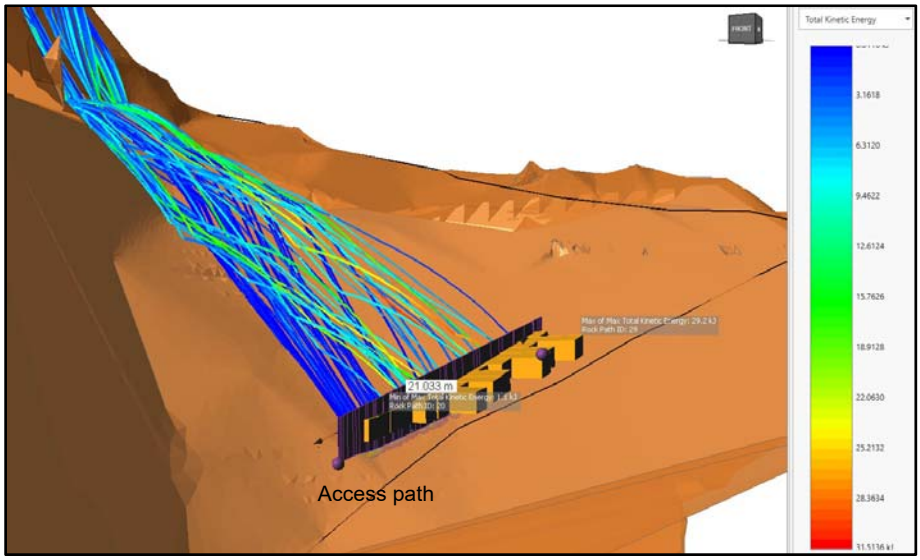


Fig. 8 Simulation of block trajectories and interaction with the protective barrier.

4 Conclusion

This paper presents the results of geotechnical investigations conducted for the reconstruction of the medieval town of Mileševac. Due to the limited accessibility of the site, the limestone rock mass on which the fortress is built was surveyed using drone-based photogrammetry. A scaled and oriented point cloud was generated, enabling detailed structural analysis. By combining the results of field investigations, point cloud analysis, and laboratory testing, geotechnical parameters were defined for the performance of a three-dimensional rockfall analysis. The results indicate that the potentially unstable blocks are relatively small and exhibit low kinetic energy, suggesting that they can be effectively intercepted by rockfall barriers with relatively low energy absorption capacity. The analysis further shows that, for the selected barrier location, the highest loads induced by block impacts are concentrated in the central and upper parts of the barrier structure.

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