Foreword

The International Symposium Rock Slope Stability 2016, held in Lyon (15-17 September 2016), takes place following the previous editions Paris 2010 and Marrakech 2014.

In the continuity of the previous sessions, RSS 2016 is an international forum which aims to bring together all actors of the academic and professional sectors over three days. The topics covered include, among others, site investigation and rockfall hazard modelling, monitoring techniques, rockfall trajectory analysis, risk management, protection structures and case studies. To affirm the strong connexion between researchers and practitioners, this edition includes, in addition to the regular sessions, an afternoon fully dedicated to technical presentations and exhibitions.

In the context of climate change, illustrated by frequent extreme climatic events, the international community must consider the risks related to rockfall and landslides with a new perspective. This is one of the major goals of the French national project C2ROP (Chutes de blocs, Risques et Ouvrages de Protection), under the auspices of which the Symposium is placed. C2ROP demonstrates in an outstanding manner the necessity to bridge all the components related to the relevant topics, both in academic areas and industrial sectors, to make rapid progress and to benefit from knowledge developed in related fields.

Among the various Sponsors, we would like to thank specially IREX (Institut pour la recherche appliquée et l'expérimentation en génie civil), INDURA (Infrastructures durables Rhône-Alpes), and FRTP (Fédération régionale des travaux publics, Rhône-Alpes).

We are also very grateful to the four invited Lecturers, who highly and diligently contribute to the success of the Conference.

The Organizing Committee will find here also our warm gratitude: M-A. Chanut (CEREMA), M. Gasc (CEREMA), B. Delaporte (IREX), J. Gilbert (INDURA), D. Fabre (CNAM), S. Lambert (IRSTEA), T. Berger (Geolithe), N. Villard (GTS), P. Plotto (IMS-RN) and P-F. Adam (Cluster Montagne). Special and warm thanks to A. Hardouin (IREX) for making possible this event!

With over 100 communications and 150 attendees, the Organizing Committee hopes to make this workshop a memorable event to bring together engineers and scientists from all over the world on a number of topics, to promote informal discussions, and to give rise to promising collaborations in the future.

Welcome to Lyon!

François Nicot, Chair of RSS 2016
Félix Darve, Chair of the Scientific Board of RSS 2016

International Symposium Rock Slope Stability
15-17 November, 2016, Lyon (France)

The Symposium RSS (Rock Slope Stability) 2016 takes place in Lyon (France) from 15 to 17 November 2016, after the Paris edition (2010) and the Marrakech edition (2014).

This 2016 edition is organized in the framework of the National Project C2ROP (blocks falls, rocky risks, and protection works). This conference is also an international forum permitting to gather all actors of the academic and professional sector during three days.

Topics covered during the Symposium:

- Site investigation and rockfall hazard modelling
- Monitoring techniques
- Rockfall trajectory analysis
- Risk management
- Protection structures

C2ROP

Chutes de Blocs
Risques **R**ocheux
Ouvrages de **P**rotection

In a sustainable and proven climate change context, an increase in rockfall events, landslides, rock falls is feared.
Therefore, it has become urgent to bring together all stakeholders in the field of rock risks, to propose a collaborative research framework and a platform of integrated operational resources

C2ROP aims to build a coordinated toolchain (hazard – risk – protection), to bring out a repository of risk and its acceptable cost, organize and develop the community, to provide a structured results from digital and experimental equipment tools, and finally position the French expertise internationally. Started in 2015, this collaborative project will bring the products of research available to those who need them, especially the owners and infrastructures managers.

The project idea came from the cluster INDURA. Today the project is labellized "National Project" and takes a national scale, administered by IREX, and supported by the MEEM, French Ministry of Environment, Energy and Sea.
A National Project (PN) is a specific device implementation of collaborative R&D in the field of construction, supported by MEEM. Research programs launched under the label of "National Projects" gather, on the basis of a voluntary commitment, all construction players: owners, masters of public and private work, construction companies, offices studies, engineering, industries producing raw materials or building components, public and private laboratories, universities and engineering schools. Today, C2ROP has more than 40 partners.

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Summary

The abstracts are arranged session by session, following the Symposium program.

Session 1

Protective structure design (1)

Combined rockfall and snow avalanche protection measures on Klein Matterhorn (Switzerland): innovative concept and challenging setting-up at 4000 m a.s.l

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Keywords: rockfall and snow avalanche protection measures, high altitude terrain

1 INTRODUCTION

The high-altitude ski domain Matterhorn Paradise (Mattertal, Switzerland) spreads over 2200 m elevation from Zermatt at the bottom of the valley to the top of Klein Matterhorn at 3883 m a.s.l. This four-sided pyramidal peak is stretched along the North-South direction with two predominant, ice-free faces oriented to the west and east (see Figure 1). Since 1979 a double cabin cable-car provides an easy access to the summit for skiers, mountaineers and pedestrians throughout the entire year, making this place the highest point in the Alps reachable with a cable-car. This unrivalled situation has led to a continuously growing number of visitors, reaching 650 000 people in 2015. In order to increase the current transport capacity of 600 persons/hour up to 2600 persons/hour, the construction of an additional modern cable-car was decided by the Zermatt Bergbahnen AG in 2010.

In this perspective, geological and technical preliminary studies were performed since 2011. A route parallel to the existing facility was retained for the establishment of the departure station and supporting pylons. On the contrary, the limited space on the summit ridge of Klein Matterhorn prevented any additional construction. An alternative site for the new arrival station was therefore chosen at 3821 m a.s.l. in the 50-70 degree steep west face, at the exit of an existing gallery (Figure 1).

Regarding the high elevation of the construction site, the predominance of winter conditions from October to June results in high amounts of snow likely to trigger significant avalanches. Moreover, the loose rock material covering most parts of the west face implies an enhanced risk for rockfalls. In this context, the need for an integral protection, first for the construction site (foundations, crane, deposit place), and second for the permanent facilities (arrival station), requires the setting-up of perennial protective measures in the west-face. The company Geotest AG, on behalf of the Zermatt Bergbahnen AG, was asked to design these measures and to act as project supervisor.

2 GEOLOGICAL AND TOPOGRAPHICAL BACKGROUND

The west face of Klein Matterhorn consists of strongly foliated serpentinite. The pronounced dip-slope main foliation intersects a secondary ENE-WSW layering, leading to a highly undulated terrain. Superficial material to 1 m depth is composed of loose rock likely to break-off into small to 1m long unstable slabs. On the contrary, the underlying rock is compact and only marginally faulted. The terrain morphology strongly differs between the northern and southern half of the flank. In the southern part, two 40 m spaced and 5m wide, steep ravines striate the face and channelize most of the rockfalls released in the vicinity. These corresponds to two steep east-west trending zones that cross-cut the entire peak. The envisioned site for the construction of the arrival station locates directly into the northernmost gully (see Figure 1). In the northern part, the terrain shows a smoother and more stable topography. The rock is partially covered by loose slabs with a maximum o size of 1m.

3 TECHNICAL REQUIREMENTS FOR THE PROTECTIVE CONCEPT

Snow heights and snow pressures associated with avalanches released within the west flank were investigated by means of snow simulations (Ingenieurbüro André Burkard). Despite poor input data on the snow availability in the area, results clearly demonstrated that snow avalanches could seriously damage the infrastructure.

The intensity and rebound-height of rock-falls released from the summit area were investigated using the trajectory analysis software RofMod™ (Zingeller&Geotest). Calculations were achieved considering block-source dimensioned to 1 m maximum released along 4 profiles within the construction site. Assuming the installation of protective measures, results pointed out intensities ranging up to 1000 kJ and maximal rebound heights of 4 m.

In this regard, 4 rows of 4 m high 1000 kJ nets adapted to snow avalanche mitigation were retained for our safety concept. Additional mesh placed in the upper area as well as gully-nets installed in the corridor above the planed arrival station complete the system (see Figure 1).

The anchoring system is designed considering permafrost conditions in the superficial layers as well as the predominance of harsh climate (wind, temperature and precipitations). In order to prevent any effect of frost-shattering, the drillings are

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exclusively performed using pneumatic techniques without any use of water. The nails employed to fix the mesh covers are stuck into the boreholes by means of frost-resisting resin. The stabilizing anchors of the net rows are cemented in to the rock using a frost-resisting preheated mortar.

4 PREPARATION AND SUPERVISION OF THE CONSTRUCTION SITE

In addition to the harsh climatic conditions prevailing in the area and the difficult access to the construction site, this project also stands out by the fact that the envisioned protective measures are designed to protect future infrastructures (crane, material deposit place, arrival station). These unique characteristics, as well as the restricted construction space, the design, the pegging and the installation of the measures are complex and strongly affect the planning of the work as well as the material supply.

5 FIRST CONCLUSIONS AND FUTURE PERSPECTIVES

The installation of an integral rockfall and snow avalanche protective system in the west flank of the Klein Matterhorn was launched in September 2015 and at this time shows good progress. After a winter break, the work is planned to resume in March 2016, in order to start the construction of the arrival station in early May. For the company Geotest AG, this project represents a unique challenge, both in terms of technical requirements and installation conditions.

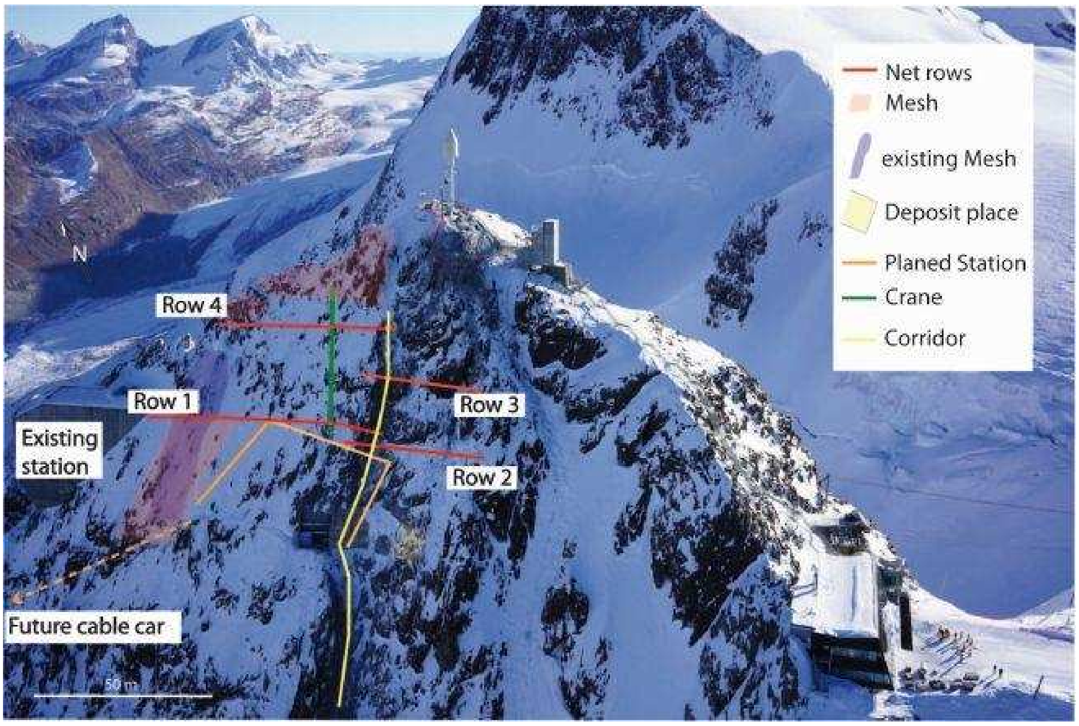

Figure 1: Frontal overview of the west face of the Klein Matterhorn with projected infrastructures and protective measures

Real-scale tests on a Reinforced Earth retaining bund impacted by a spherical projectile at energies up to 800kJ

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Keywords: retaining walls, embankment, rockfall, kinematic response, energy dissipation

1 INTRODUCTION

Different structures such as protective fences or soil embankments are used in order to protect assets from rock falls. The prediction of the behaviour of these structures under rock impact being very difficult, a real-scale experimentation is required. This paper introduces a full-scale impact tests on a Reinforced Earth protective bund with vertical faces. The paper focuses on the dynamic kinematic response of the structure when exposed to rock impact for energies ranging between 390 to 800kJ. The test method, the monitoring, and the results are presented.

2 TEST METHODOLOGY

2.1 REINFORCED EARTH PROTECTIVE BUND

The protective bund is a parallelepiped form of 9m long, 3m high and 3.4m thick (see Figure 1). This soil reinforced structure was built using the GeoTrel™ system that comprises a steel wire mesh facing (0.6m height per 3m long) behind which an 0.5m thick layer of stone is placed for the architectural finish. The compacted soil mass is reinforced with GeoStrap® reinforcement which is a 50mm wide synthetic strips made of high tenacity polyester yarn coated with LDPE. Dense and high frictional backfill was used for this project for a better energy dissipation.

Figure 1 :Reinforced Erath protective bund

2.2 TEST METHOD

The doubled vertical face protective bund is impacted by a massive 73cm diameter metallic sphere in order to simulate a rockfall. This sphere is launched against the reinforced soil structure thanks to a vehicle connected to a hydraulic catapult (see Figure 2). A first and a second impact at 390kJ with a total mass of 1700kg at a same location were done in order to assess the resilience of the structure. A third impact at a higher energy, around 900kJ with a total mass of 2700kg, was made with an impact location slightly higher than the centre of inertia of the bund. This last test was done in order to assess the performance of the structure under a severe configuration.

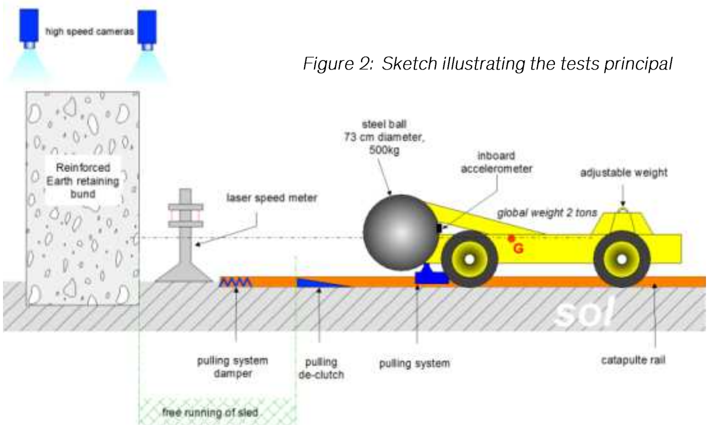

Figure 2: Sketch illustrating the tests principal

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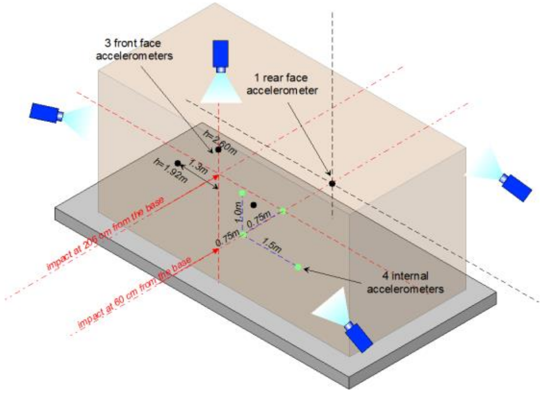
2.3 INSTRUMENTATION

The target of the real-scale test was to assess the dynamic response of the structure during impact and to estimate the residual deformation. In this regard the structures has been equipped with accelerometers and high speed cameras. Some accelerometers were buried inside the structure in order to follow the wave propagations within the soil mass, some others were place on the structure at the front and at the back. An accelerometer was also fixed on the projectile. The high speed cameras were placed around the structure as indicated in Figure 3. Topographic measurements were made before and after each test.



3 RESULTS

3.1 DYNAMIC RESPONSE OF THE STRUCTURE

The analysis of the accelerations recorded on the projectiles during the impacts at low energy (400 kJ) are showing different stages corresponding to the solicitation of three components of the structure: the facing, the uncompacted stones, and the granular fill. The double integration of the accelerations was used to determine the displacements of the fill core and of the metal sphere. The comparison between both impacts is clearly indicating that the contribution of the uncompacted stone layer to the energy dissipation and therefore to the final performance of the structure is negligible compared to the granular fill.

3.2 DEFORMATION OF THE BUND

The two first impacts at 400 kJ lead to respectively 24 (see Figure 4) and 15 cm penetration inside the bund. Sights located behind the structure indicated for both impacts a maximum residual displacement of 2cm located approximately 1.85m above the ground level. Last impact at 800 kJ under lower soil cover lead to a higher deformation. The penetration depth recorded was about 40cm with a bulging of 10cm at the back that extends on an area of about 4m diameter. For all the tests nothing revealed any evidence questioning the stability of the bund.

Figure 4: Deformation of the facing after the first impact at 400kPa

4 CONCLUSIONS

In order to assess the dynamic behaviour and the performance of a reinforced Erath protective bund a full scale instrumented structure was subjected to located impacts up to 900 kJ. The test method allows the authors to conclude on the insignificant influence of the uncompacted stone layer located behind the facing compared to the dense and purely frictional fill used in the structure core. Moreover at energies below 400 kJ only minor superficial damages were noticed. At higher energy, and for an impact closer to the top of the wall, the penetration depth is bigger due to a smaller soil cover. All the visual investigations were clearly showing no sign of serious damage that can put under question the stability of the structure under such energies.

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Investigating the behaviour of existing rockfall protection barriers

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Keywords: rockfall barriers, rockfall risk mitigation, rockfall protection management

The development of advanced full-scale testing facilities and numerical models has greatly supported the development and the improvement of new rockfall protection barriers (Volkwein et al., 2011), light and versatile steel structures, now capable to arrest falling blocks having capacity up to 8000 kJ. However, the management of existing and comparatively older rockfall protection barriers still remains a crucial issue, notably when rockfall risk assessment is performed along slopes where these structures are found (Bourrier et al., 2014). Within the context, this short note addresses the study of these protection structures, with reference to the case study of the Autonomous Province of Bolzano (PAB).

1 DETAILS OF ROCKFALL PROTECTION BARRIERS WITHIN THE PAB

Figure 1: Flexible barrier within the PAB

In the last years, the PAB has started the process of inventorying all the rockfall mitigation structures installed within its territory. Rockfall protection barriers are also inserted in the catalogue. For a convenient management, these structures can be described according to their accessibility, visibility and typology. A barrier is accessible only if reachable easily, with no need of ropes and is described as not accessible, when the use of ropes or helicopters is necessary to reach it. A barrier is also identified as visible if it is observable, also through optical devices without the aid of an helicopter. As for the type, existing rockfall protection barriers can be described of new generation, only if provided with a CE mark. The majority of the existing barriers are not CE marked, although some other documents (e.g. certificates of full-scale tests) may accompany them. These barriers are conventionally grouped and named after their deformation characteristics as flexible, semi-rigid and rigid. Flexible barriers are comparatively more recent, have high capacities (up to 5000 kJ), are provided with energy dissipating devices and hinges connects the supporting structures to the foundation system (Figure 1). Semi-rigid structures are characterized by a comparatively older technologies, have low capacities and, typically, the scope to intercept and stop the falling blocks is fulfilled by longitudinal ropes and a secondary light meshwork. Rigid barriers are also found within the PAB. These structures are historically made of steel or wooden beams in the vertical and longitudinal direction. Visible and accessible rockfall protection barriers, at present inserted in the PAB's catalogue, are about a thousand: approximately 700 are of the flexible type, 360 can be classified as semi-rigid and 50 belong to the rigid type.

1.1 STATE OF MAINTENANCE

Within the catalogue, the state of maintenance of the inventoried barriers is also described and documented. An event (e.g. rockfall, muddy of stony debris flow, fires, avalanches) might in fact have changed the initial state of the structure, modifying the structural characteristics and thus the nominal capacity. Typical consequences of an event are: the presence of rock debris and vegetation or of sole stony material of various sizes onto the interception structure, the reduction of the structure's nominal height, the rupture or corrosion structural elements. As for the visible and accessible barriers, approximately the 22% of the total number of installed barriers features one damage of the kind. Figure 2 associates a type of damage to a type of barrier. With reference to the flexible barriers, the most recurring damage is the presence of mixed materials (9.4%) and stony material (2.4%), the reduction in nominal height and rupture of structural elements has instead been detected only

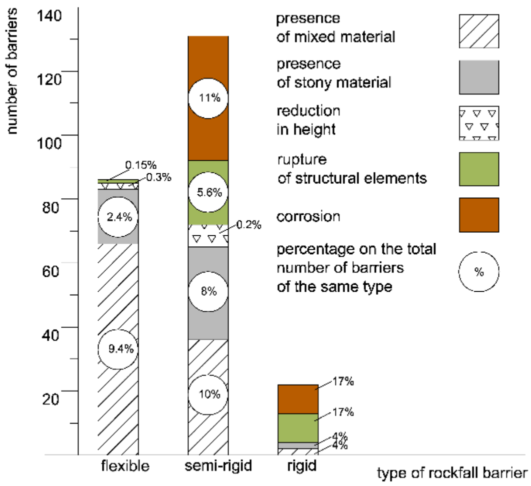

Figure 2: Types of barriers and damages within the PAB

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in three structures, and no corrosion has been observed. As for the semi-rigid structures, the type of damage observed more frequently, is the corrosion of structural elements (11%) followed by the presence of mixed (10%) and stony (8%) material. Rupture of structural elements was also found in the 5.6% of the number of semi-rigid structures. Slightly less than a half (42%) of the rigid barriers feature a damage, being the rupture or corrosion of the structural elements the most recurring ones (34%).

2 USE OF NUMERICAL MODELS TO STUDY EXISTING ROCKFALL PROTECTION BARRIERS

If proved reliable, numerical models of rockfall protection barriers provide a convenient tool to predict the capacity of a structure which has undergone an event and presents a damage of the kind described in Section 1.1. The procedure can be organised in steps: first, a model of the undamaged barrier should be devised and its reliability should be assessed, based on the available experimental results. Modifications are then introduced in the model to account for the relevant damage and the response of such a modified model is observed by virtual testing. The numerical analyses can, for instance, be carried out following the boundary and loading conditions suggested in the Annex A, of the ETAG 27 Guideline (EOTA, 2012). As an example, Figure 3 presents the results of a numerical study of the kind, performed on a flexible barrier of nominal capacity 3000 kJ. The barrier is found within the PAB territory, accompanied by a Technical report of full-scale tests, involving the central impact of a single block on a three spans prototype (energies equal to 1000 kJ and 3000 kJ). The numerical study is of the FE type. The study involves the development and calibration of the barrier model, by reproducing, numerically the full-scale tests described in the Technical report (Gentilini et al., 2013). By varying the block's speed, the model is then virtually tested in presence of other impact energies (100 kJ, 500 kJ, 1500 kJ and 2000 kJ), with the aim to describe the evolution of the maximum barrier's elongation with the kinetic energy. The resulting curve is inserted in Figure 3a (in grey dots) where the maximum experimental elongations for a 1000 kJ and 3000 kJ is also inserted (in red dots). In a similar way, the evolution of the maximum barrier's elongation provoked by the impact of an uniformly distributed mass is also inserted in Figure 3a (white dots). Figure 3b depicts the residual capacity of the impacted barrier models. The value (read along the y-axis) is determined by subjecting the previously impacted model to the central impact of a single block. For each model, several analyses are run until the impacting energy, possessed by a test block that the model is still able to arrest, reaches its maximum. The curve shows the decrease of the residual capacity of a rockfall barrier as a function of preceding impacts energies. Operatively, starting from a measured value of barrier elongation, considering the nature of the impact (concentrated, distributed) it is possible to estimate the energy at impact (Figure 3a) then, entering with such a value in Figure 3b (x-axis), it is possible to read its residual capacity.

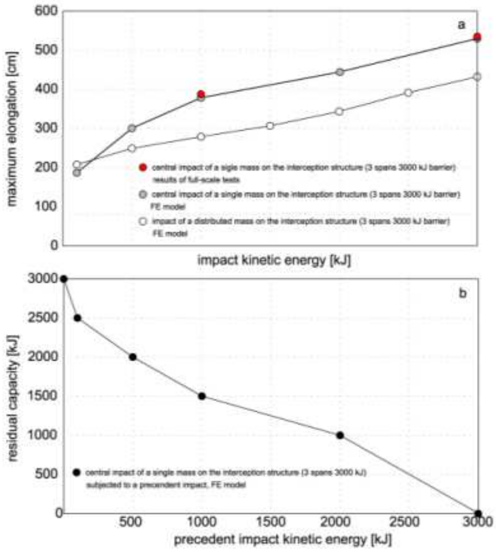

Figure 3: FE study of the response of a 3000 kJ barrier

3 CONCLUDING REMARKS

In this note some issues related to the management of existing rockfall barriers are presented with reference to the data collected by the PAB. The Province of Bolzano is at present in charge of the management of all the protection structures within its territory. In particular damage types were associated to barrier types and the potential role of numerical models within the management of damaged structures is briefly presented with reference to flexible barrier subjected to impact events. Numerical models can be also used successfully to investigate the response to impact of existing semi-rigid structures.

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Instrumentation and analysis of full-scale tests in falling rock protection barriers

Castanon-Jano, Laura [1], Blanco-Fernandez, Elena [2], Castro-Fresno, Daniel [3], Lopez-Quijada, Luis [4]

Keywords: flexible barriers, full-scale test, instrumentation

Flexible barriers are structures designed to stop falling rocks, absorbing their kinetic energy by mobilising all their components. GITECO has collaborated for several years with Malla Talud Cantabria to develop new products to become a competitive company in landslides and rockfalls protection systems. Field tests on IBT-150 and IBT-500 flexible barriers have been carried out to check their structural integrity. These impact tests have been performed according to ETAG027. A complex instrumentation has been implemented measuring forces in cables, stresses in foundations and displacements and deformations on membrane.

1 DESCRIPTION OF THE BARRIERS

Flexible barriers IBT-150 and IBT-500 both have an interception membrane made of 2 different components. One of them is a high resistance cable net with a square grid bearing and transmitting the loads from the impact. The second component is an auxiliary triple torsion wire mesh with a lower grid size having the only objective of retaining the small fragments of rocks, not having a relevant structural role. Four posts hold the net erected and divide it into 3 functional modules. In IBT-150 the functional module is 3mx10m, while in IBT-500 is 4mx10m. Lateral cables connect the cable net with the posts. The angle of the barrier with respect to the ground is accomplished by the connection of upstream and side cables to the free end of the posts. These cables incorporate energy dissipating devices of double U type, developed by GITECO and MTC (Castro-Fresno, 2009). The anchorages are different in the two barriers. A one bolt system with a specific bearing plate for post connection is used in IBT-150, whilst a 3 bolt system with a stiffer bearing plate in IBT-500. Figure 1 shows the scheme of the geometry and main elements of each barrier. A preliminary design of IBT-150 was performed through both dynamic and static analysis. Dynamic analysis was based on a simplified analytical method while static analysis was carried by means of finite element method (Lopez-Quijada, L., 2007).

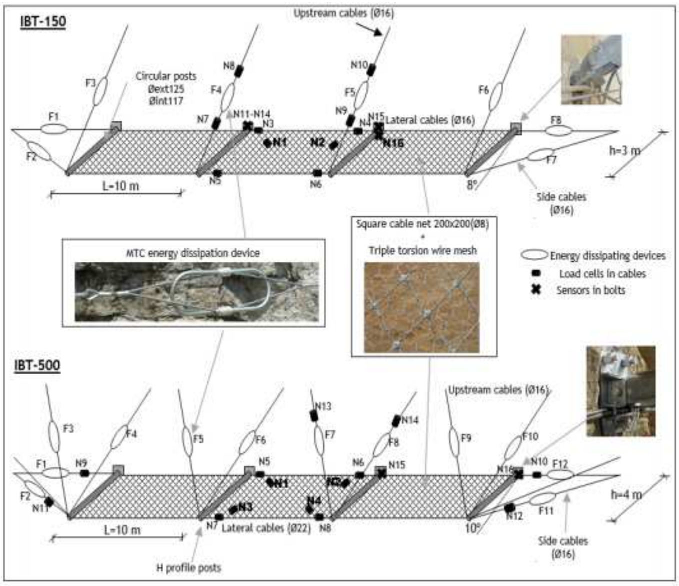

Fig 1: Geometry and sensors location for barriers IBT-150 and IBT-500. Detail of interception structure, brake system and connection of the post to the ground.

2 DESCRIPTION OF THE TEST

2.1 STANDARDS

Since 2008, full-scale tests in flexible barriers can be performed following the procedures described in ETAG 027, a guideline developed by the European Organisation for Technical Approvals (EOTA) with the aim to unify the testing criteria of these structures. This standard defines two types of test, a vertical impact test and an inclined slope test. Vertical impact test has been selected for both barriers. Three sequential tests of two different energy levels are defined in this guideline, service energy level (SEL) and maximum energy level (MEL), being MEL energy three times the SEL energy. Maximum energy level is chosen by the manufacturer before the test. Therefore IBT-150 denotes for a barrier with a MEL of 150 kJ, while IBT-500 represents a barrier with a MEL of 500 kJ. At first, two SEL tests are performed, in order to check the resistance of the kit to sequential impacts. For the third test (MEL) the kit can be repaired or also replaced by a new one after SEL tests. The objective is to characterize the maximum capacity with a unique impact of the block. In these two barriers, no repairing was carried out after second SEL impacts.

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2.2 SET UP AND INSTRUMENTATION

A vertical drop type test is considered the most appropriate since the all the potential energy of the block is transferred to the barrier in the impact, contrary to what happens in an inclined test, where the block loses energy in its rebounds with the ground and the block energy in the impact instant becomes unknown. The impact velocity and the mass of the blocks are summarized in Table 1.

Barrier type	m_{SEL} (kg)	V_{imp} (m/s)	E_{SEL} (kJ)	m_{MEL} (kg)	E_{MEL} (kJ)	V_{imp} (m/s)
IBT-150	148	25.27	47.25	504	165.02	25.59
IBT-500	460.5	26.94	167.15	1429.5	518.87	26.94

Table 1: Characteristics of the SEL and MEL test of both barriers

The following data will be recorded in each test: block velocity in the last meter before the impact, maximum elongation, residual height of the net and forces in cables and anchorages. A total of 16 sensors have been installed, based in the electrical extensometry technology. Axial force in cables is measured with a sensor without cutting the cable (Blanco-Fernandez, 2013). Forces in the bolts are measured using load cells and also directly sticking strain gauges. In IBT-150 load cells were specially designed to measure forces in the bolts close to the foundations. In IBT-500, the gauges were directly glued to the bolts. In the IBT-150 one only high speed camera was used, while in IBT-500 two high speed cameras recorded the test in a frontal and lateral view.

3 RESULTS

Data recorded during the test in IBT-150 revealed that load cells in the left bolt did not work correctly because of an inadequate transmission of stress between the bolt and the load cell through the bolt nut. Sensors located in the cables worked adequately for both barriers tests. Maximum force measurements in IBT-500 did not exceed the ultimate strength values; this was also checked by a visual inspection of the barrier after tests. On the opposite, lateral right sensor (N4) of IBT-150 reached, in the MEL test 169.81 kN, a slightly higher value than the ultimate strength of the cable (160 kN). This might indicate the cable was nearly about to fail, however, due to variations in the cable properties, this cable might have a slightly higher strength than the average. Energy dissipaters did not fail in any case. First launching of SEL in IBT-150 resulted in the activation of only one brake of the central module. In MEL test the other central brake and lateral brakes worked. A small asymmetry made the top right brake work slightly while the top left brake didn't move. A different pattern occurred in IBT-500. After first SEL test, two lateral brakes worked (2 and 11). In MEL test all the brakes activated except 3, 4, 9 and 10 (top left and top right). Although they were completely activated, they didn´t break thanks to their laced geometry. Residual height of the barrier is under 70% of the nominal height in SEL tests, and under 50% in MEL test, stablishing an Energy Level Classification A for both barriers. The first SEL impact in both barriers (IBT-150 and IBT-500) leads to permanent deformation in the central module of the barrier, both in residual height and in elongation without the block. This fact can be due to their flaccidity in the installation. A small impact might induce to the accommodation of the cables to a new equilibrium position.

4 CONCLUSION

An instrumentation for flexible barrier testing has been developed and implemented in two different barriers, IBT-150 and IBT-500. The most suitable way to measure stress in the bolts is by directly gluing gauges in the clean polished surface. Sensors for tension measurements of cables work adequately and were found to be a practical solution since they can be placed directly over the cables without cutting them. The results obtained demonstrate that both barriers satisfy the requirements of ETAG027. The preliminary design of IBT-150 showed a maximal force in perimeter cables of 146 kN, considering the maximum value of obtained from both dynamic and static calculations. This value was overcome in the experimental test, getting close to ultimate strength of the cable, being advisable to increase its diameter. Depending on the place for the location of the barrier, maximum elongation can be limited by the closeness of roads or human transit areas. In that case a pretension in the lateral and side cables could be applied. The cost of IBT-150 is around 400€/linear meter, while IBT-500 is between 800 and 900€/linear meter (Gobierno de Cantabria).

5 ACKNOWLEDGMENTS

We would like to thank MallaTalud Cantabria for the information provided and SODERCAN for the financial support.

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Efficiency of flexible structures against small landslides

Stéphane LAMBERT[1], Adel ALBABA[2], Bruno CHAREYRE[3], François NICOT[4]

Keywords: Numerical modelling, Discrete Element Method, rock slide, metallic net fences,

1 INTRODUCTION

For decades flexible structures made of metallic components have been used for stabilizing snow packs and intercepting rockfall, and were recently used for catching debris flows. The aim of this article is to present the numerical modelling of flexible structures intended to contain a flowing volume of a granular and coarse material, such as a rock slide or a debris slide. Both the flowing granular material and the flexible structure are modelled using a discrete element method (DEM). The modelling of both the flowing granular material and the flexible structure is detailed before presenting some results.

2 MODEL

The numerical model is developed based on a discrete element method (Yade-DEM.org; Šmilauer et al., 2016), where any system is described as a collection of discrete particles. The parameters governing the interaction between the particles associated to each of both the flowing material and the flexible structure were calibrated using previously published experimental data.

2.1 FLOWING MATERIAL

The flowing material consists of a dry polydisperse granular material with elongated grains (Albaba et al., 2015). The model parameters were calibrated based on laboratory experiments consisting in a granular and angular material flowing down a flume before hitting an instrumented rigid wall. The non-sphericity of the grains was accounted for using unbreakable clumps of two spheres of same radius, with an overlapping equalling one radius. The grain size distribution fitted that in the experiments. The model calibration was based on the flow characteristics (velocity and height). The flowing material model was then validated by comparison of forces acting on the wall over time, recorded at different heights on the wall.
After validation of the granular material model, the rigid wall was replaced by the flexible structure. The force exerted on the wall by the flow was very close to that from experiments (Albaba et al., 2015).

2.2 FLEXIBLE STRUCTURE

The flexible structure consisted in a metallic net hanged on main cables, connected to the ground via anchors, on both sides of the channel. Dissipators were positioned between the main cables and the anchors to prevent anchors from excessive pull-out forces. These are activated as soon as the tensile force reaches a threshold value. The connection between the net and the cables is insured by sliding rings. All the components were modelled as flexible beams (Bourrier et al., 2013) or wires, with mechanical parameters defined from literature data.
The virtual structure considered in this study is 15.2m long and 5.5m high. The net is made up of cables 16mm in diameter, forming square meshes 300mm in side. It is supported by 5 cables 32mm in diameter. The yield force of the dissipators is 250 kN. It is installed vertically in a thalweg of slope angle 40°.

3 RESULTS

The described model was used varying different parameters related to the structure (inclination of the fence, with/without brakes, mesh size opening), but also to the channel (inclination). The structure response was mainly investigated

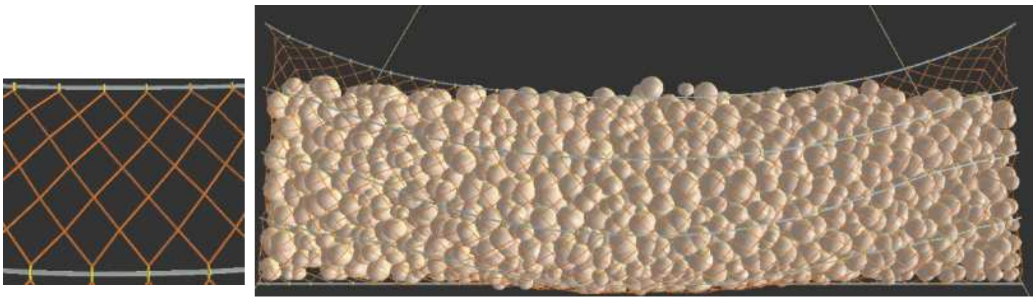

Figure 1: Detail of the structure and structure filled with the coarse granular material

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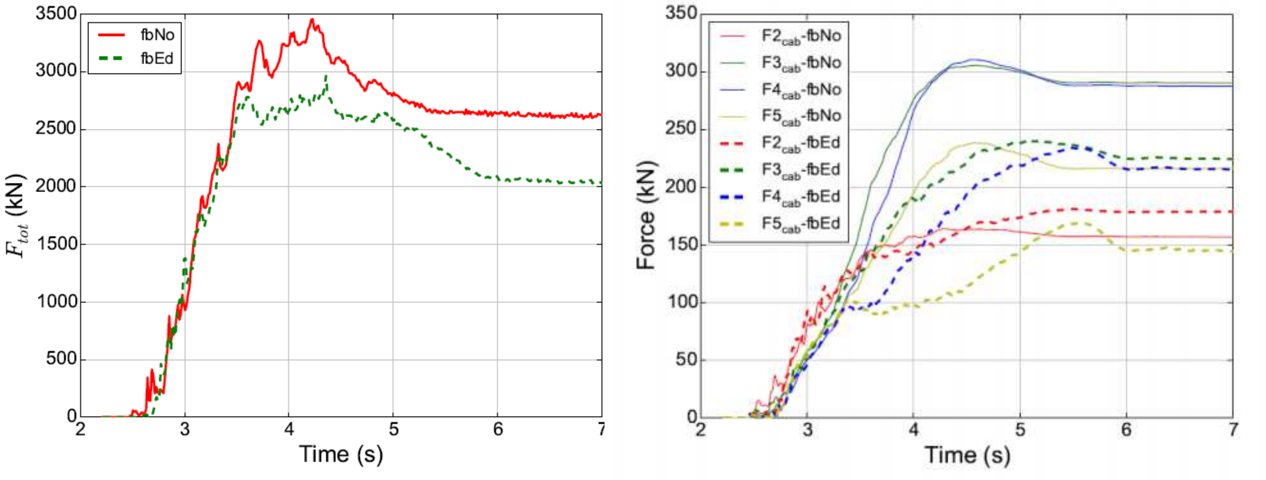
in terms of the total force acting on the fence, forces developed in the different components of the structure and also in terms of the ratio of granular material caught behind the net.



For illustration purpose, Figure 2 shows the influence of the presence of dissipators on the structure response. Dissipators tend to reduce the total force acting on the structure. This is related to the higher deformation of the structure that increases the period of time required for stopping the flow. Dissipators also reduce the average force in the cables and lead to a more uniform distribution of forces from one cable to the other. This is due to a change in the way the forces are transmitted from the net to the cables, this change resulting from the extra-length provided by the dissipators, leading to the deformation of the structure.

After analysing the effect of the different parameters on the structure response, the simulation results were compared with the load as estimated by available guidelines in a similar application field (Volkwein, 2014). Discrepancies were observed, mainly revealing that the dynamic component of the force acting on the structure was overestimated by these guidelines, contrary to the static component. This suggests that the analytical models proposed by these guidelines are too simple to catch all the mechanisms involved.

4 CONCLUSION

A DEM model for simulating the behaviour of flexible structures intercepting granular flows has been developed and validated. The influence of various parameters has been addressed, providing strong conclusions in view of optimizing the design of these structures. Besides, confrontation with existing guidelines in a similar field revealed that more complex analytical models are necessary for design purpose.

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The influence of bloc rotation on the design of rockfall protection embankments

Bernd KISTER[1]

Keywords: rockfall protection embankments, impact, bloc rotation, impact disturbed zone, angle of impact

Today the design of rockfall protection embankments is often based on the equivalent force method. For example in Switzerland a formula, which was developed for rock sheds and is described in the guideline "Exposure of rock sheds due to rockfall" (ASTRA, 2008), is used very often. But there are significant differences concerning the geometry and the boundary conditions as well as the construction materials in the two cases "rock shed" and "rockfall protection embankment". The Austrian standard ONR 24810 (2014) is based on small-scale experiments done by Hofmann & Moelk (2012). But also in this paper of Hofmann & Moelk (2012) an equivalent static force is used. And the use of an equivalent force method for the design presumes the knowledge about shape and size of the Impact Disturbed Zone (IDZ). But there exists no uniform idea concerning the geometry of the IDZ as shown by Kister (2015). Additional a common characteristic of all those models used up to now is that bloc rotation will be disregarded. Therefore a research project including small-scale quasi-2D-experimental studies and half-scale 3D-tests was established to follow up the following questions:

- How does bloc rotation influence the impact process on a rock fall protection embankment?
- What kind of shape occurs for the Impact Disturbed Zone (IDZ)? How does the IDZ grow with time?
- How does embankment geometry influence the trajectory of a bloc overcoming the embankment?

1 SMALL-SCALE QUASI-2D-EXPERIMENTAL STUDIES

Quasi-2D-embankment models made of sand and with a thickness of 20 cm had been constructed in a box with one side made of acrylic glass to observe the impact by a high speed camera. Totally 46 of these small-scale quasi-2D-tests had been done. Two types of embankment cross sections had been used in the tests. A symmetric embankment cross section has been used in 16 tests (Fig. 1a). An asymmetric cross section has been used in 30 tests (Fig. 1b). Additional for every type of cross section 3 different crest sizes had been studied in the tests.

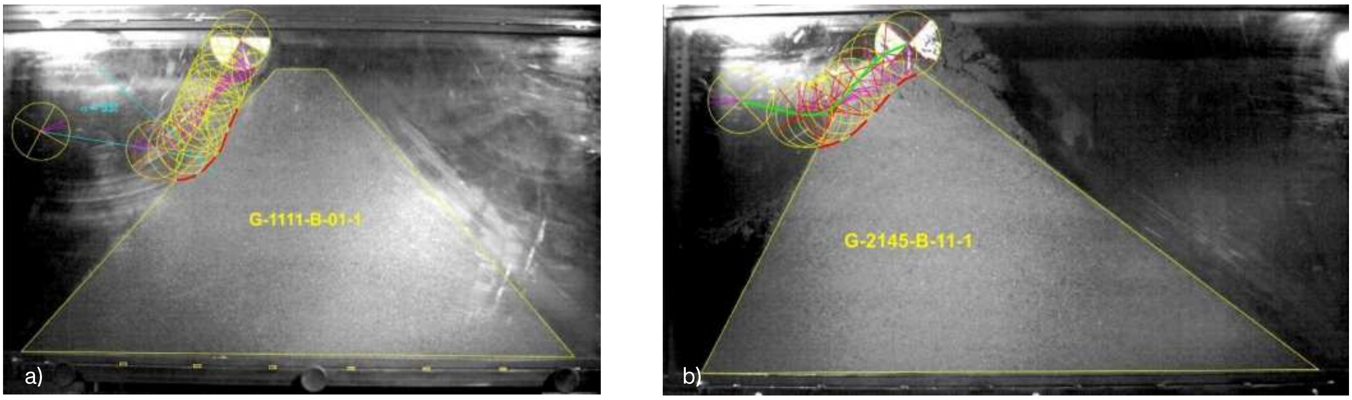

Figure 1: Embankment geometry and impactor trajectory after impact, a) symmetric embankment cross section in test G-1111-B-01-1 (slope angles 49° each), b) asymmetric cross section in test G-2145-B-11-1 (slope angle 62.6° resp. 37.3°)

Six different impacting bodies had been used in the tests. Eight tests had been done with a small concrete cylinder K, diameter 11 cm. A larger concrete cylinder G with a diameter of 16 cm was used in 11 tests. Additional a cylinder St, made of steel and a diameter of 11.3 cm, had been used in six tests. To measure the deceleration during the impact process a concrete cylinder with a 16 cm diameter was drilled out and a sensor had been placed inside the cylinder. This cylinder GS was used in 10 tests. Additional 9 tests had been done with a concrete body with an octahedron cross section (OKT). For experimental reasons all those bodies had been constructed with a height of 16 cm, which is smaller than the thickness of the embankment models. Because of that difference, the tests are not exactly plain strain tests and were named as quasi-2D-tests. For comparison 2 tests had been done with a sphere made of Aramith resin. The impact process was recorded by a high speed camera with a frame rate of 500 Hz. To analyze particle velocities and displacement fields inside the embankment the Particle Image Velocimetry (PIV) had been used (Fig. 2).

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2 HALF-SCALE 3D-EXPERIMENTAL STUDIES

The small-scale quasi-2D-tests had been complemented by half-scale 3D-experiments to verify the 2D-results also for the 3D-case. For the 3D-half-scale tests embankment sections 3 m in length and 1.5 m in height had been constructed at the terrain of the Lucerne University of Applied Sciences and Arts. 10 tests in total had been done, 6 with a symmetric cross section and 4 with an asymmetric cross section. As impactor a concrete sphere with a diameter of 0.35 m and a weight of 50.2 kg had been used

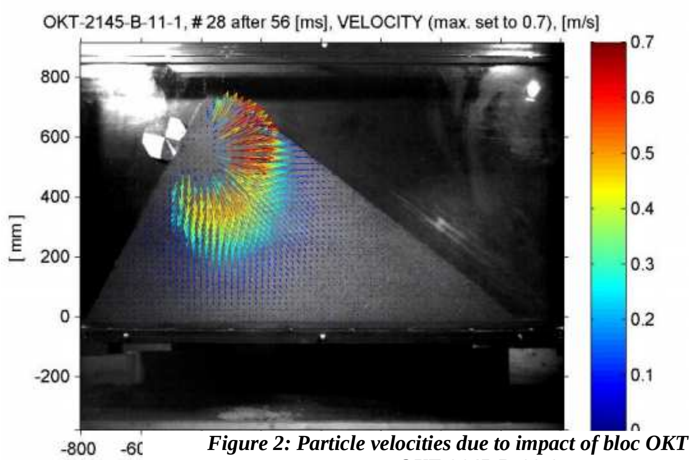

Figure 2: Particle velocities due to impact of bloc OKT in test OKT-2145-B-11-1

3 CONCLUSION

The results of the small-scale quasi-2D-experiments as well as the results of the half-scale 3D-experiments showed the bloc rotation as a fact of utmost importance for the impact process of a bloc onto a rock fall protection embankment.

The PIV analysis of the small-scale quasi-2D-tests showed that an impact process consists of different phases. At the beginning of this process compaction occurs nearby the point of impact. But during the further procedure the shock wave produced by the impact may generate loosening at the opposite slope of the embankment. This occurs because the embankment slope at the downhill side is a free boundary. If the embankment thickness at the point of impact in the 2D-tests had an extent of about a factor 3 compared with the bloc diameter, the displacements at the downhill slope of the embankment did not come up to a critical value and no embankment failure occurred.

As a summary of the results of the small-scale quasi-2D-tests as well as of the half-scale 3D-tests the following conclusions may be drawn for the design of rockfall protection embankments:

- The uphill slope of a rockfall protection embankment should be at least 60°. Therefore pure soil-made embankments should be avoided and construction methods should be used which allow the construction of steep uphill embankment slopes.
- The dimension of the embankment crest should be at least 1.2-times the diameter of the dimensioning bloc. This reduces the risk the embankment will be punched through by the bloc.
- If rockery is used at the uphill slope, it has to be taken into account, that the masonry blocs will transfer impact energy only with low damping inside the embankment. Therefore very slender constructions should be avoided.

Neither the results of the PIV analysis of the quasi-2D-experiments nor the displacement measurements done in the lower measurement plane during the 3D-tests showed significantly large displacements in the embankment's contact area or the embankment key. Therefore a stability problem at the embankment's contact area as a consequence of an impact could not be seen in the tests.

4 ACKNOWLEDGEMENTS

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Session 2A

Protective structure design (2)

From local properties to global behaviour of rockfall barriers

Cyril DOUTHE[1]**,** Leyla GHOUSSOUB, Romain BOULAUD, Karam SAB

Keywords: discrete element method, homogeneisation, structural analysis, sliding cable.

This paper investigates the influence of the micro-structure of usual nets on the global behaviour of simple rockfall barriers. Based on homogeneisation techniques, the first section gives a new insight on intrinsic mechanical properties of nets. Then the second section illustrates on different examples how these properties influence the stress distribution and the deformation of the whole structure. Finally the conclusion emphasises the potential of such equivalent modelling for preliminary design of barriers.

1 INTRINSIC MECHANICAL PROPERTIES OF USUAL NETS

1.1 IDENTIFICATION METHOD

The nets of rock-fall barriers are usually made of a small unit cell that is repeated through the width and height of the net (think for example of the rings in an ASM barrier). From far, they can be seen as a homogeneous structure, actually a continuous membrane, with a periodic micro-structure. It seems thus interesting to identify the mechanical properties of these equivalent membranes for the most spread nets. To this end, homogeneisation techniques developed in Florence & Sab have been used to derive these properties. The various micro-structures are taken from actual discrete models available in the literature (for ex. the one developed by Volkwein or Nicot) and some variants proposed by the authors.

The theoretical framework of this work is elasticity and the small displacement theory. Some precautions are thus necessary considering the strong non-linearity of the barrier when it is impacted. Nevertheless, the insight that the method gives to the intrinsic mechanical properties of the nets and the various ways these structures redistribute stresses, is to the authors opinion for the analysis and understanding of rockfall barriers.

1.2 INTRINSIC PROPERTIES

This work focuses on ring nets, cable nets, double torsion wire mesh, Omega nets. Many discrete models are changed into equivalent membranes. It evidences:

- significant variation for a given structure depending on the discrete model,
- some families of nets (those with no shear rigidity, the quasi-isotropic, the orthotropic and the quasi-incompressible),
- significant changes in the behaviour (especially the Poisson ratio) during the loading.

2 INFLUENCE OF THE LOCAL PROPERTIES ON THE GLOBAL BEHAVIOUR

2.1 NECESSITY OF AN EQUIVALENT DISCRETE MODEL

To investigate the influence of the identified behaviours on the stress distribution, the implementation of these behaviours within advanced FEM program (including large deformation theory of shells) has been done and it gave satisfactory results for fixed boundaries, even for large displacements. However, when introducing sliding elements on the edges of the net, some numerical stability problems appeared and could not be solved yet. As handling this kind of sliding elements is easier in discret method, it was decided that the comparison will be done on equivalent discrete models based on a unique geometry, on a unique unit cell. The different behaviours should be managed only by changing the stiffnesses of the constituing elements. Doing so, it will be possible to link the differences in the stress distribution and deformation of the net directly to the local properties and hence to get insight on the behaviour of the barrier.

2.2 DESCRIPTION OF THE EQUIVALENT MODEL

In this section, the geometry of the equivalent discrete model is detailed, together with the relative stiffnesses that must be chosen to fit the behaviour of the various nets. Some aspects of the convergence of the results with the size of the unit cell are also shown.

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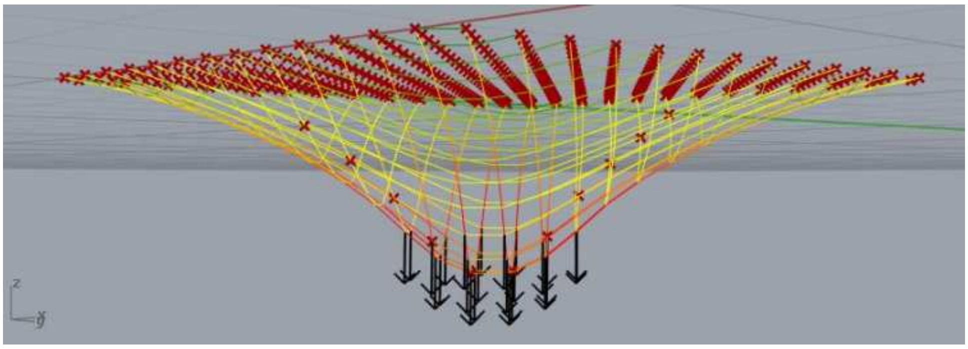
2.3 STRUCTURAL ANALYSIS OF SIMPLE ROCKFALL BARRIER

The behaviour of some simple barriers is finally investigated. This section focuses on two issues: stress distribution and deformation. The main parameters are: the local properties of the net and the boundary conditions (from fixed to completely sliding cable on the edge). Most comparisons are made on square nets (for which experimental data is available), but still an application to a three moduli barrier is presented. All simulations are static.



3 CONCLUSION

A theoretical framework for the identification of the intrinsic behaviour of nets is presented. It allows for the comparison of nets and nets models at local scale, but also at the barrier scale. It provides hence a new insight on their behaviour which is a first step toward economic and performance design of barriers.

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Generic modelling of rockfall catch fences under impacts

Jibril COULIBALY[1], Marie-Aurélie CHANUT[1], Stéphane LAMBERT[2], François NICOT[2]

Keywords: generic modelling, rockfall, fences, impacts, DEM, mechanical models

Rockfall catch fences are efficient and cost-effective protection structures to mitigate hazards in mountainous areas. The design of such structures requires a sound understanding of their complex mechanical response under dynamic loading. This document introduces a generic approach for modelling most types of catch fences un order to provide a deeper insight into their structural behaviour.

1 DESIGN AND REGULATIONS: DETERMINATION OF CATCH FENCES PERFORMANCES

Performances of rockfall catch fences are determined by a full-scale testing. Since 2008, the European Technical Agreement Guidelines, ETAG 27, prescribes a procedure to test actual kits for different energy levels; performances of a kit can only be determined based on the ETAG 27 testing procedure. Should a given feature of the kit (energy level, residual height...) meet the requirements, it would be labelled with the "CE" mark which is compulsory for public procurement use of rockfall fences. Parametric studies, carried out by means of numerical simulations, can contribute to the design of efficient and optimal structures for *in situ* loadings (Bourrier, 2014) and ETAG 27 testing.

Several modelling tools have been developed in order to provide a numerical description of the general dynamic behaviour of catch fences (Nicot, 2001, Trad, 2011). Each one of these models was created to simulate the response of one given technology and cannot be used to precisely model various structures, given the diversity in the technologies. Nonetheless, regulations cannot be technology dependent and need to settle general procedures that apply to the majority of fences.

2 A GENERIC APPROACH FOR MODELLING CATCH FENCES

So that the modelling of different fences can be harmonised, Chanut (Chanut, 2012) developed a generic modelling platform of rockfall catch fences. Based on the explicit dynamic Discrete Element Method (DEM), any fence is modelled as an abstract object made out of interacting nodes, arranged in a way to represent the constitutive elements of the fence : posts, net, cables and energy dissipating devices. The presented work aims at developing the mechanical models of the elements as well as the experimental protocol to calibrate them. Additional effort is made to improve the convenience and performances of the platform.

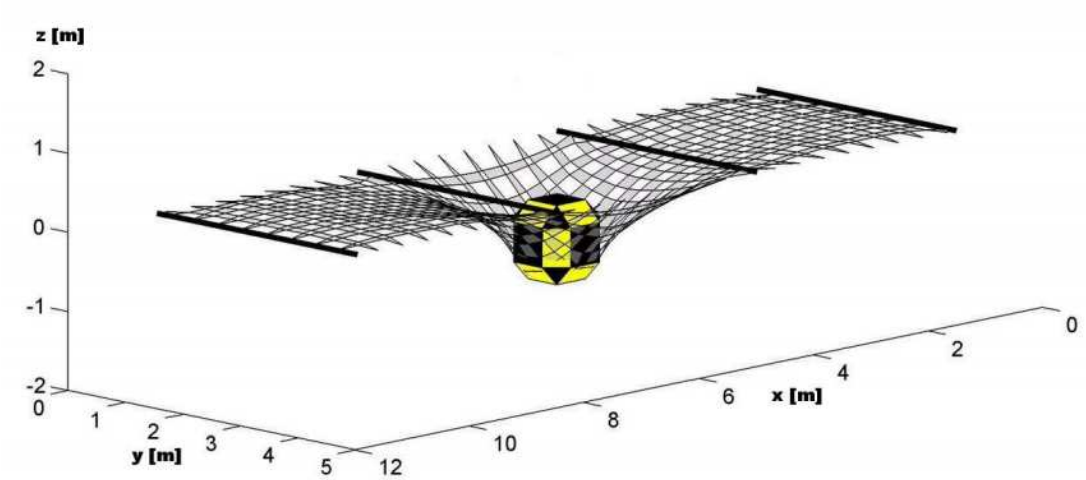

Figure 2 : Impact simulation using the developed ring net model

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2.1 MECHANICAL MODELS

Complete mechanical models for interlaced ring nets and single torsion wire mesh nets have been developed, accounting for material and geometrical non-linearities. DEM models of a single elementary pattern (ring, wire cell) are then assembled to form a multi-scale model of the net (Figure 1).
A new model for sliding cables has also been created, allowing friction between the cable and the sliding elements, as well as plastic deformation of the cable. This model is capable of reproducing the sliding of the net on the cables known as "curtain effect" (Figure 2).

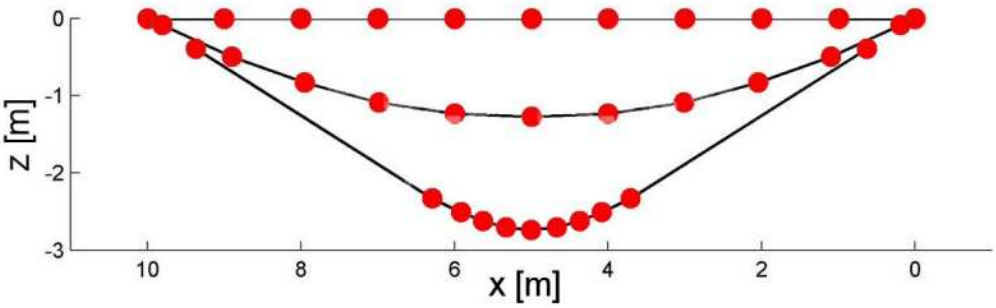

Figure 3 : curtain effect : manifestation (left) and sliding cable simulation (right)

The mechanical models offer a general description of the elements and may adapt to variations in the technologies through their input parameters. For each model of an element, a testing protocol is defined in order to provide a given manufacturer with the procedure to follow to determine the model parameters fitting best its product.

2.2 SOFTWARE DEVELOPMENT

Low calculation time is necessary to ensure efficiency in the use of the software. Fast algorithms, as well as parallel computing are used when possible. A rendering program was also created to convert simulation raw data into colormap and realistic renders as shown in Figure 1. A graphical user interface will soon be added to make the input more convenient for users.

3 CONCLUSION

The generic modelling platform provides a unified and versatile tool to study the dynamic behaviour of rockfall catch fences. Parametric simulations will contribute to a better understanding of the structural response and to the determination of influential parameters. The results may eventually help with the definition of a safety factor regarding the impact energy and with the writing of a Harmonised European standard. Manufacturers may also use the platform as a design and numerical prototyping tool.

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Dynamic non-destructive evaluation of rock anchorage

François WILQUIN [1,2], Corinne HORB [1], Zhi-Qiang FENG [2], Gérard PORCHER [2]

Keywords: Rock anchorage, dynamic testing, frequency response function, dynamic stiffness, FE simulation

Maintenance is one of the main tasks for structure and natural site administrators. It is particularly true for rock anchorage. This type of anchorage is massively used to provide stability of engineering structure such as tunnels, dry docks, pre-stressed structures and also stability of natural site such as rock mass and cliff. A lot of rock bolts are installed each year throughout the world. Administrator needs to have a quick, economical and effective method for monitoring their integrity. Actually static method are use to test these structure. Static method is risky, time consuming and should deteriorate auscultated structure. Rincent ND Applications[1] has developed a non-destructive method to evaluate stability and load of ground anchorage (Horb, 2005). This paper presents through an in situ testing example and a dynamic FE simulation this method for anchorage auscultation and for rock anchorage in particular.

1 PRESENTATION

1.1 ROCK ANCHORAGE

A rock bolt consists of a bar inserted in a borehole that is drilled into the surrounding soil or rock mass and anchored to it by means of a mechanical or grout (chemical or cement) fixture. This system is composed of three parts, a fixed part of the bar which is bonded into a stable surrounding soil or rock mass, a free unbonded part of the bar and a steel head plate with nut to allow load transfer at the unstable rock area.

1.2 DYNAMIC IMPULSE RESPONSE TESTING

Dynamic non-destructive tool presented in this paper is based on the frequency response function obtained by the impulse response measurement. Impulse response technique was initially used in early 70s to give information about piles (length, adhesion with surrounding soil) (Davis et al, 1974) and more recently to detect voids in concrete slab and pavement (Ottosen et al, 2004). This technique was adapted to in situ test to ground and rock anchorage systems. Transfer function of the structure is obtained by Single Input-Single Output (SISO) test. Vibration is generating with an impulse on head plate by an instrumented impact hammer. Response to the input stress is measured on the same plate using a velocity (geophone) or acceleration (accelerometer) transducer (Figure 1). Data of both instruments are transferred by wireless technology to computer for storage and future data treatment.
A fast Fourier transform (FFT) algorithm is used to convert force and velocity in time domain into frequency spectra and velocity is divided by force to provide the mobility function. Data of mobility curve against frequency is used to characterize anchorage.

Figure 1: Experimental setup (From Rincent ND Applications)

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2 EXPERIMENTAL IN SITU TESTING

Experimental test were performed at National Road 3 in Meaux (France). The experimental site is a nailed wall composed of 80 nails split in two height levels. 10 of these anchorages were testing dynamically as explain in chapter 1.2. Study of frequency response function obtained allows attesting if anchorage is broken. Comparative studies of dynamic response of anchorage also indicate if there is singularity in these structures. A complementary test was carried out to calibrate variation of dynamic response of anchor versus static load. Study of the relationship between square root of dynamic stiffness (define as ratio between load and displacement in frequency domain) at low frequency and static load show a linear evolution which could be used to determine anchor load of which have the same boundary condition.

Figure 2: Experimental setup for calibration test

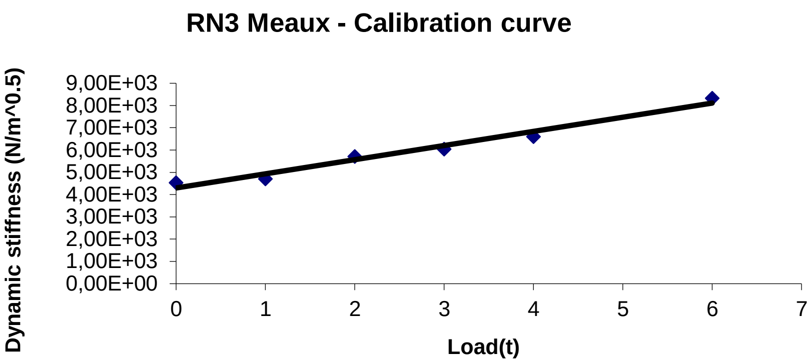

Figure 3: Evolution of dynamic stiffness versus static load

3 DYNAMIC SIMULATION

A parametric dynamic FE simulation has been done to evaluate the influence of geometrical parameter and prestressed on the dynamic response. Dynamic simulation was carried out with the finite element software FER/Impact (FENG et al, 2006) which is developed at LMEE laboratory. This software uses a first order implicit algorithm for time integration and the Bi-potential method for the frictional contact solution (FENG et al, 2010). It allows simulating efficiently impulse response tests.

4 CONCLUSION

This paper presents an application of impulse response technique to characterize rock anchorage. Compare to classic static test, dynamic testing is fast, safe and convenient. It gives information for maintenance administrator about anchorage such as length continuity of the rod without any break, singularity from statistical sample and load with static calibration.

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Tree-anchored rockfall fences: experimental and numerical studies

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Keywords: rockfall protection, forest, trees, impact experiments, discrete elements

In mountain areas, the protective capacity of forest against rockfall hazard is widely recognized (Volkweinn et al., 2011). However, the protective function of forest cannot completely guarantee the security of infrastructures and people. Moreover, the protection capacity of forests can fluctuate due to temporary tree density decrease due to silvicultural works, or damages caused by wind-storms or large snow avalanches. To secure such zones exposed to rockfall hazard, rockfall protection kits can be installed. These structures are largely implemented in the European Alps and other mountain areas and are significantly effective against rockfall hazars. However, the installation of these devices is often technically complex. In particular, these structures generally require building concrete-made anchors. The ecological impact of these protection works is thus significant. In this paper, we propose a more ecological and economical solution for the protection of zones surrounded with forest and affected by low energy rockfall events consisting of a flexible rockfall fence using trees as post supporting the flexible net.

The proposed structure design is voluntary simple and is characterized by a reduced number of anchors and simple components allowing a fast and economically feasible setting. The structure presented in this paper has been designed to resist to rock impact energy of 150 kJ which corresponds to a low impact energy level, as described in the ETAG027 classification (class 0). The structure design is based on a typical European forest in terms of tree density, tree diameters and bending strength.

Numerical studies have been carried out in the past for a better comprehension of the physical processes involved in the impact of blocks on flexible fences (Volkwein,2005, Bertrand et al.,2012, Thoeni et al.,2013), stand trees (Jonsson et al.,2007), or felled trees (Olmedo et al.,In press). These studies provide important information for the design of optimized structures. Nevertheless, experimental results remain necessary for numerical model calibrations.

In order to develop an optimized product, in terms of rockfall energy dissipation and structure commercialization, experimental and numerical investigations are currently being carried out. The results from these experiments will be used to calibrate and validate a numerical model describing rock impacts on tree-anchored flexible fences. Both the experimental and numerical studies are introduced in the followings.

1 EXPERIMENTAL STUDIES

The innovative temporary protection solution proposed consists of a rockfall catchment fence composed of different modules separated by standing trees (Fig.1). The distances between contiguous trees, ranging between 8 and 15 meters, determine the module length. The fence height ranges between 3 and 5 meters. Upper and lower longitudinal cables support a flexible net (simple twist mesh). The longitudinal cables are attached to trees at the extremities and are free to move horizontally around the central trees. Complementary cables are used as guy-lines to guarantee the stability of the extremity trees. No civil engineering works, such as concrete foundations, are needed.

In-field real-scale experiments are carried out to simulate real impacts on forest flexible rockfall fences. A zip line is installed perpendicularly to the fence to guide and accelerate an impacting mass (polyhedral concrete block of 690 kg following the specifications of the European guideline ETAG 027). This 60 meter long zip line allows accelerating the block to 18 m/s before impacting the fence. A quick release system is used to free the block from the zip-line before impacting the fence (Lambert et al., 2009).

The experimental device is instrumented to obtain data from each impact (Fig.1). High speed cameras allow identifying the precise impact location on the fence and the block and fence displacements during impact. Moreover, force sensors are installed on the upper and lower longitudinal cables to measure the loading of these components.

Complementary experiments are foreseen to characterize the mechanical response of trees to bending loadings. This response depends on tree species, growing conditions and on the loading axis. For this reason, it is necessary to obtain additional information. Thus, winching tests (Peltola et al., 2000) are carried out on trees presenting similar physical and geometrical

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characteristics as those tested on the impact experiments. This data is used as input parameter of the numerical model developed. Moreover, laboratory biaxial tensile tests are carried out to characterize the wire net.

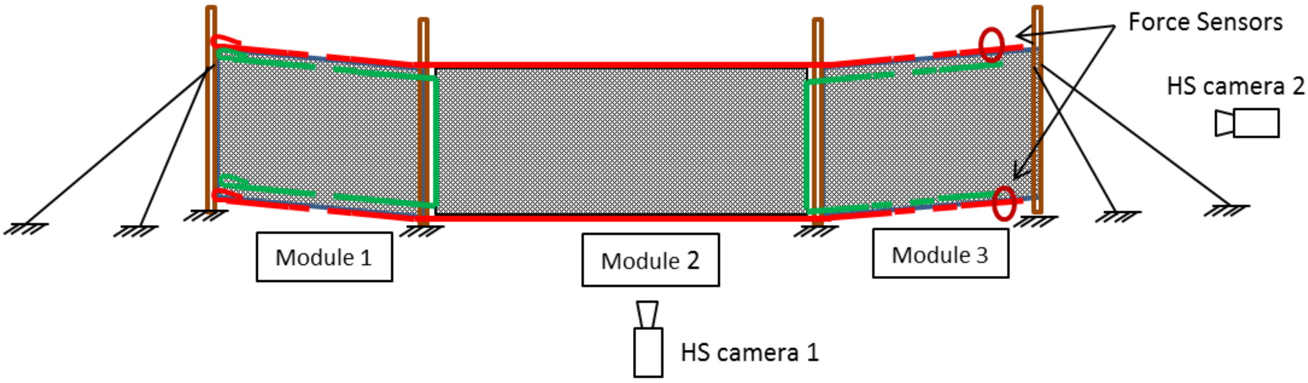

Figure 1. Tree-anchored flexible fence and instrumentation considered for the experiments.

2 NUMERICAL STUDIES

A numerical model of a block impact on a tree-anchored fence based on the Discrete Element Method (DEM) has been developed using the Yade-DEM code. The numerical developments take advantage of previous investigations concerning, on the one hand, impacts on trees and wooden structures (Olmedo et al., In press), and on the other hand, block impacts on flexible steel fences (Thoeni et al,2013). Thus, the numerical model of forest flexible fence integrates the mechanical description of trees (i.e. fresh wood) and the simple twist net response. Given the complexity of this formulation, experimental data are required to calibrate the numerical model. In particular, the resisting bending moment associated with the tree anchorage is assessed by the winching tests results. And the net mechanical characteristics will be defined from the results of the biaxial tensile tests. After this calibration process, the model is validated by the in-field full scale experiments results.

3 ACKNOWLEDGEMENTS

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ELITE® facing system: a new approach for superficial rock slope stabilisation

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Keywords: slope stability, facing, test bed

1 ELITE DEVICE DESCRIPTION

With many experiences in building embankment, GTS has developed its own facing system to prevent the superficial instabilities. Composed of wire nettings and steel rope, the assembly to anchorages use a connecting plate developed to meet the requirements of SNCF, the french railway infrastructure manager. In particular, the strength of the connection prevents the propagation of a rope breaking to the neighbouring cells.

This product is designed with a specific geotechnical model developed in partnership with the 3SR laboratory at the University of Grenoble. An experimental campaign has rated the mechanicals characteristics of the ELITE® design. The test protocol is designed specifically for a flexible facing system loading.

Figure 1: ELITE® facing concept

2 SIZING MODEL OF THE FACING

The ELITE® facing system prevent surface instabilities consecutive to the surface soil degradation of slopes which are, moreover, globally stable. This phenomenon is modelled with a gradient of cohesion applied to a superficial layer of a few meters. The thickness is selected according to the site.

The initial stability is determined by the Bishop-Fellenius method, then the pressure on the facing system and the anchorages forces are calculated to achieve the overall stability coefficient desired by the customer.

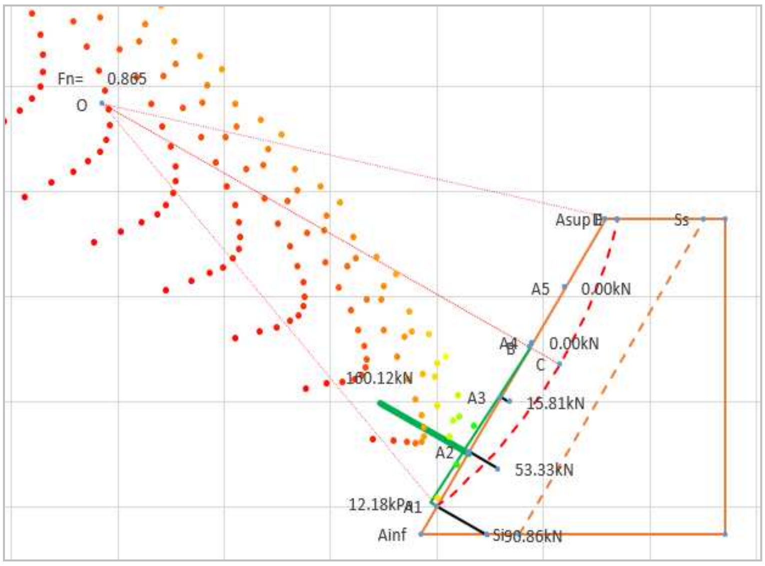

Figure 2: sizing model

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3 MECHANICAL CHARACTERIZATION TEST

To characterize the ELITE® facing resistances, a specific test device was developed. The facing sample is placed inside a horizontal metal frame which is representative of a mesh (size: 3m x 3m). The soil action is modelled by a flexible water bladder tank which guarantees equal pressure throughout the model.
Two types of tests are performed.

3.1 PUNCTURE TEST

Figure 3: Puncture test

This test allows assessing the puncture limit resistance of the facing at the anchor point. The facing is placed on the free surface of the pocket of water and a cylinder centered on the connecting plate applies an increasing effort to failure.

With the water tank use, there's no interference between the test device and the facing action.

The instrumentation, consisting in a central compression force sensor and a linear displacement sensor, provides the Effort / displacement curve.

Kind of facing	Minimal puncture resistance
100/120 steel wire mesh + standard plate 200x200	21kN
Mesh 100/120 + 1 cable ϕ12 + 1 Elite® plate	58kN
High tensile steel mesh + Specific plate	64kN
100/120 steel wire mesh + 2 cables ϕ12 + 1 Elite® plate	121kN

Table 1: mains puncture resistances

3.2 PRESSURIZED TEST

This test allows assessing the pressure limit resistance of the mesh localised between the ropes. The mesh is placed on the free surface of the flexible water bladder tanks and the water pressure is increased until failure (about 25kN/m2).

This test prove that this failure mode is not the weak link of the system.

The instrumentation provides Pressure / central displacement.

Figure 4: Pressurized test

Session 2B

Case study (1)

Methodology for numerical simulation of Tolosa rock avalanche in Argentina

Dominique DAUDON[1], Stella Maris MOREIRAS[2], Xavier BODIN[3], Stiven CUERVO[4]

Keywords: Andes, rock avalanches, Discrete element modelling, terrestrial photogrammetry

Because of their tectonic settings and the presence of an extended high-altitude cryosphere (snow, glaciers, permafrost), the central Andes are highly impacted by gravitational movements like rock avalanches, rockfalls or landslides. Between Mendoza (Argentina) and Santiago de Chile (Chili), the Road 7 is a strategic axis for the South American economy, suffering from a large variety of endangering phenomena. Recently, Moreiras et al. (2005) mapped past (Holocene and Pleistocene) events using geomorphological evidences. In order to increase the pertinence of this work for elaborating hazard risk maps, numerical modelling is a promising tool (Corominas 2014). We used a Discrete Element Model (DEM) code developed at 3SR to simulate real rock avalanches propagation in France (Cuervo 2014). Because of scaling effects in geometry and processes, the application to events of the may require adaptation of the parameters introduced: mechanical energy restitution rate, enlargement of the topography, volume of the rock blocks, high elevation difference, and poor initial data availability. The choice of the Tolosa pleistocene rock avalanches relies on the accessibility of the valley and the deposits, the strategic place near the road, the ease for the photographic data collection. Preliminary steps to run simulations include: Structure-from-Motion (SfM) processing of images collected on the field in May 2014 in order to get a high resolution (5m pixel spacing) topography (especially for the scar rock wall); merging of the resulting Digital Terrain Model (DTM) into a 20-m resolution SPOT DTM in order to fill the gaps in the SfM's DTM; manual (xpert-based) reconstruction of the initial topography of the terrain at the rock avalanche's deposit place; choice of the mechanical parameters in relation with geological considerations in order to simulate the blocks propagation and run-out distance.

1 3D ACQUISITION OF TOPOGRAPHIC DATAAND MESHING

Located at Las Cuevas locality (Mendoza, Argentina; 3.557m - 32°48′50″S, 70°02′59″O), the Tolosa rock avalanche (fig 1) has a deposit more than 50m thick, for 0.7km wide and 1.5km long, mostly made of pyroclastic rocks (andesite). The surface is covered with huge blocks (7 to 50m length) for an estimated volume of $50.10^6 m^3$. Regarding the origin of the rock avalanche, Peyrera (1997) suggested that it could have been triggered by an earthquake, whereas Rosas (2008) discussed the possible role of glacier or permafrost melting during the deglaciation period after the Late Glacial Maximum.
In order to use SfM techniques (for ex. Bodin et al., 2015) for reconstructing high resolution DTM of the study area (from the upper parts of the scar to the snout of the deposit), more than 40 pictures were taken with a fixed-focal 24mm lens mounted on Nikon D7100 digital camera from different altitudes and different view angles. The SfM processing was completed in Photoscan software (Agisoft), with an alignment procedure yielding an RMS error of 0.42m. The 3.9 Mio point cloud we generated was then meshed and gridded on a raster with a 5m pixel size (Fig 2).

Figure 1: Tolosa rock avalanche, from the Cristo Redentor pass.

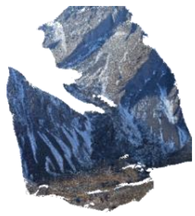

Figure 2 : SfM mesh with the ortho-image overlaid Non reconstructed shielded areas due to view angles.

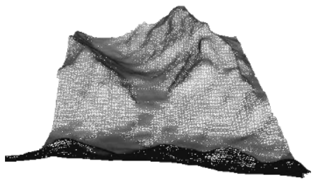

Figure 3 : the final 5-m grid, after merging with 20-m SPOT-DTM, used for the first DEM simulation

Gap filling of the initial SfM-derived 5m grid was performed merging this latter with a 20m DTM made from SPOT satellite imagery into a Geographic Information System (GIS). Visual inspection shown a clear improvement of the terrain geometry representation, especially in the source area and some parts of the deposit (Fig 3). A decimation step is then done to get a mesh compatible with reasonable numerical calculations times (Fig 4-left). The deposit geometry is estimated by manually interpreting the continuation of the lateral taluses below the deposit. The initial volume is reconstituted by intercepting a

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rectangular parallelepiped body with the tridimensional terrain at the departure zone then split into layers of cubic spheropolyhedron blocks with the preprocessing tool- CARVER- (Fig 4) developed by Richefeu (2012).

2 DEM MODELLING AND PARAMETERS CHOICE

The Tolosa rock avalanche has been chosen in order to estimate the capability of using the discrete element method in modelling the avalanche. The numerical parameters were defined following the methodology proposed by S.Cuervo in his PhD (2015).

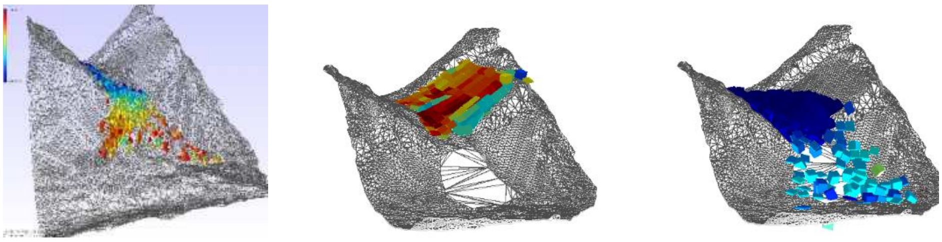

Figure 4 : final reconstituted topography ; left) 467 blocks of 30m side simulation velocity at 200s on coarse mesh. center) 344 blocks of 80 m realistic volume and slope without deposit same simulation at 10 s. right) same simulation

The particles interact between themselves and the substratum by a dissipative law including normal and tangential elastic thickness, friction dissipation, restitution coefficient (Richefeu & al 2012). No rolling dissipation law to consider the propagation of blocks into very soft soils has been used in this first simulation,.

Table 1: first set of mechanical parameters chosen to optimise the calculation time

	WEIGHT $*10^3$ T (MINI-MAXI-AVERAGE-TOTAL)	RESTITUTION COEF.	VOLUME M M^3	RIGIDITY $1E^9 KN/M^2$	TIME STEP (SECONDE)	DENSITY (KG/M3)	DURATION FOR 0.1S –TOTAL (65S)	FRICTION COEF.
464 BLOCKS OF 30 M (FIG 4 LEFT)	1.8-231-44.7-20700	0.1	10	1	10^{-05}	2000	40 MN (17 DAYS)	0.1
344 BLOCKS OF 80 M (FIG 4 RIGHT	162-1390-904-222000	0.05 /0.1	79	25	$2.5\ 10^{-05}$	2800	12 MN (6 DAYS)	0.2

3 RESULTS AND DISCUSSION

The first simulation has been carried-out with ¼ of the initial volume in satisfactory calculation time (7 days on standard PC computer). Complete initial volume modelling is still necessary but the deposit was supposed to incorporate 40% of matrix material and so coarse granulometry that can't be taken into account in DEM. A great part of the blocks stops in the Hombre Cojo valley. The flow turns towards the Cuevas valley with velocities around 60m/s (200km/h) at the front position which is the range of large rock avalanches according to literature (Jibson &al 2006) for longer propagation tracks. More calculation has to be done as the topographic mesh still contain the old deposit, and the numerical run out is larger than the real one even if few real blocks crossed to the other side of the valley.

4 CONCLUSION

A GIS methodology to get topographic data on large area combining terrestrial photogrametry data and Spot image patching has been developed in order to get a slope triangle mesh. A first simulation by feedback analysis has been done using a simplified case in Argentinean Andes to verify the capability of DEM model into smaller gravitational movements. The simulations show encouraging results in terms of propagation direction and velocities ranges according to the literature. However, it seems that it is necessary to better assess the real initial volume, by varying the mechanical parameters to fit correctly with the actual deposit. It may be necessary to calibrate them and introduce other contact laws including rolling resistance or to consider the crushing of blocks as matrix is not considered. The analysis of very old events like this are unusual that shows their difficulty specially in discriminating the part of uncertainty of the results due to "expert" choice of parameters. Sensitive studies are needed to estimate the range of variation of the influence.

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Microseismic and meteorological monitoring of Séchilienne (French Alps) rock slope destabilisation

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Keywords: microseismicity, source location, rainfall threshold

Rockfalls and deep-seated gravitational slope deformations are recognized as a major natural hazard across the mountainous regions with strong economical and social impacts on regional land settlement and transportation policies. In recent years these geohazards are increasing as a result of climate change and rapid expansions of human habitats and critical infrastructure. Unfortunately, the physical mechanisms and trigger of slope destabilization are rather complex and strongly site variable and thus, difficult to predict. As a result, some of the current researches aim to design efficient monitoring techniques able to detect changes in the slope conditions in order to anticipate catastrophic failures. Microseismic, meteorological and geodetic monitoring approaches are tested in this context.

Here, we present ongoing investigations on the microseismic and the meteorological monitoring approach for Séchilienne case study (south-east of Grenoble city), being the most instrumented French hazardous slope for more than 20 years. The Séchilienne slope is characterised by a deep progressive deformation controlled by the network of faults and fractures and induced by rainfall (Vallet et al. 2015b). A particularity of this landslide is the absence of a well-defined basal sliding surface (Leroux et al, 2011). Two microseismic networks have been installed. A permanent seismic network, installed in May 2007 (Helmstetter and Garambois, 2010) by the the Multidisciplinary Observatory of Versant Instabilities (OMIV), to supplement the geodetic and geotechnical monitoring system operated since 1985 by the French "Centre d'études et d'expertise sur les risques, l'environnement, la mobilité et l'aménagement" (CEREMA) (Durville et al., 2009). Later in November 2009, a temporary microseismic network was set up by the National French Institute for risks and environment (INERIS). It consists of experimental in-depth probes and it is only one of the components of the INERIS experimental multi-parameter system, placed along the West border of the very active zone of the landslide (Dunner et al., 2010).

1 MICROSEISMIC MONITORING

Observations from the permanent and temporary network both indicate that microseismic monitoring is a useful tool in order to survey slope activities as rock falls and other destabilizations mechanisms. Accordingly, two major microseismic crisis (with more than 100 events a day) have been recorded during significant rock fall activity in 2012 and 2013.

Beyond data qualification and quantification of the seismic rate, location of the corresponding microseismic sources is the second step in microseismic analysis but is quite complex above all with temporal networks with no dense stations number and poor azimuthal coverage. In addition, the location is a challenging task mostly because of: the lack of clear seismic phase arrivals times and strong heterogeneities in the local velocity structure.

In this study, we tested the potential of an amplitude based location approach (Kinscher et al. 2015) with some data coming from the 2013 activity. Results of this approach are consistent with previous location results (available on http://omiv.osug.fr/SECHILIENNE/SISMO/data.html) obtained by using a beam-forming approach (Lacroix & Helmstetter, 2011) (Figure 1). Even though amplitude based location seem to be somewhat less precise than the beam-forming approach, it provides automatic event location in quasi real time, which could significantly improve local hazard monitoring. Other efforts are currently done to better resolve internal destabilization dynamics of these rockfalls by using the high frequency component sensors installed in boreholes.

2 METEOROLOGICAL MONITORING

Rainfall threshold is a widely used method for estimating minimum critical rainfall amount (rainfall index) which can yield a slope failure. Vallet et al. (2015b) showed that a threshold definition based on rainfall and effective rainfall coupled with antecedent and precedent rainfall index is the most appropriate approach for the Séchilienne landslide. Indeed, the displacement velocity of the Séchilienne instability is mainly controlled by the hydrodynamic behaviour of the landslide hydrosystems (Vallet et al. 2015a, b). Moreover, previous works (Helmstetter and Garambois, 2010 and Klein et al, 2014) showed a potential weak correlation between local microseismicity and rainfall.

To better understand the relationship between microseismic activity and rainfall, effective rainfall, in addition to precipitation, was considered for threshold definition as being the input signal of the landslide hydrosystems. Literature reviews show that

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most of the threshold studies are subjective and not optimal. Support vector machines (SVM), a supervised learning method, is used for establishing an objective threshold for the landslide. The calculation of the is in progress: all our data from 2009 to 2014 are precipitation data coming from weather station (CEREMA station includes a rain gauge gauge and is located on the our microseimic network.

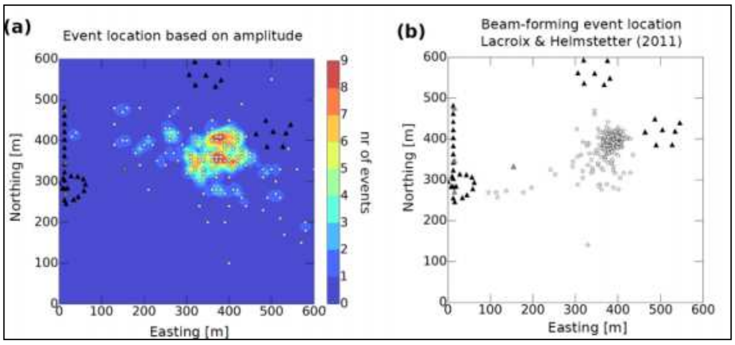

Figure 1: Example of event location results using an amplitude based (a) (Kinscher et al 2015) and a beam-forming based (b) (Lacroix & Helmstetter 2011) approach for ~ 250 events detected on the 20/10/2013. Black triangles refer to seismic stations of the permanent OMIV network, while gray triangles mark station positions of the temporal INERIS network.

3 CONCLUSION

We present current investigations aiming to improve microseismic and meteorological data analysis of the Séchilienne rock slope. We provide a new automatic location tool providing event location in real time, whose results are consistent with previous location ones (OMIV web page) and the in situ observations. Complex slope failures are predisposed and triggered by hydrogeological conditions. Our combination of microseismicity with groundwater recharge is ongoing and detailed results will be present in the next future.

4 ACKNOWLEDGEMENTS

Special thanks are extended to the CEREMA for the meteorological data, OMIV, and the French Ministry of Ecology, Sustainable Development and Planning for financial support.

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The Chambon landslide (2015) and its consequences

Alexandre MATHY[1], Valentin LE BIDAN[2], Lionel LORIER[1], Denis FABRE[3]

Keywords: natural hazards, landslide, monitoring, road construction, embankment, excavation, blasting

1 A LANDSLIDE CUTS THE DEPARTEMENTAL GRENOBLE-BRIANCON ROAD

Following the collapse of part of the roof of the Chambon tunnel (total length: 750 m) near its east portal on 10th April 2015, the road connecting Grenoble (Isere Department) to Briançon (Hautes-Alpes Department) on the right bank of Lake Chambon (RD1091) was closed to all traffic. As a consequence the Isere Department urgently adapted planned remedial works which had originally been scheduled for June 2015. Despite several attempts to strengthen this zone of the roof, the rapid acceleration of the movements meant that the planned reinforcement solution had to be abandoned. The tunnel collapsed completely over a 20 meter long section at the beginning of July 2015.

In early May 2015, emergency geological and geophysical surveys were conducted by SAGE (Société Alpine de Géotechnique) and CD38 (Conseil Départemental de l'Isère). These showed that the tunnel defects were directly related to the presence of a large landslide that affected the slope down to the Lake (Fig. 1). Just a few weeks after the discovery of this slide, which has a volume of around 600,000 m^3, it was put under surveillance using crack gauges and automatic target tracking with automated topographic theodolites. Three weeks after the introduction of this monitoring, a forecast was made by SAGE that failure in early July was likely (see Fig.2). Large movements actually occurred during the night of 5th to 6th July 2015. That night, some targets recorded a maximum displacement of 45 meters down to the lake.
The instantaneous travel speeds measured in real time before failure (D-Day) were as follows:

- to 7 cm/day to D-25,
- to 20 cm/day to D-10,
- 10-45 cm/day at D-5,
- 40 cm to 7 meters/D-1,
- 3-20 m/day at break with maximum speed
- of 3 to 5 m/hour)

Figure 1: The slide on the right bank of the Chambon Lake.

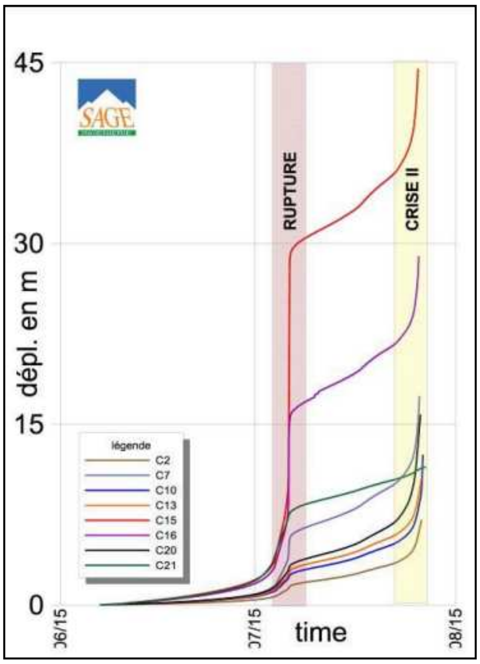

Figure 2: Displacement curves (June-August 2016)

To accelerate the purging of the moving part of the slope and to allow the shuttle service witch had to be suspended due to high slip activity (boat services were implemeted urgently to ensure temporary connection between the two Departments), it was decided to raise the level of Chambon Lake, following protocol having previously demonstrated that the sliding reacted

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positively to this type of triggering. This slight hydraulic load increase in the lower part of the slide will be effectively responsible for the geological crisis ("crise II") in late July (see Fig.2).

2 THE RELIEF ROAD: A LARGE-SCALE CONSTRUCTION MADE IN A RECORD TIME

Facing a major geological hazard on the right bank of the Lake and considering the magnitude of the resulting unprecedented social and economic crisis for the valley, the Isère Department set out in late June 2015 to create a relief track on the opposite side of the valley, so that emergency access between the Departments of Hautes-Alpes and Isère could be maintained.

To meet the challenge of developing this 5.3 km long backup road within four months, engineering studies were immediately started running in parallel with the initial preparatory works. The critical issues were:

- the securing of the future road against rockslides and avalanches,
- the need to cut a platform in a largely inaccessible cliff (see Figure 3 and Figure 4), with a volume of 15,000 m^3 without causing further instability,
- the reinforcement of the works (excavations and embankments), stream crossings and road connections,
- the creation of a usable width of 5.50 m carriageway enabling two light vehicles to cross at reduced speed; shelters were required at a maximum of 150 m and the longitudinal slope was to remain below 10%.

Figure 3: General view of the left bank before work

3 CONCLUSION

All means were used to restore light vehicle access before the winter season 2015-2016. Twenty to forty operators worked every day on the site for a project organized in shifts of 6 to 7 days, with working hours from 06:00 to 21:00. The construction of the relief road on the left bank of the Lake, initiated on 20th of July, allowed the road to reopen on 24th November. The total cost of the operation reached € 7 million.

Figure 4 : Mining of the central spur on the left bank

Currently, the slip on the right bank continues to move at a residual speed of 5 to 11 cm/month. In order to restore the main road in the long term, complementing the backup route, the selected option is to construct a 950 m long diversion tunnel using the first very stable 500 meters of the existing tunnel. The aim is to allow a controlled recirculation of the RD1091 before the 2016-2017 winter season.

Optimization of a rockfall protection embankment above the A43 highway

Jean-Philippe JARRIN[1], Camille HALBWACHS[2]

Keywords: rockfall, embankment, optimization, highway management, tunnel

The A43 highway between Lyon and Chambery crosses the Pre-Alps by 2 tunnels. The west end of the Dullin tunnel is located at the base of a 200-m-high limestone cliff. The tunnel is extended by 20-to-30-m-long galleries, supporting a service road (figures 1 and 2). The highway administrator wanted to protect the highway against rockfall and, if possible, the service road. GEOLITHE was the project manager of these works.

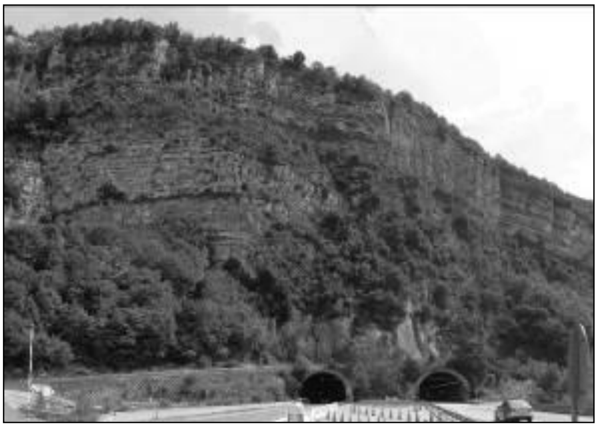

Figure 1: Sight of the cliff from the highway

Figure 2: Sight of the highway from the top of the cliff

1 DISCUSSION ABOUT THE DIFFERENT SOLUTIONS

Rockfall simulations revealed that, at the position of the service road, the trajectories are very high: up to 25 m. As a consequence, it was not possible to stop all the blocks only with rockfalls barriers or embankments.
A first solution was to build deflectors with high resistance wire mesh, and to intercept blocks at the foot of the cliff with an embankment. This solution, which would have covered the cliff, was unthinkable for environmental reasons.
It was so decided, in agreement with the highway administrator, to anchor the more unstable rock blocks above 1 m^3 volume, and to build an embankment at the foot of the cliff which would certainly be lobbed. As a consequence, the outside galleries can be damaged by a rockfall. The aim of the design was then to optimize the embankment in order to intercept as many blocks as possible.

2 OPTMIZATION DURING THE DESIGN

A first design was a 9-m-high embankment, situated at the end of the galleries (figure 3). It allowed to stop about 99.5% of less than 1m^3 rockfalls.
In order to match the resistance of the embankment and the kinetic energy of rockfalls, it was planned with a thickness of 4m, and had to be made with gabion cages ligatured one by one, filled with a crushed quarry stone.
In order to limit the dynamic load on the gallery structure due to the rockfalls impacts, a multi-layer filling was imagined, with rolled gravel on top,

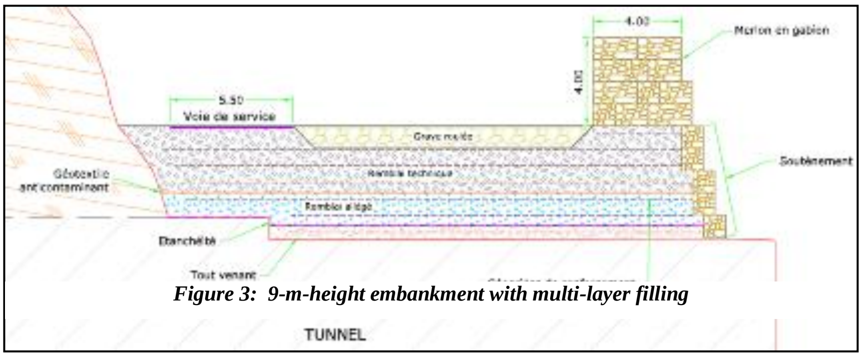

Figure 3: 9-m-height embankment with multi-layer filling

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reinforced soil with geotextile layers to spread dynamic strains, and lightweight material to reduce static strains.

This solution was finally not feasible, because the foundations of the galleries could not support the weight of the whole structure.

It was so decided to build a gabion embankment, as high as possible (4 m), at the end of the galleries. It allows to stop about 99% of less than 1m³ rockfalls. The service road has been located just behind the embankment, in order to be less reached by small rockfalls (probably the most frequent). In order to limit the dynamic load on the gallery structure due to the rockfalls impacts, a 2-m-high filling of rolled gravel was planned on the galleries structure, between the cliff and the service road. See figure 4.

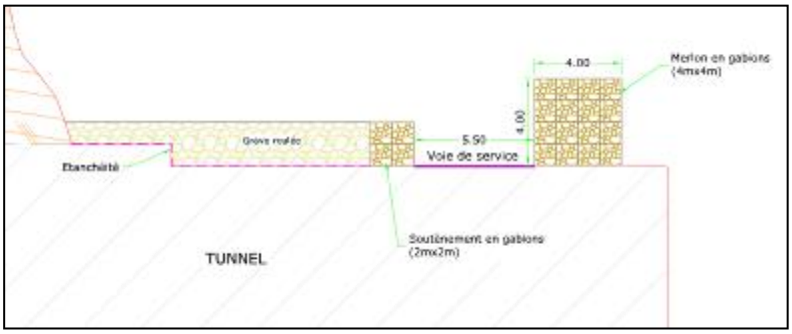


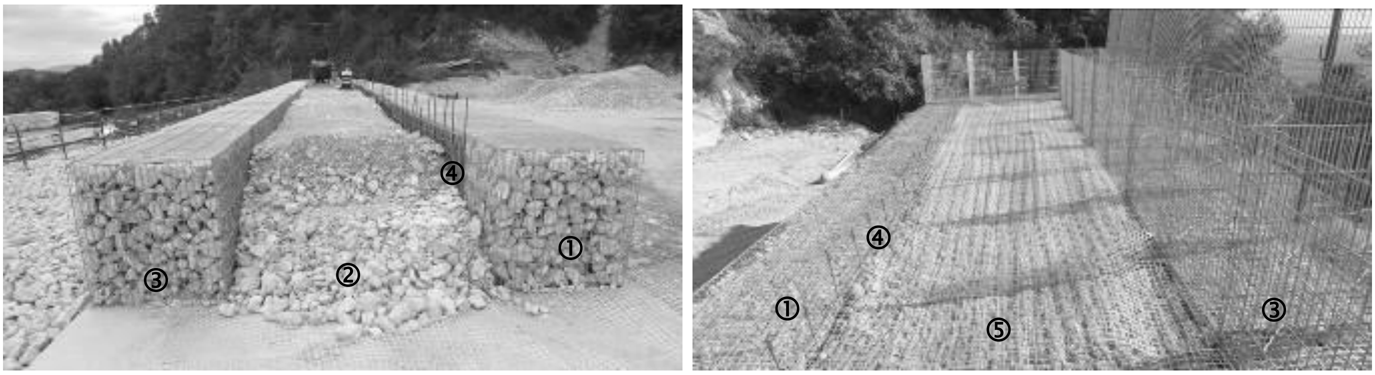
Figure 4: 4-m-height embankment

3 OPTMIZATION DURING THE BUILDING

During the construction, the working company Forézienne d'Entreprise (Eiffage) wanted to limit the gabion volume with a sandwich structure composed of two gabion walls around a crushed material. Such structures are less resistant than a structure entirely composed of gabions cages. Indeed, the structure is less intertwined and the lateral spread of dynamic strains is less significant. A reinforcement of the structure had to be imagined.

First, the structure stability has to be ensured by horizontal geogrid layers added between the two gabion walls.

Then, a vertical welded mesh, made of 9-mm-diameter bars, is positioned just behind the cliff-side gabion wall, in order to constitute a rigid and resistant layer which will be able to spread the strains due to an impact. In order to allow the



implementation of the geogrids, the vertical continuity of this reinforcement is ensured by vertical metallic bars.

Figure 5: Different sights of the building

① gabion cages on cliff side – ② crushed material – ③ gabion cages on highway side – ④ reinforcement structure made of vertical welded mesh) – ⑤ geogrid

4 CONCLUSION

The different optimizations carried out during the design and during the building resulted in an embankment which fully met the expectation of the highway administrator, realised in a short time and at a limited cost.

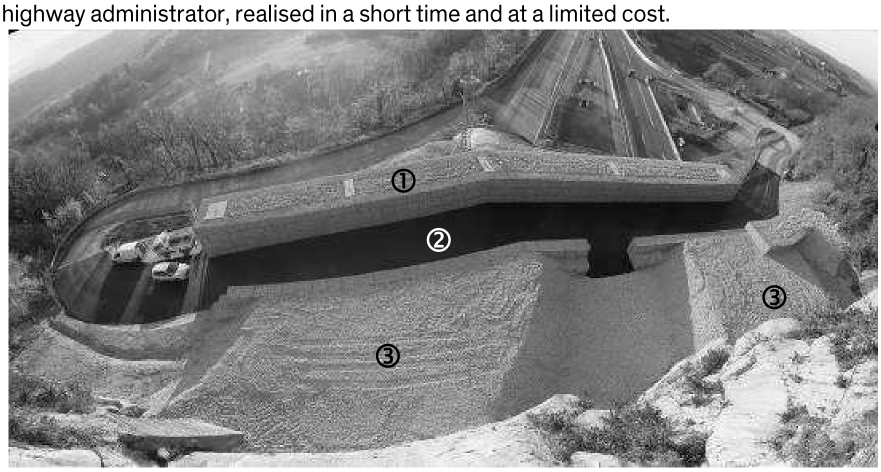


Figure 6: Sights of the finished embankment

① principal embankment – ② service road – ③ rolled gravel on galleries structure)

Securing the Cliff of Bon Voyage- NICE- A case study of 3D Laser scan contribution in an urban blasting

Tristan BOURDIN[1]**,** Gregory MESLY[2], Thierry PARTOUCHE[3], Philippe BOUDON[4]

Keywords: **emergency works, laserscanning, rockfalls, blasting, cliff reinforcement, mitigation**

1 INTRODUCTION

Our subject is located in the eastern part of Nice in French Riviera. The site consists in an anthropic cliff made of limestone and dolomites from the Jurassic stemming from a quarry exploited in the first half of the twentieth century. The cliff dominates the Nice-Tende railway line and numerous residential buildings. Because of several collapses, this cliff has been studied since the early 2000s. From 16th until 19th of January 2014, a very intense Mediterranean depression occurred, leading to heavy rainfalls, 270 mm were recorded in this area.

The activity of the cliff increased on January the 30st and it partially collapsed in a 6000 m^3 rockfall.

This paper gives a report on mitigation activities in the aftermath of the cliff collapse which pulled the closure of the railway line and evacuation of several housing of this inhabited area.

2 METHOD

Directly after the event, an emergency investigation was carried out by GEOLITHE, and showed that the cliff presented in its upper part two large instabilities forming an arc thus preventing any purging. The volume of the instabilities (about 1590 m^3), their degree of fracturing and their high risk of collapse led to the interruption of the railway line and to the evacuation of several families. The geological conditions prevented any reinforcement works in a secure way, thus the remaining solution chosen was to blast the upper part of that cliff.

The preparation phase took 4 months in difficult and challenging conditions. The working zone was protected at first by numerous punctual rock reinforcements and a 90 meters long ETAG like class A rockfall barrier. An instrumentation of the cliff with extensometers was also implemented for surveying; 600 linear meters of boreholes were performed by acrobatic drilling to prepare the blast. The density of fracturation required odex technics to perform the boreholes.

This very special blast occurred at 70 meters high and 70 meters above the nearest buildings. In order to fire with maximum safety, several methodologies were implemented :

- A statement of fracturing was lead for each drilling ;
- An inclinometer measurement was lead for each drilling ;
- A detailed exhaustive 3D laserscan acquisition was carried out from 6 test points. This measurement led to structural analysis of the fracturing and gave to the miner a 3D model with the geometry of the boreholes;
- 3D loading calculus and 2D flyrock simulations were performed.

This method allowed to supply to the miner the volume of rock covering along each borehole, the lengths which can be loaded and the fracturing network. DCI has been chosen to destabilize this arch with a carefully controlled blast, leaving gravity to deal with the rest.

A special protection was designed and implemented with metallic nets doubled by wire nettings and by geotextiles in the cliff to protect the buildings from the projections. On the loading area, protections were placed previously to the loading stage and opening made in the geotextile above the hole to allow loading (cf. Figure1).

As a precautionary measure, a wide security perimeter was deployed around the site. The 1400 concerned residents were asked to leave this area during the day of the mining and to not leave their vehicles in outdoor car parks.

The good progress of this operation was made possible by an important campaign of information and preliminary communication of the population with the support of the municipal reserve of civil safety (RCSC) and by the implication of the

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police forces (360 people were involved in the operation). More than 450 residents were taken care by the city services and buses transported them towards one of the three home sites (schools).

The coordination of the operation was led from an operational headquarter situated nearby, with a view of the cliff, under the piloting of the direction of Risks Prevention Service of Nice City Concil and GEOLITHE.
A meteo forecast was assured in connection with the forecaster of Météo-France and radio transmissions allowed the supervision of the operation.

Figure 1: View of the cliff during the preparation of the mining

3 RESULTS

This blast was done successfully the 25th of June 2014 despite a very bad weather report which delayed the hour of the firing (cf. figure 2). The results were beyond the expectations since no projection was observed in the direction of neighboring structures, or even on the railway. After the blasting, on the basis of detailed diagnostics (cliff diagnostic conducted by GEOLITHE and buildings diagnostic conducted by DPGR and a technical auditor), inhabitants were allowed to come back home in the early evening.
During summer the cliff was temporarily protected by wire nettings. A temporary bund was constructed to protect the railway and the buildings and the families evacuated after the event were reintroduced. Complementary studies are actually underway to design definitive protection structures as well as the monitoring of the cliff face by 9 extensometers. The protection works should begin this year and should last till 2018.

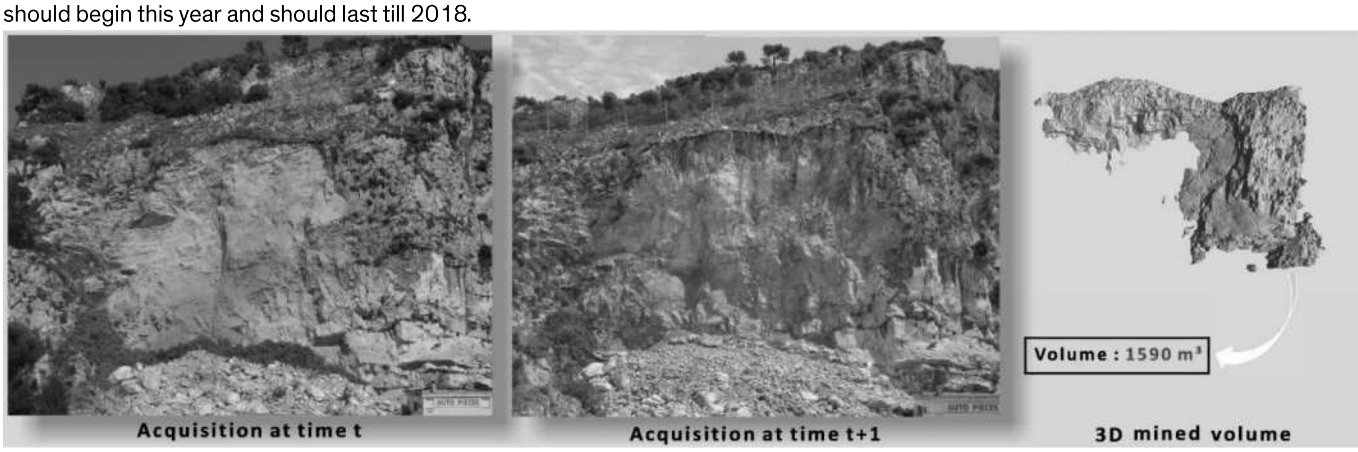

Figure 2 : The cliff before and after the blast + volume of mined rocks resulting from a laserscan of the cliff

4 CONCLUSIONS

The presented methodology allowed to secure temporarily a big cliff in an urban area by blasting using a unique combination of field measurements and advanced technologies. The contribution of 3D laser scan allowed to master the mining and the associated geological hazards.

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Session 3A

Protective structure design (3)

Rock debris flow mitigation using flexible structures

Nicolas VILLARD[1], Mathieu SCHMITT[2], Stéphane LAMBERT[3]

Keywords: debris flow; torrent; cable anchors; instrumentation; DEM modelling

1 BACKGROUND

Rock and debris flows are a complex and dangerous phenomenon in mountain, nowadays exacerbated by climate change, and causing regular damages to urban areas and infrastructures. It is a mixture of mud and rock boulders with high energy levels, met in some torrents after heavy rainfall.

Using flexible structures derived from CE / ETAG rockfall barriers offers sometimes efficient alternates to conventional mitigation structures (concrete dam, gabion). In some particular cases it could be the sole relevant option (due to difficult access, geological requirements or financial issues).

Figure 1 : Rock debris flow deposit – Valgaudemar / IRSTEA 2003

2 DESIGN APPROACH

2.1 GEOMETRY & COMPONENTS

Main components are derived from rockfall barriers tested according to CE / ETAG 027 guidelines, using wire rope nets, dissipation devices absorbing energy and cable anchors for foundations.

2.2 LOADING

Some experimental pilot sites have already been equipped in France with such structures [3]. The engineering design has been led using an analytical approach: starting from a conventional geotechnical soil pressure calculation, dynamic effects have been taken into account using empirical coefficients [1].

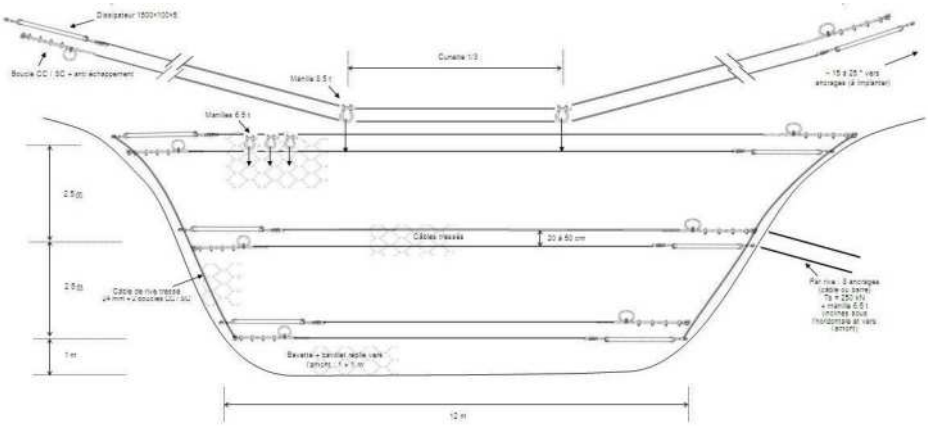

Figure 2 and 3 : example of wire rope barrier against debris flow

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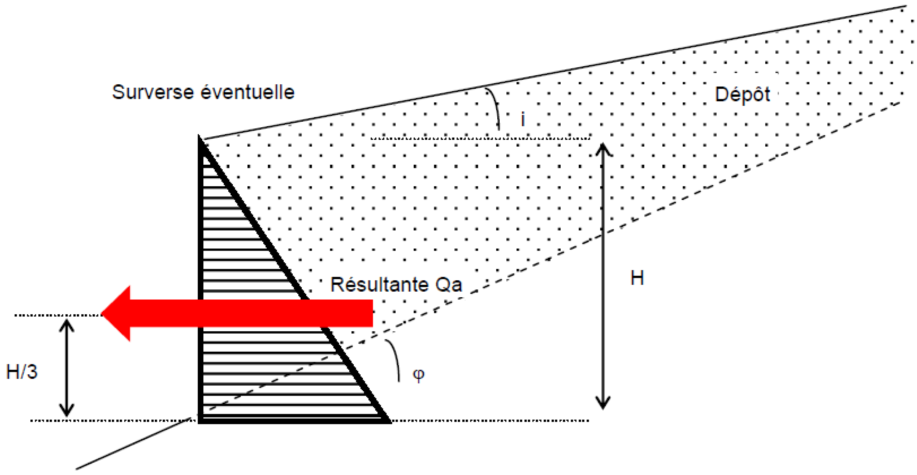

Figure 4 : cross section of loading

3 NEW DEVELOPMENT ISSUES

Representative events on these sites will now be used for back analysis. It is proposed to install monitoring systems in to get feedback information, and at the end improve scientific knowledge on such complex impacts / structures, and engineering methods for the design step.

3.1 ENGINEERING AND R&D

Main objectives of the program would concern:

- Dynamic / static effects
- Back calculation analysis using sensors
- Design of supporting ropes and breaking elements
- 3D numerical modelling of the structure (deformation, forces). [2]
- Limit range of use

3.2 INSTALLATION WORKS AND MAINTENANCE

Finally but with short term contracting issues, this concern as well field improvement such as:

- Installation of cable anchors with shearing deformation of weak soils
- Testing procedures and control for such foundation
- Cleaning procedure once the flexible dam is filled (diggers, micro explosives…)
- Components replacement, methods and costs

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Rockfall impact load dissipation within a cushion made of a granular material

Mathieu FEREGOTTO[1]**,** Laurent DUBOIS[2], Eric DIMNET[3]

Keywords: **Rockfall, granular material, impact load, dynamic behaviour.**

Protection structures like rockfall embankments or galleries use the dissipating properties of granular materials. As a consequence, designing these structures according to their ability to dissipate stresses caused by the fall of a boulder requires the characterization of the materiel under impacts of high energy.
A full scale testing device for rockfall protection works is hosted in an area close to Chambéry (France). This site is the property of the French Institute of Science and Technology for Transport, Development and Network (IFSTTAR), who is also responsible of the exploitation, together with the Center For Studies and Expertise on Risks, Environment, Mobility, and Urban and Country Planning (Cerema). The dropping device has been designed to get impact energy up to 13 900 kJ (a 20-ton boulder falling from a height of 70 meters).
Two campaigns of vertical impacts on a cushion made of coarse material (40-125 mm) have been organised on this site. Different parameters were tested: energy of the impact, shape of the boulder and thickness of the cushion.

1 IMPACT EXPERIMENTS

1.1 AIMS

The objective was to characterise the behaviour of a granular material in the specific case of an impact such as the fall of a boulder on a rockfall embankment or a gallery. A better understanding of this behaviour would lead to an optimization on the choice and the implementation of granular materials for rockfall protection devices, but would also improve the design of these structures.

1.2 EXPERIMENT METHODOLOGY

Fourty tests have been realised during two different campaigns. Three thicknesses, eight levels of energy and five different boulders were used to assess the influence of these parameters on the shock absorption.
The material is set up on a double plate sitting on top of a concrete slab and specially designed for the purpose of theses tests. It allows the positioning of ten force transducers at the base of the cushion (Figure 1). Accelerometers were set up into the boulder, and the instrumentation is completed with two high speed video cameras calibrated to allow the measurement of distances from recordings.
The drop with the chosen amount of energy is performed thanks to the positioning device of the boom crane (Figure 2), which allows a precision of about 10 cm in height.

Figure 5 : Positioning of the ten force transducers

Figure 4 : Set up before a drop

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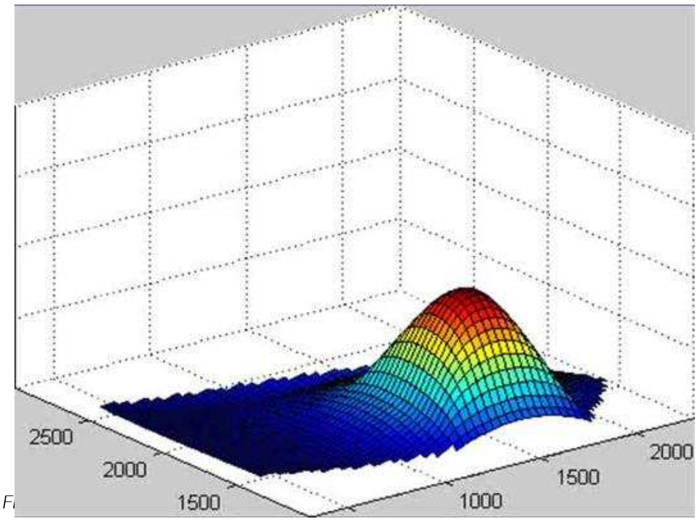
2 MEASUREMENTS

At the end of each test, the boulder is removed from the cushion thanks to the boom crane. The depth of the penetration is measured. Recording of stress measurements at the base of the cushion, deceleration of the boulder and high speed videos are stored together with common time reference for perfect synchronisation between the different data.

3 RESULTS

3.1 DATA PROCESSING

Some routines have been specially developed to process the important amount of measures coming from the fourty tests carried out. One of these routines is applied to the results given by the ten force transducers; it enables the visualization on an animated sequence of the evolution of the stress distribution at the base of the cushion during all the time of the impact (Figure 3). Another one is based on the processing of the high speed videos and gives the evolution of the depth of penetration (Figure 4).



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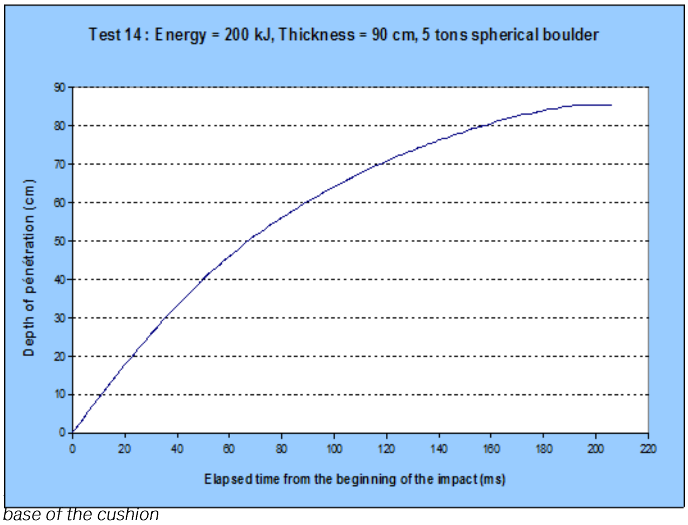

base of the cushion

3.2 ANALYSIS

The influence of the energy of the impact, the shape of the boulder and the thickness of the cushion have been observed by comparing stress distributions, accelerations and depths of penetration between the different tests. The dissipation of the energy can be quantified for each test through the shape and the time evolution of the 3D stress distribution. The behaviour of the granular material tested here is thus characterized for dynamic impacts such as rockfalls. These mechanical properties can be used for the design of protection structures.

4 CONCLUSION

Impact tests leaded on a coarse granular material have brought further understanding on its dynamic behaviour, regarding different parameters such as the thickness of the cushion or the shape of the boulder. Work is still in progress to improve and refine this results. Some tests are currently realised on a sandy material to characterise it and measure differences with the coarser one. In the end, this work should lead to recommendations for the dynamic aspect of the design of protection structures implying granular material.

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Investigating the effectiveness of semi-rigid protection fences

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Keywords: low-energy rockfall barriers, FEM modelling, residual rockfall risk

Semi-rigid, low energy rockfall protection fences, can provide in certain circumstances a convenient alternative to flexible falling rock protection barriers, if their installation is planned in an adequate and comprehensive manner. The response of these structures has been recently investigated (de Miranda et al., 2015; Mentani et al. 2016). Results of these studies have shown a dependency of the structure capacity on impact conditions. Realistic impact conditions can be applied to barrier models once they are suitably incorporated in an advanced rockfall simulation tool (Bourrier et al. 2014). Results of the simulations can then provide an effective support to intervention plans. The implementation of barrier models in a rockfall simulation model is a complex task and should rely on a thorough understanding of the fence structural behaviour. In this short note, the numerical response of a simple semi-rigid protection barrier is explored to the scope.

1 FE MODELLING OF A SEMI-RIGID BARRIER

The fence considered in the study is a semi-rigid barrier, made of structural steel posts, connected through longitudinal cables to which a steel hexagonal meshwork is attached. Posts are placed vertically and fully restrained at their base and connected to the ground through side cables which connect the head of the external posts to the ground. This barrier type can be easily encountered along the Alpine arc. The FE modeled barrier is 3.2 m in height and has 15 evenly spaced longitudinal cables of 12 mm diameter. Internal posts are IPE 200 while the external posts are IPE 300. Post spacing is 5 m. The model is three-dimensional and made of one-dimensional elements, whose behaviour is implemented as elasto-plastic. Constitutive parameters are based on available experimental data. In particular, the response of the hexagonal meshwork is calibrated based on tests carried out on net portions (Mentani et. al., 2016). The net may rupture, while the cables undergo indefinite deformation once the yielding threshold is reached.

1.1 IMPACT CONDITIONS

The analyses are run by impacting the fence model with a block of known mass and velocity, with the scope to identify the minimum value of kinetic energy at which the fence is no longer able to arrest the block. The condition is described as failure condition and the corresponding impact kinetic energy is described as failure energy. A scheme of the explored impact conditions is given in Fig. 2. Parameters considered in the analyses are: the block size (Fig. 1 a), the inclination of the block velocity at impact (Fig. 1 b) and the block location at impact (Fig. 1 c). As for the block size, 8 different external lengths were taken into account (0.5 m, 0.6 m, 0.65 m, 0.7 m, 0.75 m 0.8 m, 0.9 m and 1 m). The speed inclination is varied between minus 60° to 60° using a 30° interval The explored impact locations are: central, and, with respect to the central position, 0.5 m upward, 0.5 m downward and 1 m left. When the effects of block inclination is explored, the block dimension is kept constant and equal to 1 m and the impact is central. The influence of block position is investigated using the test block of 1m length impacting the fence with no velocity inclination.

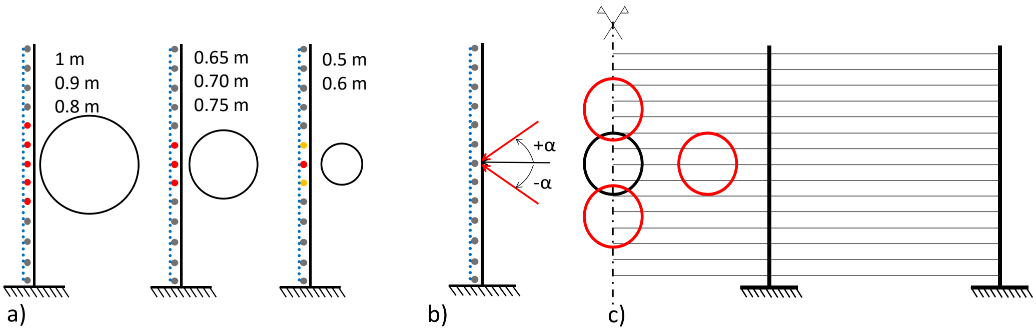

Figure 1: Parametric study of a low energy protection barrier: a) block size, b) block inclination at impact and c) block position at impact

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2 DISCUSSION OF THE FEM RESULTS

The analyses have shown a great variability in the barrier response. Values of failure energy were observed to vary between 80 kJ and 360 kJ. As for the block size, the maximum values of failure energy were associated to the largest blocks (from 0.8 m to 1 m of external length). Within this range, the impact energy remains approximately constant and equal to 250 kJ. Time histories of the velocity for the tests carried out until failure with the 1 m block are given in Fig. 2. A reduction in failure energy is observed as the block size is reduced. Medium size blocks (0.65 m to 0.75 m) produced failure with impact energy about 160 kJ. To bring the fence to failure, smaller blocks (from 0.5 m to 0.6 m) had to impact with failure energy equal to 80 kJ. The different response is related to a different failure mode, triggered by the different number of cables involved in the impact, as it is shown in Fig. 1a. The relation between block inclination and failure energy showed to be linear, with the minimum value reached at an inclination of minus 60° (about 170 kJ) and a maximum value achieved at 60° (about 360 kJ). This trend can be explained with a different failure mode observed by varying the parameter. As depicted in Fig. 3, if the block is moving upward (negative angles, Fig. 3a) the block overcome the net by rolling out from the top, causing non rupture in structural elements, while a downward movement of the block (positive angles, Fig. 3b) made the fence fail by causing plastic hinges at the internal posts. As for the block impact location, the failure energy was 220 kJ downward and upward along the centreline. A high energy resulted necessary to make the fence fail when moving the block 1 m left to the centreline (300 kJ). In Table 1, the horizontal impact velocity which causes failure is inserted along with the relevant parameters.

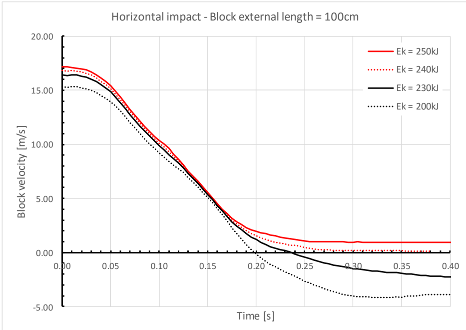

Figure 2: Velocity versus time, central, horizontal impact of 1 m block

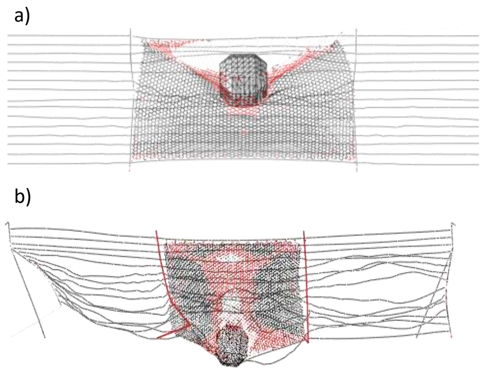

Figure 3: Fence failure mode: block inclination of a) -30° and b) +30°

Table 1: Failure data

mass [kg]	1700	1238	869	718	583	466	366	213	1700	1700	1700	1700	1700	1700
inclination [°]	0	0	0	0	0	0	0	0	-60	-30	30	60	0	0
position [m]	0	0	0	0	0	0	0	0	0	0	0	0	±0.5 centre	1 side
velocity [m/s]	16.8	20.1	24	22.4	24.2	26.2	22.1	27.4	14.4	16.8	17.8	17.8	16	17.8

3 CONCLUDING REMARKS

In this note the response of a low energy, semi-rigid rockfall protection barrier is explored numerically. The analyses show a large dependence of the barrier response to the impact conditions. The block size, the velocity direction and the block position induce different modes of failure of the fence, which in turn result in different values of failure energy. As a result, the barrier capacity cannot be established in a deterministic way. The behaviour is typical of this structure type, whose presence and use is wide within the Alpine space. The effectiveness of structures as such can be more successfully evaluated through a reliability probabilistic approach. Results can be used to create a meta-model of the barrier response which can be incorporated into rockfall simulation models.

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Flexible slope stabilization systems: the experience in the use of high performance membranes reinforced with horizontal cables.

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Keywords: flexible membrane, slope stabilization, reinforced system, TUTOR®

The theoretical development on flexible slope stabilization systems and the experience acquired from their use, particularly in important jobsites with great success, made that reinforced systems with horizontal cables are well known and widespread in Spain, Andorra and Portugal. Based on the experience and an important theoretical work, it has been demonstrated that reinforced systems are the most efficient way to get high unitary supports for slope stabilization with flexible membranes.

1 BACKGROUND

The use of flexible membranes for slope stabilization was introduced in Switzerland in the early 80's of last century with the first wire rope net panels (isotropic membranes). But it was in Spain, in the mid 90's, where these solutions were improved based on a theoretical research that gave them a "system" concept. As a result, physical models of behaviour were developed and validated in full-scale test, and a rational calculation basis which allowed dimensioning was given. Moreover, in Spain at the beginning of this century, high performance steel wire meshes were started to be used as anisotropic membranes: high tensile strength and low deformation steel wire meshes, as TUTOR® Mesh. These flexible membranes are usually reinforced with horizontal cables to get flexible slope stabilization systems according the unidirectional physical model. These kinds of systems can reach unitary support up to 40 kN/m^2. Anyway, slope stabilization systems installed under conditions based on non-efficient physical models in which the membrane is used in non-sense way have been promoted all around the world. This is the case of the membranes directly connected to the anchors by a punctual connection under the physical model. For this model, the punching strength directly over the mesh with the used plate is the mechanical parameter that controls the capacity of the system: as a result lower unitary support systems are obtained, lower than 10 kN/m^2 with the need of a close anchor grid and a reduction of the cost efficiency of the whole system.

2 BRIEF INTRODUCTION TO FLEXIBLE SLOPE STABILIZATION SYSTEMS

A flexible system is a geotechnical application used for slope stabilization purpose. It is composed by a solution of ground anchors combined with a flexible membrane, which prevents the mobilization of the unstable mass inside the anchor grid. The flexible membrane shall offer an adequate strength for the specific working conditions in which is going to be installed, regardless on the preferred directions of working. Moreover, the flexible membrane must present a load – deformation behaviour controlled and limited according the use as a flexible membrane inside a stabilization system. The membrane should present continuity in its whole surface, especially in the unions between the different panels as well as a uniform and homogeneous properties and behaviour. Flexible membrane supports the efforts generated by the ground and has to transfer them towards the anchor heads, and from there to the stable area of the slope through the anchor bar. The membrane must be connected to the anchor heads and this connection can be performed by two different ways: through plates directly installed over the membrane (direct or punctual connection) or through reinforced cables to transmit the efforts generated along the membrane to the anchor heads (indirect or reinforced connection). This way of connection should constraint the physical model of the system. Moreover, during its installation, the flexible membrane must be disposed over the slope with its entire perimeter braced and connected to the anchors in the perimeter lines. This installation procedure, combined with the irregularities of the surface and the ground pressure provokes a curvature and a small displacement of the membrane to the outwards. As a result, the tensile efforts generated inside the membrane in addition to its curvature produce a uniform stabilizing pressure over the ground surface. These requirements invalidate the employment as flexible membranes inside slope stabilization systems of certain kinds of solutions which show excessive deformation in working conditions (for example double twisted meshes or ring nets), neither other solutions which are based on the employment of heterogeneous meshes which present a lack of continuity in the mechanical properties and behaviour in the totality of its surface, without homogeneous and uniform properties.

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3 PHYSICAL MODEL AND MATHEMATICAL RESOLUTION

Depending on the physical model of behaviour assumed for a flexible membrane, specific mechanical parameters must be determined in order to know the unitary support provided by the membrane inside a system. Based on an extensive theoretical work, physical models that explain the flexible membranes behaviour inside a slope stabilization system were obtained and its mathematical resolution was validated by conducting full-scale tests. The mathematical resolution of the physical model using the mechanical parameters of the flexible membranes allows obtaining for a particular membrane the unitary support of the system and the displacement values when it is installed in a system to stabilize a slope. These parameters are unique and representative of the system, for a particular membrane and for a particular installation procedure and border conditions according to the physical model considered. So for anisotropic membranes, the physical model is different depending on the type of inner connection between the membrane and the ground anchors.

3.1 PUNCTUAL MODEL: DIRECT CONNECTION BETWEEN MEMBRANE AND ANCHORS

The unitary support of the system depends on the anchors density and the direct punching strength of the membrane transmitted by the plates used to connect bars and mesh. Flexible systems installed according to this model offer low unitary support, as the punching strength of the membrane is limited. To obtain higher unitary support for flexible systems the anchor density should be increased as well as the final cost. For this reason, in order to get higher unitary support, the unidirectional model is recommended.

Figure 1: Punctual configuration

3.2 UNIDIRECTIONAL MODEL: INDIRECT CONNECTION

The membrane is continuously reinforced through horizontal cables (as stiff elements inside the model), connected in a continuous way to the mesh and disposed in the system along the anchor lines. When the ground pushes over the membrane, it experiences a deformation similar to a cylindrical sector. This deformation provokes internal forces in the main direction of the mesh (disposed on the vertical direction of the slope). These forces in the membrane are transmitted to the anchor heads through the reinforcement lines connected to the membrane. The unitary support of the system depends on the tensile strength of the membrane. Flexible systems installed according to the unidirectional model are more efficient than systems installed according to the punctual model: higher unitary supports are achieved with a wider and more rational disposition of the anchors.

Figure 2: Unidirectional configuration - Reinforced system

4 CONCLUSION

Although the concept of reinforced systems was introduced in the slope stabilization field, this technological advance have been hidden for years due to trade policies conditioned by large multinationals. So outside the Iberian Peninsula countries, less efficient solutions have been widespread. It is necessary to consider geotechnical models of interaction membrane–terrain that represent the natural phenomena as realistically as possible. Moreover, the mechanical parameters that characterized the membrane should be obtained through rational test laboratory procedures that represent its real mechanical behaviour when is used inside a slope stabilization system and same border conditions and controlling the deformation of the system. The reinforced system is the most efficient way to use the high performance membranes for slope stabilization. From the experience getting from different jobsites where Tutor Reinforced systems have been installed, we can assure and conclude that these systems are highly efficient and reliable.

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Rapid Response to Post Fire Debris Flow Event

Laura Y. COIFFE[1], William F. KANE[2]

Keywords: rockfall, debris flow, flexible barrier, protective measures

Post-wildfire debris flow events are common in the southwestern United States of America. Typically, thousands of hectares of forested land are burned of their vegetative cover and may then be followed by monsoonal rains or intense winter storms. Large debris flows follow carrying boulders, trees, and other materials are very hazardous. In the past, rigid barriers or large detention basins were constructed. In recent years, rockfall barrier technology has been modified to develop mitigation that can be rapidly and relatively inexpensively deployed with little environmental impact. A small Native American community in New Mexico had its water supply reservoir severely impacted by post-fire debris flows. Three flexible debris flow barriers were quickly installed to lessen the severity of the damage.

1 FLEXIBLE DEBRIS FLOW BARRIER DEVELOPMENT AND IMPLEMENTATION

1.1 RESEARCH IN SWITZERLAND

Geobrugg Protection Systems began as part of a wire-rope manufacturing firm, Fatzer A.G., of Romanshorn, Switzerland. Early on, Brugg, as it was called then, began fabricating nets made from wire rope to use as snow nets for avalanche protection in the Swiss Alps. During spring net maintenance, the nets were often full of rock from rockfall. The connection was made and Brugg began manufacturing barriers made of wire rope nets for the purpose of rockfall protection.

In 1989, Brugg opened its first North American factory in Santa Fe, New Mexico to manufacture wire rope net rockfall barriers. In the early 1990s, the California Department of Transportation (Caltrans) began using the rockfall barriers with a high degree of success. Caltrans also experienced a number of debris flow events that were unintendedly stopped by the rockfall barriers. About the same time, ring net barriers, which were much stronger than wire rope nets and could absorb more energy, began to replace the rope nets in rockfall barriers.

In the winter of 2005, devastating floods and debris flows impacted Switzerland. As a result, Brugg, now Geobrugg, and the Swiss Federal Institute for Forestry, Snow and Landscape Research (WSL) embarked on a multi-year, several million Euro program to develop, test, and install debris flow barriers. These barriers were to be engineered according to the dynamics of debris flow.

As a result of this research, Geobrugg developed two systems of engineered debris flow barriers. These barriers are designed to fit within stream flow channels, or chutes. They are engineered to absorb the initial dynamic impact forces and the subsequent static loads imposed on the barriers. Their flexible design allows for much of the impact energy to be absorbed in deformation of the flexible net and brake rings.

The two barrier types are referred to as VX and UX barriers. VX barriers are intended for use in relatively narrow V-shaped chutes, up to about 15-m wide. They consist of wire rope anchors between which are suspended wire rope support ropes with braking elements. A high-strength steel ring net is hung on the top, middle, and bottom support ropes. Like rockfall barriers, the system is designed to flex outward downstream on impact to absorb the dynamic energy of the debris flow. However, the barriers also have to be designed to withstand a static load after impact, much like a retaining wall. A UX barrier is similar in construction except that it includes two support posts. It is designed for channels too wide for the VX barriers. The purpose of the posts is to maintain the height of the barrier. They are not intended to supply any additional structural functions. The barriers are engineered so that all loads are dissipated through the net, support ropes, brakes, and anchors.

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1.2 NEW MEXICO WILDFIRES

In 2011, Santa Fe National Forest, New Mexico was impacted by the Pacheco Fire which scorched around 50% of the Nambe Reservoir, FIG1. As a result of the fire, the area's vegetative cover and soil characteristics were massively modified. These modifications, as a consequence, caused debris flows in the Rio Nambe which feeds the Nambe Reservoir in the Nambe Reservation.

FIG1: Site and Pacheco Fire Location

1.3 BARRIER DESIGN

With concern from the community, the Government of Nambe Pueblo assembled a team of geotechnical professionals to develop a mitigation design to minimize the impact on wildlife and of water pollution.

After an initial field investigation and estimation of debris quantities, locations, barrier sizes and pressure capacities were determined. Anchor depths were determined based on loads and construction plans and specifications were prepared.

2 CONSTRUCTION

After completing the initial site investigation, analysis and design, the mitigation construction proceeded and was completed in 18 working days.

FIG3 shows a similar barrier constructed at nearby Santa Clara Pueblo entirely filled after a recent debris flow event.

FIG2: Debris Flow Barrier in Nambe Pueblo

Barrier Location	Top Width (m)	Height (m)	Storage Estimates (m^3)	Debris Flow Protection System
Site 1	13,71	4,42	2662	VX160-H6
Site 2	12,19	2,29	312	VX100-H6
Site 3	19.05	2.90	615	VX160-H6

3 CONCLUSION

Good communication and team work from all parties dedicated to protecting the wild life and the community of the Nambe Reservation were the reason of the project success.

FIG3: Filled Debris Flow Barrier at a neighbouring Pueblo.

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Session 3B

Case study (2)

Monitoring of a rockslide (2003-2015) until failure (Gorges de l'Arly, Savoie, France)

Lionel LORIER[1], Anne LESCURIER[2], Alexandre MATHY[1]

Keywords: survey, monitoring, rockfall, failure prediction

The Provincial Road RD1212 is an important link between the departments of Savoie and Haute-Savoie and, more generally, is giving access to the ski resorts of Val d'Arly, the Chamonix valley and the Mont Blanc tunnel.

1 PRESENTATION OF THE SITE

The itinerary follows on 15 km narrow winding gorges carved by the Arly river. Locally, in the Cliets sector, the road passes through a tunnel of 60 m long, with portals dominated by impressive cliffs consisting of steep rocky slopes on a nearly vertical height of 120 m.

From a geological point of view, these cliffs are composed of highly fractured micaschists ("Série Satinée de Belledonne"), consisting of hard beds alternating with more deformable soft levels. The entire slope of Les Cliets is affected by a large-scale toppling movement.

This area is known to present many rock falls and major landslides, volumes between 1000 m^3 (rockfall, 1996) and 1500 m^3 (rockfall, 2003), which have historically cut off the road for several months. The site is the subject of a particular geological monitoring since 2003, provided by SAGE (Société Alpine de Géotechnique) on behalf of the Conseil Départemental de la Savoie (CD73) road management service, since the second phase of decentralization in 2006.

2 MONITORING AND SURVEY

Regular monitoring consists essentially of geodetic measurements, based on 9 targets spread over the site and recorded manually with a monthly frequency (see location on Figure 1).

Right from the start of this first follow-up after the 2003 landslide, a number of targets have clearly demonstrated site activity with periods of movement during the winter months alternating with quieter periods. However, these movements (some centimeters for each period) did not show acceleration. It is only after 2010 that the quieter times (corresponding to the summer months) gradually disappeared and that the overall trend shows a clear acceleration of movements with notable changes in behavior of certain targets, like targets 6 and 7 that show movements of 25-45 cm over an annual period between 2011 and 2012 (see Figure 2).

After a detailed geological analysis (structural study and combined geophysical measurements), it appeared that a volume of materials of about 7500-8500 m^3 was potentially unstable, with highly fractured rock on a total thickness up to 9 m. Given the movements affecting the site for about 15 years, the present materials are like a big pile of disorganized large blocks and screes with many hollows, some sectors having moved down more than 2 m cumulatively.

During the period 2013, annual movements measured on targets 6 and 7 are between 75 and 85 cm with a marked acceleration after summer. In October 2013, a first minor landslide forced the CD73 to close the road to traffic until December 18th, time to purge and repair protection devices. During the works, facing the accelerating movements still monitored, the CD73 decided to implement a continuous monitoring of the site so as to anticipate an acceleration of the phenomenon that could lead the disorganized rocky mass to failure and, as well, to close the road wisely to avoid exposing users.

This enhanced monitoring, designed by SAGE in partnership with the company Dynaopt, was on duty in December 2013:

- 14 new topographic target lifted by an automated theodolite installed in the upper part of the site (the old targets were kept);
- 5 wire strain sensors installed on the main fractures on long (10 to 15 m) and short bases (a few meters);
- 1 data acquisition system coupled with traffic lights leading to the closure of the road when a speed or acceleration threshold is obtained. An alert protocol was also developed allowing the manager of the road to react quickly in the case of a sudden acceleration.

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Figure 1 : Picture of the site and localization of the targets

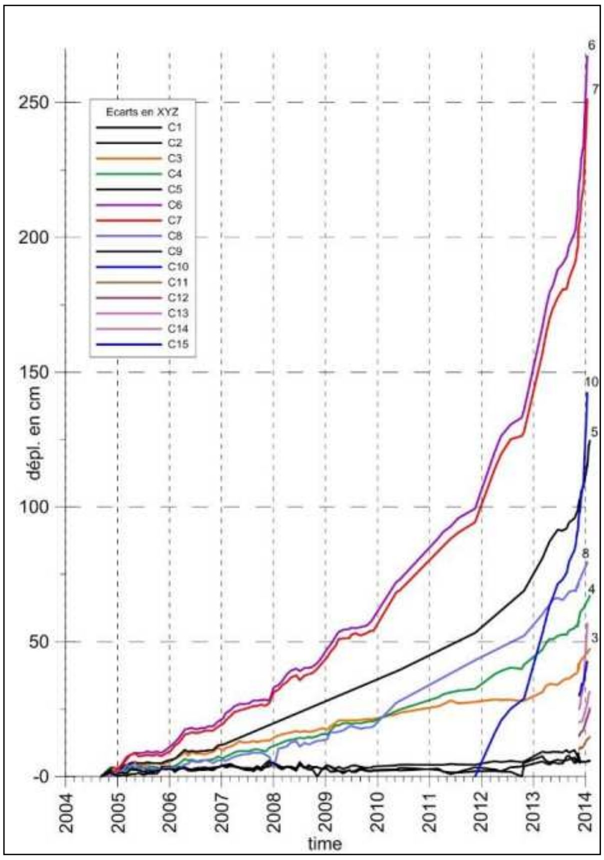

Figure 2 : Evolution of recorded movements (2004-2014)

3 THE ROCKSLIDE OF JANUARY 2014

Right after the start of monitoring, the movements accelerated with values order of 20 to 25 cm/month during the month of December 2013 (for targets 6 and 7) suggesting that a large landslide could occur soon in early 2014.

In consultation with the services of CD73 and local elected officials, the road was closed to traffic on January 13th 2014, in full winter touristic season, nine days before the major landslide that occurred during the night of 22th to 23th January 2014. The automatic monitoring which operated until failure reached an average speed of about 15-20 cm/day on January 21th and of 1 m/day a few hours before the slide. The rockslide was preceded by numerous rock falls and major disorders in the catchment (opening of fractures, subsidence of rocky compartments). The total volume of the slide is close to 10000 m3, the scar forms a large gash in the hillside up to 60 m high; its limits are related to the existence of large geological accidents. The fallen materials form a large scree cone which completely covered the roadway and the tunnel head by several tens of meters.

4 CONCLUSION

Thanks to the topographic monitoring and to the interpretation of the acceleration of motion (to be developed in the presentation), a prediction of a rock breaking of 10000 m3 was satisfactory. After the slide, the sector's safety work started quickly and led to recirculate the RD1212 on July 17th 2014, 6 months after the landslide, for a total amount of 1 million Euros.

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Rock fall mitigation strategy over Korbous village (Tunisia)

Fabien BINET[1], Kamel ZAGHOUANI (others)[2]

Keywords: Rock fall, Tunisia

Korbous village, in Nabeul province of Tunisia is located in a narrow valley, between the see shore and the foothill with steep slope and cliff. The village and its access have always been subject to rock fall hazards. The touristics developpement of the village leads to hazards mitigation works over the village.
Rock fall hazard is still an issue on the village access road.

1 GENERAL SETTING AND SITUATION

Tow cliffs dominate the village. They consist of hard sandstone in the middles of the cliffs and weathered to crisp sandstone at its base. The slope, are made of marls.

2 GENERAL MITIGATION STRATEGY

The treatment of the risk was aiming favour passives parades every time that was possible, completed with actives parades when necessary.
Execution studies focus by order on:
1 Passives parades,
1.1 Dynamic screen,
1.2 Pending mesh,
2 Actives parades,
The type of reinforcement depends on blocks typology and settings.

2.1 PASSIVE PARADES

Wherever the foothill shows enough room between cliff and housing, Dynamic screen have been implanted. The rock fall calculations, indicates that maximum speed of blocks one each profile tends to the same maximum speed, whatever is the weight of the block launch. It is than possible to define a maximum lock weight threshold that could be catch by dynamic screen. It was than possible to limit the number of blocks to reinforce with in site anchoring.

The local topography makes difficult the implantation off the screen, because of the rectangular pattern of the screen that can be change in a trapeze pattern, this leads to implanted 2 overlapping screen.

2.2 ACTIVE PARADES

Active parades have been realized when they were oversizing passive parades downhill.
Due to the geological settings (soft sandstone locally) the shape off blocks leads to a wide range of active parades.

- Anchor in classical situation (wedge)
- Warping net, on isolate or stuck blocks
- Buttress, on overhanging mass.

Figure 1: Example of stuck block stabilization

3 DIVERSITY OF TYPOLOGIES OF BLOCKS

During the works on the cliff over Korbous and during the preliminary visits along the access road to the village, the variety of blocks situation in the slope, let spot some difficulties:

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- Open voids: the injection of anchor face open void along fractures. To avoid loss of mortar, it was necessary to adapt technique injection with thicker mortar and injection socks.
- Isolated mass : blocks isolated from their base have been reinforce with anchor or anchored wire
- Chaotic mass : along tectonic gauge, discontinuous rock mass with blocs, cemented with soils and void
- Erodable base: blocs isolated in marls can be launch in the slope due to erosion of their base,
- Fine morphology : Erosion in cliff can show very fine structure, like arch and long column


Figure 2: Example of particular rock mass to stabilized, Arch and isolated blocs

For each of those morphologies, the choice of reinforcing technique is not obvious, and their design doesn't match with situation currently developed.

4 CONCLUSION

The works on the site off Korbous show a wide variety of block size, shape and settings.
The possible extension of the works will face to a variety of situation that are not common and without any completion off the whole project.

Mont Saint-Michel : Unusual rock slope assessment & mitigation option using CE rockfall barrier

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Keywords: qualitative approach; probabilistic assessment; CE rockfall barrier; touristic site

1 BACKGROUND

The small granitic island of Mont Saint-Michel in Normandy is the 2nd touristic site in France (up to 3 million visitors a year). A recent inspection of the western slope revealed a large unstable rock mass overlooking tourist itinerary (tens of cubic meters 30 m above sea tide level) [1]. Therefore this exceptional classified site required the installation of temporary rockfall barrier with strong specifications.

Figure 1 and 2 : Big issue with hundreds of tourists crossing below unstable rock mass

2 ASSESSMENT

2.1 OBJECTIVES

An emergency study including temporary works has been decided to reduce the risk [5], due to high density of visitors on a limited zone. This unusual issue led to a very sensitive operation, conducted within a contracting background and specific responsibility according to the NF P 94-500 guideline.

Indeed it is assumed that pedestrians represent a “very important” vulnerability (Table 1). Then the design of mitigation options should be “balanced” regarding the “resulting risk”, itself defined using both qualitative and quantitative methods for the “resulting hazard” step (failure × propagation).

Table 1: Semi-qualitative approach for resulting risk assesment

Resulting risk		Resulting hazard (failure × propagation).				
		Very high	High	Medium	Low	Very low
Vulnerability	Very important	Very High	Very High	High	Medium	Low
	Important	High	High	Medium	Medium	Low
	Moderated	High	Medium	Medium	Low	Very Low
	Limited	Medium	Medium	Low	Very Low	-

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2.2 DISCUSSION: QUALITATIVE VS. QUANTITATIVE METHODS

Practically, qualitative methods with an expertise approach are used for estimating the failure hazard, the occurrence timescale, and then the vulnerability of pedestrians [4].
On the opposite, quantitative methods have been used including probabilistic tools for estimating rock volumes (photogrammetry) and propagation hazard (ballistic trajectography modelling).

For the final client, the choice of mitigation options has to be made depending on the "resulting risk" obtained for each size of instability. Large rock mass would be stabilised but it is especially sensitive for smaller boulders leading to lower risk, especially regarding the exposure of hundreds of tourists.
Despite the high "qualitative" vulnerability of people on this site, it could be expected to use probabilistic methods to estimate the vulnerability of pedestrians for smaller boulders, such as shown by recent applied research works [4]. This would allow optimising the design and budget options. However these options have been decided here regarding other requirements : historical walls, landscape integration and archeologic impacts do not allow conventional protection such as anchored mesh.

3 MITIGATION STRATEGY

3.1 DESIGN OPTIONS

Two steps are expected at this stage:

1. Installation of temporary rockfall fences for short term protection
2. Active retaining works on rock mass limiting failure hazard

3.2 STRONG REQUIREMENTS AND FIELD ADAPTATION FOR ROCKFALL BARRIER

Because of this exceptional classified site, the installation of a rockfall barrier should fulfil strong requirements:

1. Adaptation to the field on an irregular slope (no earthmoving due to classified site)
2. Limited foundations (no concrete, optimized anchors due to archeology)
3. Easy installation with limited helicopter (weight) and low impact after removing the fence (anchors).

However, such a rockfall fence should respect CE / ETAG 027 as well (CEREMA recommendation) [2]. Only a few kinds of barriers could then be used such as ELITE® 5000 kJ.

Figure 3 and 4 : Rockfall fence installed on a difficult slope and its integration with limited impacts

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Reinforcement of the face cutting of the tunnel of Djebel El-Kantour (highway east-west, Algeria)

Salah MESSAST[1], Mustapha HAZZAM[2] and Nawel BOUSBIA[1] et Hichem Mokhbi[3]

Keywords: Reinforcement, Soil, Homogenization, Tunnel, Displacement

Abstract: The extension of communication ways (roads, highways, railways) often involves difficult crossings that usually lead to the construction of important structures such as tunnels. Similarly congestion on ground surface in cities necessitates the construction of underground structures (roads and metros). Terrain stability during construction especially in the coalface is one of the major problems related to the construction of these structures. This paper presents numerical modeling of the mechanical behavior of the rock face of the DJebel El-Kantour tunnel (East West Highway) reinforced by fits. In order to minimize on maximum the cost of numerical simulations, the reliability of three methods of homogenization of reinforced soil (improving cohesion, improving the cohesion and angle of friction and ultimately improve the Young's modulus) is tested by comparing the results of numerical simulations to the displacements measured insitu.

1 INTRODUCTION

In this study we propose to reinforce the cutting face by Fits (Mechanical properties of fiberglass tubes. This technique is to use tubular inclusions "GRFP" high length-based polymer reinforced with glass fibers sealed in the ground by an injection system using a cement slurry in order to stabilize the sections of the gauge cap and Stross and oppose deformations and stresses generated by the movement of the ground in different directions:

2 NUMERICAL SIMULATIONS

The reinforcement of the ground core through tubes can be schematized in a mathematical model of two different ways.
A first possibility is to take to the field reinforced equivalent mechanical characteristics. In this case four main methods are included:

- increasing the elastic modulus of the advancing core (Lunardi et al, 1992, Lunardi, 2000).
- increase soil cohesion in the core (Korbin & Brekke, 1976).
- increasing the friction angle and cohesion of the core (Indraratna & Kaiser, 1990).
- applying a pressure cutting face (Peila, 1994).

The other possibility consists in producing a mixed mesh with three-dimensional elements to the ground and the one-dimensional elements for the tubes.
In this study, the first three methods of homogenization will be tested in order to minimize the cost of numerical simulations. Figure 1. show the geometrical model and the figure 2 illustrate the variation of the vertical displacements according to the PK for 9m of the excavation the longitudinal direction.

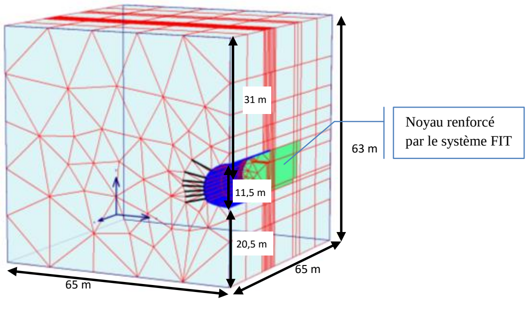

Figure 1: Geometric Model of the structure

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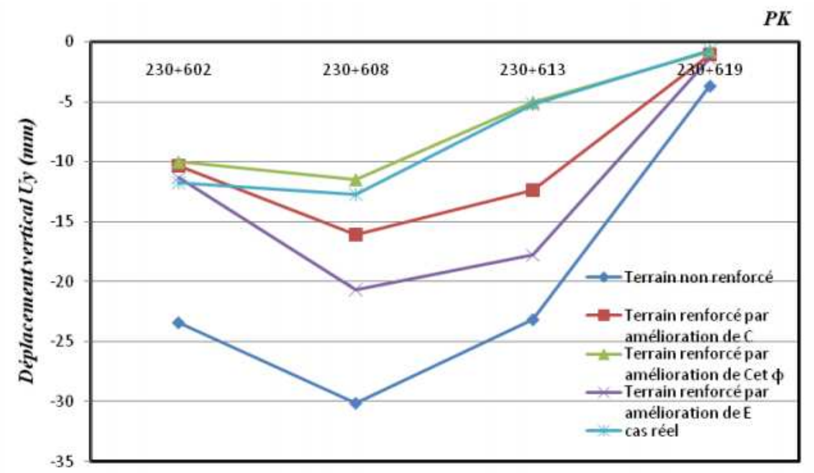

Figure 2: Vertical displacement Uy (mm) according PK (case of 9m of excavation).

3 CONCLUSION

The reinforcement of the tunnel cutting face is an important factor in the safety of workers and the preservation of the digging equipment against possible landslides from the face.

In this article, tunnel size of the front of Jebel El-Kantour, part of the highway is west, is reinforced with GFRP tubes. To better understand the behavior of reinforced areas, a calculation using the finite element calculation code Plaxis 3D Tunnel has helped to highlight the adaptation of the homogenization of the working face of the land by improving the friction angle and cohesion. This technique can significantly save time numerical simulations.

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Paillon bank reinforcement- Nice- A case study of extensive investigations and observational method application

André SQUILLACI[1]**,** Tristan BOURDIN[2], Damien PORRE[3], Brigitte MAHIEU[4]

Keywords : emergency, landslide, reinforcement, auscultations, observational method

1 INTRODUCTION

The site is located in the Trinité City, at the Est of Nice (France). It borders the A8 highway, and consists in an anthropic excavation slope composed of soft Cenomanian marl. From 16th until 19th of January 2014, a very intense Mediterranean depression occurred and brought a total of 270 mm rainfalls. These exceptional precipitations generated a 2000 m^3 rocky landslide, which required emergency excavations works.

Figure 1: View of the cliff

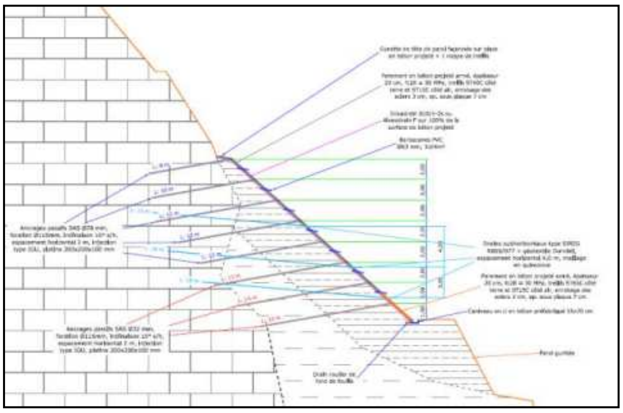

Figure 2: Scheme of the projected works

2 PROJECT STUDY

The bank consists of soft altered marls topped with benches of fractured and altered marly limestones. These materials present a strong water sensibility, thus the behavior can be either raid or soil like. The eastern part bank presents a heavily faulted corridor. The various investigations and observations indicate the presence of heterogeneous ground flows within the bank.

Due to the topographic configuration, the study required heavy geotechnical investigations most of them with the help of helicopter. The first investigations campaign led for design studies contained four borehole with pressuremeter tests led between 10 and 20 m of depth (2 vertical boreholes with one equipped with piezometer and 2 subhorizontal boreholes) and two core samples led to 15 m of depth. Laboratory tests (soil identifications, CU+U triaxal tests) were also realized on samples taken from core drillings.

These investigations highlighted a complex geological/geotechnical model presenting important side variations in term of layers positions and geomechanical characteristics of grounds. They also highlighted the presence of groundwater flows, more important in a faulted zone. This accident seems to play a drain role, guiding and concentrating groundwater flows at the level of the slide zone.

Eventually after an extensive study and discussion with ESCOTA and its assistant CEREMA, the reinforcement solution by a soil nailing wall has been chosen.

3 EXECUTION STAGE

The geological/geotechnical and hydrogeological model of the site, deducted from the first investigations campaign presented uncertainties, in particular the groundwater flows and interstitial pressure led by heavy rainfalls.

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Before the starting up of the works, an important additional geotechnical investigation campaign with devices implementation was carried out. These investigations, realized in topographical difficult conditions, included the following monitoring devices:

- 15 nails extraction tests ;
- boreholes with pressuremeter tests (vertical and sub horizontal) to a 20 m depth ;
- core samples (20 to 25 m depth). They were implemented with 2 inclinometers and 4 interstitial pressure cells ;
- ⇨ 12 topographic targets.

These investigations allowed to specify and to raise a large number of uncertainties off the initial model. The stability modelling achieved during the execution phase showed, as in the design study, that the worst combination corresponded to the accidental water phase (like the observed phenomenon). In order to confirm or counter theses hypothesis, observational method was applied.

Figure 2: Works under progress

4 APPLICATION OF THE OBSERVATIONAL METHOD

The observational method application is based on the analysis of weekly statements of the monitoring devices constituted by inclinometers, interstitial pressure cells (CPI) and topographic targets.
This wall was the object of an interactive design (different water table hypothesis have been taken into account). Additional nails can be added if the resulting pressures monitored overpass the hypothesis criteria.
Besides the CPI statements and nails drilling logs allowed adapting the drains network and their characteristics.

5 CONCLUSION

Works have been in progress since June 2015. The observational method coupled to extensive investigations enabled to optimize the works costs by specifying the geological, geotechnical and hydraulic site conditions. GEOLITHE has been carrying out the works management for ESCOTA company.

Session 4A

Rockfall analysis (1)

Predicting the behavior of landslide in a schist area

H. ZERARKA[1], M. AKCHICHE[2]2, F. PRUNIER[3]

Keywords :Landslide . Second order work, Finite element, Schist, Inclinometers.

1 INTRODUCTION

The landslide of Ain El Hammam (AEH) is characterized by a complex geology and a high hydrogeology hazard. The several reactivations compel us to look closely at its triggers in order to better understand the mechanisms of its evolution in mass. In this study we built a finite element model with Plaxis software. For a finer analysis of instabilities, we use Hill's criterion (Hill 1958), based on the sign of the second order work. This criterion allows detection of unstable areas that can be reached before the limit of plasticity of Mohr-Coulomb (Louafa et al.2010), (Prunier et al. 2009). To validate the numerical model, an analysis of inclinometer measures is performed. The direction of movements and kinematics of the sliding mechanism have been assessed.

2 STUDY AREA

The city of Ain El Hammam (AEH) is located 50 km east from the capital of the city of Tizi Ouzou (Algeria) (Figure1).The landslide under study is the reactivation of an old one. Its geology is essentially characterized by dark gray satin schist belonging to the metamorphic crystallophyllian base of the massif of the Great Kabylia. Satiny schists have a mean direction of schistosity oriented ENE-WSW with a depth that varies from 40 to 60 ° to the southeast (ANTEA 2011).

The schists of AEH are more or less altered. The alteration is facilitated by the fracking of the rock in its upper part, by the presence of flowing water, as well as by physical and chemical mechanisms of desegregation in joints. The alteration product is a reddish clay silt that contains fragments of schist and makes up the layer of the embankment. The rate of schist alteration depends on the depth of the layer, on the exposure to climate hazards, and on the circulation of groundwater. The permeability of the schist formation is a priori in one direction of the cleavage.

Figure .1 Geographical location of the landslide of Ain El Hammam area

3 MODELING

We present the numerical model of a cutting in the middle of the unstable area, with construction at actual state (Figure 2).
From a geotechnical survey realized in the site we can discern the following layers:

- a thick clay embankment with schist debris (1 to 5m deep),
- a destructed schist layer, blend with clay and silt(5 to 10 m deep),
- an altered schist layer with the passage of clay layersfrom 10 to 50m deep,
- a base substratum of satin shale rock. Its cleavage has a dip of 40 to 60 ° pointing in the direction of the slope.

3.1 NUMERICAL MODEL

The profile of the Ain El Hammam slope is modeled with the morphology and current exploitation loads as well as the climatic loads. The computation is carried out under plane strains assumption. A hydro-mechanical coupled formulation is adopted. The non-saturation of the layers above the water table is described using the Van Genuchten model (1980) to represent the water retention curve. The Van-Genuchten parameters are determined using correlations with the particle size and the permeability of the soils.

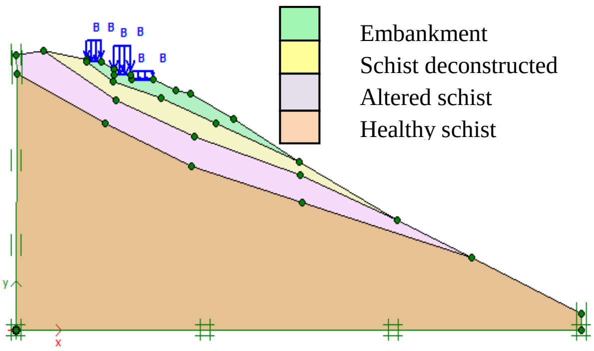

Figure .2 Cut C-C in middle of landslide of AEH (view Plaxismodel).

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3.2 HILL CRITERIONS

Hill (1958) proposed a stability criterion for elasto-plastic materials. After this work, a strain-stress state is said to be unstable if the deformation can be continued in a loading direction without external energy input. As a result, a volume V of a material system is stable, if any pair (dσ, dε) connected by their constitutive model satisfies:

$$\int_V d\boldsymbol{\sigma} \cdot d\boldsymbol{\varepsilon} dV > 0 \qquad (1)$$

at the scale of representative elementary volume (REV), this expression reads:

$$w_2 d\boldsymbol{\sigma} \cdot d\boldsymbol{\varepsilon} > 0 \qquad (2)$$

The results of numerical modeling under the effect of hydro-mechanical conditions, introducing the fluctuation of the groundwater effect and a strong precipitation in the transitional flow regime, enables the study of the evolution of the AEH slope failure. The results in term of the second order work w_2 are presented for the cut C-C (figure 3). These results show a large zone of instability appearing in the embankment and the destructed shale layer were $w_2$is negative.

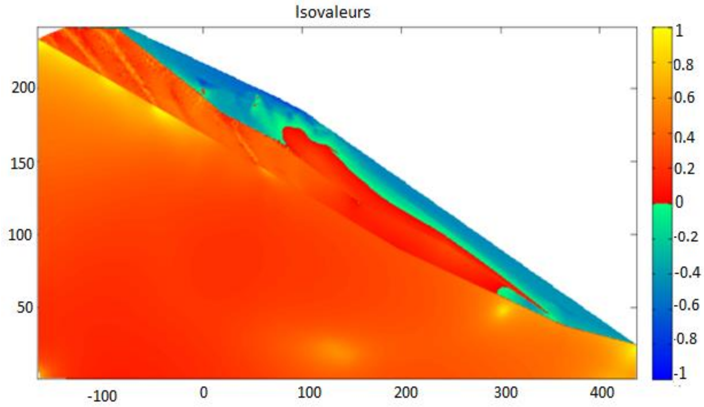

Figure.3Second order work of section C-C

3.3 INCLINOMETERS MEASUREMENT

The inclinometer implanted in section c-c of the area of AEH slide, are used to determine subsurface movement of landslides (Bonnard et al 2008). We recorded ground deformations on a slice of 28 m. These results coincide with the depth of the slip surface numerically predicted (see figures 3). This slip surface is observed along the interface between the healthy schist as well as in the altered and deconstructed schist.

4 CONCLUSION

Based on the numerical analysis and on the in situ measures, we confirm the existence of a new slip surface liable on the manifestation of AEH landslide and independent from the first surface (2008-2009).

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Rockfall hazard in the Mont Blanc massif increased by the current atmospheric warming

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Keywords: Rockfall, Permafrost, Mont Blanc massif

In the last two decades, numerous rockfalls and rock avalanches occurred in permafrost-affected rockwalls worldwide. Due to their steep topography, mountains are indeed affected by significant gravity-related transfers of material, in particular by rockfall in supraglacial areas. Rockfall is the detachment of a rock mass (volume exceeding 100 m^3) from a steep rockwall, according to the sets of discontinuities affecting the rock mass, which travels down slope to a variable distance (Fort *et al.*, 2009). It is one of the most hazardous geomorphological processes because of its high speed and the related risks for infrastructure and population up to valley floors through cascading effects. Three major factors possibly combined can trigger rockfalls in high mountains: seismic activity, glacial debuttressing due to glacier retreat and permafrost degradation *i.e.* the warming of bedrock whose temperature remains at or below 0°C for at least two years. It generates physical changes of the potential interstitial ice and increases the hydraulic permeability of the rock mass (Ravanel and Deline, 2011).
The characterization of the rockfall dynamics is a prerequisite for risk mitigation (Ravanel *et al.*, 2012). However, data on rockfall at high elevation are rare and discontinuous, thus non-representative. In order to investigate the correlation between permafrost and rockfall, we collected and analyzed a dataset of more than 600 rockfalls in the Mont Blanc massif, a 550 km^2, granitic, widely glacierized and permafrost-affected massif, where fracturing and slope steepness make it prone to rockfall.

1 METHOD

A network of rockfall observers in the Mont Blanc massif (Deline *et al.*, 2012; Ravanel and Deline, 2013) was set up in 2005 (Ravanel *et al.*, 2010). It consists in mountaineers sensitized to rock-fall observation. Fully operational since 2007 and regularly reactivated, it focuses on the central part of the massif. Observers fill a paper reporting form or a form on Internet/smartphone (http://www.alp-risk.com/pages/events), indicating the main characteristics of the rockfall and the rockwall conditions. To ensure a higher completeness of the inventory, fieldwork is con-ducted every fall. Elevation of the scars, slope and orientation of affected rockwalls, and area of the deposits were calculated in a GIS. Possible presence of permafrost was determined from a statistical model (Magnin *et al.*, 2015).

2 RESULTS

Since 2007, a network of observers surveys the occurrence of rockfall in the central area of the massif, with volume in the range 100 – 60,000 m^3 (Fig. 1).

3 DISCUSSION

There is a close relationship between the 2007-2015 rockfalls and the air temperature of those years. 50 % of the rockfalls have been precisely dated. Among them, 95 % occurred between June and September, i.e. during the hottest months of the year. It is also striking to note that the cool months that have characterized certain summers have resulted in a delay of rockfall. With 160 rockfalls in the central part of the massif surveyed by the observation network, the rockfall frequency of 2015 has been exceptional, as was the temperature of that summer.

The strong correlation between rockfall occurrence and hottest periods at the time scales of the year strengthens the hypothesis of the relationship between permafrost degradation and rockfall. Moreover, modelling suggests the presence of permafrost in nearly all of the affected rockwalls, and massive ice was observed in many scars. The very high frequency of the Summer 2015 rockfalls can only be explained by permafrost degradation, since no significant seismic activity was measured and only 15 % of the 2015 affected rockwalls were formerly glaciated at their foot (during the Little Ice Age in particular). Several other elements support permafrost degradation as main triggering factor of rockfall. The mean elevation of scars is much higher than the mean rockwall elevation (2880 m a.s.l.), while very few detachments occur below 3000 m a.s.l., which suggests that the main triggering factor is not ubiquitous. The most affected altitudinal belt is 3200-3600 m a.s.l. (55 % of the total number of rockfalls in 24 % of the rockwalls located between 2000 and 4800 m a.s.l.), with warm permafrost. Moreover, the hotter the summer is, the higher the scar elevation. The sharp contrast in scar elevation between north and south faces is

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also consistent with permafrost distribution. Finally, rockfall especially affects topography prone to permafrost degradation such as pillars, spurs and ridges.

Figure 1: The Tour Ronde, August 2015 (Ph. G. Marra)

4 CONCLUSION

Permafrost-induced rockfalls finally differ in space (volume) and time according to three main types of permafrost degradation: (i) vanishing of ice/snow cover generates very superficial rockfalls, due to the formation of an active layer; (ii) several meter thick detachments result from the deepening of the active layer; (iii) several decameter thick detachments may result from the permafrost degradation at depth by the slow rising rock temperature or the quick formation of thaw corridors.

The climatic control on rockfall in high mountain rockwalls, which involves the current permafrost degradation, means that this hazard should significantly grow during the 21st century.

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Influence of meteorological factors on rock fall frequency

Julie D'AMATO[1], Didier HANTZ[1], Antoine GUERIN[2], Michel JABOYEDOFF[2], Jean-Pierre ROSSETTI[1], Laurent BAILLET[1], Armand MARISCAL[3]

Keywords: rockfall, frequency, meteorological factors, freeze-thaw, rainfall

The processes leading to rock slope failures are poorly known but it has been observed that some meteorological phenomena increase the rockfall frequency and consequently the risk for people standing under rock cliffs. This study presents rockfall frequency measurements which have been carried out using annual terrestrial laser scanning and a continuous photographic survey, allowing studying the influence of meteorological factors on rockfall occurrence.

1 THE ROCKFALL DATA BASES

This study focuses on the South-East facing lower cliff of the Mont Saint Eynard (1308 m), consisting of thinly-bedded limestone, because of its high rockfall activity. Rockfalls have been detected using annual terrestrial laser scanning. A photographic survey from 1 km to the cliff allows us to date the occurred rockfalls. It consists of high resolution photographs taken every 2-11 weeks (periodic survey) and lower resolution photographs taken every 10 minutes (continuous survey). 609 rockfalls have been detected between 16/11/2012 and 15/07/2014. Each rockfall has been checked on high-resolution photographs, and dated in 2-11 weeks long periods (Figure 1). They constitute the database 1 (DB1).

Among these rockfalls, 125 have been dated more precisely thanks to the continuous photographic survey. They occurred between 1/2/2013 (installation of photographic survey) and 15/07/2014. They constitute the database 2 (DB2). Around 75% of DB2 rockfalls have been dated in periods of 10 min to 20 h. The volumes of most of the rockfalls for the DB1 are between 0.01 and 0.1 m^3, whereas volumes of most of the rockfalls for DB2 are above 0.1 m^3.

2 METEOROLOGICAL DATA TREATMENT

A rainfall episode begins when the rain begins and continues until the beginning of the first dry period longer than 24 h. So it may include dry periods shorter than 24 h. This value has been chosen from field observations showing that after 24 h, no water seeps on the cliff surface.

The presence and quantity of ice in the rock joints (defining a freeze-thaw episode) can be approached using the freezing potential (FP) (Montagnat et al. 2010), defined by:

$$FP = \int_{t0}^{t} -\theta dt \qquad (1)$$

A freeze episode begins when the temperature θ becomes negative ($t = t_0$), and ends when it becomes positive again. A thaw episode begins when the temperature becomes positive and ends when the temperature becomes negative again (new freeze period) or FP reaches 0 (the ice has melt). A freeze-thaw episode begins when the temperature becomes negative and ends when FP reaches 0 (it includes at least one freeze-thaw cycle).

The influence of ice on rockfall occurrence can be explained by the pressure it exerts in rock discontinuities, either when the ice forms in a confined environment (joints, cracks or pores) or when it dilates during warming episodes (Bost, 2008). In order to investigate these processes, the freeze-thaw episodes have been divided in three types of periods: cooling periods (when negative temperature decreases), warming periods (when negative temperature increases) and thawing periods (when temperature is positive).

3 ROCKFALL FREQUENCIES DURING PERIODS OF THE YEAR

It appears in Figure 1, that the 6 periods with the highest rockfall frequencies are the periods which have the highest freeze-thaw relative duration (freeze-thaw duration / period length). But they don't correspond to the periods with the highest relative rainfall duration. A multivariate analysis shows that 50% of the rockfalls can be explained by freeze-thaw or rainfall. The other 50% induce a "base" rockfall frequency, which is multiplied by influence factors of 7 during freeze-thaw episodes and 4.5 during rainfall episodes (these episodes are different from the observation periods). Thanks to a more precise dating, the DB2 allows to distinguish different conditions during the freeze-thaw or rainfall episodes.

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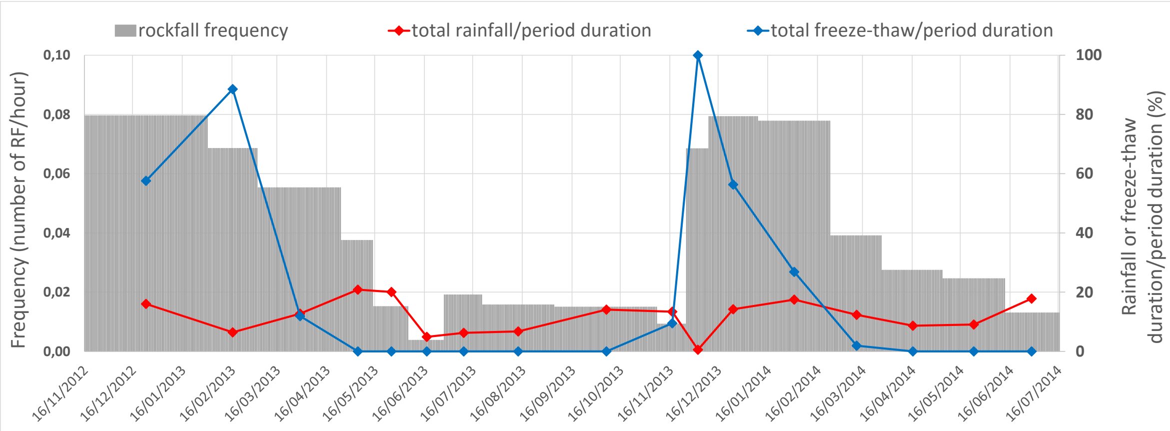

Figure 1: Rockfall frequencies for each period of datation, and freeze-thaw and rainfall relative durations (freeze-thaw or rainfall duration/period duration).

4 DAILY VARIATIONS OF ROCKFALL FREQUENCY

It appears that during the freeze-thaw episodes, the influence factor is not significant during the negative cooling periods and increases mainly during thawing periods (influence factor of about 10). During rainfall episodes, it appears that the base rockfall frequency is multiplied by an influence factor of 7.5 when the rainfall amount is between 30 and 40 mm, and by 30 when the mean rainfall intensity since the beginning of the episode is larger than 5 mm/h. From these results, a 3-level rockfall frequency prediction scale has been proposed (D'Amato et al., 2016): The highest level (influence factor $>$ 16) is reached when the rainfall intensity since the beginning of the rainfall episode is higher than 5 mmh^{-1}; the medium level (influence factor $>$ 4) occurs during negative warming, thawing or when the cumulative rainfall reaches 20 mm; in other cases the frequency level is defined as low (influence factor $<$ 4).

The influence of ice on rockfall occurrence can be explained by the pressure it exerts in rock discontinuities, either when the ice forms in a confined environment (joints, cracks or pores) or when it dilates during warming episodes. But the ice induces a cohesion in rock joints and the falls may occur only when it melts.

5 CONCLUSION

The influence of meteorological factors on rockfall frequency has been quantified for a limestone cliff under an Alpine middle mountain climate. The results obtained need to be precised with more extensive data bases.

Acknowledgements. The authors thank the Région Rhône Alpes and the Federation VOR for fundings.

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Root architecture and growth of candidate plants to stabilize marly embankments of the Fez-Taza motorway (Morocco)

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Keywords: motorway embankments, marly substrate, erosion, bioengineering, root system, biomass

The role of vegetation in the reduction of water erosion has been the subject of several studies. The results show the effectiveness of different vegetation types and affirm the importance of the vegetation cover rate to fight against erosion (Rey et al., 2004; Burylo and Rey, 2009; Lavaine et al., 2011). Plants can act as a protective barrier between the soil and the natural elements that stimulate erosion (Stokes et al., 2008). The architecture of the root system and its biomass determine the quality of the anchorage and the ability of the plant to uptake and transport soil resources (Bécel, 2010). The passage of the Fez-Taza motorway (Morocco) through marly substrates has resulted in embankments particularly sensitive to water erosion. As a matter of fact soon after their construction, these embankments become subjected to various forms of erosion with varying severity depending on rain intensity. Since mechanical stabilization proved to be ineffective or too costly, the authors have decided to use bioengineering based on plants whose root system enables soil consolidation and thus reduces erosion. The purpose of this study is to evaluate the effect of several plant species, planted on the marly embankments of the motorway Fez-Taza, on soil stability. The plants that showed promising results were closely monitored under more controlled conditions to better understand the kinetics of their root development. These plants are: *Arundo donax, Agave americana, Acacia cyanophylla, Spartium junceum, Hedysarum coronarium, Prosopis pubescens, Medicago arborea, Retama monosperma, Ceratonia siliqua, Vetiveria zizanioides* and *Atriplex halimus.* This monitoring was made on a substrate taken from the marly embankments of the Fez-Taza motorway, according to two experimental protocols: mini-rhizotrons and containers with transparent walls to follow growth and architecture of the root system.

1 EXPERIMENTAL PROTOCOLS

1.1 CONTAINERS WITH TRANSPARENT WALLS

This experiment was conducted in a greenhouse in June 2013. Its purpose was to study the quantitative development of the roots of species tested in the marly embankments of the Fez–Taza motorway. The containers were filled with the soil taken off from the marly embankments. The species tested are shown in table 1 with a root depth (initial state) varying from 15 to 20 cm depending on the species.

Plant species	*Leucaena leucocephala*	*Vetiveria zizanioides*	*Ceratonia siliqua*	*Spartium junceum*	*Tamarix aphylla*	*Atriplex halimus*	*Agave americana*	*Retama monosperma*
Initial depth(cm)	20	20	20	15	15	15	15	15

Table 1 : Initial rooting depth of the species tested

1.2 MINI-RHIZOTRONS

This is a non-destructive method for the study of growth dynamics and architecture of the root system through the follow-up of their importance and distribution along the glass sides. Several authors have used this technique (Pagès, 1992; Silva and Beeson, 2011). Mini-rhizotrons used in this study were made of wooden boxes with two glass sides protected from light by a removable black plastic film to ensure the conditions for root development and the possibility of their observation. The glass sides are inclined to allow the roots to grow against the glass thanks to geotropism (Figure1). They have been designed as follows: 30x30x60cm with 7 holes in the bottom for drainage. They were placed outdoors in Taza, to monitor the root system of the plants under the same conditions as those installed on the motorway embankments. The tested plants were: *A. donax, A. americana, A. cyanophylla, S. junceum , H. coronarium, P. pubescens, M.*

Figure 1: Mini-rhizotron

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arborea and *R. monosperma.* It should be pointed that *V. zizanioides* and *L. leucocephala* have perished two months after planting in the mini-rhizotrons. They were replaced by *P. pubescens and M. arborea.*

2 RESULTS

Vetiver is the species that showed the highest root density among the species studied in the containers with transparent walls. Their fresh weight was 288 g with a rooting depth of 80 cm. The other species tested in these containers produced fresh weights varying from 5.07 g for *R. monosperma* to 190.68 g for *L. leucocephala* with rooting depths varying from 32 cm for the agave to 80 cm for *L. leucocephala*.
Root architecture studied in the mini-rhizotrons showed a root system with a vertical growth for most species. Lateral growth was limited to a few root ramifications. (Figure 2 and 3)
Root growth rate calculated during the observation period varied between 0.94 cm/month in the case of agave and 3.33 cm/month for *L. leucocephala* and *V. zizanioides*. In the mini-rhizotrons root growth rate varied between 0.96cm/month for *S. junceum* and 16.2 cm/month for *R. monosperma.*

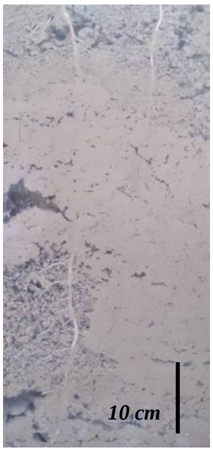

Figure 2: Root growth of Retama monosperma against the glass side

Figure 3: Root growth of Acacia cyanophylla against the glass side

3 CONCLUSION

The measurements and observations from experimental protocols show that the behaviour of the plants studied regarding the marly soil differs from one species to another. The rate of vertical growth reached more than 16 cm/ month for *R. monosperma.* The root of this species had the lowest weight among the studied species in containers with transparent walls. *H. coronariumn* had an estimated root growth rate of 14 cm/month. This can be considered as an interesting asset for the marly soil since the roots of this species were very fine expressing a potential for soil consolidation unlike *A. cyanophylla* that had thicker roots, but a slower growth. To fight against erosion in marly substrate, it is suggested to combine plants with complementary advantages. Therefore, we can suggest the combination of species with a high growth rate to colonize the topsoil, and species with an important root density to consolidate the soil and sustainably fight against erosion.

4 ACKNWOLEDGEMENTS

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Session 4B

Risk analysis

Building a road in a cliff: the case study of the Chambon road

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Keywords: Rock fall protection, mountain road, mining, verticality constraints, safety, risk management, rope access works, rocky outcrops

In April 2015, a massive landslide on the right bank of the Chambon Lake (France) occurred and caused the closure of its sole tunnel. This disaster led to the brutal interruption in traffic between the cities of Grenoble and Briançon and the isolation of local populations, implying strong social and economic impacts. The rehabilitation of the collapsed tunnel being deemed unfeasible in a reasonable delay, local authorities opted for the building of a new road on the left bank of the lake. With two major constraints: an extremely tight schedule for the construction of this 5 km road and a major rocky outcrop to blast, inclined at a 60-degree angle and located in the middle of a landlocked thalweg.

In the face of the emergency, the real challenge was to set up the appropriate strategy so that the road would be delivered in a short 4-month time as requested, without neglecting any aspect regarding the safety of on-site personnel and equipment.

1 ANTICIPATING PREDICTIBLE RISKS AND DEALING WITH UNCERTAINTIES

Works in a natural site - and all the more in a mountain environment, mean that specific procedures be designed. Before works implementation and on a daily basis, close scrutiny was paid to site facilities, scaling of loose rocks and bush clearing, drilling and anchoring, installation of rock fall barriers, concrete grouting and injection, mining with explosives and lifting, hauling and helicopter transportation. New steps sometimes had to be rethought and redesigned to adapt to the procedures and to match the short delay for completion.
But such anticipated procedures were not sufficient in that case. Indeed, unlike other projects where engineering offices most often have time to proceed to all necessary geotechnical surveys and recommendations, the emergency situation for the Chambon road had not allowed to establish a complete diagnosis. Usual risks were assessed and reduced ahead of time, whenever possible. But on-site teams have dealt on a daily basis with a great amount of uncertainties for which adaptation, flexibility and decision-making skills were needed.

Table 1: Chambon road project - Specific procedures

SPECIFIC PROCEDURES IMPLEMENTED
Enabling works to be conducted at night time (specific equipment and lighting)
Building new embankments and ensuring slope stability in extremely narrow zones and thalweg
Manipulating heavy equipment in narrow areas and thalweg
Safely accessing and mining a massive rocky outcrop
Preventing possible personnel or equipment falls into the void
Managing access solutions to the working area (helicopter, foot or boat via the lake)
Establishing alert and evacuation procedures of working areas in case of emergency (collapse of rocks...)
Managing the high-level submersion threat of a 17-meter wave as a result of possible new massive landslide
Managing the risk of causing instabilities and overhanging rocks during mining of outcrop

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2 OPTIMIZING OPERATIONS AND PHASING STAGES

To ensure the delivery of the road on time, it soon appeared inconceivable to work in a linear manner, starting from one side of the road and all the way to the other end. Works would have to be started from both sides at the same time, and their junction would happen after final mining of the rocky outcrop. That would lead to three consequences; first, that both material and personnel resources be significantly increased. Second, that phasing stages overlap with one another. And at last, that working time be expanded as much as possible into longer daily shifts - from 6 am till 9 pm, six days per week.

The first stage consisted in the compulsory installation of temporary protections to prevent any rock fall on future working areas – with single / double twist wired mesh, barriers and nets. That being accomplished, the first mining operations could start to open larger access zones and create paths for heavy equipment. The wide range of operations along the 5-km road could then been carried out in parallel: installation of additional rock fall protections, bush clearing, anchoring, drilling, mining with explosives (approx. 20.000 m^3), earth embankments and slopes stabilization works, civil works including sprayed concrete or nailed walls, enlargement of narrower portions of the road with ACROSOLS® structures, spider digger earthworks, building of crossing devices on water courses and connexion of the new road onto the existing ones.

Figure 1: Phasing stages of the Chambon road project

3 OPTIMIZING DELAYS WITHOUT COMPROMISING QUALITY AND SAFETY

Delays could never have been achieved without a strong commitment, organization and cooperation of all personnel, from the client to the operators in charge of works. Whenever facing new situations, strategic decisions needed to be made with as much responsiveness as responsibility. Such a continuous dialogue between all actors of the project saved a large amount of time. For example, some materials have been traded for other ones with similar features, depending on supply time and availability at supplier warehouse. Projected quantities have been modified to better correspond to the geotechnical reality of the site.
The major challenge was the undetermined daily rate for mining and excavating the rocky outcrop. Methods were reconsidered (drilling mine holes at night time to allow carry out operations as early as possible in the morning) and equipment was adapted (smaller machines than expected were eventually easier to work with in narrow areas), etc.

4 CONCLUSION

The Chambon road can be remembered as one of those few recent projects in France to have condensed such an unusually large scope of rope access operations and public works. The success of this global project mostly resides in HYDROKARST's capacity to coordinate its technicians and all partner companies, redefining stages to meet deadlines and adapting methodology whenever possible.

But beyond that, the remarkable speed of execution of this mountain road as well as good management of all associated risks are to be pointed out. Safety prevention and heightened vigilance by all and at all times were key factors. And with more than 50 operators and technicians simultaneously working on site on various tasks, no accident has been reported.

Indirect Vulnerability Assessment for transport infrastructures: assessment of constraints on roads exposed to rockfalls

Jean-Marc TACNET[1], Eric MERMET[2], Frédéric BERGER[3], Elodie FORESTIER[1], Félix PHILIPPE[1]

Keywords: rockfalls, natural hazards, mountains, indirect vulnerability, roads, network structural properties, constraints

In mountain areas, roads and transport infrastructures are exposed to natural hazards such as rockfalls. Road networks are essential for economic, social, environmental and safety reasons and can therefore be considered as part of the critical networks as well as energy (gas/petrol), electricity, communication and water networks. As they are parts of critical infrastructures, the effects due to linking function losses can be very harmful for territories (cf. Chambon Tunnel, Isère, France – Summer/Fall 2015 ; South East rainfalls - October 2015). At the moment, main approaches focus either on hazard assessment or local risk management considering mainly direct vulnerability. When dealing with territories, indirect techniques, considering indirect vulnerabilities are required.

1 INDIRECT VULNERABILITY AND NETWORK STRUCTURAL PROPERTIES ANALYSIS

Risk depends both on hazard and on direct and indirect vulnerability, which is also more difficult to assess. In case of roads, consequences of natural phenomena can affect either users, road infrastructures or the linking function itself
This last case corresponds to indirect vulnerability and relates to the consequences of road closures inducing loss of linking functions. The criticality and importance of road sections is directly linked to the consequences of road closures (Figure 1).

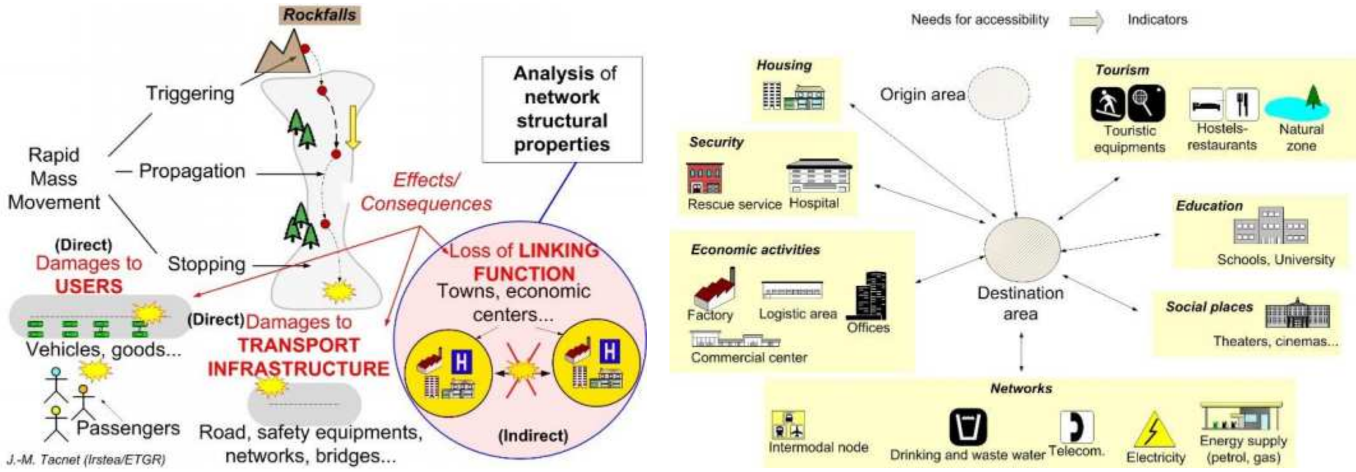

Figure 1: Concept of direct and indirect vulnerability (left) – Types of domains and activities possibly affected by linking functions losses (right)

Structural properties analysis (Gleyze, 2005 ; Mermet, 2011) combines spatial analysis, graph theory and produces indicators to (e.g.) evaluate the importance and criticity of road sections in the context of natural risks management in mountains. The centrality indicator (e.g.) measures the level up to which a road is used to reach any point (importance factor). The average (mean) topological distance indicator shows how easy or difficult it is to reach a point on the network. GeoGraphLab (GGL) is a versatile software which implements this methodology (Mermet, 2011) for the exploration of any infrastructure network (transport, energy, telecommunications). It implies to evaluate the attractivenes factors on nodes, constraints on edges (physical, exposure to natural phenomena) and then calculate the structural indicators such as centrality... (Figure 2). This paper deals with the methodology to assess constraints due to rockfalls on roads using both multicriteria decision-making methods and rule-based approaches.

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2 ASSESSMENT OF CONSTRAINTS ON ROADS DUE TO ROCKFALLS

Rockfall hazard is assessed thanks to local and regional methods based (e.g.) on energy line principles – see Berger et al. in (Tacnet, 2012), multicriteria decision making methods (Figueira et al., 2005) and rule-based approaches. An integrated approach combines constraints on roads due to rockfall phenomena in the structural properties analysis methodology dedicated to indirect vulnerability assessment (Figure 2).

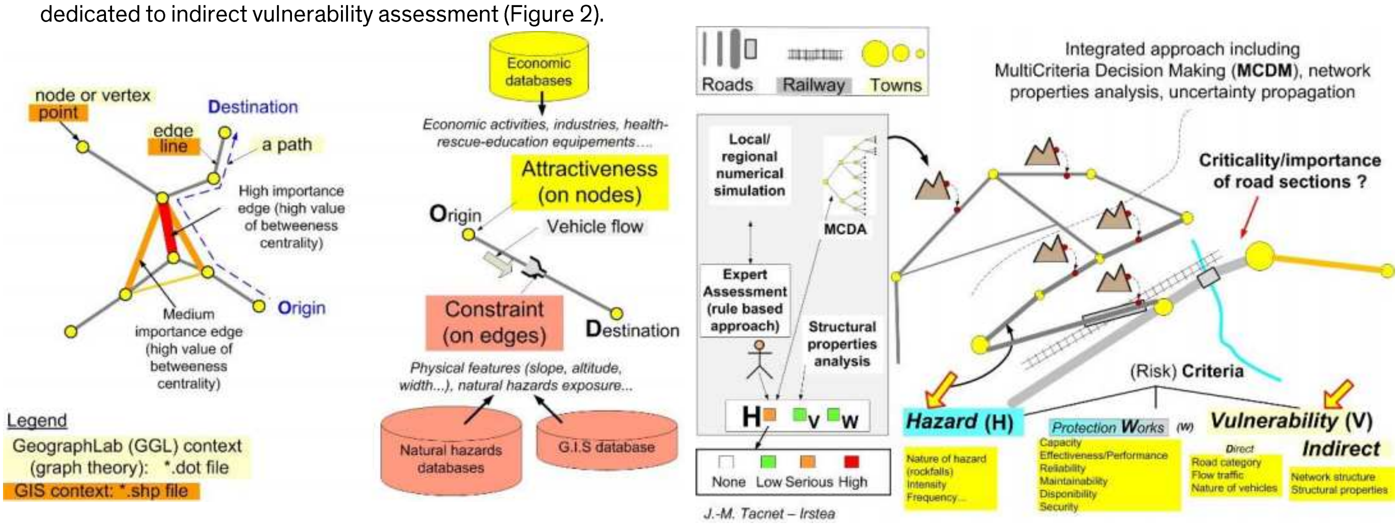

Figure 2: Structural properties analysis principle (left) – Proposed integrated approach (right)

3 CONCLUSION

Constraints on roads due to rockfall phenomena are assessed through a combination of large-area modelling techniques and expert-based multicriteria decision making methods. Those index are combined in an original way with structural properties analysis methodology to assess the importance of road sections but also to compare risk management strategies and take best decisions for investment, maintenance.

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INSTABILITE DES PENTES EN ROCHES EVOLUTIVES: L'EXPERIENCE DU MAROC

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Keywords: pélite, marne, flysch, schiste, érosion, glissement, éboulement, clouage, grillage, gabion

Dans le cas des sols ou des roches non évolutives, on applique les modèles classiques pour pouvoir se prononcer sur leur stabilité et arrêter les confortements nécessaires en cas de besoin.
Ainsi, pour les sols on les caractérise par leur cohésion et par leur angle de frottement et on applique, en général, les critères de Mohr-Coulomb pour faire les calculs de stabilité au glissement.
Pour les massifs rocheux, on les classe selon les classifications usuelles et en particulier celle de l'indice Q (Barton) ou du RMR ; et on applique le critère de Hoek et Brown pour se prononcer sur leur stabilité.
Quand la roche est évolutive, à savoir lorsqu'il y a changement de ses caractéristiques en fonction du temps, soit par aération soit par apport d'eau en général, aucun des deux modèles cités ci-haut ne peut être appliqué ; en effet :

- les essais de cisaillement au laboratoire ne sont pas représentatifs de la réalité pour déterminer C et ϕ ;
- la classification des roches n'est pas applicable à ces roches compte tenu de leur variabilité évolutive.

Parmi les roches évolutives rencontrées au Maroc, on trouve principalement :

- des pélites,
- des marnes,
- des flyschs,
- des schistes .

Photo n° 1 : Pélite évolutive

Ces formations tant qu'elles sont confinées et protégées possèdent de très bonnes caractéristiques mécaniques. D'ailleurs, les essais in-situ, dès qu'on rentre dans les tranches saines de ces formations, fournissent des pressions limites au pressiomètre ou des valeurs de SPT élevées.
Au début, des grands chantiers au Nord du Maroc, on a opté pour des pentes différentes à savoir 2H/1V dans la partie altérée et 3V/2H dans la partie saine (Figure 1).

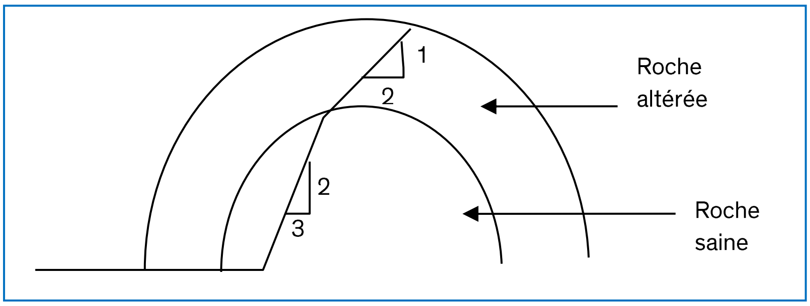

Figure 1 : Pentes initialement retenues pour les déblais au Nord du Maroc

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Mais dès l'ouverture des premiers déblais on a eu systématiquement des glissements même dans la partie saine et ces glissements ne sont stabilisés dans les pélites qu'après avoir atteint une pente de 3H/1V. Actuellement, tous les déblais de pélite au Maroc sont pentés à 3H/1V.
De même, plusieurs ouvrages de soutènement ont été fondés sur des formations saines évolutives et ont fini avec le temps par s'effondrer. En effet, si aucune disposition n'est prévue pour éviter les infiltrations d'eau dans les formations initialement saines, elles s'altèrent avec le temps et finissent par céder.

Photos n° 2 et 3 : Effondrement différé d'un mur de soutènement

Ainsi, dans ces formations, il y a toujours des désordres et des glissements différents. En effet, si l'évolution est rapide tel que dans le cas des pélites, les désordres sont observés en cours de travaux ; par contre si l'évolution est moins rapide, tel que dans le cas des schistes ou des marnes, les désordres n'apparaissent souvent que plusieurs années après les travaux.

Photo n° 4 : Glissement différé dans les pélites

Photo n° 5 : Glissement différé dans les marnes

Confronté avec les roches évolutives, il faut surtout se fier à l'expérience du terrain, en se basant sur des exemples vécus et sur les observations de stabilité naturelle des versants qui donnent de très bonnes indications sur les angles limites de stabilité. La présence d'eau est très pénalisante ; et il y a lieu de prévoir la collecte des eaux de surface et de capter d'une manière générale toutes les circulations d'eau.

Session 5A

Rockfall analysis (2)

Characterization of localized hazard in a marble rock slope using a Remotely Piloted Aircraft System (RPAS) and terrestrial laser scanning

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Keywords: RPAS photogrammetry, Terrestrial laser scanning, 3D point cloud, fracture mapping, slope stability analysis

1 DESCRIPTION OF WORK

Workplace safety in quarries can be evaluated and improved through an accurate mapping of localized hazard. Rock slopes stability is dominantly influenced by the structural setting which, in combination with exploitation methods, affects the risk. This paper describes the application of a rotary wing Remotely Piloted Aircraft System (RPAS) and a terrestrial laser scanner for monitoring the stability of a marble quarry located in the central part of the Apuan Alps Metamorphic Complex, in the municipality of Seravezza (Lucca, Italy). The site, named Piastrone, is u-shaped quarry with the main walls approximately NNE-SSW oriented and an area of about 1.5 ha; elevation ranges from about 1,150 to 1,300 m a.s.l.. The quarry consists of very appreciated ornamental marbles, well known all over the world for their aesthetical and physical-mechanical characteristics. The marbles belong to Mt. Altissimo syncline described at first by Lotti & Zaccagna (1881), Giglia (1967), and more recently by Meccheri et al. (2007) and Lorenzoni et al. (2011). In the open pit it is possible to appreciate good examples of the Altissimo White Statuario marble: a white coarse grained marble, with thin and regular layers not thicker than 2 m, characterized by low concentrations of pyrite and very thin and discontinuous yellow or greyish-green veins of phyllosilicates. The walls of the quarry are somewhere densely fractured, with discontinuity traces of variable lengths, from metric to decametric, that penetrate inward the slopes with thickness difficult to estimate, often exceeding few meters in depth. The intersection between different fracture systems and the slope surface generates rocky blocks and wedges of variable size that may be subject to phenomena of gravitational instability due to the variation of meteorological, hydraulic and dynamic conditions. In such an environmental context, data obtained from traditional engineering-geological surveys, collected at the foot of the slopes and along the wall by climbing technicians, may be used for rock mass characterization. This data, which holds important statistical value, does not provide deterministic information of the complete slope necessary to localize the hazard and plan the remediation works. There are limitations and technical difficulties in the implementation of such rope access survey methods, including the close proximity to the quarry walls which compromises visibility and the ability to recognize large-scale geological features, as well as the unavailability of suitable cartographic maps. In this study, these difficulties were overcome by integrating available data with geometrical and structural info derived from the analysis of geo-referenced 3D point cloud acquired using photogrammetric techniques. Also a terrestrial laser scanning was carried out from two different point of view, even if the elevation of the slopes creates shadows in the data, limiting the feasibility of geo-structural survey. To overcome such a limitation, we utilized a rotary wing Aibotix Aibot X6 RPAS geared with a Nikon Coolpix A camera. The drone flights were executed in manual modality and the images were acquired, according to the characteristics of the area, under different acquisition angles, both from the top and in front of the quarry escarpments. Furthermore, photos were captured very close to the slopes, allowing to produce a dense 3D point cloud by image processing. A topographic survey was carried out in order to guarantee the needed spatial accuracy to the process of images exterior orientation. The coordinates of GCPs and check points were calculated through the post-processing of data collected by using two GPS receivers, operating in static modality with an acquisition time of up to 3 hours, and a Total Station. The photogrammetric processing of image blocks allowed us the creation of dense 3D point cloud, DTM, orthophotos, and 3D textured model with high level of cartographic detail. Post-processing included contemporary observations of 3 permanent GPS stations, and the conversion of GCPs elevation from ellipsoid to orthometric heights using ConveRgo software. The topographic survey included targets used for registering two point clouds acquired by a Leica Scanstation2 laser scanner. By this way, the co-registration of the point clouds from laser scanning and RPAS image processing makes possible the comparison and integration of data. An unique 3D repository was created including updated images of the walls and their morphological and structural setting. By the analysis and interpretation of the 3D point clouds and high detailed and updated photographs, all the rock discontinuities were deterministically characterized in terms of attitude, persistence, and spacing and their distribution mapped on the orthophotos. Furthermore, the main discontinuity sets were identified through a density

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analysis of attitudes in stereographic projection. The size and shape of potentially unstable blocks identified along the slopes were also measured. Data from the photointerpretation was validated by and integrated with results of scan lines from traditional engineering-geological surveys which were conducted in accessible outcrops. Joint systems were fully characterized and used to classify the rock mass using the Rock Mass Rating method (RMR - Bieniawski, 1989) and the Geological Strength Index (GSI - Hoek, 1994). Finally, the kinematic and dynamic analysis of possible rock failures was performed for all the slopes. The characterization of localized hazard has indicated the deterministic safety factors of rock blocks and wedges, used by the company responsible for the exploitation activities to plan the protection works for safety guarantee and the continuation of marble extraction.

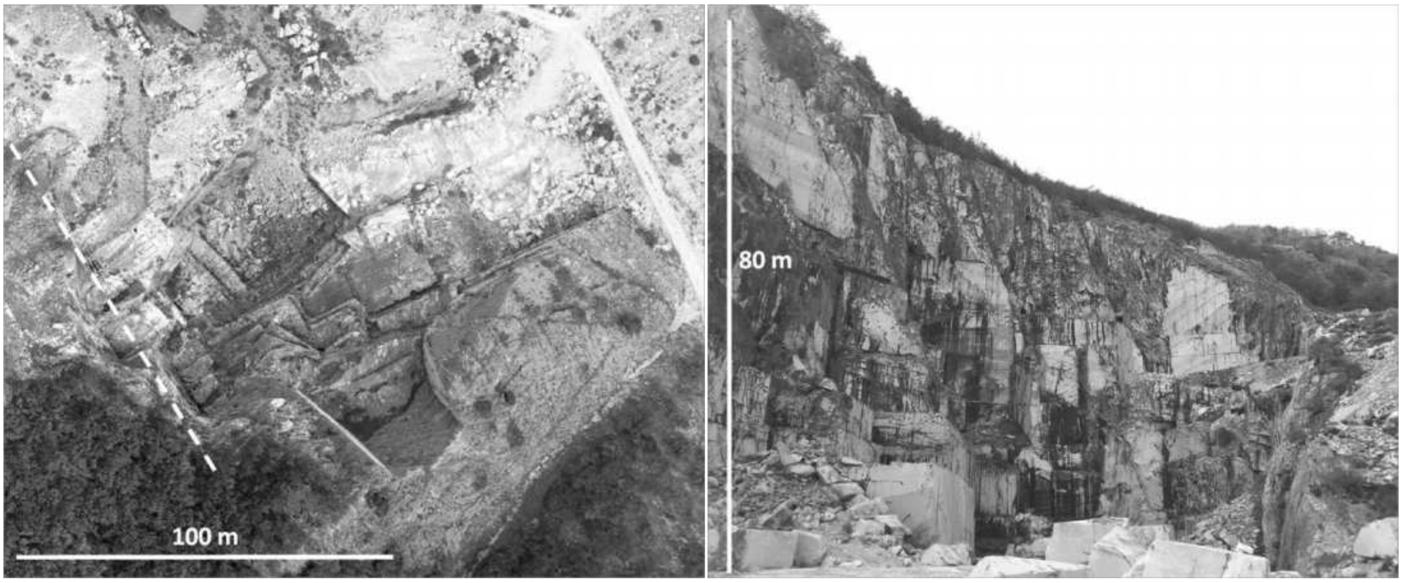

Figure 1: The study area from different perspectives: aerial view from RPAS (left); the dashed white line indicates the walls shown in the frontal view from TLS (right).

2 CONCLUSIONS

Outputs from this application show the great advantage of modern RPAS that can be successfully applied for the analysis of vertical slopes, especially in areas difficult to access with traditional techniques. The executed flights permitted rapid acquisition of a great number of high resolution photos, with normal and parallel directions of shooting with respect to the slopes. The analysis and interpretation of the 3D point cloud and photographs allowed the creation of large-scale topographic and thematic layers, as well as the inspection of the whole study area in order to identify critical conditions. Moreover, the integration with data from terrestrial laser scanning allowed to assess the reliability of the RPAS outputs, to create a complete and realistic 3D model of the slopes, and to generate updated orthoimages of the quarry walls at high spatial resolution. Data from the application of geomatics techniques, permitted to characterize the rock mass, perform the slope stability analyses, and map the hazard for a proper planning or remediation works.

3 ACKNOWLEDGEMENTS

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Contribution in the risk management of undermined rock slope

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Keywords: rock slope, cavity, undermined, modeling, hazard

A wide variety of landslides is present in the French territory including those related to undermined rock slope. The objective of the approach developed in this paper is to better characterize this natural hazard and to develop a reliable methodology. One difficulty is to specify how the landslide hazard linked to rock slope interacts with the landslide hazard induced by underground cavity.

1 CONTEXT

1.1 PROBLEM

In the current approach, landslides related rock slope and those linked to the underground cavity are often independently assessed and superimposed to the final hazard in the investigated area. Other approaches rather choose to retain the hazard upper bound as the final hazard. The objective of our work is to provide factors to build a relevant methodology in order to evaluate the undermined rock slope hazard and to get a common basis or reference throughout the French territory.

1.2 APPROACH

Among the parameters defining a rock slope, three major arise (Figure 1): the height of the rock slope (Hf), the distance d between the rock slope and the cavity measured horizontally, and the size of the cavity H corresponding to the diameter of the largest inscribed circle of the cavity. In a first approach and following Terzaghi (1943), the influence extent of the underground cavity M1, was taken to 5xH and the influence extent of the rock slope M2 is Hf (Didier and Tritsch, 1999). If influence extents M1 and M2 are distinct, each hazard can be assessed independently but if the influence extents intersect, the hazards must be combined into one and rock slope-cavity interaction should be taken into account.

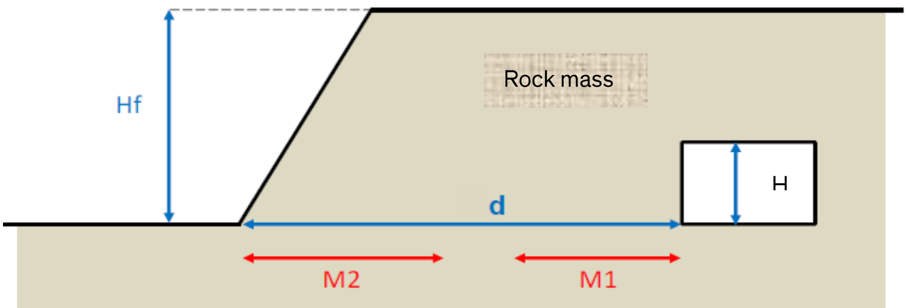

Figure 8 : Schematic representation of undermined rock slope

This influence distance is based on a strong assumption, thus we aimed to evaluate more accurately using numerical modeling.

2 DETERMINATION OF INFLUENCE DISTANCE USING NUMERICAL MODELING

2.1 MODELING OF AN UNDERMINED CLIFF

Numerical simulations were performed using the UDEC 2D software. We have identified four reference cases: a stable slope with a stable cavity, a stable slope with a damaged cavity (cavity with many signs of instability), an unstable slope (slope with many signs of instability) with a stable cavity and finally an unstable slope with a damaged cavity. Simulations were performed in 3 phases: stressed initialization, slope realization and excavation of the cavity. The impact of the cavity on the rock slope behavior is analyzed in phase 3. Among the result of simulations, two seem to be particularly interesting: the displacement field which shows the collapsed volume considered where large displacements are observed and the number of open joints which means an internal damage of the rock mass considered in the case of small displacements.

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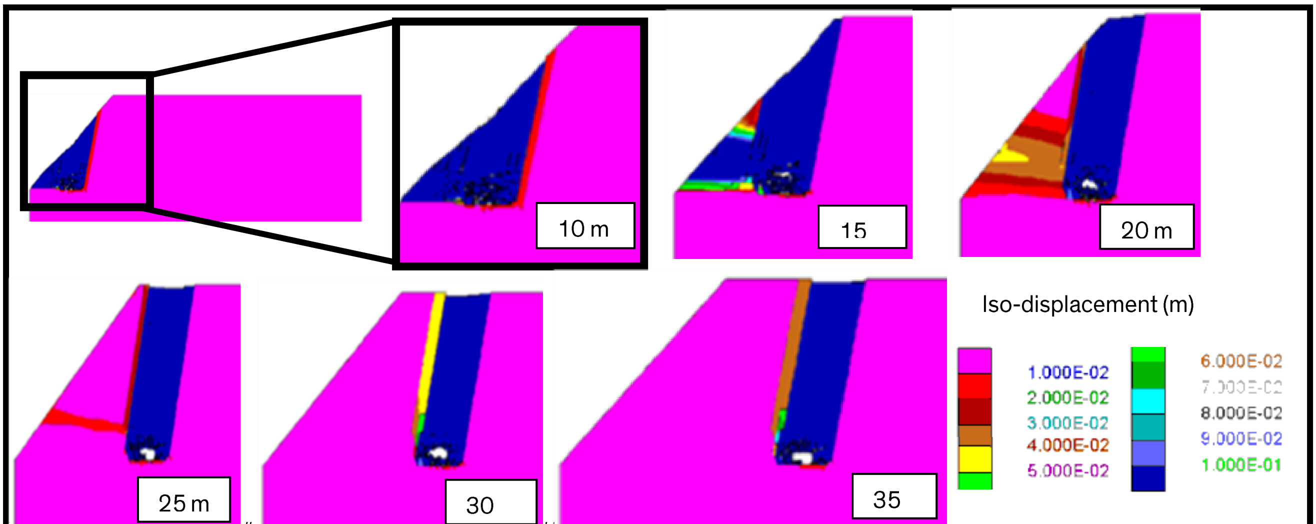
For example, for the reference case of stable slope with damaged cavity, where large displacements are observed, it is noted that for a distance of 30 m and beyond, the modeled cavity appears to have no impact on the cliff stability (Figure 2).



igure 9 : Visualization of the displacement depending on distance d between rock slope and cavity

For case of unstable rock slope with a stable cavity where small displacements are observed, we note that for a distance of lower than 35 m, the cavity has an impact on a rock slope causing a failure of joints inducing the instability of the rock slope (Figure 10).

These simulations are performed for the four reference cases and other cases are added by changing the angle of the slope, the cavity dimensions, the height of the rock slope, the fracture network of rock mass for instance. For each case, we got a critical distance of influence (D). If d<D, the cavity influences the slope stability. From a statistical analysis it is possible to determine this distance D taking account of the most important parameters. Finally, we got the following equation of D in meter:

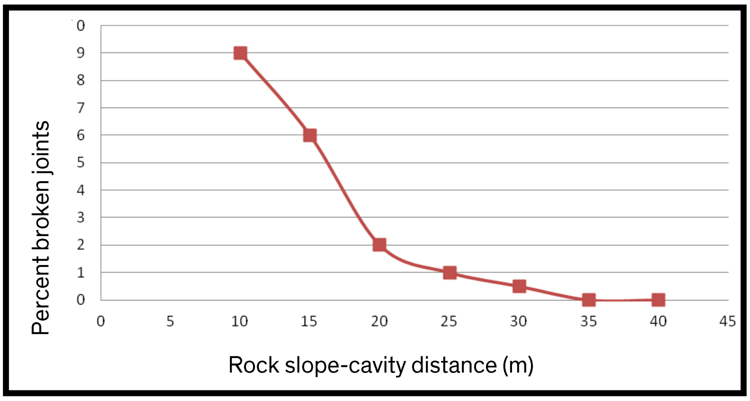


Figure 10 : Percent broken joints in the rock mass according to the rock slope-cavity

D = 3H + Hf + [5 (if the cavity shows signs of instability)] + [5 (if the rock slope is unstable)]

with H the height or the equivalent diameter of the cavity and Hf the height of cliff.

2.2 APPLICATION OF THE INTERACTION DISTANCE FORMULA ON TWO NATURAL SITES

The relationship giving the minimum distance D between a rock slope and an underground cavity is an important factor because it allows to determine which configuration is suitable to apply the methodology of undermined rock slope. Furthermore, this distance also defines a domain within which the stress field in the rock mass may appear more complex. The fracturing mode examined on the walls of the pillars located in the influence area D is not affected by a classic compression failure but rather by a shear mode failure. This approach has been verified on both sides of undermined rock slope namely a limestone quarry in the Dordogne and a chalk quarry in the Paris region.

3 CONCLUSION

The use of numerical modeling allows us to verify the complex interactions between a rock slope and an underground cavity. Starting from four reference cases, we were able to determine a critical influence distance D according to the following formula: D = 3H + Hf + [5 (if degraded cavity)] + [5 (if unstable slope)]. From feedbacks, the influence of the rock slope affects particularly the failure mode of pillars. Indeed near a highly sloped recovery, traction stresses may appear. The failure mechanism of undermining is then modified. It is important because it will better characterize this natural hazard in the future to build a methodology for evaluating the undermined rock slope hazard and to better manage risk.

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A real-time seismic and displacement monitoring system for rock instabilities assessment : Applying in the french Alps

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Keywords: Monitoring system, Seismic noise, Rockfall,

1 ABSTRACT

The cliff instabilities are a common problem for the local authorities in the french alps, affecting significantly its local economy. The implementation of protections against rockfalls is not a comprehensive solution. The instrumentation remains an important alternative to the monitoring of instabilities and very useful to evaluate the safety of hazardous areas. The classic and most common method is to follow the surface movements of the cliff using displacement measurement sensor such as crackmeter, EDM or radar. The monitoring of the internal deformations are a difficult variable to apprehend. The seismic noise method can meet that need.

Several instrumentation systems have been implement in the french Alps where the danger is resulting in the construction or improvement of roads, railway lines and buildings nearby steep slopes or cliffs. Access to isolated mountain area are now more and more used by everybody, forcing local authorities to improve their capacities in the safety management. Instrumentation becomes a key element in the decision chain, provided that the monitoring will be done in real time. The instant prediction of rockfall incidents can help significantly the local authorities to take all the necessary safety measures in case where traffic is interrupted.

We have develop a multi-sensor instrumental device to form a sound basis for the continuous acquisition of the relevant parameters for monitoring the stability of rock masses. This self-contained device is capable of continuously measuring and storing high and low frequency data generated by different types of sensor over long periods (several months). Moreover, it is linked up to a remote-monitoring system for supervising the site and checking the operation of the instrumentation. The measurement station developed can also carry out pre-processing of the data collected and includes monitoring algorithms combining different criteria for a warning management procedure. A meshed-network transmission system and client-server oriented remote supervision system have also been developed within the context of this project. Today, this monitoring system was implemented on more than fifty sites in France.

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SiteMonitor4D: Automated rock slope stability monitoring using high-resolution time-series Terrestrial Laser Scanner data

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Keywords: Terrestrial Laser Scanner, Rockfall, Automated, Monitoring

1 INTRODUCTION

In the last decade, terrestrial laser scanners (TLS) have emerged as an important tool for 3D spatial data acquisition and rock slope hazard assessment. TLS offers the capability to capture very detailed time-series measurements of unstable slopes that can be used to generate 3D change-detection maps. In addition, TLS data can be used to derive important metrics for assessing the evolution of a rock slope over time, enabling the enhancement of slope stability models.

This paper presents an overview of the SiteMonitor 4D TLS slope monitoring solution, highlighting automated processing techniques for measuring changes in 3D slope geometry, rockfall frequency and fluctuations in slope hydrology. To demonstrate the advanced monitoring and analytical capabilities of SiteMonitor 4D, we present case studies from a coastal environment and a mining environment.

1.1 SITEMONITOR 4D

SiteMonitor 4D is a powerful system designed as a bespoke slope stability tool combining state of the art TLS alongside powerful analysis software to detect and measure small 3D changes across a surface. The system works through the creation of a stable scanning platform in a fixed position (a hut or pillar), which then automatically captures repeat 3D measurements of an area of interest through time. The software processes and identifies changes in these measurements and outputs the results as difference maps.

SiteMonitor 4D combines multi-level based measurements to include range, movement and displacement, rockfall and acceleration functions, focused filtering models, geo-technical models and a highly specialized alarming system that delivers multiple alarming formations to the decision maker.

1.2 COASTAL MONITORING

Understanding the nature and mechanisms of cliff erosion is of vital importance to predicting the likely future movement of the coastline (Lim et al., 2010). This project uses continuously captured high-resolution, 3D data to generate unprecedented detail on the changes experienced at a coastal cliff site located at the historic town of Whitby, North Yorkshire coast, UK.

One of the principal objectives of this project was to provide continuous monitoring of the cliff face, allowing changes resulting from rockfall to be automatically analysed inreal-time. To meet this objective, the 3D Laser Mapping SiteMonitor system was installed in a lighthouse, providing a secure location with clear line-of-sight to the cliff face (Figure 1). The SiteMonitor system automatically schedules the capture and analysis of 3D laser scan data 24 hours a day at 30 minute intervals. Within each scan, measurements of the cliff face are taken at approximately 10 cm intervals, generating over 2 million points per scan. Captured data is wirelessly transmitted to a server, where automatic processing algorithms detect and characterise 3D surface change and rockfall activity (frequency and magnitude). Analysis of time-series data revealed a number of significant rockfall events that were well correlated with elevated levels of rainfall.

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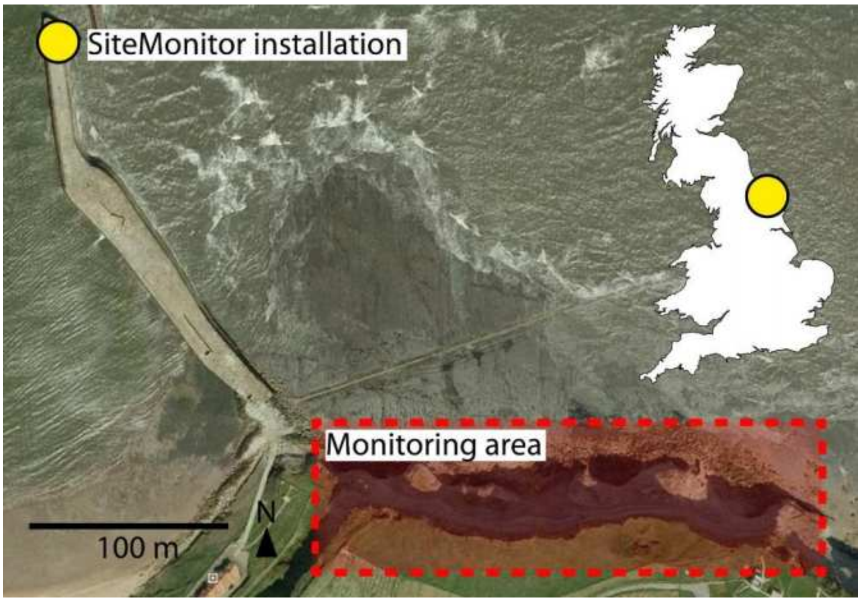

Figure 1: Location of the SiteMonitor installation and monitoring area at the Whitby (UK) study site.

1.3 MINE MONITORING

Slope monitoring forms an integral, safety critical part of slope management in open pit mining operations. This is to provide information to detect potential unstable ground, assess the performance of slope design and identify any slope instability and/or failure mechanisms that develop. If failure mechanisms are understood and the slopes are properly monitored, the risk of slope movement and the subsequent consequences can be considerably reduced. This allows for optimal mining conditions that are safe for mine personnel as well as working equipment.

3D Laser Mapping has successfully implemented SiteMonitor 4D in safety critical deformation monitoring regimes across mining sites worldwide. Here, we present the results of a TLS monitoring system in one of our mining client sites situated in Brazil.

The objective for this project was to provide a continuous safety assessment of a portion of the pit slope which had shown evidence of instability due to mine evolution. The installation of the SiteMonitor system enabled real time monitoring and geotechnical analysis of slope movement while daily mine operations continued in close proximity. The project captured TLS data at a resolution of 0.9 m on the pit wall with an hourly monitoring schedule, enabling a high resolution model of the slope instability to be determined. Time-series analysis of the TLS data, alongside prism and weather station data for a 12-month period documents a slow moving, large wedge failure advancement with displacements approximating 2m over the 57000m^3 wedge volume.

2 CONCLUSIONS

SiteMonitor 4D can be demonstrated to meet the objectives of safety critical monitoring systems not only through its integration of high resolution and high precision TLS technology, but also in providing a high level suite of software tools for the geotechnical engineer.

The safe management of the large-scale wedge failure evidenced in Brazil demonstrates the benefits in the implementation of high-resolution time-series TLS data and geotechnical tools to track the rates of rotation, translation and displacement forming in this region. In addition, high spatio-temporal resolution monitoring at Whitby is providing invaluable information for the improvement of coastal rock slope stability models.

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Characterisation of the rock mass for subsurface rock slope design based on a multidisciplinary approach

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Keywords: Subsurface rock slopes, Stability analysis, Structural analysis, Numerical photo-interpretation, Classification of the rock mass

For the design of rock slopes along road or railway project, the characterization of the rock mass is a preliminary step to determine the type of the potential failure. For deep underground structures or when the dimensions of the cuts are greater than the discontinuities spacing (great height open pit mine), continuous modelling of the rock mass is commonly applied and semi-quantitative classification or model are used (Bieniawski, 1992 ; Barton et al 1974; Singh and Goel, 2011). For subsurface project, rock slope from heights of several tens of meters, stress is low and the discontinuities are mainly open. The calculation of the stability depends mostly on the structural conditions: the discontinuities control the behaviour of rock masses (discontinuous modelling), or the rock mass is very heavily jointed, and weak and the rock masses is analysed as a soil (continuous modelling). Then the preliminary steps of the stability analysis are: (1) a structural analysis in order to determine the main joint sets and faults, (2) a classification of the rock masses according to its degree of weathering.

During preliminary design studies, the structural data collection, well known and documented (Hoek and Bray, 1981), is usually very limited. Thanks to technological advances in topographical measurements (LIDAR), the numerical photo-interpretation can be a very effective tool for the structure analysis of the rock mass. The major results are discussed from practical examples of road projects.

Estimates of the rock mass behaviour of the rock mass are generally based upon the judgement of an experienced engineer. Based on multidisciplinary criteria from common geotechnical field survey (geological and geophysical data, logging tools) and the experience of rock slopes projects, a simple and operational classification of the rock mass is proposed. Its gives rock mass boundary between soil and heavily jointed rock to low fractured rock in order to determine a continuous or a discontinuous modelling for the stability analysis.

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Coupling of geophysical methods for weathered rock masses characterization in railway field (Morlaix railway cutting case study)

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Keywords: SNCF, Rockslide, Morlaix, railway, slope, fracturing, survey

After January 2013, large rockslide (3000 m^3) occurred on the railway cutting of Morlaix (North of Britanny, France), SNCF made ADRGT responsible for establishing a geophysical approach adapted to railway Rock-cutting context, to evaluate fracturing and weathering of the slopes. This study is part of a research project on weathered rock characterization (Bottelin et al., 2012 ; Lorier et al., 2012 ; Meric et al., 2005)

1 SITE AND SURVEYS DESCRIPTIONS

1.1 SITE DESCRIPTION

On January 25th 2013, 3000 m^3 of rocks fell down (Figure 1B) from the railway cutting of Morlaix (North of Britanny) (Figure 1C) which links Paris to Brest and caused 2 months of train traffic interruption. The rockslide seemed to occur near a fault which delimits two discordant geologic units, such as, in the North, schists and felspathic greywacke sandstones with volcanic elements (from upper Devonian), and, in the South, aluminous carburetted shales (from lower Devonian).

1.2 SURVEYS DESCRIPTION

Four geophysical methods were used to characterize banks fracturing in the railway cutting: seismic refraction (Plus-Minus and tomography analyses), Electrical Resistivity Tomography (ERT), Induced Polarization (IP) and Ground Penetration Radar (GPR). These methods have been coupled along thirteen profiles located within four studied areas (Z1, Z2, Z4 and Z5, Figure 1A). Only results of Z1will be presented because of its proximity to the landslide area and to the different geotechnical surveys. Z1 is composed of five profiles acquired with seismic, IP and ERT methods. One profile (Seismic profile 1 (PS1) and IP-ERT profile 1 (PE1), Figure 1A) is located along the slope. Two profiles (PS6/PE6 and PS7/PE7) are located on top of the slope. Finally two profiles (PS4/PE4 and PS5/PE5) are located at the base of the slope.

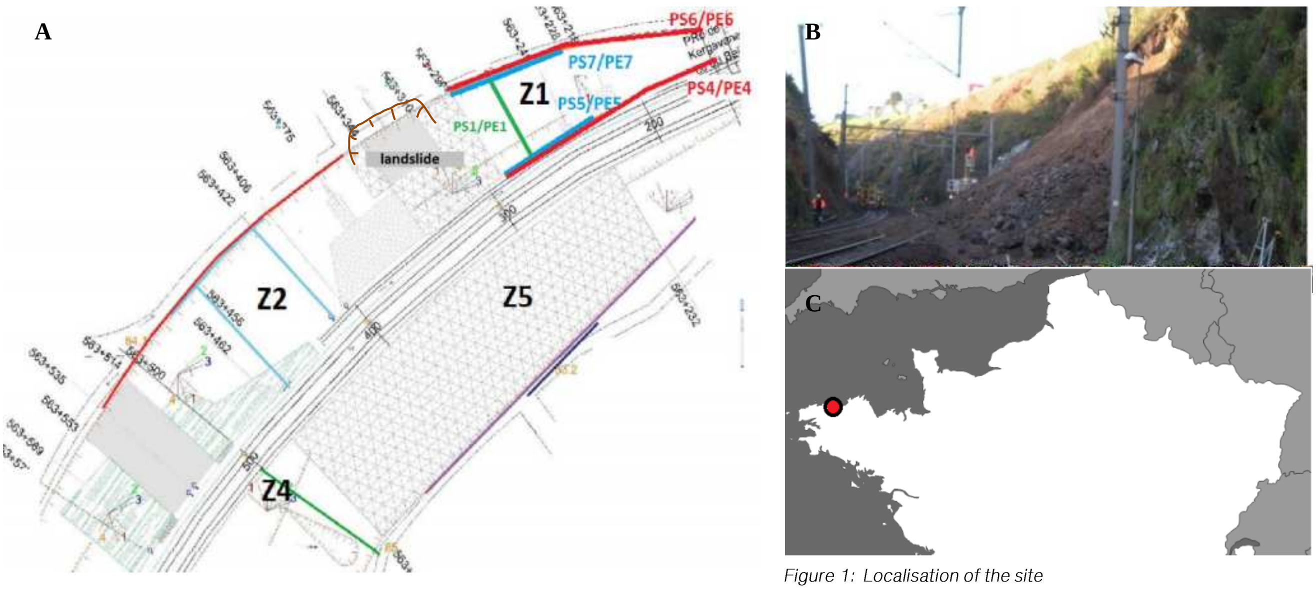

Figure 1: Localisation of the site

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2 DATA ANALYSIS AND RESULTS

Only results of Z1 will be presented. IP measurements are very noisy so that their interpretation is not consistent. Results will be presented depending on their location along the slope. Results which are not usable will not be displayed.

2.1 TOP OF THE SLOPE

On top of the slope (Figure 2), the different geophysical methods are complementary: high contrasts of both resistivity and seismic velocity allow deducing the geologic distribution of materials until 25 m depth. At shallow depth, a resistive material (around 1200 Ω.m) with low seismic velocity (around 400 m/s for shallow layers until 2m depth and 1500 m/s deeper) overlays a lower resistive material (around 350 Ω.m) which has a velocity of 3000 m/s.

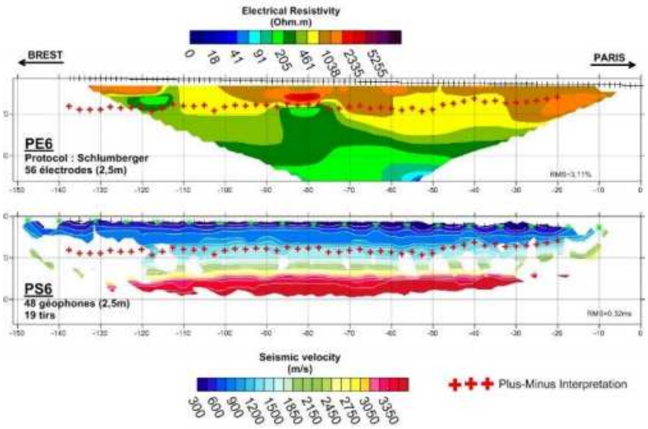

Figure 2: Results on top of the bank: PE6 and PS6

2.2 BASE OF THE SLOPE

At the base of the slope, PE4 and PE5 were acquired on bottom (in a first time), and at 1 meter high in the rock-cutting (in a second time). In both cases, ERT and IP measurements are not interpretable due to surrounding noise. However, seismic measurements give the velocity of the bedrock which is around 3500 to 4000 m/s at 1.5 m depth.

2.3 ALONG THE SLOPE

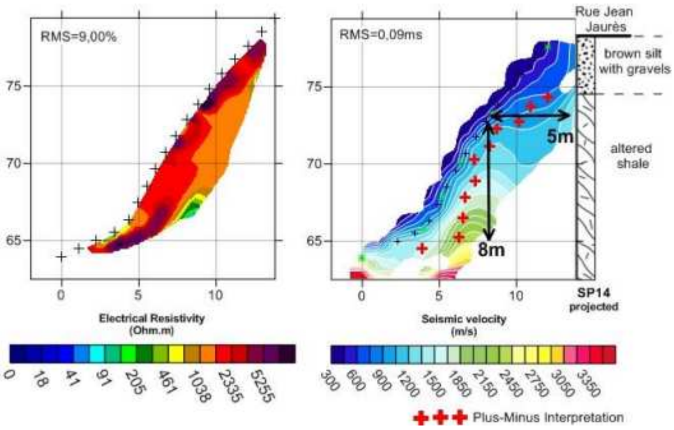

Figure 3: Results across the bank: PE1 and PS1

Along the slope, ERT measurements do not have a sufficient investigation depth to clearly see the bedrock. Seismic results seem to match with the geometry of the rockslide, about 8 m high and 5 m width. Indeed, at 8 m high from the base, there is an anomaly of low velocity which extends over 5 m. It can characterize a fractured area. In comparison with the destructive sounding SP14 (Geologic Log on Figure 3) realized on top of the bank, near the rockslide, Plus-Minus interpretation fits with depth of geologic horizons (a layer of around 4 to 5 m of brown silt with gravels fits with 450 m/s of seismic velocity overlaying altered shales with a velocity around 1300 m/s to 2500 m/s). Assuming a bedrock velocity about 4000 m/s, PS1 shows too small velocities to reach the bedrock depth.

3 CONCLUSION

To conclude, during this research project, some points are highlighted concerning the establishment of a geophysical approach adapted to railway context. First, IP measurements are very noisy and are not adapted to the kind of materials. Moreover, IP-ERT surveys on the slope base give unusable results. However, ERT and seismic refraction methods are complementary to characterize railway cutting. Seismic cross sections along the slope inquire about highly altered materials in agreement with rockslide geometry. Also, ERT and seismic surveys on top of the slope show contrasts that give information about the geological composition of the slope. Analyses are still underway due to the fact that measurements are recent.

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Session 5B

Trajectography analysis (1)

Discrete modelling of rock impact on trees to improve forest integration into trajectory analysis tools

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Keywords: Forest, impact, modelling, Discrete Element Method

1 INTRODUCTION

In the field of rockfall hazard assessment, the integration of forest effects on rocks propagation and the assessment of forest protection function are increasingly being studied. Research studies were mainly dedicated to integrate forest effects into empirical rockfall models or lumped-mass rockfall models. Both modelling approaches present efficient computational times and global accounting of forest effects. However, they do not allow integrating all the physical processes occurring during the interaction between the blocks and the tree. In particular, they do not account for the respective contributions of the different tree components - stem, root system and crown - on the impacting block velocity changes and energy reduction.

Integrating all these contributions requires developing simulation tools based on mechanical approaches. The main difficulties related with the practical use of these models remain the in-situ characterization of the numerous parameters involved and their integration into trajectory analysis tools.

This research work (Toe, 2016) is dedicated to propose a mechanical model of the interaction between a block and a tree and to identify the key parameters controlling the kinematics of the block after impact. This model is finally integrated into a trajectory analysis tool based on the Discrete Element Method (DEM) to assess forest effects on rock propagation.

2 MODELLING ROCK IMPACTS ON TREES

Modelling the impact of a block on a tree using the DEM (open-source code Yade-Dem) allows taking into account large displacements, material non-linearities and contacts between the block and the tree. Tree stems are represented by flexible cylinders model as elasto-plastic beams sustaining normal, shearing, bending, and twisting loading. The root system is modelled using a non-linear rotational spring acting on the bending moment at the bottom of the stem. The crown is taken into account using an additional mass distributed uniformly on the upper part of the tree. The block is represented by a sphere or an unbreakable assembly of spheres. The contact between the block and the stem is finally explicitly modelled.

The DEM model was validated using laboratory impact tests carried out on 41fresh beech (Fagus Sylvatica) stems. Each stem was 1.3 m long with a diameter between 3 to 7 cm. Wood stems were clamped on a rigid structure and impacted by a 149 kg Charpy pendulum.

After the model calibration, an intensive simulation campaign of blocks impacting trees was done to identify the input parameters controlling the block kinematics after impact based on the calculation of Sobol indices that correspond, for each output parameter, to the ratio between the partial variance associated with each input parameter and the total variance of the output parameter. 19 input parameters were considered in the DEM simulation model: 11 parameters were related to the tree and 8 parameters to the block. The results highlight that the impact velocity, the stem diameter, and the impact point horizontal location are the three input parameters that mainly control the block kinematics after impact. These parameters correspond to those classically used in the model of rock impacts on trees of classical trajectory analysis tools. However, other parameters used in the previous models (tree species, in particular) are not identified as prevailing parameters in the sensitivity analysis.

3 FOREST INTREGRATION INTO ROCKFALL SIMULATION MODELS

A trajectory analysis tool based on the DEM and integrating the rock-tree impact model presented above was developed. The modelling of the rock rebound on the slope surface is modelled using the approach developed by (Thoeni et al., 2014). The slope terrain is modelled as a surface made of fixed spheres which allows modelling both the site topography and the local soil roughness. Cubical rocks made of 8 unbreakable spheres with the same diameters are considered.

All parameters of the rebound model were set at fixed values corresponding to impacts on a soft scree (Thoeni et al., 2014). All parameters of the impact on tree model that were not identified as prevailing in the previous analysis were set at fixed mean values in their varying range. The only prevailing parameter that is not explicitly calculated for each impact on a tree, i.e. the tree diameter, is characterized for each tree of the virtual forest stand.

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A specific algorithm was developed to generate virtual forest stands. This algorithm requires defining the tree diameter distribution and indicators of the spatial distribution of the trees location (Ripley functions). These data have been measured for 8 forest stands from field inventories on 50mx50m plots to assess forest effects on rock propagation.
A trajectory analysis for a given slope and forest configuration consists of releasing successively *n* rocks. The only variable parameter between the successive releases is the rock initial orientation.

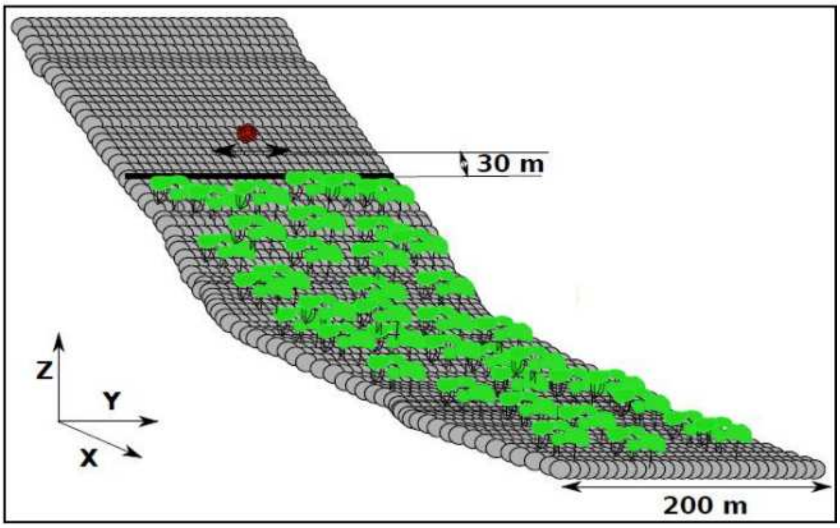

Figure 1: The DEM based trajectory analysis code integrating the effect of forest.

Trajectory analyses without forest and for the 8 forest stands have been done. Additional trajectory analyses have also been done using the software Rockyfor3D for comparison purposes. Rockyfor3d is a 3D lumped-mass rockfall model that allows explicitly modelling the block interaction with each tree of the forest stands using an empirical interaction model. The values of Rockyfor3D parameters have been set to obtain similar rock propagation paths without forest.
The comparison of the simulations using the model presented and Rockyfor3D shows a significant effect of the forest on the rock propagation: the mean decrease in rock propagation length is always larger than 30%. In addition the influence of forest simulated is very similar using both models.

4 CONCLUSION

The use of the DEM model of rock impacts on trees developed and calibrated allowed identifying the parameters that control the block kinematics after impact (impact velocity, stem diameter, and impact point horizontal location). The analysis of forest effects on rock propagation using a trajectory analysis code based on this model showed the significant effect of forest. This trajectory analysis tool provides results similar to another code integrating the effect of forest (Rockyfor3D) for a smaller amount of forest input parameters and by integrating a smaller number of variable parameters (only the initial orientation of the rocks). The limitation of this approach is the important computational effort required for doing the simulations.

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Using trajectory analysis tools for embankments design

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Keywords: trajectory analysis, embankment, design

1 INTRODUCTION

Trajectory simulation codes are widely used in view of land use planning as well as for the design of rockfall protection structures such as embankments. This article investigates the relevance of currently used trajectory analysis tools for this latter purpose through three case studies. The first case corresponds to the initial design of an embankment. The two others correspond to the assessment of the efficiency of existing structures: one which current dimensions do not correspond to the initial design due to building problems, while the other was built without any trajectory simulation results. A general framework for the use of trajectory analysis tools for embankment initial design or efficiency checking is first presented and second applied to the above mentioned case studies.

2 GENERAL FRAMEWORK

Classically, embankments are positioned uphill the elements at risks preferably where the passing height and energy of the blocks are minimum even if technical considerations can also influence the location of the structure. In this initial phase, trajectory analysis tools can provide information for the optimal location of the structure in terms of rock energy and passing height. In a second time, it is used to assess the embankment protection efficiency, i.e. the residual hazard down the embankment, after its insertion in a digital terrain model or a slope profile of the study site. In this phase, simulation results using classical rockfall simulation tools reveal abnormal trajectories in the vicinity of the embankment suggesting limitations related to both the spatial resolution of the terrain model and to the rebound models (Lambert et al., 2013). To cope with these limitations, simulation models can be adapted with specific rebound models for impacts in the ditch or on the mountain-side facing on the embankment (Lambert et al., 2013).

In a practical context, the best choice in terms of cost-efficiency for the trajectory analysis tool depends on the available data for the study case, in particular related with the slope terrain representation (2D-slope profile, 3D-Digital Terrain Model…). In addition, in case of previous trajectory analyses, the simulation tool should also be chosen so that the results from previous analyses could be used or, at least, so that the comparison with the previous analyses is possible. The optimal use of existing previous studies and data is crucial in a practical context to optimise the field expertise and simulation efforts.

In the context of the study, three rockfall simulation tools have been used:

- 3D rockfall simulation model (Rockyfor3D) for the simulation of rock propagation along the study site in case of an available Digital Terrain Model with appropriate resolution (smaller than 10m).
- 2D rockfall simulation model (CRSP) for the simulation of rock propagation along the slope profile of the study site extracted from the Digital Terrain Model or measured in the field.
- rockfall simulation model adapted to model impacts on embankments ditch or mountain-side facing for the simulation of rock propagation in the vicinity of embankments (Lambert et al., 2013). The main specificity of this model consists of using a rebound modelling approach better adapted to normal impacts than classical ones (Lambert et al., 2013).

3 CASE STUDIES

3.1 INITIAL DESIGN

The use of trajectory analysis tools for the initial design of embankments is illustrated by a case study located in the commune of Morzine (France). Following prescriptions from the Risk Prevention Plan of the municipality, a study for the design of protection structures in the locality of Les Prodains was conducted. The study benefited from a previous 2D trajectory analysis in the site and from a Digital Terrain Model with a 4m resolution.

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Given the available data, 3D rock propagation analyses were done to identify the optimum zone for the implantation of an embankment in accordance with technical feasibility. In this initial phase, 3D rockfall analysis was done to identify the main rock propagation corridors. Complementary with the 3D analysis a detailed 2D trajectory analysis was done for slope profiles corresponding to preferential propagation paths identified from the 3D analysis. This 2D analysis allowed characterizing the distributions of rocks velocities and passing heights at the entrance of the ditch of the embankment.

Finally, another 2D analysis, at the vicinity of the embankment was done with an adapted model considering as input parameters the results from the previous 2D trajectory analysis. Different embankment geometries have been tested and, finally, a 5m high embankment with a 7m large ditch was chosen as an optimum allowing only 0.05% of the rocks to overpass the structure.

3.2 EXISTING EMBANKMENTS EFFICIENCY

3.2.1 Embankment different from the initial design due to building problems

Technical problems have been encountered during the building of an embankment in the commune of Le Freney d'Oisans (France). The embankment profile in terms of height and slope of the mountain-side facing was changed. Trajectory analysis tools have been used to check the efficiency of the embankment as constructed.

For this study, no Digital Terrain Model was available but a detailed 2D trajectory analysis was already done for the initial design of the embankment. Thus, only the specific rock propagation model adapted to model impacts on the vicinity of embankments was used considering as input parameters the results from the previous 2D trajectory analysis at the entrance of the ditch. Only the distributions of the norm of the velocity and of the rock passing height were available. The definition of the input parameters thus required doing assumption on the velocity direction, rotational velocity, and on the correlations between rock velocity and passing height.

The comparison between the efficiency of the design and existing embankments shows a minor difference between the 2 works: for 2m^3 (resp. 5m^3) rocks, the design embankment is overpassed by less than 0.01% (resp. 3.28%) of the rocks whereas the existing one is also overpassed by less than 0.01% (resp. 3.39%).

3.2.2 Embankment initially not designed using trajectory analysis

Due to recent events, the efficiency of a 30 years old embankment protecting a railway, located in the commune of Les Houches (France) needed to be clarified. A complete trajectory analysis was done using both 2D and 3D approaches to answer this question and to provide solutions for the improvement of the embankment efficiency.

The analysis of the initial project showed that the existing embankment geometry was not in accordance with the initial project (height ranging from 2.5 to 3.5m instead of 6m, ditch width ranging from 2 to 3m instead of 3 to 4m). A Digital Terrain Model with a 1m resolution being available, a similar approach as presented in section 2.1 was chosen (3D global trajectory analysis, 2D trajectory analysis on slope profile determined from the 3D analysis, and local trajectory analysis with input parameters from the 2D analysis).

The results reveal a significant probability of rock overpassing the existing embankment (2.6%, 1.4%, and 0.3% for blocks 30 m^3, 5m^3, and 0.5m^3 in volume, resp.). An increase of the embankment height and of the ditch length to the values prescribed in the initial project would reduce the overpassing probability to 1%, 0.4% and less than 0.01% for blocks 30 m^3, 5m^3, and 0.5m^3 in volume, resp.). Such improvements of the existing embankment could be done using present technical solutions such as an increase of the uphill mountain facing slope.

4 CONCLUSION

The case studies presented illustrate the framework proposed for the use of trajectory analysis tools for embankment design purposes. It also emphasizes the importance of the availability of data (Digital terrain Model) and previous studies during the choice of both the analysis tools potentially usable and the definition of assumptions concerning model input parameters, in particular.

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Significance of digital elevation model resolution for numerical rockfall simulations

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Keywords: digital elevation models DEM, roughness, terrain, trajectory modelling, rock shape, RAMMS

The current state of the art in numerical rockfall modelling permits full three-dimensional simulation of real rock shapes and their interactions with the terrain. The digital elevation models (DEM) represent the three-dimensional terrain, providing the geometric terrain information on which the spatial model parameters are assigned. This is fundamental to numerical simulations of mass movements. DEM's can be obtained from a number of sources, today mostly remote sensing, and offer spatial resolutions ranging from centimetres up to 30 - 90 m. The spatial resolution representing the terrain morphology can have a large impact on modelling results. In particular if finer scale morphologies (centimetres to meters) such as the roughness of a scree slope or a boulder field are included (fine resolutions) in the DEM or not (coarse resolutions) (Bühler et al. 2011).

1 RAMMS::ROCKFALL

RAMMS::ROCKFALL is the new Rockfall module in the RAMMS-Software (RApid Mass Movement Simulation). In collaboration with the ETH Zürich we developed a novel contact-algorithm, including slippage, to model the rockfall trajectories (Leine et al. 2014). The model accounts for real rock shapes, computing their runout trajectory over three-dimensional terrain, including their jump heights, velocities, rotational velocities, total kinetic energies and contact-impact forces. All possible modes of rockfall motion (jumping, rolling and sliding) are deterministically simulated. Dynamics of single trajectories can be individually inspected or multiple trajectories can be statistically analyzed. The user-friendly interface does not require additional GIS-Software. Results are easily exported into GIS software and visualized in 3D (Christen et al. 2016).

2 TEST SITE DEM DATA AND SIMULATION SETUP

To test the influence of the DEM resolution on RAMMS::ROCKFALL simulations, we acquire high spatial resolution digital surface models (DSMs) with a Falcon 8 unmanned aerial system (UAS) over the relatively smooth high-alpine test site Strela, ranging from 2280 to 2637 m a.s.l. At this location, occasional rockfall is observed and different freshly deposited rocks are present. As a second test site we choose the rough rock glacier surface at the Flüela pass, where rockfall occurs frequently. This test site lies between 2400 to 2550 m a.s.l. containing large fields of rough blocks with diameters of 0.10 to 2 m. Both test sites are close to Davos, GR, Switzerland. From the input imagery with an average ground sampling distance (GSD) of 0.02 m we calculate DSMs with 0.10 m spatial resolution applying structure from motion photogrammetry (Koenderinkand A. van Doorn, 1991). The DSMs are geo-referenced applying 10 artificial reference points localized with differential global navigation satellite system (GNSS) measurements. We also applied this technology for high precision snow depth mapping (Bühler et al. 2016). The achieved positioning accuracy is better than 0.03 m. This source DSM is resampled to 0.25, 0.50, 1.00, 2.00, 5.00 and 10.00 m, applying cubic convolution resampling. On these DSMs we perform rockfall simulations with RAMMS using 0.10, 0.50 and 1.00 m^3 rocks of equant, flat and long shapes.

3 RESULTS

The first test site at Strela is relatively smooth, the runout distances and impact energies are quite stable between the different DEM resolutions. We find a different picture at the rough test site Flüela. Fine DEM resolutions (0.10, 0.25 and 0.50 m) decelerate and stop the rocks earlier than the coarser DEMs (1 - 10 m). The mean kinetic energies of the simulated rocks increase by nearly 100% from the 0.10 m to the 10 m simulation (Fig. 1). This result can be explained by the meso-scale roughness. This roughness in the range of 0.10 to 0.50 m, which is caused by single rocks or other small-scale terrain features, has a high impact on the trajectories of rocks smaller or equal to this rock size. At the test site Flüela this roughness is prevailing while it is not at the test site Strela. These results illustrate the role of DEM resolution for rockfall simulation results.

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4 CONCLUSION

New physics based, three-dimensional rockfall simulation tools such as RAMMS::ROCKFALL are able to account for detailed rock shapes and slippage of the rocks at the ground. However, the resolution of the three-dimensional terrain has a big impact on the results, in particular within areas with a high meso-scale roughness. Single rocks or small terraces at the terrain surface can cause this meso-scale roughness. In smooth areas, we do not find a significant influence of the DEM resolution on the results. Therefore a roughness factor could be introduced to address the meso-scale roughness if rough terrain is present and coarser DEM resolutions are used for rockfall simulations. Such a roughness factor could be calibrated using very high resolution DEM acquired by unmanned aerial systems (UAS) at different test-sites.

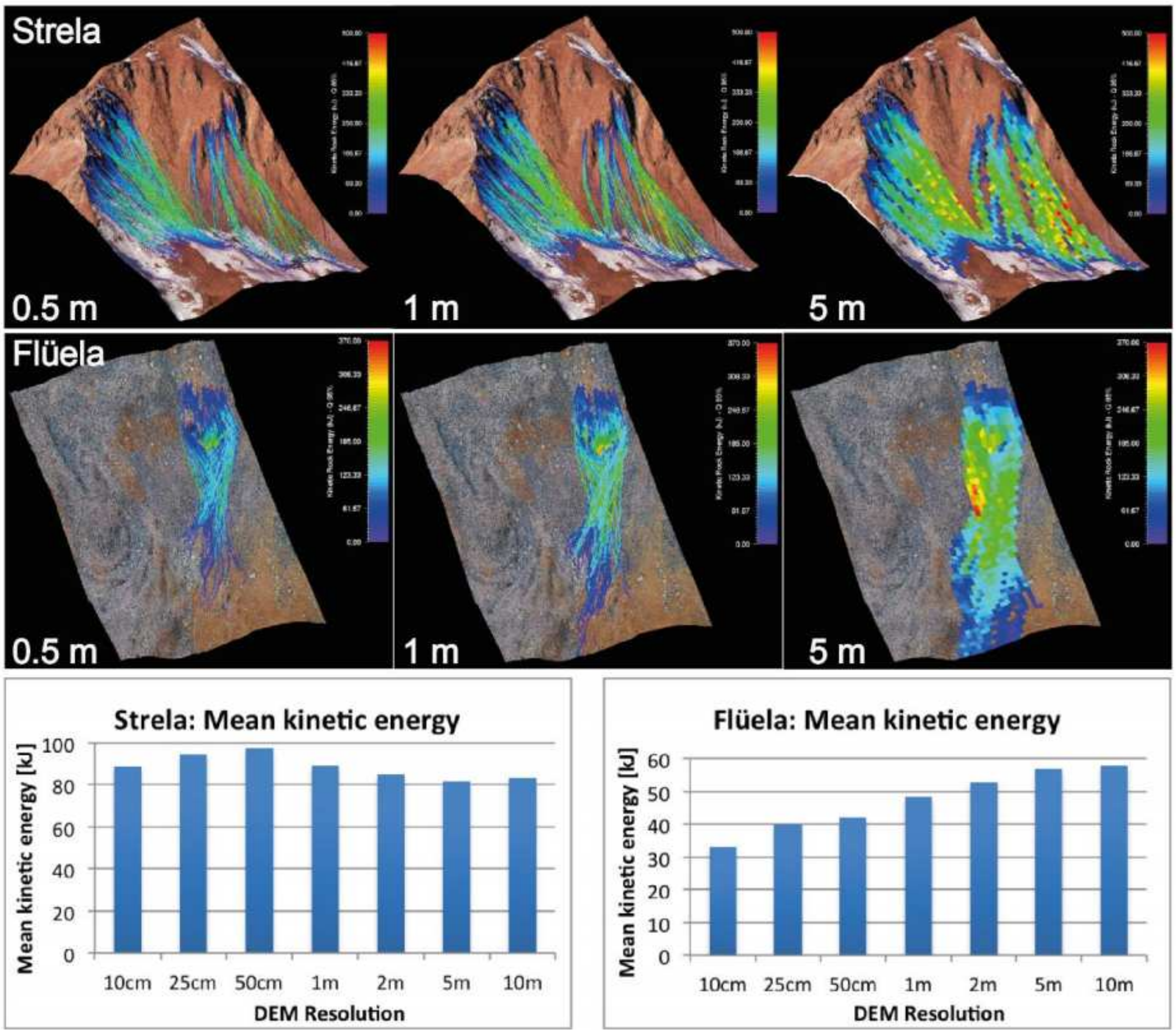

Fig. 1: Simulation results of the 0.5 m^3 rocks at the smooth Strela test site (top) and the rough Flüela test site (middle). Mean kinetic rock energies for the different investigated DEM resolutions (bottom).

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Real case of rock avalanches modelled by discrete element method

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Keywords: **discrete element method, numerical modelling, natural hazards, rock-falls, real events**

Understanding and predicting rock falls and rock avalanches are key elements in risk management when developing mountainous areas. Due to the complexity of the mechanisms involved, developments of numerical and operational tools are useful and necessary to properly estimate block trajectories and define riskiest areas.

1 NUMERICAL MODEL

The numerical approach proposed is based on the discrete element method, which allows simulating the collective behaviour of a group of rocks by using realistic block geometries and three-dimensional slope and cliff topographies (Richefeu 2012, Mollon 2012, Cuervo 2014). The numerical contact model handles interaction between two blocks and between the blocks and the natural terrain by using a limited number of parameters that can be estimated by in situ measurements or feed-back analysis of ancient rock-falls events. The most influential parameters in the motion of the blocks are: the geometry and shape of blocks and topography, the contact parameters such as the normal restitution coefficient which acts in the normal direction to the contact, the friction coefficient acting in the tangential direction to the contact surface and the rolling resistance coefficient of blocks with the slope that somehow accounts for the impact mechanisms in case of soft soil as on Neron's event (Bottelin 2014).

2 METHODOLOGY

A specific procedure allowing the modelling of the whole unstable volume involved in a rock fall is presented through a real application by using three-dimensional terrain model as input data. The parameters analysed concerned mainly the propagation distances and energy dissipation mechanisms, deposit zones, interaction forces and impact energies on protective structures. We present here the results of Perrière (38) site, located next to the city of Grenoble in France. The site consists in an almost vertical rocky cliff of 60 m high of Paleocene limestone layers of 30 cm thick approximately that exhibits currents signs of instability, with frequent rock falls. Some residential buildings and roads are located next to the cliff and increase the potential risk of this region (Figure 1, left). A protective structure was designed by the "energy line method" and built in 2007 in order to protect the near residents of small rock fall events (< 200 m^3). The wall is located approximately at 30 m from the cliff and the exposed face is 3 m height. Significant cliff displacements were recorded by some sensors situated on the top of the cliff that suggests a possible large event making necessary a re-design study of the structure. A geological survey was carried on in 2013 and a potential instable rock mass was noticeably observed. It consists in a 20 m high and 40 m wedge limited by two important fractures planes striking 1°N/66° and 105°N/76° respectively. As a result, a potential unstable volume was estimated to be near 2 500 m^3.

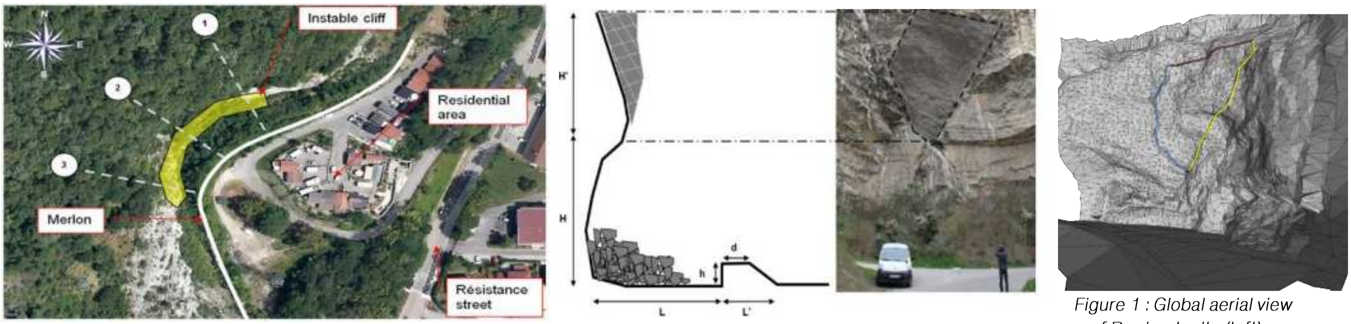

Figure 1 : Global aerial view of Perriere's site (left), a view from the ground (centre), and numerical mesh (right). dimension of the protective dam at profil 1 are L = 27 m, L' = 9.6 m, H = 37 m, H' = 18 m, h = 3.1 m, and d = 3 m.

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3 RESULTS AND DISCUSSION

For the numerical simulation, the discrete element method code DEMbox was used (Cuervos 2015). The numerical contact parameters are (1) a 10% energy restitution at normal collision, (2) a frictional coefficient equals to tg μ=1.1 taking account of friction and abutment, and (3) both normal and tangential stiffnesses of 2.10^{10} N/m, whether they relate to block/block, block/protective structure or block/natural terrain. To get a better idea of the interest of the protective structure and its actual ability to contain a mass movement, numerical simulations were made with and without the protective structure (Figure 2, left and right respectively). As expected, without the protective wall, we get a more spread deposit, with some isolated blocks browsing much greater distances. Only 8 blocks have passed over the structure (Figure 2, left) while more than 20 blocks have crossed the same region if the protective wall is absent (Figure 2, right).

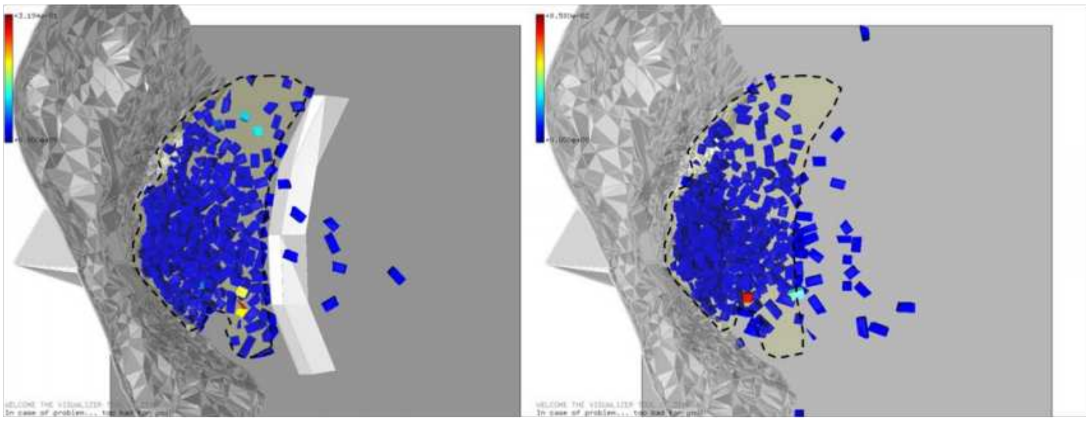

Figure 2. DEM simulation of two configurations of the site at end of the fall: (left) with protective dam (as designed by means of the energy line in 2007), and (right) with no protective dam .

The method of the energy line may be effective, despites its simplicity, for this type of phenomena, since we observe that the position of the actual wall corresponds more or less to the position of the main deposit front (deposit containing more than 90% of blocks) when the protective wall is not there. However, power line method does not allow predicting the presence and position of isolated blocks that are detached from the main mass due to specific kinematic phenomena (such as the transformation of rotation into translation velocities after a collision). Moreover, it was demonstrated that the maximum velocity of some blocks inside a mass might reach larger ranges (23 m/s in translation and up to 10 rad/s in rotation) than in classical trajectory studies (13 m/s, no rotation taken into account). The impact energy of the granular mass on different points of the reinforced structure was compared with the impact energies of isolated blocks, showing a clear role of the multiple interactions of the block within the mass, and the consequences on the kinematics and the total impact energy.

4 CONCLUSION

Field results of an unstable rocky cliff are presented. Comparing results with and without protective structure, and with granular mass or isolated blocks, highlights the mechanisms occurring during the rock fall and when the mass interacts with a structure. These two last points may be of primary importance for the design and the survey of this kind of structure.

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The RAMMS::ROCKFALL MODEL

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Keywords: Rockfall, non-smooth contact, rigid body, RAMMS, rock shape, rock orientation, ground friction

The RAMMS (RApid Mass Movements Simulation) software contains simulation modules for fast moving gravitationally driven mass movements such as snow avalanches, debris flows and landslides. The primary application is to help engineers prepare hazard maps and design mitigation structures. In 2015 a new rockfall module was introduced into the RAMMS software system. This model uses many features of the RAMMS user-interface including easy specification and visualization of three-dimensional digital terrain models (DTM) that can be combined with geo-referenced maps and aerial photographs. Many GIS-like functions have been introduced into the software system, facilitating a quick definition of hazard scenarios and the visualization of simulation results without using other GIS software. However, the primary difference between RAMMS::ROCKFALL and existing rockfall models is the use of non-smooth rigid body contact impact methods to describe the rock/ground interaction.

1 RAMMS::ROCKFALL MODEL

A significant problem with existing rockfall models is that the rock-ground interaction is based on simple lumped-mass *rebound* physics or empirical *shadow angle* type methods (Dorren, 2003; Volkwein et al., 2011). Furthermore, they often use only simplified geometries (spheres, ellipsoids) and stochastic methods to describe the variability of the rock-ground interaction. The implicit assumption of rebound physics, however, is that rocks jump, *a-priori*. Coefficients of restitution are used to define the relationship between the incoming rock velocity and the post-impact velocity vector. They are defined with respect to the translational velocity/energy only and are independent of the impact configuration, which clearly depends on the rock shape and rock orientation at the time of impact (Glover, 2014).
To overcome the problems associated with rebound models, the RAMMS::ROCKFALL module employs newly developed, non-smooth rigid body contact impact methods to treat the rock/ground or rock/tree interaction (Leine, 2014; Schweizer, 2015).
The application of non-smooth methods allows the simulation of:

- **Rock shape, size and impact configuration.** Rocks are described by point clouds that represent the complex surface topology. Rocks can therefore have different forms, varying from equant to plate like. Rock size can vary from 0.05m^3 (rockfall) up to 100 m^3 (blockfall). Impact forces are applied on the rock surface, depending on the impact configuration, leading to rock rotations and a realistic modelling of lateral dispersion. Rock shapes can be specified using the Rock Builder (Fig. 1)
- **Rock rotations, quaternions and gyroscopic forces.** Rock rotations are considered not only in the in-flight phase, but also during the contact/impact phase. Thus, rolling and sliding can be modeled. Computationally efficient quaternion algebra is used to define the orientation of the rock. Gyroscopic forces resulting from the rock rotations can upright plate-like rock shapes producing dangerous wheel-type trajectories.
- **Set-valued contact laws with friction.** Set-valued force laws are used to describe stick-slip type phenomena. They allow sliding with friction. These laws are essential to initiate rock-jumping and therefore long rock runout. Set-valued force laws are non-stochastic and therefore can be transferred to similar terrain. The contact laws are not defined with respect to the rock's centre-of-mass, rather with respect to the complex surface geometry of the rock. Contact forces, including friction, are therefore introduced at the surface of the rock and cause rock rotations. The magnitude of the rotation is therefore dependent on the orientation of the rock when it hits the ground. A restitution coefficient ε is applied at the rock surface to account for energy dissipation in the normal direction. The model uses $\varepsilon = 0$ (fully plastic) for most terrain types, especially soft soils. Bounce heights are purely a function of the impact orientation and the friction in the tangential direction. Soft ground allows more sliding and therefore true rock stopping without ad-hoc conditions.

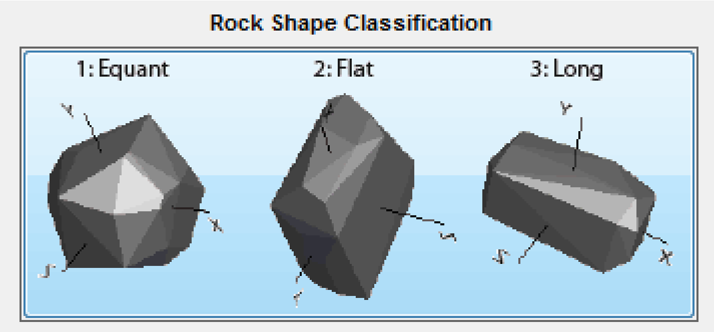

Figure 1: Example rock shapes from the RAMMS Rock Builder. Currently, more than 20 predefined rock shapes are available from the integrated rock library in RAMMS, including laser scans of real rocks. Users can upload their own point cloud files.

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- **Trajectory and statistical results.** RAMMS::Rockfall allows the definition of random release orientations to describe the variability of release. Ground parameters are completely deterministic and depend on the hardness and roughness of the terrain. Simulation results can be reproduced, an important aspect in the analysis of thousands of possible rockfall trajectories. The trajectories can be analysed to find the statistical distributions of velocity, rotations, jump heights and kinetic energies (Fig. 2). Individual trajectories can be animated to depict exact contact points with detailed information on the entire rock kinematics, including rock orientation, rotational speed and rebound heights (Fig. 3).

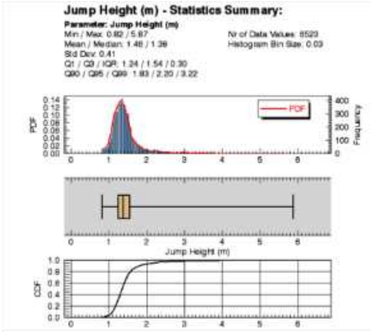

Figure 2: Example statistics plot of 100 RAMMS simulations, showing jump height distributions.

2 EXAMPLE

In November 2014, a large block (2x2.2x2.4m) released in a steep slope (45°) near the highway A13 in Grisons, Switzerland and came to rest below the highway (Fig. 2, left). For the simulation below we used a similar, laser scanned rock with a volume of 5m^3 (Fig. 2, lower right). Medium-Soft (Forest) and Hard (Road) ground parameters were used to back calculate this event. Jump heights of 2.5 m, velocities of 12.7 m/s and energies of 1250 kJ were reached (Fig. 2, upper right).

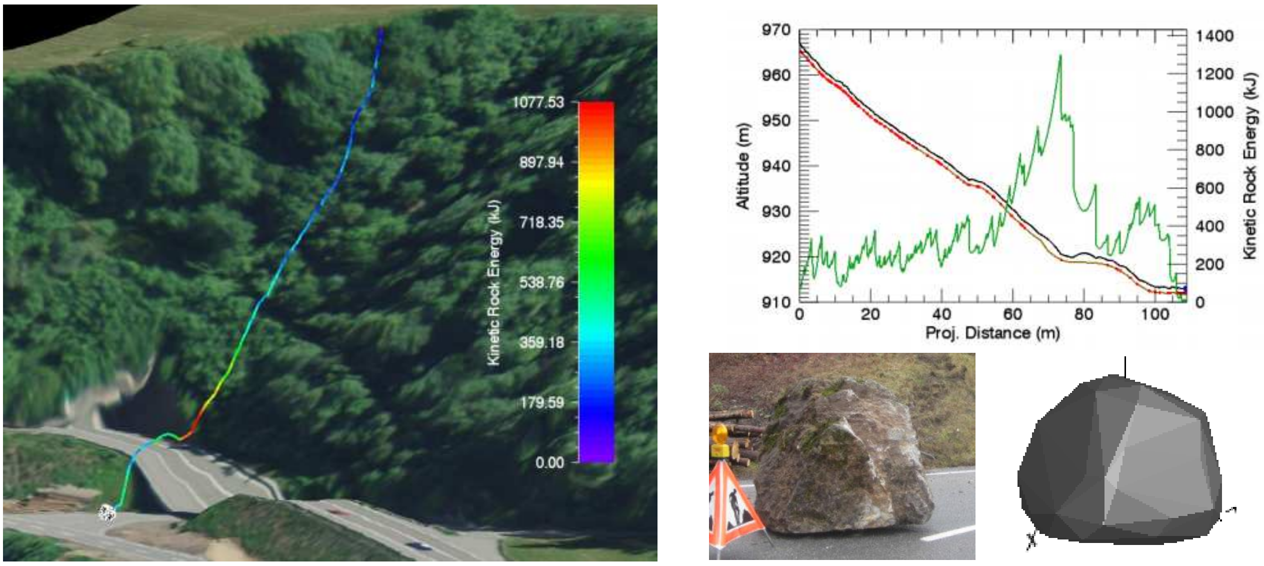

Figure 2: Left: Trajectory of back calculated rock; Upper right: Trajectory XY-Plot depicting profile and kinetic energy data; Lower right: Real rock and simulated rock (convex hull of point cloud data).

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Analysis of rock block fragmentation by means of real-scale tests

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Keywords: rockfall test, fragmentation, quarry

Rockfalls are frequent instability processes in road cuts, open pit mines and quarries, steep slopes and cliffs (Cruden & Varnes, 1996). Quite often the detached rock masses become fragmented after the first impacts on the ground. Knowing the size and trajectory of the blocks resulting from the fragmentation is critical in determining the impact energies on the exposed elements and protection structures (Jaboyedoff et al. 2005; Corominas et al. 2012). In this work we present the results of two real scale rock fall tests carried out as part of the activities of the research project Rockrisk (rockrisk.upc.edu/en). The aim of the project is to quantify the risk induced by rockfalls, improving the tools to prevent, to protect and to mitigate its effects.

1 REAL-SCALE ROCKFALL TEST DESIGN

Two real scale rock fall tests were carried out in a limestone quarry located at Vallirana (Barcelona, Spain). Test site 1 (TS#1) was a single benched slope (Figure 1, left) while test site 2 (TS#2) was a three benched slope. The total fall height, including the bulldozer blade height, was 16.5 m for TS#1 and 27.5 m for TS#2. A total of 56 blocks with volumes ranging between 0.2 and 4.8 m^3 were released (30 for TS#1 and 26 for TS#2). The block size and the strength of rock surface (Schmidt L-hammer rebound) were measured before the tests. A 'circular photogrammetric survey' was realized block per block which allowed to build 3D models of each block for precise volume and center of gravity measurements (Figure 1, center).

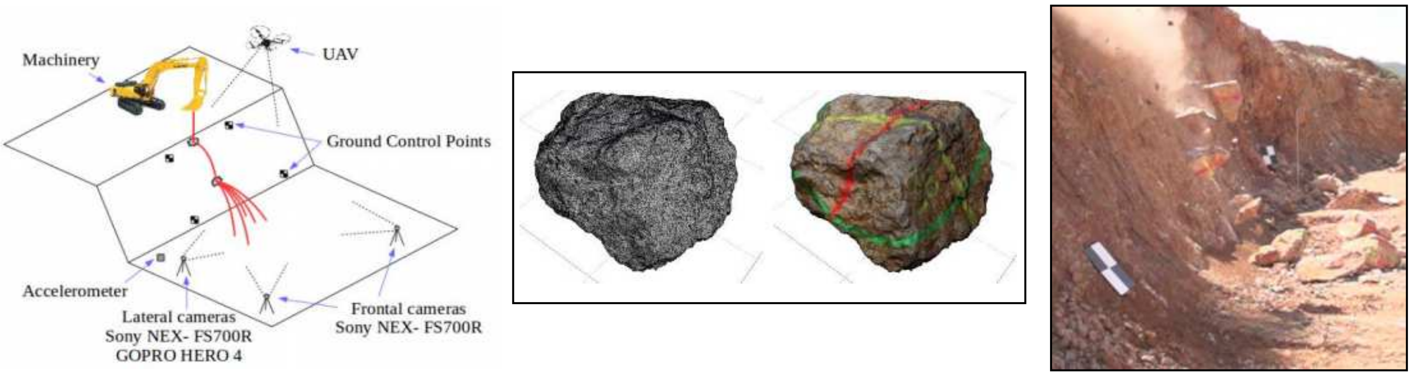

Figure 1: Left: TS#1 monitoring scheme; Center: 3d model of a block; Right: Frame of TS#1 block #19

The tests were recorded using three High-Speed (HS) and High-Definition cameras Sony NEX-FS700R at 400fps and two GOPRO Hero4 for a general view of the scene. Six Ground Control Points (GCP) where placed on the slopes to georeference the images while the location of the rock fragments was recorded using Unmanned Aerial Vehicles (UAV). An accelerometer was also installed to capture the seismic signal induced by the impacts. Figure 1 left, shows the location of the sensors for TS#1. The HS cameras were strategically placed to provide a three-dimensional multi-view (with one extra camera). The shots were synchronized using a flashlight visible from all cameras. The high frame rate allows visualizing precisely how a block is fragmented (Figure 1, right).

2 TESTS RESULTS

The kinematic reconstruction of each rock release has been made by video-triangulating the approximate position of the center of gravity of the block using a code developed by us. The velocity and the energy at the impact point have been computed using the trajectories obtained with this methodology. 19 over 30 blocks and 21 over 26 blocks broke at TS#1 and 2 respectively, generating a Block Size Distribution (BSD) of fragments. In Figure 2 we plot the original block volumes and the BSD for all the fragments generated in TS#1 and 2.

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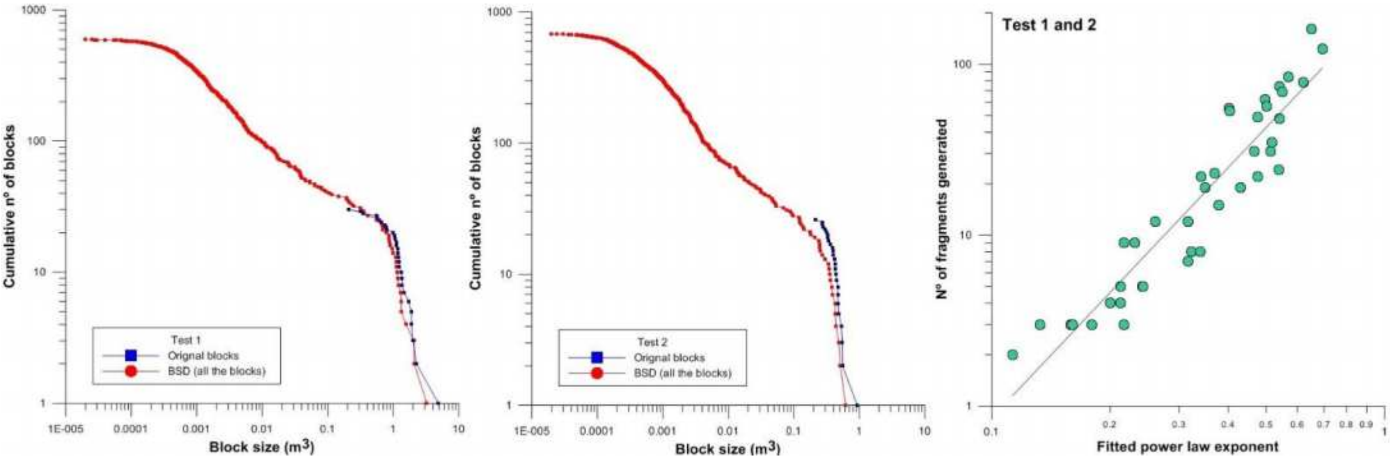

Figure 2: Original Block Volumes and the Block Size Distribution (Left: TS#1: Center: TS#2); Right: Nº of fragments generated versus the exponent of the fitted power law for each fragmented block

The BSD generated from each broken block may be well fitted by a power law. It shows a rollover effect under 10^{-4} m^3. The total volume of the generated fragments under 0.2 m^3 (the smallest block tested) represents the 33% of the total volume of the blocks. The exponent of the fitted power law at each BSD increases with the total number of fragments generated (Figure 2, right). Using the high speed videos, the velocity of each block before the impact was measured, allowing the calculation of the kinetic energies, which range between 100 and 600 KJ. No meaningful relation was observed between the kinetic energy or the Schmidt Hammer values and the number of fragments generated (Figure 3). These results suggest that other factors such as the presence of fissures in the blocks or the impact location must be taken into account. The maximum run-out distance measured from the bottom of the wall was 20 m and the maximum dispersion angle measured from the impact point was 120º.

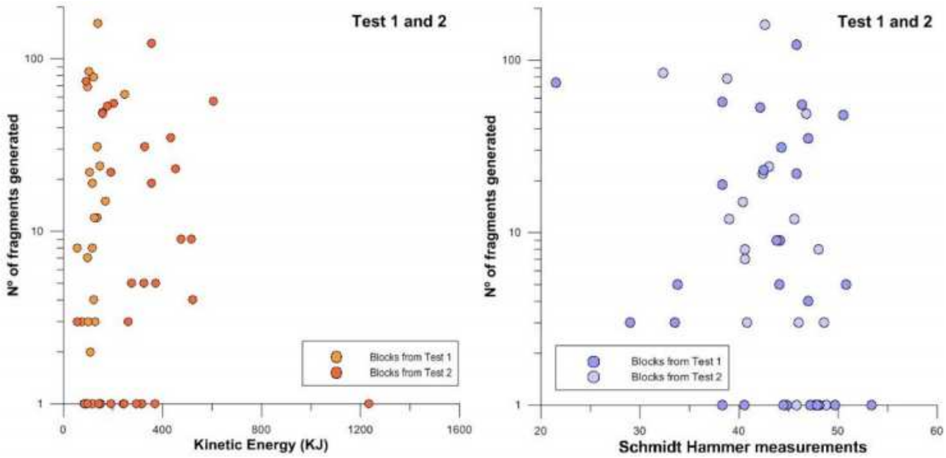

Figure 3: Nº of fragments generated versus Kinetic Energy (Left) and versus Mean value of Schmidt Hammer measures (Right)

3 CONCLUDING REMARKS

The results indicate that the size distribution of rockfall fragments can be expressed by power laws, which exponents are indicators of the degree of fragmentation. The fragmentation of the blocks induces a higher dispersion angle. Although an energy threshold could be expected for the breakage of the blocks, the kinetic energies measured and the Schmidt hammer values shows poor correlations with the number of fragments generated.

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Session 6A

Rockfall analysis (3)

Topographic surveillance of a rockfall with a 3D Terrestrial Laser Scanner

Jean-Philippe JARRIN[1], Camille HALBWACHS[2]

Keywords: rockfall, surveillance, laser scanner

On 1 May 2015, heavy rains affected the Haute-Savoie County in France, with 170 mm measured during 24h in Les Gets. It caused many landslides and rockfalls which mainly affected roads.
The site of La Ravine is known since 1995 for its recurrent rock instability, so that facilities have been builded, in order to protect the departmental road 328: wire meshes, storage trench behind the upstream retaining wall.

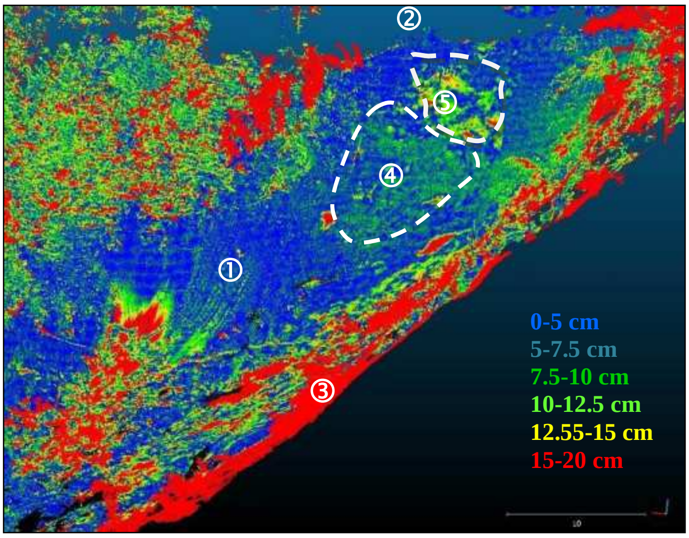
1 DESCRIPTION OF THE EVENT

Figure 1: Sight of the rockfall – 2 May 2015

The rockfall happened in 4 times within 5 days, mobilizing a total volume of 2 000 m^3. The protecting facilities were very efficient, and almost all the materials remained stored above the road (figure 1). Only a few materials but a lot of water exceeded the protections, which clogged the road. The circulation on the road has been cut immediately by the road administrator. During the first few days, earth-moving works of the crumbled materials started, in order to repair the protecting facilities. The workers were protected by a look-out. Then, in view of the number and the size of the rockfalls, came a double question of how to insure the safety of the workers, and how to reopen the road in good conditions. So, it was decided, in agreement with the road administrator, to set up a topographic surveillance, with the aim of detecting a movement in the slope.

2 DESCRIPTION OF THE TOPOGRAPHIC SURVEILLANCE

Data acquisition was carried out using a laser scanner Riegl VZ-400 coupled with a camera Nikon D800. Three measuring points were positioned: on the road, on the left bank of the rockfall, and above the rockfall scar. The measurements were carried out, with an interval of a few days. The point clouds were compared with the software CloudCompare.



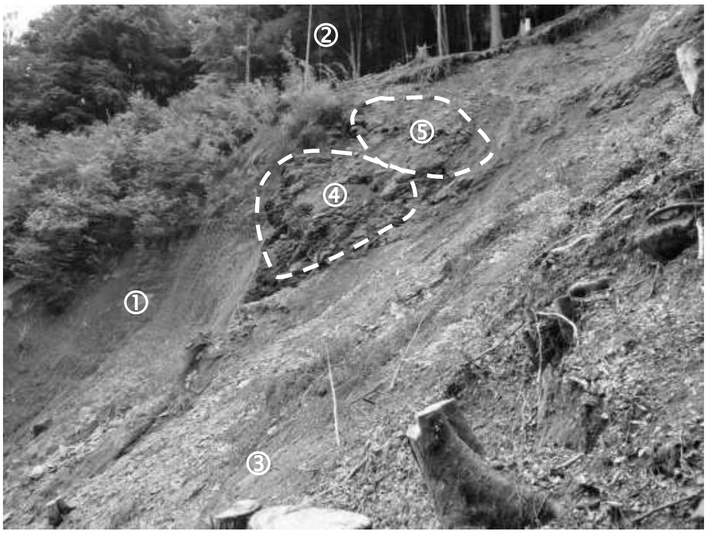


Figure 2: Point clouds comparison between 5/7/2015 and 5/19/2015
① wire mesh - ② top of the rockfall scar - ③ scree - ④ moving area (7.5-10 cm) - ⑤ moving area (> 10 cm)

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Quickly, a moving area was detected, with high velocities: about 10 cm in 10 days (see ④ and ⑤ in figure 2).
Note: the red areas in the figure 2 (movements up to 15 cm) match either bushes, little volume rockfalls (less than $0.5m^3$), or scree creeping amplified by the earth-moving works of the crumbled materials.

The volume of the moving area was about 400-600 m^3. These movements match decompression cracks with a gap about 10 cm noticed in situ. A new rockfall was highly probable within a few days. This did not allow to continue the clearing works and to reopen the road in good safety conditions. It was then decided to mine this zone (figure 3).
The mining was very successful, as about 1 000 m^3 went down, half during the mining, and half a few hours after.

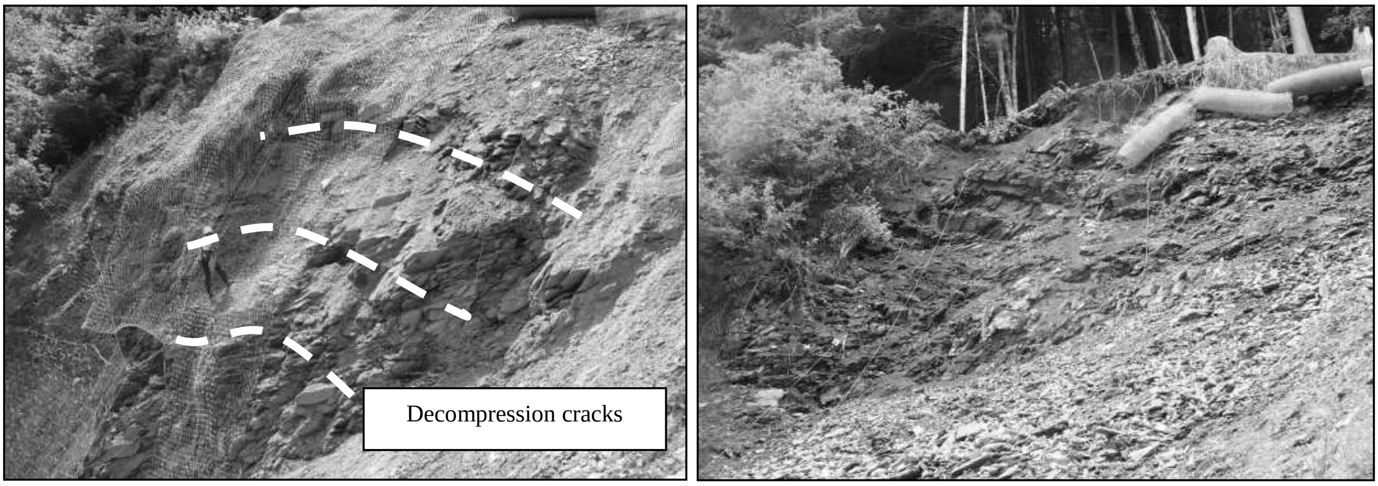

Figure 3: Sight of the site before and after the mining

After this mining, the topographic surveillance started again (figure 4). It revealed that within 12 days, the rock slope moved less than 5cm (see ④ in figure 4), except local rockfalls (volume lower than 0.2 m^3), and a little zone of about 10-20 m^3 with a 10cm movement (see ⑤ in figure 4). This zone was probably disrupted by the manual earth-moving works at the top of the rockfall scar, and no movement was measured between the last 2 measures. This allowed to finish the works of repairing the protecting facilities, and to reopen the road.
Note: the red areas in the figure 4 (movements up to 15 cm) match either little volume rockfalls, scree creeping or manual earth-moving works at the top of the rockfall scar.

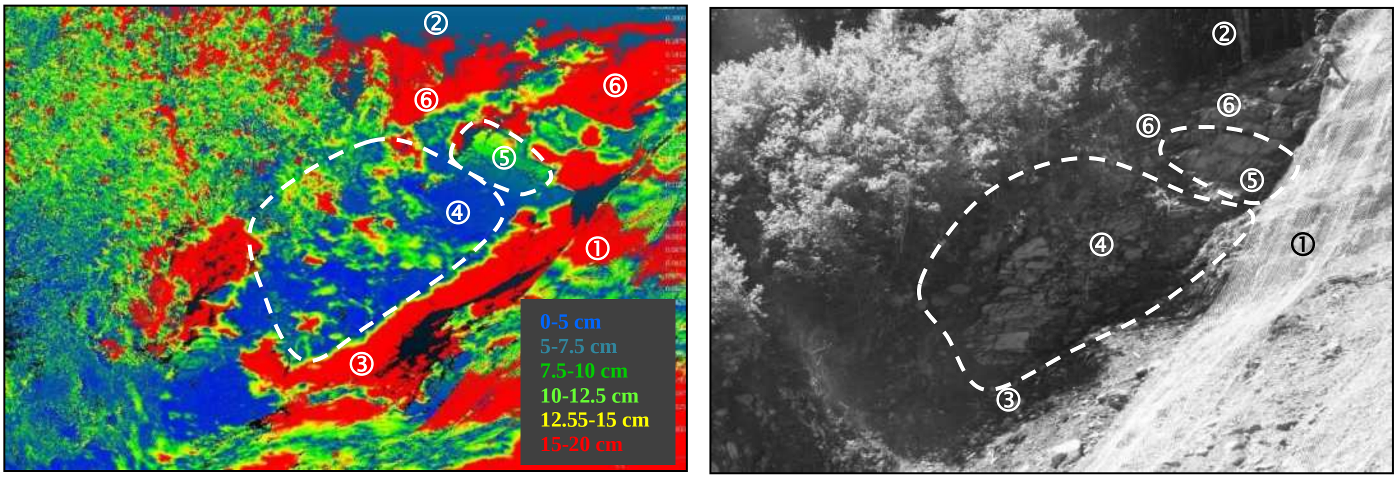

Figure 4: Point clouds comparison between 6/12/2015 and 6/24/2015
① wire mesh - ② top of the rockfall scar - ③ scree creeping - ④ low movement area (< 5 cm) except local rockfalls (red points) - ⑤ moving area (~ 10 cm) – ⑥ slope moving back due to the manual earth-moving works at the top of the rockfall scar

3 CONCLUSION

The topographic surveillance by Laser Scanner was very efficient for this case, as it allowed to predict an imminent rockfall of several hundreds of m^3. It allowed too to carry on securing works and to open the road in good safety conditions, as it was known that the slope movements were slow and limited.
Today, the slope is surveyed with topographic reflectors, because the Laser Scanner cannot cross the wire mesh. Unfortunately, this not allows to get global data, but only isolated data.
The Haute-Savoie County envisage building a protection gallery in the next years.

Rockfall frequency in different geomorphological conditions

Didier HANTZ[1], Thomas DEWEZ[2], Clara LEVY[2], Antoine GUERIN[3], Michel JABOYEDOFF[3]

Keywords: rockfall, frequency, causal factors

The knowledge of the rockfall frequency in a cliff is needed for quantitative rockfall hazard assessment (Hantz et al., 2016). It can be estimated from historical data bases (e.g. Hantz et al., 2003) or from diachronic comparison of digital cliff topographic models (e.g. Dewez et al. 2013, Guerin et al. 2014). An empirical approach is proposed to be used when historical data bases are largely incomplete (not enough events) and diachronic digital models not available.

1 VOLUME-FREQUENCY RELATION AND EROSION RATE

It is well recognised that the frequency of the rockfalls occurring in a rock wall decreases when the volume considered increases. It has appeared that the best law for describing this decrease is a power law, which can be applied either to the temporal frequency or to the spatial-temporal frequency (number of rockfalls per year and per m^2 of cliff):

$$F = A\,V^{-B} \qquad (1)$$

Where F is the frequency of rockfalls bigger than V, A is the frequency of rockfalls bigger than 1 m^3, and B is the scaling exponent, whose value is usually between 0.4 and 0.8. Note that the relation is bounded to a finite value of V because there is a maximal rockfall volume (V_{max}) which depends on the size of the cliff. This maximal volume is rarely observed because its return period is usually larger than the observation period. Integrating the volume V from 0 to V_{max} (assuming $B < 1$) gives the volumetric retreat rate of the cliff (Hantz et al., 2003):

$$W = V_{max}^{(1-B)}\,A\,/\,(1-B) \qquad (2)$$

If the spatial-temporal frequency is used ($F_{st} = A_{st}\,V^{-B}$), Equation (2) gives the linear retreat rate (or erosion rate) of the cliff ($E = V_{max}^{(1-B)}\,A_{st}\,/\,(1-B)$). As this rate includes the biggest possible volumes, it is usually higher than the rate obtained by summing the observed volumes or measuring the retreat of the cliff crest. V_{max} is difficult to estimate, but one can note that if its uncertainty is a factor 10, the uncertainty on the erosion rate is lower (factor $10^{(1-B)}$).

The retreat rate given by Equation (2) includes very large rockfalls (flow like movements) as well as smaller rockfalls (with negligible interaction between individual particles). As the methods used to simulate the propagation of these phenomena are different, it is relevant to calculate different volumetric retreat rates using this relation (Hantz et al., 2003):

$$W = [V_2^{(1-B)} - V_1^{(1-B)}]\,AB\,/\,(1-B) \qquad (3)$$

Where W is the volumetric retreat rate due to rockfalls whose volume is between V_1 and V_2. A method to estimate the frequency of impacts due to small rockfalls is given by Hantz et al. (2016).

2 ESTIMATING THE ROCKFALL FREQUENCY PARAMETERS

Three parameters are necessary to characterize the rockfall activity of a cliff: V_{max}, B and E or A_{st}. V_{max} and B are geometrical parameters. It is reasonable to assume they depend essentially on the rock mass structure (notably discontinuities extension and spacing). A_{st} (or E) is a temporal parameter ($year^{-1}$) which reflects the activity of the processes leading to rockfalls. The processes leading to landslides (of all types) have been listed by Popescu (1994). They can be divided in 3 groups: geomorphological processes, physical and chemical processes and human processes. Effendiantz et al. (2004) have listed the processes leading to failure in rock walls. The more relevant ones have to be considered for estimating the erosion rate.

Note that the Slope Mass Rating system (Romana, 1985) doesn't seem adequate for estimating the erosion rate, because it doesn't consider most of these processes (except groundwater action). It is useful to predict the stability of a rock cut rather than the evolution of an existing rock slope.

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The rockfall frequency parameters B, A_{st} and E have been determined for different types of rock masses, geomorphological and climatic conditions (Table 1). It seems that B is lower for massive rocks (massive limestone, chalk, gneiss) than for bedded rocks (bedded limestone). This finding is in agreement with the results obtained by Hungr et al. (1999) who concluded that a B value in the order of 0.7 is characteristic in moderately to highly jointed metamorphic, igneous and strong sedimentary rocks and that a lower value, of the order of 0.4, appears appropriate for massive felsic intrusive rocks (which possibly produce a relatively greater proportion of large-magnitude, structurally controlled failures).

The erosion rate (and A_{st}) is strongly influenced by the geomorphological context: Erosion rates of some cm/year to some dm/year are obtained for coastal cliffs submitted to the waves action, while values of some tenth of mm/year to some cm/year are derived for middle mountain cliffs.

Table 1: Rockfall frequency parameters (from Pinard & Isnard, 2013, Guerin et al., 2014, Dewez et al., 2013)

Rock	Massive limestone (Urgonian)	Massive limestone (Urgonian)	Bedded limestone (Valaginian)	Bedded limestone (Sequanian)	Massive chalk	Massive gneiss
Site	Isère	Haute-Savoie	Gorgette	Saint-Eynard	Mesnil Val	Venosc
Cliff area (m^2)	$6.1\ 10^7$	$7.8\ 10^7$	$5.1\ 10^4$	$1.3\ 10^5$	$3.7\ 10^4$	$3.5\ 10^5$
Period length (year)	62	22	3.2	3.2	2.3	3.2
Number of rockfalls	87	45	147	344	8567	12
B	0.52	0.60	0.57	0.75	0.54	0.38
Ast ($m^{-2}.year^{-1}$)	$1.4\ 10^{-7}$	$2.2\ 10^{-7}$	$9.6\ 10^{-5}$	$9.8\ 10^{-5}$	$1.1\ 10^{-3}$	$8.4\ 10^{-6}$
Maximal volume (m^3)	10^7	10^7	10^6	10^6	10^5	10^6
Erosion rate ($m^{-1}.year^{-1}$)	0.0007	0.0004	0.085	0.012	0.48	0.07

3 CONCLUSION

Orders of magnitude have been proposed for the rockfall frequency parameters according to the rock mass properties and geomorphological context.

Acknowledgements. The authors thank the national project C2ROP and the Federation VOR for fundings.

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The small rock avalanche of January 9, 2016 from the calcareous NW pillar of the iconic Mont Granier (1933 m a.s.l., French Alps)

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Keywords: small rock avalanche, calcareous French alps, Mont Granier

On January 9, 2016 at 4:57, inhabitants of the municipality of Entremont-le-Vieux were awakened by the sound of a large rock collapse (100,000 m^3) detached from the Mont Granier (1933 m, Savoie, France), iconic mountain of the Chartreuse massif located between Chambery and Grenoble.

1 GEOLOGICAL CONTEXT

The north face of the Mont Granier (1933 m a.s.l.), a 900-m-high natural geological cross section in Urgonian limestone, Hauterivian marls, Valanginian limestone and Berriasian marls, was affected in 1248 by a huge collapse (500 million m^3) that caused hundreds of fatalities (Pachoud, 1991; Fort *et al.*, 2009).
The collapse of 2016 is not comparable to the historical event of 1248, which had reshaped the entire northern side. However, its dimensions make the January 2016 event one of the major events of recent decades in the limestone western Alps.

2 THE EVENT

The NW pillar, shaped in the upper Urgonian limestone, climbed for the first time in 1964 and several times during the warm and dry autumn 2015, collapsed on January 9, 2016, throughout its height (180 m) over a width of up to about 85 m with a volume certainly much higher than 100,000 m^3 (Fig. 1). Blocks rolled down the western slope on about 700 m before stopping in the forest. It is now notched over a hundred meters wide. No infrastructure was affected.
There is now a significant residual risk of falling boulders while a large overhang just formed suggests probable collapses in the short or medium term, with volumes exceeding 10,000 m^3.

Figure 1: The NW pillar of the Mont Granier after the event

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3 CAUSES

The Mont Granier has many predispositions to large instabilities. This is the western remaining part, largely fractured, of a much eroded perched syncline whose inclination is oriented to the east. Fracturing and orientation of the strata contributes to pre-cut limestone and marls. The Granier is also a major karstic area: the Granier plateau with its multilevel karst network (90 km of galleries explored) is a model of polyphased karst network. The presence of big underground chambers nearby the walls is supposed to strengthen the instability of the mountain. But the starting zone of the collapse in the Urgonian part shows no trace of karstification. The karst network also allows the monitoring of some big open vertical fractures cutting the karstic galleries. Untill the January 2016 collapse, no movement was recorded by the crack-meters installed in 1995 close to the north face of the Mont Granier. New measurements are expected after this event. The region is also frequently affected by small earthquakes but seismicity does not appear to be a triggering factor. Conversely, the collapse has produced a seismic signal with an equivalent magnitude M=2.2. Besides rock fatigue related to what has just been mentioned and a vertical or overhanging topography, it is likely that heavy rains at abnormal high altitude of days before the event and after a long period of drought have unleashed the destabilization.

4 CONCLUSION

Events of this size are very rare in the currently available rockfall data series.

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Session 6B

In-situ instrumentation and survey (1)

UAV systems for linear outcrop inspection

Pierre ASSALI[1], Antoine FIVEL[1], Nicolas Pollet[1], Flavien VIGUIER[1]

Keywords: UAV, visual inspection, 3D modeling, structural survey, automatic data processing, protection measures

As it develops quickly over the past years, the use of civil UAVs (Unmanned Aerial Vehicles) offers new prospects for collecting multi-sources data (optical and infrared imagery, LiDAR point clouds, etc.) especially for surveying purposes. Improvements and miniaturization of sensors, combined with post-processing performances, give nowadays the flexibility to exploit these new measurement means and to integrate them into the workflow for the maintenance of the French railway network. The railway asset of earth works and rock outcrops represents a wide application scope.

1 UAV SYSTEMS IN RAILWAY ENVIRONMENT

Rock instabilities located along the national railway network are a real risk for safety and regularity of rail traffic. About 4.000 sites are concerned, representing more than 2.300 km of rock outcrops requiring a specific monitoring and a regular maintenance. As a matter of fact, natural hazards are the second most frequent cause of traffic disruption. Human and social impacts of these incidents highlight thus the need of an efficient management of the rock hazard issue (Pollet et al, 2010) or (Pollet, 2012).

In the broader context of network surveillance, the use of civil UAVs has been experimented for more than 4 years. The available fleet of aircrafts (see examples in Figure 1) and sensors leads to the optimization of survey data used for risk management.

a. b. c.

Figure 11 : Some of the UAVs used by the Drones Department
a. AscTec Falcon 8 ; b. Helipse 190E ; c. DT26 (mainly used for safety checks over large distances)

2 ROCK FACE DIAGNOSIS

Rock mass characterization is obviously a key element in rock-fall hazard analysis. Managing risk and determining the most adapted reinforcement method require a proper understanding of the considered rock mass. One of the major difficulties for the survey of outcrops in railway environment is the difficulty of direct access for a classic walking operator. Even for ground based remote sensing devices, data acquisition can be disturbed by the spatial configuration of the sites (low-angle view, bottom view, vegetation, etc.). UAVs can overcome these limits by embarking sensors for visualization and survey from optimal points of view. Rotary-wing devices are used for this kind of application most of the time.

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2.1 THE EXPERT'S REMOTE EYE

The first concept is simply to use the on-board digital camera as the expert's remote eye. That is, the operator responsible for inspecting the outcrop is able to analyse the rock face thanks to the live video stream (see Figure 2). Digital photos can also be captured at the same time, and used for photogrammetric post-processing helping with the geological and geotechnical diagnosis.

2.2 UAV-BORNE REMOTE SENSING FOR 3D MODELING

UAV-borne laser scanning and photogrammetric techniques - dense image matching - both generate a large amount of 3D digital information that can be used to produce a 3D model of any visible part of a rock face. Such 3D models are then used to enhance the geometrical analysis of the studied outcrops, using fully-automatic (or at least semi-automatic) processes.

2.3 TOWARDS AUTOMATING ROCK MASS STRUCTURAL CHARACTERIZATION

For the needs of geologists in charge of rock-fall risk management along the French railway network, a specific software application has been developed, called Gaia- GeoRoc (Assali, 2014) which includes the following features: advanced visualization of 3D point clouds, semi-automatic extraction of discontinuity sets through direct segmentation, estimation of persistence and spacing parameters, and creation and exploitation of solid-images for reliable analyses, etc. (Assali et al, 2014). Other open-sources packages like the Facets plugin of CloudCompare (CC qFacets, 2015) are also available.

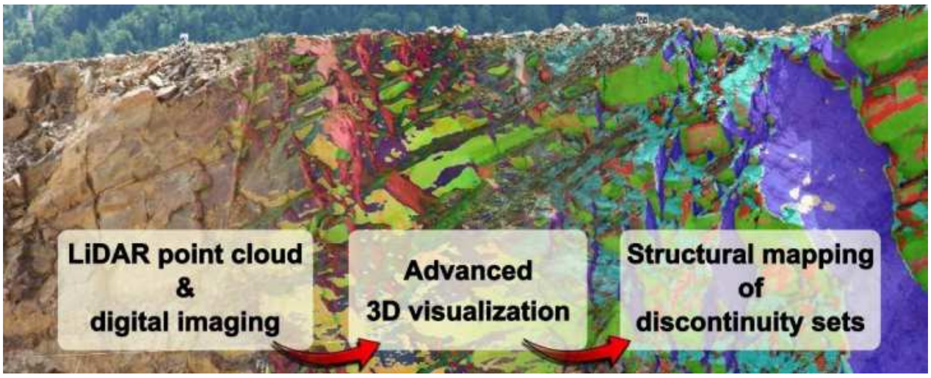

Figure 12 : Automating rock mass characterization using Gaia-GeoRoc or other packages

2.4 MAINTENANCE OF REINFORCEMENT AND PROTECTION MEASURES

As a remote eye solution, UAV-borne sensors are also used to inspect and maintain reinforcement or protection installations. Security structures are checked (nets corrosion, fasteners wear, estimation of blocks amount into ditches, etc.) using this quicker and safer manner compared to classical ground-based inspections.

3 CONCLUSION

This paper reviews the ongoing projects and the various approaches investigated by the Drones Department of SNCF Réseau for the inspection of linear outcrops in railway environment. UAV-borne sensors are used to get geometrical and geotechnical input data, as reliable as possible for the best price, in order to optimize the process of rock-fall risk management. Combining the remote eye concept with further automated processes provide a more complete and more reliable analysis, which can then be used to choose the most appropriate and most cost-effective risk-treatment measures.

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The use of seismic noise for assessing the rock-fall hazard on cliffs

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Keywords seismic noise, spectral analysis, resonant frequency, rock-fall

The seismic noise (ambient vibrations) is characterized by a wide frequency range that allows investigating geological and civil engineering structures in a wide variety of sizes, from the lithospheric slab to the metric or decametric scale of landslides and buildings (Brenguier et al. 2008; Lévy et al. 2010; Clinton et al. 2006; Starr et al. 2015). On unstable slopes, different parameters can be extracted from the spectral analysis of ambient vibrations for investigation or monitoring purposes (Del Gaudio et al. 2013; Burjánek et al. 2010; Bottelin et al. 2013). In stiff rocks, like the limestone making the high cliffs around Grenoble (western Alps), the most common landslide type is rock-fall. The aim of this study is to identify the pertinent seismic parameters able to characterize the decoupling of a prone-to-fall column. 2D Finite Element numerical modelling is first applied to identify the pertinent seismic parameters able to characterize the column decoupling. In a second step, these numerical results are compared to data acquired at one limestone cliff site.

1 NUMERICAL MODELING

This study aims to identify the pertinent and applicable seismic parameters that could be extracted from ambient vibrations in order to gain information on the column geometry and decoupling. We first used 2D FE numerical modeling (Comsol software) to better understand the influence of the unstable column characteristics (width w_1 and depth L_1 of the rear fracture) on the horizontal motion $H(f)$, as well as on the spectral ratios $H(f)/V(f)$ and $H(f)/H_r(f)$, where $H_r(f)$ is the horizontal motion measured at a stable reference site . We identified three seismic parameters able to characterize the column decoupling: the main resonance frequency (f_0 shown in Figure 1a) and the corresponding maximum amplitudes of the spectral ratios ($A_{H/V}$ and $A_{H/H}$ shown in Figures 1b and 1c, respectively). Surprisingly, the spectral amplitudes and the inverse of slenderness (w_1/L_1) exhibit a linear relation (log scale), providing the two relations:

$$\text{Log}\,(AH/V) = -0.901 \log (w1/L1) + 1.473 \qquad (1)$$

$$\text{Log}\,(AH/H) = -0.707 \log (w1/L1) + 2.545 \qquad (2)$$

with correlation coefficients of 0.95 and 0.94, respectively. The two lines with ± one standard deviation interval are drawn in Figure 1b and 1c. These results suggest that the spectral ratio amplitudes are mainly controlled by the w_1/L_1 ratio and that these theoretical relations could then be applied on real data to estimate the slenderness of a potential unstable column.

2 COMPARISON WITH EXPERIMENTAL DATA

These numerical results were compared with the seismic noise parameters measured at a prone-to-fall limestone column (Figure 1d to f). Knowing the column width w_1 and the Young's modulus E obtained by seismic prospecting (4.5 m and 10 GPa, respectively), the variations of f_0 with w_1/L_1 were obtained by modal analysis (Figure 1d). The two measured amplitudes parameters are variable with time (not shown), with significant fluctuations seeming to be controlled by wind conditions. Using the mean parameters values (dark green line), w_1/L_1 values of 0.34, 0.38 and 0.30 were estimated from f_0, $A_{H/V}$ and $A_{H/H}$, respectively, providing L_1 estimations of 13, 12 and 15 m. The three parameters provide similar L_1 values, which are close to the estimation obtained from the outcrop observation (15 m).
The application of these theoretical results must be however tempered by some difficulties. Concerning the amplitude parameters, their high sensitivity to wind conditions (see the variation range shown by the light green area in Figure 1d and 1e) requires a long period of recording and a thorough analysis of the data to estimate correctly their amplitudes. The first resonance frequency f_0 is the more reliable parameter for assessing the fracture depth L_1 but the Young's modulus has to be known (seismic prospecting)

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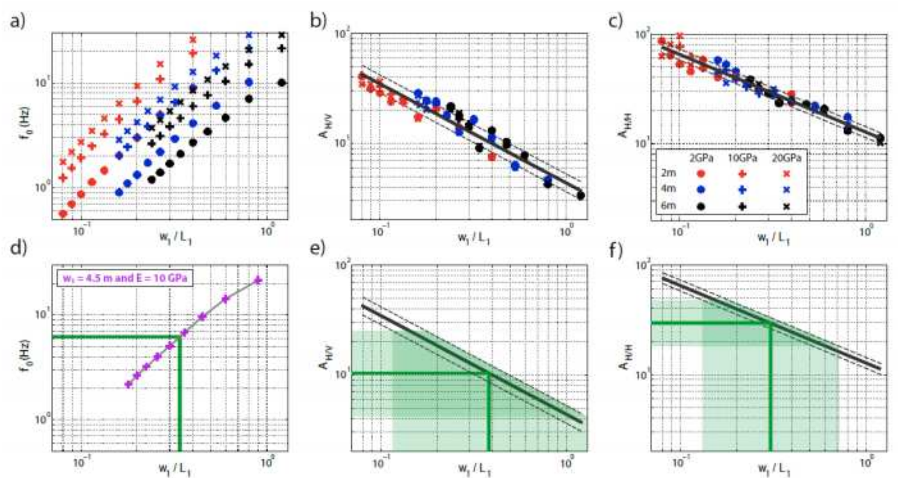

Figure 13

a) to c) Variations of the parameters f_0, $A_{H/V}$ and $A_{H/H}$ as a function of w_1/L_1 for three values of w_1 (2, 4 and 6 m), nine values of L_1 (from 5 m to 25 m) and three values of Young's modulus E (2, 10 and 20 GPa). The legend is shown in Figure 1c. d) to f) Comparison between the three measured parameters (f_0, $A_{H/V}$ and $A_{H/H}$, in green) and the theoretical curves (black) shown in Figure a to c. The measured parameters are given with their variation range (light green area).

3 CONCLUSION

The amplitudes parameters $A_{H/V}$ and $A_{H/H}$ vary linearly (in log scale) with the inverse of slenderness w_1/L_1 of the column. Assessing the thickness w_1 from field observation, the rear fracture depth L_1 can then be directly estimated. However, the amplitude parameters turned out to be sensitive to the environmental conditions, while the resonance frequency f_0 slightly varies with temperature. Despite these limitations, the three parameters gave a consistent estimate of the fracture depth at a limestone site, close to the observation. The parameter f_0 seems to be the most robust to estimate the rear fracture depth, but it necessitates determining the rock Young's modulus. The monitoring of f_0 theoretically allows the progressive decoupling of the prone-to-fall column to be followed. However, reversible thermal variations in f_0 make it difficult the detection of this irreversible damage. An autoregressive model, based on the similarity between the temperature and f_0 curves, was proposed to detect realistic drops of f_0.

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Commune of La Roque Gageac (France)
Works for securing an underground fort

Marc VELU [1], Vincent CHAUTY [2]

Keywords: works, underground fort, collapse, reinforcement, design and built contract, material ropeway

1 CONTRACT

Located in Perigord, La Roque Gageac is one of the most beautiful villages of France.
Since 1920, several massive rock collapses have occurred, causing heavy casualties.
To remedy the hazard, the commune has floated a tender on Design and Build basis with prequalification.

The Employer :		Commune of la Roque Gageac (24)
Consulting Engineers :		ABCS / INTECH
Consortium :	Contractors :	CAN (lead member) / VLM / AMAK
	Engineering :	MECANROC / REULET
	Architect :	DODEMAN

2 INTRODUCTION TO THE WORKS

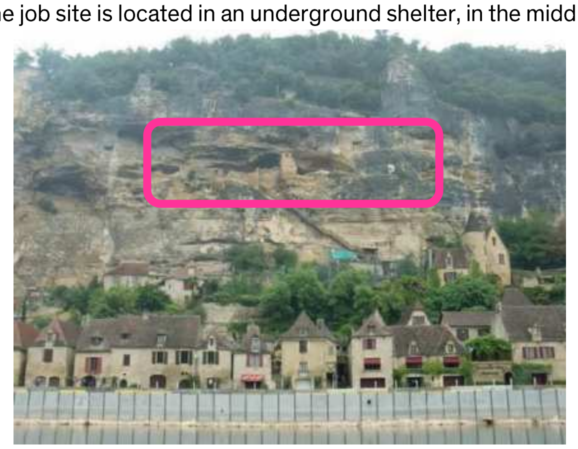
The works are due to repeated events that occurred during the 20th century and early 21st , close to the underground living:
- 1920 – partial collapse of the eastern section of the cave.
- 1957 – partial collapse at the western side of the village
- 1994 – crumbling in the middle of the village au centre du village,
- 2010 – partial collapse of the roof of the underground living cave

Due to the latest collapse in 2010, part of the village has been completely closed to public and villagers have been evacuated. Temporary protection with rockfall barrier was installed.

The works consist in reinforcement of the residual beam of the 2010 collapse, plus the total reinforcement of the roofing of the cave.

3 SITE AND WORKS CONDITIONS

The job site is located in an underground shelter, in the middle of a cliff above the village of La Roque Gageac.



One monument (Manor of Tarde) in La Roque Gageac is listed among the national heritage sites in France. Hence the design had to be approved by the National Heritage Committee. The entire site is listed as a national natural site.
La Roque Gageac is a tourist destination with up to 2 millions visitors every year between June and September.
Regardless of compliance with environmental requirements to preserve natural habitats and species, the contractor had to include in its project, restoring the initial state of all areas in which it intervened

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The works mainly dealt with:

- Installation of means of access and supply, including a material ropeway
- Installation of provisory support
- Fragmentation and removal of rocks that collapsed in 2010: 320 m^3
- Installation of 9 concrete gantries for final supporting
- Partial reinstatement of the fort masonries

4 EXECUTION OF THE WORKS

It is a Design and Build contract. The concept (supporting gantries + access with material ropeway) was already approved through the tendering process.

The first phase of Design included geotechnical investigations, topographic survey, designing of concrete structure, study of the material ropeway, architectural design, environmental impact study and administrative file.

Then, the works began with securing of access: the large rock scales above the cave have been nailed: same size scales have already fallen in 2010. A temporary stairway made of scaffolding has been installed as a collective protective equipment. The existing wooden staircase was not suitable for access to the job site.

Then, the material ropeway (capacity 2,2 tons) has been installed to remove the debris of the old collapsed vault (320m^3) and to supply the works. The rock debris have transported in big bags with the material ropeway. This solution has been chosen rather than a mobile crane (jib 35m minimum) that would have been installed on top of the cliff. It was a solution to minimize the visual impact during the works, to reduce the construction period and to reduce the environmental impact. The top of the cliff is located in a Natura 2000 area which means specific environmental impact assessment and reinstatement if a temporary access trail would have been constructed (700 m long).

Reinforced props were placed under the unstable vault. The blocks that fell from the vault were fragmented with jackhammers and expansion cement. Control of vibrations have been performed to check the possibility to use hydraulic rock breaker without disturbances to the unstable vault. After approval, a mini hydraulic excavator has been transported with the material ropeway for more efficient fragmentation. Disposal of material has been done with rubble chute and the material ropeway.

After clearing of the floor, we proceeded with the construction of the concrete supporting gantries: Ø 600 and 800mm, height from 4 to 8m.

The concept of the concrete gantries was: an era a style. It means the visual impact should be minimum but not fake. Nice with modern style; as designed by Jean Nouvel for the nearby Halles of Sarlat (10kms from La Roque Gageac).

Concrete was mixed on site as it was not possible to use a concrete pump for such a long distance (horizontal > 50 m and vertical rise >40m). Transport of concrete with the material ropeway was not considered. Cardboard formworks have been used. Concrete gantries were covered with coloured plaster

Installation of a material ropeway for removal of debris and supply

Installation of 9 concrete gantries for final supporting, formworks with tubular cardboard - on site concrete mixing

After construction of the concrete gantries, we reinstated the fort masonries with similar techniques as in the ancient times. Then we proceeded with dismantling of the material ropeway, of the access scaffolding staircase and restoring of the site.

The completion period was 6 months, starting from January 2015 until June 2015.

5 CONCLUSION

This case study demonstrates the possibility of using different techniques to remedy geotechnical hazards associated with different equipment to supply the works. This case can provide new ideas to consulting engineers and contractors with projects where historical monuments have to be reinforced or protected against rock massive collapse. This is the case for many medieval castles built above cliffs or caves, or where tunnels have been excavated.

Session 7A

In-situ instrumentation and survey (2)

Morphosense, a new technology to monitor the geometry of critical structures and areas: a rock slope application

Mikael CARMONA[1], Alexandre PALEOLOGUE[2]

Keywords: geometrical monitoring, high resolution, sensor network

Based on an innovative MEMS (*Micro Electro-Mechanical Systems*) technology, this system enables real-time assessment of 3D geometry and vibrations of critical structures (*such as bridges, dams, and buildings*) as well as zones (*as a rock slopes*).

1 ORIGIN AND DESCRIPTION OF THE TECHNOLOGY

CEA-Leti is a provider of R&D solutions related to micro and nano technologies, including their development and integration into systems dedicated to innovative applications. In the context of the monitoring of structures and critical zones (*as rock slope*), CEA-Leti as developed a high-performance system based on MEMS-accelerometers, and called Morphosense, to assess the geometry (*3D-deflection, tilt and curvature*) and vibrations (*event detection, modal and spectral analysis*) of a zone or a protective structure which needs to be monitored in order to manage the risk (*anticipation of rock fall*). The principle is based on a distribution of measurement nodes including MEMS-accelerometers on the critical area. During operation, the system provides real-time synchronous data of the 3D geometric deformation and vibrations of the protective structure or risk zones to which it is attached. Acquired data are transmitted with a wireless protocol adapted to the configuration (*3G, 4G, WiFi, LoRa, SigFox*) to a distant database in which dedicated algorithms are applied and alerts are generated if specified threshold are overpassed.

The figure bellow presents – at the top – two demonstrators dedicated to 1D [1, 2] and 2D [1, 3] geometry retrieval, respectively. A description of an industrial demonstrator of such technology is shown in figure [4] related to the context of flexible riser monitoring. At the bottom of the figure, the capabilities of the system to acquire vibrations (*and related relevant information as event/chocks detection, modal parameters and ambient frequencies*) of a critical structure or zone are highlighted with an instrumentation of an emblematic structure in the microelectronic world [5, 6]. The system is “flexible” as the number of sensors and their position can be dimensioned to cover wide linear line (*more than 100m*) or 2D areas (*more than 100 m x 100 m*) and with a high density (*mode than 100 nodes if necessary*). At the opposite of optical fiber-based technologies [7], which are the most relevant concurrent technologies, the MEMS-based monitoring technique is easier to install, more robust with respect to environment conditions, and have high performances with respect to time and temperature stability. The resolution of the system is 0.1 mrad meaning that any event generating a deformation of, for example 1 mm over 10 m (*or 10 mm over 100 m, and so on*), will be detected.

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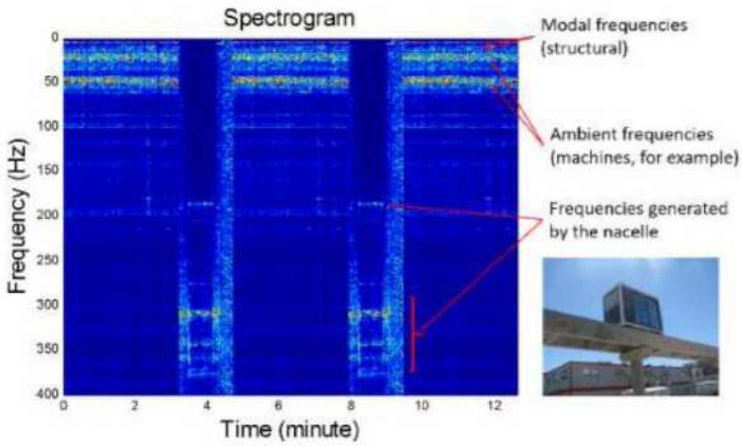

2 CONCLUSION

With the emergence of MEMS-based solutions and broader implementation of distributed sensor networks, it is now possible to monitor wide critical areas such as rock slopes with a high accuracy and efficiency. The Morphosense system will significantly contribute to optimize the management of emergencies situations classical in the rock slope domain as it provides a continuous and real-time monitoring of rock-fall hazard, cliffs and ongoing safety initiatives.

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Survey optimisation to size reinforcement in Rock-cutting

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Keywords: SNCF, rock-cutting, hidden discontinuities, survey, sizing, optimisation, reinforcement.

In some cases, along the French railway network, surveys are regularly undertaken to size reinforcement in Rock-cutting, based on the geotechnical profession's recommendations (Aftes, 2003). It occurs when the rock mass structure is hidden and the hazards unknown and for high traffic railway lines. Regarding the railway constraints and the limited financial resources, a cost-benefit analysis seems essential to optimize our surveys while keeping a high safety level. Indeed, not all these surveys are used to model mass rock or to size the reinforcement (empiric method or soil mechanic model are preferred strategies). Feedback from historical survey were analysed to improve our methods, illustrated in this paper through three examples.

1 "GRANDE FAUGERETTE" ROCK-CUTTING

The first rock-cutting example is located along the Paris-Toulouse railway built during the nineteenth century. The limestone stratas affected by calcite dissolution (karstic system) are partially covered with brickworks walls.
For many years, some damages have been identified on the brickwork structure, resulting in a hazard for the railway. This critical situation is partially generated by water inflows behind the walls from the karstic network (fig 1.b). They weather the cement joints and increase hydrostatic pressure behind the brickworks.

Figure 14 : a. photogrammetric view ; b. karst behind the walls

It was necessary to undertake a survey to determine a geologic model and then choose the best solution for reinforcement. Several boreholes and geophysical survey have been carried out between 2013 and 2014 (photogrammetry (fig. 1.a), ground penetrating radar, coring, diagraphy, endoscopy and lab tests).
Not all results have been used and the reinforcement was defined with an empirical technique and a simple model based on structural analysis, considering there was no rockslide behind the walls. Indeed, sizing and complex modelling would have been difficult considering the uncertainty of parameters and wall failure propagation (interactions between rubbles, cement joints strength, etc.).

The reinforcement technique finally chosen was rock nailing: hexagonal steel wire mesh with cables and nails.

2 "MORLAIX" ROCK-CUTTING

On the 25th of January 2013, a large rockslide happened in a 22 m high schist rock-cutting and covered the railway line between Paris and Brest, in Morlaix. This cutting presents many joints, foliation and a major discontinuity between two different structural units (Cabanis et al., 1981). 2 days later, only one of the three tracks reopened at low speed, due to remaining risks. This unexpected phenomenon (about 3000 m^3) is explained by a weathered zone, hidden about 5 m behind the facing of the rock-cutting, thus it could not be identified during site visits.

Traffic resumed after two months of surveys (monitoring, topography and 3 boreholes with mechanic tests) and earthworks (soil nailing and nailed wall). A complementary survey was conducted in 2014 (around 20 boreholes and mechanic tests) to verify the design hypotheses (using soil mechanics theory (Philipponnat et al., 2008) for sizing the reinforcement, taking into consideration how weathered the rock was). The results confirmed the hypotheses. After the discovery of a more important

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weathered area than first expected, a reinforced concrete wall with anchorage tie bars was built at the base of the cutting in 2015.

Other surveys (geophysics and boreholes) are planned to verify the presence of similar areas on the whole rock-cutting, which may cause another rockslide.
In November 2015, some geophysical tests were conducted to define the relevant acquisition and processing data protocol.

In this case, we face both practical and technical issues. Firstly we had to cope with difficult access and tight schedule during survey. Then, although weathered rock mass behaves as a soil, the usual soil mechanic survey is at its limits, considering foliation and fracturing.

3 "MERCUES" ROCK-CUTTING

This case concerns a jurassic limestone-clay rock-cutting covered with brickwork wall and shotcrete near Cahors along the Paris-Toulouse railway.
On the May 12th 2012, a derailment was caused by a 60 m^3 rockslide of brickwork and boulder on the railway line.
In order to analyse the surrounding wall stability, ground penetrating radar was used in 2012. First results showed a large anomaly (decompressed zones) behind one of the walls. These zones were reinforced in emergency with hexagonal steel wire mesh and anchors in 2012. Reinforcement was sized from structural analysis and experience of similar cases along the French Railway, without computing (Rapin et al., 2004).
Complementary surveys have been conducted in 2013 (coring, endoscopy …) to better constrain the anomalies, identify the rock quality behind the wall with a view to designing a preventive reinforcement where needed.

In this case, not all the results have been used to size an optimal remedial work keeping in mind the railway constrains. A structural analysis has been preferred to the mechanic soil model suggested by a geotechnical consultant which was less relevant considering survey results.

4 CONCLUSION

These three examples teach us the followings. Firstly, not all the results are used to model the rock mass and to size reinforcement (all cases). Then, despite these surveys, a relevant model can't be built (cases 1 and 3: too many uncertainties; case 2: limits of surveys to characterize soil-rock behaviour). Reinforcement sizing is based on structural analysis and usual cases proved to be efficient. A simple computing approach to verify the sizing gives satisfying results in case 1 (Coulomb earth pressure theory) and 2 (Slope stability). Furthermore, surveys are very expansive as implementation is often very difficult (access and railway constraints).

To conclude, we face two choices: we can conduct a great amount of surveys and models to get optimal and totally-verified reinforcements (which is costly and time consuming) or we can decide to limit surveys and models and design reinforcements using simple models with a high safety margin, approximation and feedback. Making a choice required a case-by-case analysis taking into account issues and economical resources.
Other non-destructive technics could be tested to carry out a more precise assessment. That's why we decided to conduct geophysical research project along the Morlaix rock-cutting, to evaluate if the geophysical methods coupling is relevant for this rock-mechanic topic.

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Development of a wireless sensor network for rock mass deformation monitoring in the Montserrat Massif

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Keywords: contact sensors, jointmetric monitoring, wireless network, ZigBee technology

1 SITUATION AND BACKGROUND

Montserrat Mountain is located near Barcelona in Catalonia, at the north-east corner of Spain, and its massif is formed by conglomerate interleaved by siltstone/sandstone with steep slopes very prone to rock falls. The increasing visitor's number in the monastery area, reaching 2.4 million per year, has pointed out the risk derived from rock falls for this building area and also for the terrestrial accesses, both road and rack railway. A risk mitigation plan is currently being applied for 2014-2016 that contains monitoring testing and implementation as a key point, to be further developed in a second phase. Besides well known cases for large landslides, also for rockfalls, references about premonitory deformations along a certain period before collapse are ever more usual. Considering that the mechanical behaviour of the rock mass in Montserrat seems to be very stiff and apparently brittle, it is expected that the collapse can be reached at low strain. For this reason, high precision measurements and long time series are mandatory. The goal of the monitoring tasks during the current mitigation plan consists in improving the knowledge on rock mass behaviour and the comprehension of instability propagation and failure preparation. Thus, different pilot tests have been performed to check the applicability of several techniques depending on each case and site. Four monitoring techniques were selected with different spatial and temporal continuity (Janeras et al. 2015). By one hand, 3 remote systems are currently applied in collaboration with specialized research teams of UPC (Technical University of Catalonia) and UB (University of Barcelona): Ground-based Synthetic Aperture Radar, Terrestrial Laser Scanner and Total Station. By the other hand, a contact sensors network for big blocks is being developed by ICGC.

2 MONITORING OF ROCK JOINTS

2.1 PREVIOUS WIRED INSTRUMENTATION

In Montserrat, an experience in rock mass auscultation was started on September 2010 to monitor the movement of a huge rock block called *A3-6*. To that end, 3 extensometric sensors (wire and bar type) were installed to measure movement of this mass relative to the massif, together with an air temperature sensor. These sensors were wire-connected to a datalogger provided with a photovoltaic power system and a 3G mobile communications system for remote access. Currently, after five years of operation, system continues providing continuous data demonstrating its feasibility for long term operation, despite some gaps due to technical problems with wildlife interaction. However, a major disadvantage of such auscultation technique is the effort, and the cost, of cabling from sensors to the datalogger and the electricity supply, especially in a mountainous terrain such as Montserrat massif, where this task must be carried out by personnel qualified for rope access works.

2.2 WIRELESS SENSOR NETWORKS (WSN)

In order to reduce the visual impact of wiring the rock walls, a new concept of distributed wireless data acquisition network has been implemented to extend the auscultation strategy to other potentially unstable blocks, following the priorities previously analysed. For instance, new monitoring sensors were installed in a cluster of blocks called *Castell del Diable*, where a large rockfall occurred in XVIth century destroying a dependence of the monastery with 4 fatalities, as it is known from historical documents. Three different types of displacement sensors have been installed: crackmeters, wire extensometers and beam tiltmeters. Nowadays, 5 stations are in operation and other 5 are planned (Figure 1). The acquisition of measurements (reading sensors and sending) is based on ZBLogger nodes. This is a low power, and low cost, system of ICGC own design with 3 single ended input channels (or 1 differential plus 1 single ended) and wireless connection, based on ZigBee protocols (IEEE 802.15.4 standard), which can communicate, directly or through other nodes acting as repeater, with a master or coordinator, which is finally linked to the CS datalogger. Eight LR6 1.5V rechargeable batteries and a solar cell of 0.96 W for each ZBLogger provide the system with enough energy for long term operation (5-10 years,

Figure 1: Monitoring network under development (red: hub station; green: operating station; blue: planned station; purple: repeater)

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Figure 2: WSN in Montserrat: a) installation works; b) ZBLogger for a crackmeter ; c) scheme of the network from sensors to datalogger in the hub station *through ZBLoggers, master node and repeaters*

depending of batteries service life in terms of charging cycles). ZBLoggers digitize signal of input channels, at 1 sample each 10 minutes, with a 16 bits sigmadelta A/D converter. Data from a ZBLogger can be sent directly to a datalogger across the wireless network (800 m maximum in case of direct line of sight), or via others nodes acting as repeaters (Figure 2). Once data are stored inside the datalogger, they are sent to Data Centre by GSM call, and archived into a database. Finally, *NetMon*, a proprietary web based tool developed by ICGC (Figure 3), offers a quick view of available data in real time to be harnessed effectively, and allows a future deployment of some warning system if necessary.

3 DATA ANALYSIS

3.1 SYSTEM PERFORMANCE

A first version of ZBLoggers was tested in the Diable block at the end of 2014. After learning from some hardware problems when installing, a second version was improved for the Berruga and Cisneros stations in 2015. Nowadays, after overcoming an annual cycle under operation, it is being performed a checking of all the dysfunctions and incidences before expanding the network to other 5 blocks, as it is planned according to the set priorities. By filtering the incidents like the fallen down of a repeater, the system shows its feasibility for the designed purpose. However, it should be noted that there is an irreducible loss of data due to the nature of wireless nodes with latency cycles in order to obtain the low power features. For instance, it is seen that a ZBLogger with proper sunstroke that ensures the energy recharge and proper visibility with the master, may be subject to a random data loss of at least 0.1%. In this sense, for such a wireless network, it must be considered an oversampling to obtain perfectly complete series with the desired sample frequency.

3.2 MONITORING RESULTS

The longest register, obtained in A3-6 block, clearly shows the annual cycle and also daily oscillation, as it can be identified with a FFT analysis. This behaviour can be linked with thermal deformation of rock mass superficial part, including the analysed block, but also thermal effect on sensor must be considered. The mean amplitude of annual oscillation is 2.0, 1.0 and 1.3 mm for sensors 1, 2 and 3 respectively, and 36.7°C for temperature. Sensor 3 (bar-extensometer in the sliding direction of the backwards joint) shows a completely elastic behaviour, recovering its original value after the annual cycle. In contrast, sensors S1 and S2 (wire extensometers measuring the aperture of the rear joint at the upper and medium part) have a slight tendency to accumulate displacement at a rate of 0.169 and 0.065 mm/year respectively, corresponding to a toppling movement of 5.1E-5 rad/year. Due to the small scale of the strain, there could be a creeping process of weakening of the limestone level in the base, and a refilling in the joint that prevents its fully elastic recuperation. To determine if this mechanism is leading to a long-term toppling failure, it must be monitored further to allow a second order analysis of a longer time-series by means of acceleration.

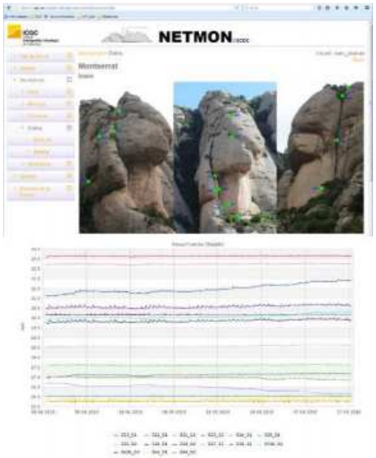

Figure 3: View in NetMon web interface for the proper data management and operational control of the system

4 CONCLUSION

The developed ZBLogger node for a distributed wireless data acquisition network constitutes a very promising system for geotechnical monitoring needs by ICGC. Some improvements are still needed for the long term operation in this highly challenging terrain. It is expected to be a proper technological basis for the auscultation strategy that is being developed by the risk mitigation plan in Montserrat Massif.

5 ACKNOWLEDGEMENTS

All this work would not have been possible without the engagement of the whole lab & field team in ICGC. Also, we appreciate the advices of Josep A. Gili from the Universitat Politècnica de Catalunya (UPC). The risk mitigation plan is promoted by the Patronat de la Muntanya de Montserrat and funded by the Catalan Government.

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Comparison of lasergrammetry and photogrammetry for rock walls diagnosis and monitoring

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Keywords: Lasergrammetry, photogrammetry, monitoring, diagnosis, rockfalls, trajectography, vegetation, risk mitigation

Gravitational instabilities are a major challenge in mountainous regions. Development of new trajectographic technics, lasergrammetric and photogrammetric 3D modeling, plays a significant role for the diagnosis and monitoring of landslides. They allow 3D surface investigation of hazardous areas, based on areas visible by the instruments.

1 DIAGNOSIS AND GRAVITATIONAL INSTABILITIES EXPERTISE:

When rock is not covered by vegetation or protective structures, Lasergrammetry and photogrammetry give a precise 3D Digital Elevation Model (DEM). They also provide a high definition photography, essential to the diagnosis. Nevertheless, most of the studied areas are partially vegetated, or covered with wire mesh. This can moderately or totally block the ground out. These elements must be removed, to do the protection studies.

<u>Elimination of vegetation and existing surface works</u>

The 3D laser scanner Lidar type (TOF) provides more information than photographic sources, thanks to advanced signal processing technics, called « online waveform processing ». Thanks to the rigorous multi-targets detection, this technology allows to extract the "off field points" from the true points corresponding to the field. Playing on the echoes of transmitted signals, enables to select points corresponding for instance to the vegetation or a wire mesh along a rock face.

Figure 1a shows a site where the vegetation totally covers the lower part of the rocky hillside even in winter In this particular sector, the photogrammetric technic does not allow the realization of a digital elevation model of the entire rocky hillside of interest. Figure 2b shows the 3D colorized cloud point obtained by lasergrammetry after the elimination of vegetation. The processing of Lidar data allows the identification and characterization of the existing merlon construction which is completely covered and blocked out by vegetation. The lasergrammetric survey on the site enables the construction of a precise DEM of the whole slope. This DEM has been used for the trajectographic modeling taking into consideration the existing protection infrastructure.

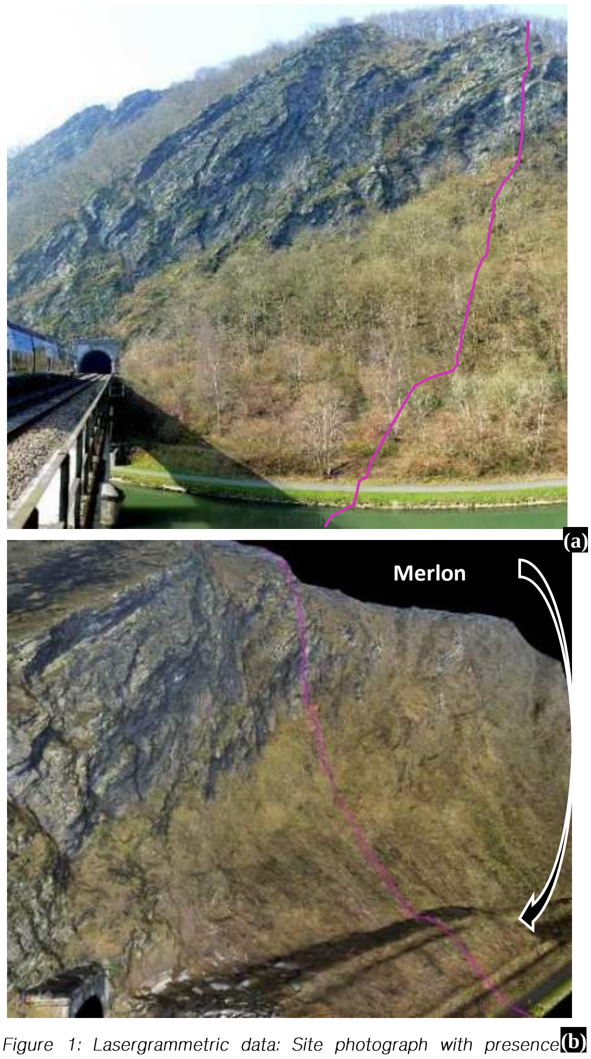

Figure 1: Lasergrammetric data: Site photograph with presence vegetation (a) and textured DEM after processing (b), The violet line represents the realized trajectographic profile.

Figure 2a shows the result of the elimination of various wire meshes on an instable rocky slope to be stabilized (Hospital Pasteur in Nice,. Figure 2b and c display the automatic discrimination of the echoes generated by the meshes from those generated by the rocky slope. Nevertheless, even though the presence of a wire mesh does not cause trouble as for the data processing, the acquisition time on the other hand is much higher.It is capital to multiply the stations so that the laser beam arrives perpendicularly against the rock wall. Furthermore, it is necessary to increase the angular resolution (with respect to a classic survey) in order to obtain enough points on the rock face to create a fine meshing.

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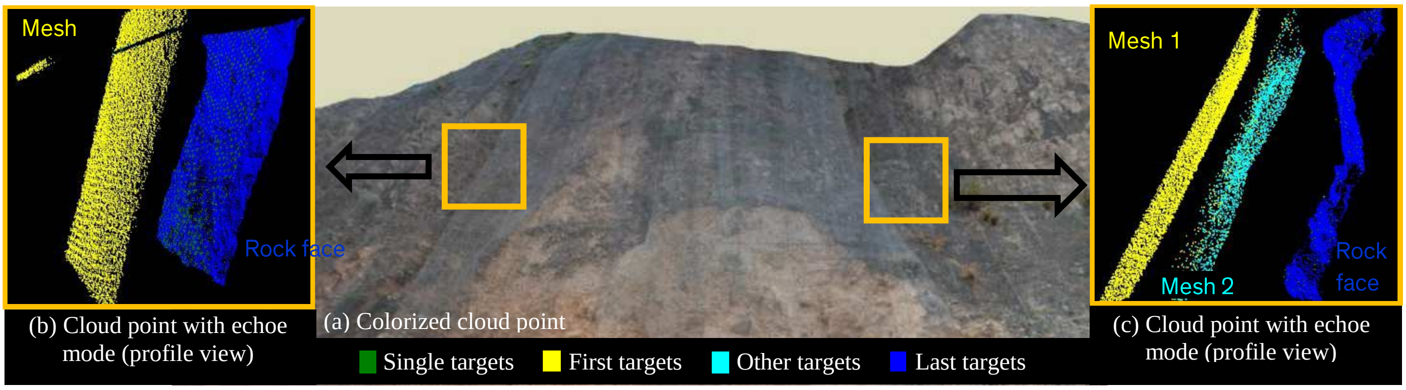

Figure 2: Lasergrammetric survey with colorized obtained cloud point (a) and discrimination of one (b) or two (c) hanged wire meshes

The elimination of vegetation and infrastructures layers allows a geo-structural analysis of the hidden rock wall. Figure 3 displays an example of result obtained by the automatic processing of lasergrammetric data.

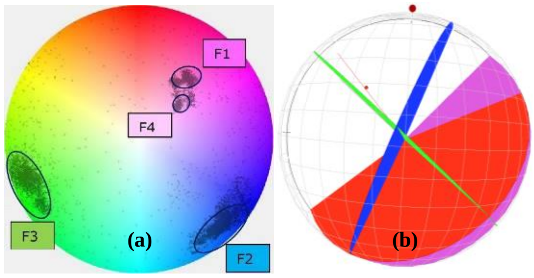

Figure 3: Geostructural analysis from lasergrammetric data: (a) Poles of different fractures families, HSV Diagram (Hue Saturation Value), (b) 3D Wülff diagram of surveyed discontinuities – lower

<u>Taking into account vegetation and existing protection structures in the trajectographic simulations</u>

In order to implement and design the passive protection structures against rockfalls, it is advisable to take into account vegetation and existing protection structures in the trajectographic simulations.

Figure 4 demonstrates the capacity of the lasergrammetric technic to qualify and quantify those elements. The precise 3D localization of the areas of vegetation, the density and type of vegetation, the size and diameter of the trees are easily measurable. The localizations and dimensions of the structures are also designed with a very fine accuracy.

2 AUSCULTATION AND GRAVITATIONAL INSTABILITIES MONITORING:

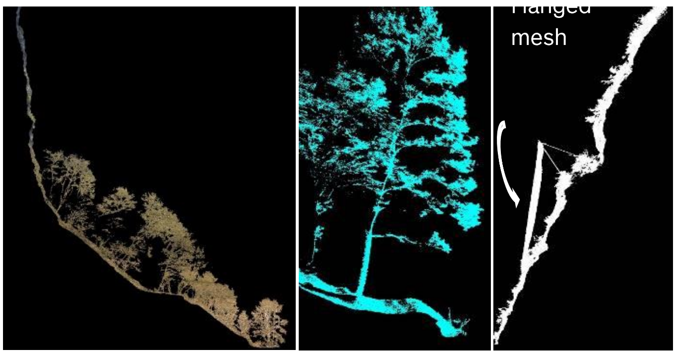

Figure 4: Overview of vegetation and a protection structure on a cross-section profile

Both technics of lasergrammetry and photogrammetry enable to compare 3D models carried out at different dates in order to quantify and evaluate displacement of a high risk site which do not present vegetation nor surface protection structures.

Nevertheless in a natural hazards prevention objective, the monitoring tools should not depend on environmental and climatic conditions since these are major factors affecting rockfalls risks.

The Laser scanner is the topographic device allowing a high speed and contactless acquisition thanks to a laser beam and a quick scanning mechanism. Some atmospheric parameters are likely to disturb and consequently to distort the data. Indeed, the temperature, the pressure, the atmospheric gaseous composition, the climatic conditions and the presence of particles in the air are the elements which influence the measurements. However some of these elements can be taken into account and implemented into the device in order to minimize the errors.

On the other hand, photogrammetry offers less flexibility for the adjustment of a precision monitoring. Indeed, this technic is not working by night, in back-light conditions, or when the air humidity is high. Therefore, the data gathering must be carried out in optimal environmental conditions to obtain quality results in order to detect displacements in the range of 1-5 cm. The environmental conditions also influence directly the lasergrammetric data but not as much as they affect photogrammectric ones.

3 CONCLUSION

Lasergrammetry and photogrammetry give high quality data. But, contrary to photogrammetry, lasergrammetry allows to create a DEM, getting rid of vegetation and existing surface protection structures, and to integrate these very elements into the trajectographic simulations, lasergrammetry is more suitable for diagnosis and the monitoring of gravitational instabilities. Despite a merely superior cost price (acquisition and processing), the additional data provided by lasergrammetry over photogrammetry bring a non-negligible added value for the project.

A cost-efficient approach to monitor rockfall activity over large areas using non-permanent single-camera system (mono-photogrammetry)

Julien TRAVELLETTI[1]**,** Frank PHILIPPOSSIAN[2], Raphaël MAYORAZ[3]

Keywords: rockfall, landslide, displacement, monitoring, photogrammetry, camera, remote sensing

1 INTRODUCTION

In the case of permanent single-camera systems (mono-photogrammetry), several studies (Travelletti et al., 2012; Motta et al., 2013) demonstrated the possibility to derive quantitative 3D information of landslides (eg. surface changes, 2D metric displacements) using existing Digital Elevation Models (DEM). Nevertheless similar quantitative applications on rockfall activity are relatively poor in the literature. In the case of non-permanent single-camera system, visual comparison is the most common application and the potential of such monitoring system is not yet fully exploited.

This work presents an original monitoring approach to highlight the potential of non-permanent single-camera systems to monitor rockfalls over large areas. The rock cliffs of Derborence located in Valais (Switzerland) are used as test site.

camera system

2 ACQUISITION SETUP

The monitoring is carried out with a 37 M pixels NIKON D800 camera and a 85 mm focal length (Figure 1). The acquisitions are realized every year during similar illumination conditions from the same base station at distances varying from 2 to 4 km. At these distances, pixel size varies from 12 to 24 cm (i.e. minimal theoretical bloc size that can be detected). Four pictures taken at each acquisition allow to cover an area of 5 km^2 (i.e. the effective area monitored by the system).

For each acquisition, the images are approximately oriented in the same direction as the reference by using natural landmarks in the rock cliffs visible in the display of the camera. Afterwards the errors of positioning and orientation are then minimized using an image registration algorithm.

3 IMAGE REGISTRATION

A least squares estimation algorithm taking into account translation, rotation and scaling corrections (affine transformation) is used to register the images in the same 2D intrinsic coordinate system of the reference (image sets of 2013). In some cases, a 2D piecewise linear geometric transformation is also realized. The residual errors between images are generally in the range of the pixel. Then the registered images are integrated in a GIS environment (ArcMap ESRI) and compared to the reference.

4 AUTOMATIC COMPARISONS OF THE IMAGES

Automatic comparison provides to the operator a global view of the main changes that occurred between two time series. It allows to:

- Detect important movements of rock compartments: the aim is to automatically track the same natural features (i.e. well contrasted surface) identified in different time series using digital image correlation (Travelletti et al., 2012) and object tracking algorithms (Gance et al., 2014) to define 2D displacements in the image plane ;
- Identify new rock fall sources: the computation of correlation coefficient associated with filtering critera allow the detection of anomalies in the image texture due to missing rock volumes (Figure 2). The detected anomalies are then validated and digitized in a shapefile by the operator.

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5 GEOREFERENCING

In order to derive 3D metric outputs from a single-camera system, the solution described in Travelletti et al., 2012 is implemented. An auto-calibration of the camera is firstly realized. Then the method consists in projecting the XYZ coordinates of an existing DEM in the image plane. The 3D coordinates of the projected DEM are then interpolated to allocate a triplet of X, Y, Z coordinates to each pixel of the image. This procedure allows to:

- Locate rock sources (failure) in the local coordinate system (Figure 2);
- Estimate the magnitude of the rock fall by computing the area of the rock source in the vertical plane;
- Evaluate the magnitude of the displacements in metric units.

All the results are stored in a GIS environment (grids and shapefile with attribute table).

6 CONCLUSION

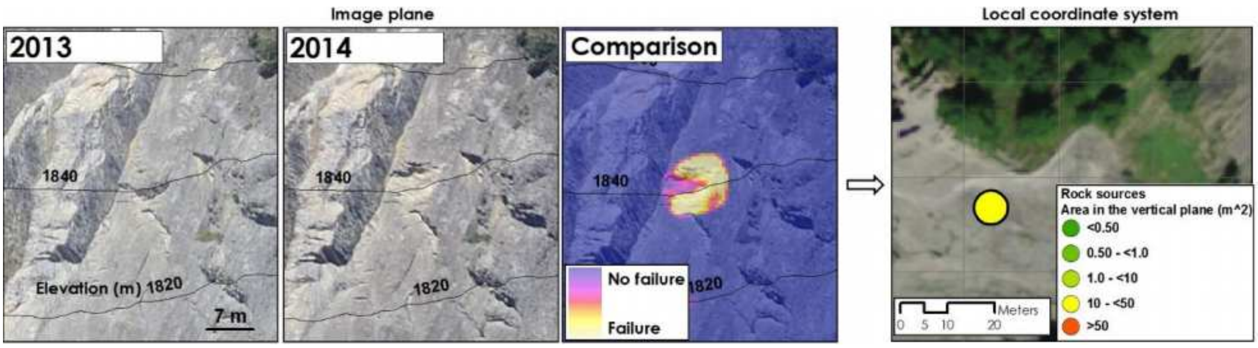

Figure 2: example of rock source detection located at 2 km.

An approach is proposed to monitor rockfall activity using non-permanent single camera. The main advantages are the following:

- Implementation, survey and equipment costs much lower than GB-InSAR or Terrestrial Laser Scanner
- The long range remote imaging technique (resolution of 24 cm at 4 km with the camera used in the study)
- Possibility to monitor large area (in the study : 5 km^2 with 4 images)
- The effective rockfall sources detection (blocs of 10^{-2} m^3 - 10^{-1} m^3 can be potentially detected at $>$1 km)
- Mostly automatic processing
- The intuitive interpretation also for non-experts (standard RGB images)
- Possibility to localite/orthorectify the results in the local reference system (e.g. CH 1903)
- Possibility to directly store the raw data and the results in a GIS environment (simplicity of data management)

The strongest limitations are related to the meteorological, illumination conditions and the presence of vegetation. Finally, the method appears particularly interesting for the evaluation of the relationships between frequency and magnitude of failure. It offers promising perspectives of development with the arrival of gigapixel imaging systems.

7 ACKNOWLEDGEMENTS

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Session 7B

Miscellaneaous topics

A quick method to evaluate the optimum mesh type for remediation of unstable rock and soil slopes

Alberto GRIMOD[1]**,** Giorgio GIACCHETTI[2]

Keywords: **mesh, rockfall, soil nailing, preliminary design**

1 ABSTRACT

Steel wire meshes are commonly used to protect highways, roads, railways, etc. from rock fall hazards or superficial soil instabilities. These solutions are used worldwide as simple or secured drapery systems, as well as superficial structural facing for soil nailing applications. Several times the choice of the best type of mesh must be defined in a short period of time (i.e. emergency situations) in order to figure out a safe and cost effective protection which may guarantee the regular traffic on roads or other infrastructures and/or may protect workers during the installation of other permanent rockfall protection systems (i.e. passive rockfall protection systems, such as dynamic rockfall barriers or embankments).

This paper describes a quick and simple methodology developed by Maccaferri to easily define the most suitable mesh for specific applications. The approach is based on evidences that users can easily analyse and define during their preliminary inspection on site. It is not required specific knowledges on the geology of the soil and/or on the geomechanical behaviour of the rock is. It can be a useful tool for a preliminary design of the best solution to be used for a specific goal. Moreover, users can do their choice directly on site.

The analysis is divided on several questions about the main characteristics of the slope, the rockfall events and the environmental conditions, such as: the type of application (i.e. simple drapery system, erosion control system, etc.), the geometry of the slope (i.e. inclination, height, etc.), the slope morphology (i.e. planar, uneven surface, etc.), the size of the potential rock boulders that can fall or rolling along the slope, the possible in-situ corrosion condition (i.e. polluted areas, marine environment, etc.), etc.. For each question, it is possible to express one preference between different proposed options, and select a coefficient function on the importance of the parameter analysed. More than 25 meshes are considered, and for each of them a certain value is calculated. The result is a number obtained by adding all the partial scores related to the different questions. In order to find out the best choice(s) the tool allows to rank all the Maccaferri mesh in a numerical scale from the best solution(s) (higher score) to the worst one(s) (lower score).

To better understand the reliability of this innovative tool, an example of a real application is presented. The example compares the results obtained during a preliminary design carried out with the presented methodology and the ones calculated with a more accurate approach.

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A method of rockfall protection in municipal monument reserve –a case study from medieval town Tabor, South Bohemia

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Keywords: Bioengineering methods, rockfall protection, Biotec Vegetation Strips

Routinely and frequently used rockfall mitigation measures often are not applicable in the zones protected by law due to nature, landscape or historic preservation. An extensive effort to find mutually acceptable solution often fails. At this case could help unconventional techniques. The BIOTEC Vegetation Strip could be one of these methods. It was also used during remediation of rock outcrops in the territory of municipal monument reserve Tabor in south of Bohemia (medieval picturesque town). The standard methods (netting with bolts or catching fences) did not have permission from National preservation trust due to aesthetic disturbances, so another technique was find. The Biotec Vegetation Strips fulfiled the demands of preservationists and began to be the main part of designed solution.

The system consist of classic hexagonal doubletwisted steel mesh, which is completed by special sacks of vegetative substrate. The net is anchored to the ground/rock, as usual for drapery systems. Pockets are made from the steel net , layers of juta and synthetic textiles. Their placement depend on situation, but generally are mounted slantwise on the main steel mesh. The way of assembling mentioned materials is covered by a patent. This method of nets compositions is not visualy noticeable and moreover this solution combine the advantages of steel nets with antierosion effect of vegetative layer.

This contribution deals with description of mentioned rockfall protection, its benefits and disadvantages.

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Description of Ax-les-Thermes cuttings for a Rock fall hazard Benchmark case

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Keywords: benchmark, rock slope, natural hazard

This paper will present the site, the geological and structural data that constitute a benchmark on rock fall hazard. All the data provided come from a site that was studied for the bypass of Ax-les-Thermes (Ariège, France).

1 AIM OF THE BENCHMARK

Starting from fracturing data and a slope (or underground) geometry, estimates of the rock fall hazard can be ran under different methodologies, using various software. The idea of this benchmark is to compare practices in:

- Regrouping fractures in families:
- Consideration or not of sampling bias
- Mechanical properties taken into account for the discontinuities (deterministic or stochastic)
- Rock fall hazard estimation
- The stability indicators given from the stability analyses. These indicators can be varied for example the results can be expressed in terms of:
  - biggest unstable block
  - more frequent block (possibly with a confidence interval)
  - total unstable volume (weight) and dispersion around this volume
  - number of unstable blocks
  - blocks displacement and movement types
  - local or Global Safety Factor
- The inclusion of a reinforcement (bolting) and the presence of water in the hazard assessment
- software used for these different steps
- How the geometrical, and mechanical properties uncertainties are taken into account

We propose to work on data collected before excavating a high cutting in a migmatitic rock excavated for a bypass in the French Pyrenees where 861 fracture measurements were performed (Gasc-Barbier et al., 2008). Another benchmark on rock fall hazard in underground context will also be proposed based on the data collected before and during the excavation of St Beat tunnel, also in the French Pyrenees.

2 DATA SURVEY

2.1 LOCATION

Ax-les-Thermes lies 130 km south of Toulouse, just 30 km from the border between France and Andorra, in the primary axial zone of the French Pyrenees. Three orogenic phases that affected the entire mountain lead in considerable fracturing of the site, which is apparent in different forms depending on the area studied.

2.2 GEOLOGICAL INFORMATION

The bypass encounters Quaternary superficial formations, fluvio-glacial materials and then the rock substratum, where the benchmark takes place. The slope under study (Esquiroulet cuttings) is mainly composed of augen gneiss and leptinic gneiss, in practice it is impossible to separate them into two distinct areas and hence the formation is best described as migmatite (in the widest sense). All these rocks are heavily fractured.

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2.3 SCANLINES AND BOREHOLE IMAGES

For several reasons, excavation of the site was stopped when the proposed 42 m high cutting was 15 m high. This provided an opportunity to realize scanlines on the new cuttings to obtain statistical information on the joint properties. A scanline is a horizontal line (of about 10 m) drawn on a rock face (Priest, 1993). The line can, in fact, be vertical or horizontal but should have spatial references (dip and dip direction if possible). As recommended by many authors (Aftès, 2003, Priest, 1993, Khanlari et Mohammadi, 2005), the standard properties of all the joints that intersect the line are mapped: dip, dip direction, spacing, trace length, weathering, aperture, infill and roughness. Several lines in different directions were drawn in order to try to intersect the maximum number of joint sets. In this particular case and because of the geometry of the site, the scanlines were drawn on three faces as depicted in Figure 1 but their lengths were quite different. It was not possible to use vertical lines because of the difficulty of access. In total, 11 scanlines (from 6 to 30 m) were plotted; including 856 discontinuities over 181.5 m. Borehole images, also allow having a kind of "scanlines". Two boreholes were inspected by camera allowing an enhancing of the data and "internal" data of the rock mass. All the data can be found on https://www.researchgate.net/profile/Muriel_Gasc-Barbier/publications.

2.4 OTHER PROVIDED INFORMATION

Other data are provided on https://www.researchgate.net/profile/Muriel_Gasc-Barbier/publications: Elastic waves velocities, uniaxial and Brazilian tests and shear tests on natural joints.

2.5 OTHER PROVIDED INFORMATION

Figure 1a shows a 3D view of the site when scanlines were drawn. Figure 1b gives the proposed geometry of the cutting to be reinforced for the benchmark.

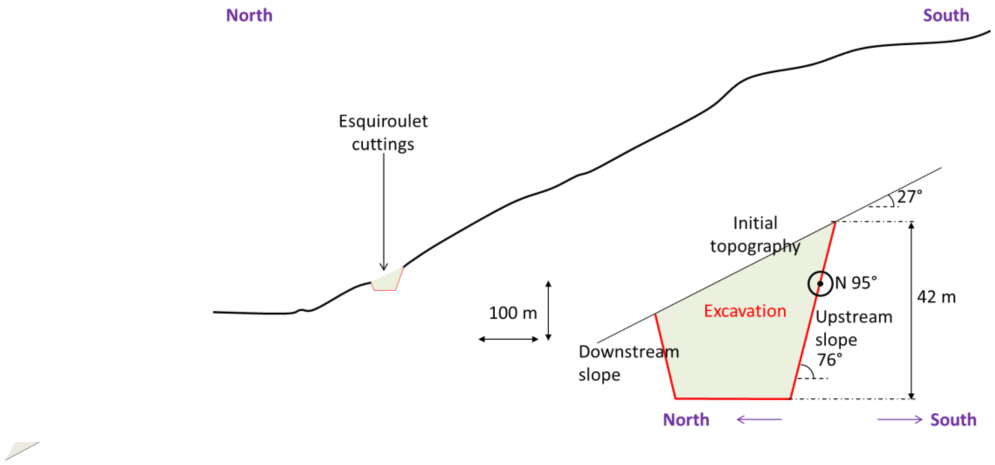

Figure 1: Ax-les-Thermes cuttings – a- localization of the different scanlines, b- proposition of geometry for the benchmark

3 CONCLUSION

This benchmark is conduct under the supervision of the co-authors of this paper in 2016 and main conclusions are awaited at the beginning of 2017. Please contact the authors for information and participation to the benchmark.

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Assessment of the slope stability of the deep road cuts excavated during the construction of Anamur – Kaledran State Road

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Keywords: anamur, state road, rock mass, slope stability, schist, cut slope

This study aims to assess the stability problems on the deep cut slopes at Anamur – Kaledran State Road created in over jointed schist and locally landslided areas according to the geological and topographic effects and to improve suitable engineering solutions. Along the project route, these cut slopes built with earlier design patterns reach 120-meters of height. In field works, it's predicted that these local landslides has a retrogressive feature and they could become larger landslides that harm the road security if no precaution is taken.

1 PROPERTIES OF SITE AND PROJECT

1.1 PROPERTIES OF PROJECT AREA

Anamur is a district of Mersin province located at the Eastern Mediterranean region of Turkey. This district is also located between the Mediterranean Sea and Taurus Mountain range. Therefore, the project area is highly affected by tectonic activities due to the continuous movement of Arabic and Eurasian plates. For this reason, structurally, rock units of this region are usually over jointed and folded. Within the compaction process, rock units are exposed to high temperature and pressure conditions. Herewith, within this period, sedimentary rock units such as sandstone became metasandstone and various types of schists. These over jointed schist and metasandstones are the typical rock units of the investigation area.

1.2 PROPERTIES OF PROJECT AND EXISTING DESIGN

The Anamur – Kaledran State Road was projected and designed in the master project named "Erdemli – Silifke – Taşucu -14th Region Boundary State Road" and contracted out by General Directorate of Highways. At the time of completion of the road construction, two major and important provinces of Mediterranean Region; Antalya and Mersin will be connected with 2x2 divide road. Therefore, completion of this road has an importance thereby for economical and touristic fields.

According to the topoghraphical profile of project route, superstructures such as Tunnels and Viaducts and deep cut slopes are designed. Due to principles of the existing design; cut slopes located on the project route have up to 120-m of height. At the surface of cut slopes, there is no support system and additional prevention for the stability of slopes. For this reason, in course of time, due to the atmospheric and gravitational conditions, local landslides started to grow up at the surfaces of slopes. The types and geometries of landslides, solution proposed for the prevention of these stability problems and principles for redesign are the main purpose and subject of this paper.

2 SITE INVESTIGATIONS AND GEOLOGICAL MODELING

Within the scope of studies, to determine geological and tectonic properties of the study area, topographic and geological mapping studies have been carried out along the cut slopes of route. To determine main values of slope design, geological – geotechnical field observation studies have been carried out at the problematic cut slopes along the project route.

With field observation and mapping studies, geological history, properties and model are performed. Within this modelling studies, depth and size of landslides, main features of rocks such as discontinuity systems, weathering and alteration zones, approximate groundwater levels are revealed.

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All these information and database are confirmed and extended with database obtained from boreholes drilled at various locations of the project route. With all these site investigations and geotechnical drilling works; geological, geomechanical and geotechnical modelling is completed.

3 ASSIGNATION OF PARAMETERS AND REDESIGN OF SLOPES

With the correlated database from site investigations and drilling works, parameters for slope design are obtained. For slope stability analyses, modified RMR based GSI and empirical GSI chart developed by Sönmez and Ulusay (2002) are used together. With these GSI values and other parameters such as “Uniaxial Compression Strength”, “Material Constant(mi)”, Disturbance Factor(Df) etc. are used to obtain “Hoek Brown Failure Criteria” as the strength and shear parameters of rock mass. With the generated rock mass parameters, problematic rock slopes are redesigned.
Firstly, slopes are redesigned with opening 3 meters of buffer-shelf area at the toe of slope in order to prevent the negative effect of local landslides controlled by joints on fluence of vehicle traffic. In new design, height of single slopes are decreased to 10 meter and the width of first berm increased to 7 meter. The gradient of slopes are taken similar as existing slopes. To reach sufficient factor of safety, the redesigned slope geometries are supported with rock slope support systems firstly. Groundwater condition is one of the most effective factors for determining of support types during the re-design process of slopes. Design of slopes is performed to prevent sudden and violent rise of hydrostatic pressure in joint system of rock masses because of the irregular precipitation index of Anamur Region. Sudden and violent rise in hydrostatic pressure causes weakening of the rock mass, decrease in shear strength of rock mass and causes irreversible and significant slope stability problems. In this context, there are two major aspects during the slope design. First of them is to avoid-covering the whole slope surface with shotcrete and rockbolts. Covering the whole surface of the slope with shotcrete creates an impermeable layer and causes rise in the hydrostatic pressure. Another major aspect is fine – porous wire mesh application. The main purpose of fine – porous wire mesh application is to prevent negative effect of joint – controlled rock falls and ruptures to integrity of the rock mass stability on non – supported areas of slopes. This situation causes and triggers major stability problems on rock slopes.

4 CONCLUSION

As mentioned above, geological – geotechnical field observation studies have been carried out at the problematic cut slopes along the project route. And the detected parameters from these studies are supported with the parameters provided from the geotechnical and geomechanical database of drilling studies. With achieved and correlated parameters, these problematic cut slopes are redesigned. As the result of all assessment during and after re-construction period of cut slopes, it’s established that the GSI values obtained from field works and the new design of slopes with these rock mass parameters and all applied methods present highly successful performance at over jointed rocks masses as well.

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Ground pressure calculation over flexible membranes for slope stabilization systems through rational geotechnical models.

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Keywords: geotechnical model, local stability, slope stabilization, flexible membrane

The flexible systems have been used for many years as a reliable solution for slope stabilization. These kinds of systems are founded in the combination of an anchor system and a flexible membrane characterized by a high tensile strength and low deformation in working conditions, generally composed by wire rope net panels or high performance chain link wire meshes. While the global stability of the slope can be guarantee by the implementation and design of an anchor solution, the dimensioning of the flexible membrane requires a deep how-to-know in order to determine the unitary pressure exerted by the terrain located inside the anchor grid. By using a specific geotechnical model for the analysis of the local stability, it is possible to calculate the local pressure of the rock-ground mass located inside the anchor grid.

1 BACKGROUND

The classical geotechnical approach allows determining the unitary support required to guarantee a specific safety factor in long term conditions of the slope which in turn provides the required solution for the anchor system design. Once the global stability of the slope is guaranteed, the calculation of the pressure generated by the instability of the most superficial materials of the slope located between the anchors which are exposed to the weathering and erosion processes is required. In this area of the slope, by degradation and weathering of the materials, the geotechnical parameters related to the stability of the slope (friction angle and cohesion) decrease over time in long term conditions. Through a geotechnical model for the analysis of local stability, it is possible to determine the local pressure of the ground over a flexible membrane which is opposed to the movement, according to the analysis of the equilibrium of the superficial unstable mass located inside the anchor grid pattern, under the assumption of the specific hypothesis of the geotechnical model. The calculation of the land pressure over Flexible Membranes for Slope Stabilization combined with passive anchor bolts has been used since 1.996 when the first version of this geotechnical model was developed by Dr. Torres Vila et Al. This model has been currently implemented and improved and enables to determine the most efficient flexible system solution to solve a specific stability problem.

2 GEOTECHNICAL MODEL

The geotechnical model is based on the following hypotheses which remain on the safety side:

- The failure sliding surface of the unstable ground mass is considered as a plane.
- The potentially unstable ground mass is limited by consecutive anchors disposed in the same vertical line, the slope surface and the lower sliding surface of the unstable soil mass (α_{SD}).
- The angle of the potential sliding surface is obtained by a process of optimization, by selecting the value where the safety factor for the equilibrium analysis rich to a minimum.

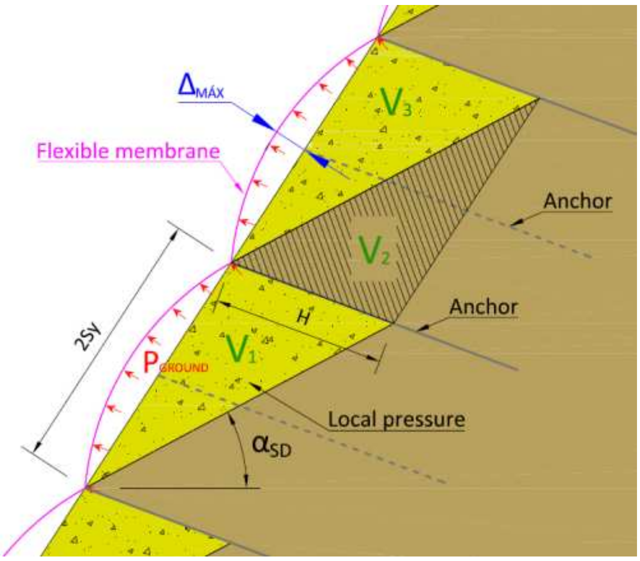

Figure 1: Scheme for the geotechnical model for local stability analysis.

- The potentially unstable ground mass (yellow area in the scheme, V1) to get the geotechnical equilibrium is affected by the loads caused by the wedge of ground located on top of the unstable soil mass (dashed area, V2) and the loads caused by wedge of unstable soil located over it (V3).

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- The intersection between the sliding surface and the plane defined by the anchor line determines the depth of the potentially unstable soil mass (H).
- The depth of the unstable mass (H) is variable and depends on geotechnical parameters of the ground, on anchor distribution, and especially on the slope angle. The influence of the different parameters is indicated below:

$$H = f\,(\beta, \emptyset, c, \gamma, S_Y, \alpha_{SD}) \qquad (1)$$

- where (β) is the slope angle, (Ø) the angle of internal friction, (c) cohesion, (γ) density of the ground material, (S_Y) vertical distance between two consecutive horizontal rows of anchors and (α_{SD}) angle of the critical slide surface.

With the solution of the geotechnical model, the next results are obtained:

- Support required for the stabilization, p_{Local} (kN/m^2).
- Required load to be transmitted to the anchor heads by the membrane.
- Deep of the potentially unstable mass, H (m).
- Angle of the anchor drilling (measured from the horizontal line), Θ.

The geotechnical model allows determining the depth of the potentially unstable mass and this value is obtained as a result of the calculation process. This model is applicable when the ground material is considered as uniform mass like rock surfaces highly fractured with high weathering level, high weathered long term rocks and conglomerates with medium-low cementation.

Other empirical calculation models commonly used in this field consider as hypotheses an unstable layer with a constant thickness previously settled and a direction of movement parallel to the slope surface as well as the movement of the anchor plate against the ground. These supposed hypotheses are far away from what happens in actual conditions.

3 SELECTION OF THE FLEXIBLE SLOPE STABILIZATION SYSTEM

Once the local pressure required by the geotechnical model (p_{Design}) is obtained, the designer have to select the flexible slope stabilization system composed by a flexible membrane and anchor solution which should be able to offer an unitary support higher or equal than the required design support ($p_{System} > p_{Design}$).

The suppliers of the flexible slope stabilization system should guarantee the capacity of the system (p_{System}). The capacity of the system must be obtained from the mechanical characteristics of the flexible membrane and the whole elements of the system (reinforced steel cables, sewing elements, flexible anchors, accessories…), using a physical model of behavior validated by laboratory tests and considering a certain safety factor and checking the level of the deformation of the membrane.

4 CONCLUSION

The geotechnical model for local stability analysis has been used for nearly 20 years, during which the model has been improved and validated as a tool for calculating the case of surface slope stability analysis for soil loose materials. The model allows determining the pressure of the terrain located inside the anchor grid in order to be compared with the capacity of a flexible system (support capacity) in order to ensure the local stability of the slope. This model is out of the range of traditional nailing, and could be considered closer to a flexible anchored wall. The model developed by 3S Geotecnia y Tecnología, S.L. team has been also implemented in an computer application which provides the complete design of the flexible system TUTOR® composed by high performance wire mesh of high tensile strength and low deformation.

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Session 8

Trajectography analysis (2)

A comparison between DEM and MPM for the modelling of unsteady flow

Fabio Gracia[1*], Vincent Richefeu[2], Pascal Villard[3]

Keywords: MPM, DEM, Plasticity, Constitutive Models, Energy Dissipation

This work deals with a comparison of transient granular flow simulations using two radically different approaches: Discrete Element Method (DEM) and Material Point Method (MPM); see Cundall & Strack (1979) and Sulsky et al. (1995), respectively. Certain parameters in the simulations were varied in order to analyze its influence. Properties of the final deposit such as center of mass and spread of the deposit were used to compare similarities, as well as differences between the two methods. Additionally, energy dissipation during the simulations was evaluated.

1 CONTINUUM APPROACH

In the field of rock avalanches, the continuum approaches are generally considered when large volumes (i.e., large number of blocks) are involved. For this study, the MPM was chosen over other continuum-based methods because of its hybrid Lagrangian and Eulerian description, which gives it the ability to manage large deformations. This makes it an ideal method for the modelling of rock-mass propagation events. In return, the continuity assumption may also limit the domain of validity of continuum approaches, specifically for the application of rock flows. The computational times required by MPM simulations are significantly low, but the dissipation means and more generally the rheologies have to be given explicitly by constitutive laws with, unavoidably, the continuity of the medium in mind.

2 DISCRETE APPROACH

A discrete element modelling, taking account of realistic grain shapes and dissipative contact laws (Richefeu, 2012; Mollon 2012) was used for this study. DEM requires an accurate definition of the initial condition for the terrain, block shapes and families of discontinuity, together with prohibitive computation time. As a consequence DEM is better appropriate for medium sized volume (i.e., with few hundred blocks in the deposit). We know that complex rheologies, inherent in the granular materials, are naturally well captured by the DEM – provided that a minimum set of features are taken into account. In our case, these features are the shapes of the blocks and adequate dissipative contact models. The price to pay is however long computational times.

3 COMPARISONS

The two numerical methods were compared on the base of numerical simulations which consist (*i*) in the release of randomly packed bricks within a parallelepiped box for the DEM, and (*ii*) in a continuum mass governed by an elasto-plastic law for the MPM. The box is positioned on a slope of 45°, with a smooth transition to a horizontal plane where the bricks stop after the door of the box is opened. DEM simulations are tri-dimensional; the flow is thus canalized so that it can be compared with the two-dimensional MPM simulations.

Comparisons between the two models were based on the run-out distances and on the deposit positions (fronts and rears). Different block shapes (A and C: Cubes; B: Bricks) were tested for different release heights (numbered 1 to 3) and different aspect ratios of the starting box (A: square box; B and C: rectangular box). A sensitivity study was performed by varying the dissipation parameters. As an example, Figure 1 shows the results for a basal friction of 0.3, which is one of the driving parameters. At first glance it was found that MPM and DEM results, in terms of run-out distances and spread of the deposit, were similar when using basal angle of friction lower than the internal angle of friction (the Coulomb cohesion being low enough to be negligible). The MPM simulations lead thus to results similar to DEM predictions (taken here as the ground truth) as long as the internal friction angle used in the elasto-plastic constitutive law is greater than the basal friction. In this particular case,

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similarities were also observed for the modes and rates of energy dissipation, with a better match for bricks that are less prone to rotate than cubes. Anyway, the rotations remain limited for small values of basal friction.

On the other hand, completely different results were found when the internal friction angle was chosen lower than the basal friction. Both run-out distances, spreads of the deposit and energy dissipated differ in a considerable manner. This behavior is due to complex mechanisms within the mass, which involve high rotations of the blocks that activates normal and tangential modes of dissipation between the blocks, and an increase in normal dissipation with the boundary along with a decrease of friction energy at the base. The Mohr-Coulomb law is not designed to account such mechanisms. It is however clear that the block shape plays a crucial role. It was observed that bricks, that are less subjected to rotation than cubes, imply a kinematic field closer to the continuum one generated by the MPM. Consequently, better agreement of MPM and DEM results was obtained for bricks. It was then confirmed that these complex mechanisms observed in DEM are the factor that determines such differences between the two methods.

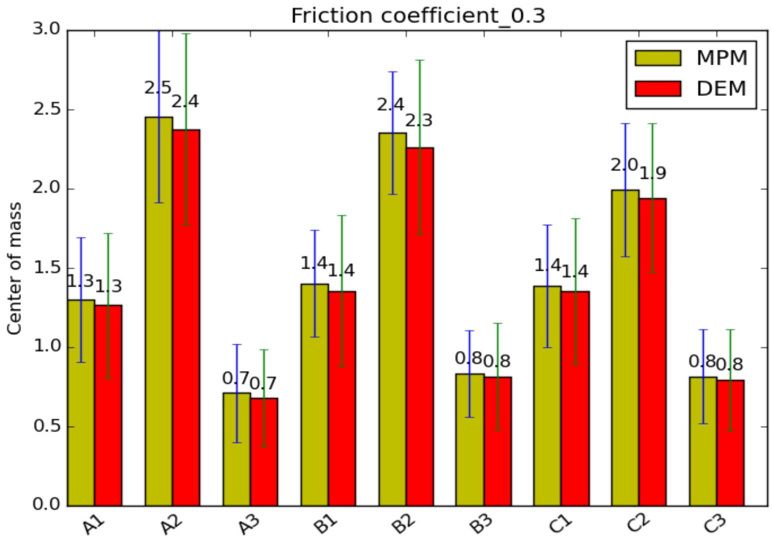

Figure 1: Run-out distance. The vertical lines show the spread of the final deposit.

4 CONCLUSION

Different configurations were tested by varying the aspect ratio of the box, the block shapes, and the release height. The results were compared in terms of run-out distance and energy dissipation by distinguishing the dissipation modes (friction at the base, normal restitution at the base, and dissipation within the flowing mass), and strong correlations were found between the two methods when certain complex internal dissipation mechanisms are avoided. Increasing the basal friction can activate these complex mechanisms. In these situations, the Mohr-Coulomb plastic law is no longer suitable but the internal friction coefficient can be adjusted as a function of the flowing regime (internal shearing or “track-system flow”), which is indirectly related to the block shapes.

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Restitution Coefficients and Roughness Parameters for Non-Smooth Rigid Body Rockfall Modelling

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Keywords: bouncing, sliding, rolling, trajectory modelling, RAMMS, restitution coefficients, roughness, sliding friction, non-smooth mechanics, rock shape, hard contact modelling

Rockfall trajectories consist of three fundamental modes of motion: *bouncing*, *rolling* and *sliding*. Long (and dangerous)-runout is primarily associated with ballistic-type, bouncing motions because the rock is mostly airborne and experiences no friction. Rolling and sliding modes of motion are encountered when the rock is in the final stage of motion, typically preceding stopping and deposition. Considerable attention in rockfall mechanics has therefore been devoted to defining apparent restitution coefficients which define the rebound behaviour of rocks on various substrates (Asteriou et al., 2012;Ansari et al., 2015). This allows rockfall models to predict all-important bouncing-mode trajectories that essentially define the extent of rockfall danger. Cut-off criteria (rock energy, velocity, jump height and/or combinations) are often used to model non-bouncing modes that define the final runout distance.

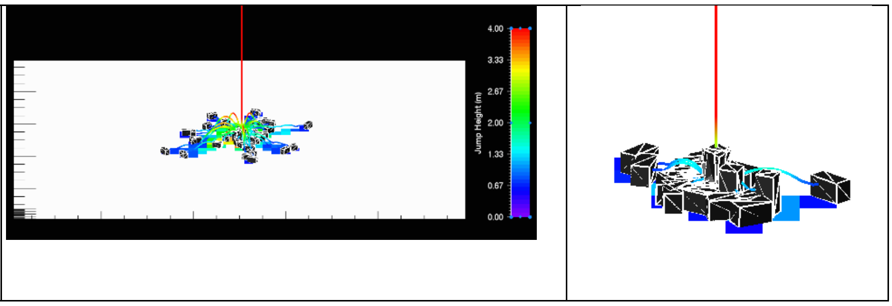
Restitution coefficients are difficult to define in real terrain because they lump a wide variety of physical processes into one "apparent" rebound coefficient. These characteristics include (a) slope geometry, (b) slope roughness, (c) sliding and rolling friction, (d) slope hardness, (e) rock shape and incidence angle and (f) rock density and strength (Ansari et al., 2015). In-situ tests often reveal a wide scatter of values, even for the same rock type and substrate, see Asteriou et al.(2012) or Bourrier et al.(2012). In theory, restitution coefficients should only define the elasticity of the rock and substrate. However, in rockfall modelling they are used to control the whole (and very complex) rebound dynamic from initiation to runout.

In this paper we apply a hard contact, rigid body modelling approach (Leine et al., 2014) that includes slope geometry, sliding and rolling friction, rock shape and incidence angle. The outcome of a rock impact is to change not only the translational motion of the rock but also the rotational velocity. Our goal is to separate "apparent" restitution coefficients from "true" restitution coefficients defined by rock/substrate hardness.

1 PARAMETERIZATION OF REBOUND AND GROUND FRICTION

In non-smooth models, ground impact is considered over a finite sliding distance S, which we term the *scarring length*. The scarring length S is defined when any point on the *surface* of the rock is in contact with the ground. Contact is defined with respect to the rock's complex surface geometry. Contact forces, including friction, are introduced at the surface of the rock and cause rock rotations. The magnitude of the rotation at exit (when the rock departs the scar) are therefore dependent on the orientation of the rock when it hits the ground. A restitution coefficient ε is applied at the rock surface to account for energy dissipation in the normal direction (Fig. 1).

Figure 1: Fifty cuboids are dropped from a height of 100 m with an initial rotation and random orientation, They impact a flat plane. Left: ε=0.15, Mean jump height 1.30m; max jump height 3.85m. Right: With ε=0 the rocks still displace (and bounce) because of the transfer of rotational energy into translation energy: Mean jump height 0.67 m, max jump height 1.55m. Surface roughness can also introduce bouncing, but requires an energy source and geometric perturbation.



If $\varepsilon = 0$(fully plastic) then bounce heights are *purely* a function of the impact orientation and the friction in the tangential direction. Hard ground does not allow sliding *and therefore can produce bouncing modes of propagation.* The ground scar is being created at the speed $\dot{S}$ and is given by

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$$\dot{s} = \begin{cases} \|v\| & \text{if contact} \\ -\beta s & \text{if no contact} \end{cases} \tag{1}$$

where $\|v\|$ is the sliding speed of the rock and β is a parameter controlling ramping effects, created when ground material is being displaced during the scar formation. When the ground is soft β is small because the ground is softer and the rock remains in contact with the ground longer (Fig. 2). The scar parameter β serves to extend the time that ground friction, or any other drag process, operates on the rock.

When the rock is in contact with the ground, friction is described by a combination of Coulomb friction S_N and viscous friction S_V. These forces act in the tangential direction; that is, in the direction opposite to the rock velocity. The Coulomb friction coefficient $\mu(s)$ for $s > 0$ is given by the transcendental function

$$\mu(s) = \mu_{\min} + \frac{2}{\pi}(\mu_{\max} - \mu_{\min})\arctan(\kappa s) \tag{2}$$

This function contains three parameters: $\mu_{\min}$, $\mu_{\max}$ and κ. The parameter $\mu_{\min}$ defines the initial friction at the beginning of the ground interaction. Softer ground has lower $\mu_{\min}$ values. We then assume that during the ground interaction the rock penetrates the ground and, because of confining pressures, ground material cannot be easily displaced out of the scar. This leads to an increase of friction, or *scar hardening*. The maximum friction is defined by $\mu_{\max}$ which we regard as a limit friction value, constant for all soils, but very low for easily deformable and porous ground materials like snow. An important parameter is κ which defines how quickly the ground material changes from $\mu_{\min}$ to $\mu_{\max}$, which is a function of how easily a ground material compresses (in the sliding direction) during ground penetration. The Coulomb friction term is supplemented with a viscous, rate-dependent friction,

$$S_V = -\nu M\|v\| \tag{3}$$

where M is the total mass of the rock and ν is the viscous drag coefficient.

2 EXAMPLES

In the first example problem we release 50 equant shaped rocks from rest on a 40º slope (Fig. 2). The rocks are randomly oriented to obtain a natural lateral dispersion from the starting point. Two different restitution coefficients are applied: $\varepsilon = 0.00$ and $\varepsilon = 0.15$. The sliding friction parameters remain the same in both simulations: μ_{min}=0.35, μ_{max}=2.00, β=150 1/s, κ=2 1/m, ν=0.6 m.

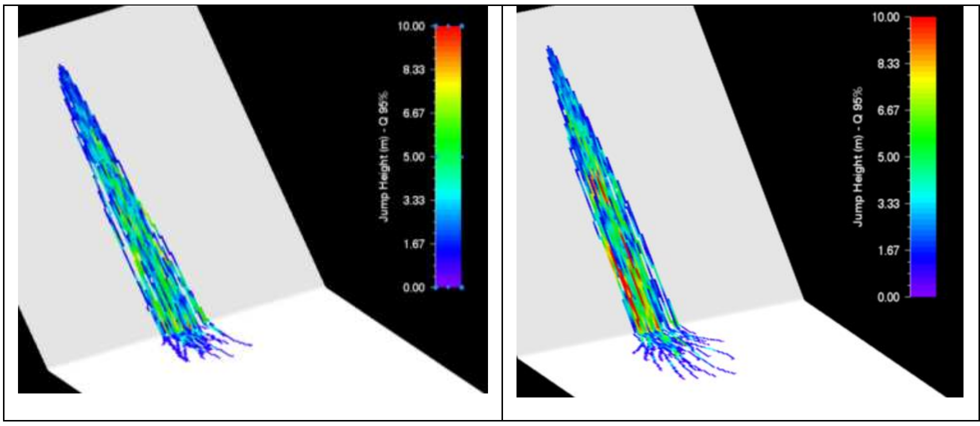


Figure 2: Fifty real shaped equant rocks are dropped from a height of 10 m and random orientation. They impact a flat 40º plane: Left: ε=0 Mean jump height 1.91m; max jump height 9.28 m. Right: ε =0.15 Mean jump height 2.19 m, max jump height 13.10 m. The "roughness" or friction parameters are: μ_{min}=0.35, μ_{max}=2, β=150 1/s, κ=2 1/m, ν=0.6 m. Runout is a function of hardness and roughness. On the slope, trajectories consist of bouncing, rolling and sliding motions, even when ε=0.

We find only a slight increase in mean jump heights, from 1.9 m ($\varepsilon = 0.00$) to 2.1 m ($\varepsilon = 0.15$). The maximum jump height increases significantly to 13.1 m ($\varepsilon = 0.15$) from 9.3 m ($\varepsilon = 0.00$). There is only a slight increase in runout for the more bouncy case ($\varepsilon = 0.15$). The important result of this simulation, however, is to demonstrate that realistic bouncing modes are possible even for fully plastic restitution coefficients. This (somewhat counter-intuitive) result indicates that ground friction converts rolling kinetic energy (spin) into translational kinetic energy (jumping), even when the restitution coefficient is $\varepsilon = 0.00$. Thus, "apparent restitution coefficients" (with values much higher than $\varepsilon > 0.15$) can *appear* to exist when the full complexity of the ground interaction is considered in the numerical modelling. Apparent restitution coefficients will demonstrate much variability on the same ground because the impact orientation of the rock and rotational velocity varies from impact to impact (Leine et al., 2014). The true restitution coefficient ε should be determined in simple drop tests without spin. Recent work of Gerber and McArdell (2016) indicates that true restitution coefficients for ground are typically near zero, $\varepsilon = 0.00$ and much less than $\varepsilon < 0.10$.

In the next example problem we simulate the real rock experiments of Volkwein et al. (2016) performed in the upper Rhine valley of the Swiss canton of Grisons. In these experiments rocks of variable shape were release down a grassy slope. The slope ran out onto a flat runout zone; the same rock was dropped several times in order to obtain some statistical information on lateral spreading and runout. Here we present the results of two particular rock shapes: an equant shaped rock (Fig. 3) and a platy shaped rock (Fig. 4).

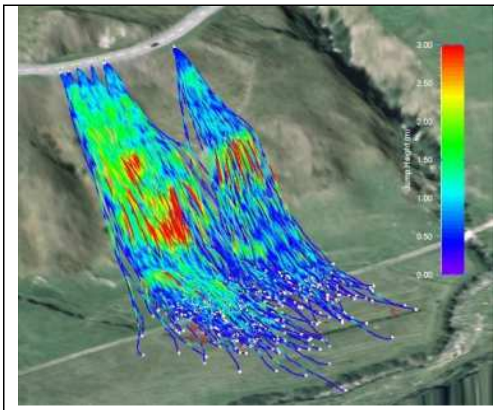
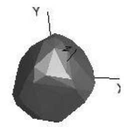
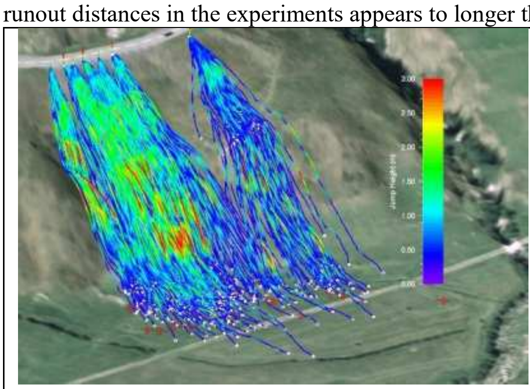


Figure 3: Simulation of the Tschamut grass slope with 700 equant shaped rocks with volume 0.028 m3. The coefficient of restitution in the simulations is $\boldsymbol{\varepsilon} = 0.0$.



In good agreement with the observations, the calculated jump heights in the runout zone are all less than h < 1.5m. Comparison between calculated (white dots and measured (red crosses) runout locations.

Different rock shapes are ideal to parameterize ground conditions because the statistical distribution of runout positions must depend only on the parameter that varied in the experiment, in this case rock shape. In general we find that the equant shaped rock led to longer runout distances, both experimentally and in the numerical computations with the hard contact model. For the equant shaped rocks the lateral dispersion, jump heights and runout distribution appears to be correctly modelled (Fig. 3). We emphasize that only 700 rocks were used in the numerical computations. Similar results are obtained for the platy rocks (Fig. 4); however, the mean runout distances in the experiments appears to longer than the mean runout of the calculations.



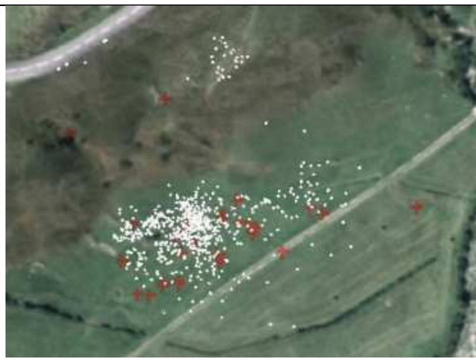


Figure 4: Simulation of the Tschamut grass slope with 700 rock plates. shaped rocks with volume 0.016 m3. The coefficient of restitution in the simulations is $\boldsymbol{\varepsilon} = 0.0$.

Comparison between calculated (white dots and measured (red crosses) runout locations.

3 CONCLUSIONS

Physics based rockfall models have been developed that will employ “real” (i.e. measured) restitution coefficients. Runout distances, jump heights and lateral dispersion can be modelled with these real coefficients; however more work remains in parameterizing the shear forces which govern the transformation of rotational energy into translational energy. This transformation governs the transition from rolling to bouncing or from bouncing to rolling. More experiments with different rock shapes and in different terrain are planned to investigate this process in the near future.

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Non-smooth contact mechanics exposed: Detailed insights of rock-ground interactions

Werner GERBER[1], Andrin CAVIEZEL[2]

Keywords: Rockfall, impact kinematics, non-smooth contact, field experiments

In this work, a rockfall trajectory extracted from high speed video analysis is examined with focus on the rock ground interaction. Investigation of the trajectories and modelling of the impact processes yield information about the acceleration and velocity distributions during the transition. The presented data shows a more accurate treatment of rock-ground interactions than simple rebound physics.

1 ANALYSIS OF ROCK-TERRAIN INTERACTIONS

Existing rigid-body rockfall models rely mostly on simple rebound physics where coefficients of restitution govern the pre- and post-impact velocity vector. These coefficients hence define the key parameters such as the jump direction, height and length. In rebound models the entire rock ground interaction is contracted to a single point which does not allow for sliding or rolling during the contact phase. However, numerous parameters govern the bouncing phenomena such as slope and rock characteristics as well as the incident kinematical configuration (Labiouse, 1996). Therefore, the reliability of simulated rockfall trajectories depends on the calibration of these rebound parameters (Bourrier, 2015). Detailed modelling of ground penetration, sliding and rotation require rigid body or discrete element approaches. Such non-smooth contact/impact models handle the impact via parameterization of ground friction (Leine, 2014).

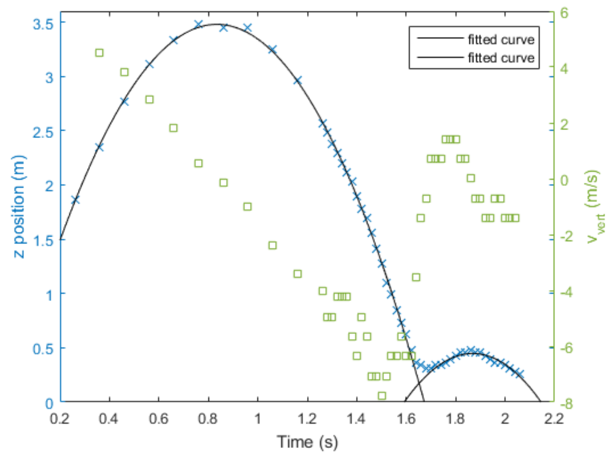

Fig. 1: Measured vertical positions as well as velocities during flight and impact with a parabola fitted to both airborne phases.

Here, field experiment data are presented revealing an impact process distributed in time. Analysis of the acceleration and velocity vectors unravels part of the impact dynamics. The experiments took place at Tschamut, GR, (Switzerland) with an ellipsoidal rock of dimensions 0.5x0.39x0.3 m and a weight of 79 kg. The ground material is a soft soil typical of Swiss alpine meadows with a slope angle of 27 degrees. The rockfall trajectory is recorded by a high speed camera with 50 frames/s. By analysis of the footage the kinematical parameters in 2D are determined.

Figure 1 depicts the extracted vertical positions as well as velocities during flight and impact. Equivalent data points exist for the horizontal directions. A parabola is used to define the in-flight phase. Deviations from these parabolas mark the beginning and end of the contact phase and are set to the time stamps ranging from 1.58 s to 1.70 s resulting in a ground contact time of 0.12 s. The deceleration process during the contact is studied in greater detail. The fitting supplies the coordinates and velocities in horizontal and vertical directions at the start and end point of the impact. The rock travels with absolute velocities of 9.21 m/s before, and 7.43 m/s after the impact respectively. The vertical direction of our coordinate frame is defined parallel to gravity. Table 1 summarizes these values which serve as boundary conditions of the impact treatment. In the horizontal direction the impact results in an increased velocity of 1.68 m/s, vertically the rock is decelerated to 0.0 and leaves the ground again with a velocity of 1.59 m/s.

Under the assumption of Gaussian pulse shapes effective accelerations in both directions are introduced such that the boundary conditions are fulfilled (Fig 2(a)). Partial integration combined with a linear least-squares fitting yields the

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Table 1

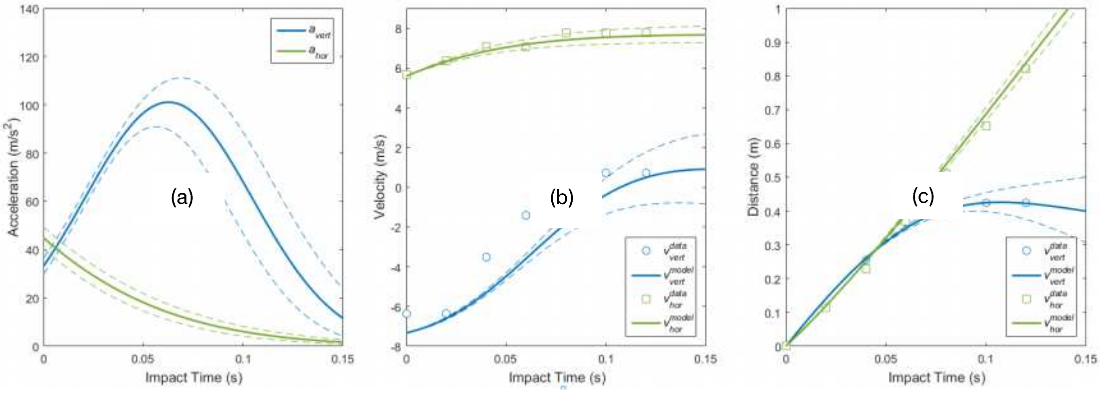

Fig. 2: (a) Experimental accelerations under the assumption of a Gaussian shape determined by minimizing the differences to the measured velocities (b) and distances (c) during the impact.

velocities plotted in Fig 2(b) and finally the travelled distances in both coordinate directions depicted in Fig. 2(c). Calculation of the directions and magnitudes of both horizontal and vertical accelerations form the plot shown in Fig. 3(a). Figure 3(b) depicts the effective travelled distances of the centre-of-mass during ground contact (red) with the acting accelerations which finally return the rock to its second flight phase after the impact (blue). Note, that the obtained distances are to be understood within the defined coordinate system. The effective impact scar has a length of roughly 90 cm with a depth of 15-20 cm.

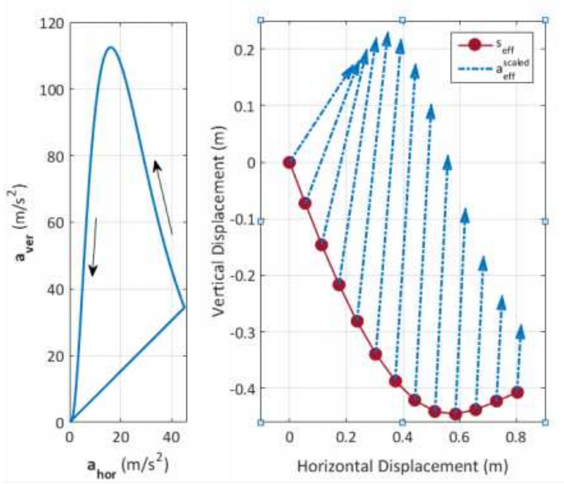

Fig.3: (a) Acting accelerations and (b) Effective translation and acting accelerations (scaled) during impact.

The calculated impact accelerations heavily depend on impact angle, surface roughness, and rock shape. The presented fitting routine only focuses on translational parameters and therefore does not consider the influence of the rotational modes. Future experiments and analyses have to be extended to rotational degrees of freedom. A time-resolved picture including information about all of the six degrees of freedom of motion would present a valuable calibration foundation for currently used non-smooth models.

2 CONCLUSION

Experimental data of rock fall impact are evaluated in order to identify ongoing processes during the rock/ground interaction. The data not only shows the acting forces, respectively accelerations during the impact but also the notion of a finite ground contact time. It becomes obvious that simple rebound models which reduce the contact to a single time point are a drastic oversimplification. Hence, in order to map realistic impact mechanics non-smooth impact models are favourable. Such models bear the possibility to track and simulate sudden changes in the trajectory owing to irregular rock shapes and thus complicated ground interactions. Such analysis should allow the separation of the complex rock ground interactions such as slipping, scarring, rebound and rolling.

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Assessment of rockfall hazard in open-pit coal mines: practical application

Federica FERRARI[1], Anna GIACOMINI[2], Klaus THOENI[3] & Cedric LAMBERT[4]

Keywords: in-situ evaluation, rockfall frequency, rockfall energy, impact distance, qualitative method

Rockfalls pose serious threats to workers, machineries and structures in open-pit mines, despite that no standard methods exist to deal with rockfall hazard in coal mining environment. For this reason, a new rigorous methodology was specifically developed for defining different hazard levels at the toe of Australian coal highwalls. The methodology, named Evolving Rockfall Hazard Assessment (ERHA), involves a first qualitative assessment for the identification of the most hazardous areas, where a second more robust quantitative analysis is necessary. This paper summarizes the qualitative step of the methodology, which can be applied for a quick in-situ survey and hazard classification, and shows a practical application.

1 QUALITATIVE EVOLVING ROCKFALL HAZARD ASSESSMENT

The qualitative ERHA is a simple and quick tool capable of identifying the most dangerous highwall sections from in situ observations. According to the Swiss guidelines on rockfall hazard assessment (Raetzo et al., 2002), the methodology defines three hazard levels (i.e., low, medium and high) as function of a probability-intensity matrix (Figure 1). Both rockfall probability (i.e., frequency) and intensity are subdivided into three classes. Within the qualitative ERHA, the rockfall frequency is evaluated through the state of activity of the highwall, which describes how much a highwall is prone to block detachment. High, medium and low state of activity classes indicate that frequent, occasional and few rockfalls are expected, respectively. The rockfall intensity is described by means of the translational energy expected at the base of a highwall. The thresholds for intensity classes were chosen according to the impact resistance of PPE helmet (AS/NZS, 1997), machinery (ISO, 2005) and concrete portal structures. Therefore, the energy thresholds are: 0.05 kJ (lower limit), 11.6 kJ, and 300 kJ. Intensity values greater than 300 kJ are included in the high hazard level. The combination of state of activity and intensity classes gives the hazard level. Besides the hazard levels, the methodology provides information about the expected distance of the first impact at the base of the highwall.

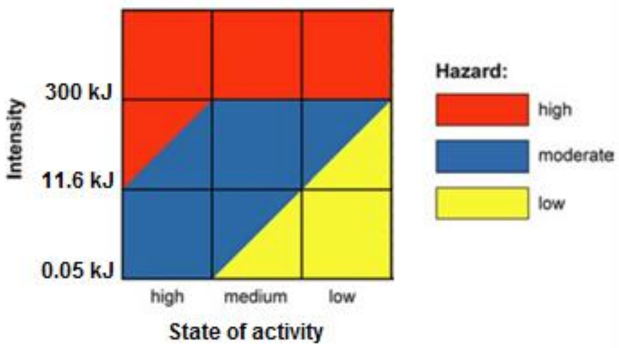

Figure 1: Matrix for defining hazard levels

ERHA was implemented into a web app (freely available on erha.newcastle.edu.au after setting up a personal account), which generates hazard zoning maps based on in-situ observations.

1.1 STATE OF ACTIVITY

The state of activity of a highwall is defined by means of rapid in situ observations, using a rating based approach. The approach considers the geological structure of the rock mass, the potential instability mechanism (due to undercutting, block toppling and/or sliding), the slope performance (deviation from design) as well as signs of recent block detachments. All these parameters can be easily observed and quickly rated (see example in Section 2). The resulting value defines a probability class (i.e., high, medium or low).

1.2 ROCKFALL INTENSITY

The estimate of energies at the base of the highwall is based on results of a sensitivity analysis, which was carried out by performing 78,400 stochastic simulations with the software Rocfall 4. The sensitivity analysis was carried out by systematically

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varying the slope height, the slope angle, the surface roughness, and the outcropping material properties (in terms of normal and tangential restitution coefficients). Regression coefficients were inferred in order to relate the slope geometry and block volume to the expected kinetic energy at the base of the highwall (Ferrari et al., 2016).

2 APPLICATION

The qualitative step of ERHA can be applied using either the single block approach or the geological domain approach. In the former, the most dangerous potential unstable block is identified on the highwall and the hazard is evaluated, using the own block dimensions and location. In the latter, the highwall is subdivided into geological homogenous domains and the hazard is calculated for each domain, according to its representative features.

An example of hazard assessment by means of the single block approach is reported in the following. The analysed highwall (Figure 2) has an average slope angle of 80° and high roughness. The hazard is given by a potential unstable sandstone block, which has a mass of about 1200 kg and is located 30 m above the toe of the highwall. The block could fail due to undercutting. The rock mass has a blocky structure. Several signs of already fallen blocks are observable. From these observations, the Web App automatically calculates the expected hazard level, the energies and distances at the base of the highwall (Figure 3).

Figure 2: Application: highwall with a potential unstable block

Figure 2: Application: highwall with a potential unstable block

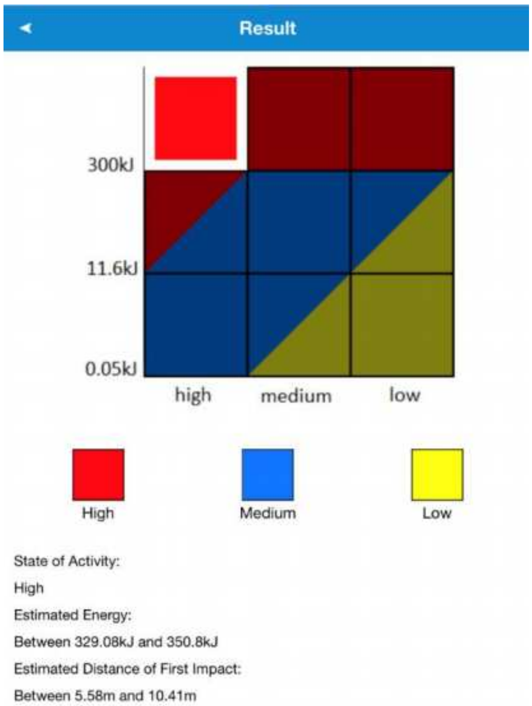

Figure 3: Application: results from the web app tial unstable block

3 CONCLUSIONS

The qualitative ERHA allows for a quick identification of hazard levels at the bottom of highwalls from simple in-situ observations. ERHA provides practitioners with a rigorous and effective approach for evaluating and managing the rockfall hazard in open-pit mines. In particular, ERHA gives a greater confidence in locating personnel, machineries, and structures over the working areas at the base of highwalls. The ERHA methodology has been specially developed for rockfall hazard assessment in open-pit mines and it shows a great potential for a quick qualitative hazard assessment, by providing indication about impact energy and horizontal distance of the first impact at the base of a rock cliff. The approach can be used for both natural and constructed slopes and it can be easily extended to more complicated and articulated slope geometries.

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Measuring rockfall motion

Axel VOLKWEIN[1], James GLOVER[2]

Keywords: rockfall, testing, trajectory, measurements

1 INTRODUCTION

Rockfall is a fascinating process to observe with dynamic combinations of translational movements, rotation and jumps. To quantify these highly dynamic elements of the process, innovative measurement solutions are required. It is the benefit of such detailed measurements of rockfall dynamics that numerical rockfall simulation software can be calibrated and/or validated. Furthermore, the design of protection barriers can rely on fully dynamic loads rather than quasi-static extrapolations or static equivalent loads. Different approaches to detect accelerations, the spatial rotation and the rock's trajectory are presented in this abstract.

2 MEASURMENT POSSIBILITIES

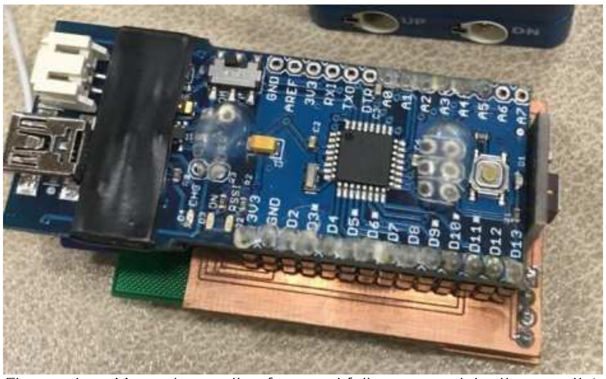

Figure 1: Measuring unit of a rockfall sensor detecting spatial accelerations and rotational velocity. over time.

Acceleration: The development of effective "in-block" measurement systems has benefited from advances in micro sensor technology in recent years. One of the first applications of accelerometers for rockfall testing was Grassl (2002), who installed eight acceleration ±50g sensors arranged in three orthogonal directions sampling at 2 kHz into a spherical block. It was used to measure the deceleration process within flexible rockfall protection barriers which experienced a maximum deceleration of <20g. The spatial arrangement of the sensors within the sphere also allowed the determination of the actual spatial orientation. A temporary wire connection charges the battery of the system, activated the measurements and allowed the post-event data retrieval. The measurement was triggered automatically after 0.5 secs of free fall.

Another possibility to measure accelerations of one-dimensional accelerations is a wired connection between on-block acceleration sensors and external data retrieval (Schellenberg, 2008; Gerber & Volkwein, 2010). However, sensor units are prone to damage due to the exposed cable connectors, and one is limited to simple 1-D impacts. The development of small and effective accelerations sensors (see Fig. 1) – of course mostly pushed by the evolution of smart phones in the past decade – enables the installation of compact on-block measurement units as for example used by Lambert (2007).

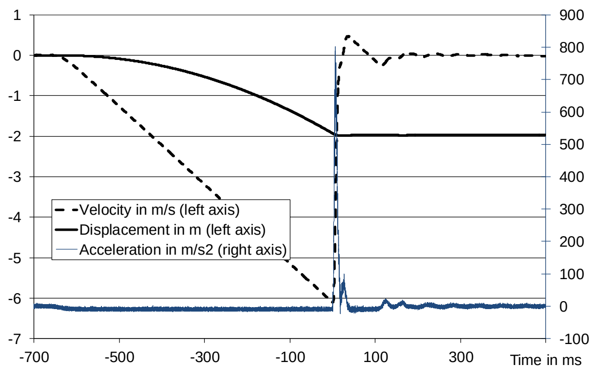

Figure 2: Impact test of a 4000kg block on gravel being dropped from 2m height: measured vertical acceleration with velocity and displacement obtained through integration over time.

Position and trajectory: To determine the spatial trajectory with velocity and position of a block the accelerations can be double integrated (for example Fig. 2). A pure forward integration of a block's accelerations always results in a significant drift. However, this procedure works well for mostly one-dimensional movements e.g. a vertical drop. In this case, the boundary conditions at the start and end of the process (block's not moving) are known and can be used to eliminate the unavoidable progressive error resulting from the integration over longer time intervals. For spatial movements the acting accelerations also have to be corrected according to the current orientation of the block in relation to the current direction of gravity:

Rotation: The block's current spatial orientation can be detected by the use of an additional gyroscope sensor e.g. Caviezel & Murri (2013) and Glover (2015). Measuring up to 2000 deg/secs and with sampling rates of 20 kHz and 600 Hz, respectively. The integration of the orientation delivers the rotational velocity of the block that allows the determination of the rotational part of the block's total kinetic energy (see Fig. 3). Volkwein & Klette (2014) and Sabathy & Malacarne (2013) directly measured the angular velocity up to 2000 deg/secs using a different type of sensor with

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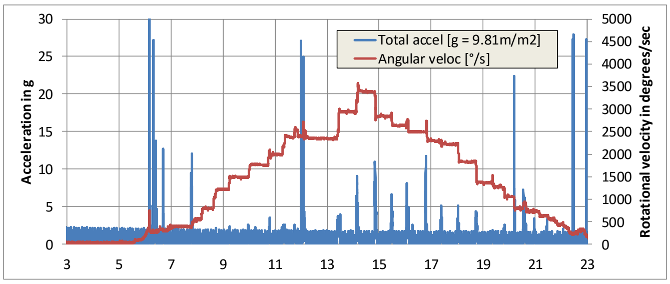

Figure 3: Rockfall test in the field: the graph shows in blue the measured accelerations resulting from the single impacts (left axis in g) and in red the angular velocity of block over time (right axis in deg/s).

a sampling rate of about 1.0 kHz. To obtain the current orientation the sensor's data have to be differentiated. However, the typical sensor noise reduces the usability of the results.

Video and tracking: The position of the falling rock in the field over time is traditionally retrieved through video records with a sufficient precision (Volkwein et al., 2012). However, video post-analysis is very time consuming. Special attention to the conversion scale between pixels and physical distances must be made. It can be set based on reference distances placed in the field with known preset reference points. Another option is to adjust the scale in a way that the vertical component of a rock jump over time results in a free flight parabola with the vertical acceleration that corresponds to g (9.81 m/s^2). This approach also produces reliable result even if the camera view is not orthogonal to the plane of the rock's trajectory.

The position in the field during a rockfall test can also be tracked through GPS (Sabathy & Malacarne, 2013) or a specially setup localisation system (Volkwein & Klette, 2014). Both systems record the block's trajectory with 10-15 points per second and allow for an estimation of the translational velocities. However, the systems do not report jump heights. Tests have been conducted with using a barometric sensor to track the block's elevation over time (Glover, 2015). However, the air pressure around the rock falsifies the measurements, in particular, where the sensors are placed inside a hole in the rocks and the rocks are rotating.

3 APPLICATION IN THE FIELD

Above measurement systems can be used to produce field data that in turn are valuable to calibrate or validate rockfall trajectory simulation software regarding the predicted impacts, rotational velocities, translational energy, run out etc. Their installation usually requires a drilled opening with a diameter of 40-70 mm and a depth of 10-25 cm, while with advances in micro-sensor technology it is expected that smaller diameter holes can be used in the future; test blocks might even be re-usable. An excentric installation of the sensor might result in centrifugal accelerations during rotation. Single rock blocks enhanced with sensor techniques and manually released during rock face scaling operations can also deliver valuable experimental data that will be shown in a full contribution regarding all specialities even a single trajectory can provide. Furthermore, the results of repeated testing allow for a corresponding statistical analysis. Another usage for in-block sensors are the tests with protection measures. In all cases, a robust sensor unit has to be used because loads of 500g easily can be reached during hard impact.

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The effect of forests on rockfall occurrence frequency

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Keywords: Rockfall, release probability, magnitude, power-law, protection forests, risk analysis

Many forests in mountain regions protect people, settlements and transportation corridors from rockfall and play an important role in risk prevention. The protective effect of forests can be grouped into three types: firstly, the prevention or reduction of the initial mass that is released in the natural hazard process, secondly, the barrier effect against the released mass and thirdly, negative effects of the presence of trees on the natural hazard process. Depending on the process, the three above mentioned effects of forests can affect natural hazard risks in various ways. In the case of rockfall, forests can influence the occurrence frequency of an event at the element at risk, its intensity and thus the extent of damage, and finally the probability of the event actually impacting the elements at risk (spatial probability of occurrence) (Wasser and Perren, 2014). Despite recent advances regarding forest-rockfall interactions, open questions still remain, namely on how these mitigating effects of forests can be quantitatively integrated into rockfall risk analyses. The quantification of the influence of forests on rockfall occurrence frequency is particularly demanding. The goal of this study is to quantify the effect of forests on the occurrence frequency based on rockfall simulations with the three-dimensional, process-based model Rockyfor3D (Dorren, 2015).

1 METHODS

1.1 MODELLED ROCKFALL MAGNITUDES AND FREQUENCIES

We defined a rockfall release probability based on a power-law magnitude-frequency distribution (Eq. 1), which serves as input for the simulation of rockfall events over a period of 1000 yrs on a virtually constructed slope. The exponent of the power-law distribution (b) was derived from literature (Carrea et al., 2015).

$$F(V_i) = F_0 V_i^{-b} \qquad (1)$$

where $F(V_i)$ is the frequency of volume i (V_i) and F_0 the expected number of rockfalls per year (here set to 12).

The minimum volume of the simulated single rock is 0.05 m^3, the maximum volume is 2.0 m^3. The simulations are conducted for different forest and non-forest scenarios under varying terrain conditions. These simulations provide input data for the determination of rockfall occurrence frequencies at five different evaluation zones situated at 0, 150, 300, 450, and 530 m from the downslope part of the release area.

1.2 STATISTICAL ANALYSIS

For each volume class and simulation scenario, we summed up the number of rocks passed per cell (number of passages) for the five evaluation zones. We calculated the rockfall occurrence frequency (Freq) by dividing the sum of the number of passages (Nrp) by 1000 yr. The return period (RP) is its reciprocal value. Based on multivariate statistical models, we assessed how specific forest and terrain characteristics control the rockfall occurrence frequency along a slope. We related the reduction in Nrp under forested compared to non-forested conditions to terrain and forest variables using linear regression models as well as regression trees (Breiman et al., 1984). The following explanatory variables were considered: block volume, soil type, soil roughness, cumulative basal area (basal area x slope length; cbA), mean tree diameter (dbh), mean number of trees per ha (Nha), horizontal forest structure (“homogenous”, “clustered”, “aisle”, “gaps”) and the percentage of conifers.

2 RESULTS

The rockfall occurrence frequency is significantly reduced under forested compared to non-forested conditions. For a 0.5 m^3 block, for example, the occurrence frequency (expressed as a return period in years) at a distance of 530 m from the release area changes from 30 yrs on a non-forested to more than 1000 yrs on a forested slope (Figure 1). The difference in the frequency increases with increasing distance from the release area and is less pronounced for larger block volumes ($\geq 1.2\ m^3$).

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Furthermore, the distance from the release area required to significantly reduce the occurrence frequency increases with decreasing stem density and decreasing tree diameter.

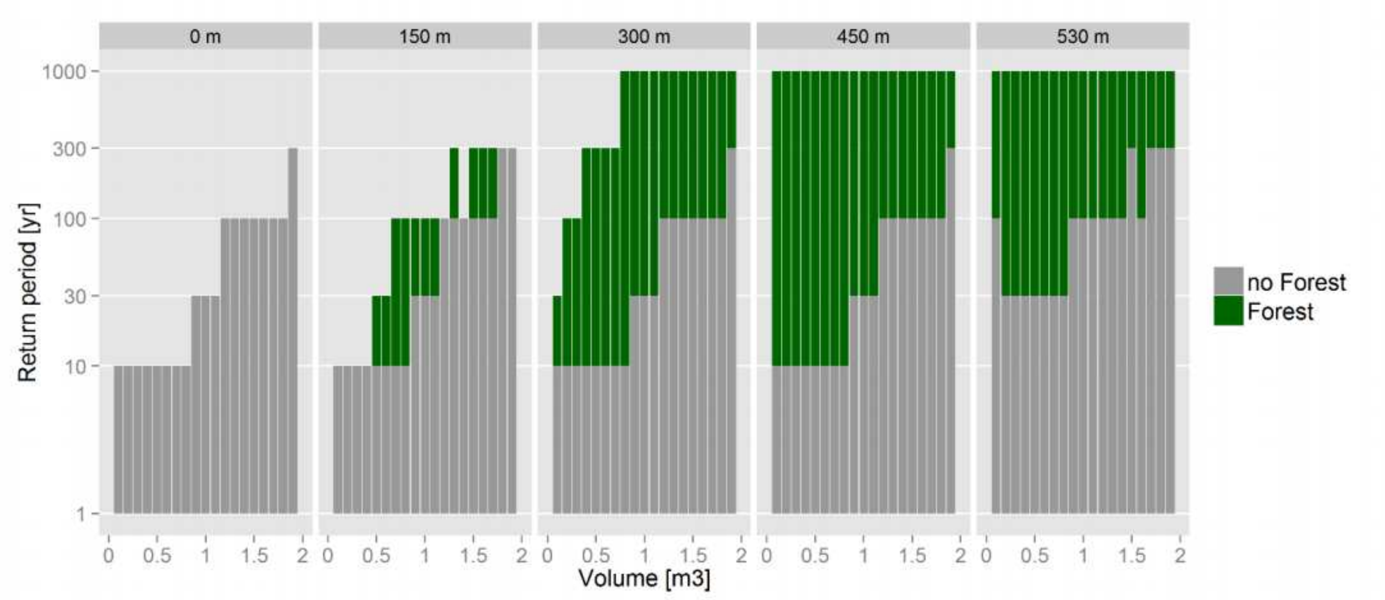

Figure 1: The effect of forests on the return period (y-axis) of different rockfall volumes (x-axis) at 5 different distances from the release area.

3 CONCLUSION

The results of this study demonstrate that forests can significantly reduce the rockfall occurrence frequency at elements at risk. This also applies to relatively short distances from the release area (~ 150 m) and for relatively open forests (~ 200 trees/ha). The statistical analyses allow quantifying the effect of specific forest and terrain characteristics on the reduction of the occurrence frequency. For example, a forested slope length of 100 m with a dense forest (basal area = 50 m^2/ha) is required to reduce the occurrence frequency of blocks < 1.0 m^3 with 70 %. Based on these results, conclusions can be drawn for real rockfall slopes. They further provide a quantitative basis for determining the effect of forests on rockfall risks.

4 REFERENCES

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