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REINFORCEMENT AND PROTECTION OF A ROCK-CLIFF: METHODOLOGICAL APPROACH, ANALYSIS AND DESIGN

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ABSTRACT

This paper outlines the basic principles for the analysis and design of protection measures of a rock cliff against rock toppling and rock falls. The investigated case study is a rock cliff, characterized as natural monument, at the entrance of a village at Cyprus. Due to a considerable number of project restrictions, it was decided that the measures should be of passive character. The proposed measures include installation of a rock barrier of well-defined geometry, position and energy capacity, local strengthening of the existing fence, excavation of a shallow-depth, energy-absorbing ditch and localized installation of a protective rock retaining wire mesh. However, in cases where the demand or the financial surcharge of these measures is unjustifiable, active measures are proposed. All possibilities of rock instabilities are thoroughly checked (plane / wedge sliding and rock toppling) under static, extreme hydraulic and seismic conditions. Results from numerical analysis, carried out at a critical cross section, corroborate results from ultimate limit state analyses, under static and seismic conditions.

Keywords: rockmass, fence, rock bolt, limestone, toppling, barrier.

INTRODUCTION

At the entrance of Episkopi village of District Administration Office Paphos, at Cyprus, an enormous rock-cliff consisting of a thick bedded limestone is located, where rock toppling and other kind of rock instabilities and hazards have occurred; usually at the end of the wintertime, after intense rainfalls or occasionally during seismic events. Given that this rock-cliff is a source of meaningful hazard, due to rock toppling and other kind of rock instabilities, added to the fact that it has been characterized as a natural monument and it is a tourist attraction for the community, it was decided by local authorities that protective measures should be adopted in order to make safe transit of walkers and residents, as well as any kind of transit of vehicle. For this reason, a sophisticated project was assigned to the Institute of Engineering Seismology and Earthquake Engineering, located at Thessaloniki (Hellas) in order to elaborate a number of appropriate countermeasures that would increase the level of safety and could apply to this case study, taking into account the technological abilities of the local market and the local site conditions, not especially convenient for any kind of protective measures. Given the number of heterogeneous constraints such as technological, financial, social, cultural and

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environmental, the choice of an acceptable solution is a real challenge in terms of methodological approach and deployment of analysis, design and constructability.

SCOPE OF THE WORK

The main goal of this work is the adoption of appropriate measures to ensure transit of residents, and vehicles, as well as the safety of residents, without major alteration of the natural environment, which is preserved as a natural monument. The prevailing measures for this case study are of passive character, i.e. protection against hazards resulting mainly from rock toppling and rockfalls, by installation of a protective system.

The entity of measures adopted for rock cliff reinforcement, stabilization and protection from rockfalls or rock toppling, can be described as follows:

1. Installation of a new rock barrier for protection against rockfall hazards.
2. Placement of rockfall protection netting, fulfilling special requirements, on a limited surface of the rock-cliff, oriented towards North.
3. Selective reinforcement or stabilization of unstable / potentially unstable rock masses of the extended east front of the rock cliff.
4. Selective operations aiming to the reinforcement or stabilization of potentially unstable rocks located at the broad area of the crown of the scarp. The above operations consist of rock-blasting, in situ rock fragmentation combined with transfer to a safer location, or improvement of their foundation, by rock bolting or by concrete.
5. Formation of a shallow depth energy-absorbing ditch, between an existing fence and the proposed barrier, which will be backfilled with clayey soil or mixture of a sandy-clayey soil, planted with bushes or trees.

LOCAL GEOLOGICAL, GEOTECHNICAL AND SEISMOLOGICAL CONDITIONS

The steep rock cliff at the entrance of the village of Episkopi, District Administration Office Paphos at Cyprus, consists of a thick-bedded hard limestone of high mechanical parameters. A network of structural discontinuities, such as bedding, fractures and joints affect unfavourably the mechanical behaviour of rockmass, as the shear strength along structural discontinuities is drastically reduced. Rock instabilities (i.e. plane failure, wedge failure, rock toppling) are also occasionally generated, whenever the combination of structural discontinuities orientation and the rock slope inclination results in favourable kinematics, and as well as, when shear resistance developed along some structural discontinuities, is smaller to destabilizing actions applied on the above tectonic discontinuities.

The main scarp oriented towards East is of an approximate length of 250m and a height fluctuating between 30 and 70m. An assiduous tectonic and geological investigation of the rock frontage resulted in a segmentation of the examined area into five (5) different sections (B, B', C, D and E: figure 1). In each of these sections, a statistical elaboration of tectonic measurements of structural discontinuities allowed the stability analyses, which are dependent on the occasionally fulfilled kinematical conditions. In each case, the specific sites were located where the different types of rock instability are prone to appear, and a quantitative check of stability is demanded. Another useful result was the localisation of rock instabilities on the tectonic mirror, whereas cautious observation permitted the conclusion that passive measures should be encountered for isolated rock fragments of volume $\leq 4m^3$.

Two geotechnical boreholes located at the crown and the toe of the scarp near the northern part of the rockmass, allowed selection of rock cores. An appropriate program of laboratory tests was carried out in order to calculate the uniaxial compression strength of intact rock cores, Young modulus and Poisson ratio of intact rock cores and shear strength parameters of discontinuities on rock samples.

Geotechnical characterization of the rockmass classified it in class IA-IB (very good quality) and locally in class II (slightly weathered). However, in a few shear zones, where rockmass is heavily weathered, it is, classified into class IV or V, unlike the general trend. Classification of the rockmass according to the RMR (Rock Mass Rating) system resulted in values: RMR_{89} = 50 to 70 which also correspond to a rockmass with good to very good mechanical behaviour. By taking into account the results from the laboratory tests performed on intact rock cores, the mean value of the intact rock core uniaxial compression strength is σ_{cm}=57.0 MPa, whereas the statistical min and max characteristic values for a confidence level of 95%, correspond to 43.0 and 71.5MPa respectively.The following mean values were derived from the direct shear tests performed on structural discontinuities of the rock samples: c'_m=0.31MPa και $\varphi'_m=26.5^0$. Min and max characteristic couple of values for a confidence level of 95%, are the following: c'_{min}=0.20MPa, $\varphi'_{min}=21.5^0$ and c'_{max}=0.42MPa, $\varphi'_{max}=31.5^0$ respectively. The above values,appropriately scaled will be used as design values for stability analyses developed on a second stage.. The aforementioned geotechnical boreholes have been carried out within the limits of sections C and D, where rock instabilities have been registered in the recent past and the quality of the rockmass is slightly inferior to the general trend. Therefore, the resulting design values of the mechanical parameters are rather conservative as they are used for the entire rockmass stability analyses.

Lastly, based on the seismological data of the broad area of interest, two different models concerning seismic sources have been used (PSM: Point Source Model, and FSM: Finite Source Model) in order to carry out a seismic hazard analysis and assessment of seismic scenarios of the project area. It resulted that PGA target for rock outcrop conditions for a mean return period of 60 years with a possibility of exceedance 10%, is calculated equal to **0.26g**. Stability analyses under seismic conditions have been carried out in conformity to EAK 2000 (Hellenic Seismic Code): for this purpose the free field effective ground acceleration for rock outcrop conditions was crudely approximated as 75%xPGA = 0.75x0.26g ≅ 0.20g. The horizontal component was then: $a_h=0.5xa_{max}$=0.10g and the vertical $a_v=\pm0.25a_{max}=\pm0.05g$. Pseudostatic stability analyses under seismic conditions have been carried out for the critical cross sections with the above additional inertia horizontal and vertical forces, whereas, for the most critical one (cross section T9) a numerical analysis based on a finite difference software code (Flac 2D, V5.0) was employed to refine stability results.

METHODOLOGICAL APPROACH AND ANALYSIS PRINCIPLES

According to the data collected and produced for the examined area (i.e. topographical drawings, cross sections of the rockmass facing, geological and tectonic measurements, geotechnical investigation, probabilistic seismic hazard of the broad area), it has been clarified that the major rock hazard is rock toppling. However, other type of rock hazards, such, as wedge and plane failures exist at specific locations of the rockmass, despite of its very good quality. Rotational or multi-wedge failures are not susceptible to occur due to the rockmass texture.

Therefore, a systematic search of all potential rock hazards should take place, after designation of the design values used for relevant analyses. Namely, for instabilities prone to occur along structural discontinuities (plane and wedge failure), the design values of shear strength parameters are calculated according to Barton criterion (Barton, 1976; Barton and Choubey, 1977):

$$\varphi_\kappa = 28^0 \quad \text{and} \quad c_k \cong 15.0 \text{ kPa}$$

The above values lean on rock mechanics laboratory tests and geological – tectonic measurements of structural discontinuities (basic friction angle φ_b, JRC_o, JCS_o, length of discontinuities L_n and normal stresses applied σ_n). The effect of scale was taken into account via appropriate modification of coefficients JRC_n (Joint Roughness Coefficient for discontinuities length L_n) and JCS_n (Joint Compression Coefficient for discontinuities length L_n) according to Barton and Bandis (1982).

Figure 1 Horizontal layout of the examined area with deployment of generalized passive protection systems

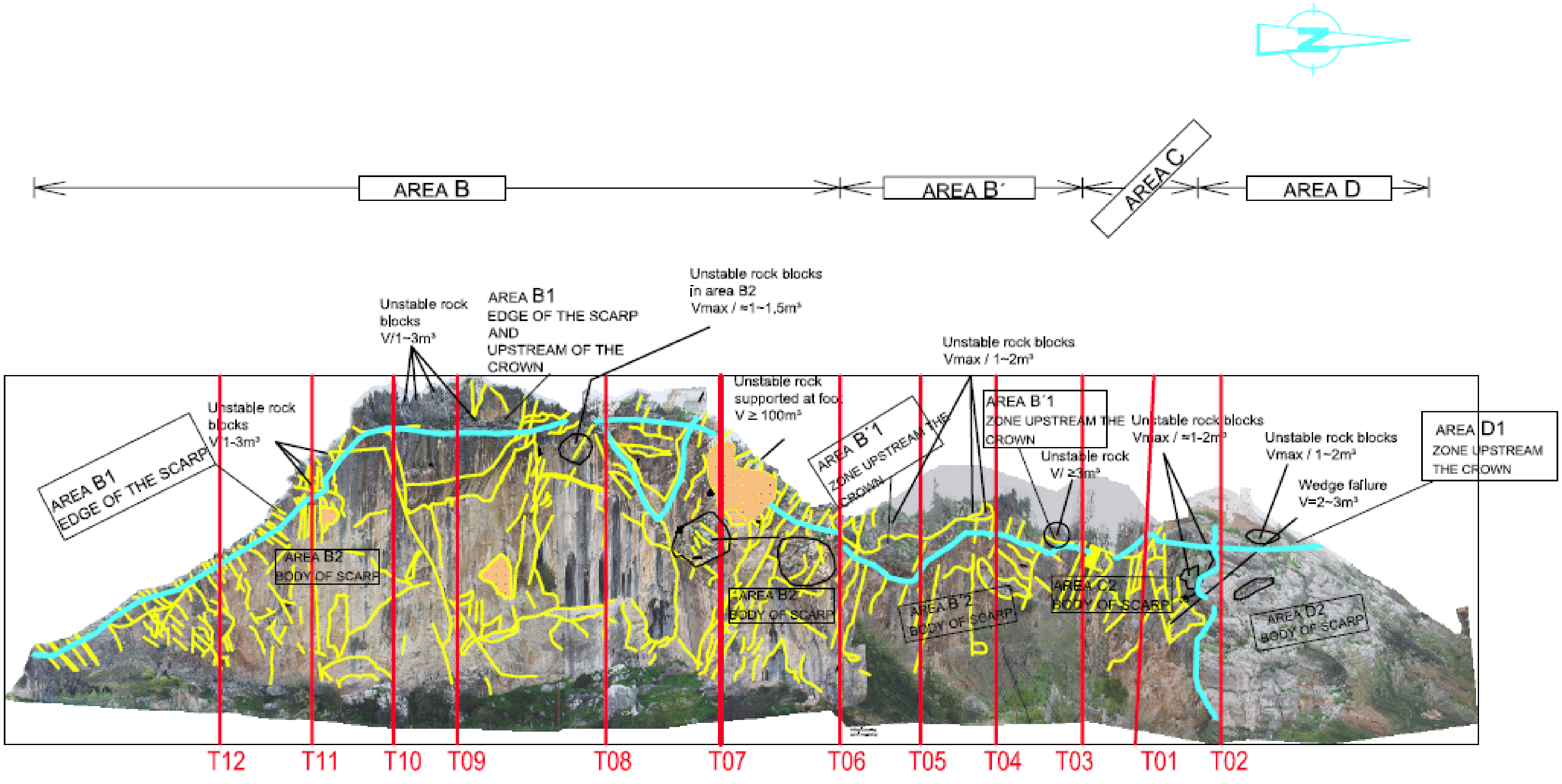

Figure 2 Development of the rockmass face view from east, combined with orthophotography of the scarp

Stability analyses focusing on plane and wedge sliding are based on Hoek and Bray (1981) principles. Three combinations of loading conditions are adopted according to DIN 4054 and are summarized in Table 1 with required safety factors per case.

Table 1 Minimum required factors of safety for rock slopes per loading combination

Combination number	1	2	3	4
Earthquake	Y	Y	N	N
Extreme hydraulic conditions (50 years)	Y	N	Y	N
Required F.S.	/	1.0	1.2	1.3

Earthquake: Y corresponds to design earthquake according to EAK 2000; N means no seismic conditions.
Extreme hydraulic conditions: Y corresponds to the highest expected pore pressure of structural discontinuities (saturation of discontinuities of 50 years: 30%); N corresponds to yearly highest pore pressure (estimated percentage 0%, since limestone is a pervious material).

As for rock toppling hazard, tests of interception of toppling boulders or rock blocks by a new rock barrier have been achieved by specialized software RocFall V4.043 (Rocscience, Inc). These tests have been carried out to almost all of the cross sections (T1 to T13 and NTA; see figures 1 & 2) in an attempt to take into account variations due to geometry, texture and shape of the rock toe and downstream geomaterials, combined with technical demands of the project.

In order to quantify all possible trajectories potentially followed by boulders, an important number of tests (software runs) were carried out, as follows:

- For a generated random number of rock blocks (volumes of 1, 2 and 4m^3 are examined) detached from the rockmass facing or from the scarp crown, trajectories are solved based on classical physics equations
- Geometry of the examined cross section is entered as data for a sufficient length, in such a way, that all locations with potential unstable rock fragments, kinematically prone to topple, are included. Linear segments simulate toppling trajectories. Characteristic properties for each linear segment are used in the framework of the equations solved.

Characteristic properties that are used for simulation of the trajectories are: a) friction angle φ (deg), b) coefficients of normal (R_n) and tangential (R_t) restitution, and c) slope roughness (equal to inclination angle of each segment). Table 2 presents the design values of these properties.

Table 2 Coefficients of restitution, friction angle and slope roughness of different segments

Geomorphologic conditions	R_n / Std	R_t / Std	φ (deg) / Std	Roughness / Std
Smoothed limestone	0.45 / 0.05	0.90 / 0.05	30 / 2	Segment inclination / 1
Rough limestone	0.45 / 0.05	0.90 / 0.05	30 / 2	Segment inclination / 2
Talus No1	0.32 / 0.04	0.82 / 0.040	29 / 2	Segment inclination / 4
Talus No2	0.31 / 0.04	0.81 / 0.04	28 / 2	Segment inclination / 3
Talus covered by vegetation	0.30 / 0.04	0.80 / 0.04	28 / 2	Segment inclination / 5
Absorbing-energy ditch soil cover	0.30 / 0.04	0.80 / 0.04	28 / 2	Segment inclination / 3
Road asphalt	0.40 / 0.04	0.90 / 0.04	30 / 2	Segment inclination / 0

The methodology that was adopted in order to assign the geometric and energy needs of the new rock barrier is summarized in the following steps for every cross section tested for rock toppling hazard:

- Initial positioning of a new rock barrier taking into account the different components of the project.
- Tests for three different rock blocks of volume 1, 2 and $4m^3$ representative of site conditions under static conditions.
- Repeat of the above tests for seismic conditions (simulation of local seismic conditions by assignment of an initial horizontal speed u=1m/sec within a standard deviation of $\pm$0.5m/sec) to rock blocks prone to toppling.

For each test, a random number of 1000 detached blocks was examined per cross section and per rock volume. The analyses results provide maximum energy demand and maximum rebounding height of the boulders, which determine the required energy capacity, the positions, and the height of the rock barrier.

STABILITY ANALYSES RESULTS

Plane sliding

Plane sliding analyses were performed based on the loading combinations of Table 1. According to tectonic measurements, the most unfavourably structural oriented discontinuities satisfying kinematical instability for plane sliding, with the biggest length and the densest occurrence, were examined. Simulation of extreme hydraulic conditions corresponds to filling of the discontinuity with water up to 30% of its total length, whereas seismic conditions are simulated by a free-filed peak ground acceleration of 0.26g for rock outcrop conditions.

A synoptic overview of plane stability analyses results is summarized in the following conclusions:

1. Areas B, B' and E do not satisfy kinematical conditions.
2. In the area C, plane sliding kinematical conditions are satisfied. Volumes that could be detached, based on tectonic measurements could be easily dealt with passive nature measures, such as the new rock barrier.
3. Finally, in area D, even though kinematical conditions are favourable for plane sliding occurrence, the resulted factors of safety are high enough and no visible risk of plane sliding is reported.

It is therefore concluded, that plane-sliding hazard is of minor interest since it occurs sporadically. Nevertheless, whenever it does, passive measures in the form of a rock barrier designed for rock toppling needs, are sufficient enough to face this hazard, for rock blocks inferior to $4m^3$.

Wedge sliding

Wedge sliding analyses were also examined in all cross sections of the rockmass. In this way, areas B, B', C, D and E, were carefully tested against wedge sliding. Whenever kinematical conditions for wedge sliding occurred, stability analyses were carried out based on tectonic observations and laboratory tests scaled in order to specify shear resistance along wedge sides. A synopsis of wedge analyses results concludes that areas B and B' are the most vulnerable for wedge sliding. Mainly, the overhanging part of the main scarp, in the interval of cross sections T5 and T7 (figures 1 & 2), suffers the most. In a marginal way, interval between cross sections T10 and T12, also suffers from wedge sliding hazard (figures 1 & 2). However, in few cases where calculated factors of safety were not satisfactory (less than requirements of Table 1), it turned out that the rock barrier, foreseen for interception of rock toppling, is a sufficient preventive/protecting system, provided that the wedge volumes are less than $4m^3$. However, in the extreme case of a wedge of 8 or even $10m^3$ that might be detached from the vertical rock scarp (see figure 2), it could be well assumed that it is going to break into pieces that would not exceed $4m^3$.

The only exception to the aforementioned is a potential unstable wedge of enormous volume, crudely approximated around $150m^3$, near cross section T7 (figures 2 & 3). In this case, rock barrier is not a safe protection system; therefore, measures of active character such as, reinforcement and anchoring of the toe of the voluminous wedge, need to be adopted to improve safety.

DESIGN OF PASSIVE PROTECTION SYSTEMS

The prominent rockmass emerging as a tectonic mirror is characterized as a natural monument. The adopted policy for the design of the whole project consisted mainly of the development of protective systems that would ensure safety despite the number and randomness of the hazards related to the project. Anyway, the adopted protective systems should not alter shape and character of the preserved natural monument. Therefore, the development of passive protection systems seems to be the most appropriate solution for this project, since they do not affect the physiognomy of the natural monument, whereas they could withstand natural deterioration of the monument and its implications. Another semantic privilege opting for development of passive protection systems is the financial aspect of the problem: given the randomness of the hazards, a large number of active systems should be applied with ambiguous efficiency in terms of safety demand, for a disproportioned amount of budget. The basic trendline of the design focused in the development of passive protection systems, appropriately adapted to the project requirements. The main parts of the solution were as follows: installation of an appropriate rock barrier system, local strengthening of the existing retaining fence, construction of a shallow depth energy- absorbing ditch and placement over a steep rock slope of a protective heavily galvanized maintained on place by fully bonded rock bolts.

Locally, whenever generalized passive protection systems were not sufficient or demand of safety level induced outrageous increase of finance or local over-design, and then active protection measures were taken in an attempt to overcome a localized, deterministic problem.

Rock barrier system

Based on a considerable number of rock toppling analyses, which were carried out, on the cross sections, three different categories of a new rock barrier have been considered satisfying energy and geometrical criteria. The following categories were positioned for different lengths downstream the existing fence (figure 1) on a south to north direction:
Section NF-1: length L=60m, height H=3m, capacity of energy absorption E=500kJ
Section NF-2: length L=40m, height H=4m, capacity of energy absorption E=500kJ
Section NF-3: length L=60m, height H=3m, capacity of energy absorption E=500kJ
Section NF-4: length L=50m, height H=3m, capacity of energy absorption E=1000kJ
Section NF-5: length L=40m, height H=3m, capacity of energy absorption E=500kJ

Location of the new rock barrier system is depicted in figure 1. The new rock barrier forms an angle of 10^0 with the vertical towards downstream. Details of the proposed rock barrier with technical specifications are presented in figure 3.

Strengthening of existing fence

The contribution of the existing fence was taken into account in the design of the new rock barrier. However, for a small part, in the interval of cross sections T5 to T2, strengthening of the existing fence for an approximate length of 50m was considered as necessary, in order to avoid financial surcharge by a meaningful increase of energy demands of the new rock barrier. The strengthening is going to be implemented by installation of diagonal wire ropes between vertical posts.

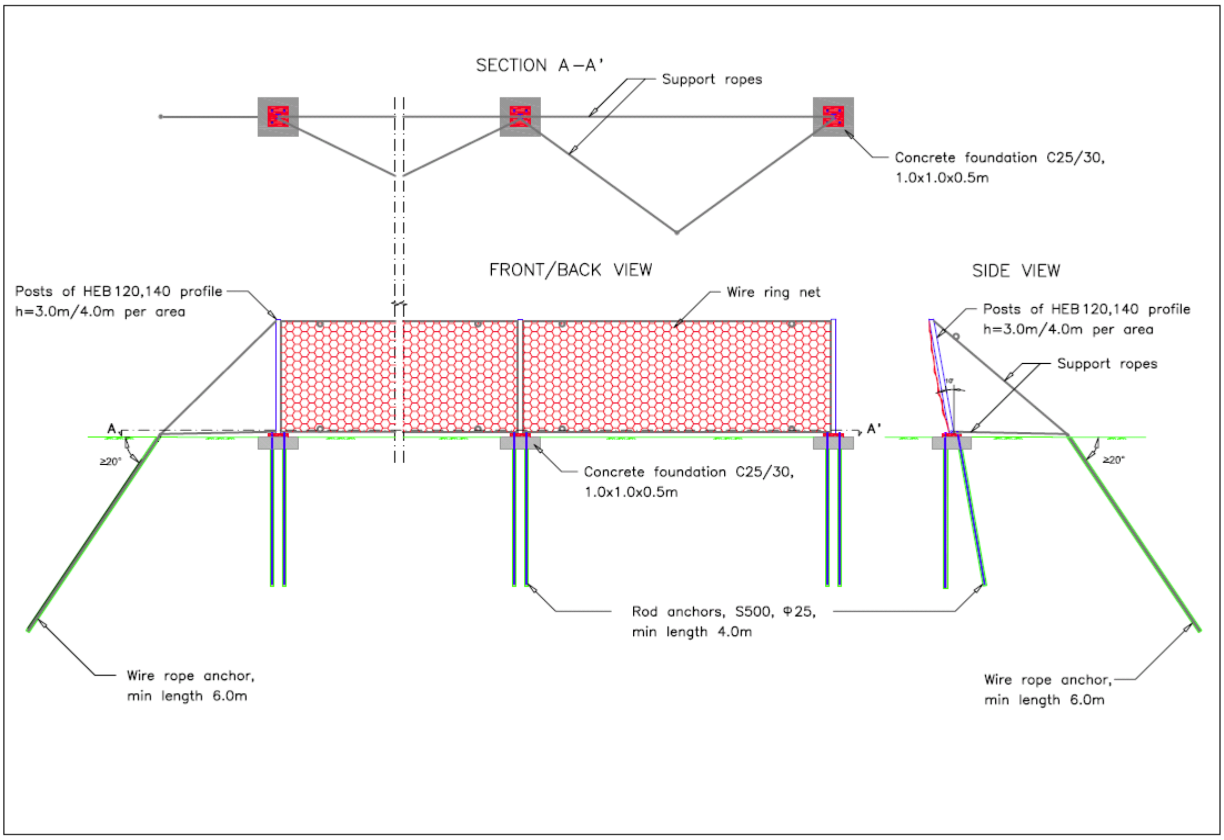

Figure 3 Front / back view, side view and horizontal layout of a rock barrier designed for passive protection against rock falls and rock toppling

Energy-absorbing ditch

The proximity of the existing road restricts positioning of the proposed rock barrier. Thus, in an attempt to reduce energy and geometrical requirements of the new rock barrier, the construction of a shallow depth ditch of approximately 80cm is proposed. A typical cross section of the proposed trench is presented in figure 4.

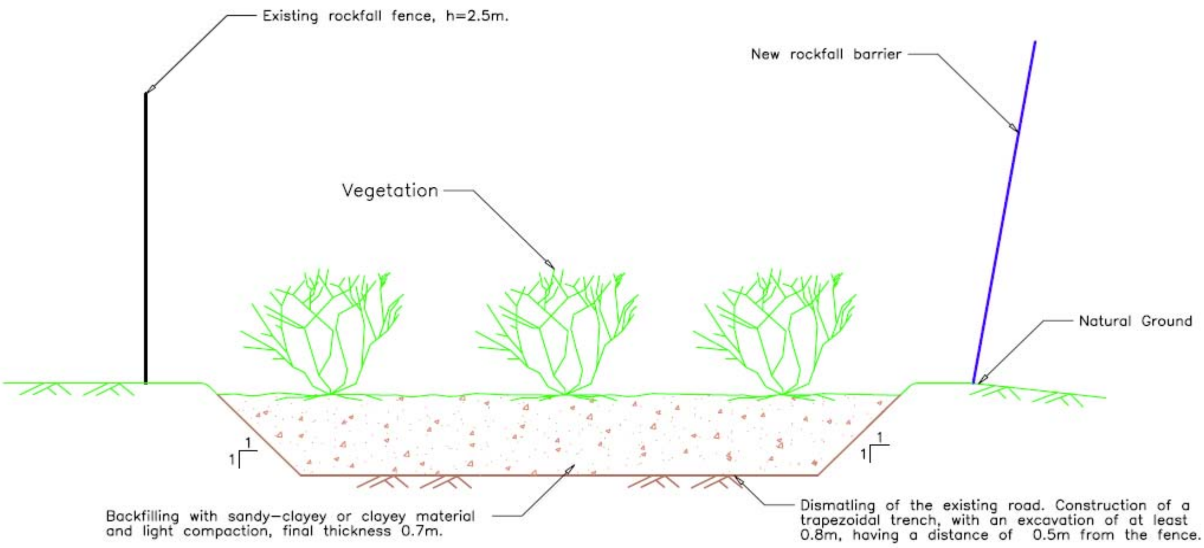

Figure 4 Typical cross section of shallow depth (0.8m), energy-absorbing ditch.

Protective retaining wire mesh

Analyses against different types of hazards (mainly rock toppling) for the areas D and E, corresponding to NE and N part of the rockmass, indicate that these areas are not sufficiently protected by the proposed rock barrier and that the north part is not protected by any fence or rock barrier. Also, existence of housing, proximity of the existing road and the ruins of the old church, are considerable reasons to intensify all necessary protective systems to ensure safety. Albeit, plane and wedge failures are not justified based on the analyses carried out under all load combinations (see Table 1), inhabitants testified that a number of rock fragments and boulders were destabilized and thus felt or toppled usually at the end of the winter time, or occasionally, after some seismic events.

Taking into consideration the above, as well as the imposed project restrictions, installation of a retaining wire mesh, was considered as best solution. Rock bolts spaced on a grid of 2.5x2.5m and a minimum length of 3m ensures retention of the mesh. Rock bolts are passive anchors, fully bonded in the rockmass, consisting of high strength, steel S_{500} and a diameter of 25mm. All metallic elements of the retaining system are heavily galvanized and protected by a mixture of Zn and Al. A typical cross section of the retained area after installation of the wire mesh is depicted in figure 5.

Figure 5 Cross section of rockmass protected by wire mesh kept in place by a grid 2.5x2.5m of 3m length rock bolts, fully bonded into the rockmass

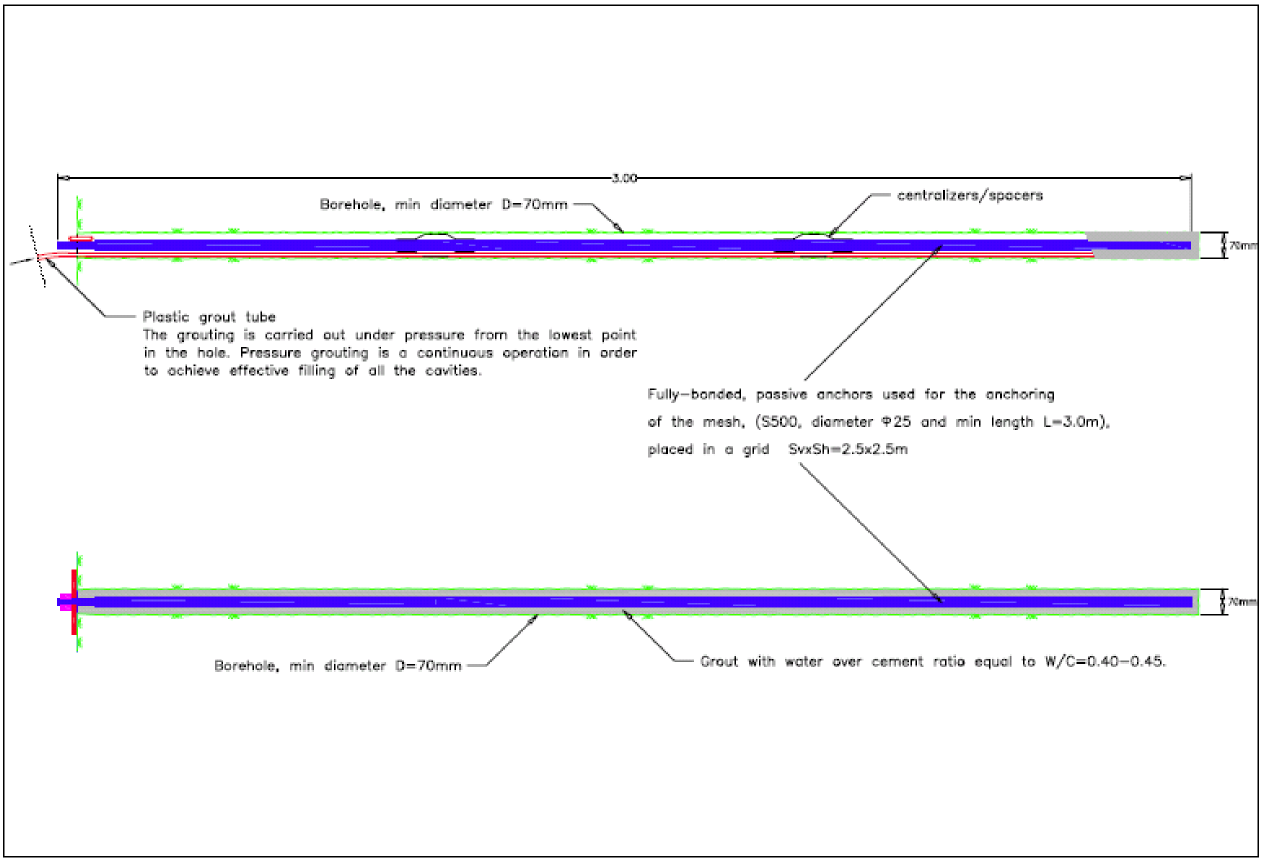

Figure 6 Details relevant to construction of fully bonded rock bolts used to keep in place protective retaining wire mesh

The density of the proposed grid is derived based on the rock bolts capacity. The required diameter results from imposed restrictions (Stillborg, 1994; Papaspyrou, 1985): the cross section of the rebar should not exceed 20% of the borehole cross section in order to avoid concentration of shear stresses. Grout cover of the rebar must be at least 20mm. Consequently, a borehole of 70mm is deemed necessary. After implementation of all tests (rebar failure, sliding along grout-tendon interface, sliding along grout-rockmass interface, rockmass failure from rock bolt pullout test both checked as a stand alone unit or as a group) the minimum required bonding length was calculated to 1.25m. Considering the fact that a surficial zone of approximate 1.5m should be considered as disturbed, design bond length is proposed at 3m.

DESIGN OF ACTIVE PROTECTION SYSTEMS

In the present case study, the use of active protection systems are recommended either when there is an outrageous demand of passive protection systems which is unacceptable with the character of the project, or unjustifiable increase of financial surcharge. A characteristic case is that of a potential unstable wedge or better expressed as a wedge of potentially reduced level of safety, with an enormous estimated volume crudely approximated at $150m^3$. By implementation of back analysis under static conditions, a stability analysis was performed for a wedge formed by two tectonic discontinuities simulated as following: J1($40^0/70^0$) and J2($90^0/70^0$), a slope plane ($64^0/90^0$), an upwards plane slope ($64^0/35^0$) in terms of dip/dip direction, so that a wedge of almost 4130kN weight is formed. Assuming that shear characteristics of structural discontinuities are: $\varphi=28^0$ and cohesion $c=28kN/m^2$, back analysis under static conditions, results in a safety factor $F_S \cong 1.04$. In order to achieve an adequate factor of safety under seismic conditions, a horizontally applied anchoring strength of 2850kN is required. Analyses were carried out by SWEDGE software (Rocscience Inc.).

Construction difficulties lead to the selection of self-drilling anchors made of stainless austenitic steel, of diameter Φ40/16mm (40mm: outer diameter and 16mm: diameter of inner hole of the anchor). The proposed anchors should have satisfactory length in order to safely transfer the loads in deeper locations of the rockmass, far away of the wide-open side gaps. By implementation of all necessary tests for the above anchors of an ultimate load of 660kN and yield load at 525kN, it was concluded that for working load under static conditions T_{WLS}=525/2=262,5kN, the adopted design value was 250kN. For seismic conditions, the working load was roughly approximated at a value of 150kN (T_{WLD}) for a magnitude of earthquake, M=6.5, corresponding to a number of cycles N=7 (Gazetas, 1995). The grid of the proposed active protection system is given in figure 7.

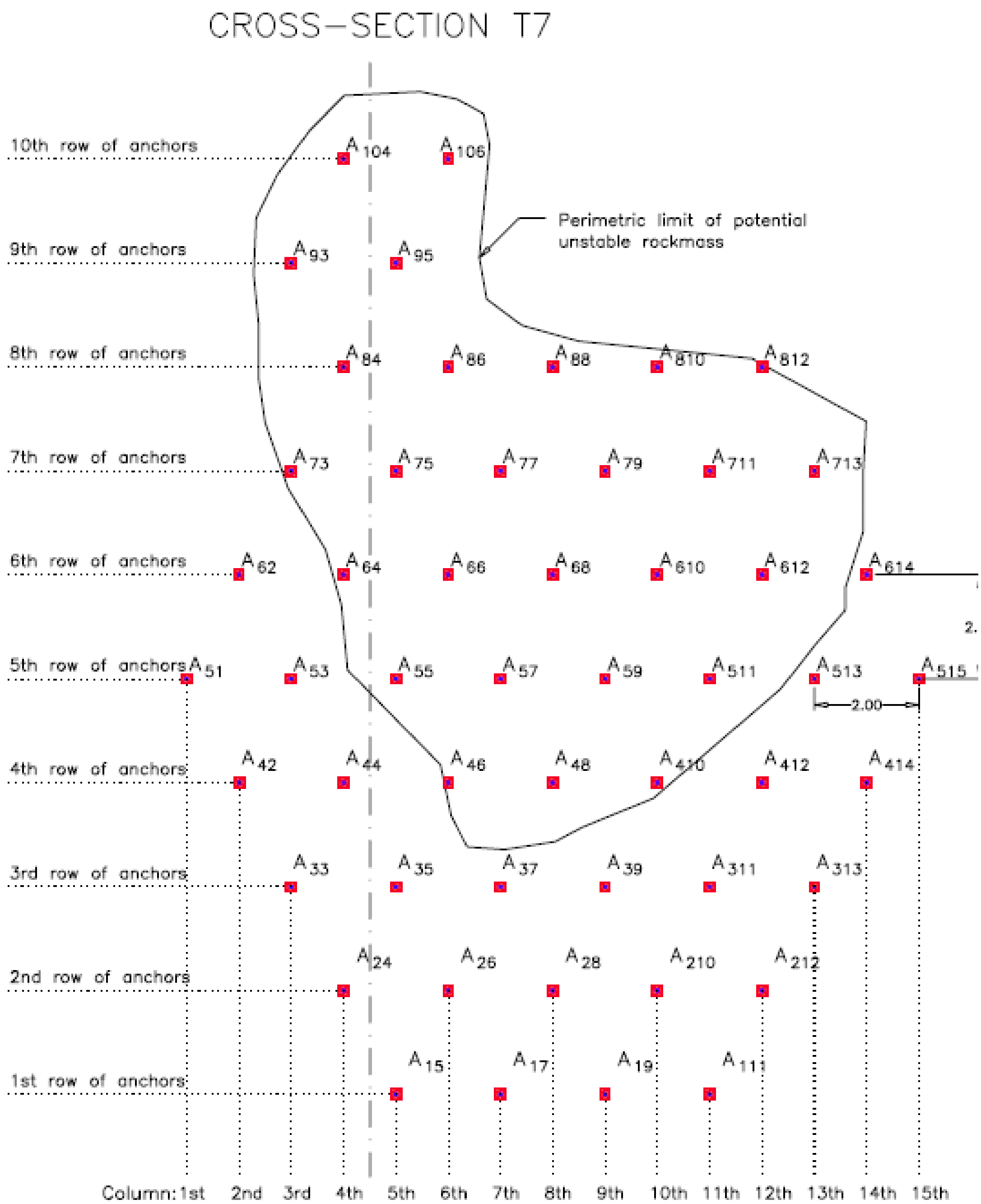

Figure 7 Face view of a potentially unstable rock wedge, near cross section T7; location and layout of rock bolts designed for stabilization and reinforcement.

Selective rock fragmentation & stabilization of unsafe boulders

The above works are part of the active measures foreseen for special cases in the vicinity of crown area. Rock fragmentation is undertaken in special cases where boulders located in the broad crown area and prone to fall, are broken into smaller fragments that could be intercepted by the rock barrier installed downwards. Fragmentation is undertaken either mechanically, or chemically.

Another type of selective operation aiming in a stabilization process of an unsafe boulder can be described by the content of figure 8. This figure shows improvement of foundations conditions and stabilization by use of passive compact anchors (rebar), made of stainless austenitic steel S_{500S}, fully bonded and of estimated length of 4 to 6m. Rebar diameter is Φ25mm, ultimate load of 270kN, minimum limit of yielding load 200kN, centrally positioned in borings of 60mm, and fully grouted.

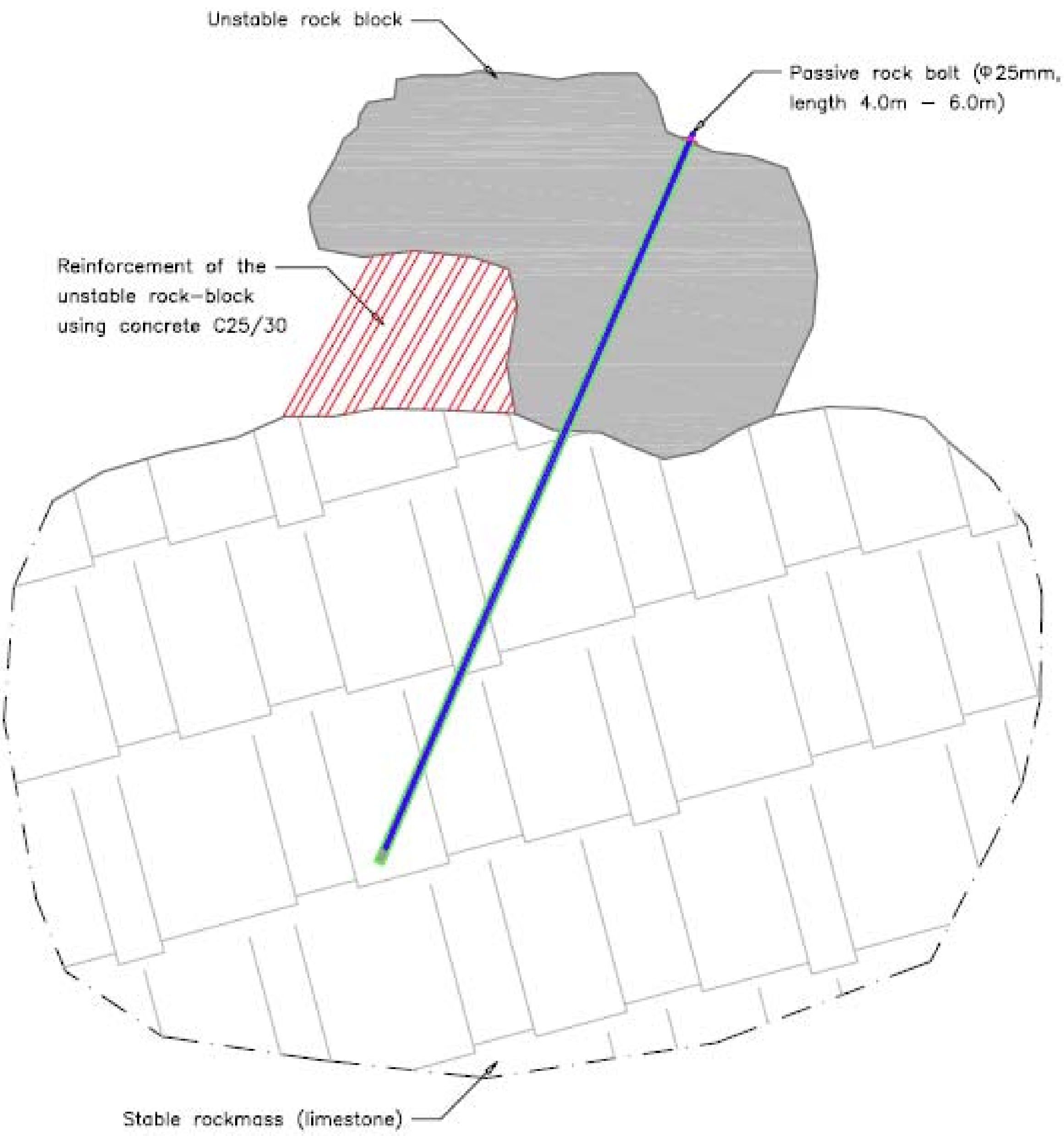

Figure 8 Selective reinforcement and stabilization of unsafe boulders at the area of crown

NUMERICAL ANALYSIS IMPLEMENTATION

In the framework of the present project a numerical analysis of cross section T9 was performed with FLAC v5.0, 2-D finite difference software oriented in geotechnical engineering applications. A non-linear, two-dimensional plane strain analysis was carried out on a detailed mesh consisting of 14500 elements. The scope of this approach was to estimate the anticipated values of stress and kinematics field and to specify the most vulnerable part of the rockmass, as well as kinematical field of the cross section.

After a thorough elaboration of geotechnical investigation and laboratory tests data, a non-associated Mohr-Coulomb law modelled soil cover, whereas a ubiquitous joint simulation inclined at 70^0 modelled limestone. A friction angle $\varphi=28^0$ and an apparent cohesion c=200kPa were used as mechanical properties of the limestone bedding. These shear strength design values were defined based on laboratory tests on rock discontinuities and the apparent cohesion high value was attributed to the combined effect of roughness and interlocking along discontinuities. In order to simulate seismic conditions, rotation of the gravity was applied for both horizontal and vertical seismic component according to the Hellenic Seismic Code as described for slopes (EAK 2000). In the upper part of the rockmass a tensile zone was observed, attributed to steep inclination of structural discontinuities and the low value stress field. As for the area beneath rock prominence, it seems that there are local zones of failure, which could explain small-scale detachments or rock falls during seismic events.

Resulting displacements, presented in figure 9, based on pseudostatic approach, are of the order of 2cm. The kinematical field could be attributed to the tensile failure on the left part of the figure. In any case the previous results do not indicate generalized or extended failures. Therefore, a different approach seems to corroborate results based on the initial point of view of local rock toppling and rock falls.

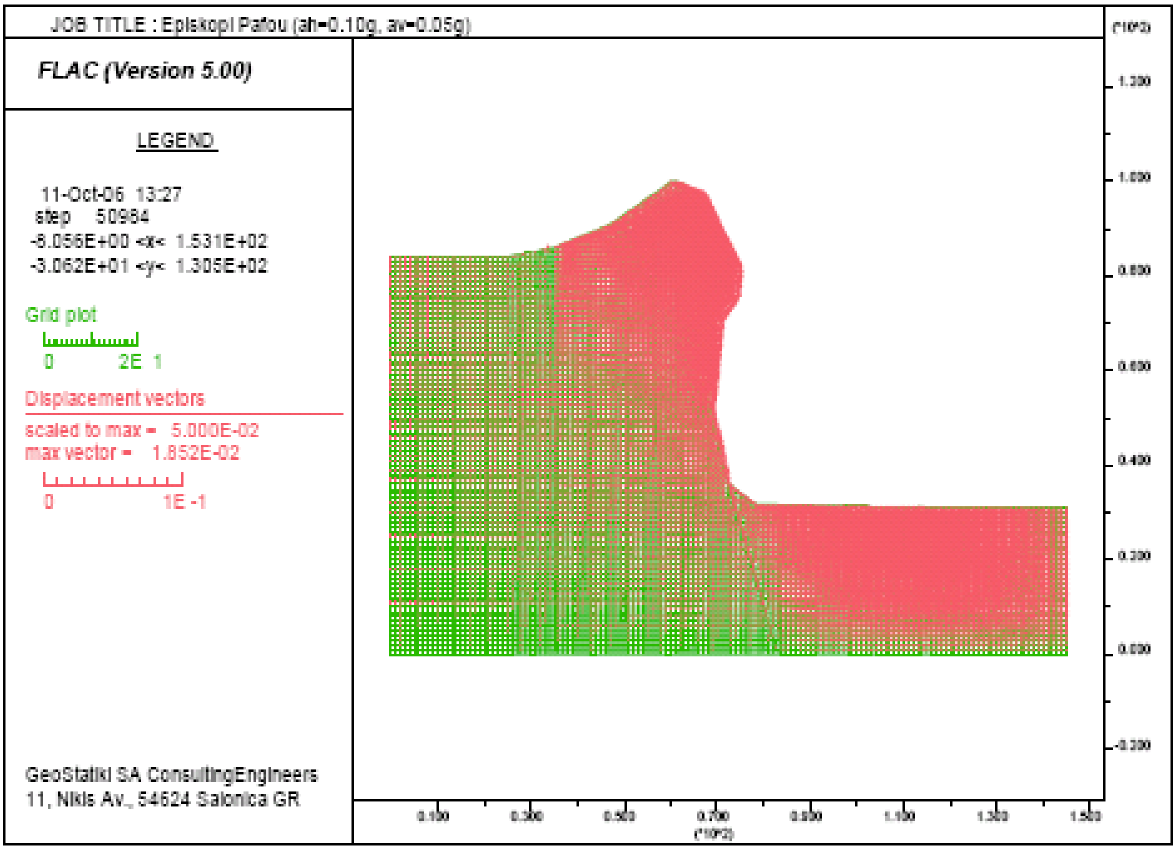

Figure 9 Kinematical field attributed to simultaneous effect of horizontal and vertical seismic action

CONCLUSIONS

Based on the available topographical survey, geological-tectonic, geotechnical and seismological data of the area of interest, an integrated research was carried out in relation with the analysis and design of protection measures. The main conclusions could be summarized as follows:

- The examined rockmass is a thick-bedded limestone of very good mechanical behaviour, intercepted by structural discontinuities of lower shear strength properties, which might determine its stability under kinematically favourable conditions.
- By consideration of all available data, as well as the complexity and randomness of possible hazards, the adoption of passive protection measures seemed to be the most reasonable solution, since the examined rock cliff is characterized as a natural monument. Installation of a rock barrier at a specified position of well defined geometrical and energy requirements, local strengthening of the existing fence, localized installation of a protective rock retaining wire mesh and construction of a shallow-depth, energy-absorbing ditch, constitute the prevailing measures of passive protection.
- Locally, for optimization and rationalization of the quantities and the financial surcharge, complementary active measures are proposed which include: reinforcement of an enormous, potentially unstable wedge with stainless steel, self drilling anchors of appropriate dimensions and installed on a suitable grid, as well as, selective fragmentation or stabilization of unsafe rock boulders at the crown area.
- All reinforcement, stabilization and protection measures, either active or passive, have been designed against toppling, falls and plane or wedge sliding, based on extensive stability analyses under static, extreme hydraulic and seismic conditions.
- A special numerical analysis of a critical cross section, based on a 2D plane strain hypothesis, concluded, using a different approach, that only rock toppling and falls of rock fragments are possible, and no generalized failure is justified under both static and seismic conditions.

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