# SEISMIC VULNERABILITY OF NATURAL STRUCTURES: STONE PINNACLES ON THE AMALFI COAST, A CASE STUDY.

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**ABSTRACT :**

The landscape beauty and the dangerousness of some stone formations with a cylindrical shape (pinnacles) leaning on the underlying National Road 163 (the Amalfi Coastal road) in Southern Italy, requested a seismic vulnerability study. Due to the erosion of the rock and the presence of sub-vertical cracking planes in the rocky slope, particularly close to Positano town, some pinnacles more than ten meters in height arose as a danger to the transit. Based on some inspections performed to have a detailed survey of the pinnacles, using the Laser Scanner 3D technique and with the collaboration of geologist-mountain climbers, numerical models and different structural analyses to assess their seismic vulnerability have been performed and compared: starting from simplified mechanical models of rigid blocks, passing through elastic analyses and finally to Finite Element (F.E.M.) analyses. Numerical dynamic and static analyses, particularly, the modal dynamic analyses for the elastic continuum and the non-linear static analyses, considering both cracking and plasticity behavior of the rock have been performed. The analyses concerned the pinnacles both under the actual in-situ conditions and according to some strengthening interventions to evaluate the effectiveness and safety of the design. The overall study involved several pinnacles with different shapes and different dimensions; in the present paper only one of them, the most relevant for its dimensions, location and structural issues, is presented.

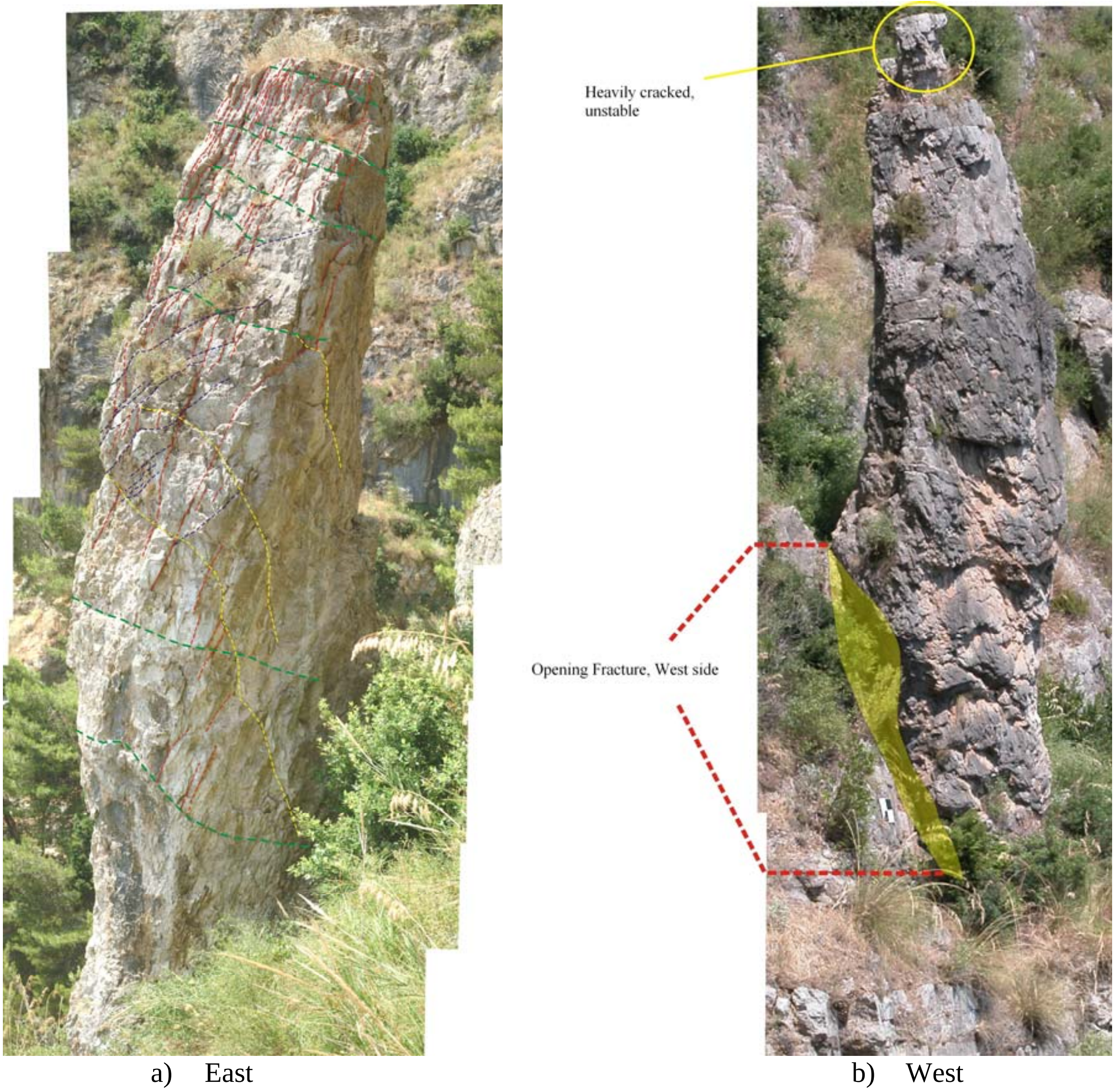
**KEYWORDS:** Seismic vulnerability, Natural stone, Nonlinear analysis, FEM, Strengthening

## 1. INTRODUCTION

In the southern part of Italy, along the National Road 163 (the Amalfi Coastal road), and particularly close to Positano town, some stone formations are present with a cylindrical shape (pinnacles) due to the erosion of the rock and the presence of sub-vertical cracking planes in the rocky slope. As a result of some collapse events of stone blocks above the road in the last years, whose trajectories have often crossed the aforesaid National Coastal road, particularly close to Positano town, in Campania, a study about the hydrogeological emergency has been undertaken within the overall program for the improvement of the rocky versant stability.
The landscape beauty and the dangerousness of these pinnacles, more than ten meters in height, leaning on the underlying road, requested a seismic vulnerability study. Within such activity the vulnerability check of these pinnacles has been performed indeed. Some possible interventions for the strengthening of these natural elements have been designed. For the safety of the area and the National Road, to avoid further failures of unstable rocks, it has been preferred not to demolish the pinnacles but anchoring them to the stable rocky slope lying behind. Both the seismic vulnerability as the evaluation of the effectiveness of the consolidation interventions have been developed by means of a variety of structural modeling, some of them simplified while others more sophisticated, in the linear and non linear field (Clough and Penzien, 1993). The specific weight, the tensile (for bending) and the compressive strength, as well as the Young Modulus, considering various directions with respect to cracking planes, were analyzed during laboratory tests on the specimens sampled on site. The dynamic behavior of these natural solids is influenced by the notable mechanical inhomogeneity due to the nature of the rock, to the presence of diffused crack patterns and to the position along the slope.

## 2. DETAILED SURVEY OF THE PINNACLE AND THE INVESTIGATED AREA

The investigated area presents a particular type of landscape, it is strongly characterized by steep slopes interested by numerous faults and cracking planes in calcareous-dolomite rocks with piroclastics coverage.
The rock masses, fractured and having sub-horizontal stratifications and systems of sub-vertical discontinuities, are interested by collapses and turnovers of the more fractured sectors and by the underground circulation of water. Another important element is the state of fragmentation of the rocks: the more the fractures, the bigger the volume of the rocks affected by this phenomenon. These column shaped formations became in precarious equilibrium because of the selective erosion and dissolution of the rock. This dangerousness along with the notable difficulties accessing the slopes lead to a survey mapped through photogrammetry with the technique of the Laser Scanner 3D. The pinnacle taken into consideration is approximately 11 meters in height with an almost elliptic shaped cross section with least diameter variable from about 1.0 meter in the upper part to about 2.3 meters in the inferior part. In fact the observations at the base of the pinnacle, conducted with the assistance of geologist-mountain climbers, has allowed to notice the stratifications and a series of tied up fractures (in figure 1a it is depicted the east side of the pinnacle). The most fractured sector resulted to be the highest above the base, in the direction of the National Road where a spacing of few centimeters of the fractures determines the formation of isolated small stone blocks.




a) East b) West

Figure 1. The analyzed pinnacle: (a) east side; (b) west side.

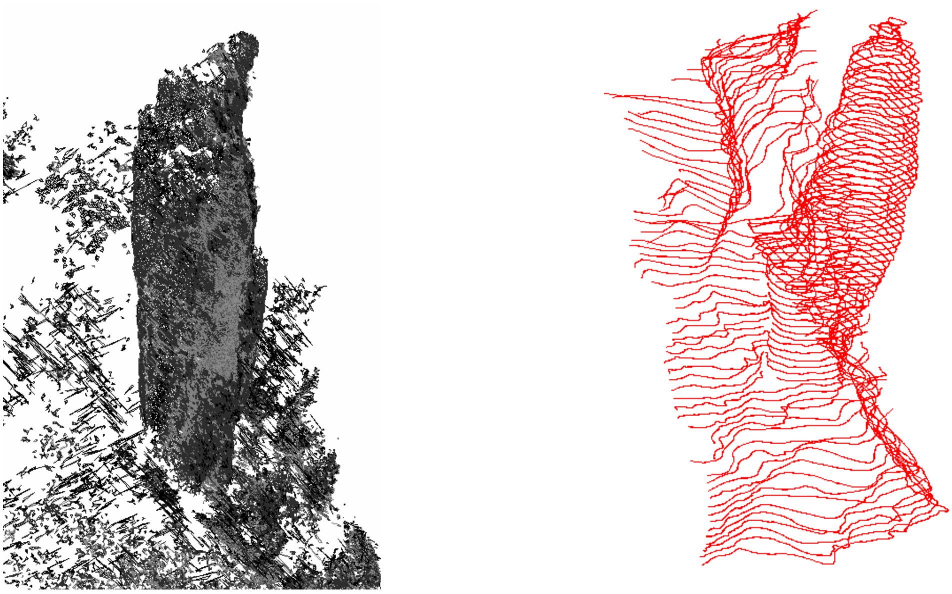
Table 2.1 Mechanical Properties after lab tests

| | |
|---|---|
| Specific Weight $\gamma$ | 28.0 kN/m$^3$ |
| Young Modulus E | 50÷150 GPa |
| Compressive Strength $f_c$ | 45÷65 N/mm$^2$ |
| Tensile Strength $f_{ct}$ | 4.2÷16.7 N/mm$^2$ |

In general, however, in the central part of the pinnacle the fractures are very close and the stratification is interlocked; great discontinuities, which can cause separations or collapses, are not meaningful, while as clearly visible in figure 1b, in the highest part of the pinnacle it is present a small portion of extremely fractured rocks close to collapse. In figure 1b the west side of the pinnacle is reproduced and it is evident a wide opening fracture at the base that practically separates it from the slope behind; however, frontally, it results anchored to the carbonatic substratum of the slope in the basal portion. The fracture actual width results about 50 centimeters in length and closes progressively going about 80 centimeters inside.
Some specimens of rock have been sampled on site. These specimens have been used to perform strength and deformability tests in the laboratories of the Department of Structural Engineering of the University of Naples "Federico II". In table 2.1 the average mechanical properties, obtained in the laboratory, are reported.

## 3. STRUCTURAL MODELING

The modeling has been performed adopting the limit criterion to address the uncertainties on the mechanical properties of the rock, a natural constituent material of these "structural" element, the pinnacles (namely uncertainties on the presence of planes of internal fracture and on the real degree of constraint at the base). This criterion foresees to appraise the range of variability of some crucial mechanical properties and the range of the possible degrees of constraint: therefore, some analyses in various limit configurations are performed. In this way it is possible to carry out some parametric analyses with a good level of knowledge about the sensitivity to these various parameters: the elastic spectrum (pseudo-acceleration of the ground and ground typology), the tensile strength $f_{ct}$, the compressive strength $f_c$.
Both the tensile and compressive strengths influence the degree of restraint at the base, in fact, under the assumption of no-tension, the pinnacle base is a simple unilateral support and the seismic analysis and safety check comes down to an overturning check, verifying that the resultant force acting on the element is inside to the base (and of course, in this case, it is also assumed that the compressive strength is unlimited).
Assuming instead some tensile strength in the rock, the base constraint is assumed as a fixed end and to perform these analyses they are both considered a simple triangular distribution of horizontal seismic actions ("linear static" analysis on the principal mode only), and the real distribution of the horizontal seismic actions due to natural Eigen modes ("modal dynamic" analysis, considering the effective dynamic characteristics of the structure on its seismic behavior). Moreover some finite element (FEM) analyses on discretized 3D solid numerical models in the linear and nonlinear field (based on 3D laser scanner survey, figure 2) have been developed.
In the analyses the cross section at the base of the pinnacle resulted the critical section because of the presence of the highest flexural moment and shear in the first mode and because of the reduction of the cross section of the pinnacle due to the aforementioned opening fracture. It has not been considered any "weak plan" at different height above the base.
From the very first rigid block analyses without tensile strength it is clearly shown that very small horizontal accelerations may lead to the overturning of the pinnacle. The following analyses unquestionably show that a significant tensile strength is required, particularly in the cross section at the base and along the cracking planes at various heights, to withstand the horizontal accelerations prescribed by the Italian Code for the examined site.


a) b)
Figure 2. 3D laser scanner: (a) Collected point cloud data; (b) Processed level curves.

### *3.1. Rigid Block Analysis Method*

Assuming a lack of tensile strength for the rock material, it is checked that the force resultant, the vectorial sum of the weight force and the horizontal seismic action, both acting in the center of the masses of the pinnacle is inside to the base cross section, or in other terms, with respect to the base, the overturning moment due to the seismic action (equal to the seismic force multiplied by the distance above the base, h') must be in equilibrium with the stabilizing moment due to the weight force (equal to the weight force multiplied by the least distance of the vertical projection of the center of the masses on the base from the perimeter of the base section, d'). In this case the seismic force is equal to the mass of the pinnacle $M_{pinnacle}$ multiplied by the seismic acceleration expected at the ground level, $a_g$, since there is no amplification effect as the structure is supposed rigid.
The center of mass is about h' = 4.98m above the base and the least distance of the vertical projection on the base of this point from the perimeter of the base section is d' = 154cm on the road side and d' = 14cm on the mountain side. Considering the equilibrium equation

$$a_g \cdot M_{pinnacle} \cdot h' = g \cdot M_{pinnacle} \cdot d' \quad (3.1)$$

accordingly the maximum pseudo-acceleration which does not cause the turnover of the pinnacle, and therefore the failure, in the most severe case of d' = 14cm is $a_g = d' / h' \cdot g = 0.028g$.
The value of ground acceleration found is smaller than the value expected in the examined area (zone 3, soil type A -rock-, where $a_g = 0.15g$ according to the Italian Code O.P.C.M. 3431/05).

### *3.2. Elastic Continuum Analysis Method*

The analysis is conducted on a continuous cantilever column where the mass is uniformly distributed, the height is equal to the greatest height determined during surveys (h=11m), elliptic constant shaped cross section is assumed so that the equivalent volume is equal to the real volume of the pinnacle. The cross-section is elliptic because it is the geometric figure that better approximates the shape of the pinnacle and the ratio between the diameters (3.94 x 2.38 $m^2$) has been chosen so that the first natural frequencies of vibration observed in the FEM model in the two orthogonal directions of seismic excitation and the frequencies calculated on a simplified model of column are matching.

Assuming a tensile strength in the rock, equal to $f_{ct}$, the seismic action which does not lead to tensile stresses higher than the tensile strength of the rock material is evaluated.
Since the real vertical projection of the center of the masses on the base does not lie on a principal axis, it is worth noted that the weight force induces a combined biaxial bending and compression in the base cross section of the real pinnacle; although the seismic analysis is performed considering the two principal directions of the equivalent cylindrical column.

*3.2.1 Linear Static Analysis*

The distribution of the seismic actions is triangular with zero force at the base and has a resultant equal to the total mass of the pinnacle multiplied by the seismic acceleration. The actual frequency of vibration of the structure and the spectral shape amplification factor S(T) for soil type A (in agreement with Eurocode 8/2004), where T is the natural period of the pinnacle are considered. In this way it is possible to plot (figure 3a) the relationship between the tensile strength of the rock and the peak seismic acceleration $a_g/g$ that leads to a tensile stress at the base equal to the respective tensile strength.

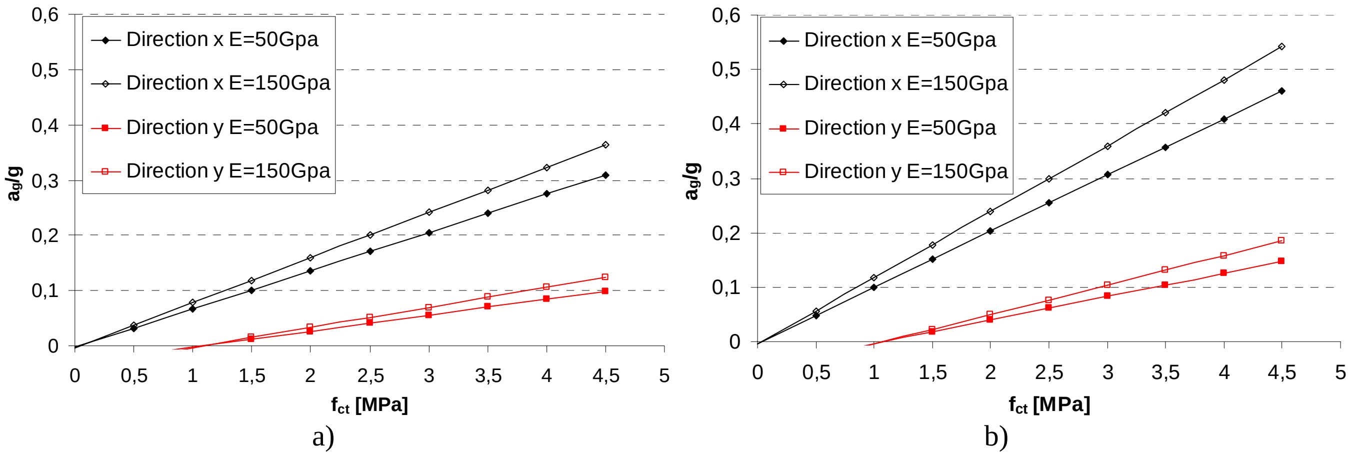



Figure 3. Max. PGA vs. tensile strength: (a) Linear Static Elastic Analysis; (b) Modal Dynamic Analysis.

In direction y (weak axis) with the weaker rock material, a tensile strength of about 6 N/mm$^2$ is requested to satisfy the seismic demand ($a_g$=0.15g). In reality, according to the recent Seismic Hazard Map introduced in the O.P.C.M. 3519 (2006), the considered acceleration value expected in the examined area now corresponds to a return period higher than 2475 years.

*3.2.2 Modal Dynamic Analysis*

The distribution of the seismic loads is variable along the column and depends on the considered modal shape.
The first three Eigen modes response in terms of shape, natural frequencies, axial, bending and shear load are evaluated according to the elastic spectrum and continuous elements theory (Clough and Penzien, 1993). These maximum values are then combined opportunely, for instance through the Square Root of Sum of Squares (S.R.S.S.) criterion.. It is highlighted that the natural frequencies (in table 3.1 is an Eigen frequencies comparison) are always smaller than the code value $T_B$=0.15 sec, which characterizes the beginning of the plateau in the elastic spectrum of the pseudo-accelerations. Comparing the linear static analysis with the modal dynamic analysis it is observed that for the first mode a reduction of the shear, that is proportional to $0.613 \cdot h$, rather than h, is attained. Similarly for the flexural moment a reduction is attained as it is proportional to $0.445 \cdot h^2$, rather than $2/3 \cdot h^2$. It is plotted again (figure 4b) the relationship between the peak seismic acceleration $a_g/g$ that leads to a tensile stress at the base equal to the respective tensile strength $f_{ct}$, according to the dynamic modal analysis.
The most unfavorable condition is still represented by the check along direction y (weak axis) in the case of weaker material: a tensile strength of about 4.5 N/mm$^2$ is requested to satisfy the seismic demand ($a_g$=0.15g).

Table 3.1 Eigen Frequencies Comparison

| *FEM 3D Model* | | | *2D Dynamic modal analysis* | | |
|---|---|---|---|---|---|
| *Mode* | *Period [sec.]* | | *Mode* | *Period [sec.]* | |
| | *E=50GPa* | *E=150GPa* | | *E=50GPa* | *E=150GPa* |
| *1° (trasl. y)* | *0.0924* | *0.0534* | *1° (trasl. y)* | *0.0886* | *0.0511* |
| *2° (trasl. x)* | *0.0562* | *0.0324* | *1° (trasl. x)* | *0.0535* | *0.0309* |
| *3° (rotat.)* | *0.0194* | *0.0112* | - | - | - |
| *4° (trasl. y + rot.)* | *0.0149* | *0.0086* | *2° (trasl. y)* | *0.0141* | *0.0082* |
| *5°* | *0.0117* | *0.0067* | - | - | - |
| *6° (trasl. x + rot.)* | *0.0104* | *0.0060* | *2° (trasl. x)* | *0.085* | *0.0049* |
| *7°* | *0.0066* | *0.0038* | - | - | - |
| *8°* | *0.0060* | *0.0030* | - | - | - |
| *9° (trasl. y + rot.)* | *0.0053* | *0.0023* | *3° (trasl. x)* | *0.0050* | *0.0029* |

### *3.3. F.E.M. 3D Analysis Method*

The Finite Element commercial code HyperStudio 7 by Altair Engineering has been adopted to model the pinnacle starting from on site digital 3D surveys, through the extrusion of the acquired level curves and point cloud data (figure 2). The pinnacle has been modeled in 3D framework using WEDGE solid elements with five sides and six nodes. The finite elements solver TNO, DIANA version 9.1 (de Witte et Kikstra, 2005) has been adopted.
The pinnacle has been restrained, on safe side, only in the base section as a cantilever. The mechanical properties for the linear elastic analysis are the same used in the previous analyses (table 2.1), while some non linearities in the material (smeared cracking and ideal plasticity) have been inserted (table 3.2), appropriate to sharpen the results.

Table 3.2 FEM analysis non linear properties

| | | | |
|---|---|---|---|
| *Tensile / Compressive Strength* | *2 MPa / 50 MPa* | *Shear Retention* | *Full* |
| *Stress cut-off* | *Linear* | *Compressive Behavior* | *Ideal Mohr-Coulomb* |
| *Tension Softening* | *None (brittle)* | *Cohesion* | *5 MPa* |
| *Ultimate Tensile Strain* | *4 E-5* | *Friction / Dilatancy Angle* | *67° / 0°* |

#### *3.3.1 Dynamic modal elastic analysis*

The Eigen modes and the natural frequencies of the cantilever model have been evaluated. In table 3.1 the comparison between the FEM 3D model results and the modal dynamic 2D analysis is summarized both for E=50 GPa and for E=150 GPa; good agreement with previously evaluated periods was found.
The 3D first and second modes were translational along the weak and strong axes (first mode), respectively. The 3D third mode was rotational. The 3D fourth and sixth modes were translational along the weak and strong axes (second mode), respectively. From the fourth mode on, in the 3D solid model, a combination of translational modes is generally observed with the rotational modes or extensional ones and this justifies the slight discard in between the modal dynamic FEM 3D analysis and the 2D analysis on the elliptic column.

#### *3.3.2 Non linear Push-Over analysis*

A displacement controlled load has been applied to the pinnacle on the cross section at 2/3 of the height from the base section. All the nodes of such section (control section) have been tied and they had the same displacement in the y direction. In figure 5 the curve of the displacement of the control section versus the base shear along the

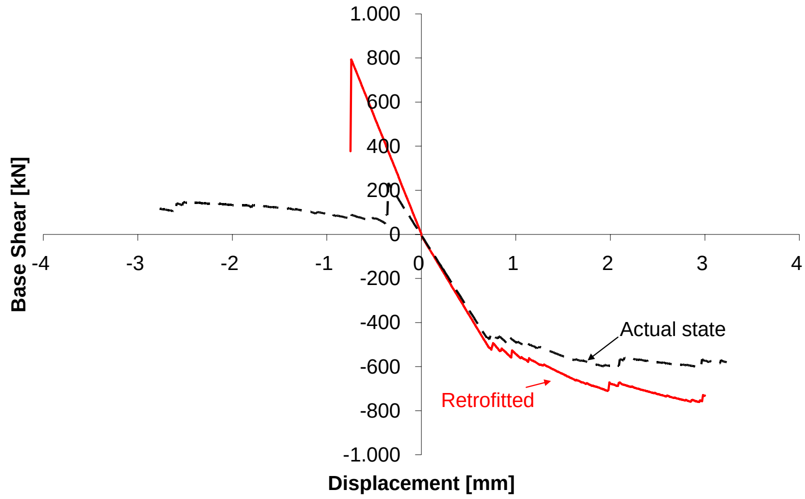
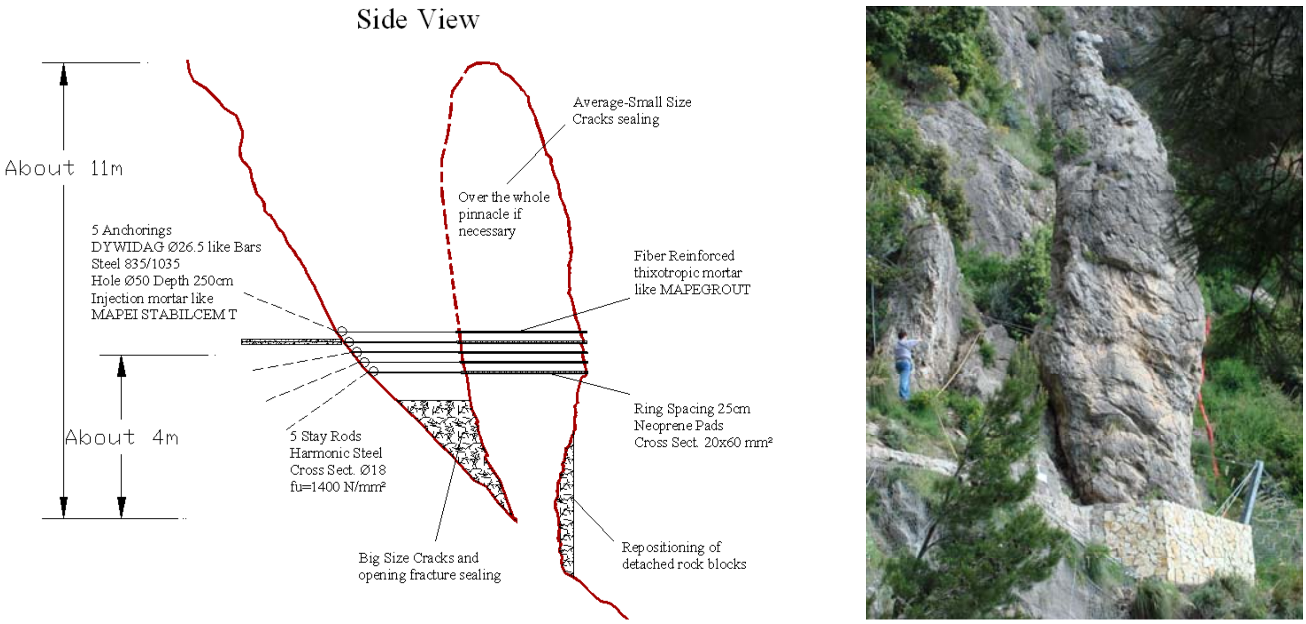



Figure 5. Push-Over Curve along weak axis: actual state and Retrofitted.

weak axis direction has been plotted, where the non linearity of the rock material (i.e. diffused cracking and plasticity) has been considered. For negative displacements the behavior is definitely more brittle because the center of pressure is already close to the perimeter of the section on the pushing (negative displacement) side. The pinnacle cannot withstand the code shear demand (according to Italian code) equal to about 400 kN. This underline the necessity to design a retrofit intervention, described in the following section.

## 4. RETROFIT INTERVENTION

Two classes of operations have been recognized (figure 6): sealing of the cracks and the opening fracture with the purpose to increase the strength at the base constraint of the pinnacle; insertion of additional constraints through stay rods made of harmonic steel. With the first class of interventions, the cracks in the rock have been sealed to guarantee tensile strength in the rock material and to guarantee for the first meters from the base, where is the opening fracture, the direct contact of the pinnacle to the rocky slope behind.




a) b)

Figure 6. (a) Design intervention concise scheme; (b) The pinnacle during works: first stay rod placement.

In the upper and central portion of the pinnacle the many tiny fractures contribute to create conditions of potential instability for small volumes of rock: a combination of fiber reinforced thixotropic mortars with microsilicate and an epoxy superfluid resin for injections have to be applied.
The second series of interventions consider the realization of ten stay rods anchored in the rocky slope at the back. Such stay rods are tied around the pinnacle at the middle-low part at about 4 meters from the base section of the pinnacle with neoprene pads avoiding any local damage due to direct contact with the steel. Stay rods (least cross section of about 250 $mm^2$) are made of harmonic steel with tensile strength of 1400 $N/mm^2$. Cables around the rock are modeled with monodimensional spring type elements, while the sealing of the cracks (i.e. opening crack) has been modeled in compression as a rigid contact.
It can be easily demonstrated that the axial stiffness of the stay rods is about thirty times smaller than the shear stiffness of the pinnacle evaluated at the application point of the constraints. This means that the scope of the cable is not to increase the stiffness of the structural system, but to hold back the pinnacle in the case of horizontal actions higher than those assumed. The effectiveness of the intervention is highlighted in figure 5, where the Push-Over curve of the retrofitted pinnacle (in solid line) is compared with that (in dashed line) of the same pinnacle in the actual state. The effectiveness of the intervention is negligible in the direction of the strong axis, but in such direction the seismic check resulted to be already satisfied.

## 5. CONCLUSIONS

The landscape beauty and the dangerousness of natural pinnacles, more than ten meters in height, leaning on the underlying National road 163 requested a vulnerability study. A detailed analysis of one of them, the most relevant for its dimensions, location and structural issues, has been presented. The dynamic behavior of these natural solids is influenced by the notable mechanical inhomogeneity due to the nature of the rock. The modeling has been performed adopting the limit criterion to address the uncertainties on the mechanical properties of the rock, a natural constituent material of the "structural" element that is the pinnacle. This method appraises the range of variability of some crucial mechanical properties and the range of the possible degrees of constraint: therefore, some analyses in various limit configurations have been performed and compared: starting from simplified mechanical models of rigid blocks passing through elastic analyses and finally to Finite Element (F.E.M.) analyses. The analyses performed both in the linear and non linear field confirmed the results assessed through the simple model of a cantilever column and they underlined the necessity to design a retrofit intervention of the pinnacle to upgrade the lacking seismic performance of this natural rock element. The effectiveness of the designed retrofitting intervention, consisting mainly in the sealing of the diffused crack pattern and the opening fracture at the base of the pinnacle and in the insertion of some additional stay rods, was finally checked.

## 6. ACKNOWLEDGMENT

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