Interdisciplinary Workshop on Rockfall Protection

Co-sponsored by

IABSE

Organised by

Axel Volkwein
WSL Swiss Federal Institute for Forest, Snow and Landscape Research

Vincent Labiouse
Swiss Federal Institute of Technology Lausanne

Kristian Schellenberg
Swiss Federal Institute of Technology Zürich

WSL

ÉCOLE POLYTECHNIQUE FÉDÉRALE DE LAUSANNE

ETH
Eidgenössische Technische Hochschule Zürich
Swiss Federal Institute of Technology Zurich

Preface

Compared to other landslide movements, rockfall is an extremely rapid process with long travel distances. When an event occurs, the ability of persons to take evasive actions is almost reduced to zero and the risk of injury or loss of life is high. Damages to buildings and infrastructures are likely as well. In many cases suitable protection measures are thus necessary.

The workshop brings together international researchers and practitioners to discuss and evaluate existing and new findings, methods and results that are relevant for a proper protection against rockfall. Furthermore, the workshop aims to discuss and evaluate new visions and ideas for future rockfall research. It is therefore valuable for people involved in projects on rockfall protection of infrastructure, working in the fields of civil or environmental engineering, risk and safety, earth and natural sciences.

When a risk analysis related to possible rockfall hazards reveals a threat to people, buildings or infrastructures, suitable structural protection measures have to be selected according to the expected event frequency and impact energies. For proper design and dimensioning of the measures, it is essential to know the magnitude of the impact loads and the performance of the structures, such as walls, fences, galleries or dams. This knowledge can be obtained from numerical simulations, experiments, models or existing guidelines.

However, rockfall protection considerations do not only comprehend structural protection measures. At first it has to be clarified whether, why and where rocks are released and to which amount. The rockfall initiation depends also on different mostly not yet quantified factors such as weathering, frost and melting cycles or heavy rainfall. Subsequent trajectory analyses determine the areas that have to be protected by additional measures. To account for their high sensitiveness to only small changes in the landscape such as bedrock, dead wood, small dips etc, statistical analyses of rockfall danger are usually performed, at best including a plausibility analysis on the accuracy of the results.

It is therefore the aim of the workshop to deal with the methods and measures that are used to protect from rockfall effectively and efficiently. The workshop will therefore focus on existing and new protection approaches that are evaluated and discussed regarding their performance, reliability, validation, extreme loads etc. Contributions to this topic cover different aspects such as rockfall danger, structural protection measures, tests, modeling and simulations.

A. Volkwein
V. Labiouse
K. Schellenberg

Scientific Committee

***T. Vogel**, Switzerland, Chair*
***A. Volkwein**, Switzerland, Vice chair*
***K. Schellenberg**, Switzerland, Secretary*

***N. Banthia**, Canada*
***E. Cadoni**, Switzerland*
***L.K.A. Dorren**, France*
***M.H. Faber**, Switzerland*
***M. Jaboyedoff**, Switzerland*
***N. Kishi**, Japan*
***T. Krauthammer**, USA*
***V. Labiouse**, Switzerland*
***M. Langseth**, Norway*
***H. Masuya**, Japan*
***J. Mazars**, France*
***T.-C. Pan**, Singapore*
***Y. Sonoda**, Japan*
***S.M. Springman**, Switzerland*
***F. Toutlemonde**, France*
***B. Wang**, Canada*
***J. Weerheijm**, Netherlands*

Main Financial Sponsor

isofer ag
conveying, lifting, securing

Table of Contents

QUANTITATIVE SPATIAL DISTRIBUTION OF BLOCK FALLS USING GIS-TREE DECISION MODELS

Chadi Abdallah[1], Rania Bou kheir[1], Damien Dhont[2]

SUMMARY

Block fall are considered a significant aspect of surficial instability contributing to losses in land and socio-economics through their damaging of the natural and human environments. This paper focus on building simple, realistic, practical and informative decision tree - regression models for a quantitative mapping of the volume of block falls (m^3) using the predefined terrain parameters. This work was applied on a study site within Lebanon (762 km^2, 8% of the Lebanese territory) using remote sensing and GIS. Satellite imageries helped in discriminating existing block falls, while GIS was used to extract ordinal (slope, curvature, etc.,) and nominal (lithology, karst, etc.,) variable terrain parameters. Decision-tree models were constructed based on different input terrain parameters: (1) all terrain parameters, (2) topographic parameters only, (3) geologic parameters only, and adopting various processing techniques (pruned and unpruned trees). The most powerful model proved to be the regression unpruned (exploratory) tree-model based on all considered parameters explaining correctly 86% of the variance in the trained data. Once pruned, this model classifies 50% in block falls volumes by selecting just four parameters (lithology, slope gradient, soil and land cover/use). The highest predictive decision-tree model was then converted to a quantitative 1:50,000 block falls' map with six classes; starting from Nil (no block falls) to more than 4000 m^3. This map can be used to prioritize the choice of specific zones for further measurement and modeling. It is extremely useful fitting management needs and helping in the adoption of measures to reduce the occurrence of harmful block falls, specifically in 18% of the studied area with block falls' volumes exceeding 2000 m^3.

Key Words: Block falls, Quantitative mapping, GIS, Regression models

INTRODUCTION

Despite of the danger effect of block falls, most worldwide studies have been done on landslide research, and there is a little on evaluating other types of Mass Movements (MM), and especially block/rock falls [1]. A GIS decision-tree method is tested to produce a volumetric map (volume in m^3) of block falls, which is considered as a machine learning, probabilistic and non-parametric method. The decision-tree method has been extensively exploited for vegetation mapping [2], ecological modeling [3], gully erosion modeling, soil mapping and in remote sensing studies (e.g. land use classification based on threshold values of various band data) [4]. However, its use in predictive estimate of the block falls is still at its early stages. In this context, this work focuses on building decision-tree models for quantitative mapping of block falls in a study site within Lebanon. The constructed map describes the potential size of block falls (volume in m^3) in each location depending on field measurements of existing ones (what is called predictive mapping). The GIS decision-tree models comprise a set of rules to classify (predict) a dependent target

[1] National Council for Scientific Research/Remote Sensing Center, P.O. Box 11-8281, Beirut, Lebanon.Tel: +961 4 409 845/6; Fax: +961 4 409 847; Email: chadi@cnrs.edu.lb
[2] Université de Pau et des pays d'Adour CNRS-FRE 2639: Imagerie Géophysique/CURS-IPRA, Avenue of the University, 64013 Pau cedex France. Tel: +33 5 59 40 74 25 Fax: +33 5 59 40 74 15; Email: ddhont@univ-pau.fr

variable using the values of independent variables (predictors). The predictor variables are both ordinal (elevation, slope angle, slope aspect, slope curvature, proximity to fault line, distance to the drainage line, proximity to roads) and nominal (lithology, karst type, soil type, land cover/use). It is important to mention that the selected variables were defined in earlier work for obtaining landscape unit, building susceptibility and hazard maps for the same area [5]. The landscape unit is the foundation on which the results of decision-tree models are converted to predictive volumetric map of block falls.

WORK FLOW

Mapping the volumetric size of block falls was conducted in several steps: 1) Field measurement of dimensions (volume) of existing block falls (target dependent variable) detected through visual interpretation of satellite imageries [6]; 2) intersect the block fall layer information (point location) with the maps of predictor independent parameters prepared under GIS environment. The result of this intersection was a layer of the block fall locations and the corresponding parameters in an attribute table related to it. This table was saved in an ASCII format for software considerations; 3) perform on this ASCII file several decision-tree models; and 4) convert the model that explains the highest variance in block falls to a predictive volumetric map of block falls under GIS environment and using landscape analysis.

RESULTS AND DISCUSSION

Sixty-two block falls were found in the field, ranging from 234 m^3 to 5436 m^3, with a mean volume being equal to 2139 m^3 and a standard deviation of 1959. Most of them occur under following conditions: on elevations ranging between 500 and 1000 m (37% of block falls), with moderately steep slopes oscillating between 12 and 18° (48%), facing southward (16%), in concave (50%) forested areas of coniferous type (21%), growing over highly fissured and joined dolomites and dolomitic limestone of the Jurassic formation (24%), mixed soils on alternating marls, limestone and sandstone (48%), of highly faulted terrain and non karstic (79%). The block falls density is 0.08 events per km^2. Seven decision-tree models were constructed in the context of this study based on different input terrain parameters and processing techniques (pruned and unpruned). The most powerful one was the regression unpruned (exploratory) tree-model based on all considered parameters explaining correctly 86% of the variance in the trained data. Once pruned, this model classifies 50% in block falls volumes by selecting just four parameters (lithology, slope gradient, soil and land cover/use). The unpruned model built using only 4 geological parameters (lithology, soil type, proximity to fault line, and karst type) seems interesting, since it is created using similarly 4 parameters, but it has a higher predictive accuracy (68%). The relative importance of the predictor parameters in building those trees (model 1a &1b, model 2a & 2b, model 3a & 3b) and splitting corresponding nodes is shown in Tab.1.
Finally and depending on the obtained decision-tree models results, the preferred unpruned (model 1a, highest predictive power, classifying 86% of the data correctly) was used in order to establish the volumetric map of block falls in the chosen studied site at 1:50,000 cartographic scale, in addition to the pruned using only 4 parameters (model 1b).

CONCLUSION

The produced predictive quantitative block falls' maps at a scale of 1:50,000 can be used to prioritize the choice of specific zones for further measurement and modeling. It is extremely useful fitting management needs and helping in the adoption of measures to reduce the occurrence of harmful block falls, specifically in 18% of the studied area with block falls' volumes exceeding 2000 m^3.

Tab. 1 Summary characteristics of built regression tree-models [1 - based on all parameters, 2 - based on topographic parameters only and 3 - based on geologic parameters only].

Predictor variables (%)	**Model 1a (exploratory tree)**	**Model 1b (pruned tree)**	**Model 2a (exploratory tree)**	**Model 2b (pruned tree)**	**Model 3a (exploratory tree)**	**Model 3b (pruned tree)**
Elevation	0%*	0%	18%	0%	N	N
Slope gradient	70%	78%	100%	100%	N	N
Slope aspect	11%	0%	70%	0%	N	N
Slope curvature	0%	0%	50%	0%	N	N
Lithology	100%	86%	N**	N	100%	100%
Proximity to fault line	50%	0%	N	N	22%	0%
Karst type	10%	0%	N	N	10%	0%
Soil type	61%	40%	N	N	41%	49%
Distance to drainage line	0%	0%	N	N	N	N
Land cover/use	60%	50%	N	N	N	N
Proximity to roads	6%	0%	N	N	N	N
Proportion of variance explained	**86%**	**50%**	**57%**	**19%**	**68%**	**45%**
Number of total nodes	23	7	31	3	27	5
Number of terminal nodes	12	4	15	2	14	3
Minimum cost error – node number	-	0.07 – node 3	-	0.8538 – node 2	-	0.8057 – node 3

* = relative importance of predictor variables; N = not included in building the model

REFERENCE

[1] Guzzetti, F., Reichenbach, P., Ghigi, S. (2004) Rockfall hazard and risk assessment along a transportation corridor in the Nera Valley, Central Italy. *Environmental Management*, 34(2), 191–208.
[2] Franklin, J. (1998). Predicting the distributions of shrub species in California chaparral and coastal stage communities from climate and terrain-derived variables. *Journal of Vegetation Science*, 9, 733-748.
[3] Michaelsen, J., Schimel, D.S., Friedl, M.A., Davis, F.W., Dubayah, R.C. (1994). Regression tree analysis of satellite and terrain data to guide vegetation sampling and surveys. *Journal of Vegetation Science*, 5, 673-686.
[4] Friedl, M.A., Brodley, C.E., Strahler, A.H., 1999. Maximizing land cover classification accuracies produced by decision trees at continental to global scales. *IEEE Transactions on Geoscience Remote Sensing*, 37, 969-977.
[5] Abdallah, C., Chorowicz, J., Bou Kheir, R., Khawlie, M. (2005). Detecting major terrain parameters relating to mass movements' occurrence using GIS, remote sensing and statistical correlations, case study Lebanon. *Remote Sensing of Environment*, 99, 448-461.
[6] Abdallah, C., Chorowicz, J., Bou Kheir, R., Dhont, D. (2007). Comparative use of processed satellite images in remote sensing of mass movements: Lebanon as a case study. *International Journal for Remote Sensing*, 28:19, 1 - 20.

INTEGRATING ROCKFALL RISK SCENARIO ASSESSMENT AND COUNTERMEASURE DESIGN BY 3D MODELLING TECHNIQUES

Federico Agliardi[1], Giovanni B. Crosta[1], Paolo Frattini[1]

Rockfall risk analysis for mitigation action design requires defining rockfall hazard, the spatial probability and intensity of impacts on structures, their vulnerability, and the related expected costs for different scenarios. We integrated these tasks by using the 3D mathematical model HY-STONE. We discuss the case study of Fiumelatte (Varenna, Italy), where a large rockfall in November 2004 caused 2 deaths, the destruction of several houses and damage to transportation corridors. We calibrated a 3D rockfall numerical model by back analysis of the 2004 event, and we performed simulations for the whole area at risk by considering scenarios without protection (0), with a provisional embankment (1), and with a planned long-term protection embankment (2). The spatial distribution and energy of impacts against each exposed building have been computed and used to derive the degree of expected damage according to its vulnerability. Then, the costs and benefits associated to different scenarios and rockfall probabilities have been estimated. Our approach allows assessing both the technical and the cost efficiency of different mitigation options, and shows the possibility to adopt single modelling tools to achieve quantitative evaluation of rockfall risk.

Keywords: Rockfall, scenario, hazard, impact, risk, optimisation

INTRODUCTION

Rockfalls are widespread phenomena threatening human beings and causing significant damages to structures. Thus, rockfall protection of elements at risk is important for administrators and stakeholders. Rockfall protection includes different tasks, namely: rockfall risk assessment for land planning and prioritization of actions; identification of mitigation options able to achieve a specified risk reduction; and evaluation of their cost efficiency to optimise budget and design. Accomplishing these tasks involves assessing: (1) rockfall hazard over the affected area; (2) the distribution and intensity of impacts on exposed structures; (3) the vulnerability of impacted structures; (4) the expected cost of specific risk scenarios.

We tried to integrate all the relevant stages of the rockfall protection process in a comprehensive approach, taking advantage of 3D mathematical modelling tools, that provide useful insight and tools for the analysis of rockfall process and related risk [1, 2, 3, 4, 5]. We present the case study of Fiumelatte (Lecco, Italy), where a large rockfall caused 2 casualties, the destruction of several houses and the interruption of roads and railway [6]. This area has been recurrently interested by a series of small rockfall events. Following the disaster, the authorities in charge built a first emergency protection embankment, and started the design of a longer and higher embankment to ensure long-term protection. The aim of our analysis is to verify the technical suitability and the cost-efficiency of the countermeasures through both mathematical modelling and cost-benefit analysis.

[1] Dipartimento di Scienze Geologiche e Geotecnologie, Università degli Studi di Milano-Bicocca, Piazza della Scienza 4, I-20126 Milano, +39 02 644820-06 (fax: -73), federico.agliardi@unimib.it; giovannib.crosta@unimib.it ; paolo.frattini@unimib.it

THE FIUMELATTE EVENT

On November 13, 2004, a rock fall of about 4000-5000 m^3 occurred at Fiumelatte (Varenna, Lombardy; Fig. 1). The rock fall detached from a rocky cliff as a single toppling block which exploded impacting on a rocky area just at it base. Then, a large number of blocks were thrown down-slope and spread for a 400-500 m distance along 40° slope reaching a urbanized area at the slope toe. The rock fall involved a 170 m long span of two transportation corridors (i.e. a railway track and a road of regional importance) and several houses and structures, claiming two casualties (Fig.1). Despite rock falls are common phenomena in this area and all along the of Como Lake, no rock falls of this size had been reported in historical times.

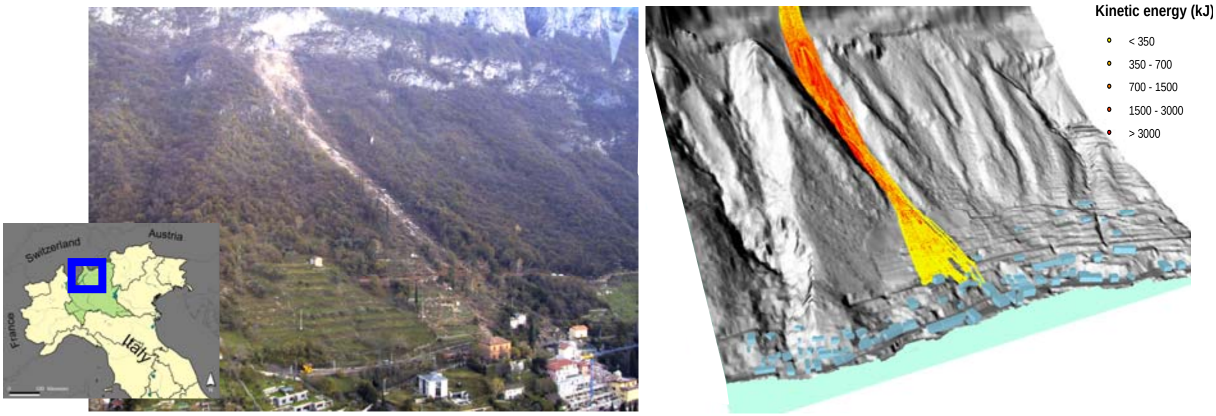


Fig. 1 Overview of the Fiumelatte area and event back analysis based on a LIDAR topography (cell size: 1 m)

MODELLING APPROACH

Rockfall modelling has been performed by the code HY-STONE [2, 4], able to take advantage of high-resolution 3D topography and to perform multi-scale stochastic modelling. The code incorporates kinematic and hybrid algorithms, modelling free fall, impact and rolling, with different damping relationships available to simulate energy loss. Topography is described by raster DEM, and all the relevant parameters are spatially distributed. The stochastic nature of rockfall processes is introduced as a function of the model resolution and by random sampling most parameters from different PDF. The model accounts for the interactions between blocks and countermeasures or structures by introducing their geometry and strength. Routines accounting for elasto-plastic deformation of soft ground, fragmentation and vegetation are also included. Rockfall modelling in the study area has been based on a LIDAR topography with 1m spatial resolution, and on detailed mapping of rockfall source areas, surface lithology and vegetation. Field surveys of single fallen blocks and damages to structures has also been accomplished.

First, a back analysis of the 2004 rockfall event has been performed (Fig. 1) in order to calibrate model parameters by event data (observed runout and damages). The calibrated model gives a satisfactory fitting of the actual location of the mapped blocks and of the transversal and longitudinal limits of the spreading zone (Fig. 1). Then, calibrated parameters have been used to model rockfalls hazard for the whole neighboring area.

SCENARIO-BASED RISK ASSESSMENT

Risk analysis has been performed by considering three scenarios, namely: a reference scenario without countermeasures (scenario 0) and two scenarios with the emergency-stage embankment (scenario 1), and the planned embankment (scenario 2), respectively.

For the reference (0) scenario, rockfall numerical modelling has been performed for the whole Fiumelatte village, and the related hazard has been computed by the RHV (Rockfall Hazard Vector) physically based methodology [2, 5] as a combination of reach probability, fly height and kinetic energy. The spatial distribution and energy of impacts on buildings and infrastructures were computed for each element at risk of a specifically collected database. The spatial distribution of impact energy (Fig. 2) was used to derive an expected damage degree according to simplified vulnerability functions. Finally, the expected cost of the scenario 0 has been evaluated, providing a quantitative estimation of risk. The analysis has been repeated for scenarios 1 and 2, using modified LIDAR topographies including embankments. This allowed to update hazard maps, impact analyses and cost estimates. Finally, we calculated the costs (evacuation, loss of property, deployment of defensive works) and benefits associated to each scenario, allowing for comparison and optimisation of the mitigation options.

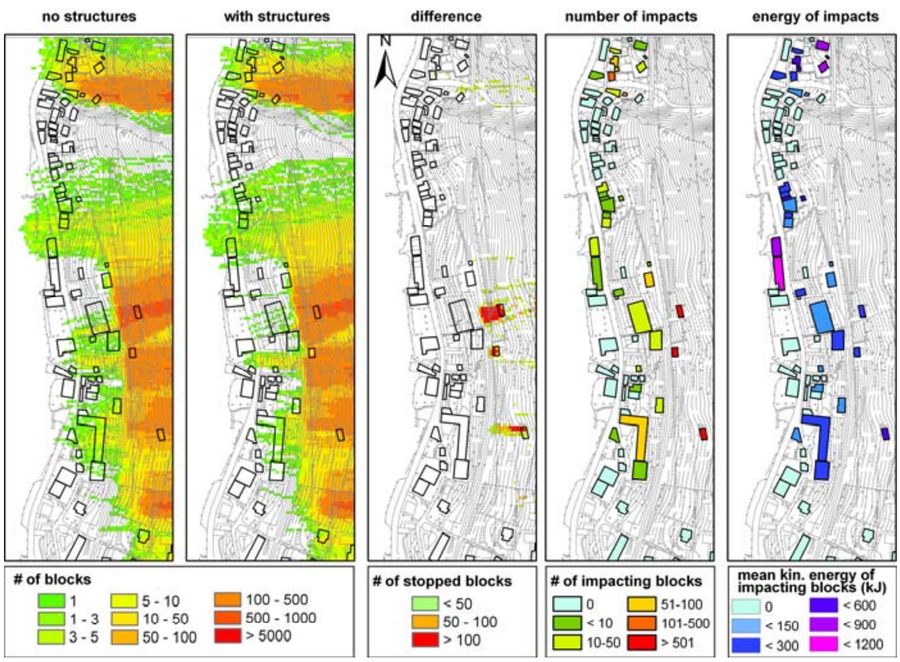


Fig. 2 Analysis of rockfall impacts on selected structures, including probability and kinetic energy (i.e. intensity). The analysis of observed vs. simulated impacts allowed defining simplified vulnerability functions.

CONCLUSIONS

This study shows the possibility to achieve quantitative evaluation of rockfall risk as an input for the design and cost/benefit analysis of different mitigation scenarios. A single modelling tool has been used to assess the physical components of risk, namely: hazard, probability and energy of impacts on structures. These are mandatory data for a complete risk assessment through the evaluation of the expected losses. For this case study, the approach allowed to assess both the technical efficiency (i.e. capability of intercepting incoming blocks) and the cost efficiency (i.e. cost/benefit) of different mitigation options.

REFERENCES

[1] AGLIARDI, F., G.B. CROSTA (2003) High resolution three-dimensional numerical modelling of rockfalls. International Journal of Rock Mechanics and Mining Sciences 40, 455-471.
[2] CROSTA G.B., F. AGLIARDI (2003) A new methodology for physically-based rockfall hazard assessment. Natural Hazards and Earth System Sciences 3, 407-422.
[3] CROSTA G.B., F. AGLIARDI, P. FRATTINI, S. IMPOSIMATO (2004) A three-dimensional hybrid numerical model for rockfall simulation. Geophysical Research Abstracts 6, 04502.
[4] JABOYEDOFF M., J.P. DUDT, V. LABIOUSE (2005) An attempt to refine rockfall hazard zoning based on the kinetic energy, frequency and fragmentation degree. Natural Hazards and Earth System Sciences 5, 621-632.
[5] FRATTINI P., G.B. CROSTA, A. CARRARA, F. AGLIARDI (2008) Assessment of rockfall susceptibility by integrating statistical and physically-based approaches. Geomorphology 94, 419-437.

INTERDISCIPLINARY APPROACH TO ROCKFALL FORECASTING

Mario Alba[1], Diego Arosio[2], Laura Longoni[1], Monica Papini[1], Marco Scaioni[1], Luigi Zanzi[2]

Our work addresses a new methodology for assessing and monitoring rockfalls through an integrated multi-disciplinary approach, taking advantage of innovative devices and sensors (terrestrial laser scanners, ground-based interferometric SAR systems, ground penetrating radars, seismic and acoustic sensor networks) in order to increase the achievable information. The research project promotes the integration of different techniques to facilitate a better analysis of complex processes as rockfalls. This means either to combine datasets collected with different sensors and to develop expert systems capable to merge diverse measurements as well as to support decision makers. Field activities are undergoing experimentation in prealpine and alpine test sites.

Keywords: Engineering Geology, Rockfall, Monitoring

INTRODUCTION

Among the many natural hazards in mountainous regions, *rockfalls* are frequently occurring events characterized by their suddenness and difficult of prediction. Bearing in mind the key role of structural protections in defending both infrastructure and human settlements from released rocks, the possibility of identifying areas featuring a high rockfall risk, and the monitoring of all critical situations are complementary important tasks in this research field.
In recent years, new instruments and techniques based on ground remote sensors have been employed for deformation/displacement monitoring (e.g. *Terrestrial Laser Scanning* - TLS, and *Ground-Based Interferometric Synthetic Aperture Radar* – GBinSAR), and their application to rock slopes assessment represents a current challenge for researchers. This is mainly due to the high accuracy that is required to detect small deformations or displacements affecting an instable rock slope (commonly a few mm), to the long distance from sensor to the monitored rock face, and to the harsh conditions that are typical of mountain areas.
On the other hand, modern *Ground Penetrating Radars* (GPR), in association with related data processing techniques, can be powerful tools to investigate rock mass condition, in order to map subsurface discontinuities where a rockfall could originate. Moreover, *distributed sensor networks* (microseismic or acoustic sensor networks employing, for instance, *Micro-Electro-Mechanical Systems* - MEMS) could allow to detect weak acoustic emissions or vibrations which may be earlier signals just before a rockfall is about to happen. As a matter of fact, fracturing processes start with the breaking of internal structures inside the rock mass, resulting in signals that can be recorded with suitable sensors..

INTEGRATED MONITORING AND ASSESSMENT OF ROCKFALLS

At the Politecnico di Milano, a multisciplinary research group supervised by geologists and including geophysical, geotechnical, micro-electronic, risk-management and geodetic

[1] Politecnico di Milano , D.I.I.A.R., via M. d'Oggiono 18/a, 23900 Lecco (Italy).
[2] Politecnico di Milano , D.I.S., p.za L. da Vinci 32, 20155 Milano (Italy).
e-mails: {mario.alba, laura.longoni, monica.papini, marco.scaioni, diego.arosio, luigi.zanzi}@polimi.it

engineers, started working on rockfall monitoring issues in 2005. A specific project (GPE – "*First Emergency Management*" - which is carried out in the frame of the university internal project *PROMETEO*, concerning public and civil protection – www.polimi.it/prometeo/) has been focused to assess and to improve the application of the above-mentioned techniques to rockfall forecasting. Moreover, the exploitation of integrated information obtained through the contemporary application of several investigation devices is believed to be the most efficient solution to this problem. This means, firstly, to achieve finer performances from each monitoring systems, hence requiring, in some case, the development of new data processing software, operational methodologies as well as hardware solutions. Secondly, the integration of different types of sensors has to be performed considering a twofold level:

1. combination of diverse datasets to improve their quality (i.e. accuracy, observation rate and spatial density, data reliability) and to achieve enhanced information; an example of this is represented by the contemporary use of TLS and GBinSAR, which would allow to exploit the higher point density of the former sensor and the better accuracy of the latter;
2. merging information derived from different assessment and monitoring systems, in order to improve decision-making processes. In this case, the use of a *Decision Support System* (DSS) to carry out this task in an almost automatic way is under development [1].

After the development of the investigation techniques, the effective application of the *Integrated Monitoring and Assessment of RockFall* (IMARF) approach to real unstable rock slopes requires the definition of *alarm thresholds*. Traditionally, thresholds are applied on the original observed readings on the basis of both empirical considerations and site history. IMARF approach makes use of a further threshold determined on the basis of the DSS analysis (the so called "*Observed Risk Degree*" - ORD, see Fig. 1). This solution has the advantage of integrating different classes of observations, exploiting possible correlations (and not only independent values), and keeping into account boundary conditions (e.g. meteorological data) as well. The alert activation strategy is depicted in Fig. 1. On the basis of both levels of information achievable by the monitoring systems (direct measurements and ORD), some *pre-alarm thresholds* are foreseen to activate "I level actions" accounting for:

- consolidation and protection works;
- intensification of the observation rate of monitoring systems and activation of further non-permanent monitoring systems;
- pre-activation of emergency plans for evacuation of the population and for alternative transport roads.

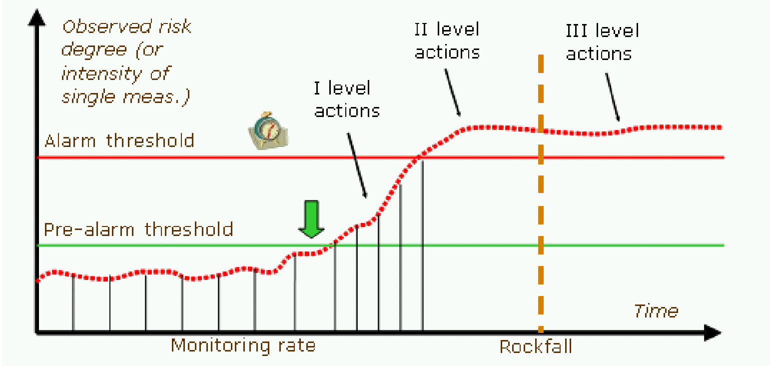


Fig. 1 Strategy for the definition of alarm thresholds and involved actions implemented in IMARF

Secondly, some *alarm threshold* are fixed when the risk of a rockfall would become high enough. In this case the following “II level actions” are engaged:

- people evacuation and interruption of transport ways;
- alert of emergency teams;
- withdrawal of monitoring system involving risk for operators and of expensive instruments.

The definition of both pre-alarm and alarm thresholds is a very complex and time-consuming process, which should be based on the availability of historical data concerning several rock slope morphologies and structures. Moreover, an intense on-the field testing is needed to provide further data.
Eventually, some “III level actions” are planned when the rockfall is partially happened. This are aimed to analyse the “*residual risk*” concerning the possibility that the process may still evolve in the next future. This aspect represents a specific feature of the GPE project, since the analysis of recent natural and anthropic hazards (e.g. terrorist attacks) has shown that the number of injured or dead people belonging to emergency teams is largely due to the underestimation of the residual risk. Obviously, this is still true in case of rockfall events.

ON-THE-FIELD TESTS

Currently two case studies have been selected in the Lecco mountain area (Lombardia Prealps), in the nearby of a county road which is affected by rockfalls in different areas. Here the application of TLS for periodical (monthly) deformation/displacement monitoring is on-going. The adopted strategy is based on the stationing of the instrument over a removable steel pillar, which allows to overcome errors due to georeferencing. Currently, displacements in the order of ±5 mm can be detected over rock slopes extending for some tens of square metres. Moreover, GPR acquisitions have been carried out in the same sites to investigate subsurface discontinuities [2]. Finally, microacoustic emission induced by soliciting cracks with an hydraulic jack where detected with different kinds of seismic sensors (piezoelectric as well as MEMS accelerometers and geophones) deployed according to diverse configurations with the aim of assessing propagation conditions and of comparing sensors’ performances.

CONCLUSIONS

The proposed IMARF approach to cope with rockfall forecasting and prevention is highly innovative and ambitious, because it is based on multisource observations and technologies which allow one to investigate the problem under diverse point of views. Work carried out so far concerned the definition of general strategies and focuses on the improvement of different tecniques in separate way. Further on-site experimentations are planned for the close future, where also sensor integration will be applied.

REFERENCES

[1] GRANDE, O., TRUCCO, P., LONGONI, L., PAPINI (2008). A Bayesian-based Decision Support Tool for assessing and managing rock fall disasters. Proc. of BEAR 2008, Kandalama (Sri Lanka), 10-15 Feb, pp. 566-575.

[2] SCAIONI, M., AROSIO, D., LONGONI, L., PAPINI, M., ZANZI, L. (2008). Integrated Monitoring and Assessment of Rockfall. In Proc. of BEAR 2008, Kandalama (Sri Lanka), 10-15 Feb, pp. 618-629.

HYBRID BARRIER SYSTEMS FOR ROCKFALL PROTECTION

Thomas C. Badger[1], John D. Duffy[2], Francesco Sassudelli[3], Peter C. Ingraham[4], Peter Perreault[5], Balasingam Muhunthan and Harish Radhakrishnan[6], Oreste S. Bursi and Marco Molinari[7], Eugenio Castelli[8]

This study examines hybrid barrier systems, incorporating field performance, limited rock-rolling tests, and finite-element modeling. A hybrid barrier is a passive rockfall protection system consisting of a flexible fabric suspended from a top horizontal cable raised off the ground by posts or by anchoring across a chute; it includes no internal, side or bottom anchoring of the fabric. They address rockfalls occurring both underneath the fabric and upslope of the installation, controlling their descent under the fabric and into a containment area at the base of the system. Systems installed in North America have proven to be durable and highly effective. Limited rock-rolling tests demonstrated these systems to be effective for moderate energies (< 500 kJ) requiring no maintenance during repeated impacts with less than 0.5 m height loss. Finite-element modeling examined deformation and loads for a variety of system and slope configurations exposed to a range of impact energies. Modeling results showed comparable deformation behavior and performance results as the rock-rolling tests. These systems protect more slope area with less coverage than would be required with a full drapery, and they capture higher energies with less robust fence infrastructure. These combined benefits typically result in highly cost-effective and low-maintenance systems.

Keywords: hybrid barrier, drapery, rockfall protection, numerical modeling

INTRODUCTION

A hybrid barrier refers to a passive rockfall protection system consisting of a flexible, woven-wire or -cable fabric suspended from a horizontal top support cable that is raised off the ground by posts or by anchoring across a chute (Fig. 1A). No internal (patterned) or side and bottom anchoring of the fabric is generally included, allowing for controlled deformation of the fabric and providing either full containment or attenuation of the rockfall trajectory at the

[1] Washington State Department of Transportation, P.O. Box 47365, Olympia, Washington 98504-7365, +01-360-709-5461, badgert@wsdot.wa.gov
[2] California Department of Transportation, 50 Higuera Street, San Luis Obispo, California 93401, +01-805-549-3663, John_D_Duffy@dot.ca.gov
[3] Igor-Paramassi, Via A. Volta, 38015 Lavis, Italy, +39 0461 245098, fancesco.sassudelli@igorparamassi.it
[4] Golder Associates Inc., 540 N. Commercial Street, Suite 103, Manchester, New Hampshire 03101, +01-603-668-0880, pingraham@golder.com
[5] Mountain Management Inc., Financial Plaza Building, 1135 Terminal Way, Suite 106 Reno, Nevada, 89502-2145, +01-866-466-7223, Peter@MountainManagement.biz
[6] Department of Civil and Environmental Engineering, Washington State University, Pullman, Washington 99164-2910, +01-509-335-3921, muhuntha@wsu.edu
[7] Department of Mechanical and Structural Engineering, University of Trento, 38100 Mesiano, Trento, Italy, +39 0461 882503, oreste.bursi@ing.unitn.it
[8] Department of Civil and Environmental Engineering, University of Trieste, Italy +39 040 558 7157, ecastelli@units.it

base of the installation. Hybrid barriers address rockfall source areas both underneath and upslope of the installation and control the rock's descent under the mesh, combining the performance of standard unsecured draperies [1] and flexible rockfall fences. Consequently, they protect more slope area with less coverage than would be required with a full drapery, and they capture higher energies with less robust fence infrastructure. These combined benefits often result in highly cost-effective and low maintenance systems. To date, these systems have been designed empirically in the absence of quantitative design data, full-scale field testing, or documented performance. This study examines hybrid barriers, incorporating existing field performance, limited rock rolling tests, and finite-element modeling.

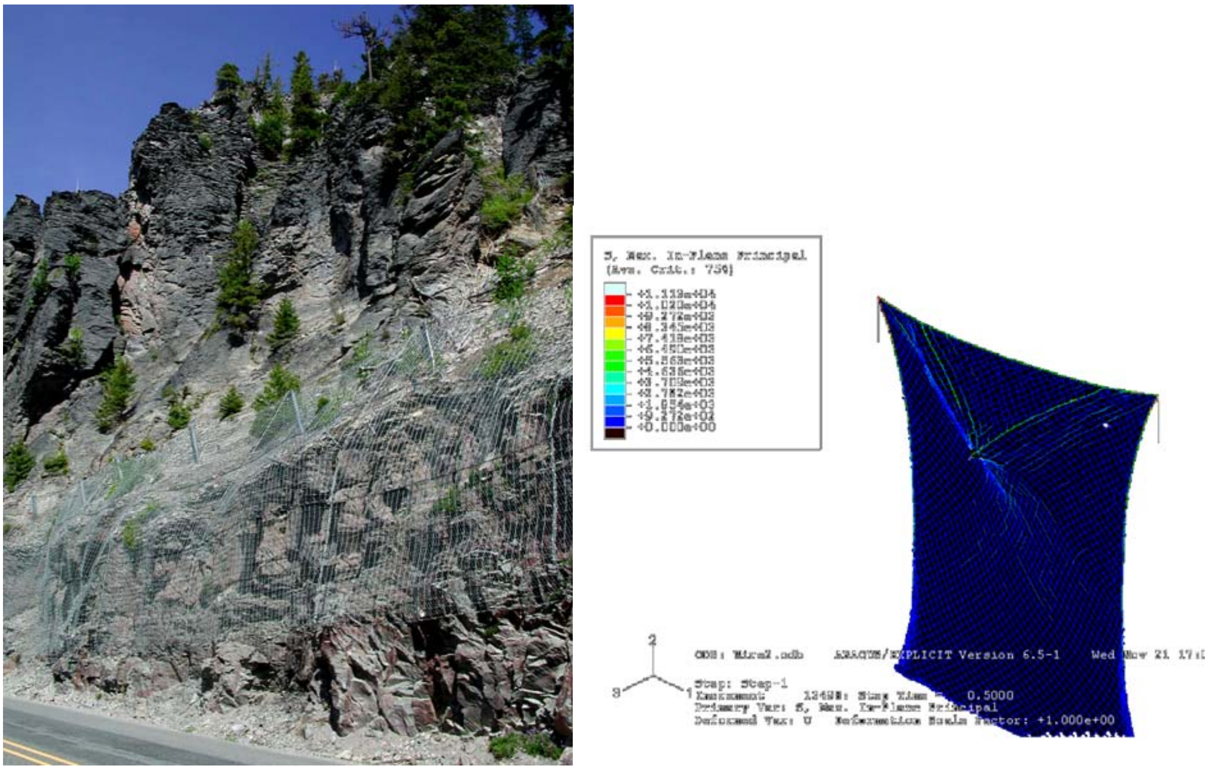


Figure 1. (A) Hybrid barrier controls rockfall initiating both upslope and underneath of the installation, containing the debris beneath the fabric or discharging it into a ditch at the base of the slope. (B) Simulation of 12 m wide by 15 m high cable net system impacted by 15 kJ rockfall; simulations up to 200 kJ were made.

PERFORMANCE OF EXISTING INSTALLATIONS

Numerous hybrid barrier systems have been constructed in western North America to control rockfalls along highways using both lightweight and high-tensile-steel chainlink fabrics, double-twisted hexagonal mesh, and more robust cable nets. Badger [2] reported systems in Washington (USA) to be durable and highly effective for containing rockfalls generated both beneath and upslope of the installation. Observed problems included minor puncture failures near the top horizontal support rope, damage to post supports, debris accumulation above abrupt slope convexities and horizontal support ropes, and maintaining slope coverage with long narrow drapes. Based on these observations, design efforts have sought to avoid restraining the mesh, raise systems to reduce perimeter impacts, minimize vulnerable post supports, diminish effects of slope convexities, reinforce the impact area, and cautiously secure narrow systems. Similar favorable results were reported by Duffy [3] for numerous systems installed in California (USA) using different fabrics and infrastructure, and under varied site and loading conditions.

FIELD TESTING

Recent full-scale field tests were performed on a 45º slope with rolled rocks [4]. The barrier used three 6 m-wide by 4 m-high panels supported on four 4 m-high posts suspending an approximately 15 m-long drapery of ring nets. The nets were secured to the top horizontal rope and at the upper third points along the sides. Five tests were performed with concrete "rocks" ranging from 1 to 5 tonnes with estimated impact velocities up to 14.3 m/s, resulting in impact energies up to about 500 kJ. All of the rocks were successfully stopped in a flat containment area within three meters of the base of the slope. Load cells on the top horizontal rope and end tie-back anchor were measured for several impacts with rocks up to 2.6 tonnes, yielding a maximum impact load measured in the top horizontal rope of 83kN. Despite several impacts to support ropes and a post, no system maintenance was required, remaining wholly functional during the entire succession of tests.

NUMERICAL MODELING

Finite-element analyses (ABAQUS) evaluated loads on the support posts and fabric deformation for a range of impact energies (≤ 200 kJ), slope orientations (70º to 90º), slope-fabric interface friction values (0, 1 and no slip), and system configurations (6 to 15 m in width and 12 to 30 m in length) [5]. Cable nets, consisting of a 250mm diagonal grid of 8mm diameter wire ropes, were selected for the analysis. Static benchmark modeling calibrated the model with puncture tests performed on 3 m-square panels at the University of Trento, Italy. Wire ropes were modeled using 2-dimensional truss elements, and static runs were solved using the Newton-Raphson technique with slight viscous damping included. Modeling results showed comparable enveloping of the rock by the fabric and dramatic attenuation of the horizontal trajectories as was observed during the field testing.

CONCLUSIONS

Observed field performance of existing systems along with limited rock rolling tests and numerical modeling results demonstrate the effectiveness of these systems to control repeated rockfalls of low to moderate energies (<500 kJ) with little to no damage or requisite maintenance. The system functions by attenuating energies through extensive flexibility and minimal restraint of the fabric, dampening the trajectory, and containing the rock either beneath the mesh or in a catchment area at the base of the system. Further full-scale field testing and numerical modeling are needed to refine the design of these systems to a broader range of field conditions, and to more fully evaluate the required containment area.

REFERENCES

[1] MUHUTHAN, B., S. SHU, N. SASIHARAN, O.A. HATTAMLEH, T.C. BADGER, S.L. LOWELL, J.D. DUFFY (2005) Analysis and Design or Wire Mesh/Cable Net Slope Protection, Washington State Transportation Center, Seattle, Washington, WA-RD 612.1, 267p.
[2] BADGER, T.C. (2007) Suspended draped rockfall protection: an overview of system performance in Washington, Assoc. of Environ. & Engr. Geologists Program with Abstracts, pp. 68.
[3] DUFFY, J.D. (2007) Hybrid rockfall systems, Assoc. of Environ. & Engr. Geologists Program with Abstracts, pp. 80.
[4] SASSUDELLI, F., P. PERREAULT, P.C. INGRAHAM (2007) Recent rockfall barrier testing in Italy: structural component tests and dynamic testing of barrier kits and hybrid barrier/drape systems in the framework of European technical guidelines, Assoc. of Environ. & Engr. Geologists Program with Abstracts, pp. 122.
[5] MUHUNTHAN, B., H. RADHAKRISHNAN (2007) Finite Element Analysis of Hybrid Barrier for Rock Fall Slope Protection, Final Report, Washington State University Dept. of Civil & Environ. Engineering, 13 p.

AN APPROACH TO THE DESIGN OF NETS AND NAILS FOR SURFICIAL ROCK SLOPE REVETMENT

P. Bertolo[1], G. Giacchetti[2]

In Europe, both for historical and technical reasons, superficial revetments are usually installed using nets together with the systematic nailing of the rock slope surface.
At first, the paper points out why superficial revetments have to be classified as passive systems. After that the net design method is examined: the correct design procedure must take into account not only of the verification of failure loads of the system components (ultimate limit state), but also of the maximum deformation of the facing (serviceability limit state condition). Due to the complexity of the problem, the large number of parameters involved, and their interaction, which at present is not well known, it is necessary to use full scale and *in-situ* test results to better understand and describe the system behaviour.
The use of experimental load-displacement curves of the facing is therefore necessary for the correct design of the structure.

Keywords: rockfall, nets, design, limit state, physical models.

INTRODUCTION

The installation of surficial systems is one of the most common ways of protecting roads and infrastructures against the detachment of small rock elements in areas prone to rockfall. Despite their frequent and worldwide application, there are no guidelines or technical standards to help engineers for the correct design and for an effective choice of the best systems and products to avoid this type of instability.
In order to pursue a suitable design of these protection works, a project approach based on the real behaviour of a specific surficial system is needed, but there is still lack of experimental data and comparative tests on different facing products.
At present, no field experiences are available where the whole “system”, which is composed of several structural components that interact, is tested. Test field need to be set up to evaluate the full-scale behaviour, taking into account the site boundary conditions, of a cortical system retaining a falling rock block.
Furthermore a correct geotechnical approach has to take into account of the passive behaviour of this kind of protections and the design method has to follow the guidance given by the Eurocode 7, that means not only check for the ultimate resistance of the components and of the whole system, but also check the serviceability limit state conditions.

ACTIVE AND PASSIVE SYSTEMS

Usually, interventions against rock falls are classified as either “active” or “passive” protection systems, without a clear-cut between these two classes. The aim of an active protection system is to prevent the instability from occurring, while a passive system is designed to mitigate the effects of a previous movement by containing, intercepting and stopping falling rock blocks.
Only pre-stressed wire anchors, can be classified as active elements, since they start to apply a force before the movement of blocks. On the other hand, embankments, ditches, net fences

[1] OFFICINE MACCAFERRI S.p.A., Via Agresti, 6, 40123, Bologna, ITALY, +39 0516436000, paola.bertolo@maccaferri.com

[2] Consultant, Giorgio.giacchetti@maccaferri.com

and rock sheds are passive systems, since they do not directly interfere in the process of rock detachment from the slope, but control the dynamic effects of blocks that have already been detached while they are moving downhill.
Surficial revetments can be considered as something in-between the two above mentioned classes, since they mainly act by controlling the movement of rock fragments thus preventing them from freely falling onto infrastructures, but from the geotechnical point of view they are absolutely "passive", because they start to retain the material only after the first movement has occurred. Nevertheless, if the components of the surficial system are sufficiently stiff, the intervention can prevent the detachment and the free fall of the unstable elements from shallow layers of a rock slope [1].

FULL SCALE TESTS

In order to obtain a correct design procedure, it is necessary to combine both a geological and geomechanical characterization of a site to determine the load conditions, but also the surficial system mechanical behaviour is necessary for a design engineer to know how a specific system reacts when loaded.
Only full-scale tests on large elements can be a representative and valuable tool for both the industry and designers. A specifically designed full size test device (Fig. 1) for the characterization of superficial reinforcements was built [2] and used for different tests. This field test has proved to be able to provide effective data and to allow the comparison of different types of products. From the performed tests the load-displacement curve for different surficial reinforcing systems (mesh or cable panel facing combined with nails) were obtained (Fig. 2).


Fig. 1 - On site tests

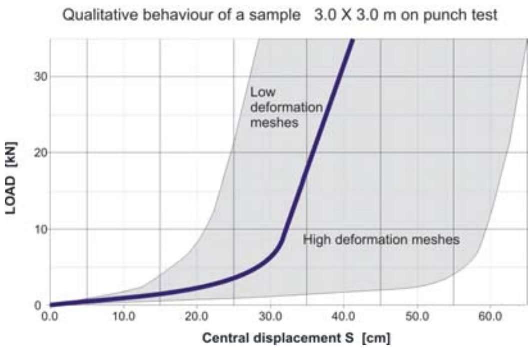


Fig. 2 - Real behaviour of surficial revetments

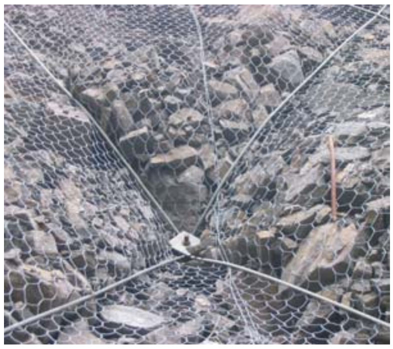


Fig. 3 - "Quilt-like" behaviour of a net

DESIGN METHOD

From the load-displacement curve of punching test on 3.00 m x 3.00 m net samples (Fig. 2), it can be clearly seen that the facing of the surficial revetment has a non-linear behaviour: after a first stage in which the facing has an important deformation under low loads there is a sudden increase in the system stiffness. In this paper the nail design is not analysed, because it can be done following the usual rock mechanics methods.
For loads less than 10kN deformations of the system are in the order of 200 to 600 mm, depending on the net type. This behaviour allows for two different serviceability limit states to be reached:

- the over-deformation of the facing system
- the gradual but continuous detachment of blocks from rock mass, which can cause a material loss around the nail and the consequent failure of the nail itself.

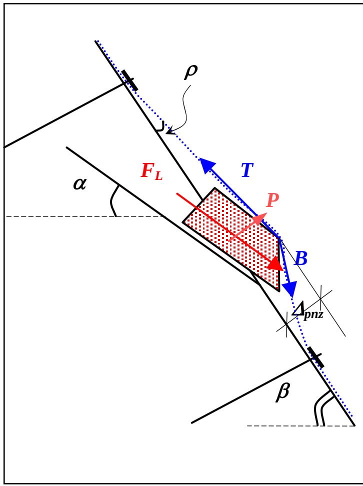


Fig. 4 - 2D scheme of the force vectors on the net between the anchors

The net covers the exposed face of the reinforced rock mass and has to provide a stabilizing function to retain the debris material and the small unstable blocks between anchors.
β is the rock mass inclination and α is the inclination of the most dangerous joint, along which a block can slip between the anchor pattern. The reaction Nr ($\vec{Nr} = \vec{B} + \vec{T}$) opposed by the net to the movement of the unstable mass (which loads the net with F_L) can be evaluated starting from the stability condition of the rock mass, taking into account of the passive behaviour of the surficial revetment system (facing cover and nails):

$$((Wi \cdot sin\,\alpha \cdot (1 - c)/\gamma_{MWi} + Nr) \geq (Wi \cdot \gamma_{DWi} \cdot (sin\,\alpha + c \cdot cos\,\alpha))$$

where Wi is the weight of the unstable block (or blocks), c is seismic coefficient, γ_{MWi} is a reduction coefficient to take into account for the continuous rock weathering (e.g. γ_{MWi} = 1.15), γ_{DWi} is a partial increasing coefficient to take into account of the model uncertainty (e.g. must be γ_{DWi} >1.3). $P=F_L \cdot \sin(\beta-\alpha)$, is the component of the load of the rock normal to the ideal rock mass inclination. The net deformation arises from load-displacement curves from full-scale tests; $\rho=\arctan(\Delta_{pnz}/1.5)$, where $\Delta_{pnz} = f(P)$ from punching load-displacement curve (Fig. 2: X-axis). The net is verified if $T_{net}/\gamma_{dnet} \geq T$ (ultimate limit state check) and if $\Delta_{pnz} \cdot \gamma_{d\Delta} \leq \Delta_o$ (serviceability limit state check), where T_{net} is the tensile strength of the net,γ_{dnet} is a safety coefficient of the tensile strength of the net (suggested $\gamma_{dnet} \geq 3$), $\gamma_{d\Delta}$ is a partial coefficient on the net bulging, to take into account of unexpected disadvantageous conditions and Δ_o is the design admitted bulging. This value can vary depending on the facing position (e.g. along a road) and working conditions.
The presented model is very simple, because doesn't take into account of the 3-D of the problem, which heavily influences the load distribution and the vector composition ($\Rightarrow$ the stresses on the net): this will be future development of the model.

CONCLUSIONS

To evaluate the real behaviour of a netting system it is necessary to develop full-scale tests, since the boundary conditions usually adopted in laboratory tests (load, boundary conditions, constraints, size) influence the results to a great extent. The knowledge of how much a mesh deforms under a defined load is a key value for the evaluation of the behaviour of a system, and guarantees that the designed protection system will not collapse (ultimate limit state) or deform too much (serviceability limit state) when it contains the design block.
In addition in situ tests are also representative of the punching condition of the system at the anchors, when the system is loaded by debris material and the anchoring (or the eyebolt with the ropes) plate maintains the net or the panel in contact with the slope, producing a "quilt-like" aspect (Fig. 3).

REFERENCES

[1] SMITH, D.D., J.D. DUFFY (1990) Field Tests and Evaluation of Rockfall Restraining Nets, Report CA/TL - 90/05, Office of Transportation Materials and Research, California Department of Transportation, Sacramento, California, U.S.A.
[2] BERTOLO, P., F. FERRAIOLO, G. GIACCHETTI, C. OGGERI, D. PEILA, B. ROSSI (2007) *Metodologia per prove in vera grandezza su sistemi di protezione corticale dei versanti – GEAM Geoingegneria Ambientale e mineraria, N. 2, 2007* (*in italian*).

A STOCHASTIC FRAMEWORK FOR THE MODELLING OF THE IMPACT OF A BOULDER ON A COARSE SOIL

Franck Bourrier, François Nicot[1], Félix Darve[2]

Rockfall hazard assessments are increasingly done using trajectory analysis. A key parameter in such approaches is the modelling of the interaction between the falling rock and the soil. The presented study focuses on this phase in the case of a boulder impacting a scree slope. In the context of trajectory analysis, the available information on the local configuration of soil particles near the impact point and on the incident kinematic parameters of the boulder are not sufficient to perform a deterministic prediction of boulder bouncing. Stochastic approaches are then necessary to properly model the impact phase.
Numerical investigations using a Discrete Element Method are performed in order to investigate the impact process. The statistical treatment of the results leads to the definition of a stochastic impact model linking the block kinematics parameters before and after its interaction with the granular soil. The comparison between the predictions of the model and the results from half-scale experiments emphasizes the relevance of a predictive use of the stochastic impact model and the interest of using stochastic approaches in the field of trajectory analysis.

Keywords: trajectory analysis, impact, stochastic, half-scale experiments

INTRODUCTION

The prediction of the trajectories of the potentially falling boulders is one of the key points in the field of trajectory analysis. The large variability of slope properties and rock removal conditions pleads for a stochastic modelling of potential trajectories of falling rocks. In particular, the bounce of falling rocks has to be modelled as a random process ([1], [2]).
Numerical approaches are used in order to investigate the impact of a block on a coarse granular soil and a numerical simulation campaign is held. The analysis of numerical results is carried out using advanced statistical methods leading to the definition of a stochastic impact model. The validity of the stochastic impact law is evaluated by comparing predictions of the model to results from half-scale experiments.

STOCHASTIC MODELLING OF THE IMPACT

Numerical simulations are performed in order to provide a large dataset usable for the statistical definition of an impact model. Assuming that rocks composing the talus slope are rigid locally deformable two-dimensional bodies, the software Particle Flow Code 2D [3] based on a Discrete Element Method is used. The boulder is assumed to be spherical and the boulder diameter R_b varies from 0.65 m to 3.25 m. The mean diameter of soil particles is 0.65 m. The properties of the sample and the numerical simulation of an impact event are defined following the approach used in [4].

[1] Cemagref, UR ETNA, 2 rue de la Papeterie, BP 76, 38 402 Saint Martin d'Hères Cedex, France, franck.bourrier@cemagref.fr, francois.nicot@cemagref.fr
[2] L3S-R, INPG - UJF - CNRS, Domaine Universitaire, BP 53, 38 041 Grenoble Cedex 9, felix.darve@inpg.fr

In a two-dimensional frame, kinematic parameters of the boulder are properly described by a generalized velocity vector $\mathbf{V}$ that is composed of a velocity component v_y along the normal direction to soil surface, a velocity component v_x along the tangent direction to soil surface and a rotational velocity ω such as: $\mathbf{V} = \begin{pmatrix} v_x & v_y & R_b\omega \end{pmatrix}^t$. The incident $\mathbf{V}^{\mathbf{in}}$ and reflected $\mathbf{V}^{\mathbf{re}}$ velocity vectors of the block are related by the operator $\tilde{f}$: $\mathbf{V}^{\mathbf{re}} = \tilde{f}(\mathbf{V}^{\mathbf{in}})$. The first order Taylor series expansion of $\tilde{f}$ leads to the definition of the operator $\mathbf{A}$ composed of the coefficients a_i of the Taylor series expansion of $\tilde{f}$:

$$\mathbf{V}^{\mathbf{re}} = \mathbf{A}\mathbf{V}^{\mathbf{in}} \text{ with } \mathbf{A} = \begin{bmatrix} a_1 & a_2 & a_3 \\ a_4 & a_5 & a_6 \\ a_7 & a_8 & a_9 \end{bmatrix}$$

The high variability of the local configurations of the soil and of the incident kinematic conditions induces that the operator $\mathbf{A}$ cannot be considered as deterministic variables. A stochastic approach using Bayesian inference is therefore adopted in order to define a unique impact model that can capture the variability associated with the random variables $\mathbf{A}$. The operator $\mathbf{A}$ is assumed to take a constant value for a given impact point. The variability of a_i parameters from one impact point to another is described by a normal law.

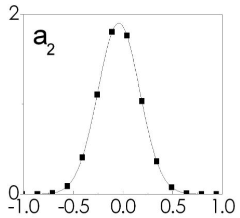


Fig. 1 Probability distribution function of the parameter a_2

The results obtained using the free software Winbugs® [5] highlight that the random variable $\mathbf{A}\mathbf{V}^{\mathbf{in}}$ models more than 75% of the variability of the results. Moreover, the marginal standard deviations of the coefficients a_i are small with respect to the validity range of the parameters (see Fig. 1) which confirms that physical processes are very similar from one impact point to the other. An efficient predictive use of the impact model is therefore envisigible.

EXPERIMENTAL VALIDATION OF THE STOCHASTIC IMPACT MODEL

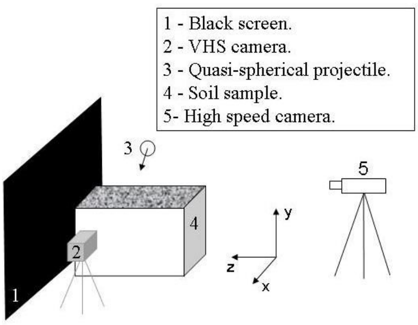


Fig. 2 Experimental set-up

The predictions of the model are compared to results from half-scale experiments consisting in the impact of a 10 cm diameter boulder on a coarse soil. More than 100 impact are performed for varying incidence angles and incident velocity magnitude V_i. A specific projectile dropping device has been designed in order to control the incident velocity of the projectile. The component of the incident velocity along the z axis and the incident rotational velocity are set to 0. Impact experiments are both filmed using a high speed camera and a VHS camera (see Fig. 2). Image processing of films allows the calculation of reflected velocities.

Experimental results are compared to the components of the reflected velocity predicted using the stochastic impact model under the same incident conditions (see Fig. 3). The results highlight a satisfying accordance between the reflected velocities predicted and the experimental results. The tangential velocity is particularly well predicted using the stochastic impact model. The predictions for the normal reflected velocity are slightly larger than the experimental results. This slight difference can be related to smaller energy dissipations inside the numerical sample either due to differences between the porosity of the experimental soil and of the numerical sample or to the still unsatisfying numerical modelling of energy dissipation associated with contact attrition or particle breakage.

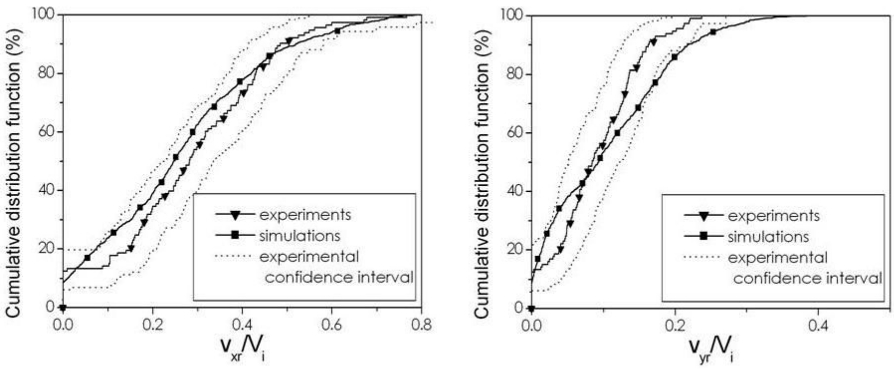


Fig. 3 Comparison between the experimental results and the predictions of the impact model

CONCLUSION-DISCUSSION

Based on a DEM numerical model of the impact of a boulder on a coarse granular soil, an extensive simulation campaign has been performed providing a large set of results for impacts associated with different soil particles spatial configuration and boulder incident parameters. Assuming that a relation exists between the incident and reflected velocities of the falling rock, the statistical analysis of the numerical results leads to the definition of a first order stochastic impact model that allows quantifying most of the variability of the reflected velocity vector. The comparison between the predictions of the model and the experimental results emphasizes that the first order impact model provides reasonable predictions.

The stochastic impact model proposed constitutes an extension to classical laws which allows modelling the coupling between the incident kinematic parameters. Moreover, contrary to classical approaches, the model proposed is directly developed in a global stochastic framework. Contrary to standard models, the mean restitution coefficients predicted by the model are not constant; they depend heavily on the incident velocity vector of the boulder.

In order to evaluate the applicability of the model for trajectory analysis, it has been implemented inside the trajectory analysis software Rockyfor [6]. The predictions of the model have been compared to real-scale experimental results [7]. The results of the comparison between the predictions and the experimental measurements are very satisfying. In addition, the use of the proposed model shows the advantages of the procedure developed. As the values of the model parameters are previously defined using numerical simulations, field data collection is reduced to the determination of the mean size of rocks for each point of the site.

REFERENCES

[1] AGLIARDI, F.; CROSTA, G.B. (2003) High resolution three-dimensional numerical modelling of rockfalls. Rock *Mechanics and Mining Sciences 40*, pp 455-471.

[2] JABOYEDOFF, M.; DUDT, J.P.; LABIOUSE, V. (2005) An attempt to refine rockfall zoning based on kinetic energy, frequency and fragmentation degree. *Natural Hazards and Earth System Sciences 5, pp* 621-632.

[3] Itasca Consulting Group. (1999) PFC2D User’s manual. Minneapolis: Itasca.

[4] BOURRIER, F.; NICOT, F.; DARVE, F. (2007) Rockfall modelling: Numerical simulation of the impact of a particle on a coarse granular medium. *Proc. 10th Int. Congr. on NUmerical MOdel in Geomechanics Rhodes (Greece)*, pp 699-705.

[5] SPIEGELHALTER, D.J.; THOMAS, A.; BEST, N. (2000) WinBUGS Version 1.3 User Manual, MRC Biostatistics Unit.

[6] DORREN, L.K.A.; BOURRIER, F.; NICOT, F.; BERGER, F.; DARVE, F. (2008) Towards objective rockfall trajectory modelling using a stochastic rebound algorithm. *Workshop on Rockfall Protection Morschach (Switzerland).*

[7] DORREN, L.K.A.; BERGER, F.; Putters, U.S. (2006) Real-size experiments and 3-D simulation of rockfall on forested and non-forested slopes. *Nat. Hazards. and Earth Syst. Sc.* 6, pp. 145-153.

ROCKFALL PROTECTION BARRIER: A COMPARISON BETWEEN FULL SCALE TESTS AND NUMERICAL MODELING

Benoit Boutillier[1], Ludovic Idoux[2]

For many years, lots of companies have been testing their rockfall protection barriers using full scale tests. Due to the expense of such tests, numerical modeling could provide a useful alternative. With different simulations, we were able to study the effects of several parameters: distance between posts, net height, applied forces on anchors, etc. to reduce the cost.
A numerical model of a rockfall dynamic barrier was developed by Nicot [1]. For this modeling, the study was based only on static and dynamic tests on ring net set and breaking systems. An elasto-plastic model with hardening was obtained to simulate the mechanical reaction of the ring net. A threshold function was used for dissipating devices.
To validate this model and obtain a first adjustment, we compared the full scale test of the 250kJ barrier with the simulations (Idoux [2]). We obtained a good match between the experimental tests and the numerical simulations, which enabled us to improve the numerical model and carry on full scale tests for other comparisons.

Keywords: rockfall barrier, dynamic, numerical modeling, full scale test

INTRODUCTION

The present paper deals with experimental full scale tests and the numerical simulations carried out on rockfall dynamic barriers. For several years, all producers have been improving their rockfall barriers with expensive full scale tests. The numerical model could greatly assist companies to study the effects of different parameters: the distance between posts, slope angle, maximum displacement of net set, applied forces on anchors, etc., at a reduced price.
A numerical model was developed by Nicot [1]. To develop this model, Nicot [1] studied the mechanical behavior of a ring net set and breaking systems with separate tests.
To adjust this model, experimental (full scale tests) and numerical data were compared. We obtained a good fit which allowed us to continue our study.

ROCKFALL DYNAMIC BARRIER

The studied barrier (see Fig. 1) is composed of ring nets, posts, cables and specific breaking systems. Ring nets are downhill posts, and breaking systems are installed to each uphill and lateral anchor (between the anchor and the cable). A Ring net set is composed of assembled rings with a different quantity of adjacent rings. In our study, we considered a standard ring net which corresponds to 1+6 type, i.e. one ring directly linked to six other rings.
Breaking system is shown on figure 2. It is constituted of two cables which glide between two specific steel plates.

[1] HC Systec, 19, avenue Victor Hugo, F-38 800 Le Pont de Claix, +33 4 76 33 61 10 (fax: +33 4 76 92 37 75), benoit.boutillier@groupehc.com
[2] HC Systec, 19, avenue Victor Hugo, F-38 800 Le Pont de Claix, +33 4 76 33 61 10 (fax: +33 4 76 92 37 75), hc.systec@groupehc.com

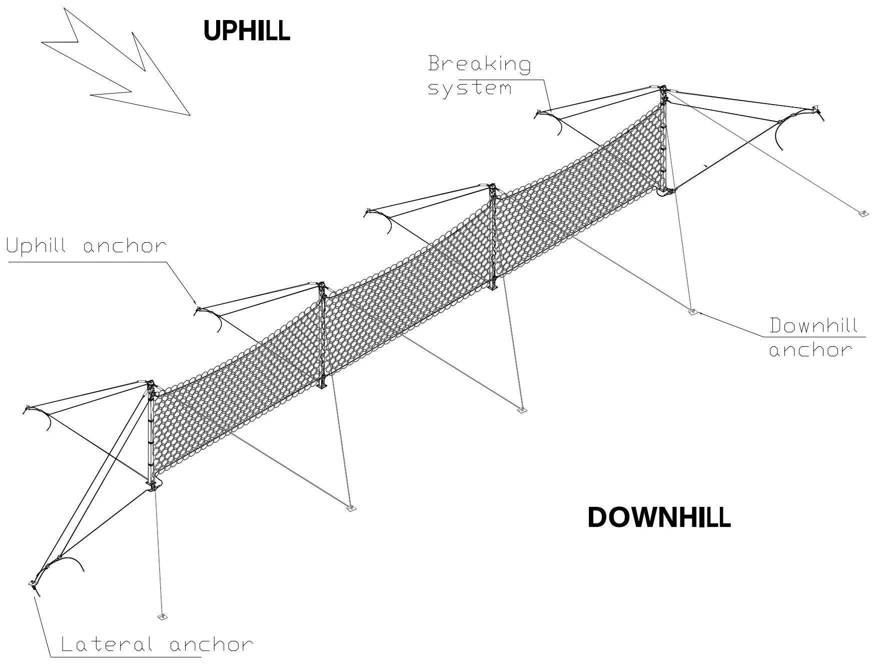


Fig. 1 Scheme of studied rockfall barrier

Fig. 2 Studied breaking system

NUMERICAL SIMULATIONS

For modeling, some simplifications had to be chosen.
For breaking system, a threshold function was chosen. This means energy dissipation occurs when a level of applied force is reached and up until this continues to be applied.
An elasto-plastic model with hardening was developed to represent the mechanical behavior of a ring net (Nicot [1], Nicot et al. [2]). Static tests were done on ring net set to improve this model (Nicot [1]). The discrete element method was used to represent all elements of the barrier, then by fixed or mobile nodes. For example, anchors are modeled by fixed nodes and ring net is composed of mobile nodes, which correspond to the ring centers.
Numerical simulations allow us to determine the main parameters: applied forces on anchors, running lengths of dissipating devices, maximum displacement of ring net, schemes of different steps of simulated impact (see Fig. 3), etc.

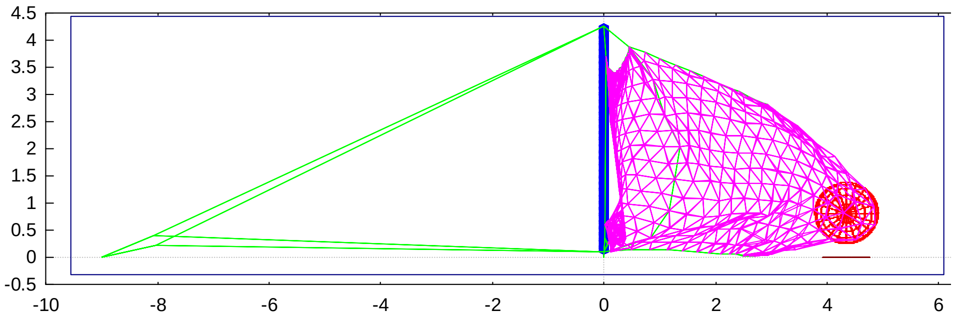


Fig. 3 Lateral view of numerical simulation – View of maximum displacement

EXPERIMENTAL DATA

Experimental data was conducted with full scale test on structure of 250kJ (vertical test – see Fig. 4) of 3 identical ring sets of 2.4m high. The released rock weighed 1 170kg and fell from a height of 22m above the structure, and then reached it at an approximate speed of 20.8m/s. During this test, we measured the applied forces on anchors at a frequency of 2 kHz. Before and after impact, we also measured global geometry of the structure. The maximum applied force on the anchors was 75kN (see Tab. 1) and the total running length of breaks was 7.89m.

Fig. 4 Experimental full scale test on barrier of 250kJ

Tab. 1 Main measurement during experimental test VS Simulations (S.R.S.: Static release strength of breaks)

	TESTS	**SIMULATIONS**		
Type of measurement	**Measure**	**S. R. S. 75kN**	**S. R. S. 21kN**	**S. R. S. 17kN**
Global running length (m)	7.89	3.60m / -54.4%	7.45m / -5.58%	7.87m / -0.25%
Maximum force (kN)	75	75m / 0%	75kN / 0%	75kN / 0%
Maximum displacement of impacted net set (m)	5.90	4.5m / -23.7%	5.5m / -6.8%	6.3m / +6.2%

COMPARISON

As the numerical model had not yet been correctly adjusted, we performed several simulations with different values of running lengths of dissipating devices and their static release strength to obtain a good match between the model and the full scale test. Finally, the right agreement was obtained when we only changed the value of the static release strength of the breaking systems (see Table 1 and Idoux [3]). For breaking system with 21kN of static release strength, the best agreement is obtained with a numerical value of 17kN.

CONCLUSION

As full scale tests are expensive, numerical tools could be very interesting to study the effects of several parameters. Even if dynamic impact induces many difficulties, a numerical model was developed by Nicot [1]. This model was developed with independent mechanical tests.
To adjust the model, a full scale test is obviously necessary. Several measurements were performed during 250kJ full scale test. Latest data was compared with the numerical ones. A good fit between these data was obtained with the break static release strength of 17kN. The model is now available for structures using these breaking systems. New experimental tests will be done in the future to improve the numerical model for other types of structures.

REFERENCES

[1] NICOT, F. (1999) Study on mechanical behavior of flexible structures of protection against falling rocks (in French). Ph D thesis, Ecole Centrale de Lyon.
[2] NICOT, F., B. CAMBOU, G. MAZZOLENI (2001) From a Constitutive Modelling of Metallic Rings to the Design of Rockfall Restraining Nest. International Journal for Numerical and Analytical Methods in Geomechanics, n° 25, pp. 49-70.
[3] IDOUX, L., (2004) Help to design rockfall dynamic barrier (in French). Internship thesis, Ecole centrale de Nantes.

IMPACT BEHAVIOUR OF MATERIALS: A TOOL FOR THE DESIGN OF ROCKFALL PROTECTIVE SHELTERS

Ezio Cadoni[1], Carlo Albertini, D. Forni

The optimum design of rockfall protective shelters resistant to quasi-static, strong earthquake and impact loading, requires the use of modern computer codes implemented with the constitutive equations developed and calibrated on the mechanical properties of the materials; they should be measured with precision tests in a large range of strain rate in the most common deformation modes like tension, compression and shear and under combined loading.
It is not obvious to underline the requirement of measuring the material properties with precision experiments because, especially under impact loading, in case of not well designed testing experiment the records of material response are strongly affected by the stress wave interactions and reflections inside the testing equipment which render the record analysis very difficult and of low accuracy; therefore in this paper special attention is given to the definition of a correct impact testing rig and the relative results obtained on plain and fibre reinforced concrete.

Keywords: impact, concrete, FRC, experimental technique, high strain-rate

INTRODUCTION

High loading rates, caused by the impact of flying objects, rockfall, vehicle collisions, earthquakes or blast, provoke in the structures a regime of high strain-rates.
Towards the prediction of the structural responses under this type of loading conditions, the utilisation of laboratory experimentation with innovative techniques can provide a strong base for the validation of existing computational models, or the implementation of new ones. In order to assess the computational model is necessary a deep knowledge of the high strain-rate behaviour of the materials as concrete and FRC.
It is then essential to understand which type of the traditional materials (like concrete) are capable of improving the safety of the structures and in which direction it is better to drive our efforts. In this way is fundamental the following question: is it better to have a structure in which the material dissipates the impact or a material that withstands it and then transmits to the rest of the structure important quantities of energy? The choice is between the higher resistance and the higher energy dissipation. The alternative is not decisive and should be discussed case by case. As well as in the passive safety of cars, for some structures it is worthwhile to have some elements that can be sacrificed in a strategy of energy dissipation (i.e. crash absorber elements).
After these considerations and because of the rockfall protective shelters are often made of concrete, it is then very important to know the influence of important mix parameters in concrete subjected to high loading rate, such as aggregate size, water content, etc. These mix parameters play an important role on the high strain-rate behaviour of concrete, influencing its strength, ductility and energy absorption characteristics in dynamics , properties that are fundamental for the impact design of rockfall structures.

---

[1] University of Applied Sciences of Southern Switzerland, Department of Environmental Constructions and Design, DynaMat Lab., via Trevano, CH-6952 Canobbio, +41 58 6666377 (fax: -359), ezio.cadoni@supsi.ch

Because of the restricted space at disposal this paper cannot be a state of the art review as [1-3] of the mix parameter influence of concrete at high strain rate, but it would be only a contribution to direct the design to the right type of concrete and to the right experimental technique to obtain constitutive law in case of impact.

EXPERIMENTAL METHODS TO STUDY MATERIAL AT HIGH STRAIN-RATE

The accurateness of the results in high strain-rate tests is strongly influenced by the experimental technique used for the mechanical characterization of materials. In fact, the precision of the constitutive law in reproducing the actual behavior of the materials (concrete and steel) has important consequence in the correctness of design and assessment of concrete structures subjected to impact loading. Several experimental techniques can be used for the impact testing of concrete (plain or reinforced) as falling mass, gas gun, instrumented Charpy hammer, etc., but the Hopkinson bar technique has shown distinct advantages and results in particular for the own capability to measure the wave propagation in materials produced during high strain-rates tests avoiding any filtering of signals, which could potentially cut out important information about the material behavior. For these reasons the Hopkinson bar technique is widely used to determine the mechanical properties of structural materials under high loading rates[1-4].

MIX PARAMETER INFLUENCE ON THE IMPACT CONCRETE BEHAVIOUR

The materials, generally used in the rockfall protective shelters (concrete and steel), resulted very sensitive to strain rate where the strain rate sensitivity consists of a remarkable increase of the stress at a given strain by increasing strain rate; uniform strain and fracture strain are in general also sensitive to strain rate but in a more complicated manner than the flow stress. In Fig. 1 the stress vs. strain of concrete in tension at different strain-rate is reported.
Nowadays FE codes implement simple phenomenological constitutive models of steel taking into account strain rate sensitivity like those of Cowper-Symonds or Johnson-Cook which do not include any material internal structure parameter; only recently the strain rate sensitive constitutive equation of Zerilli-Armstrong makes the attempt of taking into account one internal structure parameter like the grain size. Also the mechanical properties of plain concrete under impact loading depends in a marked manner on internal structure parameters like aggregate size [5], water-cement ratio; a further complication is introduced by the strong dependence of the dynamic mechanical properties of plain concrete on the internal relative humidity [6,7] a parameter which varies very much from construction to construction. Also for plain concrete FE codes implements simple phenomenological constitutive models not taking into account internal structure parameters and in-service damage parameters.

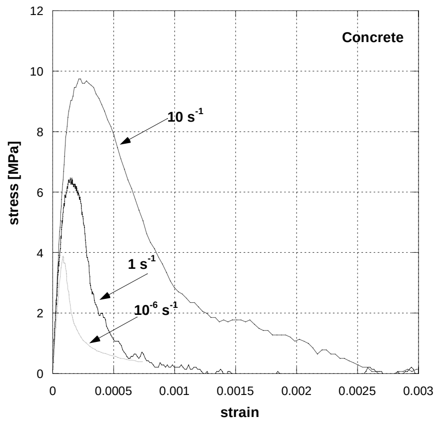


Fig. 1 Effect of the strain-rate in tension

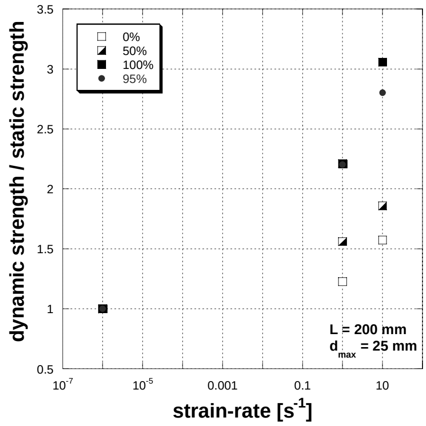


Fig. 2 Effect of the free water on strength

The free water in concrete influences the concrete property under dynamic loading. Water in concrete exists as free water and as chemically combined water. In order to explain the rate effect for concrete different hypothesis have been given: the Stefan effect [5] and the wave propagation [6]. In Fig. 2 are shown the growing curves of the ration between the dynamic and static strength, called here the Dynamic Increasing Factor (DIF) , for the four different curing history described in [6]. The aggregate size influences the behaviour of concrete both in tension and compression.
Comparing the dynamic strength normalized with the respect to the static strength as DIF in function of the strain rate it can be observed that smaller maximum size of aggregate improve the strength during the impact loading (see Figs. 3-4).
This increment is more remarkable in tension [2,7] rather than in compression [1,8].

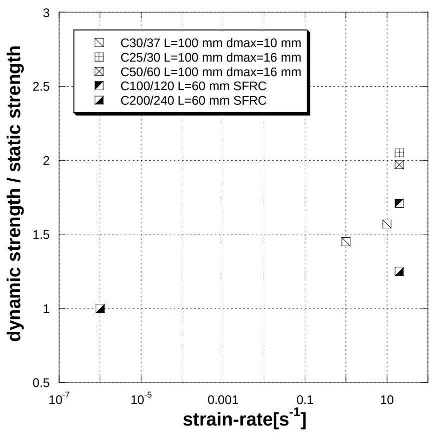


Fig. 3 Effect of the strain-rate in compression

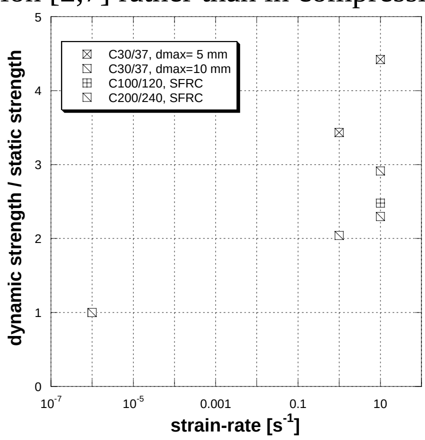


Fig. 4 Effect of the strain-rate in tension

CONCLUSION

It has been shown that precision impact testing based on the Hopkinson bar technique can provide an accurate phenomenological description of the steel and concrete response at high strain rate also in function of internal structure parameters of such materials. Attempts have been done with some success to implement the experimental results obtained for steel in dynamic constitutive equations which have with success implemented in realistic numerical computations of automotive crash or in reactor accidents simulations validated by structural experiments. It is believed that the development of theoretical models of constitutive equations capable of representing the phenomenological experimental results for concrete in function of internal structure parameter, like those reported in the paper, is needed in order to achieve realistic progress in the design of impact resistant concrete rockfall protective shelters structures.

REFERENCES

[1] BISHOFFS P.H., PERRY S.H. (1991) Compressive behaviour of concrete under high strain rates. Materials and Structures. 24, 425–450.
[2] MALVAR L.J., ROSS C.A. (1998) Review of strain rate effects for concrete in tension. ACI Material Journal. 95 (6), 735–739.
[3] ROSS, C.A., TEDESCO, J.W., KUENNEN, S.T. (1995) Effects of Strain Rate on Concrete Strength, ACI Materials Journal, 92(1), 37-47.
[4] ALBERTINI C., CADONI E., LABIBES K. (1999) Study of the mechanical properties of plain concrete under dynamic loading, Experimental Mechanics, 39 (2), 137-141.
[5] CADONI E. et al. (2001) Influence of the Aggregate Size on the Strain-Rate Tensile Behaviour of Concrete, ACI Materials Journal, 98(3), 220-223.
[6] ROSSI P., VAN MIER J.G.M., BOULAY C. AND LE MAOU F. (1992) The dynamic behaviour of concrete: influence of free water, Materials and Structures, 25, 509-514.
[7] CADONI E., LABIBES K., ALBERTINI C., BERRA M., GIANGRASSO M. (2001) Strain-rate effect on the tensile behaviour of concrete at different relative humidity levels, Materials and Structures, 34(235), 21-26.
[8] CADONI E., SOLOMOS G., BERRA M., ALBERTINI C. (2005) High strain-rate behaviour of plain concrete subjected to tensile and compressive loading, Proc. 3rd Int. Conf. on Construction Materials. ISBN 0-88865-810-9.

AN UNCOUPLED APPROACH FOR THE DESIGN OF ROCKFALL PROTECTION TUNNELS

Francesco Calvetti[1], Claudio di Prisco[2]

This paper concerns the definition and the discussion of an uncoupled approach for designing rockfall protection tunnels. The experimental results obtained by performing in situ impact tests on a real structure are first presented, in order to highlight the main features of the complex impact phenomenon and to introduce and justify the hypotheses that characterise the proposed design approach. This latter yields to the definition of the time history of stresses acting on the upper face of the reinforced concrete slab during an impact, starting from the evaluation of the impact force acting at the boulder-soil interface, and subsequently considering the stress propagation through the soil stratum.
In order to critically discuss this approach, the impact force on the boulder, the vertical stresses on the upper face of the reinforced concrete slab and the displacements of the structure due to the impact of the boulder were measured and compared with results obtained by performing a dynamical analysis of the shelter slab where the stresses provided by the design approach were used as input.

Keywords: rockfall shelters, impact force, stress propagation, dynamic structural response, uncoupled design approach

INTRODUCTION

This note presents a new design approach for rockfall shelters, which has been recently proposed by the Authors [1] with the aim of bettering standard design approaches. These latter take into consideration only static loads (soil and structure weights) or, in some cases, take fictitiously into account impacts loads by practically using a quasi-static approach.
The conceived method is based on the assumption that the event can be approached by considering separately (I) the impact of the rock block on the surface of the soil stratum placed on the top of the rockfall protection structure, (II) the propagation of impact waves within the dissipative cushion and (III) the dynamic response of the underneath reinforced concrete structure. Steps (I) and (II) can be performed with the help of design charts that have been conceived ad hoc, and take into consideration the design impact (block size and impact velocity/falling height) and the mechanical and geometrical properties of the soil layer (relative density, stiffness, thickness). Considering the widespread typologies of sheltering structures (open tunnels, cantilever shelters, polycentric tunnels) step (III) has to be performed numerically, with the help of an adequate dynamical numerical code.
For the sake of brevity, in this note only the fundamental aspects of the phenomenon and of the proposed design approach are presented. Full details are given in the reported references.

[1] Politecnico di Milano, Dipartimento di Ingegneria Strutturale, piazza L. da Vinci 32, I 20133, Milano, +39 0341 48-8861 (fax: -8771), francesco.calvetti@polimi.it
[2] Politecnico di Milano, Dipartimento di Ingegneria Strutturale, piazza L. da Vinci 32, I 20133, Milano, +39 02 2399-4298 (fax: -4220), claudio.diprisco@polimi.it

EXPERIMENTAL EVIDENCE

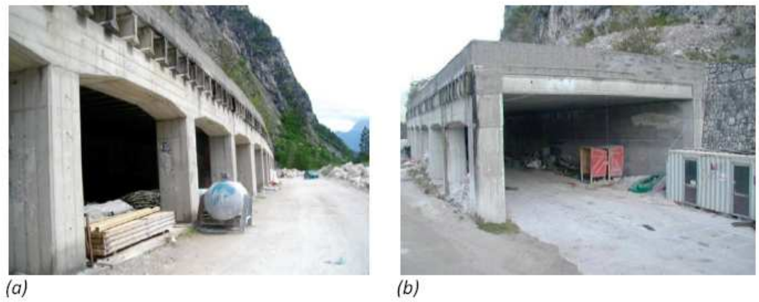


Fig. 1 Side and front view of the rockfall shelter

Selected results will be presented with the aim of demonstrating the relevance of the main hypotheses of the approach, and in particular: (*i*) the possibility of uncoupling the evaluation of the impact force from the analysis of the subsequent impact wave propagation through the soil stratum; (*ii*) the possibility of uncoupling the evaluation of the stresses acting on the reinforced concrete slab from the deformation of the slab. The impact experiments were performed on a dismissed portion of a regional road in the Dolomites, in the vicinity of Listolade (BL) by using a RC sphere (mass: 850 kg, diameter: 0.9 m; falling height 5-45 m). The following measurement devices were used: (a) one accelerometer in the RC sphere; (b) two load cells on the shelter slab; a series of LVDT and strain gauges underneath the slab in order to measure its deflection. Prior to testing the soil stratum has been leveled to a thickness of 2 m.

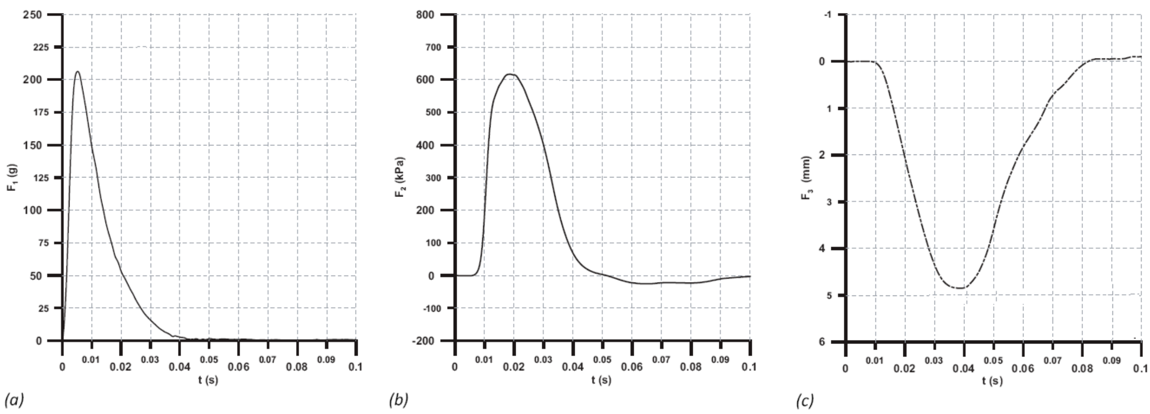


Fig. 2 Impact #5: (a) block deceleration; (b) stress increment on the slab; (c) slab deflection

Typical impact results are shown in Fig. 2 (test #5, falling height 36.4 m). The following points should be emphasized: the maximum value of the block deceleration is attained before the impact wave has propagated to the slab (point *i*); the initiation of the slab deflection is simultaneous with the impact wave arrival at the slab, i.e. the slab cannot be considered as a rigid boundary for the stress wave propagation. However, even if the recorded peak acceleration of the slab is as large as 6 g, the maximum velocity of the slab itself is small (0.2 m/s in this case) and it can be conservatively neglected (point *ii*).

DESCRIPTION OF THE DESIGN APPROACH

Within the framework of the proposed design approach, and considering the complexity of the phenomenon under investigation, a series of numerical analyses have been conducted with the aim of providing usable information in the form of design charts for evaluating the impact force (as a function of time) and the stress acting on the slab (as a function of time and distance from the impact point). A simplified numerical approach is proposed for obtaining the time evolution of the impact force starting from the results obtained with the rheological BIMPAM model [3] of the penetrating boulder (Fig. 3a). All the parameters of Fig. 3 can be determined with the help of abaci as a function of boulder size and falling height for different

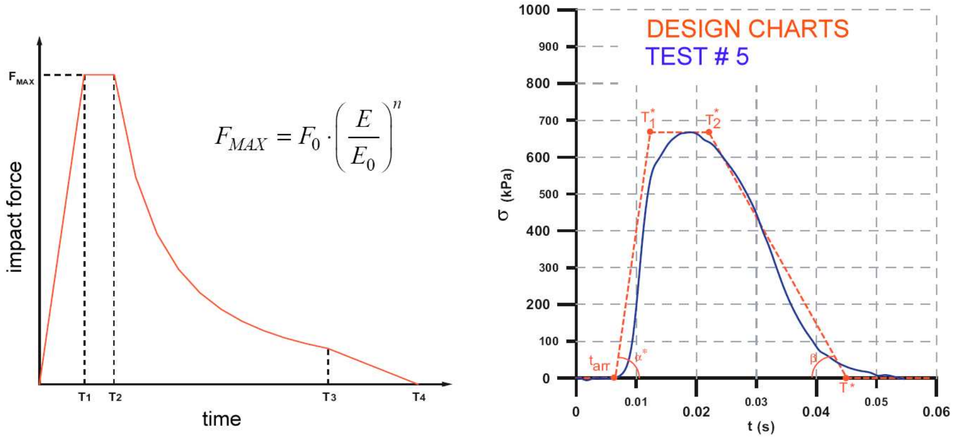


Fig. 3. (a) Simplified time evolution of impact force; (b) stress increment on the slab under the impact point

type of granular soils (dense or loose, see [1]). A similar approach is followed as far as the time evolution of the stress distribution on the slab is concerned (Fig. 3b, the comparison with the results of test # 5 is also shown). The maximum value of the stress on the plate, which is a function of the distance from the impact point, can be determined as $\sigma_{MAX}=\sigma^i_{MAX}\, f_g\, f_a$, where σ^i_{MAX} is the maximum nominal stress at the boulder-soil interface, f_g is an elastic lateral diffusion factor and f_a is a dynamic amplification factor which is a function of the elastic stiffness of the soil [1].

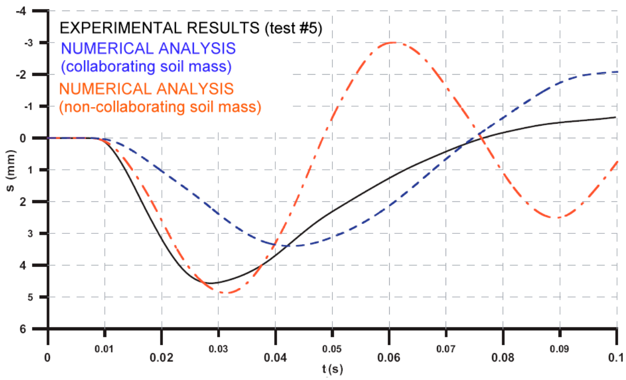


Fig. 4 Test # 5. Slab deflection.

CONCLUSION

As a concluding example, we show an application of the proposed approach. Test # 5 is simulated by applying the previously defined input load to the slab of the shelter, which is modeled as an orthotropic elastic plate in order to account for the presence of the girder elements (see Fig. 1). Two analyses are performed with the GeoELSE code [4]: in the first one the mass of the soil is considered to follow the oscillation of the slab; in the second, the possibility of a soil-slab detaching due to the large initial acceleration of the slab itself is considered. In this latter case, the initial structural response is well reproduced, while the first hypothesis involves a better performance of the model during the following rebound of the slab.

REFERENCES

[1] CALVETTI, F., C. DI PRISCO (2007) Linee guida per la progettazione di gallerie paramassi. Starrylink Editrice, Brescia. ISBN: 978-88-89720-79-0.

[2] CALVETTI, F., C. DI PRISCO, M. VECCHIOTTI (2006) Experimental and Numerical Study of Rock-Fall Impacts on Granular Soils. Rivista Italiana di Geotecnica, 39 (4), 95-109.

[3] DI PRISCO, C., M. VECCHIOTTI (2006) A Rheological Model for the Description of Boulder Impacts on Granular Strata. Geotechnique, 56, 7, pp. 469-482

[4] STUPAZZINI, M., C. ZAMBELLI (2005) *GeoELSEvp: a Spectral Element Approach for Dynamic Elasto-viscoplastic Problems*", Rivista Italiana di Geotecnica 39(4), 70-82.

TOWARDS OBJECTIVE ROCKFALL TRAJECTORY MODELLING USING A STOCHASTIC REBOUND ALGORITHM

L. Dorren, F. Bourrier, F. Nicot, F. Berger[1], F. Darve[2]

A challenge in rockfall modelling is to develop a robust rebound algorithm for which parameter values can be estimated objectively in the field. In this paper, we tackle this challenge by presenting and testing a stochastic impact law that requires limited parameterisation by the model user. The approach provides promising results. We will present the simulation results and their validation as well as remaining unknowns and future developments.

Keywords: probabilistic modelling, impact law, real size experiments, rockfall trajectory

INTRODUCTION

To reduce the risk posed by rockfall, rockfall simulation models are increasingly used for calculating required strengths of protection measures such as nets and dams or for making hazard maps. A key process that has to be simulated in such models is the rebound, for which a wide range of algorithms is currently available. Each of these algorithms requires its specific set of parameters. In general these parameters are difficult to estimate objectively in the field. This explains the large variation in the results when applying different models, or even the same model used by different operators, at the same site. To come up with a more objective procedure we tested a stochastic impact model [1], which differentiates only 5 different impact cases, determined by the ratio of the radius of the falling rock to the mean rock radii constituting the impacted surface.

To test the performance of this stochastic impact model, we implemented it in a mixed probabilistic/deterministic, 3D rockfall simulation model. The specific objective was to evaluate whether this model is able to reproduce the rockfall data from real size rockfall experiments on an Alpine slope.

METHODS

The used 3D rockfall simulation model is Rockyfor, described in detail by [2], which is a mixed probabilistic/deterministic model. The rebound algorithm in this model is essentially based on Pfeiffer and Bowen (1989), which decomposes the incoming velocity before the impact in a normal and tangential component. Two energy restitution coefficients consequently reduce the magnitude of both components to account for energy loss during the impact on the surface. The essence of a well functioning simulation of rockfall rebounds is the exact attribution of the restitution coefficient values for different parts of the slope where rockfall is active. To increase the objectivity of the attribution of the tangential restitution coefficient (r_t) values, Rockyfor demands a description of the slope (top sub-) surface based

[1] Cemagref Grenoble, 2 rue de la Papeterie, BP 76, 38402 Saint Martin d'Hères Cedex, France.
luuk.dorren@cemagref.fr, Tel: +33 4 7676 2806
[2] L3S-R, INPG - UJF - CNRS, Domaine Universitaire, BP 53, 38 041 Grenoble Cedex 9, felix.darve@inpg.fr

on three probability-size classes. The user needs to provide the surface roughness (average height difference between particles covering the slope surface and the mean level of the slope) of the material that the falling rock encounters during 70%, 20% and 10% of the impacts on the surface. Experience learned that these three classes were very useful in practice. Rockyfor then calculates the r_t using the radius of the falling rock and the value given by one of the three probability-size classes. A randomly drawn number determines if the value of the 70%, 20% or 10% is used. A second probabilistic factor is included by a uniform random variation of 10% of the calculated r_t and the given normal restitution coefficient (r_n). Finally, the local slope angle at the position of impact is varied randomly with 4º [3].

The stochastic impact algorithm implemented for this study is described by [1]. For five ratios between the size of the falling rock and the mean size of the soil particles (1:1 up to 5:1; larger ratios fall outside the validity domain of the model), 3 impact parameter sets have been derived. These 3 impact parameter sets describe multidimensional functions for: 1) the reflected velocity after rebound (V_r), 2) the angle between the slope and the rock after rebound (α_r) and 3) the reflected rotational velocity (ω_r) in relation to the incidence velocity (V_i), incidence rotational velocity (ω_i) and impact angle (α_i). These functions are multi-dimensional, normal probability density functions that account for the local variability in the soil and the variation in responses to changing combination of velocities, rotational velocities and impact angles. Figure 1 present a graphical presentation of such a function for V_r. Similar functions are implemented for ω_r and α_r. In summary, as input this algorithm only needs the size of the falling rock and the average size of the material in the slope (top sub-) surface. Estimating values for the r_t and the r_n is not necessary.

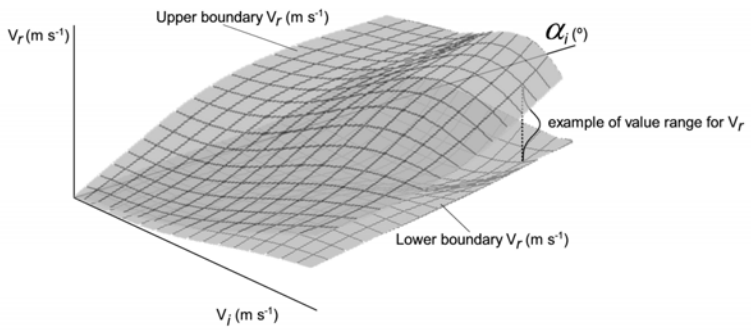


Fig. 1. Graphical representation of the multi-dimensional normal probability distribution of V_r as function of V_i and α_i.

We carried out 100 – 50000 simulations using the Rockyfor method of probabilistic attribution of the encountered surface roughness during a rebound. During each rebound one of the three probability-size classes were expressed in one of the 5 size-ratios between the radius of the surface material and the simulated rock (1:1 … 1:5).

To validate the 3D rockfall simulation we used data from real size rockfall experiments in an avalanche couloir [4]. Data available included the translational velocities (m s−1), the vertical passing heights (m) and the translational kinetic energies (kJ) at two "evaluation lines". Evaluation line 1 is located after 185m, measured over the slope, from the rock release point and evaluation line 2 is located after 235 m. We created a Digital Elevation Model (DEM) with a resolution of 2.5 m, covering the experimental site from the release point to the opposite river bank in the valley bottom, which is used a 3D topographical basis for the rockfall modelling in this study. The slope surface polygon map was mapped in the field on the basis of a generated contour line map. The rock volumes used during the experiments follow a normal distribution with a mean of 0.8 m^3 and a standard deviation of 0.15 m^3. This distribution was used for the simulations as well.

RESULTS

All the simulations reproduced the observed mean velocities and passing heights at the two evaluation lines with a mean error of respectively -15% and -20% (Tab. 1). The mean translational kinetic energy was underestimated with a mean error of -23%. The observed maximum velocities and translational kinetic energies were relatively well simulated with mean errors of respectively 0.5% and 30%. The maximum passing heights were largely overestimated with a mean error of 119%. Another interesting result was that the mean variation in the simulated results became less than 5% after 1000 simulations.

Tab. 1 Results after 5000 simulation runs.

	Velocity (m s^{-1})			Passing height (m)			Kin. E. trans (kJ)		
	mean	stdev	max	mean	stdev	max	mean	stdev	max
EL* 1 observed	12.5	5.2	28.1	1.4	1.1	5.0	205	169	789
EL 1 simulated	11.5	4.2	27.7	1.2	0.9	9.9	175	126	1081
EL* 2 observed	13.8	5.5	28.9	1.6	1.4	6.2	245	197	958
EL 2 simulated	10.9	4.7	29.6	1.2	1.0	14.9	167	139	1174

* EL = Evaluation line

DISCUSSION

Although simulation errors expressed in percentages seem quite high (especially those exceeding 20%), we are satisfied with the performance of the stochastic impact model. The overestimation of the maximum passing heights can be due to the limited number of real size experiments. Additional analyses show that observed and simulated distributions match very well. Looking at the data in Tab. 1, the results produced by the model would be perfectly useable for designing protective measures at the position of the two evaluation lines. This is because the maximum values were overestimated while giving the correct range of velocities, energies and passing heights required for engineers (to be discussed at the workshop).

The presented approach is interesting since it allows for a more objective assignment of the r_t. Moreover, the r_n or additional parameters are not needed in this algorithm. A critical point is that fine soils do not occur at our test site. It would be interesting to test the algorithm on a site with meadows, compacted soil, etc. Our next step is to evaluate whether the fieldwork can be simplified by measuring only one dominant size class per homogeneous terrain unit. This would imply that quite some information on the variability of the material sizes in the terrain would be lost. However, with the estimation of the three probability-size classes being redundant, the field phase would be significantly optimized.

REFERENCES

[1] BOURRIER, F., F. NICOT, F. DARVE (2008) A stochastic framework for the modelling of the impact of a boulder on a coarse soil. Interdisciplinary Workshop on Rockfall Protection. June 23-25, 2008 in Morschach, Switzerland.

[2] DORREN, L.K.A., F. BERGER, U.S. PUTTERS (2006) Real size experiments and 3D simulation of rockfall on forested and non-forested slopes. Natural Hazards and Earth System Sciences 6, 145-153.

[3] PFEIFFER, T.J., T.D. BOWEN (1989) Computer simulation of rockfalls. Bull. Ass. Eng. Geol., XXVI, 135–146.

[4] DORREN, L.K.A., F. BERGER, C. LE HIR, E. MERMIN, P. TARDIF (2005) Mechanisms, effects and management implications of rockfall in forests. For. Ecol. Manage. 215(1-3), 183-195.

IMPACT OF ROCKFALLS ON REINFORCED FOAM GLASS CUSHION SYSTEMS

Werner Gerber[1], Axel Volkwein[1], Matthias Denk[2]

The trajectory of a falling rock is mainly determined by bounces on the ground or with trees reducing continuously the rock's kinetic energy thus braking it. Free fall experiments on a horizontally positioned concrete slab covered by a special cushion system allow an analysis of the braking process. The process has been recorded using high-speed video and acceleration sensors attached to the rock. These interdependent measurements allow deduction of the kinetic process, i.e. velocity and displacement, obtained from integration of the rock's acceleration and independently from differentiating the video displacements. This contribution shows the validity of both methods and additionally suggests how to evaluate and to classify e.g. novel cushion systems.

Keywords: rockfall, experiment, impact, deceleration, protection structure

INTRODUCTION

The stopping process of falling rocks by protection systems causes deceleration effects that can not predicted easily [1]. The mass and velocity of a falling rock constitute its kinetic energy. The absorption/transformation of this energy into deformation work or heat energy happens during a certain impact time along the braking distance. The latter can reach several meters in a flexible protection system. On the contrary, an impact on almost rigid surface results in a very short braking distance and impact time with very large impact forces. Therefore, concrete galleries usually are protected by an additional cushion layer, which mostly consists of granular soil. In this paper we report on the performance of a new type of cushion material made from foam glass (density = 130-160 kg/m^3, grain size = 10 / 25 mm, friction angle = 55°) has been tested under different load conditions [2]. The cushion material is used normally as aggregate for light-weight and heat-insulating concrete. The test results are compared to the guidelines for loads on rockfall protection galleries [3, 4].

Figure 1 Instrumented 800 kg model rock with cushion system (diameter = 3m, height = 1.2m) prior to tests [2]

[1] WSL Swiss Federal Institute for Forest, Snow and Landscape Research, Zuercherstr. 111, CH-8903 Birmensdorf, +41 44 7392-469 (fax: -215), werner.gerber@wsl.ch, axel.volkwein@wsl.ch
[2] Geobrugg AG Protection Systems, Hofstr. 55, CH-8590 Romanshorn, +41 71 4668196, matthias.denk@geobrugg.com

TEST METHODS AND ANALYSIS

Three layers of the cushion material (foam glass) with a thickness of 0.4 m each were placed on the concrete slab (Figure 1). Between the layers and as lateral bound a high-strength steel net was used. The test specimens were equipped with 6 acceleration sensors connected by wire with the central data logging unit. A total of 12 tests were carried out with two different impact masses and falling heights ranging between 2 and15 m (Table 1).

Table 1 Performed tests on special cushion system

Test No.		C1	C2	C3	C4	C5	C6	C7	C8	F1	F2	F3	F4
Mass *m*	(kg)	800	800	800	800	800	800	4000	4000	4000	4000	4000	4000
Falling height	(m)	15	5	5	5	10	15	2	5	7.5	7.5	7.5	7.5
Energy	(kJ)	120	40	40	40	80	120	80	200	300	300	300	300

For a subsequent comparison with the existing FEDRO[3] guidelines [3, 4] the peak force has to be converted into a corresponding acceleration. The guidelines formulate the relation between the characteristic brake force F_k, the impact velocity v and the brake distance d as

$$d = mv^2 / F_k \qquad (1)$$

Assuming F_k to be the maximum acting brake force F the acceleration a is calculable by the use of the 1st law of Newton ($F = m \cdot a$) and equation (1) as

$$a = v^2 / d \qquad (2)$$

stating that the deceleration of a falling rock only depends on the impact velocity and the (measured) impact depth.

RESULTS

It is now possible to analyse the preciseness of both kinetic measurements. The video records (V) have a temporal and aerial resolution of *4 ms* and *~1 cm*, respectively, resulting in a grade of accuracy on the order of a few *cm* for the braking distance and a noise of *2 m/s* for the velocity calculated from the videos. The acceleration sensors (A) with a range of ±*500 g* and a resolution of ±*2 g* restrict the exact definition of the impact time to a few *ms* despite the sample rate of *10 kHz*. The resulting precision of the integrated brake distance is again defined on the order of a few *cm* which also corresponds to the precision of the measured falling height. The comparison between video and accelerations shows very similar values (Table 2).

Table 2 Comparison of measured braking from video records (V) and acceleration sensors (A)

Test No.		C1	C2	C3	C4	C5	C6	C7	C8	F1	F2	F3	F4
Impact time V	(ms)	48	40	36	40	-	36	48	52	68	56	64	48
Impact time A	(ms)	50	44	38	39	37	39	46	49	70	56	64	51
Brake distance V	(cm)	51	26	22	24	-	36	20	29	56	46	49	37
Brake distance A	(cm)	52	28	24	25	33	40	21	30	57	47	50	39
Acceleration A	(m/s^2)	530	370	390	420	615	680	170	260	330	475	320	440

The calculated values of the braking distance (i.e. the impact depth) at the given impact velocity are now compared to the impact characteristics given in the FEDRO guidelines for protection galleries [3, 4]. The deceleration curves show clearly that the existing guidelines for traditional cushion soil can also be used for the dimensioning of a gallery underneath such new cushion systems (Figure 2).

[3] FEDRO = Swiss Federal Roads Office

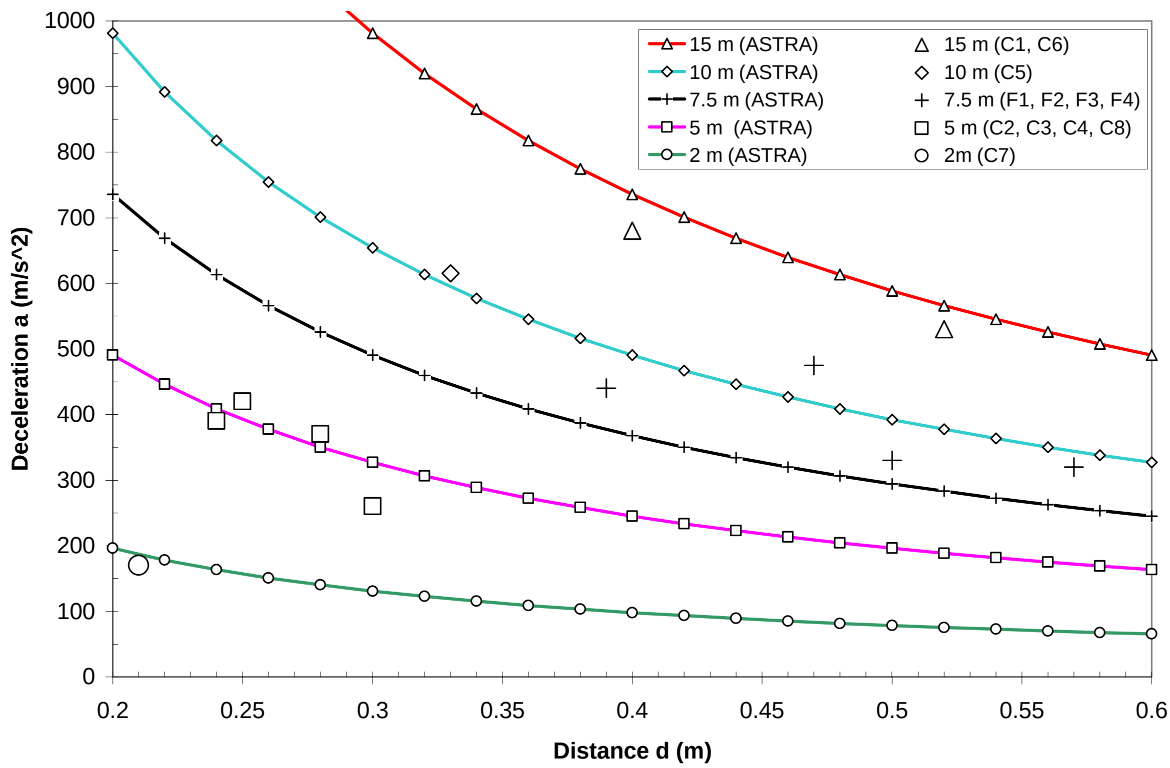


Figure 2 Measured accelerations compared with accelerations for different falling heights calculated according the FEDRO guidelines [3, 4].

CONCLUSION

The results shown above represent the first results of an extensive analysis of the tests described in [2]. After the usability of all measurements has been checked and proven, new models regarding the impact loads on galleries, the performance of different cushion layers and the non-linear dynamics of the impact will be analysed in detail.

Using different cushion material, analysis of the experiments works as shown above. A summary of all tests and some additional results are given in [2]. It can be summarized that the times for braking of the falling boulder are smaller by a factor 10 for the tested foam glass protection system compared to a traditional gravel layer. Thus, decreasing the deceleration of the falling rock by also a factor of 10. Finally, it was demonstrated that the existing guidelines for the dimensioning of protection galleries are also valid using different cushion systems.

REFERENCES

[1] Gerber, W. (2002) Peak Forces in Flexible Rockfall Barriers. International Congress INTERPRAEVENT 2002 in the Pacific Rim – Matsumoto/Japan. Congress publication, volume 2, pp.761-771.
[2] Schellenberg, K.; Volkwein, A.; Roth, A.; Vogel, T. (2007) Large-scale impact tests on rock fall galleries. In: Huang, F.L.; Li, Q.M.; Lok, T.S. (eds) Proc. 7th Int. Conf. Shock and Impact Loads on Structures. 17-19 October 2007. Beijing, China. 497-504.
[3] ASTRA (1998) Richtlinie: Einwirkungen auf Steinschlagschutzgalerien. Bundesamt für Strassen, Baudirektion SBB. Eidg. Drucksachen und Materialzentrtale, Bern. 18 S.
[4] ASTRA (2008) Richtlinie: Einwirkungen infolge Steinschlag auf Schutzgalerien. Bundesamt für Strassen (www.astra.admin.ch). 21. S.

SWISS GUIDELINES FOR THE APPROVAL OF ROCKFALL PROTECTION KITS – 7 YEARS OF EXPERIENCE

Werner Gerber[1], Reto Baumann[2], Axel Volkwein[1]

Since the introduction of the Swiss Guideline in 2001, flexible rockfall barriers consisting of wire nettings and steel posts have been systematically tested. This now results in seven years of experience concerning type-testing and the mechanical properties of individual structures but also on measuring procedures and data. In accordance with the guideline, the barriers were dynamically tested by free-falling test bodies and characterized by energy absorption classes ranging from 250 kJ to now 5000 kJ. For each test a number of different data were gathered before, during and after the actual impact. This includes forces acting on the supporting ropes, wire netting deformations and brake times and distances of the test specimen.

Keywords: rockfall protection, guidelines, type testing, experience

INTRODUCTION

In 2001 the Swiss Guidelines for the approval of rockfall protection kits [1] became operative as the first governmental guideline worldwide. It was the result of 13 precedent years of field testing of flexible rockfall barrier systems. As also written in [2, 3] various types of wire nettings of various constructions have been developed in Switzerland. In order to enable a quantitative assessment of their respective characteristics and qualities, the Swiss Federal Office for the Environment, BAFU, which subsidizes these protective measures, has issued guidelines for the testing of wire nettings that protect against rockfall. So far eighteen different types of wire nettings have been submitted for tests, fifteen of which succeeded in complying with the norms set by the guidelines. In this report we present individual test results that may answer various theoretical and practical questions. It will be shown that a relationship exists between the kinetic energy of the impacting stone and the deformation of the protective net, varying with the length of the trajectory and the moment the falling body was first braked. These data enable us to evaluate the degree of stiffness or softness of the braking process (abrupt- / gentle braking processes). The results shown in this contribution can be found more detailed in [3].

METHODS

In order to test the capacity of wire net barriers under realistic conditions, a test facility that meets certain requirements is needed. Stones falling from slopes mostly move in a bouncing motion. Maximum velocity of the stones occurs at the end of the trajectory, shortly before they hit ground. Previous research has found that the impact angle of the trajectory is often about 15° greater than the angle of the slope [4]. When these premises are applied to various slope angles, the relevant test conditions may be derived from them (see Fig. 1). The experience resulting from different tests on different slope arrangements has proven that a vertical

[1] WSL Swiss Federal Institute for Landscape, Snow and Forest Research, Zuercherstr. 111, CH-8903 Birmensdorf, +41 44 7392-469 (fax: -215), werner.gerber@wsl.ch, axel.volkwein@wsl.ch
[2] Bundesamt für Umwelt BAFU, Sektion Schutzwald und Naturgefahren, CH-3003 Bern, +41 31 3247838, reto. baumann@bafu.admin.ch

test installation also enables test circumstances 'approaching nature', provided the correlations between the slopes remain the same. This means that the protective nets in a vertical test setting have to be mounted at an angle of 30° to the horizontal.

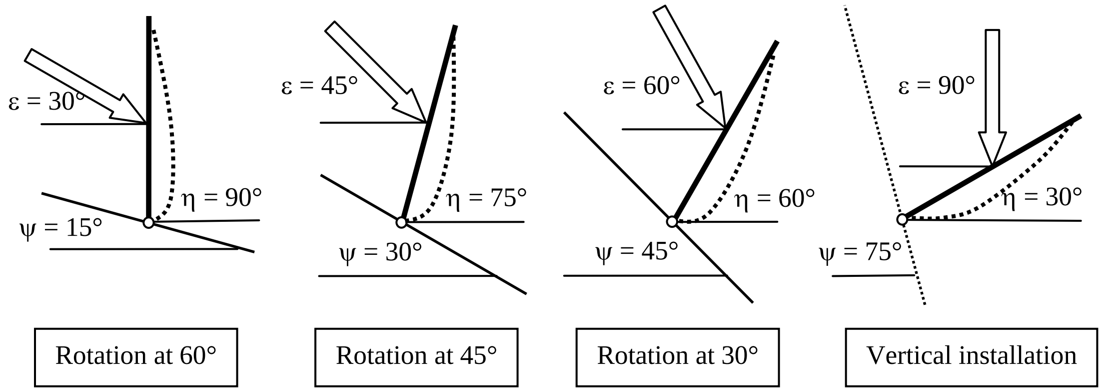


Fig. 1 Schematic representation of various sloping arrangements and a vertical arrangement. Rotation in respect of vertical installation.

In the test facility Lochezen at Walenstadt the examined protection systems are mounted on an almost vertical slope, four steel posts attached on foundation bases with three fields of 10 m width. The post heads are retained by retaining ropes, the support ropes along the barrier hold the net. Depending on net type, the support ropes usually contained incorporated braking elements in order to keep the impact forces at a predetermined level and to slow down any further increase of these forces. Measuring cells are also incorporated in the ropes in order to register these forces (Fig. 2) sampled with 2000 kHz during the test. Additionally, the movements of the bodies falling into the net as well as the resulting deformation of the net are filmed with high-speed cameras (250 frames / sec). The cameras are activated by the same signal that triggered force measurement..

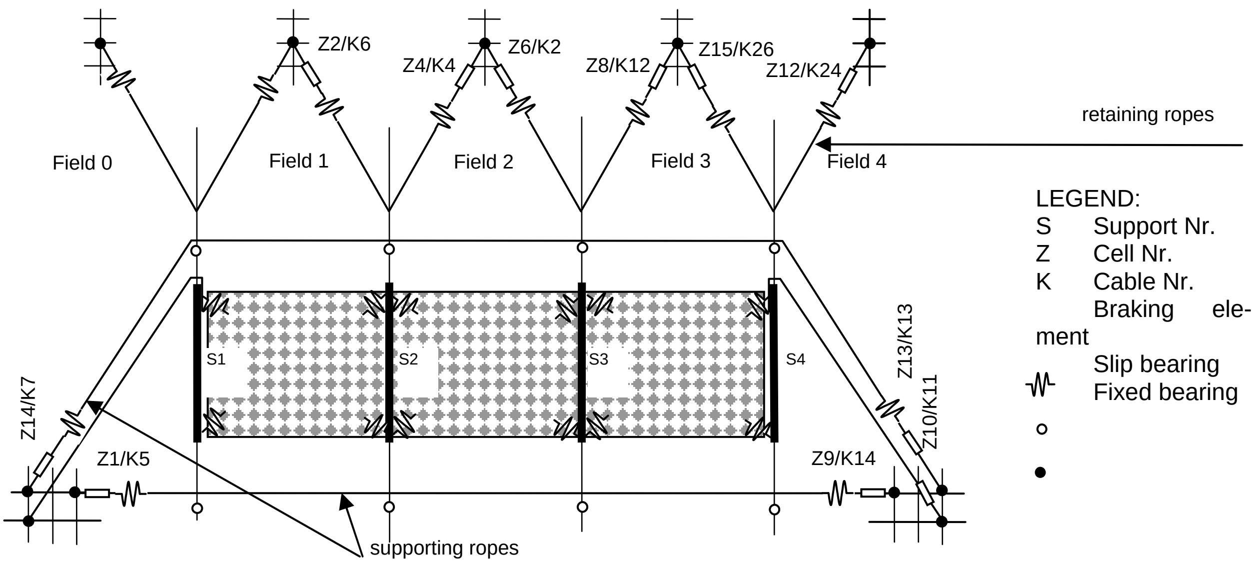


Fig. 2 Schematic positions of the supporting elements and the measuring cells.

A type-testing procedure consists of the following subtests:

a) Small energy tests (test bodies with a lateral length of 10 cm, 20 cm, and 50 cm); to test the deformations of the laid-on mesh

b) 50% energy tests (test bodies with 50 % mass); to establish the required repair effort
c) 100% energy tests (test bodies with 100 % mass); to test bearing capacity and deformability
d) Tests according to special criteria; to test the practical suitability

RESULTS

So far, out of 18 type tests, 15 net types have turned out to comply with the guidelines and so are eligible for subsidy by the Federal Office for the Environment. The most important test results from the published data in the relevant certificates (http://www.umwelt-schweiz.ch/typenpruefung) are summarised in [1]. They are the results concerning the following issues:

- The braking process of the falling bodies (braking distance and braking time)
- Maximum forces on the upper and lower supporting ropes and the retaining ropes
- Net height and remaining effective height of the net's centrefield (field 2)
- Labor required for repairs, in working hours after a 50%-test

Fig. 3 shows the brake times and distances as well as the maximum load on the retain ropes as an example for the measurement values. More results will be shown and discussed in the presentation.

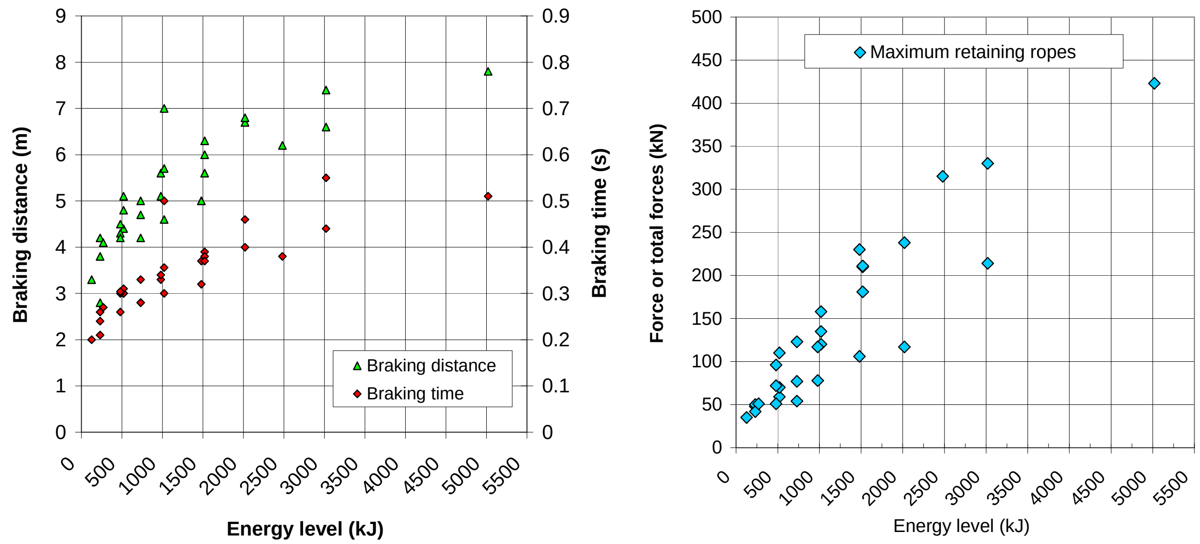


Fig. 3 Increase of braking distance and braking time of the falling bodies in the protective net in proportion to kinetic energy and measured maximum load in the retain ropes.

REFERENCES

[1] Gerber, W. (2001/2006) Guideline for the approval of rockfall protection kits. Environment in practice. Swiss Agency for the Environment, Forests and Landscape (SAEFL), Swiss Federal Research Institute WSL. Bern.
[2] Gerber, W., Boell A. (2002) Peak forces in flexible rockfall barriers, INTERPRAEVENT 2002, volume 2, p.761-771.
[3] Gerber, Werner, Böll, Albert (2006) Type-testing of rockfall barriers – comparative results, Int. Symp. Interpraevent, Niigata, Japan
[4] Zinggeler, A. (1990) Steinschlagsimulation in Gebirgswäldern. Modellierung der relevanten Teilprozesse. Diplomarbeit im Hauptfach Geographie, Geographisches Institut, Universität Bern, 114 S.CONNOR, R. J., J. W.

HALF-SCALE EXPERIMENTAL STUDY OF ROCKFALL IMPACTS ON SANDY SLOPES

Barbara Heidenreich, Vincent Labiouse[1]

In the framework of rockfall trajectory modelling, the bouncing phenomenon occurring when a rock block impacts with the slope surface is the most difficult to predict, owing to its complexity and its very limited understanding. To date, the rebound is commonly quantified by means of (one or) two coefficients of restitution estimated from a rough description of the ground material. To acquire a better knowledge of the bouncing phenomenon and to investigate the influence of the various impact parameters, a comprehensive laboratory testing campaign was undertaken at the LMR-EPFL. After a short description of the testing device and data processing, this paper focuses on the influence of some impact parameters. It is observed that the rebound and the commonly-used coefficients of restitution depend not only on slope material characteristics, but also on factors related to the kinematics (slope inclination and impact velocity) and to the block (weight and geometry).

Keywords: Rock fall, trajectory analyses, impact, coefficient of restitution

INTRODUCTION

Rockfalls are a major hazard in densely populated mountain areas, such as the Alps. It is particularly important in these areas to have the best possible knowledge of rockfall trajectories (in particular maximum path length, height and velocity) and energies in order to determine the areas at risk and construct adequate defence systems.

On condition of correct calibration, computer codes can be considered as valuable tools for quantitative analyses of block trajectories. They usually distinguish four types of block motion: free falling, bouncing, rolling and sliding. The bouncing phenomenon that occurs when the block impacts with the slope surface, is probably the less understood and the most difficult to predict. Its behaviour is governed by the geometry and the mechanical characteristics of the blocks and the slope. Up to now, the latter properties are usually not directly represented in the computer analyses, but through one or two coefficients, called the restitution coefficients, which express the amount of energy dissipated during the ground impact.

HALF-SCALE EXPERIMENTAL CAMPAIGN

To get a better understanding of the bouncing phenomenon, a half-scale experimental campaign has been carried out at the LMR-EPFL in a shaft with a diameter of 5 m and a depth of 8 m. Blocks of different weights (up to 10 kN) and shape were dropped from various heights (up to 10 m) on a horizontal or inclined layer of sand (Figure 1). The impact process has been filmed by a high-speed camera. The analysis of the block movement before, during and after the shock allowed to gather information concerning the impact process itself (velocity and acceleration of the block, penetration into the ground material, duration of impact etc.) and to

[1] Swiss Federal Institute of Technology Lausanne EPFL, Rock Mechanics Laboratory LMR, Station 18, CH-1015 Lausanne, +41 21 69-32323 (fax -34153), vincent.labiouse@epfl.ch

determine a criterion for which the impact process is completed. By means of this criterion, the normal (R_n), tangential (R_t) and energetic (R_{TE}) coefficients of restitution have been evaluated for the mass centre of the block according to the most common formulations (ratio of the normal or tangential velocities respectively the total energies before and after impact).

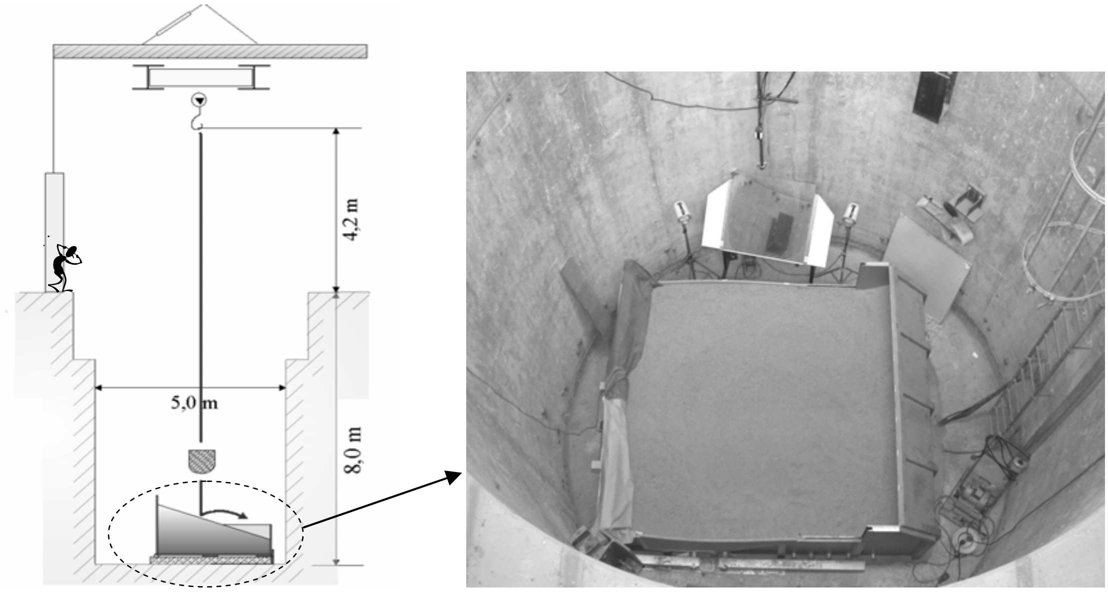


Fig. 1 - Scheme (left) and picture (right) of the half-scale testing device.

INTERPRETATION

As already pointed out in previous small-scale testing campaigns [1, 2, 3], the interpretation of the half-scale tests confirmed a very clear dependency of the coefficients of restitution on parameters related to ground, block and kinematics. The following results were obtained:

- Concerning the kinematical characteristics of the impact, the clear dependency of the coefficients of restitution on the slope inclination is confirmed (Figure 2). Furthermore, the dropping height (corresponding to the impact velocity and thus the impact energy and impulsion) is found to have a very clear influence.
- Concerning the block characteristics, the block shape has been shown to have a certain influence, especially on steeper slopes (due to the block rotation). The block mass is also confirmed to have a certain influence, depending on the radius of the block.

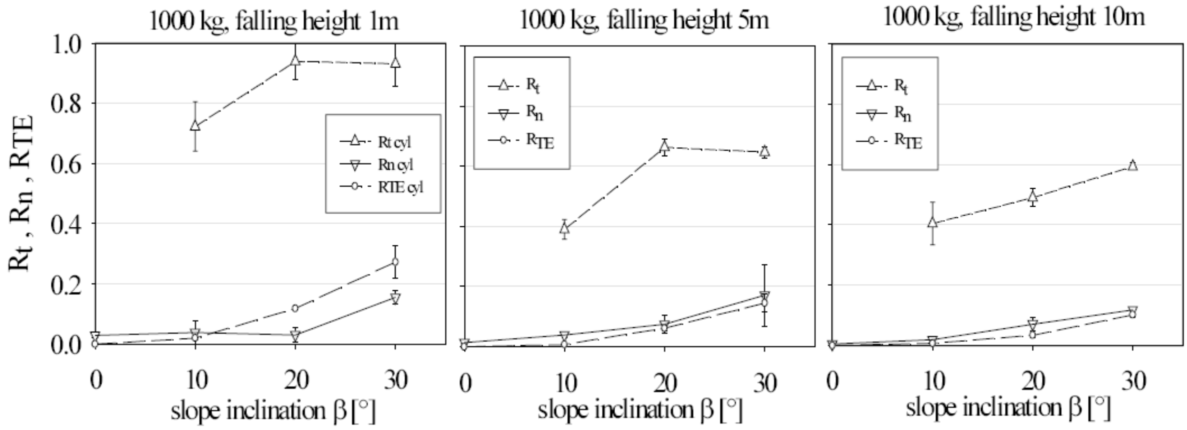


Fig. 2 - Dependency of the coefficients of restitution (normal R_n, tangential R_t and energetic R_{TE}) on the slope inclination and the impact energy (impact velocity increasing from the left to the right diagrams).

The movement of the block during impact has been found to consist of three main mechanisms acting partly antagonistically:

- a normal translation (penetration), as a function of the bearing capacity of the ground material,
- a tangential translation (sliding), depending on the mass and shear resistance of the ground material in front of the block,
- rotation of the block, as a function of the slope inclination, the impact direction and the impact velocity (by means of the depth of the crater generated during impact).

It has been shown that these mechanisms are very much interdependent, as illustrated by the following four examples: for vertical impacts on horizontal ground, no rotation and lateral translation occur. For impacts of small incident velocity (corresponding to a free fall of about 1 to 2 m) on a stable slope, no distinct crater is formed (only little vertical translation) whereas the lateral translation and the rotation of the block are very clear. For an impact of high velocity on the same slope, the vertical translation preponderates (as the penetration is large), whereas the lateral translation (due to ground failure) and rotation are less clear. If the slope inclination is close to the internal friction angle of the ground material, the ground fails superficially at impact and causes a very distinct lateral movement of the block, whereas penetration and rotation are comparatively less important. These different mechanisms occurring during impact emphasise the complexity of the impact process and explain the difficulty encountered during the development of the formulations expressing the coefficients of restitution.

Based on the aforementioned observations, the maximum penetration of the block into the sandy slope, the maximum contact force between block and ground as well as the rotational velocity of the block at impact end, have been quantified. Then, from these formulations and inspired by the principle of the conservation of linear momentum, expressions for the normal and tangential components of the coefficients of restitution were developed [4]. Their subsequent implementation in rockfall computer codes should lead to a better prediction capacity for rock block trajectories and finally to a better delineation of areas at risk by hazard maps.

CONCLUSION

The analysis of half-scale impact tests (maximum energy of 100 kJ) proves that the rebound of rock blocks as well as the coefficients of restitution commonly used to characterise the rebound depend not only on the ground characteristics, but also on parameters related to the block (weight, geometry) and the kinematics (impact velocity and angle). A thorough observation of the impacts has shown that the block motion during impact is governed by three mechanisms (penetration, sliding, rotation), acting partly antagonistically. For different impact conditions, one or another of these mechanisms is privileged, governing on its part the block motion after impact.

REFERENCES

[1] Chau, K. T.; Wong, R. H. C.; Liu, J.; Wu, J. J.; Lee, C. F. (1999): Shape effects on the coefficient of restitution during rockfall impacts. Proc. 9th Int. Congress on Rock Mechanics, ISRM, Paris 1999. Vol. 1, pp. 541-544
[2] Chau, K. T.; Wong, R. H. C.; Wu, J. J. (2002): Coefficient of restitution and rotational motions of rockfall impacts. International Journal of Rock Mechanics and Mining Sciences 39, pp. 69-77
[3] HEIDENREICH, B., LABIOUSE, V. (2004) Small-scale experimental study of rockfall impacts on granular slopes. Rivista Italiana di Geotecnica. Anno XXXVIII, n°2 (2004), pp. 80-91
[4] HEIDENREICH, B. (2004) Small- and half scale experimental studies of rockfall impacts on sandy slopes. Swiss Federal Institute of Technology Lausanne, PhD thesis N° 3059, 231 pp.

IMPACT RESPONSE ANALYSIS OF PROTOTYPE RC GIRDERS APPLYING THE EQUIVALENT FRACTURE ENERGY CONCEPT

Norimitsu Kishi[1], Abdul Qadir Bhatti[2], Hisashi Konno[3], Yusuke Kurihashi[4]

In order to establish a modification method for material properties of concrete for rational analyses using coarse mesh, an equivalent fracture energy concept for concrete elements is proposed hereby. The applicability of the method is discussed by comparison of results of numerical analysis and falling-weight impact experiments. An outcome of this study, it is observed that using coarse mesh for prototype RC girders, results similar to those obtained using fine mesh are achieved. These results are in good agreement with the experiments.

Keywords: equivalent cracking energy, impact resistant design, impact response analysis, Drucker-Prager yield criterion, prototype RC girder

INTRODUCTION

In Japan, many reinforced concrete (RC) rock-shelters were constructed to cover the highways to protect people’s lives and transportation network from falling rocks. So far, these infrastructures were designed replacing maximum impact load due to falling rocks with static one. In order to establish rational impact resistant procedure considering elasto-plastic dynamic response characteristics, not only experimental but also numerical study should be conducted.

From this point of view, we have performed falling-weight impact tests for small and prototype RC members as well as the numerical simulation of small scale RC girders by means of three-dimensional elasto-plastic finite element (FE) analysis method. From these studies, small scale RC girders under falling-weight impact loading can be rationally analyzed. However, for prototype RC members, analytical method has not been established yet, because mesh size of concrete element plays very important role in non-linear transit response behavior of RC structures. Proposing an equivalent fracture energy concept to determine the input data of material properties of concrete elements, the applicability of the method was investigated by comparing with the experimental results of a prototype RC girder. In this study, LS-DYNA code was used for numerical analysis.

EXPERIMENTAL OVERVIEW

In this study, a RC girder was chosen for falling-weight impact test of prototype RC structures. The girder is of rectangular cross section having dimensions of 1 m x 0.85 m, and the clear span is 8 m, which is similar to the width of real RC rock-shelters as shown in Fig. 1.

[1]Dept. of Civil Engrg, Muroran Institute of Technology, Muroran 050-8585, Japan, +81 143 46-5226 (Fax: 5227), kishi@news3.ce.muroran-it.ac.jp
[2]Department of Civil Engineering, Muroran Institute of Technology, Japan, bhatti@news3.ce.muroran-it.ac.jp
[3]Civil Engineering Research Institute for Cold Region, 1-3-1-34, Hiragishi, Toyohira, Sapporo, 060-8602, Japan, +81-841-4698 (Fax: -3502), konno@ceri.go.jp
[4]Dept. of Civil Engrg, Muroran Institute of Technology, Muroran 050-8585, Japan, +81 143 46-5225 (Fax: 5227), kuri@news3.ce.muroran-it.ac.jp

The thickness of concrete cover is assumed to be 150 mm similar to the real rock-shelters. D13 stirrups are arranged with intervals of 250 mm. To assure the flexural failure of the RC girder, interlayer stirrups were arranged to upgrade the shear load-carrying capacity of the member.

In this experiment, a 2,000 kg weight was lifted up to the prescribed height of 5 m by using the track crane, and was dropped freely to the mid-span of girder through a desorption device. The RC girder was set on the supporting gigues, which are made to rotate freely but not to move toward each other. The supporting points of RC girders were fixed in the upward direction using steel rods and cross beams to prevent it from jumping at the time of impact due to a heavy weight. Figure 2 shows a numerical model of this impact test.

ANALYTICAL MODELING

In numerical investigation of dynamic response behavior of the RC structures by means of elasto-plastic FE analysis method, mesh size may have a very important role if cracking occurs in the concrete elements. This work constitutes an effort directed towards the development of an objectivity algorithm for tensile failure of concrete element based on the smeared crack formulation. Thus enabling the control of energy dissipation will be associated with each failure mode regardless of mesh refinement. The advantage of the proposed technique is that mesh size sensitivity on failure is omitted leading to the results, which converge to a unique solution, as the mesh is refined.

Here, a detailed formulation and numerical implementation of an objectivity algorithm on an equivalent fracture energy G_f concept for numerical analysis of prototype RC girders will be discussed. The equivalent fracture energy G_f concept was derived based on the following assumptions:

(1) One through crack occurs in a reference solid element with longitudinal length y_0 as shown in Fig. 3(a);
(2) Even though a solid element has arbitrary longitudinal length y_i as shown in Fig. 3(b), one through crack occurs in the element as soon as reaching an equivalent tensile fracture energy for the reference element;
(3) The fracture energy for the element with arbitrary length y_i can be defined introducing fictitious tensile strength f_{ti} of concrete as $f_{ti} = f_{t0} (y_0 / y_i)^{1/2}$, in which f_{t0} is the tensile strength of reference element which is equal to one tenth of compressive strength of concrete obtained from material test; and
(4) Longitudinal length of reference element is set to be 35 mm following numerical analysis using fine mesh [1].

Figure 4 shows the comparison between experimental results and numerical ones with/without G_f concept for the mid-span displacement, adopting longitudinal length of concrete elements as 250 mm. From these results, it is confirmed that even though the spacing of stirrups was taken as the longitudinal length of concrete elements, time history of the displacement could properly be evaluated considering proposed G_f concept.

CONCLUSIONS

(1) To rationally analyze dynamic response behavior of prototype RC girders under falling weight impact loading, the concrete elements with arbitrary longitudinal length may be fractured reaching the same strain energy to that of reference concrete element; and

(2) Using the fictitious tensile strength for coarse mesh based on the proposed equivalent fracture energy concept, similar results to those obtained using fine mesh can be assured. These results were in good agreement with the experimental ones.

REFERENCES

[1] Bhatti, A.Q., Kishi, N., Konno, H. and Okada, S., (2006). "Effective finite element mesh size distribution for proposed numerical method of prototype RC girders under falling-weight impact loading", Proceedings of 2nd International Conference on Design and Analysis of Protective Structures, Singapore, Nov 13-15, 261-272.

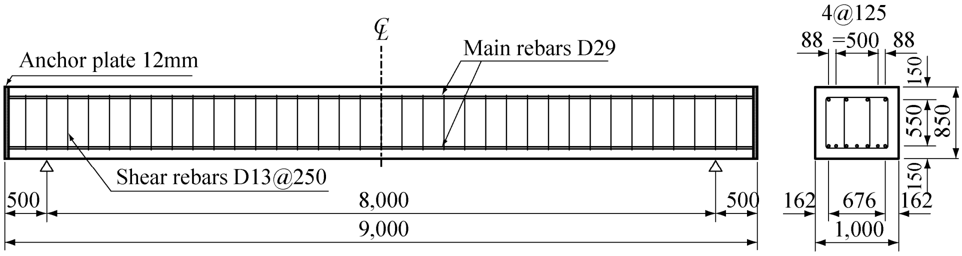
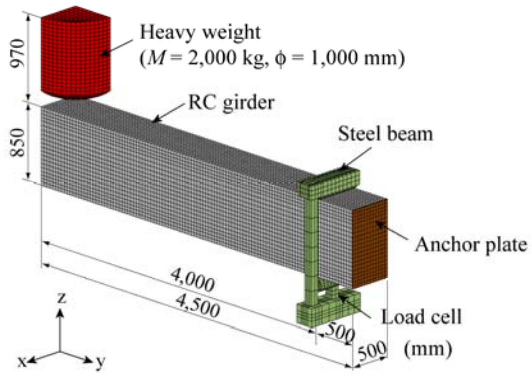
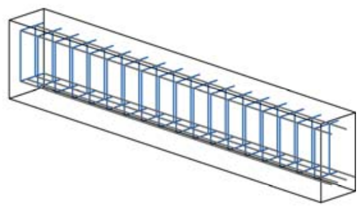


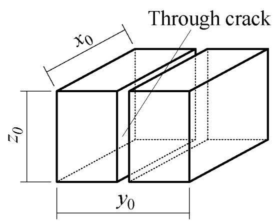
Fig. 1 Full-scale rock-shelter type RC girder subjected to weight falling



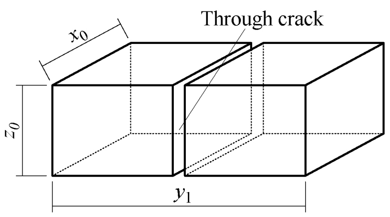


(a) FE model (b) Rebar arrangement

Fig. 2 Full-scale numerical setup model of falling-weight Impact test



(a) Crack model for reference element



(b) Crack model for element with arbitrary length y_i

Fig. 3 Diagram of equivalent G_f concept

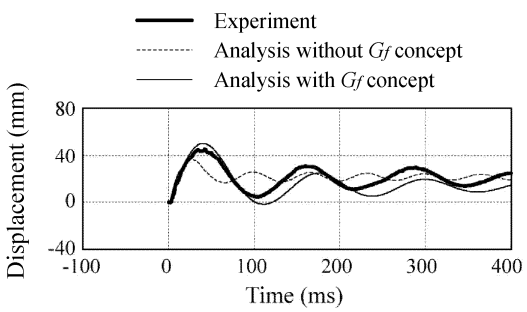


Fig. 4 Comparison between experimental and analysis results with/without G_f, adopting the longitudinal length of concrete element as 250 mm

EUROPEAN TECHNICAL APPROVALS – THE WAY FOR CE MARKING OF ROCKFALL PROTECTION KITS

Dr. Georg Kohlmaier[1]

SUMMARY

According to the Construction Products Directive 89/106/EEC [1] the European Technical Approval (ETA) is a harmonized technical specification and is considered as a favourable assessment of a product for its intended use. For rockfall protection kits the European Technical Approval Guideline 027 "Falling rock protection kits" [2] is serving as reference document for issuing ETAs. The ETA is giving the basis for the certification procedure, which is laid down in the approval document. Once, having passed the certification procedure, the manufacturer is allowed to affix the CE marking on the product according to the concerned rules.

Keywords: Construction Products Directive, European Technical Approval Guideline, European Technical Approval, Content of ETA, Energy level classification, Deformation characteristics, Evaluation of conformity, CE marking.

INTRODUCTION

In the near future designers, executors and administrative people will be confronted in their "daily business" with CE marked rockfall protection kits. This is due to the fact, that on European level the European Technical Approval Guideline (ETAG) of Falling Rock Protection Kits has been elaborated under the responsibility of the European Organisation for Technical Approvals (EOTA) over the last years. Finally, the Commission services confirmed February 1st, 2008 as the date of availability and applicability of the ETAG 027. It is used by the notified approval bodies for issuing European Technical Approvals for rockfall protection kits.

CE MARKING ON BASIS OF EUROPEAN TECHNICAL APPROVAL

The Construction Products Directive (CPD) is offering two types of harmonized technical specifications: Harmonized European standards and, as an alternative for products and product families which are not (or not yet) standardized, European Technical Approvals. Each of these two specifications forms the basis for CE marking of the covered product(s). As examples for harmonized standards those for cement (EN 197-1) and for aggregates to be used in concrete (EN 12620) are quoted.
The European Technical Approval, issued by a notified approval body, is forming the individual harmonized technical specification for a product. "Individual" because the ETA is related to an approval holder, a clearly defined product or range of products and to the concerned manufacturing plant(s). It includes individual elements for the factory production control. Fig. 1 shows the information in the CE marking label for rockfall protection kits.

[1] Austrian Institute of Construction Engineering (OIB) - Approval Body for European Technical Approvals in Austria, Schenkenstraße 4, A-1010 Vienna, +43.1.533 65 50 (fax +43.1.533 64 23), kohlmaier@oib.or.at

CE nnnn
XXXX 09 nnnn-CPD-zzzz
ETA-XX/YYYY ETAG No 027 YYY Energy level classification: 3 Residual height category for maximum energy level : A

Full legend given in cl. 3.3 in ETAG 027

Fig. 1 Example of CE marking affixed on each post of the rockfall protection kit

The abbreviation *ETA-XX/YYYY* indicates the related individual ETA (*XX* … year of issuing the ETA; *YYYY* … ongoing number) according to ETAG 027. The accompanying information to the symbols CE provide a set of information, partially related to formalistic items, partially to technical issues, specifying the key points for the possible use of CE marked products. Details are given in cl. 3.3 of ETAG 027.

CONTENT OF THE EUROPEAN TECHNICAL APPROVAL

The ETA comprises the definition of the product (description of functional modules, use of additional layers, if any) and its intended use. The intended use is referred to the verified energy level (in the ETA the service energy level (SEL) and the maximum energy level (MEL) are to be addressed) and related to an ambient temperature range (– 40°C to +50°C). If the manufacturer wants to go beyond this temperature range, additional assessment for the concerned kit, e.g. appropriate tests for its materials, would be necessary. The ETA also addresses an intended working life (provided the installed product is subject to appropriate maintenance), which is according to the ETAG 25 years, except if the product is to be used in environmental aggressive conditions. In the latter case the intended working life may be reduced to at least 10 years. Furthermore, the ETA includes information about packaging, transport and storage of the kit, just as about correct maintenance and repair is included or at least reference is given to a set of information to be attached to the delivered kit.
Detailed consideration is given to the assessment for the real performance level of the kit by means of the following performance characteristics: Energy absorption, expressed as energy level classification (see fig. 1 and tab. 1) as a combination of SEL (see tab. 1) and MEL (see tab. 1); Deformation characteristics, expressed by means of residual heights and maximum elongation; Action on the foundations of the assembled system to be declared by means of the peak force, including time force diagrams. Due to the regulations in the Member States attention is also paid to the release and/or content of dangerous substances (e.g. cadmium).

Tab. 1 Technical classes for energy level according to ETAG 027

Energy level classification	0	1	2	3	4	5	6	7	8
SEL	-	85	170	330	500	660	1000	1500	>1500
MEL≥	100	250	500	1000	1500	2000	3000	4500	>4500

The minimum content of a kit covered by ETAG 027 consists of a 3 functional module, which means 3 fields of net fences and 4 posts. The spacing of the posts is not fixed and shall be chosen by the producer.The key point is the assessment of the energy absorption by means of three impact tests, one for verification of MEL, two for SEL. The MEL level shall be chosen by the manufacturer in advance. The retained SEL value needs to be derived from the MEL value. The testing procedure shall be started with verification of SEL and continued with verification of MEL. It is possible to carry out the MEL test either in the kit used for SEL testing after reparation or in a new kit. For MEL the two following assessment criteria apply: The net fence shall stop the block and the block is not allowed to touch the ground until the kit has reached its maximum elongation during the test. The SEL level is based on assessment of two successive impacts. The assessment criteria are laid down in cl. 2.4.1.2 in ETAG 027 and cl. 4.1.5 and 4.1.6 in annex B of ETAG 027. For the 2nd SEL test the criteria are the same as for MEL. It is important to note that there is no maintenance allowed between the 1st and 2nd launch! Deformation characteristics: The residual height to be met after the MEL launch is classified as category A, B or C (see fig. 1). Category A indicates a residual height of at least 50 % of the nominal height, category B is referred to 30 % to 49 % and category C to less than 30 % of nominal height. This means that no value but the concerned category will be stated in the ETA and the CE marking label (see fig. 1). The residual height for SEL needs to be expressed as declared value on the basis of a threshold value of 70 % of the nominal height. Verification is to be done after the 1st SEL launch, but without removing the block. The maximum elongation shall be stated for all cases (1st and 2nd SEL launch and MEL launch) by means of declared values.

EUROPEAN TECHNICAL APPROVAL AND EVALUATION OF CONFORMITY

For the evaluation of conformity the attestation of conformity system 1 according to the CPD applies. This means certification of the conformity of the product by a notified certification body on the basis of defined tasks for the manufacturer and for the notified body. The elements for initial type testing (ITT), the factory production control (FPC) and the continuous surveillance of FPC are included in the ETA. Normally the approval tests are substituting the ITT. A detailed control plan for the tasks of the manufacturer and the certification body to be used in the certification procedure thereafter by the certification body, has to be deposited at the approval body. The control plan and information about identification parameters are not part of the public available ETA itself.
The basic document for affixing the CE marking is the EC certificate, issued by a notified certification body. On request, it has to be provided to the authorities in the official language of the Member State(s) of destination. In opposite to that, for the CE marking label itself an obligation for translation in the languages of Member States of destination does not exist. The CE marking label of such products will provide basic information about the performance of the product. But this does not supersede the need to consider the ETA in detail and carefully!

CONCLUSION

The ETA will operate as technical specification for the certification procedure for rockfall protection kits. Due to the fact that the concept for the verification of the energy levels in the approval procedure is rather new, it will be interesting to compare the results with those based on already existing national guidelines. In particular, it could occur that the approach for the energy level classification will lead to a new orientation in the public procurement and consequently in the availability of the range of products in the future.

REFERENCES

[1] 89/106/EEC: Council Directive of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products.
[2] ETAG 027 Guideline for European Technical Approval of Falling Rock Protection Kits. Edition February 2008.

PROTOTYPE IMPACT TEST OF STEEL-CONCRETE COMPOSITE TYPE ROCK-SHEDS

Hisashi Konno[1], Hiroyuki Ishikawa[1], Shin-ya Okada[1], Norimitsu Kishi[2]

In recent years, steel-concrete composite structures have been employed often in road bridge decks and other infrastructures. Since such structures are of high load-carrying capacity and toughness, various studies using full-scale models toward development of steel-concrete composite rock-sheds have been carried out. Following advantages can be expected for such kind of rock-sheds: 1) greater toughness than that of RC or PC rock-sheds; 2) high punching shear load-carrying capacity; 3) a simpler substructure because of lighter weight; and 4) cost reduction by reduced construction time and labors.

The steel-concrete composite rock-sheds were first applied as a new-technology pilot project in 1999. This type of rock-shed was applied for Byo-Bu-Iwa Rock-Shed (216.0 m) and Kan-Non-Iwa Rock-Shed (132.0 m) on route No. 39 of National Highway near So-Unkyo Gorge in Hokkaido, Japan. In order to verify the impact-resistance performance of the real steel-concrete composite rock-shed, a field falling-weight impact test on Byo-bu-Iwa Rock-Shed was conducted after its completion. Here, the experimental results are discussed and it is confirmed that the steel-concrete composite type rock-sheds possess high impact resistant capacity and ductility.

Keywords: rock-shed, falling-weight impact test, steel-concrete composite structure, transmitted impact force

OUTLINE OF FALLING-WEIGHT IMPACT TEST

Byo-Bu-Iwa Rock-Shed:

Byo-Bu-Iwa Rock-Shed has been constructed on route No. 39 of National Highway near So-Unkyo Gorge in Hokkaido, Japan. The rock-shed is composed of base slab and a RC sidewall in the mountain-side, concrete filled steel-tube columns, and steel-concrete sandwich structure for roof slab (hereinafter, sandwich slab). The rock-shed is 216 m long and is composed of 18 blocks of 12 m length. The field test was conducted on the eighteenth block (hereinafter, 18-BL), which is the terminal one.

Figure 1 shows the cross-section of Byo-Bu-Iwa Rock-Shed and the location of falling weight at 18-BL. The lateral span of the rock-shed in the transverse direction of road is 15.3 m. Steel panels for sandwich slab were built up by connecting pre-fabricated panels with dimension of 2,000 x 15,300 x 318 mm in the road-direction. After setting continuous steel panel at the field, high fluidity concrete was cast in the steel panel. Columns were built using high fluidity concrete filled square steel tube, the dimensions of which were 400 x 400 mm

[1] Civil Engineering Research Institute for Cold Region, 1-3-1-34, Hiragishi, Toyohira, Sapporo, 062-8602 JAPAN, +81-11-841-1698 (fax:-3502), konno@ceri.go.jp; hi-iskw@ceri.go.jp; okada@ceri.go.jp
[2] Muroran Institute of Technology, 27-1, Mizumoto, Muroran, 050-8585 JAPAN, +81-143-46-5226 (fax:-5227), kishi@news3.ce.muroran-it.ac.jp

and the thickness was 12 mm. The filled concrete was continuously cast in the roof of sandwich slab and columns and those were combined with each other.

Sandwich panel was composed of the upper plate with SS400 steel and lower plate with SM490 and those were of 9 mm thickness. Both plates were fixed installing spacers with the intervals of 300 mm in the cross-directions. Steel pipes were used for the spacers, which were 65 mm in diameter, 9 mm thick, and 300 mm long. The spacers were tightened using high tensile bolts with 22 mm diameter for the upper and lower plates to behave as single structure. Here, 70.6 kN of pretensioning force was introduced in each bolt. Figure 2 shows cross-sectional view of steel panel. On the roof of rock-shed, 900 mm thick sand cushion and 200 mm thick shatterproof soil were set.

Experiment overview:
Total seven cases were tested varying falling height and location of a heavyweight. The tests were conducted lifting a 20-ton round-headed falling-weight having 1 m in diameter up to a pre-determined height using truck crane and free falling. Fall locations of heavyweight were determined along the centerline of 1 panel and the connecting line of two panels with each other. Test cases considered here are listed in Table 1.

Based on the design value and/or three-dimensional dynamic response analysis, falling height of a heavyweight was determined so that rock-shed behaves elastically. The maximum falling height was h = 17.5 m in case C2. In this experiment, following time histories were measured: 3 chs. of weight impact force through acceleration, 15 chs. of transmitted impact stress through load-cells set on the roof, 122 chs. of strain of roof plate and column tube and 22 chs. of displacement of roof and column.

CONCLUSIONS

In this study, in order to confirm an impact resistant capacity of the steel-concrete composite type rock-sheds, falling-weight impact tests were conducted on Byo-Bu-Iwa Rock-Shed, Hokkaido, Japan. From this study, following results were obtained:

1) Maximum weight impact force and transmitted impact force through sand cushion can be better estimated by using Hertz's contact theory with Lame's constant of λ = 1,500 kN/m^2. The results were different from those obtained in cases of sand cushion set on the rigid base and/or roof of RC type rock-sheds;
2) Duration time of each impact force is almost twice of that in cases of sand cushion set on the rigid base; and
3) It is confirmed that response stresses of plates in roof slab are less than that of design values for all measuring points considered here, and the measured values keep more than twice of safety margin for allowable stress at design.

Therefore, even though this steel-concrete composite type rock-shed was developed in order to reduce the construction cost, duration, and labors, it is confirmed that it also retains a plenty of impact resistant safety margin than that of RC/PC type rock-sheds.

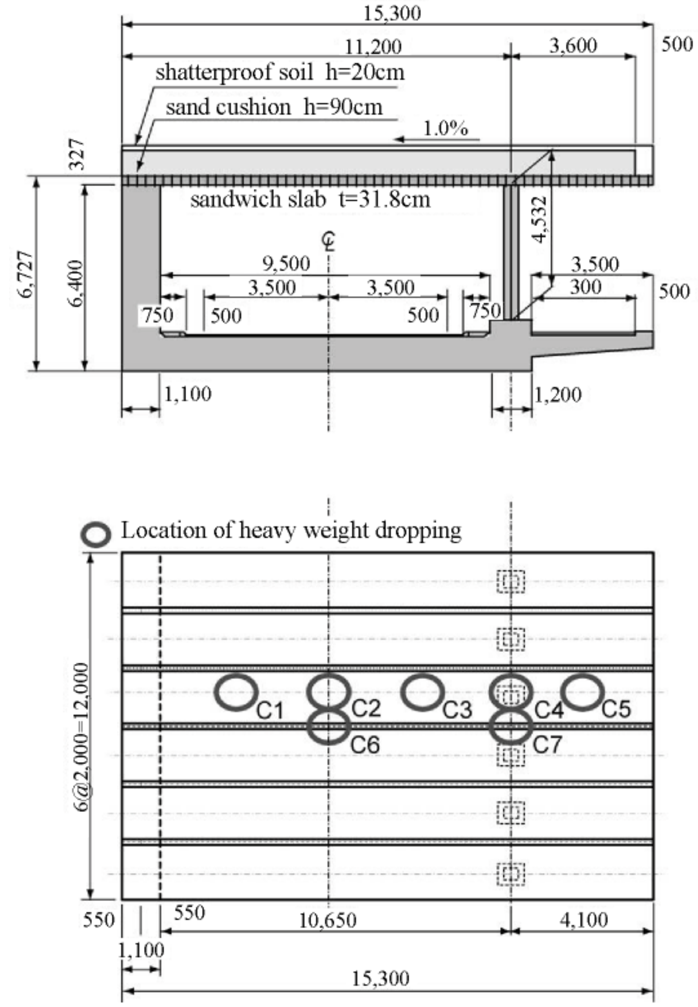


Fig. 1 Cross-sectional view of rock-shed and loading points

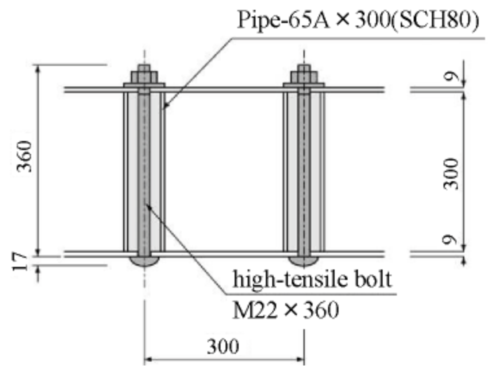


Fig. 2 Detail of sandwich panel

Table 1 List of test cases

Test case	Thickness of sand cushion (cm)	Weight of heavyweight (kg)	Location of weight falling	Falling height (m)
C1			1/4 in wall- side	6, 10
C2			Center	2, 4, 6, 8, 10, 17.5
C3			Column-side	6
C4	90	2,000	Above column	6, 8
C5			Overhang	2, 4
C6			On splice plate at center	6, 7
C7			On splice plate above column	6, 8

ASSESSMENT OF NATURAL HAZARDS BY 3D-CALCULATION OF ROCKFALL BEHAVIOUR

Bernhard Krummenacher[1], Severin Schwab[2], Fabian Dolf[3]

Keywords: 3D rockfall modelling, protection measures

INTRODUCTION

In alpine regions very often settlements, roads as well as electricity and telecommunication infrastructure are closely situated to instable rocks. The distance between potential rockfall sites and exposed persons, traffic and other infrastructure is often very small and the relief energy is high. The analysis of rockfall processes is getting more difficult within complex landscape morphology with small structures and strongly changing and inhomogeneous forest. Therefore different problems concerning the assessment of rockfall hazards have to be solved adequately. Moreover the planning of protective measures calls for more detailed information concerning intensity and probability of expected rockfall incidents, due to the actual discussion about cost efficiency.

This paper presents the rockfall simulation model GEOTEST - ROFMOD 4.1 (2D and 3D). This simulation model allows, besides the production of intensity maps, to calculate the decisive parameter to dimension and determine the exact placement of protection measures. Trajectory, fall energy and height of bouncing blocks are being calculated covering a defined area or among slope profiles.

DETERMINATION OF BASIC SIMULATION DATA, MODEL DEFINITION

As a first step a map of natural hazards phenomena has to be investigated. This is the major basic data for the hazard analysis and shows the decisive burst location (release zone) of rockfall as well as ‘silent witnesses’ (e.g. debris of different age, damaged trees) which were assessed and mapped by a specialist in the field. Information about historic incidents (e.g. from chronicles or human witnesses) is being collected, analysed and structured in an event documentation.

Moreover, a reliable geologic-geomorphologic model of the situation at the release zone as well as in the transit and deposition area (geologic structure / geomorphology) has to be elaborated before using any computer simulation model. This knowledge enables to determine the model parameters in a correct and realistic matter.

IMPACT ANALYSIS, MODELLING

Within the impact analysis the relevant scenarios according to their intensities are determined using a 3D rockfall model (ROFMOD 4.1, [1]). It calculates the trajectories of rocks and blocks based on physical laws from defined starting points within the mapped rock area and steep slopes with loose blocks. Thereby, the rolling and bouncing fall movements are strongly influenced by the spatial resolution of the digital terrain model (DTM), the used block sizes and their axis length ratio as well as the reactions of blocks with the underground and possible

[1] GEOTEST AG, Promenade 129, CH-7260 Davos Dorf, +41 81 420 15 59, bernhard.krummenacher@geotest.ch
[2] GEOTEST AG, Birkenstrasse 15, CH-3052 Zollikofen, +41 31 910 01 01, severin.schwab@geotest.ch
[3] GEOTEST AG, Promenade 129, CH-7260 Davos Dorf, +41 81 420 15 59 fabian.dolf@geotest.ch

contacts with trees. Therefore the following parameters are implemented in the rockfall model: geometry of blocks, plasticity of the underground (value of damping), roughness of the relief as well as existing forest (amount of trees per area and trunk diameter).

The simulation model of the rockfall trajectory consists of several parts. If moving through the air the trajectory is a ballistic parabola including spatial rotations but neglecting the air resistance until the next contact with the topography is detected. Then, the interaction with the ground is divided into the horizontally projected movement and the vertical impact [2]. The horizontal orientation of the movement mainly follows the slope line defined by the actual inclination of the area at this specific location calculated using the anticline defined by the four nearest DEM grid points. This procedure results in kind of a 2D mountain profile that defines the basis for the subsequent rock soil interaction.
The after-impact rotational, horizontal and vertical velocities are calculated from pre-developed impact models considering penetration into the ground, leverage effects for non-spherical boulders, friction and energy absorption. Another model-part calculates the interaction of the boulders (mass points) with trees modelling e.g. a protection forest. For these deterministic models the falling rocks are categorized by their mass and the block dimensions in the three axes. The block volume and the moments of inertia are then defined using a sphericity factor classifying the block to something between a cuboid and an ellipsoid which are the two extreme corpora for the given dimensions. Figure 1 gives an overview on the algorithm calculating the trajectory.

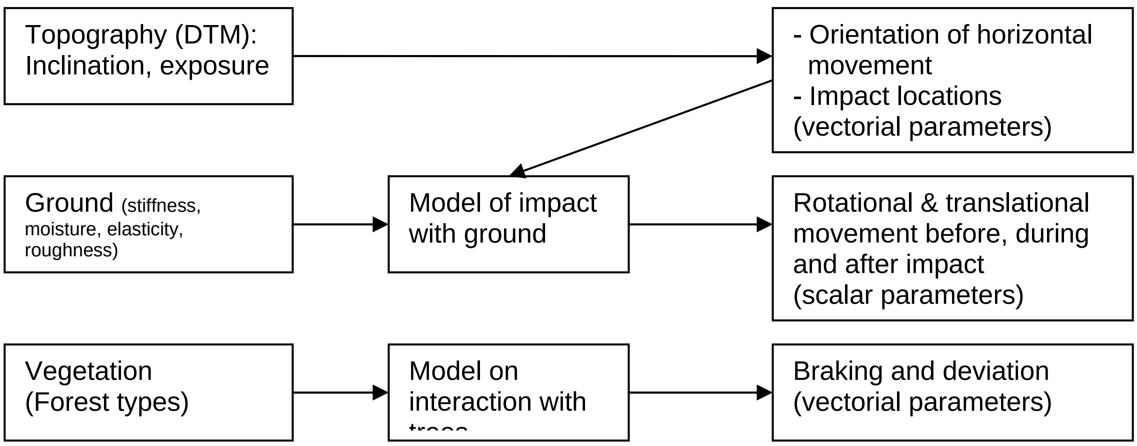


Figure 1: Calculation procedure for the calculation of the trajectories [2]

Once the after-impact velocities have reached a certain minimum level the bouncing process or if the duration between two impacts is smaller than the period of rotation the process changes to a pure rolling until a new jump is detected because of leverage effects or until the almost full stop of the boulder.

EXAMPLE STUDY

Area of investigation The investigated area covers the steep slope and rock steps above the settlement of Spino (Community of Soglio, Canton of Grisons, Switzerland) as well as the entire settlement Spino and Sott Punt. It includes different traffic infrastructure and a hospital at the base of the slope.
As a result of the existing cleavage system in the starting area mainly cubic blocks are being formed. At many places these blocks are loosened from the stable rock formation and are in ultimate readiness to fall. Traces of old rockfall incidents as well as recent disruptions cover the entire rock formation. 'Silent witnesses' of ancient incidents show a block size smaller than 8 m^3. Due to the high intensity of cleavage the disaggregation of the rock body is high.

On account of the geological disposition, it is more likely for small block sizes to fall than for big rock masses. Hence the block size varies from 0.2 to 8 m^3 of volume.

Definition of the incident scenarios To determine hazard levels for different classes of incident probability, calculation parameters for falling cubature have to be established. According to the recommendations of state and cantonal governments (Federal Office for the Environment (FOEN), Forest Office of the Canton of Grisons, Switzerland) an observation period shall span 300 years. Hence for hazard assessment potential incidents with this return period have to be taken in to account. On this basis the different potential and proven burst locations are allocated to specific block sizes.

Data acquisition For the Soglio project area the model was calculated on the basic of the DTM-AV airborne laser scanning (Digitales Terrain Modell der Amtlichen Vermessung, Swiss Federal Office for Topography, Wabern) with a spatial resolution of 2 x 2 meters (x/y-axis) and a height resolution (z-axis) within decimetres.
The used parameters (forest, ground and block size) including determination of the burst areas have been defined by field investigations and area covering mapping. Within the burst location the starting points were placed in a regular grid of 3 to 5 m width, depending on the size of the location. The values for roughness and damping are variables without dimensions. They are assigned to different terrain entities covering the entire assessed area. They were digitized and processed as basic data for the modelling.

RESULTS AND FIELD OF APPLICATION

As a result the following layers are processed within a Geographical Information System (GIS): trajectories of all modelled blocks, pixel maps of classified bouncing heights and resulting maximum energies of the falling blocks. Parameter maps can be generated for all possible burst scenarios and the chosen period or reoccurrence (30, 100 and 300 years). The energy maps can be used as a basis for the essential intensity maps.
By overlaying the information layers “energy” and “bounce height” a basic map can be generated. This allows planning of necessary protection measures at best possible locations. Additionally the optimal height of protection measures (e.g. of rockfall barriers) and possible energy resolving behaviour can be determined [3].

CONCLUSION

Since the beginning of 2008 and according to the new subsidies conditions for all planned protection measures in Switzerland a cost-effect analysis has to be undertaken. To do so, intensity maps have to be available. They demonstrate the situation before and after the realization of the protection measures. The maps generated with the 3D simulation can be overlaid with the damage potential in a GIS. The intersection of both layers automatically shows the affected objects for each scenario. With a special calculation program „EconoMe 1.0“(FOEN 2007, www.econome.admin.ch) risk and cost efficiency of planned measures can be calculated.

REFERENCES

[1] Krummenacher, B., Pfeifer, R., Tobler, D, Keusen, H.R., Liniger, M., Zinggeler, A. (2005). Modellierung von Stein- und Blockschlag, FAN-Forum 18. Februar 2005, ETH Zürich
[2] Zinggeler, André (1990). Steinschlagsimulation in Gebirgswäldern : Modellierung der relevanten Teilprozesse, Diplomarb. Geogr. Inst. Univ. Bern.
[3] Perret, S., Dolf, F., Kienholz, H. (2004). Rockfalls into forests: Analysis and simulation of rockfall trajectories — considerations with respect to mountainous forests in Switzerland, Landslides, Vol. 1(2), 2, Berlin.

EXPERIMENTAL STUDY OF THE IMPACT RESPONSE OF GEO-CELLS AS COMPONENTS OF ROCK FALL PROTECTION DYKES

Stéphane Lambertl[1], François Nicot[2], Philippe Gotteland[3]

Geocells can be used to build cellular rock fall protection dykes. The aim of this paper is to investigate experimentally the impact behaviour of geocells. The response of single cells subjected to impact is analysed focusing on the influences of the cell fill material and of the cell boundary conditions. The aim is to identify the conditions for a higher reduction of the transmitted force.

Keywords: rock fall, protection dyke, impact, geocell, gabion, geomaterials

INTRODUCTION

The use of geocells offers a promising alternative for the construction of rock fall protection dykes. Geocells are composite structures combining a manufactured envelop together with a granular fill material. In fact, changing the fill material according to the geocells position allows building sandwich structures (Fig. 1). In the case of rock fall protection dykes, a sandwich structure could aim at reducing the force transmitted through the structure during the impact by a boulder. This could be attained favouring the deformation of the front face resulting in the reduction of the impact force and, consequently, of the transmitted force. In addition, the deformation of the structure would also lead to higher energy dissipation.
Compared with more classical soil reinforced rock fall protection dykes, the main difference is that the deformation, and possibly degradation, is expected during the impact. This deformation is considered for their design and their maintenance includes the possible replacement of damaged cells, which is facilitated by the cellular nature of the structure.

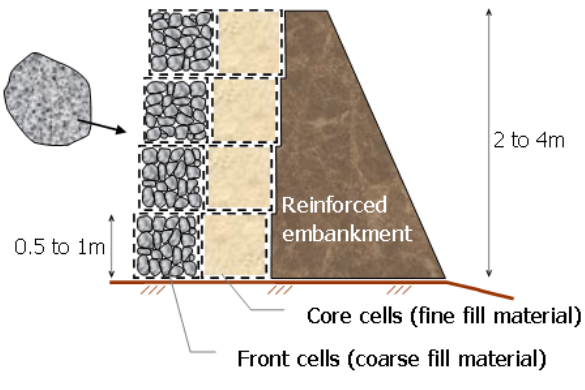


Fig. 1 Illustration of the principle of cellular rock fall protection dykes

[1] Cemagref, Unité ETNA, BP 76, 38402 Saint Martin d'hères Cedex, France, +33 476 762 794 (fax: - 513 803), stephane.lambert@grenoble.cemagref.fr
[2] Cemagref, Unité ETNA, BP 76, 38402 Saint Martin d'hères Cedex, France, +33 476 762 770 (fax: - 513 803), francois.nicot@grenoble.cemagref.fr
[3] L3S-R, BP 53, 38041 Grenoble Cedex 9, France, +33 456 528 630, philippe.gotteland@ujf-grenoble.fr

Fig. 2 Cells filled with sand (gabion cage + geotextile) and boulder (gabion cage)

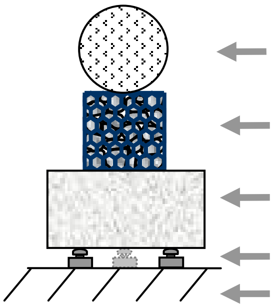


Fig. 3 Sketch of the experimental set-up

A study coupling experiments together with numerical modelling was engaged, considering the various scales, under static and impact solicitations [1]. The aim of this paper is to present and discuss the experimental results obtained at the cell scale. The response of cells subjected to impact by a boulder is analysed. The main parameters of concern are the cell fill material and the cell boundary conditions. The aim is to identify the phenomenon involved during the impact and to define which condition result in a higher transmitted force reduction.

MATERIALS AND METHOD

The geocells considered are cubic in shape, 500mm in height, and made of a hexagonal 'double twisted' wire mesh filled with coarse granular materials or fine materials (Fig. 2). The former were crushed carry limestone, hereafter referred to as 'boulder'. The latter consisted of compacted sand or a mixture of sand and shredded tyres, with a 70/30 mass ratio [2]. In the case of fine fill material, a non-woven geotextile was used in combination with the wire mesh.

The cells were submitted to impact by dropping a rigid 250kg spherical boulder 54cm in diameter. The cell was placed on a rigid pedestal made of reinforced concrete (Fig. 3).
During the impact, the main measurements were the impacting boulder acceleration and the force transmitted by the cell to the pedestal [3].
The four lateral faces of the cell were (i) free to deform – FD (ii) confined by the same material as their fill material – MC or (iii) rigidly confined – RC.

Results presented in the following concern 13.5kJ impacts obtained dropping the boulder from a 5.5m height.

MAIN RESULTS

From the impacting boulders acceleration measurement it is possible to obtain the impact force. This data is interesting for understanding the dynamic behaviour of the cell. But considering the aim of our study, the most important data is the transmitted force. Figure 4 gives

the maximum value of the transmitted force for the different situations. The cell filled with boulder and laterally free to deform transmitted the lower force. This type of cell is the most efficient whatever the cell boundary conditions. This is due to the different fill material impact behaviour. The impact leads to the fine fill material compaction whereas it leads to boulders breakage, increasing the impact duration and the cell penetration.
The tyre-sand mixture is less efficient than sand in reducing the transmitted force. In fact, the tyre-sand ratio considered was defined based on static tests which is not satisfactory for dynamical loadings.

The influence of the cell boundary conditions appears to be higher than the influence of the fill material.

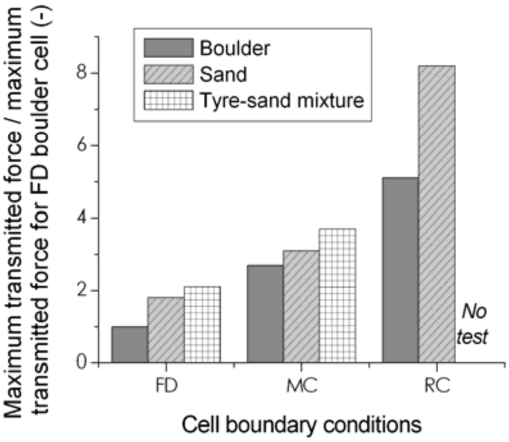


Fig. 4 Maximum transmitted force – Relative value

CONCLUSION

In order to investigate the behaviour of geocells as components of rock fall protection dykes a series of impact test by a 250kg spherical boulder was performed.
Based on the maximum force transmitted by the impacted cell, it appears that the optimum consists of a cell filled with coarse granular, whatever the cell boundary conditions. Laterally free to deform cells transmit the lower force.
Nevertheless, the transmitted force based criterion is not sufficient to evaluate the ability of a cell to reduce the stress transmitted in the impacted dyke. New developments are necessary to take into account the diffusion through the structure.

ACKNOWLEDGEMENT

The authors are grateful to the PGRN (Natural Hazard Pole of Grenoble) from the General Council of Isère for their financial support, to the research consortium VOR (Risk Vulnerability of Structures) for providing us with the experimental site.

REFERENCES

[1] NICOT, F., P. GOTTELAND, D. BERTRAND, S. LAMBERT (2007) Multi-scale approach to geocomposite cellular structures subjected to rock impacts. *International Journal for Numerical and Analytical Methods in Geomechanics*, 31, pp. 1477-1515.
[2] GOTTELAND, P., S. LAMBERT, L. BALACHOWSKI (2005) Strength characteristics of tyre chips-sand mixtures. *Studia geotechnica et mechanica*, 27(1), pp. 55-66.
[3] LAMBERT, S. (2007) *Comportement mécanique de géo-cellules – application aux constituants de merlons pare-blocs cellulaires*. Phd Thesis, University Joseph Fourier, Grenoble, *in French*.

DO WE NEED CODED LOAD ASSUMPTIONS FOR ROCKFALL PROTECTION MEASURES?

Jan Laue[1], Ravikiran Chikatamarla[2], Sarah M. Springman[1]

Rockfall protection galleries are mainly designed based on limited available information about potential hazard. Design assumptions, often given as maximum design energies [1], have been extended by identification of potential penetration depth using a mechanism derived from extremely low energy events. The latter approach in the ASTRA guidelines [2] incorporates stiffness and strength of the cushion in addition to the energy of the impact and size of the boulder, but although stiffness and strength will vary significantly for different loading scenarios. This approach represents a single mechanism and can only be valid for specific energy events. Significant errors will arise for medium to high energy events when stress-based mechanisms and dynamic effects are not considered. In addition, acceptable results will be limited to materials behaving similarly to those used to develop the guiding equations (quartz sand). To promote a wide range of innovative structural protection options against rockfalls, some broad principles are presented for category based design, without specific load assumptions or standardised formulae.

Keywords: rockfall, protective cushion materials, load reduction, centrifuge modelling

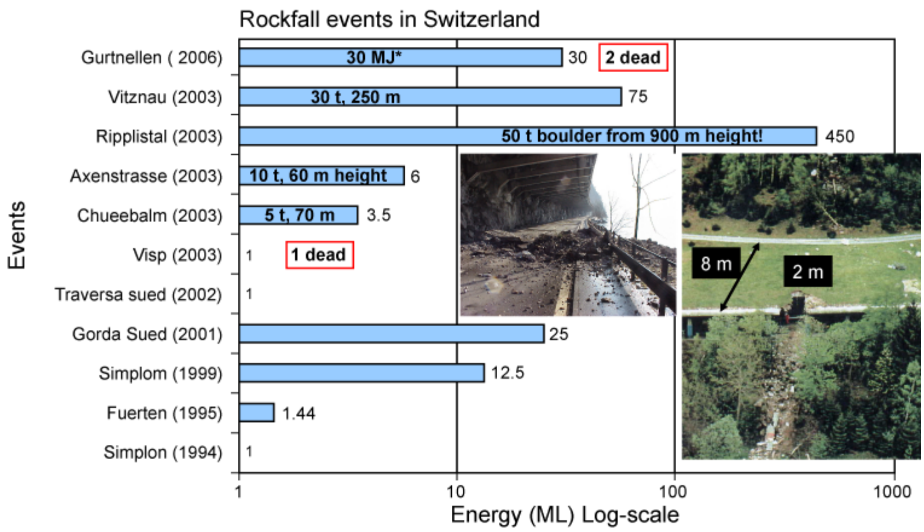


Fig. 1: Recent rockfall events in Switzerland and the damage occurring at Axenstrasse (2003) and Ripplistal (2003) after [3]. * A forensic analysis was carried out at Gurtnellen 30MJ establishing nominal 30MJ as the maximum event impacting on the road [4].

Reports of selected Swiss rockfall events (Figure 1: 1994-2006, [modified after 3]) cite potential energies prior to release, ranging from 1 to 450 MJ. While a reliable definition of the appropriate design energy event could have been made during planning phases of

[1] Institut für Geotechnik, Wolfgang-Pauli Strasse 15, ETH Zürich, 8093 Zürich +44 633 2525, jan.laue@igt.baug.ethz.ch, sarah.springman@igt.baug.ethz.ch
[2] UBS Bank AG, Zürich, ravi.chikatamarla@gmail.com, formerly at [1]

some protection structures, others cannot be predicted so effectively and designing for low probability, large events would incur disproportionately high protection costs. Design limits are sometimes formulated in the project phase and the limit design event at Axenstrasse was defined as 3 MJ, which has been significantly smaller than the majority of recent Swiss events reported [3]. An optimised design of the Axenstrasse gallery could have prevented the damage by e.g. using a cushion to reduce impact, whereas it is impossible to provide protection at the street against the scale of the Ripplistal event and only measures in the detachment area might have been able to prevent the hazard. Nonetheless, the gallery should be upgraded from existing avalanche protection levels to prevent further economic losses due to closure of this major arterial roadway through the Alps, which also happened in 2006 in Gurtnellen, in addition to any repair cost. Interestingly, the loss of life in this series was reported for a lower energy event where a boulder penetrated a car roof and for the last event at Gurtnellen, neither on road sections protected by galleries.

Figure 2 shows results from investigations into different cushion materials that were modelled in a geotechnical centrifuge [5] and tested up to 20 MJ [3]. Measurements are presented for the peak values, scaled up to the equivalent prototype, for acceleration of the falling boulder, deflection of the gallery and, in some cases, the pressure footprint of the rockfall on the roof of the gallery under the cushion. The sand cushion is adopted as a reference and performs significantly worse than most of the other feasible options. When designing against acceleration, a cushion of brittle clay lumps is most effective, although it would be less sustainable longterm, whereas a more durable sand-rubber granulate mixture (70% / 30%) reduced the deformation of the gallery most.

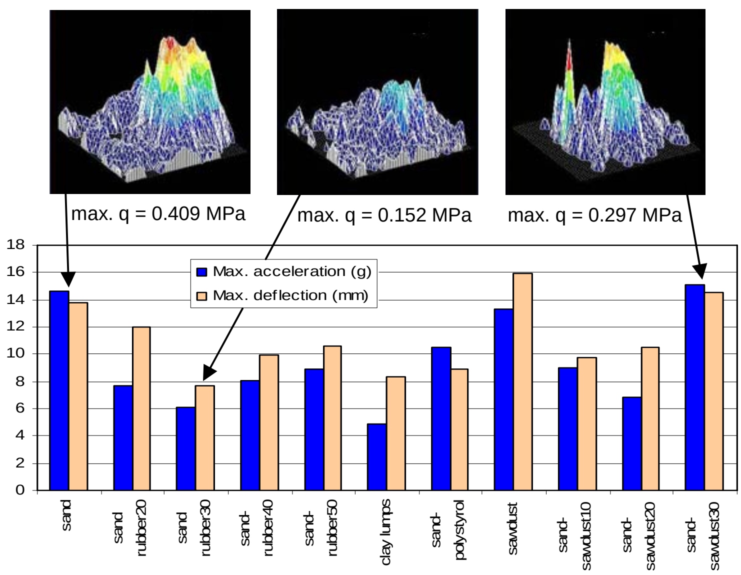
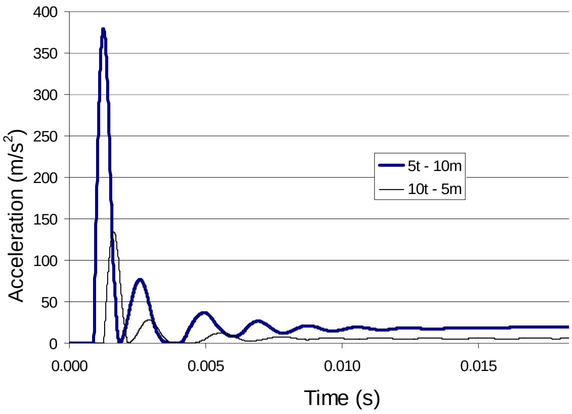


Figure 2: Acceleration, deflection and stress distribution of a 1.25 MJ rockfall event (25t x 5 m fall height) on a protection gallery with a 1.8m deep cushion simulated in the geotechnical centrifuge [after 3].

This confirms that the energy dissipation mechanism highly influences the transfer of impact to the gallery. A cushion of clay lumps fragments locally under the impact, reducing the magnitude of load, whereas a sand-rubber mixture dissipates less energy through clastic mechanisms but distributes the load over a greater roof area and will store and release more elastic energy.



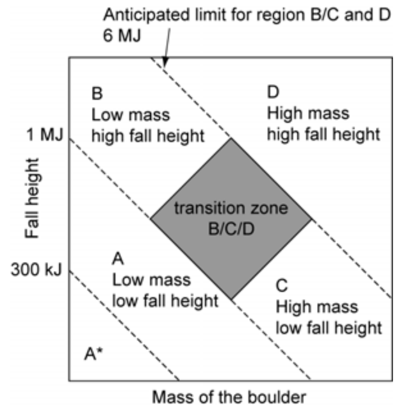


Figure 3: a) Acceleration from two 500kJ events (numerical derivation), b) proposed design guideline [3].

Fig. 3a shows the acceleration calculated for two equivalent events (same input energy, 500kJ) using a spring mass damper model [3]. A smaller boulder falling through a greater height must be decelerated a factor of three times more than for a larger boulder. Results from the physical modelling [3, 6] support this conclusion. The smaller boulder was expected to cause a bearing capacity failure or to punch through the cushion. The bigger boulder would be stopped by transferring energy via material damping and compression of the underlying soil to the gallery. A mechanical description for these options depends on the relevant soil parameters (stiffness, frictional and crushing strength) evidencing that a single formula [2] will potentially create unrealistic answers. A design chart is proposed to guide the analysis required (Figure 3b). Potential events should be classified in terms of fall height and mass of the boulder. For low masses and low fall heights (Area A^*), where enough data is available (field tests to 300 kJ by e.g. [6]), the existing code [2] could be used. Area A delimits a potential boundary to current codification. Areas B and C classify possible events that will be dominated by shear forces or crushing (B) or by the stiffness during embedment (C). A case by case approach focussing on the expected mechanism should be chosen for the transition zone. In Area D additional aspects of risk acceptance and protection at the detachment area should be incorporated into the design. Limits given in Figure 3 are open for discussion.

[1] Yoshida, H. (1999). Recent experimental studies on rockfall control in Japan. Joint Japan-Swiss scientific sem. on impact load by rock falls and design of protection structures, Kanazawa, Japan.
[2] ASTRA (2008). Richtlinie: Einwirkungen infolge Steinschlag auf Schutzgalerien. Bundesamt für Strassen (www.astra.admin.ch). 21. S.
[3] Chikatamarla, R. (2007). Optimisation of cushion materials for rockfall protection galleries. Veröffentlichung des Instituts für Geotechnik, Band 227, vdf Verlag, Zürich.
[4] Thali, U. (2008) personal communication, Göschenen.
[5] Springman, S.M., Laue, J., Boyle, R., White, J. & Zweidler, A. (2001). The ETH Zurich geotechnical drum centrifuge, *International Journal of Physical Modelling in Geotechnics*, 1 (1), pp. 59-70.
[6] Schellenberg, K., Volkwein, A., Roth, A., & Vogel, T. (2007). Large-scale impact tests on rock fall galleries. In: Huang, F.L., Li, Q.M., Lok, T.S. (eds) Proc. 7th Int. Conf. Shock and Impact Loads on Structures. 17-19 October 2007. Beijing, China. 497-504.

A NEW SANDWICH DESIGN STRUCTURE FOR PROTECTION AGAINST ROCKFALLS

Julien Lorentz[1], Pascal Perrotin[2], Frederic Victor Donze[3]

A rockfall protection structure based on a multi-layer configuration is studied. A reinforced concrete slab spreads the impact load on a second layer composed of extensible geotextile socks filled with gravel. These socks are themselves contained in columns made of car tires. This dissipative device is coupled with a reinforced concrete wall which maintains the whole structure. An experimental campaign of impact tests at different impact energies (up to 90 kJ) was carried out to measure the transmitted impact force through the dissipative part of the structure. At higher energies, the punching-shear plug process appearing on the impacted concrete slab decreases the efficiency of the structure. To overcome this limitation, it was seen that superposing two sandwich modules (referred to as "double-sandwich") dramatically increases the dissipative efficiency, thus making this structure suitable for common impact energies.

Keywords: protective structure, rockfall, impact tests.

INTRODUCTION

Passive rockfall protection structures use various technologies, such as drape nets, rockfall catchment's fences, gabions, diversion dams, embankments, etc [1]. Most of these conventional structures take up a lot of space which means they cannot be built in some places. There is a need for designing new thinner (about a meter-thick) and long lasting dissipative protection structures for moderate impact energy, and this is the aim of this study. While keeping construction costs as low as possible, a thin protective structure has been tested during an experimental test campaign.

A NEW PROTECTION STRUCTURE: THE "SANDWICH" DESIGN

After preliminary studies [2], the proposed protection structure is a multi-layer structure where a reinforced concrete slab (impacted face) is associated with a layer of gravel contained in extensible geotextile socks which are input in columns of tires. These two layers represent the dissipative system in which damage is tolerated for the structure. This system is then placed against a support made of a concrete wall (Figure 1). By dissipating the energy of impact, this system is capable of strongly decreasing the transmission of the impact loading in order to avoid oversizing the support wall. In addition, the dissipative layer can be easily removed and changed in case of intense damage whereas the support wall is permanently installed.

---

[1] Bureau d'étude IMS RN, Parc Pré Millet, rue Aristide Bergès, 38330 MONTBONNOT, julien.lorentz@imsrn.com
[2] Laboratoire du LOCIE, Université de Savoie, 73376 Le Bourget du Lac, perrotin@univ-savoie.fr
[3] *Laboratoire Sols, Solides, Structures et Risques* UMR5521, Université Joseph Fourier, Grenoble Université, BP 53, 38041 Grenoble cedex 9, Frederic.Donze@hmg.inpg.fr

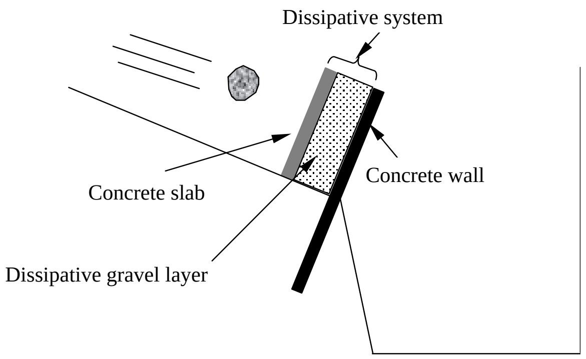
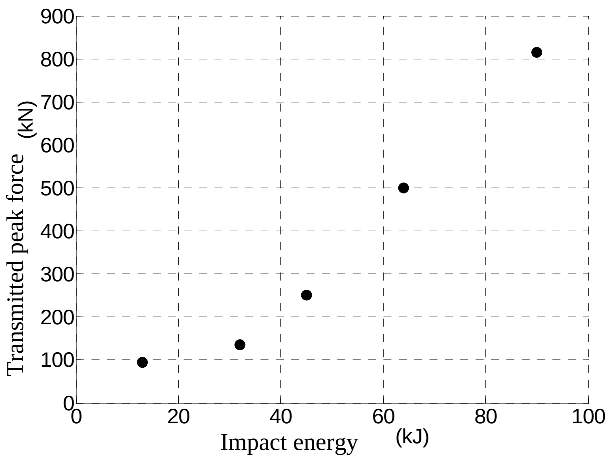


Figure 1. a) Schematic diagram of the sandwich structure b) Vertical view of the Sandwich structure

EVOLUTION OF THE TRANSMITTED FORCE VS THE IMPACT ENERGY

Load cells were placed on the support wall to record the transmitted impact force through the dissipative system during an impact event [3]. The impact tests were carried out vertically, with an impacting concrete sphere made of Ductal, which was dropped on the structure. The impact energy of the concrete sphere varies from 13 kj to 90 kJ, corresponding to drop heights ranging from 2 and 14 meters. The mass of the sphere was 650 kg.
Below impact energies of 40 kJ, the peak of the transmitted effort increased slightly, with, for example, a peak at 135 kN for an impact energy of 32 kJ. Then, from 40 and 90 kJ, a linear increase of this peak with a higher rate was observed (Figure 2). In the case of an impact e-nergy of 90 kJ, the peak of the transmitted force reached 815 kN. This sudden increase of the transmitted peak corresponds to the step where deformation not only occurs in the tires but starts appearing in the gravel-filled geotextile socks.



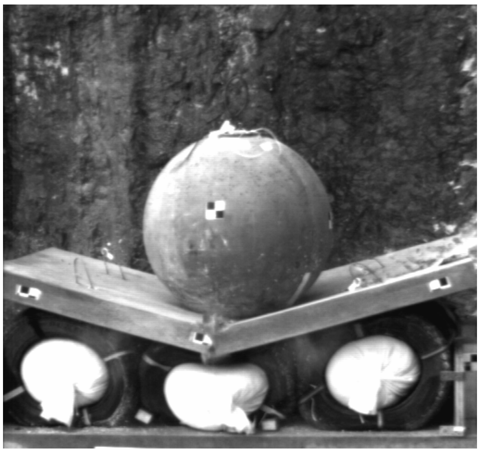


Figure 2. On the left, the evolution of the peak of the transmitted force depending on the impact energy is plotted and on the rigt the impacted structure can be seen.

ROLE OF THE IMPACTED SLAB

To show the role of the impacted reinforced concrete slab in the spreading force process, a direct impact test on the three columns of tires containing the gravel material was carried out.
Here, the maximum transmitted force reached 424 kN even for a low impact energy of 20 kJ, which confirmed the major role played by the concrete slab.
Moreover, putting the gravel material in geotextile socks instead of directly filling the tires, gives better results. The role of the geotextile in the dissipative capability of the structure was

verified by directly impacting a sandwich structure where no geotextile socks were used. The results showed that the maximum peak of the transmitted force was a least four times greater when the tires were completely filled. This is because the gravel in this case is totally confined, thus decreasing the dissipative efficiency. The voids created in the tire flanks, by the socks of geotextile, allow the gravel material to largely deform and thus dissipate more energy by friction.

EXTENSION FOR MORE IMPORTANT ENERGIES: THE DOUBLE SANDWICH

In order to increase the dissipation capabilities of the sandwich structure for energies higher than 100 kJ, the structure has simply been doubled.

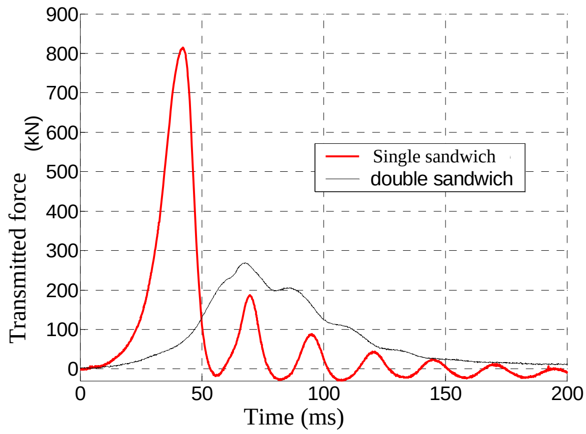


Fig. 3 Front view of the double sandwich, Comparison of the efforts transmitted by the single and double sandwiches at 90 kJ

Impact tests up to 90 kJ showed that the peak of the transmitted force by a double sandwich is approximately three times lower than for a single sandwich.

CONCLUSION

Experimental tests were carried out to evaluate the dissipative capabilities of the sandwich design structure during rockfall impacts. The single sandwich structure gave important insights about the response of the structure. The role of the different components were clearly identified: a concrete slab to spread the impact force on a dissipative layer made of columns of tires containing geotextile socks filled with gravel. It was seen that superposing two sandwich modules (referred to as a “double-sandwich”) dramatically increases the dissipative efficiency, thus making this structure suitable for common impact energies.

REFERENCES

[1] Laboratoire Central des Ponts et Chaussées (LCPC), Parades contre les instabilités rocheuses: chute de pierres, chute de blocs, éboulements, Collection environnement, Les risques naturels, Laboratoire Central des Ponts et Chaussées, 2001

[2] Lorentz J., Donzé F.V., Perrotin P. and Plotto P., Experimental study of the dissipative efficiency of a multi-layered protective structure against rockfall impact, Revue Européenne de Génie Civil, **10**-3, 295--308, 2006

[3] Delhomme F., Mommessin M., Mougin J.P. and Perrotin P., Behavior of a structurally dissipating rock-shed: experimental analysis and study of punching effects, International Journal of Solids and Structures , **42**-14, 4204--4219, 2005

INTEGRATED HAZARD MAP OF AXEN ROAD

K. Louis[1], B. Romunde[2], P. Bänninger[1]

The national road N 4 (Axenroad) and the Swiss Federal Railways-line (SBB Axen-Gotthard-line) are passing along the eastern shore of the Urnersee (eastern part of Lake of Lucerne, Switzerland) often below steep rocky cliffs. Between Brunnen and Flüelen these two major traffic routes are highly exposed to natural hazards, especially to rockfall processes. Over 70 occurred events have been recorded, extending from frequent rockfalls up to rockslides of 6'000 m^3 (1932). With the constructions of rockfall galleries in 1968, direct hits on the road by low energy events could be minimized. However, the galleries do not provide protection for high energy events and were damaged or partially destroyed several time by rockfall events (fig. 1). Statistically every second year a potentially harmful rockfall event has to be expected.

Fig. 1: Gallery destroyed by rockfall event (500-600 m3) in 1970.

Both transportation routes are part of the international north-south transit line via the Gotthard, connecting Switzerland and Italy. The transit frequency is very high with a daily average of 11’800 car/truck/bus movements and 200 trains per day, respectively. Consequently the safety requirements as well as the availability of these traffic lines are of high importance.

To investigate the 8.3 km long section from Mositunnel-Nord (Brunnen) to Gumpisch (Usser Tellen) for its exposure to gravitational natural hazards, the “Civil Engineering Office of the canton Schwyz” in cooperation with the “Swiss Federal Railways” and the “Civil Engineering Office of the canton Uri” conducted a two-phase project: The first phase should focus on the present gravitational hazard potential and is subject of this publication. In a second phase, the present collective and individual risks, including evaluation of measures to be taken, were considered. The consortium “Louis Ingenieurgeologie GmbH Weggis, Ingenieure Bart AG,

1) Louis Ingenieurgeologie GmbH, Postfach 143, CH-6353 Weggis, Tel. +41 41 391 01 91,
Fax +41 41 391 01 92, eMail: info@louis-weggis.ch
2) Dr. Spang Civil Engineering and Geotechnical Consultants LTD, Westfalenstr. 5-9, D-58455 Witten,
+49 2302-914020 (fax -9140220), dr.spang@t-online.de

St. Gallen, and Büro für Geologie & Umweltfragen D. Imper, Heiligenkreuz" was assigned with those tasks.

The hazard analysis included detailed field mapping of the rockfall source and deposit areas, as well as the evaluation of all existing mitigation measures and protection structures. Numerical rockfall modelling was conducted to investigate the expected impact energies on the traffic lines.

Due to differences in the source rock areas (i.e. lithology, bedding, joints, spacing of joints, decomposition, failture mechanisms, registered events or observed tracks), the slopes and rock faces were divided into a total of 58 different release zones, for which scenarios (expected block volumes) for the following 5 intervals had been defined:

- Highly frequent occurrence: interval of recurrence 1 - 3 years
- frequent occurrence: interval of recurrence 3 – 10 years
- less frequent occurrence: interval of recurrence 10 – 30 years
- scarce occurrence: interval of recurrence 30 – 100 years
- very scarce occurrence: interval of recurrence 100 – 300 years.

Numerical 2D rockfall simulations were conducted in 44 slope profiles. The simulations were performed for the five scenarios, according to the different block volumes expected. Most of these slope profiles included several release zones positioned on different altitudes, so that more than one simulation run per slope profile and scenario was demanded.

This procedure resulted in a total amount of about 500 single rockfall simulations. To deal with the high computational effort, a batch procedure for the simulation run as well as pre- and post-processing tools were applied. For this, the used software Rockfall 6.1 was slightly modified.

The Slope profiles are based on a digital surface model and the surface elevations were extracted along predefined paths. As the resolution of the surface model was very high, it turned out to be necessary to smoothen the slope profile and overlay it afterwards with an appropriate roughness. This process was incorporated in the pre-processing procedure.

Regarding all these parameters, the amount of calculation results was overwhelming. Nevertheless the results of the rockfall trajectory calculation had to be checked and verified for plausibility for every single simulation. But for the use and the evaluation of the results for the further steps of impact and risk analysis the usual form of result presentation in single sheets for every single simulation run is not feasible.

To integrate the results into two-dimensional maps the following post processing procedure was conducted: In all slope profiles the calculation probed every 10 meters for the full statistics of kinetic energy and jump height. From this the 90% value was evaluated and exported into the two-dimensional presentation. Thus the simulation results of the effect of rockfall events for each release zone are charted in a map of intensity, graded in 9 level of impact energy.

On the basis of the intensity maps, the risk analysis (phase two) was conducted, including evaluation of measures to be taken to reduce existing risks to an acceptable level considering the principles of cost-efficiency.

It is shown that large amounts of simulations in single slope profiles can be processed and finally evaluated into areal presentations. Pre-condition is the a-priori identification of rockfall paths which lead to the slope profiles in which the simulations are done. In general they are following the steepest gradient downhill. The huge amount of calculations which are necessary to obtain detailed and in a statistical aspect valid results, requires a rather sophisticated system of pre-, post-processing and batch procedures. Otherwise the task at hand is not only consuming time and lots of manpower but also highly error prone.

ACKNOWLEDGEMENTS

Markus Isaak, Project manager of Tiefbauamt Kanton Schwyz (“Civil Engineering Office of the canton Schwyz”), which represents the cantons Schwyz and Uri as well as SBB, for their permission to publish the results of the project and their good cooperation in this project.
Dr. Spang GmbH for their consent to provide a modified version of the ROCKFALL 6.1 Software which could be adapted for the special purpose of this project.

REGIONAL INDICATIVE ROCKFALL MAP USING LIDAR-BASED SLOPE FREQUENCY HISTOGRAM AND CONEFALL MODELLING

Alexandre Loye[1], Andrea Pedrazzini[1], Michel Jaboyedoff[1]

A factor limiting potential rockfall mapping at regional scale is often the lack of knowledge of potential source areas. Nowadays, LiDAR Digital Elevation Model can account for realistic relief details so that quantitative geomorphometric analyses become a relevant approach for detecting potential rockfall instabilities. Using DEM-based slope angle statistics over areas of similar lithologies and rocky outcrops and screes zones available from the 1:25’000 vectorized topographic maps, an indicative rockfall hazard map was obtained over the canton of Vaud (3200 km^2), Switzerland, in order to provide a relevant overview of the potential rockfall prone areas. The slope angle frequency histogram enables displaying major morphologies such as alluvial plains, U-shaped valleys, rock cliffs, etc. by fitting several Gaussian distribution function and its major pick can be considered as a slope stability parameter. Potential rockfall source areas were therefore defined by the dominant population with the highest average (cliff population) and the internal frictional angle of the rocky outcrops and scree zones. 3D modelling (CONEFALL) was then applied on the estimated source zones in order to assess the maximum runout length. Comparison with known events and other rockfall hazard assessments are in good agreement, showing that it is possible to develop susceptibility rockfall hazard map over large areas from DEM-based parameters and slope analysis.

Keywords: Rockfall, Indicative hazard map, geomorphometry, shadow angle, LiDAR-DEM.

INTRODUCTION

Susceptibility hazard mapping throughout large mountainous zones provides a fast and cost-effective overview of rockfall prone areas. In Switzerland, the Indicative hazard map consists of defining potentially rockfall sources zones and their runout zones for rockfall of small size [1]. Larger events, such as rock avalanches, are not taken into account. The outcome can be used as groundwork for territory management. The lack of knowledge of the potential rockfall source areas is however one of the great limiting factors. Sources zones are usually taken from distinctive evidence (e.g. talus slope deposits below cliff faces, field measurements, and historical register information). But rockfalls also occur on slope surfaces covered with vegetation where evidence is less distinct. The following paper introduces an approach that takes into account those hidden source zones in assessing potential rockfall prone areas.

IDENTIFICATION OF POTENTIAL ROCKFALL SOURCE AREAS

Rocky outcrops and consequently unstable rockfall source areas are found by definition in steep slopes [2]. Moreover, the angle of internal friction depends on the rock type and some morphological aspects. For that reason, failure susceptibility can be characterized through the internal frictional angle, which is function of the slope gradient [3]. From that perspective, a

[1] IGAR Institute of Geomatics and Risk Analysis, University of Lausanne, Amphipôle, CH-1015 Lausanne, +41 21 692 35-43 (fax: -35), alexandre.loye@unil.ch

slope frequency distribution analysis was carried out by computing the slope histogram over each main geological unit. This approach consists of breaking up the slope frequency histogram into several Gaussian distribution function so that their sum fits the slope histogram, following that the slope gradient frequency of a morphological unit varies randomly around its mean slope gradient [4]. Those slope populations can be therefore defined by each fitted normal curve on the histogram (fig.1). In an alpine topography come mostly across [5]:

1. Low slope gradients corresponding to the plains formed by fluvio-glacial deposits.
2. Mid/gentle slope gradients corresponding to the lower part of the hillslope (Foothills & mountain flanks). They are characterized by alluvial fans (debris flow), landslide mass and till deposits.
3. Steep slopes can be matched to the occurrence of rocky outcrops and cliff faces.

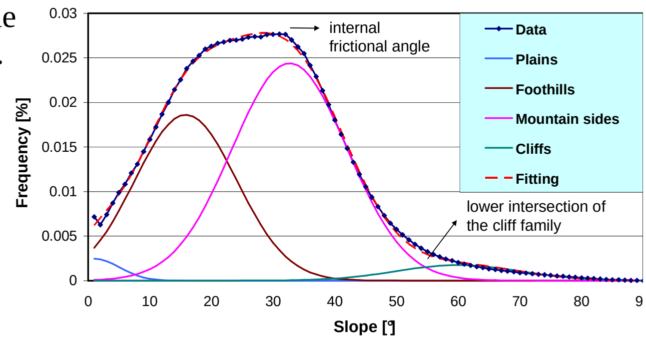


Fig.1 Slope frequency histogram featuring Gaussian populations (colored lines), where the pick of each curve represent the mean slope equilibrium of a distinctive morphological unit.

The peaks of each fitted curve reflect more precisely a distinctive threshold that can be correlated to major morphological units. Usually, those major topographical features are strongly influenced by the dominant lithologies. In a geomorphologically homogeneous landscape, the steepest part can be correlated to harder lithologies.

ASSESSMENT OF THE MAXIMUM ROCKFALL RUNOUT ZONES

Assessment of the runout zones from potential source areas described above was performed by means of the CONEFALL software [6,7], which simply implements the cone method inspired from the shallow angle method [8] in a GIS environment. Thus, this GIS-based software allows the estimation of the maximum runout length in 3D by assuming a given aperture angle (90°- ϕ_p) centred on the source point (fig.2). The cone method is empirical and doesn't require ambiguous input parameters, such as coefficient of friction and bounding velocity.

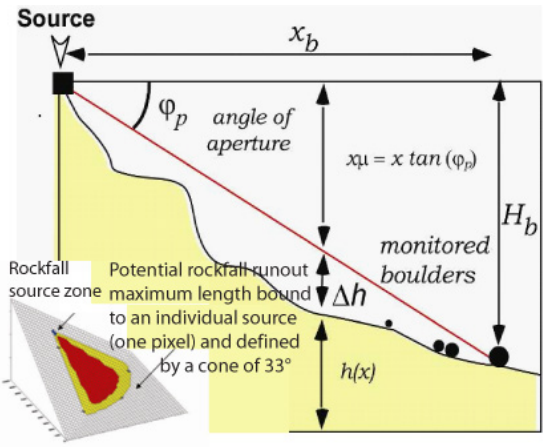


Fig. 2: Scheme of the Cone method implemented in the GIS-based CONEFALL.

DATA PROCESSING

The slope frequency histogram was performed on DEM of 1m x 1m cell size (© 2005 SIT, Vaud). The state of Vaud was divided into five study areas: The Helvetic, Ultrahelvetic and median Prealps gave three distinct slope frequency distributions of the alpine topography. Plateau and Jura gave two others. For each of those five geologically-based units, a slope

Tab. 1 Threshold angles above which the rockfall source areas are potentially considered

Tectonic unit	Threshold angle: Topographic vector map	Threshold angle: Intersection Cliff family
Helvetic	36°	54°
Ultrahelvetic	33°	49°
Median Prealps	34°	53°
Molasse Basin (Plateau)	30°	46°
Jura Mountains	32°	46°

frequency distribution was computed and plotted according to each slope angle. The pick of the slope histogram was taken as the internal frictional angle. Rocky outcrops available from the 1:25’000 topographic vectorized map, steeper than this angle were considered as potential rockfall source areas. Gaussian curves were fitted using the excel-based solver in order to compute the most-likely normal curves in an iterative way. Initial values were defined according to the shape of the slope histogram, where slope populations were obvious (unsteadiness in the distribution). Critical angles for each major unit are resumed in Tab.1. CONEFALL was applied to each potential source zones with an aperture angle of 33° for all units. This was established by comparison with rockfall events observed on orthophotos and fieldwork undertaken on test zones. The cone method performed over mountain sides that overhang alluvial plains models a maximum runout length too large compared to the reality. A correction for the valley-bottom was therefore done to limit the runout length to 60 m.

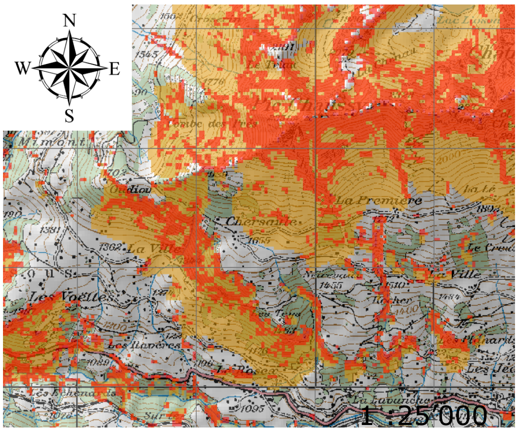


Fig. 3 Close-up of the Indicative rockfall hazard map. Rockfall source zones are drawn in red and the runout perimeter in brown.

CONCLUSION

The present method describes a global approach to develop an indicative rockfall hazard map at regional scale (fig.3), which provides a fast and cost-effective way to overview rockfall prone zones over large areas. Rockfall source zones are in good agreement to what can be observed on test zones. Moreover, they show some good conformity with the geomorphology by the fact that this method detects rockfall source zones located on steep slopes covered with vegetation as well. This is a rather conservative way to consider cliff faces and slopes surfaces to be potentially instable over a certain slope gradient. Nevertheless, this is reasonable over a long period of time. Likewise, CONEFALL allows to quickly but accurately delineating potentially rockfall prone perimeters and can hold the comparison with a 3D trajectography model, although it doesn’t require physically-based parameters. Finally, this work shows that it is possible to produce indicative rockfall hazard maps over large areas from LiDAR-DEM-based parameters and slope analysis.

REFERENCES

[1] Lateltin, O. (1997). Recommandations - Prise en compte des dangers dus aux mouvements de terrain dans le cadre des activités de l’aménagement du territoire. Office fédéral de l’Environnement, des forêts et du paysage.
[2] Dorren, L. K. A. 2003. A review of rockfall mechanics and modelling approaches. Progress in Physical Geography 27, 1: pp. 69-87.
[3] Wyllie, D. C. and Mah, C. W. 2007. Rock slope engineering. Civil and Mining, 4th ed.. Spon press, London.
[4] Strahler, A. N. 1954. Quantitative geomorphology of erosional landscapes. Compt. Rend. 19th Intern. Geol. Cong., Sec.13: pp. 341-354.
[5] Oppikofer, T., Jaboyedoff, M. et Coe, J., 2007. Rockfall hazard at Little Mill Campgournd, Uninta National Forest: Part 2. DEM analysis. 1st North American Landslide Conference, Vail, Colorado, USA. Association of Environmental & Engineering, pp 1351 - 1361.
[6] Jaboyedoff, M. and Labiouse, V. 2003. Preliminary assessment of rockfall hazard based on GIS data. ISRM 2003 - Technology roadmap for rock mechanics, South African Institute of Mining and Metallurgy.
[7] Jaboyedoff, M. 2003a. CONEFALL 1.0 : A program to estimate propagation zones of rockfall based on cone method. Quanterra, www. quanterra.ch.
[8] Toppe, R. 1987. Terrain models – a tool for natural hazard mapping. In Salm, B. and Gubler, H., editors, Avalanche formation, movement and effects. IAHS Publication no. 162, 629–38.

HISTORY AND HIGHLIGHTS OF ROCK FALL RESEARCH IN JAPAN

Hiroshi Masuya[1]

In Japan, measures against rock fall belong to the most important subjects preventing natural disasters, because there are a lot of mountainous roads. This paper gives an introduction into the history of research, technology and some related activities in Japan. The change of concepts concerning rock fall protection on progress of time and also the state of art are concretely explained. Finally, an outlook is given toward a rational design procedure for protective structures against rock fall.

Keywords: impact load, rock fall, sand cushion, performance based design

INTRODUCTION

There are many kinds of structures exposed to potential impact load by the collision of a moving body like protection structures for rock falls or avalanche of rocks and earth in Japan [1],[2],[3]. Figure 1 shows a protective structure damaged by a rock fall event that was released by the Noto peninsula earthquake M6.9 on March 26, 2007. Fortunately, there was no human damage. However, the damage for residents of this town was large, since the important national road (N249) was out of service.

Rock fall belongs to the slope disasters as landslide and debris flow. There are a lot unknown items regarding the appearance of rock fall as well as the motion on the slopes. The impact forces acting on protective structures by rock fall have always been an interesting and difficult issue for engineers. The dynamic behavior of structures under impact is complex because of different constitutive laws of the material. There are questions arisen how to design protective structures considering the risk to the public and how to maintain those public structures under a pre-defined safety level etc. In this paper, the history concerning subjects of rock fall in Japan are introduced and recent activities by engineers are shown.

Figure 1 Protection structure damaged by rock falls caused by Noto peninsula earthquake

MAIN CLASSIFICATION CONCERNING SUBJECT OF ROCK FALL

There are many subjects similar to rock fall, i.e. avalanches or land slides. However, it would go beyond the scope of considering all topics within this contribution. At least, rock falls generally can be classified to following four subjects [1],[4]:

[1]Kanazawa University, Kakuma-machi, Kanazawa 920-1192, Japan, phone & fax: +81 762 4603, masuya@kenroku.kanazawa-u.ac.jp

1) Occurance of rock fall
2) Motion of rocks along the slope
3) Impact load by rock fall
4) The design method of rock protection structures

It became clear at a some extent, that the appearance of rock falls can be classifyed by means of the relationship with the gradient of the slope and the relation of the geology. However, the most difficult problem has ever been to predict the occurance of rock falls exactly or approximatively based on probability theory. The actual risk at a certain slope, namely the estimation of the trajectory and the velocity of falling rocks is usually evaluated by special simulation methods not specified more closely in this contribution. However, the evaluation method of the impact force due to rock falls on gallery structures and design methods are introduced below.

OUTLINE OF EXPERIMENTS

Table 1 shows the outline of impact experiments for rock fall and the recorded values. The tests can generally be classified into four types [5], [6], [7]:

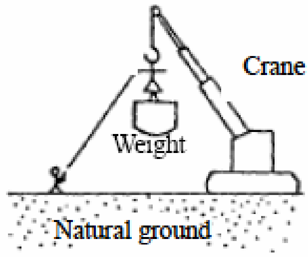
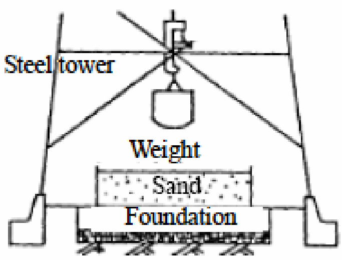
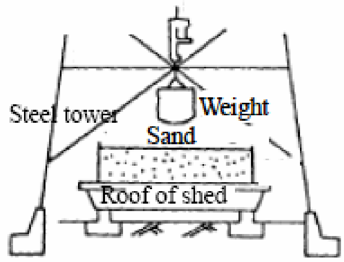
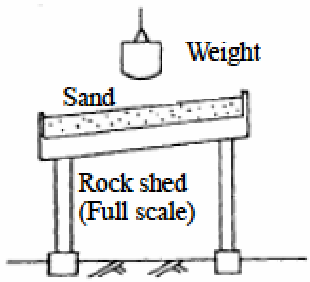
A is the simplest experiment that is done on the natural ground like a sand beach. The acceleration of the falling weight is obtained during this experiment.

B aims to reveal the propagation of impact stress within the sand layer and its ability as a buffer.

C focuses on the behavior of the roof of a rock shed under rock fall impact.

D is the full-scale experiment including a gallery and a rock shed. This experiment has the focus on the whole behavior of rock shed under rock impact.

Table 1 Outline of impact experiments to rock fall

View of experiment	Outline of experiment
A. Simple experiment at natural ground 	Experiment is done at natural ground like sand beach and space depositing sand or sand. Impact force is assumed as the product of the maximum acceleration and the mass of a weight.
B. Experiment on sand cushion over foundation 	Experiment is done on sand cushion over foundation. Acceleration of a weight and earth pressures at several locations on foundation are measured. Propagation of impact stress in sand layer and the buffer ability of sand cushion are made clear.
C. Experiment on sand cushion over the roof of shed 	Experiment is done on sand cushion over the roof of rock shed. Acceleration of a weight, earth pressures on the roof and displacements, strains of roof member of rock shed are measured. It is possible to know the dynamic behavior of the roof by impact of rock fall.
D. Full scale experiment of rock shed 	Experiment is done using full scale of rock shed. Acceleration of a weight, earth pressures on roof and displacements, strains of roof member of rock shed are measured. It is possible to know the dynamic behavior of rock shed in full scale.

EVALUATION OF IMPACT LOAD

In general and with exception of rock sheds, protection structures against rock fall are designed according to their energy absorption capacity. In contrast, rock sheds are designed by classical admissible stress design methods since they have been considered to be one of the most important structures to protect roads and railways. Therefore, permanent cracks for RC and PC rock sheds, plastic deformation for steel rock sheds aren't permitted for the largest expected falling rock event. Therefore, the evaluation of the impact load becomes important once the falling rock for the design has been determined.
The research on the evaluation of the impact load due to rock falls started about 1965 initiated by a tour bus accident in Japan. Since then, some design formulae and some ideas have been proposed for the design of the structure under impact loads.
The most general design formula for the impact load due to rock fall in Japan is drawn from the elastic contact theory. Equation (1) is recommended for sand cushion in [1].

$$P = 2.108(mg)^{2/3}\lambda^{2/5}H^{3/5} \quad (1)$$

Here, m is the mass of a falling rock (ton), H is the height of a rock fall (m), λ is the Lame coefficient of cushion material (kN/m^2) and g is the gravity acceleration (m/s^2). According to statistic data of soils and sands, λ =1000 kN/m^2 for soft soil, λ = 3000 - 5000 kN/m^2 for normal soil and λ = 10000 kN/m^2 for hard soil. Furthermore, a modification factor $\alpha = (T/D)^{-1/2}$ to consider the thickness of cushion is recommended. Here, T is the thickness of cushion (m) and D the diameter of the impacting rock (m). When a rock falls freely from a steep slope the falling height H is used like shown in Figure 2. However, the equivalent height H' expressed by eq. (2) is used, when a rock falls down from the slope with a mean gradient of slope θ.

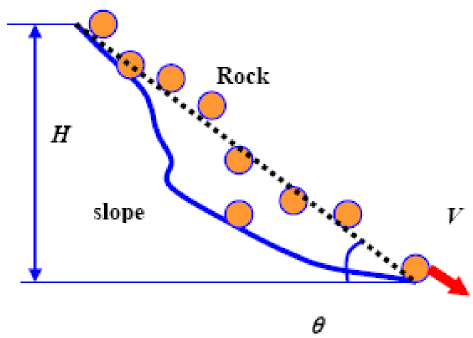


Figure 2 Slope and height of rock fall

$$H' = \left(1 - \frac{\mu}{\tan\theta}\right)H \quad (2)$$

μ is the equivalent friction coefficient of a slope with recommended values shown in Table 2. These values have generally been used for the design of protection structure, when no additional technical information is available to decide on the impact velocity.

Table 2 equivalent friction coefficient

Classification	**Characteristics of slope and rock**	**Recommended equivalent friction coefficient μ (*experimental value*)**
A	Hard rock, Small roughness, No flora Round rock	0.05
B	Soft rock, Medium roughness, No flora Round or Squarish rock	0.15
C	Earth and sand slope, Small or medium roughness, No flora Round or Squarish rock	0.25
D	Talus slope, Medium or large roughness, With flora or no Squarish rock	0.35

Series of full-scale experiments have been performed by the author and other researchers out on three kinds of sand cushions placed over a concrete foundation (see **B** in Table 1). The characteristics of the rock acceleration and the distribution of the earth pressure underneath the sand cushions were investigated. It was recommended to use the force under the cushion as design load of the rock shed rather than the acceleration of the rock. Furthermore, following formula was recommended in the utilization of the database of the rock falling tests [8]:

$$P = \beta_0 m \frac{\sqrt{2gH}}{T_0} \quad \text{(ton)} \tag{3}$$

in which W is the weight of the rock (ton), H is the falling height, T_0 is the duration of impact force (s), β_0 is a coefficient considering the effect of the depth of the sand. These parameters are expressed as follows:

$$T_0 = (0.0481 + 0.00064H)W^{0.270}\left(U_c/U_{c0}\right) \tag{4}$$

$$\beta_0 = -4.81\left(h/h_0\right) + 5.84 \qquad \text{for} \qquad h/h_0 < 1$$

$$\beta_0 = 1.03 \qquad \text{for} \qquad h/h_0 \geq 1 \tag{5}$$

$$U_{c0} = 1.53$$

$$h_0 = 90\text{cm}$$

Here, h is the depth of the sand cushion (cm), U_c the uniformity coefficient, h_0 the standard depth , U_{co} and the standard uniformity coefficient.

$$P = \beta_0 m \frac{\sqrt{2gH}}{T_0} \quad \text{(ton)} \tag{6}$$

with $\beta = 1.90\beta_0$.

It is confirmed that eq. (6) gives the upper limit of the impact force. Figure 3 shows the correlation between impact load P' estimated by eqs. (3) and (6) and the experimental value P for three types of sand cushion. Komatsuzawa's formula, Japan steel club's formula and some other formulae had been proposed for sand cushion [1].

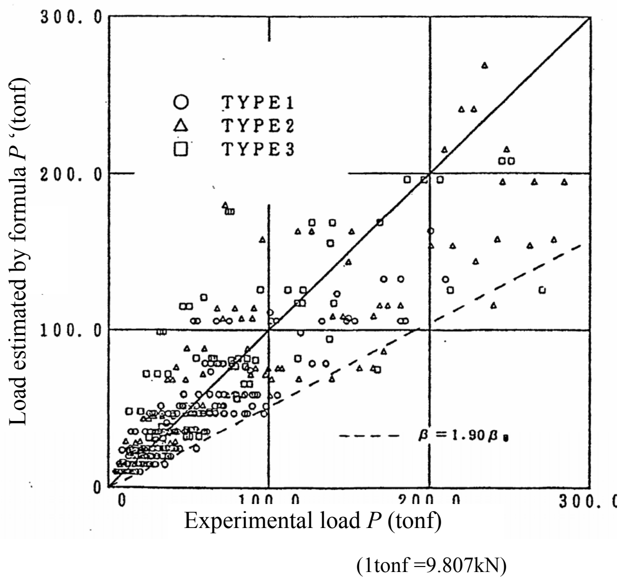


Figure 3. Correlation between experimental values and estimation by formulae

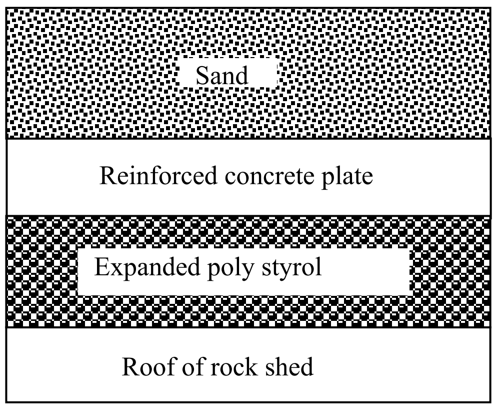


Figure 4. Three layered cushion system

Another cushion system like used tires, extended poly styrol or a composite system shown in Figure 4 also had been proposed [9]. However, it is assumed that in general there is no damage of the rock shed under the estimated impact load by a rock fall. Therefore, designers of protection measures undergo some difficulties especially for rock fall with very large energy.

DYNAMIC EFFECT ON THE BEHAVIOR OF ROCK SHEDS

Loading due to a falling rock is a dynamic and large concentrated force on the roof of a rock shed. Therefore, it is necessary to know the dynamic effects. For its practical use, the impact load should be treated as an equivalent static load.
To investigate this effect both experimental studies using a full-scale model of a prestressed concrete rock shed roof and analytical studies based on the finite element method have been carried out [10]. An impact factor i for an equivalent static design force has been proposed as follows.

$$\begin{aligned} &i = 0 && \text{for} \quad T_0/T < 0.3 \\ &i = T_0/T - 0.3 && \text{for} \quad 0.3 < T_0/T < 0.7 \\ &i = 0.4 && \text{for} \quad 0.7 < T_0/T \end{aligned} \tag{7}$$

Here, T_0 is the duration of impact force, T the first eigen period of the roof of the rock shed. The impact factor represents the dynamic effect as shown in eq. (8) .

$$i = \frac{\delta_d - \delta_s}{\delta_s} \tag{8}$$

Here, δ_s is the static displacement (or strain) of the roof of a rock shed when the maximum value of dynamic force acts statically and δ_d is the maximum of the dynamic displacement (or strain). Figure 5 shows the relationship between T_0/T and i (also i+1). From this result, it has been recommended to use the dynamic factor (i+1) to estimate the impact load.

EVALUATION OF IMPACT ACTION FROM OTHER POINT OF VIEW

As mentioned above, the estimation of the impact load due to rock fall has been ranked important for the design of rock sheds. Engineers treat this load as a static load. However, for a more rational and realistic design, the state exceeding the elastic range is necessary to be considered. Therefore, some evaluation ideas from another point of view are introduced.
Sonoda presented the idea of ultimate design method for rock shed [11]. He proposed the simple model assuming that the collision between the rock and the rock shed is perfectly plastic (Figure 6).

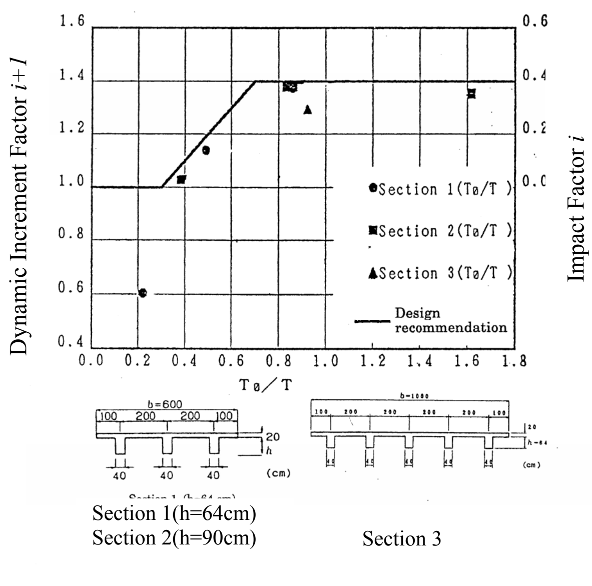


Figure 5 Impact factor and dynamic amplification factor

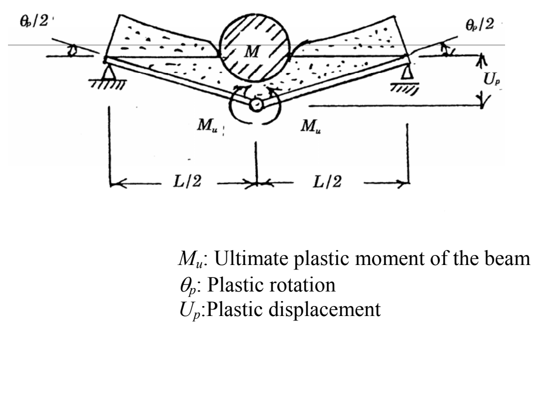


Figure 6 Simple assumption of failure mode of rock shed

For a rational estimation of the design load, the rock fall experiments have been carried out using full-scale models of a steel roof shown in Figure 7 [12]. From the results of the experiments, a strong correlation between maximum deflection, reaction forces and bending moment was observed. It was shown that these could be approximated by simple static theory. It became clear that the potential energy of a rock is absorbed in the sand cushion to than 80% and that the remaining energy is transferred to the rock shed.

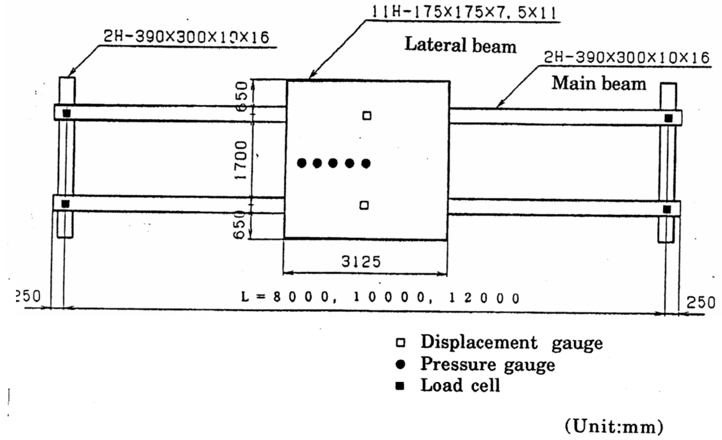


Figure 7 Steel rock shed roof model used in experiment

Analytical research using a composite simulation was achieved, in which finite elements are used for the rock shed and distinct elements for the sand cushion as shown in Figure 8 [13]. Equation (9) was proposed, which presents the energy transmitted from the rock to the rock shed (Figure 9).

$$\left(E_t/E_p\right)\left(M/m\right) = -1 + 2.5/\left(T_d/T\right) \qquad (9)$$

Here, T_d is the impact duration, T the first eigen period of the structure, *M the* effective mass of rock shed, *m the* mass of the rock, E_p the initial energy of the rock and E_t the transmitted energy to rock shed.

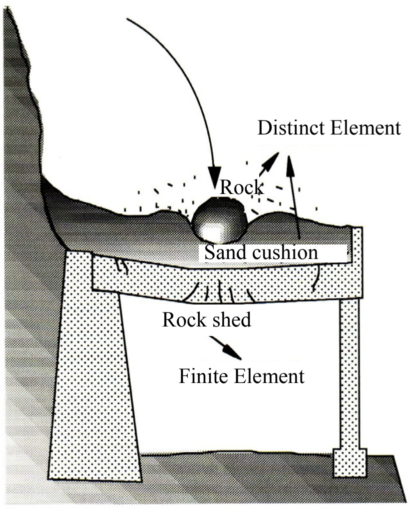


Figure 8 Analysis of rock shed by DEM and FEM

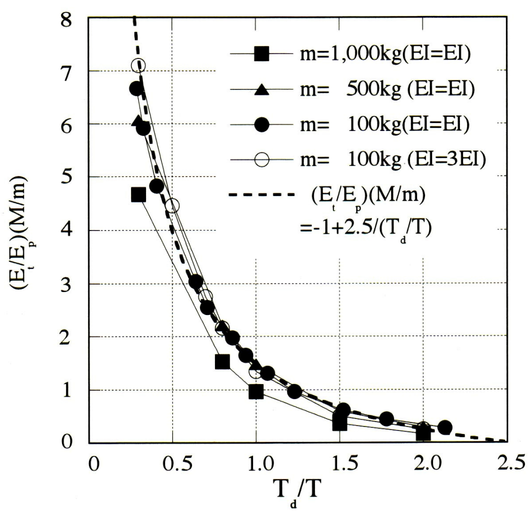


Fig.9. Estimation of energy transmission to rock shed

ACTIVITY TOWARDS PERFORMANCE BASED DESIGN

Practical design method based on the concept of performance-based design is strongly demanded. Therefore, concerning rational methods according to the performance-based design, technical investigation had to be performed not only for general structures but also for especial structures under impact load in order to secure a reliable high safety. "The foundations of the design concerning engineering works and construction" were published from the ministry of land, infrastructure and transport on October 2002 in Japan. From such a point of view, the Subcommittee for Impact Problems of the Japanese Society of Civil Engineers started the research activity concerning performance-based design of structures against impact actions by seven workgroups shown below.

1) Experimental evaluation methods for the impact performance and design of structural members
2) Analytical evaluation methods for the performance of structures.
3) Investigation of the performance of structures against explosion.
4) Performance based design of protection structures against rock falls.
5) Performance based design of dams for an avalanche consisting of rocks and soil.
6) Impact actions and the limit states of structures.
7) Other impact problems (collision of aircraft, collision between port structure and ship, impact by vehicle, damping system for earthquake, etc.)

Each working group performed investigations of the fundamental data. Between the working groups, the exchange of information, mutual support, and discussion performed remarkably [14], [15]. The committees conducted round robin analysis to estimate the results of impact experiments by large-scale reinforced concrete beam in advance. This was a competitive analysis in the correctness of calculation result. Trial design of the reinforced concrete beam by performance-based design was also conducted. Many members of this committee participated in these technical events. Fairly interesting results were obtained and valuable discussions have been done at the committee meetings. Finally, the committee published the "Comprehensive design code of structures under impact actions founded the performance based design concept (proposal)". Succeeding to this code, "Comprehensive performance based design code for dams against an avalanche consisting of rock and soil by performance based design (proposal)" and "Design code for protection structures against rock falls by performance based design (proposal)" has been also published. Table 3 shows one example of performance requirements of protection structure for rock fall introduced in this code [15]. In addition, research towards performance design of structures against explosions and investigations concerning the design method of structures against aircraft collision have also been carried out.

Table 3 Example of performance requirements of protection structure for rock fall

The action level of rock fall	Extent of importance of protection structure for rock fall		
	Most important structure	Important structure	Usual structure
Level 1 Frequent (Ex.: few times per year)	Serviceability limit state	Serviceability limit state	Restorability limit state
Level 2 Rare (Ex.: once or two times during design working life)	Serviceability limit state	Restorability limit state	Ultimate limit state
Level 3 Very rare (Ex.: Probability is low but its energy is very large.)	Restorability limit state	Ultimate limit state	Near collapse

The impact committee of the Japanese Society of Civil Engineers started new activities aiming the classification of structures under impact action and specification of the performance of each structure according its classification. It is expected that examination methods by reference for protection structures against rock fall will be investigated more detailed. The work is going to be achieved by the members of the committee [16], [17].

CONCLUSIONS

This paper has described the history of rock fall research in Japan. Methods to treat the impact force due to rock fall as an equivalent static load, the dynamic effect of structural behavior due to impacts, the energy absorption of the sand cushion, the energy transmission to the protective structure, recent activities concerning a performance based design and the view towards a rational design have been introduced. The design concept and method of protection structure differs in each country. It is considered that development of the globally common technology and design method of protection structure is desirable from now on so that the design method satisfying requested performance to the impact action in each risk level can be established. The author is therefore very pleased, if the described contents contribute to the interest or as a hint to engineers involved in the design or development in this field.

REFERENCES

[1] Japan road association: Handbook of preventatives against rock falls, 2000.
[2] Japan railway civil engineering association: Manual for preventive structures against rock falls, 1978.
[3] Express highway research foundation of Japan: The research report of preventive structures against rock falls, 1974.
[4] Japan Road Association: Reference concerning Hand Book for Rock falls -Research of the simulation method of the motion of rock falls on slope, Japan Road Association, 2001.
[5] Yoshida H., Masuya H., Ushiro T. and Shibata K.: A survey of design of rock sheds and an approach to improved design, Proceedings of Japan society of civil engineering, No.421, pp.213-222, 1990.
[6] Yoshida H., Ishizuka K. and Hosokawa Y.: Experimental study on the estimation of impact load for preventive structures against rock falls, Proceedings of 13th meeting of Japan road association, pp.35-41, 1976.
[7] Masuya H: The evaluation of the impact load on rock shed by the falling rocks, Doctoral theses, 1989.
[8] Yoshida H., Masuya H., Fujii T. and Maekawa K.: Impulsive behavior of steel rock barriers under a falling rock, Proceedings of Japan Society of civil engineering, N0.409, pp.75-84, 1989.
[9] Nakana O., Kishi N., Gotou Y. and Endoh H.: Development on 3-layed shock-absorbing system of rock sheds, Journal of Japan society of civil engineering, vol.77, pp.20-33 1992.
[10] Yoshida H., Masuya H. and Oka M.: Impulsive loading properties by falling stones on rock sheds, Proceedings of Japan society of civil engineering, No.362, pp.467-470, 1985.
[11] Sonoda K.: Limit state design and the design method of rock sheds, The report of 35 years' anniversary symposium of Hokuriku road research association, pp.22-34, 1988.
[12] Masuya H., Mitake Y. and Kajikawa Y.: Test of rock falls on steel rock shed and distributive energy of rock shed, Journal of structural engineering, JSCE, pp.1627-1636, 1991.
[13] Masuya H. and Nakata Y.: Development of numerical model combining distinct element and finite element methods and application to rock shed analysis, Proceedings of Japan society of civil engineering, No.710/I, pp.113-128, 2002.
[14] Kishi N (ed.):.Impact committee of Japan society of civil engineers: Practical methods for impact test and analysis, Japan society of civil engineers, Structural engineering series 15, 2004.
[15] Masuya H (ed.): Impact committee of Japan society of civil engineers: Practical methods for impact test and analysis, Japan Society of Civil engineers, Structural engineering technical series 50, 2007.
[16] Masuya H., Tachibana S.: Simulation of behavior of reinforced concrete beam under impact, Proceedings of the 2nd international conference on design and analysis of protective structures against impact/ impulsive/ shock loads, pp.406-416, 2006.
[17] Masuya H., Tachibana S. and Nakamura S.: Study on the performance evaluation of reinforced concrete beam under lateral impact, Proceedings of performance, protection and strengthening of structures under extreme loading, JPN29 (in CD), 2007.

THE SEISMIC EFFECT ON ROCKFALL HAZARD

Olga Mavrouli[1], Jordi Corominas[2], and Joseph Wartman[3]

A methodology is presented here for the evaluation of earthquake-induced rockfalls. It is a three-step procedure that involves: (i) the estimation of the horizontal peak ground acceleration (PGA) at the base of the slope from available seismic hazard maps, (ii) the estimation of topographic ground motion amplification at the crest of the slope based on existing guidelines and finally, (iii) the assessment of the slope stability including the seismic effect by means of limit-equilibrium analysis. The methodology is applied to the Solà de Santa Coloma slope, situated in the Principality of Andorra. The results, which reveal the effect of earthquake on the rockfall hazard, are discussed and the methodology is evaluated.

Keywords: earthquake, rockfall, hazard, limit-equilibrium analysis

INTRODUCTION

Earthquake-induced rockfalls comprise a considerable fraction of landslides worldwide, posing a significant threat to humans and infrastructures. The assessment of earthquake-induced rockfall hazards is an important part of risk assessment for mountain areas. For this purpose, the local conditions of each studied area should be considered, including both geological characteristics and the seismic hazard.

In a 1994, David Keefer [1] presented what has become one of the most popular approaches in practice for assessing seismic rockfall hazards Keefer's empirically based approach identifies geologic and topographic factors that are characteristic of large seismically induced rockfalls. Yet this approach appears to be conservative. For example, despite having many slopes meeting the hazard susceptibility criteria of Keefer, a review of the historical archives suggests that no significant rock failures occurred during the Boi earthquake of 1919, a seismic event that had one of the strongest effects in the region of Andorra, over the last hundred years. Accordingly, we have developed a refined analysis procedure intended to further investigate areas identified as being susceptible to seismically induced rockfalls based on the Keefer criteria.

[1] Ph.D student, UPC, Technical University of Catalonia, E.T.S. Department of Geotechnical Engineering and Geo-Sciences, Campus Nord, edifici D2, C. Jordi Girona, 1-3. 08034 Barcelona, Spain, +34 93401, olga-christina.mavrouli@upc.edu.

[2]Professor, UPC, Technical University of Catalonia, E.T.S. Department of Geotechnical Engineering and Geo-Sciences, Campus Nord, edifici D2, C. Jordi Girona, 1-3. 08034 Barcelona, Spain, +34 93401 6861, jordi.corominas@upc.edu.

[3]Associate Professor, Department of Civil, Architectural, and Environmental Engineering, Drexel University, 3141 Chestnut Street, Philadelphia, PA 19104 USA, 215 895 2087, joseph.wartman@drexel.edu.

In order to refine the seismic rockfall hazard assessment, so as to take into account the particular geological and earthquake properties of each area, a methodology is proposed. The methodology is based on the analytical evaluation of rock mass instabilities in the presence of earthquake ground excitation, considering existing kinematic conditions.

EFFECTS OF EARTHQUAKES ON ROCK SLOPES

During an earthquake event, rock instabilities are mainly due to destabilizing inertial forces that are developed in the rock mass. Their magnitude of these inertial forces is related to ground motion intensity, often represented by peak acceleration values. In the case of steep rocky slopes, the amplification of seismic waves can occur from topographic amplification [2] leading to higher values of the ground motion characteristics. This phenomenon accounts for the fact that even in the case of low seismicity areas, the earthquake-induced rockfall hazard can be high.

THE PROPOSED METHODOLOGY

The proposed methodology is analytical and is based on the evaluation of possible jointed rock masses. The three-step procedure involves: (i) the estimation of ground motion parameters at the base of the slope, (ii) the estimation of topographic ground motion amplification at the crest of the slope and finally, (iii) the assessment of seismic stability by means of stereographic and limit-equilibrium analyses.

The magnitude of the earthquake event is expressed in terms of horizontal peak ground acceleration (PGA). For the determination of the peak ground acceleration on the surface base of the slope, seismic hazard maps and/or codes for the studied area can be used. The PGA on the crest of the slope is estimated considering a percentage increase based on existing guidelines developed by Ashford and Sitar [3].

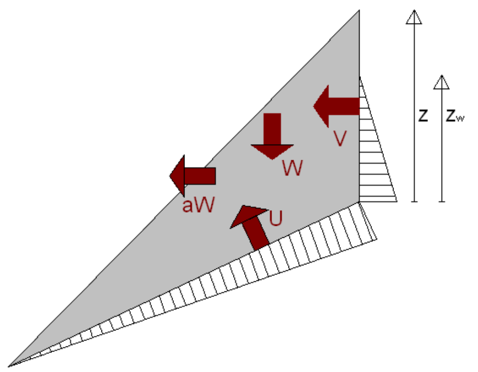


Fig. 1 Limit equilibrium conditions for the rock block

The seismically induced instability assessment is performed by means of limit-equilibrium analysis (Fig. 1). Initially, the rocky slope relief is represented using available topographical maps. The studied area is discretized into cells (i.e., sub-zones) were geological and earthquake parameters are considered constant. The dip direction and dip of the principal joint sets are obtained by in situ observations. For the limit-equilibrium analysis, water pressure is taken into account according to local conditions. The joint strength parameters: cohesion, c, and friction angle, φ, are those corresponding to rock mass rating (RMR) values, as calculated using in situ data. The slope angle is taken equal to the inclination of each cell's characteristic section. For the calculation of the corresponding safety factor, a limit-equilibrium block analysis is made, where the destabilizing earthquake effects are represented by a horizontal pseudostatic force applied to the center of a potential failure mass.

Given the obtained safety factor values for each cell, an indication of the seismic rockfall hazard can be analytically and spatially derived. The establishment of appropriate thresholds can, additionally, provide a qualitative description of the hazard level. Furthermore, a parametric comparative analysis, for earthquake or no-earthquake conditions, shows the importance of the seismic effect on the rockfall hazard.

STUDY SITE

The methodology is applied in the Solà de Santa Coloma, a steep slope located in the Andorra Principality, in the Pyrenean Range between France and Spain. The rock structural data are given by [4]. Despite the moderate seismicity of the area, the earthquake topographic amplification could trigger large rockfall events, threatening the urban area of Santa Coloma, located near the base of the slope.

CONCLUSION

A methodology for the evaluation of the seismically induced rockfall hazard of mountain areas is proposed. Instead of using empirical or statistical data, this methodology is analytical, based on limit-equilibrium analysis. The magnitude of the earthquake event is taken into account for the analysis, as well as the topographic amplification effect. The application of the proposed methodology can provide quantitative results for the seismic rockfall hazard.

The application of the proposed methodology is made for some critical slopes of the region of the Solà de Santa Coloma slope. The obtained results reveal the increase of the rockfall hazard due to earthquake, in comparison with no seismic ground motion conditions, despite the low seismicity of the area.

ACKNOWLEDGEMENTS

This work has been performed within the framework of the “Mountain Risks” Marie Curie Research Training Network, financed by the European Union (6th Framework Program). Additional support has been provided to the third author by the U.S. National Science Foundation (Grant No. CMS-0134370).

REFERENCES

[1] KEEFER, D. K. (1993) The susceptibility of rock slopes to earthquake/induces failure, Association of Engineering Geologists Bulletin, Vol. 30, p. 353-361.
[2] AKI, K. (1988). Local site effects on strong ground motion, Earthquake Engineering and Soil Dynamics II, Recent Advances in Ground Motion Evaluation, June 27-30, Park City, Utah.
[3] ASHFORD, S. A., SITAR N., ASCE M. (2002) Simplified Method for Evaluating Seismic Stability of Steep Slopes, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 2, p. 119–128.
[4] COMELLA J. (2003) Aplicaciò de les classificacions geomecàniques a l’ avaluacio de la susceptibilitat dels macizos rocosos a produir despreniments. Graduate Thesis. Civil Engineering School. Barcelona.

ROCK FALL RISK RATING FOR SETTLEMENTS: DEVELOPMENT OF A RATING SYSTEM BASED ON A CASE STUDY

Michael Mölk[1], Rainer Poisel[2]

Rock-fall risk rating systems were applied frequently for linear infrastructures such as roads, pipelines or railways in the past [3]. The application of rating systems resulting in a relative risk for populated areas such as villages in mountainous areas are rare and currently discussed intensely ([1]und [5]). Such risk rating systems would enable the authorities responsible for public safety, land use planning and/or the funding of mitigation measures to optimize their decisions and the steering of public funding. The presented method was developed to obtain an objective tool for the evaluation of the rock fall-risk of developed areas within the limits of settlements in alpine regions in the course of hazard zoning.

Keywords: rock-fall risk rating, settlements, hazard zoning

INTRODUCTION

The proposed rock fall rating system [1] aims to a comprehensive and reproducible approach to evaluate the rock fall risk, specific parts of a settlement are exposed to. By plotting the results of the evaluation of the potential rock fall source areas, the rock fall path, historical frequency of rock falls and the quality of the land-use potentially being effected by the detachment areas in a frequency-tolerance-diagram gives a clear indication of further actions required by the authorities responsible for the public safety.

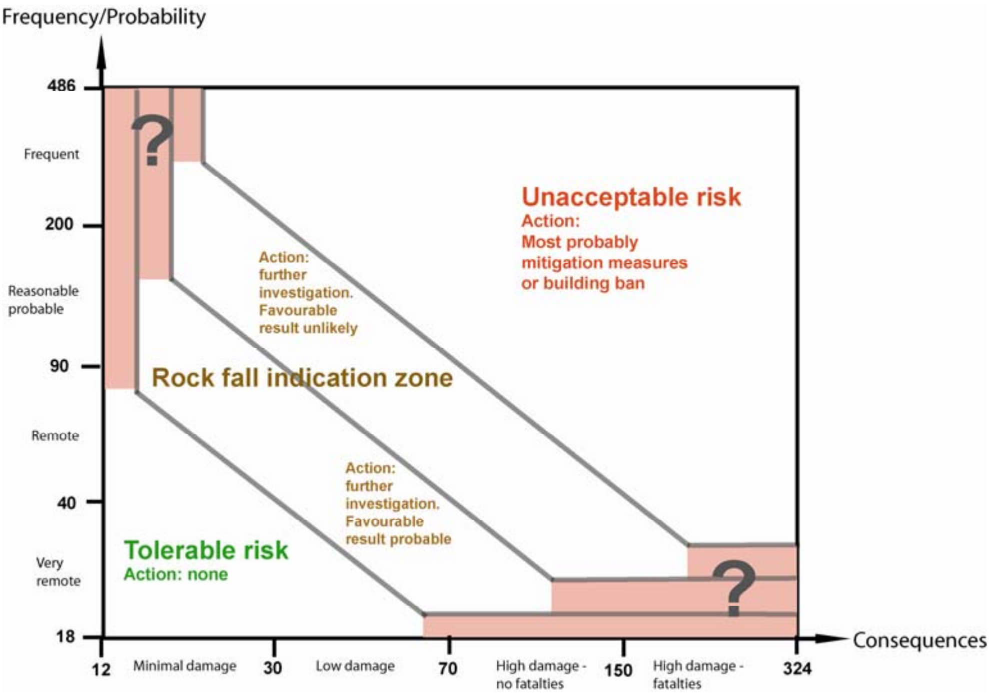


Fig. 1: Frequency/Consequence-diagram

[1] Forsttechnischer Dinest f. Wildbach- und Lawinenverbauung, Geoloigsche Stelle, Liebeneggstr. 11, A-6020 Innsbruck, Austria, +43/(0)512/584200-38 (fax: -44), michael.moelk@die-wildbach.at
[2] TU Wien, Inst. f. Ingenieurgeologie, Karlsplatz 13/203, A-1040 Vienna, Austria, +43 (0) 1- 58801-20301 (fax: -20399), rainer.poisel@tuwien.ac.at

These actions can consist of:

- no activities required due to an acceptable risk
- further investigations required – necessity to declare a rock-fall indication zone in the hazard zoning
- construction of mitigation measures or respective land use planning (building ban …)

For the hazard zoning of rock fall processes the following standard procedure is proposed:

Step 1: Surveillance by aerial photographs, DTMs – identification of scree-slopes and location of potential detachment areas

Step 2: Investigation of rock fall history (chronicle, interviews)

Step 3: field investigation:

- Identification of outcrops/source area
- Measurement of shadow angle of 26° (as proposed by [4] from highest point of each source area.

Step 4: If land use as defined in Tab. 1, line 9 lies within the pathway bordered by the shadow angle, the rating system has to be applied.

DESCRIPTION OF THE RATING SYSTEM

The rating system evaluates nine parameters and rates each respective characteristic of the slope into one of four classes. These classes reflect a semi-quantitative assessment of risk-relevant parameters.

Tab. 1 Rating criteria and score

		Parameter	3 Points	9 Points	27 Points	81 Points
Source area	1	**Loosening of rock**	joints closed	joint width mm	joint width cm	joint width dm
	2	**Joint strength**	rough joints	undulated joint-planes	planar joints	slickensides, joint gauge
	3	**Joint discontinuity and orientation**	discontinuous joints, favourable orientation	discontinuous joints, random orientation	discontinuous joints, adverse orientation	continuous joints, adverse orientation
Pathway (transition + immission zone)	4	**Vertical slope-height [m]**	<100	100-300	300-500	>500
	5	**Climate and water**	aspect of slope=north, no water present on slope	slope tends to be dry	water present on slope	aspect of slope=south, permanent water leakage
	6	**Block size [m³] d_{90}**	< 1	1 - 5	5 - 10	> 10
	7	**Pathway: roughness+damping**	high roughness, good damping	rough, forested slope, good to mean damping (i. e. scree slope)	little vegetation, smooth, mean to poor damping	no vegetation, poor damping (rocky surface)
	8	**Proof of historical events**	no events reported, no silent witnesses	silent witnesses no events reported	1 event/10 years	>1 event/10 years
	9	**Quality of landuse**	agriculture	periodically used buildings	periodically inhabited buildings	permanently inhabited buildings

In order to achieve a risk based evaluation of the developed land exposed to rock fall, the sum of the scores of the parameters influencing the probability of rock fall and the sum of the scores of the parameters influencing the damage are multiplied [4]. The parameters "pathway roughness" and "damping" influence both, probability and damage. It is therefore proposed to count vertical slope height, block size, pathway roughness and damping as well as quality of land use to the group of damage influencing parameters and loosening of rock, joint strength, , joint discontinuity and orientation, climate/water and pathway roughness and damping to the group of probability influencing parameters (table 2).

Tab. 2 Calculation of relative risk (example of worst case)

		Parameter	Influences	Sum scores damage potential	*	Sum scores Frequency/Probability
Source area	1	Loosening of rock	Frequency/Probability			81
	2	Joint strength	'Frequency/Probability			81
	3	Joint discontinuity and orientation	'Frequency/Probability			81
Pathway (transition + immission zone)	4	Vertical slope-height [m]	Damage	81		
	5	Climate and water	'Frequency/Probability			81
	6	Block size [m³] d_{90}	Damage	81		
	7	Pathway: roughness+damping	Damage + Frequency/Probability	81		81
	8	Proof of historical events	'Frequency/Probability			81
	9	Quality of land use	Damage	81		
Sum of column				324	*	486
Relative Risk = Sum damage potential x sum frequency/probability				157.464		

The above described system was tested in a touristic highly developed valley in Austria and the results give a reproducible determination of the relative risk the different settlements of the community are exposed to. The risk rating furthermore can lead to a hazard zoning taking into account the rock fall hazard of each part of the community. This enables the authorities to prioritise its actions and to control future land use planning. The application of the proposed rating system lead to a geotechnical categorization of the identified areas of the respective community at risk, that showed different urgencies concerning mitigation measures and/or actions to be taken by the land use planning authorities.

Tab. 3 Calculation of relative risk for different parts of the community used as a test bed

	Versahl Area B		Versahl Area C		Unterschrofen		theoretical tolerable case	
Parameter	Damage	Frequency/ Probability	Damage	Frequency/ Probability	Damage	Frequency/ Probability	Damage	Frequency/ Probability
Loosening of rock		27		9		9		3
Joint strength		27		3		9		3
Joint discontinuity and orientation		27		9		9		3
Vertical slope-height [m]	9		27		9		9	
Climate and water		9		9		9		3
Block size [m³] d_{90}	9		3		3		3	
Pathway: roughness+damping	9	9	27	27	27	27	3	3
Proof of historical events		27		27		9		9
Quality of land use	81		81		9		9	
Summary scores for damage and probabiliy	108	126	138	84	48	72	24	24
Relative Risk (Damage x Frequency)	13.608		11.592		3.456		576	

REFERENCES

[1] MALONE, A.W. (2007): Risk management and slope safety in Hong Kong. Thirty years of slope safety practice in Hong Kong. The Government of the Hong Kong Special Administrative Region.

[2] MOELK M. POISEL, R., WEILBOLD, J. & ANGERER, H. (2008): Rockfall Rating Systems: Is there a comprehensive method for hazard zoning in populated areas?. – Interpreavent 2008 (in preparation).

[3] PIERSON, L. A., DAVIS, S. A. and VAN VICKLE, R. (1990): Rockfall hazard Rating System Implementation Manual. Federal Highway Administration (FHWA) Report FHWA-OR-EG-90-01, US Department of Transportation

[4] WYLLIE, D. C. (2006).: Risk management of rock fall hazards. – Sea to Sky Geotechnique, Conference Proceedings, Vancouver 2006

[5] WONG, H.N. (2007): Landslide risk assessment for individual facilities. Thirty years of slope safety practice in Hong Kong. The Government of the Hong Kong Special Administrative Region.

NUMERICAL SIMULATION FOR SURVEYING INPUT ENERGY AT ULTIMATE STATE OF RC ROCK-SHED

Shin-ya Okada[1], Hisashi Konno[1], Hiroaki Nishi[1], and Norimitsu Kishi[2]

In order to establish impact resistant design procedure for RC structures based on limit state design method, a three-dimensional elasto-plastic FE analysis of a RC rock-shed designed based on allowable stress design method was conducted. Its impact resistant behavior at the ultimate load-carrying capacity (hereinafter, ultimate capacity) limit state and serviceability limit state was investigated. From this study, it is confirmed that RC rock-shed designed based on allowable stress design method retains fifty times and twenty-five times safety margin for ultimate strength limit state and serviceability limit state, respectively.

Keywords: rock-shed, three-dimensional elasto-plastic FE analysis, limit state design method, input impact energy

INTRODUCTION

In preparation for establishment of impact resistant design method for RC structures based on limit state design concept, a RC rock-shed designed based on allowable stress design method was numerically analyzed by means of 3D elasto-plastic FE analysis method. Here, its numerical results at both ultimate capacity and serviceability limit states were investigated. In this study, numerical simulation was performed using 3D FE code LS-DYNA (Ver.970) [1].

NUMERICAL MODEL

Figure 1 shows the numerical analysis model of the rock-shed. The model used in this study is the same as a design example of real RC rock-shed introduced in “Impact resistant design for rock-shed” [2]. Concrete, sand cushion and steel weight were modeled using eight-node solid elements, and rebars were modeled using beam elements. Total number of nodes and elements of the model are 87,882 and 128,890, respectively.

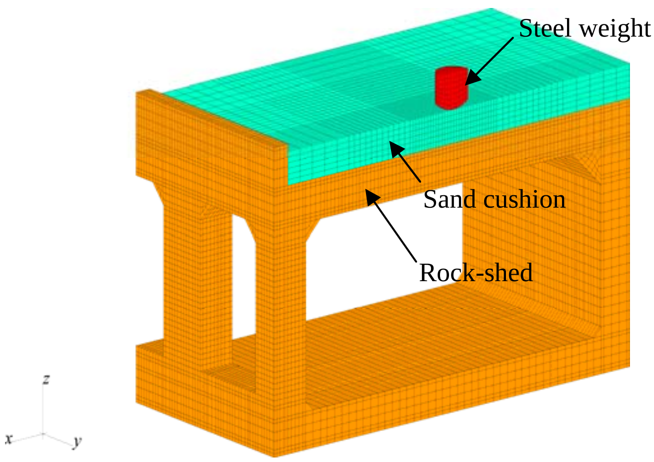


Fig.1 Analysis Model of RC rock-shed

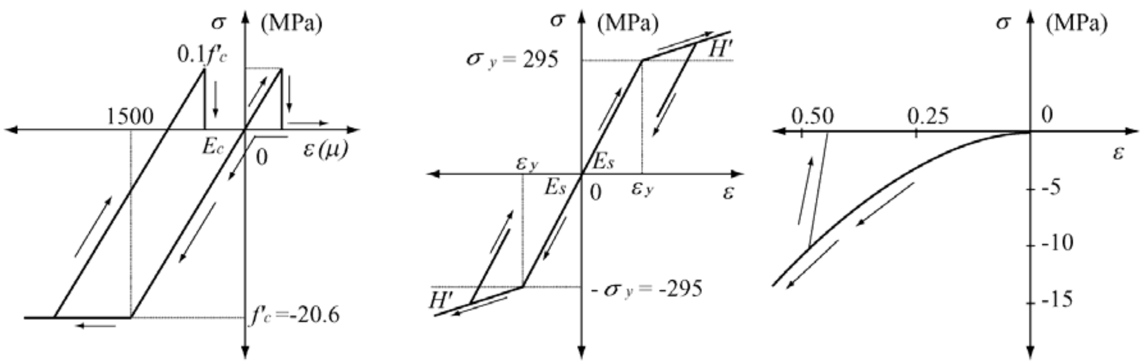
[1]Civil Engineering Research Institute for Cold Region. 1-3-1-34, Hiragishi, Toyohira, Sapporo, 062-8602 JAPAN, +81-11-841-1698 (fax:-3502), okada@ceri.go.jp; konno@ceri.go.jp; h-nishi@ceri.go.jp
[2]Muroran Institute of Technology. 27-1, Mizumoto, Muroran, 050-8585 JAPAN, +143-46-5226(fax:-5227), kishi@news3.ce.muroran-it.ac.jp

Tab. 1 List of cases of numerical analysis

Analysis case	Falling height (m)	Mass of steel weight (t)	Input energy E (MJ)
E-0.2	**20**	**1**	**0.196**
E-2.5	**50**		**2.45**
E-4.9	**100**		**4.90**
E-7.4	**150**	**5**	**7.35**
E-9.8	**200**		**9.80**
E-12.3	**250**		**12.3**

Tab. 2 Material properties

Material	Density (kN/m^3)	E-mudulus (GPa)	Poisson's ratio
Concrete	**23.0**	**13.7**	**0.167**
Rebar	**76.9**	**206**	**0.300**
Sand cusion	**15.7**	**10.0**	**0.060**



(a) concrete model (b) rebar model (c) sand model

Fig.2 Stress-strain relation of material constitutive models

Sand cushion set on the roof of rock-shed was considered in this analysis, because: 1) dead load of the sand cushion cannot be ignored comparing with that of concrete; 2) the weight of sand cushion may affect on impact response behavior of the rock-shed; and 3) impact load transmitted on the rock-shed may be widely distributed due to sand cushion. Boundary conditions of the rock-shed were defined as: 1) bottom surface of rock-shed was perfectly fixed; and 2) symmetrical condition was adopted at the central section in the road direction.

Table 2 shows the list of material properties for numerical analysis. Figure 2 shows the stress-strain relation for each material. The relationship for concrete was assumed as a bilinear model in the compression side and a cut-off model in the tension side as shown in Fig. 2(a). For rebar, an elasto-plastic model following isotropic hardening rule was applied as shown in Fig. 2(b). Figure 2(c) shows the constitutive model for sand cushion, which is formulated based on the experimental results [3] such as:

$$\sigma_{sand} = 50\ \varepsilon_{sand}^{2}$$

in which σ_{sand} is stress and ε_{sand} is the volumetric strain.

DEFINITION OF ULTIMATE STATE

In this study, to investigate the trend of axial strain of rebar and residual displacement at the loading point (hereinafter, strain and displacement) corresponding to increase of input impact energy E, numerical analyses were performed by gradually increasing input impact energy E. The cases of numerical analyses are listed in the Table 1.

Here, to evaluate impact resistant capacity of rock-shed based on the concept of limit state design method, it is defined that: 1) a RC rock-shed reaches ultimate capacity limit state when the increasing ratios in rebar strain and displacement have a tendency to be rapidly increased; and 2) it reaches serviceability limit state when the increments in rebar strain and displacement are moved from linear to non-linear variation and/or punching shear failure occurred. This is an important phenomenon considering that spalling of concrete blocks may set off the serious accident involving human lives, even if the rock-sheds have never been collapsed yet.

NUMERICAL RESULTS

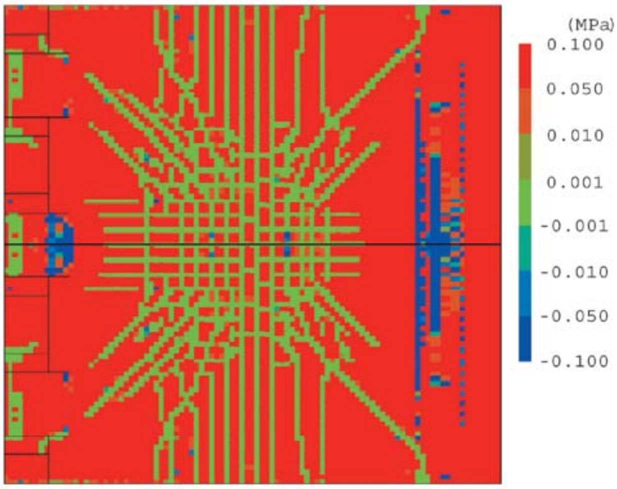
From the numerical results, it is seen that rock-shed reaches ultimate capacity and serviceability limit states at the cases of input energy E = 12.3 MJ and 7.4 MJ, respectively. On the other hand, the rock-shed considered here was designed based on allowable stress design method (current design method) assuming maximum input energy E = 0.196 MJ. Thus, it is seen that the rock-shed designed based on current design method has fifty and twenty-five times safety margin for ultimate limit state and serviceability limit state, respectively.

Figure 3 shows an example of analytical results for 1st principle stress distribution at E = 12.3 MJ. In this figure, it can be estimated that the area with green color stress contour has been damaged due to crack occurrence.

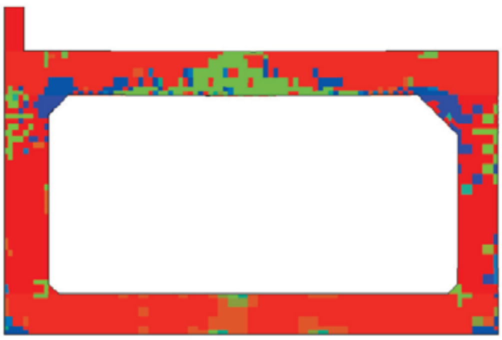
CONCLUSION

In this paper, in preparation for establishment of impact resistant design procedure for RC structures based on limit state design concept, 3D elasto-plastic FE analysis for RC rock-shed designed based on allowable stress design method (current design method) was performed.

From this study, it is confirmed that the RC rock-shed designed by current design method in Japan has fifty and twenty-five times safety margin for ultimate capacity limit state and serviceability limit state, respectively.



(a) Lower surface of roof of rock-shed



(b) Cross section along the centerline

Fig.3 Principle stress distribution of rock-shed at E = 12.3 kN

REFERENCES

[1] John O.Hallguist (2003) LS-DYNA User's Manual, Livermore Software Tecnology Corporation
[2] JSCE (1998) Impact resistance design of rock-shed, Structural engineering series8.
[3] Bhatti, A.Q., Kishi, N., Konno, H., and Okada, S. (2007) "Numerical analysis of falling weight impacting RC girders converted with partially supported sand cushion", Journal of Applied Mechanics JSCE, Vol.10, 265-272

SIMULATIONS OF ROCKFALL IMPACTS ON EMBANKMENTS USING A DISCRETE ELEMENT METHOD

Jean-Patrick Plassiard[1], Frédéric-Victor Donzé[2], Julien Lorentz[3]

The efficiency of embankments subjected to rockfall impacts is studied with a numerical approach based on the Discrete Element Method (DEM). A number of parameters were tested both with respect to the embankment itself and the rock mass. Once the reference model has been calibrated an application to a real test case is presented. The model is thus validated and shows the advantages of using such an approach to design embankments.

Keywords: reinforced embankments, rockfall impact, numerical simulation, Discrete Element Method.

ROCKFALL IMPACTS ON EMBANKMENTS

A reinforced embankment is built when impact energies in the 5 000 to 50 000 kJ range or higher are present. The upstream face is inclined up to values of 65° with respect to the horizontal axis thus preventing the rock mass from getting over the embankment. Current methods to design embankments are based on the translational kinetic energy and the trajectory of blocks both of which are given by trajectographic studies. However, none of the block parameters are usually considered (i.e. size and form, rotational kinetic energy, orientation of impact) nor are those of the embankment itself (i.e. inclination of its sides, crest thickness or properties of soil). Moreover, experimental tests are too restrictive as some parameters cannot be varied. For example the campaign of impacts done at Trento (Italy) only used a given impact orientation and did not consider a rotational velocity [2]. An alternative study using numerical simulations is proposed here. The discrete element method - DEM - has been chosen as implemented in the SDEC software [3].

NUMERICAL MODEL AND VALIDATION

In the first models it is assumed that reinforcement can be neglected so that embankments are only made of soil. The numerical model must first be representative of both the quasi-static and the dynamic behaviours of this soil, before being used to simulate embankments. Thus the validation with the DEM requires two major steps: simulations of triaxial tests (fig. 1) to reproduce the averaged properties of soil (Tab. 1) and then, the discrete model is used to simulate boulder impacts onto granular layers [4]. Both of these steps correspond to a calibration phase of the discrete model. Finally the soil model thus obtained was able to reproduce high energy impacts of rock masses onto granular layers [5] so that it can be assumed that the model can now simulate impacts on embankments.

[1] Université de Lyon, LTDS, Ecole Centrale de Lyon, 36 avenue Guy de Collongue, F-69134 Ecully Cedex, France, jp.plassiard@gmail.com
[2] Université Joseph Fourier, Laboratoire 3S-R, Domaine Universitaire, BP53, F-38041 Grenoble Cedex 9, France, Donze@geo.hmg.inpg.fr
[3] IMSRN, Z.A Pré Millet, 680 rue Aristide Bergès, F-38330 Montbonnot, France, julien.lorentz@imsrn.com

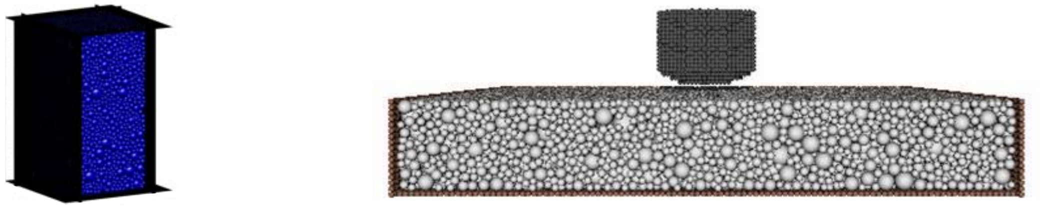


Fig. 1 Simulations of a triaxial test and of a boulder impact onto a granular layer.

Tab. 1 Averaged properties estimated from the soil of three embankments.

Elastic modulus E (MPa)	Poisson's ratio ν (-)	Friction angle at peak $\phi_i^{'}$ (-)	cohesion c' (kPa)	Dilatancy angle ψ (-)	Residual friction angle $\phi_{rés}^{'}$ (-)
100	0.3	43	10	15	35

SIMULATION OF IMPACTS: GENERAL RESULTS

The calibrated model has been applied to a reference model of embankment [6]. This model (fig. 2) is 5 m high and both upstream and downstream sides are inclined at 60° relative to the horizontal axis. The crest size is 2 m thick while the reference rock mass is a spherical boulder with a 2 m diameter. Parameters governing both the block and the embankment have been tested (fig.2), but only the embankment parameters will be discussed here.

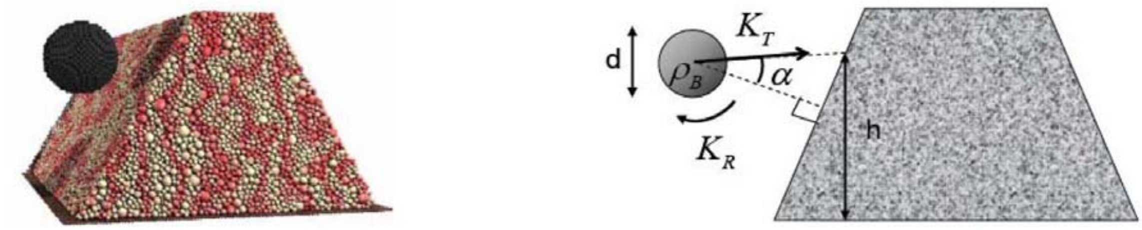


Fig. 2 Numerical model of embankment and main parameters which govern the rock mass and the embankment.

The efficiency of the embankment has proven to be governed by the following four parameters: translational and rotational kinetic energies K_T and K_R, height h and impact orientation α. Whatever the diameter size d, the density ρ_B and the translational velocity V are, the response is not controlled independently by any one of these parameters but by a combination of these that corresponds to the translational kinetic energy K_T. When $K_R/K_T > 0.1$, the boulder may go over the embankment thus making this value critical. Finally the inefficiency of the structure was also studied for critical impact orientation α. Multi parametrical studies have been performed using two major dimensioning parameters (h; K_T), coupled with either the rotational kinetic energy K_R or with the impact orientation α. A critical combination of parameters has been observed, showing that the efficiency of the structure depends on the parameters that are considered. A future study will focus on the combination of these parameters.

APPLICATION TO A REAL CASE

The same method has been used to study a real case of rockfall hazard at Val d'Isère (France), (fig. 4). Given the block size and both, its kinetic energy and its trajectory, an initial embankment was designed (fig. 4). Because of the almost 75° inclination on both sides of the construction, the cohesion value of the model had to be raised to 30 kPa [7]. A parametrical study on the crest size was done and it lead to a reduction of its thickness from 5 m to 4 m, which meant that significant space could be saved in the construction zone. Estimating the transmitted force to the ground during the impact has also been possible, thus the foundations could be designed simultaneously.

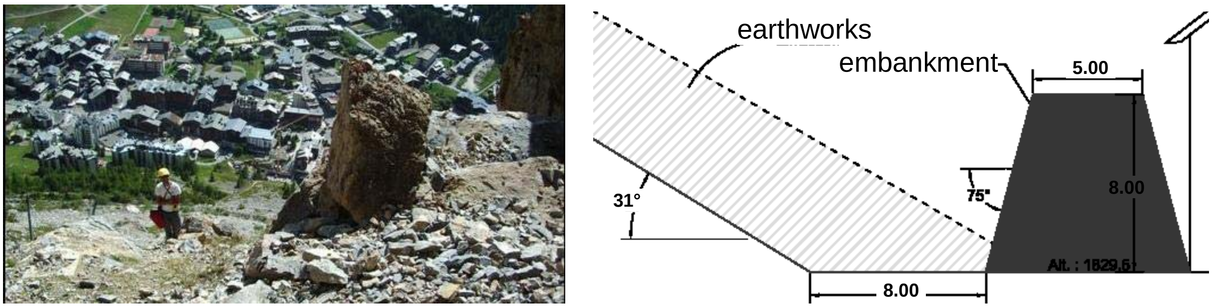


Fig. 4 Rockfalls hazards near the town of Val d’Isère (France) and numerical model of the embankment.

CONCLUSION

An efficient model of embankment impacted by rockfall has been developed. Based on the results obtained during simple and multi parametrical studies, the design methods currently in use could be revisited. Further investigations are needed to give more insights in the response of more heterogeneous embankments and the possible interaction with the different kind of inclusions (geotextile, gabions, tires, etc…).

REFERENCES

[1] DESCOEUDRES F. Aspects géomécaniques des instabilités de falaises rocheuses et des chutes de blocs, Publications de la Société Suisse de Mécanique des Sols et des Roches, Montreux, Suisse, 1997.

[2] PEILA D., OGGERI C., CASTIGLIA C., RECALCATI P., RIMOLDI P., “Testing and modelling geogrid reinforced soil embankments subject to high energy rock impacts”, Geosynthetics 7th ICG, Delmas, Gourc & Girard (eds.), 2002.

[3] DONZE, F. and S. A. MAGNIER, Formulation of a three-dimensional numerical model of brittle behavior, Geophys. J. Int., 122, 790-802. 1995.

[4] MONTANI S., Sollicitation dynamique de la couverture des galeries de protection lors de chutes de blocs, PhD Thesis, EPFL Lausanne, Suisse, 1998.

[5] PICHLER B., Parameteridentifikation als Grundlage für Prognosen im Grundbau, PhD Thesis, T.U. Wien, Austria, 2003.

[6] PLASSIARD J.-P., Modélisation discrete d’impacts de blocs rocheux sur structures de protection type merlons, PhD Thesis, Grenoble, France, 2007.

[7] SUBRIN D., “Modélistion analytique et numérique pseudo-statique des merlons de protection contre les chutes de blocs rocheux”, JNGG 2006, Lyon, 2006.

CAN THE DEMANDS OF ACTIONS OF A ROCKFALL SHED AND OF A SNOW SHED BE FULLFILLED WITHIN ONE SHED?

Katharina Platzer[1]

SUMMARY
Snow sheds as well as rockfall sheds are used in alpine regions to protect transportation routes from natural hazards. Due to different load assumptions of the two different processes the question is asked whether a shed can be build to protect roads from avalanches as well as from rockfall. Sheds must withstand both the static and dynamic forces from rockfall and avalanches. However, in this contribution the focus is given on the actions of snow sheds. Snow shed design is based in practice on simple hydrodynamic equations. The dynamic forces exerted by an avalanche are a function of flow velocities, flow height and slope deviation angle. Between 2002 and 2006 scale experiments using granular material on a laboratory chute as well as large chute experiments with snow on the Weissfluhjoch (Davos) were used to determine the relevant dynamic parameters of avalanches flowing over an inclined plane with a deviation. The performed experiments result in an improved approach to calculate the dynamic forces due to a deviation and in more detailed data on the coefficient of friction depending on the snow type. The new findings were implemented in the revision of the design guideline for snow sheds. The guideline describes the design concept, typical load cases and formulas to calculate the avalanche actions.

Keywords: snow shed, dynamic force, coefficient of friction, avalanche dynamics

INTRODUCTION
Rockfall sheds as well as snow sheds are used in alpine regions to protect transportation routes from natural hazards. In some places of mountainous regions a shed might be exposed to several natural hazards, as flowing snow avalanches during winter and spring seasons as well as to rockfall during alternating thawing-freezing periods or warm seasons. In both cases the sheds are conceived as massive structures that must withstand both the static and dynamic loads [2], [3]. The question arises whether it is possible to combine the actions of rockfall and avalanches in one "rockfall - avalanche" shed in order to account for both actions without getting out of the hand of costs.
In the case of snow sheds static loads arise from the snow cover and when avalanches, or other debris, come to rest on the roof of the shed. Dynamic loads arise from the normal and shear stresses exerted by dense avalanches as they pass over the roof. Engineering guidelines have been proposed ([1] and [2]) that define the magnitude of the static and dynamic design loads as a function of the avalanche return period and velocity [4]. Already in the 1960ies experiments on real snow sheds in the Swiss Alps have been performed [5]. However, only the maximum shear and normal forces of exerted on load cells one winter season could be measured. Maximum values of $\mu = 0.5$ were found. The problem with this measurement technique is that those measured forces can arise from several avalanche events. The mean μ value represents the average of the sum of all measured events from one winter, independent of the avalanche type. Therefore a new project has been set up to investigate the avalanche loads under more defined conditions. Recent experimental investigations with snow flows on

[1] Geo7, Geowissenschaftliches Büro, Neufeldstrasse 5-9, CH-3012 Bern, +41 31 300 44 21, katharina.platzer@geo7.ch

the Weissfluhjoch chute [4] suggest that the maximum value of $\mu = 0.5$ is a realistic value only for wet snow avalanches.
This contribution is focused on the actions on snow sheds, describing the important load cases. The point here is to highlight the most important structural characteristics of snow sheds. Thus the rockfall specialists have the possibility to compare the structural design to the rockfall processes.

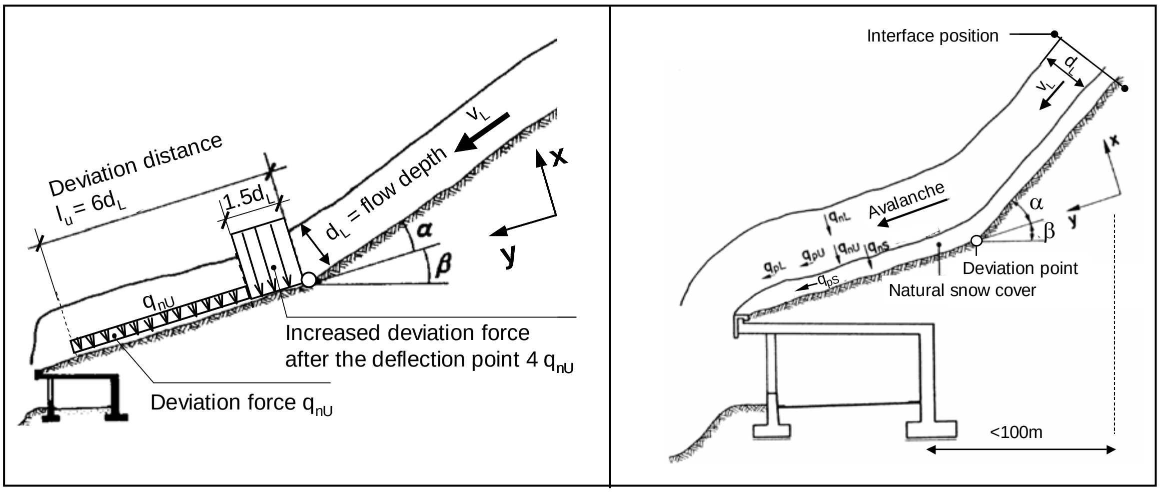


Figure 1: a) Distribution of the dynamic deviation force. b) Definitions of avalanche actions and the interface position

EXPERIMENTAL INVESTIGATION OF AVALANCHE LOADS

Between 2002 and 2006 scale experiments using granular material on a laboratory chute as well as large chute experiments with snow on the Weissfluhjoch (Davos) were used to determine the relevant dynamic parameters of flowing avalanches [6]. In the laboratory experiments were performed on a wooden chute [7]. On the Weissfluhjoch snow chute more than 50 experiments with 8 to 15 m^3 snow for each experiment were performed. Within both experimental setups a series of optical velocity sensors, ultrasonic flow height sensors and force plates - measuring the normal- and shear component of an avalanche over time - are used in combination to determine the characteristics of flowing avalanches when moving over a deflecting structure. The main findings of the experiments result in an improved approach to calculate the dynamic forces due to a deviation (Fig. 1a) and in more detailed data on the coefficient of friction depending on the snow type [2] and [3].

RESULTS

An important feature of the calculation guidelines is the prediction of the force distribution after a slope deviation. Snow sheds must often be built close to such terrain deviations (Fig. 1b) where hydrostatic normal forces can no longer be safely assumed. The Swiss guideline procedures add a "centrifugal" or excess pressure to the hydrostatic pressure [1] and then assume that this stress remains constant over the deviation length l_u, which is the distance from the slope deviation to the end of the snow shed (see Fig. 1a). The excess pressure is a function of the slope angle change, α (see Fig. 1b). To date there exists no experimental information, especially with snow flows, to suggest that this calculation procedure is correct. Many snow sheds, however, have withstood extreme avalanche events without damage leading us to believe the procedure could be over-conservative.
The main results from the chute experiments reveal that near the deviation point, meaning up to a distance of 1.5 times the flow height of an avalanche, the so far applied formula [1] underestimates the applied dynamic forces, whereas after a distance of more than 6 times the

flow height of an avalanche, the dynamic forces due to a deviation have vanished. The mean coefficient of friction μ is for dry snow avalanches 0.3 and for wet snow avalanches 0.5 [6]. The measured coefficient of friction is reduced by about 30% when the avalanche flows over a deposited snow cover (Figure 3).

LOAD CASES

The Swiss guideline distinguishes eight different load cases. All actions from avalanches like friction, normal loads and deviation loads are combined either leading action or accompanying action. The actions from avalanche deposits and sliding avalanches cannot be combined leading action. If snow sheds are loaded by avalanches and rockfall the two actions must not be combined.

CONCLUSION AND DISCUSSION

With the performed experiments on the laboratory chute as well as on the Weissfluhjoch chute it was possible to improve the calculation formula of the guidelines. Especially new findings on the snow densities, the coefficients of friction and the formula to calculate the deviation force could be introduced. The decrease of the dynamic force is of interest also for some cases of the rockfall sheds. The repeatability of the experiments was not perfect because of the varying types of snow. An important point that should be studied in more details in the future is the influence of a snow cover or of avalanche deposits on the damping of the dynamic avalanche loads. In this context it is interesting to discuss also the possible effects (change of geometry or loads) due to the presence of rock deposits on the avalanche sheds. On the other side snow deposits may have a damping effect on rockfall sheds, especially in spring when those processes are more frequent.

REFERENCES

[1] ASTRA/SBB 1994. Einwirkungen auf Lawinenschutzgalerien. Bundesamt für Strassenbau und Baudirektion GD SBB, Ausgabe 1994.
[2] ASTRA/SBB 2007. Einwirkungen auf Lawinenschutzgalerien. Bundesamt für Strassenbau und Baudirektion GD SBB, Ausgabe 2007.
[3] ASTRA/SBB (2008): Richtlinie Einwirkungen infolge Steinschlags auf Schutzgalerien. Bundesamt für Strassenbau und Baudirektion GD SBB, Ausgabe 2008.
[4] Platzer K., Bartelt P., Jaedicke C. (2007): Basal shear and normal stresses of dry and wet snow avalanches after a slope deviation. Cold Regions Science and Technology 49 (1), pp. 11-25.
[5] Salm, B. and Sommerhalder, E. 1964. Beanspruchung von Lawinengalerien durch Schnee. Strasse und Verkehr, Solothurn, Nr. 1/1964, pp.9-16.
[6] Platzer, K. and S. Margreth. 2007. Experimental investigation of avalanche forces acting on snow sheds. Bundesamt für Strassen, Bern, Bericht VSS Nr. 613. (in German)
[7] Platzer, K., Margreth, S., and P. Bartelt. 2004. Granular flow experiments to investigate dynamic avalanche forces for snow shed design. In Proceedings of the 5th International Conference on Snow Engineering V, Davos. Routledge, UK, 363–370.

EVALUATING THE LONG-TERM ROCKFALL PROTECTION EFFECT OF SILVICULTURAL STRATEGIES AT PROJECT LEVEL

Werner Rammer[1], Michael Brauner[2], Manfred J. Lexer[1]

SUMMARY

In this contribution we present the evaluation of alternative silvicultural strategies for a case study rockfall protection forest project in Austria using the coupled rockfall and forest ecosystem modelling platform PICUS. Over a period of 100 years the rockfall protection effect and the cost efficiency of management operations of a timber oriented and two protection forest management strategies were assessed. While the protection forest concepts based on unevenaged silvilculture performed well with regard to both protective and economic efficiency, the timber oriented age class forestry lacks in maintaining a high level of protection due to large scale juvenile stand development phases.

Keywords: PICUS, mountain forests, rockfall protection, project level, cost efficiency analysis.

INTRODUCTION

Alpine mountain forests frequently have, next to a site-protection function in which forest cover stabilizes the slope itself, an object-protection function, which means that forest cover has to protect objects and infrastructure located further down a slope from natural hazards such as avalanches and rockfall. In this contribution we focus on the management of rockfall protection forests. Integrated management plans for protection forests often combine biological means such as silvicultural interventions with technical protection measures such as rockfall nets or dams to optimize overall protection efficiency. The typical spatial scale of such integrated projects extends from a few to as much as hundred hectares including a variety of different sites and stand development phases. Designing silvicultural treatment plans for rockfall protection forests is a challenging task, as various stakeholders are involved. Different and often conflicting goals like economic revenues from timber production and rockfall protection effectiveness are to be considered in a rather complex spatio-temporal setting.
This contribution aims at the evaluation of alternative silvicultural strategies for a case study rockfall protection forest project of the Austrian Federal Service for Avalanche and Torrent Control using the coupled rockfall and forest ecosystem modelling platform PICUS.

[1] University of Natural Resources and Applied Life Sciences, Institute of Silviculture, Peter Jordanstr. 82, A-1190 Wien, (+43)1/47654-4053, werner.rammer@boku.ac.at

[2] University of Natural Resources and Applied Life Sciences, Institute of Mountain Risk Engineering, Peter Jordanstr. 82, A-1190 Wien, (+43)1/47654-4354, michael.brauner@boku.ac.at

METHODS AND MATERIAL

The PICUS simulation tool integrates (a) a 3-dimensional hybrid patch model [1] allowing the simulation of heterogeneous mountain forests including spatially explicit management interventions, and (b) the PICUS-Rockfall module [3] which simulates individual rockfall trajectories on project level (up to 100ha) in 3D on a digital terrain model with spatially distributed surface properties taking into account energy dissipation due to surface contact and single tree impacts. The forest model as well as the rockfall module have been tested extensively in earlier studies ([2],[4]).
The case study project comprises of 12 stands with a total area of 40 hectares and extends from appr. 800m to 1200m a.s.l. Downslope, a road as well as a village have to be protected against rockfall. Within an integrated protection forest project alternative silvicultural treatment plans for all stands for a period of up to 100 years were developed and simulated with PICUS. Alternatives included a “no management” (MS1) variant, a timber oriented treatment concept based on age class forestry and shelterwood regeneration (MS2) as well as two concepts particularly designed for protection forest management based on unevenaged silviculture and regeneration slots (MS3, MS4).
The DEM used for the rockfall simulation was acquired through airborne high resolution laser scanning, data on forest structure as well as geological mapping of rockfall source areas and surface properties was obtained through field measurements. The rockfall protection effect of the current and simulated future future forest was evaluated by estimating the probability of rocks of different sizes to reach the target and accompanying energies and jump heights of falling rocks as well as by mapping energy levels and run out distances. Protective effects as well as cost efficiency of the silvicultural concepts were evaluated by relating protection effect and opportunity cost from constrained timber harvesting.

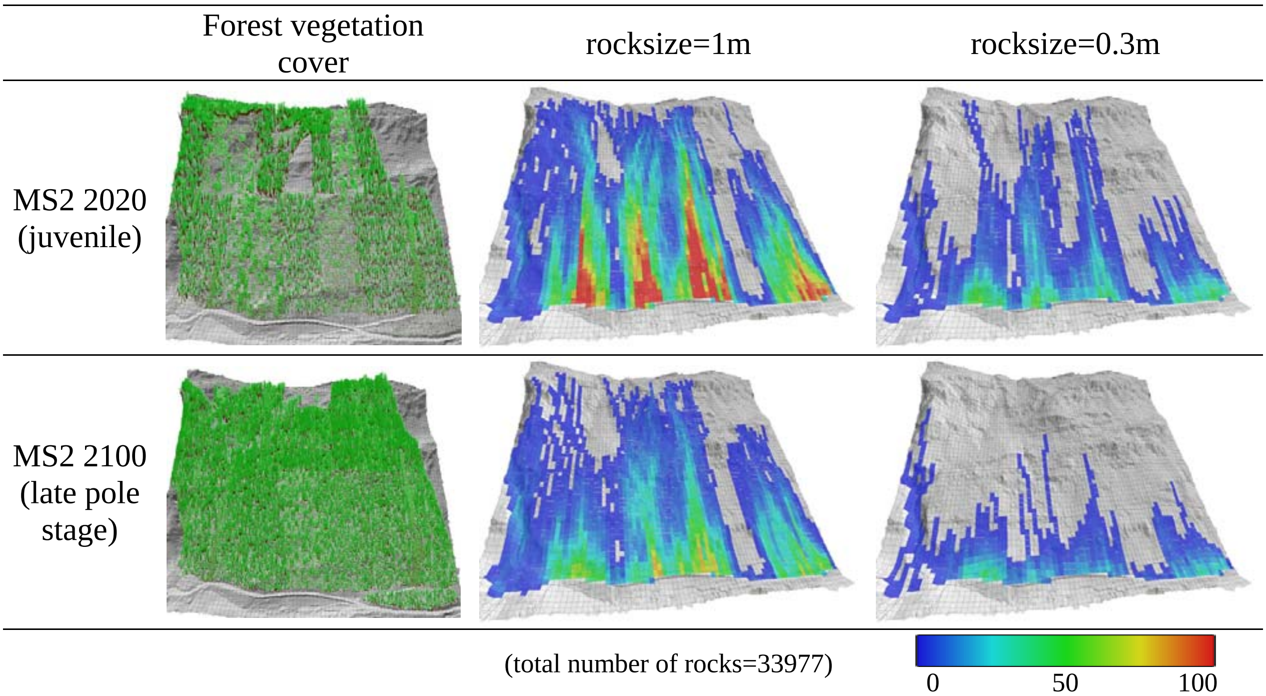


Fig 1. Visual representation of the effect of different vegetation cover on rockfall protection. The color indicates the number of rock trajectories trespassing a “patch” (10x10m cell) that subsequently reach the road (protection target at valley bottom). In 2020 of the timber oriented scenario large areas are in a juvenile stand phase (upper row) and perform poorly compared to 2100 (largely late pole stage, lower row). Larger rocks (1m, middle column) generally reach the road in a higher share than smaller rocks (0.3m, right column).

RESULTS

Each of the four management strategies yielded specific strengths and weaknesses. While the "no management" variant MS1 could sustain a relatively good protection effect particularly against larger rocks over several decades, it missed a timely regeneration and showed diminishing protective effect in later stages. The timber oriented concept MS2 concentrates on intensive harvest entries early in the planning period with subsequent favorable contribution margins, but fails to sustain balanced economic results in juvenile stand development phases. Additionally, the protective effect against larger rocks is reduced substantially over several decades due to large-scale juvenile stand development phases. The protection forest management concepts (MS3, MS4) turned out to perform surprisingly well with regard to economic efficiency while at the same time providing good protection against rocks of all sizes due to the simultaneous presence of large diameter timber as well as sufficient regeneration. In Fig. 1 the effect of different vegetation cover on rockfall protection is exemplified. The trade-offs between rockfall protection and timber production are presented for different temporal planning horizons.

CONCLUSIONS

The presented simulation approach integrates for the first time forest dynamics and rockfall simulation consistently within one simulation environment. The application of the tool is favored by the fact that the required high-resolution DEMs are becoming successively available for larger areas. Quantitative analysis of different goal aspects is possible which adds transparency to project planning. This is an inevitable advantage if stakeholders are actively involved in discussing management alternatives for rockfall protection forests. The tool provides potentially so much information that to prevent information overflow a targeted analysis approach is recommended.

REFERENCES

[1] Lexer, M.J., Hönninger, K., 2001. A modified 3D-patch model for spatially explicit simulation of vegetation composition in heterogeneous landscapes. Forest Ecology and Management 144 (2001), 43-65.
[2] Seidl R., Lexer M.J., Jäger D., Hönninger K. (2005). Evaluating the accuracy and generality of a hybrid patch model. Tree Physiology, 25, 939-951.
[3] Woltjer, M., Rammer, W., Brauner, M., Seidl, R., Mohren, G.M.J., Lexer, M.J., 2008. Coupling a 3D patch model and a rockfall module to assess rockfall protection in mountain forests. Journal of Environmental Management 87, 373-388.
[4] Rammer, W., Brauner, M., Dorren, L.K.A., Berger, F., Lexer, M.J., 2008. Validation of a 3D-coupled rockfall and forest model, in preparation.

EXCEPTIONAL ROCKFALL PROTECTION WORKS REUNION & ST HELENA

Michel Richard and Marc Velu[1]

INTRODUCTION

Because of increasing safety concerns, climate changes and the development of transport infrastructure, rockfall protection works are being extended worldwide. Two of the major projects of rockfall protection works with respect to their magnitude, are located in overseas territories, one being a French Department (Réunion) and the other a British Territory (St Helena). CAN has been extensively involved in the execution of both of these two projects, the first of which is now fully completed, while the second is currently underway.

I will introduce the risk analysis which has been carried out, the protection scheme which has been defined, and the special techniques and method statements which have been implemented for the safe execution of these two projects.

Both projects are using, on a large scale, draped rockfall containment netting, for cliffs mainly made of basaltic lava flows.

Keywords: draped netting, post supported, rockfall containment, protection, experience.

REUNION (INDIAN OCEAN)

Reunion is a French overseas territory with a population of approximately 800,000 persons. Fournaise is one of the world's most active volcanoes and is responsible for the creation of Reunion. For many years the coast motorway has repeatedly been subjected to massive rockfalls, some of them with heavy casualties. Commissioned in 1963 (2 lanes) and widened to 2 x 2 lanes in 1976, this motorway links the four major cities of the island: Saint Denis, head of the Department, the Harbour, heart of the economic activity with Saint Paul and Saint Pierre in the south. Over 80,000 vehicles per day use this road. Subject to severe climatic conditions (cyclones, heavy rains…) as well as geotechnical activity (volcanoes) numerous accidents occur on the motorway each year.

In 2006, 6 accidents occurred with 4 deaths and 11 injuries.

The local population is therefore very concerned with rockfall exposure, and at their request, the State and the local Province undertook to finance a large scale protection scheme, although other alternatives to the existing motorway route were also evaluated.

The cliff, which is in excess of 250m high in places, is made of several heterogeneous layers of basalt, tuffs and breccias. Over a 9 km section of the road, 6 kms of cliff have been protected. Three contracts were put out to tender, two of which were awarded to our joint venture. The budget for the JV was 24 million Euros (excluding supply of nets and taxes), for a contract period of 18 months.

Road traffic had to be maintained throughout the execution of the works, with the exception of Sundays for mass scaling operations.

The scope of work included:

[1]CAN SA, Le Relut, F 26270 Mirmande, +33 475 63 06 36 (fax -+33 475 63 10 10), mrichard@can.fr

- Scaling : 66 days (Sundays only)
- Controlled removal of large blocks: 5,000 m^3
- Installation of rockfall containment netting (wire mesh): 330,000 m^2
- Installation of reinforced rockfall containment netting (wire nets, all types): 290,000 m^2
- Installation of post supported nets: 3,500 m
- Catch fences (class 9): 300 m
- Over 27,000 m of anchors (Ø 28,32 & 40mm of length varying between 3 and 12m)

Due to the large volume of blocks with the potential to fail, post supported nets were considered preferable to catch barriers. Draped nets and post supported nets are particularly suitable given the height of the cliffs and the difficult access to the barriers for removing the blocks after major rockfall events. Such systems offer the advantage of possible net scouring at the base of the cliff with the combined use of gabions.


Fig. 1 Blasting


Fig. 2 Post supported nets


Fig. 3 Draped rockfall netting

Several types of wire nets were also installed as part of the works: wire nets (anti submarine nets), Rocco nets and cable netting, all of which are subject to minimum breaking load of 500,000 N.
Two Helicopters were also used throughout the works for lifting the nets and transport of personnel. Safety was a key concern during the works, both for car drivers, as traffic was allowed during the works, and for the rope access workers, whose work was continuously coordinated by a full time committed Safety Coordinator. No casualties were reported.
Results have matched expectations.
After commissioning, new events have been recorded and will improve the experience and increase the data base collected on these types of containment measures.

ST HELENA (SOUTH ATLANTIC OCEAN)

St Helena is located in the South Atlantic Ocean, 2000 km west of Namibia. The island has a population of approximately 4000 persons for an area of 120 km2. St Helena is one of the most remote territories in the world with no airport facility and no port.
The Islands sole access point is the Jamestown wharf which is situated beneath a 200 metre high cliff. As part of a programme of infrastructure development to improve access to St Helena, the SH Government has commissioned a rockfall protection scheme for the Jamestown Wharf area.
Given the isolation of the island, the risk analysis and the protection scheme proposed by the designer[2], the project has the objective of minimizing the works on the cliff and reducing the presence, in non secured areas, of all personnel.

[2]WA Fairhurst & Partners, 225 Bath Street, Glasgow G2 4GZ, UK, +44 141 204 88 00 - fax -0141 204 88 01

The main aspects of the works include:

- scaling of all rock faces: 72,000 m^2
- installation of rockfall containment netting (PVC coating): 76,600 m^2
- installation of reinforced rockfall containment netting (Spider nets): 36,900 m^2
- construction of rock catch fences.: 526m, of varying heights and capacities.
- rock reinforcement with dowels and tensioned rock bolts at discrete locations.
- reinstatement of a path across the slopes at mid elevation, which divides the two zones of works.

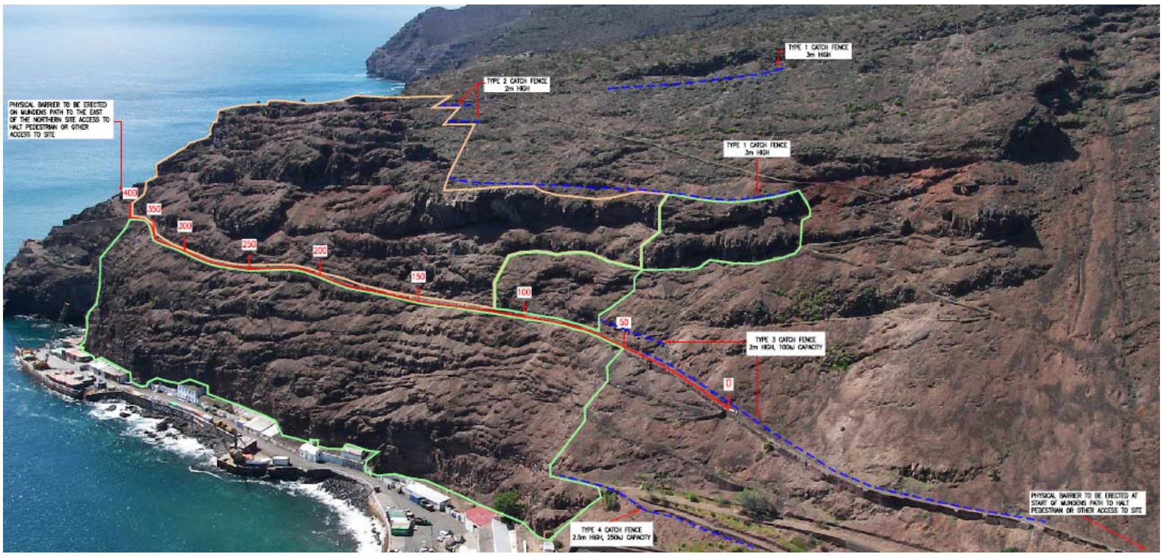


Fig. 4 Jamestown wharf rockfall protection scheme

Rockfall containment netting, will be anchored at the top and bottom without intermediate anchors in the cliff. The top section (above Mundens path) will be covered with a two layer system: a base layer of draped (hexagonal woven) netting with a second layer of reinforcement mesh (Spider S4).

The lower section (below Mundens path) will involve the installation of only draped netting. Due to the proximity of the marine environment, weather protection was considered an important factor. The protection scheme also includes 500m of catch fences, particularly at the boundaries of the protected zone.

An earlier design by another consultant was previously in place but included a large number of rockfall barriers. The design that has been adopted is considered preferable as it reduces the on-face works with respect to installation and on-going maintenance.

More details on design parameters and method statements can be provided on request.

Although the provision of a helicopter in St Helena is a first for the Island, experience obtained in Reunion makes this method of installation most suitable with respect to safety and the contract timeline.

Additionally, to solve a space problem on the wharf storage of materials on a chartered cargo vessel has also been utilised.

Works started in early January 2008 and will be finished by the end of October 2008.

CONCLUSION

The two projects described above reflect a trend for rockfall protection on volcanic terrain: draped rockfall containment is widely used. Modelling and simulation parameters will be explained with more detail. Performance assessments will also be justified to introduce new visions and ideas for future rockfall containment projects. Designers and contractors on these projects are ready to share their experience with others.

A DYNAMIC DESIGN METHOD FOR ROCK FALL PROTECTION GALLERIES

Kristian Schellenberg[1], Thomas Vogel[2]

Galleries are a wide spread measure to protect roads or railways from falling rocks. In comparison to other protection measures, the erection costs of galleries are relatively high. They can be justified since rock fall galleries protect against high energy events and at the same time they do not cause expensive maintenance after a middle intensity event. In particular, galleries can be appropriated where the endangered road section is relatively short and the risk is well defined.

A survey on Swiss rock fall galleries has shown that most of the existing galleries consist of reinforced concrete slabs and are covered by a cushion layer. In general, the design of structures to withstand impact loads requires a strong understanding of the loading characteristics and of the structural response as well. Since a detailed evaluation is too time consuming for practical engineers to be cost effective, a static design using an equivalent load became the normal approach in many countries. The difficult question is how to choose the static equivalent load. In Switzerland as well as in Japan guidelines have been elaborated, where the static equivalent load is mainly defined by the properties of the cushion layer [1], [2].

The static approach leads to relatively stiff structures. Especially since consulting engineers select the slab section in a way, that it can be built without shear reinforcement. But the stiffer the structure is, the higher will result the impact loads. Additionally high impact forces combined with the higher masses will produces higher local shear stresses that can lead to a punching failure in the early state of the impact.

This contribution proposes a simplified performance based design method that takes into account the dynamic response of the structure.

Keywords: rock fall protection, reinforced concrete slabs, impact load

ANALYTICAL MODELING WITH SYSTEM OF THREE DEGREES OF FREEDOM

A system of multiple degrees of freedom (SMDF) is proposed for the design of rock fall protection galleries. The SMDF is shown in Fig. 1 and consists of three masses and three corresponding springs, similar to the models presented for the analysis of aircraft crashes into reactor containments by researchers around Eibl more than 20 years ago [3]. For the application to rock fall impacts on galleries, the different masses and spring properties have to be adequately chosen. Additionally the properties of the cushion layer have to be implemented.

[1] ETH Zurich, Institute of Structural Engineering, Wolfgang-Pauli-Str. 15, CH 8093 Zurich,
+41 44 633 30 80, schellenberg@ibk.baug.ethz.ch
[2] Prof., ETH Zurich, Institute of Structural Engineering, Wolfgang-Pauli-Str. 15, CH 8093 Zurich,
+41 44 633 31 34, vogel@ibk.baug.ethz.ch

The three masses of the SMDF are

(i) the mass of the impacting rock M_1,
(ii) the mass of an assumed punching cone M_2 and
(iii) the effective oscillating mass of the rest of the structure M_3^*, which governs the global response of the structure.

The response of the system is determined by the three nonlinear springs K_1, K_2 and K_3 (Fig. 2). K_1 describes the properties of the cushion layer, K_2 describes the shear behavior of the assumed critical section and K_3 describes the bending stiffness of the global system, respectively. Both, the masses and the stiffness of every spring are influenced by the boundary conditions and by the section's geometries.

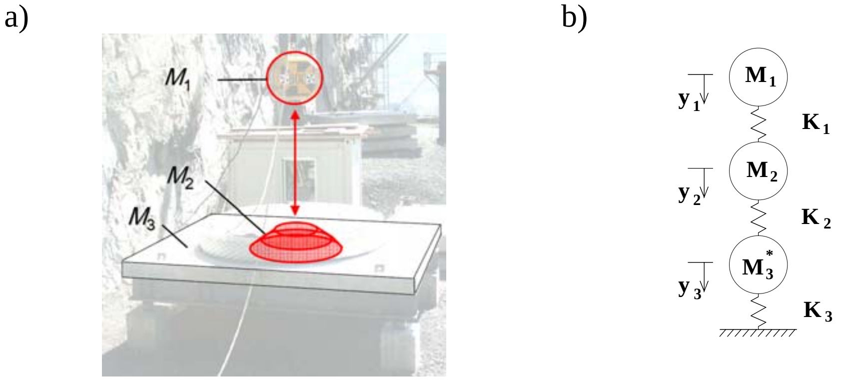


Fig. 1 System of multiple degrees of freedom a) for hard impacts proposed by Eibl b) for rock fall galleries

For the values of the spring properties physical models are used. However, simplifications lead to an effective solving of the equation of motion. The values of the springs can consider strain rate effects, which are relevant regarding resistance against punching failure in the early state of the impact.

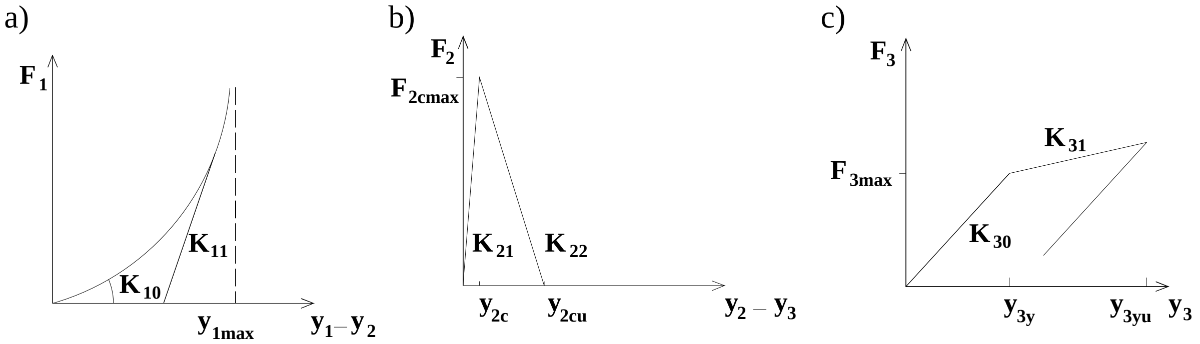


Fig. 2 Spring properties for a) K_1 - cushion layer, b) K_2 - punching behavior and c) K_3 - global behavior

EVALUATION WITH EXPERIMENTAL AND NUMERICAL RESULTS

In large-scale tests the SMDF could be verified. Reinforced concrete slabs of 3.5 x 4.5 m were subjected to rock fall impacts [4]. The falling height was increased after every impact until failure of the slab was reached. During the test series with totally 38 impacts on six slabs, the influence of several parameters was tested:

- slab thickness
- reinforcement layout
- cushion layer
- striking masses

The slabs were supported by one linear and two punctual supports corresponding to the back wall and the columns of the galleries. In the tests it could be observed, how the crossover from the elastic to the plastic range of the global response led to a decrease of the maximum total reaction force. The measured reaction forces are compared with the forces obtained in spring K_3 of the SMDF. Measured strains during the impacts allow for evaluating the dynamic increase of the material strength for typical impact velocities.

In addition, the experimental results could be obtained by means of a finite element model created in Japan [5]. Thus in further investigations, finite element simulations will be used to evaluate the parameters of the SMDF for more complex cases.

DESIGN CONCEPT

The proposed design method is an iterative procedure. Preliminary dimensions have to be selected to calculate the input parameters. For the spring properties characteristic values have to be used, since an underestimation of the yielding stresses, for instance, predicts an optimistic performance of the structure.

The proposed analytical model can easily be solved by a calculation sheet or using a simple finite element simulation with three beam elements only. The output is the force-time-history for every spring K_1, K_2 and K_3. However, the maximum load resulting from the SMDF can not be taken as design load for a static design, since the resulting section would have a different dynamic response. Instead accepted plastic deformations of the global behavior can be used as design criterion. Punching failure has also to be avoided. Structural safety can be formulated by the ratio of the forces K_2 or K_3 to the local shear or the overall bending capacity of the slab, respectively.

Once recommendations for the assumptions of masses and spring properties are elaborated for different boundary conditions as well as for typical geometrical proportions, the SMDF will be a simple and powerful design tool for a performance based design procedure.

REFERENCES

[1] ASTRA, SBB (1998). Einwirkungen auf Steinschlagschutzgalerien, Richtlinie, Bundesamt für Strassen, Baudirektion SBB, Bern.
[2] Japan Road Association (1983). Manual for anti-impact structures against falling rocks (in Japanese).
[3] Schlütter, F.-H. (1987). Dicke Stahlbetonplatten unter stossartiger Belastungen – Flugzeugabsturz – , Dissertation, Institut für Massivbau und Baustofftechnologie, Hrsg.: Prof. Dr.-Ing. J. Eibl, Prof. Dr.-Ing. H. K. Hilsdorf.
[4] Schellenberg, K., Volkwein, A., Roth, A., Vogel, T. (2007). Large-scale impact tests on rock fall galleries, 7th International Conference on Shock & Impact Loads on Structures, 17 –19 October 2007, Beijing, pp.497-504.
[5] Schellenberg, K., Kishi, N., Volkwein, A., Vogel, T. (2008). Cross-evaluation of models for impact on rock fall galleries; publication in preparation.

FALLING WEIGHT TESTS ON ROCK FALL GALLERIES WITH CUSHION LAYERS

Kristian Schellenberg[1], Axel Volkwein[2], Matthias Denk[3], Thomas Vogel[1]

Impact tests on six reinforced concrete slabs with different cushion layers were performed by dropping concrete boulders from different falling heights. A conventional cushion layer and special cushion systems consisting of high-tensile steel wire meshes filled by cellular glass were tested. The tests reveal the measured reaction forces at the supports, the accelerations in the boulder and in the slab as well as the strains at the upper slab surface and in the bending reinforcement. In addition, a high speed video system recorded the impacts.

Keywords: rock fall impact, large-scale falling weight tests, cushion system, cellular glass, reinforced concrete slabs

INTRODUCTION

The impact load capacity of existing rock fall protection galleries is of great interest when deciding on the necessity of renovation or strengthening. The Swiss design guideline for rock fall galleries was published in 1998 [1]. Older galleries are mostly designed based on over-simplifications by local engineers. The guideline is based on impact tests carried out in 1996 [2] that focused on the influence of the cushion layer. The test results were extrapolated by using finite element simulations [3]. Further research was performed on the dissipation capacity of different cushion materials [4]. The response of the structure and the interaction between the impacting rock, the cushion layer and the reinforced concrete slab are the main focus of the presented study. Therefore, large-scale field tests on reinforced concrete slabs are performed in an old quarry close to Walenstadt in the Swiss Alps.
A survey of the existing galleries in Switzerland has shown that most galleries consist of reinforced concrete slabs covered with a cushion layer [6]. Normally, granular soil from the surroundings or gravel is used as cushion layer. Protection galleries typically span 9 m with a slab thickness of approximately 0.70 m. The back side of the galleries is clamed supported at the retaining wall; the valley side is supported on columns. Typical column spacing is 7 meters.

TEST SETUP

The slab's dimensions of 3.5 x 4.5 m correspond to an average rock fall protection gallery in a scale of 1:2. They were line supported along one side and pile supported on the remaining two corners (see Figure 1). The slabs are covered by a conventional or by a special cushion system consisting of high-tensile steel wire mesh filled with a layer of light-weight cellular glass (Misapor). Two instrumented boulders of 800 and 4000 kg were used as falling weights to

[1] ETH Zurich, Institute of Structural Engineering, 8093 Zurich, Switzerland, +41 44 633 30 80, schellenberg@ibk.baug.ethz.ch
[2] WSL Swiss Federal Institute for Landscape, Snow and Forest Research, Zuercherstr. 111, CH-8903 Birmensdorf, +41 44 7392-962 (fax: -215), axel.volkwein@wsl.ch
[3] Geobrugg AG, Hofstr. 55, CH-8590 Romanshorn, +41 71 46681-96, matthias.denk@geobrugg.com

impact the slabs. The tests have been performed by gradually increasing the falling height from 2 to 15 m until plastic strains in the bending reinforcement reached a certain level or shear failure occurred. The kinematics of the impacting bodies was analyzed and the dynamic response of the reinforced concrete slab was investigated by measuring reaction forces strains and accelerations with sample rates of 3.2. The tests are recorded by a digital video camera with a recording rate of 250 frames per second with a posterior analysis of the trajectory using tracking software. The camera and the instrumentation are triggered manually.
A repetition of some identical tests staying in the elastic range of the slabs showed that the statistical spread of the test results stays small. The high-energy impact tests were carried out only once. Altogether 38 impacts were executed. The test setup and test program are described more detailed in [5].

Figure 1: Test setup with gravel cushion and 4000 kg boulder

RESULTS AND OBSERVATIONS

It was observed, that after the tests the cushion layer was less compacted than before. The separation of cushion layer and slab was clearly observable with the high-speed video recording. The stopping process of the boulder was different for the gravel cushion or for the system out of cellular glass.
In difference to real rock fall galleries, the slabs are not restrained from lifting off the supports. This was also observed in the high-speed videos.
During the latest phase of the slab response, slab, cushion system and boulder are in free oscillation. From the oscillation period, the stiffness of the slab can be deduced.
The failure mode that could be observed in all slabs was a combined bending shear failure close to the simple supported corner. According to the design of the slabs, a bending failure along the middle of the slab was expected. Punching resistance of the slab was close to the bending resistance. For the structural analysis and the design of the structure, the supposed

failure mode plays an important role [7]. The structure's response is also important for an adequate assumption of the dynamic material characteristics.

CONCLUSIONS AND OUTLOOK

Large-scale tests have been presented that simulate the impact of a falling rock onto a rock fall protection gallery. With the special system reaction forces at the supports could be reduced substantially.
From comparing the independent measurements on the slab and on the boulder as well as the recorded videos, it could be shown that the test setup produces reliable results. The obtained data using a sampling rate of 3.2 kHz is sufficiently detailed for the range of impact velocities concerning rock fall impacts. The data allows for an extensive analysis that describes the rock impact, the behavior of the cushion system and the interaction between impacting boulder and concrete slab.
The test results are used to develop and evaluate physical and numerical models. The models will lead to a design concept for rock fall galleries. Progress in the prediction of the performance of protective structures is one element, which has to be combined with the progress in the other disciplines in rock fall studies. These are for example the detachment of blocks from cliffs, trajectory analysis and geotechnical studies of the cushion layer. Collaboration between the different researchers will improve the handling of rock fall problems. Thus Teamwork will improve to mitigate the damage of infrastructure or humans lives due to rock fall.

ACKNOWLEDGEMENTS
The authors wish to express their gratitude to all involved persons, since these large-scale tests have only been possible to perform thanks to their help. Special thanks are also addressed to the highway administrations of the cantons of Grison and Uri for their indispensable financial support.

REFERENCES

[1] ASTRA, SBB (1998). Einwirkungen auf Steinschlagschutzgalerien, Richtlinie, Bundesamt für Strassen, Baudirektion SBB, Eidgenössische Drucksachen- und Materialzentrale, Bern.
[2] Montani, S. (1998). Sollicitation dynamique de la couverture des galeries de protection lors de chutes de blocs, Dissertation, No. 1899, École Polytechnique Fédérale de Lausanne, 190 pp.
[3] Bucher, K. (1997). Dynamische Berechnung von Steinschlageinwirkungen, Proceedings, Schweizerische Gesellschaft für Boden und Felsmechanik, Conference Paper Montreux,
[4] Chikatamarla, R. (2005). Rockfalls on slopes and structures, Dissertation, No. 16315, Swiss Federal Institute of Technology (ETH), Zurich.
[5] Schellenberg, K.; Volkwein, A.; Roth, A.; Vogel, T. (2007) Large-scale impact tests on rock fall galleries. In: Huang, F.L.; Li, Q.M.; Lok, T.S. (eds) Proceedings of the 7th International Conference on Shock and Impact Loads on Structures. 17-19 October 2007. Beijing, China. 497-504.
[6] Schellenberg, K., Vogel, T. (2005). Swiss Rockfall Galleries - Impact Load, Proceedings IABSE Symposium Lisbon 2005, 'Structures and Extreme Events', IABSE Zurich, 2005, pp. 302/303 and CD-ROM file LIS099.PDF, pp. 1-8.
[7] Schellenberg, K., Vogel, T. (2007). Tests and analytical model of rockfall impacts on galleries, Proceedings of Protect2007, Structures under Extreme Loading, Aug. 20-22, Whistler, p. 27 and CD-ROM file SWO04_Schellenberg.pdf, pp. 1-10.

ROCK FALL ASSESSMENT CHAPMANS PEAK DRIVE, CAPE TOWN, SOUTH AFRICA

Paul Schlotfeldt[1]

GIS was used as one of the primary analysis tools during a rock fall hazard assessment and analyses of Chapmans Peak Drive, south of Cape Town. Key methods used included 1) the development of a digital elevation model (DEM) used to delineate rock fall runout Zones; 2) rating Zones using an adapted version of the Oregon Rock Fall Hazard Rating System (RHRS) in order to determine the relative vulnerability of individual Zones to rock fall; 3) undertaking rock trajectory analyses to determine the distribution of kinetic energy for identified rock fall runout Zones; and 4) undertaking event tree analyses based on the synthesis of all data in order to establish Zones with the highest risk of fatalities. The results of this work shows that the vulnerability and risk varies considerably between Zones.

Keywords: GIS, digital elevation model (DEM), rock fall runout zone, trajectory analyses, event tree analysis.

INTRODUCTION

Chapmans Peak Drive, located between Hout Bay and Noordhoek, on the Atlantic Ocean side of the Cape Peninsula, near Cape Town, South Africa. Much of the Drive hugs the near vertical mountainside of Chapmans Peak. At least 10 people have been killed and many more injured as a result of rock falls along parts of the drive, with the majority of the deaths or injuries having occurred within the last ten years.

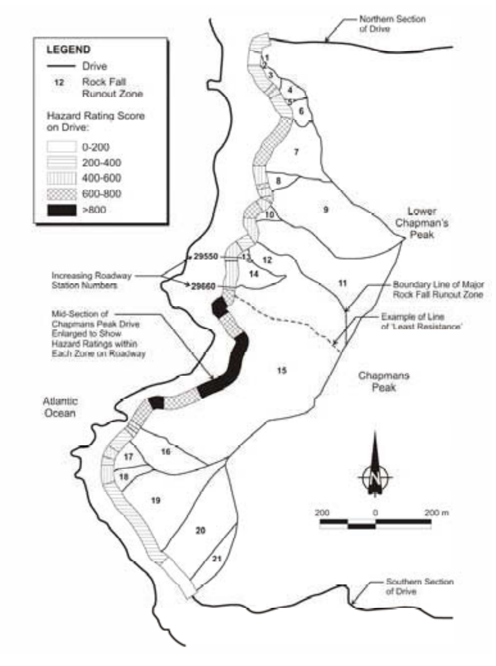


Fig. 1 DEM of Chapmans Peak showing rockfall runout Zones and hazard ratings for each Zone

BACKGROUND INFORMATION

From a rock fall hazard perspective the Drive was divided into three distinct sections (Figure 1). The mid-section was considered to be the most hazardous section of the Drive and this paper deals only with this part of the Drive (Figure 1).

The mountainous terrain on the Drive is typically complex and extremely steep. The cliff bands are frequently near vertical to overhanging and are up to 150 m high in places. Unstable blocks and wedges, some of which were car-sized or bigger, were observed to be present on most of the cliff bands.

The cliff bands are typically interspersed with boulder strewn scree slopes with slope angles of between 35^o and 55^o.

As a result of the fires, a significant percentage of the scree slopes had all binding vegetation obliterated. This exposed boulders of all sizes and shapes, frequently perched at critical angles of repose, significantly increasing the potential hazard from these slope facets.

1 Golder Associates Ltd. PO Box 2059, Squamish, BC, Canada 011-1-(604) 815-0768; pschlotfeldt@golder.com

DIGITAL ELEVATION MODEL

Before any mitigation measures could be considered, it was imperative that the spatial distribution and magnitude of the problem be understood. Initial steps entailed setting up a digital elevation model (DEM) using ArcInfo GIS. The DEM model was then used to delineate rock fall runout Zones. A rock fall runout Zone is defined as an area above the road where falling rocks are channeled into gullies and/or are contained between topographic features. The predicted boundaries of major rockfall runout Zones for the Drive are shown schematically in Figure 2.

ROCK FALL HAZARD ASSESSMENT

In order to understand key factors influencing the rock fall hazard, it was necessary to undertake some form of hazard assessment that could be used to assess the relative hazard level at each rock fall runout Zone identified using the DEM. This ensured that Zones could be ranked in order of the potential danger they presented to the highway user.

The Oregon State Highway Division's Rock Fall Hazard Rating System (RHRS) [1] was initially considered for use. The system provides a method of making informed decisions on where and how to spend construction funds. For a number of reasons, which are discussed in [2] it became necessary to adapt the RHRS system specifically for local conditions on Chapmans Peak Drive. It is assumed that the reader is familiar with / or will refer to references [1] and [2] when reading this section, since space does not permit a more detailed treatment. More details on this hazard rating system and results are published elewhere [2].

ROCKFALL TRAJECTORY ANALYSES

The path that falling rocks can potentially take, the associated kinetic energy, and the bounce heights and velocities are very important factors that influence the risk to road users, particularly when designing mitigation measures. In order to understand the potential magnitude of the rock fall problem, rockfall trajectory analyses were undertaken on all Zones shown in Figure 1 using a computer simulation program called Rocfall [3].

Method of assessment and Results of Trajectory Analyses

Typical cross-sections were generated along assumed potential trajectory paths, i.e. one for each Zone and two in Zone 15. Figure 2 shows one of the profiles generated for Zone 15 and the simulated rock fall trajectories.

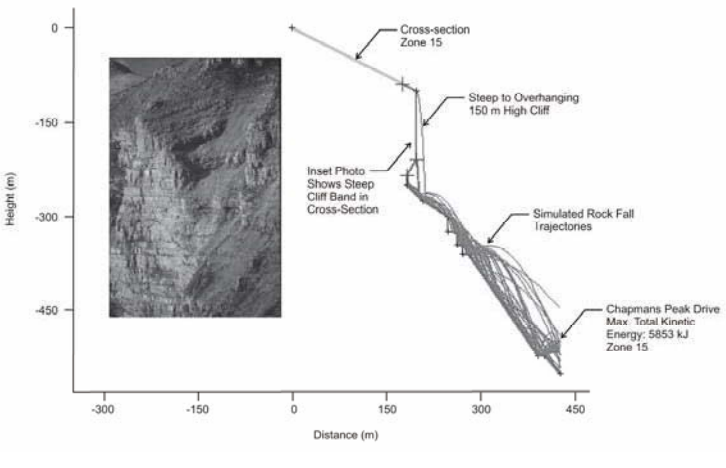


Fig. 2 Zone 15 rockfall simulation trajectory paths (shows the upper cliff band evident in the cross-section).

Values for slope friction angles and coefficients of normal and tangential restitution were typically selected as default values provided within the program. Subsequent work on another project using the Rocfall program indicates that it is possible to calibrate rock fall trajectory models in order to accommodate the observed field conditions. The result of this type of modeling indicates that the back analysis-derived slope parameter provide a high level of confidence that the number of rocks reaching the site is realistic.

RISK EVALUATION; EVENT TREE ANALYSIS AND THE RESULTANT RISK

Probability analyses were undertaken for each Zone and the risk or fatalities due to rockfalls are summarised for selected Zones in Figure 3.

Published Guidelines for Tolerable Risk

Figure 3 shows curves of annual probability of occurrence with increasing number of fatalities. The bold lines summarize published and proposed guidelines for tolerable risk in Canada / Hong Kong (shown as Hong Kong line in Figure 3), Netherlands, and UK respectively [4].

Event tree analyses were undertaken for all Zones on the Drive. For clarity, the results from only a select number of Zones are shown in Figure 3.

CONCLUDING REMARKS

The result of the study outlined above provided a powerful starting point to developing conceptualized remedial measures, producing preliminary designs, and developing schedule of quantities and cost estimates for individual Zones on the Drive. These data and designs formed the core of the design, build, finance, operate bid.

Solutions range from reinforced concrete and fill covered roadway sections, to half tunnels in rock, to multiple deflector rock fall catch fences (acting to deflect and channel falling rocks and to reduce energy), to a single catch fences (with a range of capacities and heights), to a single cantilevered catch fence (anchored on vertical rock cuts above the Drive).

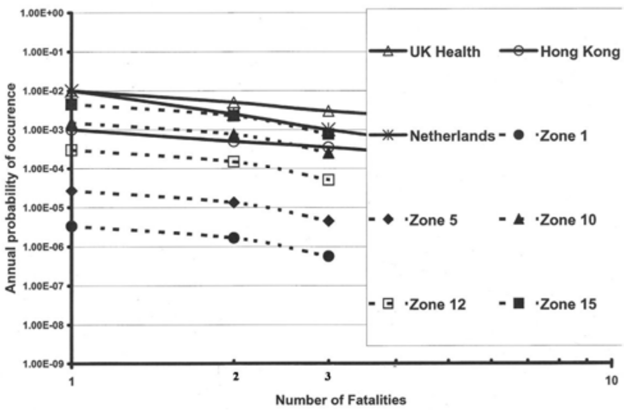


Fig. 3 Comparison between risks of fatalities due to rockfall for Zones with published acceptable risk criteria

REFERENCES

[1] PIERSON, L.A., Davis, S.A. and Van Vickle, R. (1990). *Oregon State Highway Rockfall Hazard Rating System Implementation Manual*. Federal Highway Administration (FHWA) Report FHWA-OR-EG-90-01. FHWA, U.S. Department of Transportation.

[2] SCHLOTFELDT, P (2007) Rock Mechanics: Meeting Society's Challenges and Demands - Eberhardt, Stead & Morrison (eds), pp 887-894

[3] RocFall, (2001) Instruction manual for RocFall program, Rocscience Inc. Geomechanics Software and Research Institute, University of Toronto.

[4] HOEK, E. (2000) Rock Engineering. Chapter 9, p. 115-136. Available on www.rocscience.com.

CHARACTERIZATION OF DESIGN IMPACT LOADS FOR ROCK-FALL PROTECTION

Matthias Schubert[1], Michael Havbro Faber[2]

This paper describes a methodology for the assessment of design loads for rock-fall protection structures taking explicitly into account different types of uncertainties. The advantage of this methodology is that any kind of information and models can be utilized and included in this framework. It enables a consistent description for the design load and can also directly be used for purposes of structural reliability analysis and probabilistic risk assessments.

Keywords: Rock-fall design load, probabilistic modeling, rock-fall frequency

INTRODUCTION

Rock-fall hazards are one of the most frequently occurring natural hazards in Switzerland. Considerable numbers of different kinds of protection structures aiming to protect against rock-fall hazards have been built in Switzerland during the last decades. The main purpose of these structures is to protect inhabited areas as well as transportation networks and other infrastructure lifelines; the protection structures are thus of high significance for the Swiss society. However, protection structures are costly in design and maintenance and the significant number of these corresponds to a substantial societal investment. The safety they provide should thus be in balance with the costs they are associated with. For this reason it is crucial that the design of these structures is made on a consistent and rational basis.
Rock-fall events have highly site specific characteristics and the prediction of rock-fall events underlies manifold uncertainties of different origins. To guaranty an acceptable level of safety and to develop a rational decision basis for the strategic planning of investments into protection structures it is necessary that adequate decision criteria are developed. One step in this direction is the development of a methodology for the modeling of design loads due to rock-falls including all uncertainties affecting the detachment of the rock and the falling process. Typical engineering problems such as design are decision problems subject to a combination of inherent, modeling and statistical uncertainties. As the assessment of risks due to natural hazards like rock-fall events and the assessment of the statistical characteristics of the parameters governing loads on structures due to rock-fall events in general serve as a means for establishing an improved decision basis for the planning of infrastructure and protection structures it is of tremendous importance that the uncertainties are consistently considered in the analyses.
It is commonly accepted that uncertainties should be interpreted and differentiated in regard to their type and origin; see e.g. [1], [2]. In this way it has become standard to differentiate between uncertainties due to inherent natural variability, model uncertainty and statistical uncertainty. Whereas the first mentioned type of uncertainty is often denoted aleatory uncertainty, the two latter are referred to as epistemic uncertainties, however, disregarding their classification, all uncertainties should be accounted for in the analyses. In the following it is illustrated how these uncertainties might be included in the analysis.

[1] Swiss Federal Institute of Technology Zurich ETH, Wolfgang Pauli Strasse 15, CH-8093 Zurich, +41 44 633 6109, schubert@ibk.baug.ethz.ch
[2] Swiss Federal Institute of Technology Zurich ETH, Wolfgang Pauli Strasse 15, CH-8093 Zurich, +41 44 633 3117, faber@ibk.baug.ethz.ch

METHODOLOGY

The stochastic nature of the triggering mechanism of rock-fall events is in general described by their frequency. Sometimes indicators for the description of the detachment process are available. These indicators might be small movements in the rock, changes in crack geometries, rain fall, etc. In most cases no information about such indicators is available and the frequency has to be described empirically. The main parameter for the modeling of design loads for rock-falls is the mass or the volume of the detached rocks and the detachment area (see e.g. [3]). The mass of the rock can be represented by a random variable X . Typically, it is described by its annual exceedance frequency $H_X(x)$ [4]. The exceedance frequency can be interpreted as the annual number of rock-fall events involving volumes larger than a specific value x (see Figure 1 (left)).

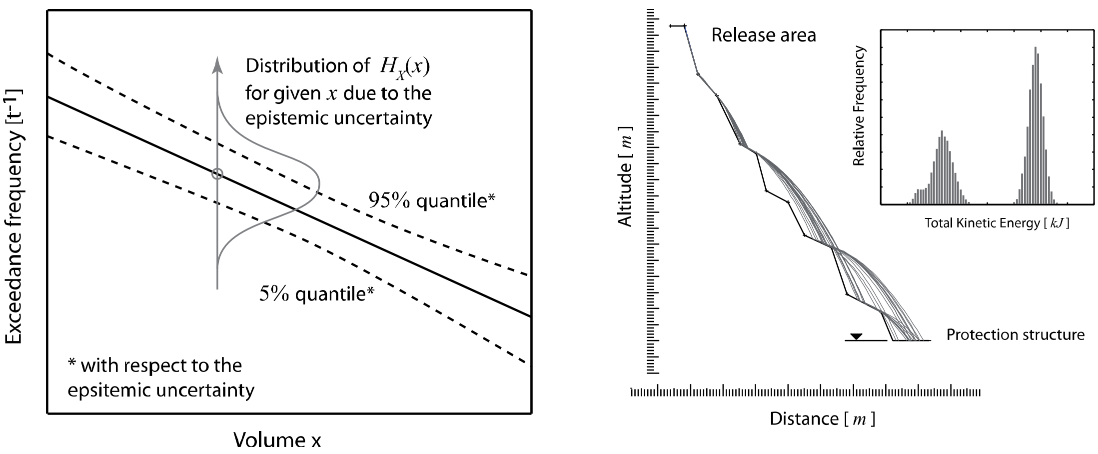


Fig. 1 Illustration of the annual exceedance frequency for rock-fall volumes (left). Illustration of the result of a trajectory simulation for a specific volume (right).

The modeling of the exceedance frequency is in general based on an insufficient data basis. Therefore, not only the detachment process itself is subject to large uncertainties but also the model describing the process. Figure 1 (left) illustrates the exeedance frequency as a function of volume. The solid line is equal to the expected value of the exeedance frequency for the different volumes and it represents the inherent uncertainties in the detachment process. This process may be considered to be an outcome of inherent natural variability the uncertainty of which cannot directly be reduced through observations and thus, the uncertainty can be considered as an aleatoric. The uncertainty associated with the model is represented by the dotted lines. The epistemic uncertainty needs to be quantified and included in the analysis even though in many applications this is not done. This might lead to a underestimation of the hazard (see e.g. [5]).

Once a rock-fall event of a given volume occurs, the trajectory of the falling rock is mainly determined by the topography, its mode of motion and the characteristics of the surfaces of the rock and the ground. All these factors contribute to the uncertainty in the prediction of the trajectory. Existing numerical tools facilitate the inclusion of this uncertainty by means of crude Monte-Carlo-Simulation, see [6] for an overview. Different choices of analyses are available including 2- or 3-dimensional models with different options for the the physical representation of the falling rock. Having selected a model the impact of the rock with the ground and other obstacles may be simulated [7]. To represent the impact of a falling rock on the ground constitutes one of the most complex parts of the modelling and is in general associated with large uncertainty. In general the analysis tools do not account for the variability in the ground material (particularly in zones covered with vegetation) and the local geometry of the ground and the rock. As a result of this the models will be imprecise e.g. associated with an epistemic uncertainty. In addition, due to the limited data basis utilized for the estimation of the model parameters these are associated with epistemic statistical uncertainty. Additional epistemic uncertainty due to a simplified modelling of the slope profile at the impact location may also influence the results.

The result of simulations using a two-dimensional trajectory model for a specific location is shown in Figure 1 (right). All the above mentioned uncertainties are reflected in the results by the relative frequency distribution shown in Figure 1 (right).

By combining the frequency model with models of the falling process, the joint probability distribution of the velocity (or energy) and the mass of a stone over the impact region can be determined. This distribution can be directly used to characterize a design load or to perform structural reliability analysis for design of new or for assessment of existing protection structures. Finally, by utilizing the above described probabilistic models for the rock-fall events in the context of structural reliability analysis and by invoking models for societal preferences into life saving investments rational decisions concerning the identification of optimal target reliabilities for protection structures may be established (see e.g. [8]).

CONCLUSION

A general methodology for the assessment of design loads due to rock-falls is outlined. This methodology facilitates the consideration of different types of uncertainties in the assessment of design loads for protection structures. The proposed model framework facilitates for the reduction of uncertainty through reliability updating based on observed rock-fall events. The generality of the approach also facilitates the exchange of parts of the model, e.g. the rock-fall simulation model, if more precise models are available; with the benefit that such improvements may lead to reduced model uncertainties and thus a basis for a more optimal identification of design parameters.

Using the joint probability distribution of the load, represented by the impact velocity of the stone and its mass, protection structures can be designed according to the recent codes and standards. It enables also to perform detailed structural reliability analysis, consistent risk assessments and the identification of optimal target reliability levels for the design and assessment of protection structures.

REFERENCES

[1] Apostolakis, G.,(1990) *The concept of probability in safety assessments of technological systems.* Science. **250**(4986): pp. 1359-1364.

[2] Faber, M.H., Maes, M.A.,(2005) *Epistemic Uncertainties and System Choice in Decision Making*. in *Proceedings ICOSSAR2005, 9th International Conference on Structural Safety and Reliability*. Rome, Italy.

[3] Hungr, O., Evans, S.G., Hazzard, J.,(1999) *Magnitude and frequency of rock falls and rock slides along the main transportation corridors of southwestern British Columbia.* Canadian Geotechnical Journal. **36**: pp. 224–238.

[4] Straub, D., Schubert, M.,(2008) *Modeling and managing uncertainties in rock-fall hazards.* Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards. **2**(1): pp. 1 - 15.

[5] Coles, S., Pericchi, L.R., Sisson, S.,(2003) *A fully probabilistic approach to extreme rainfall modeling.* Journal of Hydrology. **273**(1-4): pp. 35-50.

[6] Guzzetti, F., Crosta, G., Detti, R., Agliardi, F.,(2002) *STONE: a computer program for the three-dimensional simulation of rock-falls.* Computers & Geosciences. **28**(9): pp. 1079-1093.

[7] Dorren, L.K.A.,(2003) *A review of rockfall mechanics and modelling approaches.* Progress in Physical Geography. **27**(1): pp. 69-87.

[8] Rackwitz, R.,(2002) *Optimization and risk acceptability based on the Life Quality Index.* Structural Safety. **24**(2-4): pp. 297-331.

PREPROCESSING AND SMOOTHING OF SURFACE MODELS FOR ROCKFALL SIMULATION

Christian Spang [1], Bernd Romunde[1]

Today highly detailed surface representations are available. These data stem from Laser scanning, photogrametry or different surveying methods and are formulated as digital surface models. Modern computer technology allows to incorporate this surfaces representations without further adaptation in simulation software. First experiences with rockfall simulations based on digital surface models showed the scale dependency of the modeled process. Theoretical considerations and some preliminary research done on this topic showed that the model parameter roughness is not directly comparable to the actual slope surface. The necessity to smooth the actual surface in order to apply a appropriate roughness which meets the demands of the present simulation approach will be discussed.

Keywords: Rockfall simulation, surface representations, surface roughness

INTRODUCTION

Every model is a simplified representation of reality. The processes, which determine the behaviour of a natural system, have to be simulated by the model. The capability of prediction of any numerical model can be endangered by an oversimplified model approach or uncertainties in our knowledge of the natural system due to sparse data. Here we do not want to deal with obvious errors in the model itself. Although obvious errors are not always that obvious, if we consider for example the numeric behaviour of highly complex models and their solution algorithm.

In numerical rockfall simulation model approach and the degree of detail i.e. the scale of surface representation can not be dealt with independently. The interaction between Block and surface is highly dependent of block form and surface features in the magnitude of the block size. Therefore modelers are not free to choose a scale as can be done within certain limits in other fields of numerical modeling.

The rockfall process is a series of sliding, toppling and impacts on the subsoil. With higher velocities and steeper gradients of the slope the rock can entirely lift of from the surface and follow a ballistic trajectory until impact on the slope again. Rolling, which is a possible movement in most simulation algorithms, is already an idealization of the natural process. In nature it will not occur in this form, because rock blocks are rarely spheres or have any circular forms. Neither is the surface exactly level.

Dr. Spang Civil Engineering and Geotechnical Consultants LTD, Westfalenstr. 5-9, D-58455 Witten, +49 2302-914020 (fax -9140220), dr.spang@t-online.de

SIMULATION STRATEGY

In recent years up to now one of the possible strategies is the representation of the slope profile in a macro scale which is overlain by a micro roughness in the magnitude of the block size. Micro roughness is the stochastic model of zigzags. The spikes are characterized by length and amplitude. The roughness representation was and is due to the estimation and experience of the modeller. In effect this led to a rather detailed, explicit and computer power consuming simulation strategy.

ANALYSIS OF SURFACE REPRESENTATION

The nowadays available higher resolution of surface models first led to the hope that it may be possible either to identify the relevant characteristics from the surface representations or to model the rockfall path on the actual real life surface without a stochastic model of surface roughness. In particular the first approach seemed promising. It was expected that the smoothing process in it self can result in the identification of the relevant surface roughness, if a sufficient resolution of the surface representation is given.

The analysis of surface roughness by geostatics, yielded surface roughness which were not comparable to model parameters. Further more the resolution of the surface representation has an effect on simulation results. In the numerical model of rockfall every peak is likely to produce an impact. In addition every point of a surface model is likely to act as a peak even if it does not represent a peak of the real surface. Therefore we can state a direct effect of the resolution to energy dissipation. Different representations of the same slope surface which are only different in their resolution respectively scale will produce different simulation results because of the number of peaks which may produce impacts between block and subsoil.

Our approach is to smooth out the slope profile and afterwards overlay it with a appropriate roughness. Two different methods for smoothing were tested and applied to real world projects. From this experience we are recommending only one of these.

CONCLUSIONS

Theoretical considerations and some preliminary research done on this topic showed that the model parameter roughness is not directly comparable to the actual slope surface. At the present state of the art the roughness concept also covers up for some additional effects, which are not explicitly taken into account in the model approach. This article discussed the necessity to smooth the actual surface in order to apply a appropriate roughness which meets the demands of the present simulation approach.

DISCUSSIONS

But the discussion of the influence of surface representation in different scales is leading to a better process understanding. This may open the way for model approaches, with a higher degree of abstraction. This is insofar from great importance as the present model approach with its roughness concept can not directly be transferred to a higher dimensional approach.

The recent years showed a demand for more extensive rockfall simulations. This applies for risk mapping as well as for design and dimensioning of mitigation measures and rockfall protection structures.

ACKNOWLEDGEMENTS

Dr. M. Ruff, Bart Ingenieure St. Gallen, who came up with a first idea for the smoothing process.
Dr. H.-M. Möbus, Regierungspräsidium Freiburg, for his observations of the scale effect from surface models on rockfall results.

REFERENCES

[1] CROSTA, G. B., F. AGLIARDI (2003) A methodology for physically-based rockfall hazard assessment. – In: Natural Hazards and Earth System Sciences, **3** (5): 407-422, Katlenburg-Lindau (Europ. Geosc. Union)
[2] DAVIES, J.C. (1986) : Statistics and data analysis in Geology ; 2nd Edition, John Wiley & Sons, NY, Chichester, Brisbane.
[3] SPANG, R. M. (1988): Empirical and mathematical approaches to rockfall protection and their practical applications. Proc. 5th Int. Symp. Landslides, Lausanne, II, 1237-1243
[4] SPANG, R. M. (1994): Geologisch-Geotechnische Grundlagen des Steinschlagschutzes. Vortr. 11. Bodensee-Tag. Ing.-geol. Naturgefahren, Risikoanalysen, Schutzkonzepte, 21.10.1994

ROCKFALL PROTECTION 2008: FLEXIBLE ROCKFALL BARRIERS SUBJECTED TO EXTREME LOADS AS HIGH-SPEED ROCKFALLS OR FALLING TREES

Aron Vogel[1], Axel Volkwein[2], Werner Gerber[2], Andrea Roth[1]

Flexible rockfall barriers are proven protection systems to mitigate the hazard of rocks hitting people or infrastructure. In order to assure that such systems are able to dynamically stop falling rocks in reality, several guidelines requesting full-scale tests were introduced worldwide. These guidelines consider very standardized and repeatable load cases but do not take into account extreme loads. This paper deals with such loads acting on flexible barriers and the consequences for the designers. Two examples of such extreme loads are chosen. The first one is the impact of high-speed rockfalls with impact velocities of well above 25 m/s as described in the testing guidelines. The second extreme load is the impact of a tree trunk with the same weight as a falling rock but with a much smaller impact area.

Keywords: Rockfall barriers, extreme loads, tree fall, high-speed rockfall, full-scale tests, numerical modelling

INTRODUCTION

The current guidelines for the testing of rockfall barriers take into account the full-scale test of the kits with a standardized test body made of concrete and an impact velocity of 25 m/s (e.g. Swiss guideline [2]). This velocity was gained by the back-calculation of rockfalls but is exceeded in areas with very high slopes or cliffs. In such cases, the validity of the executed rockfall tests is not given anymore since such high-speed rockfalls can create puncturing effects on the net structure which is not observed with slower and bigger rocks, even when the energy level is the same.

The same effect can be seen with falling trees where the logs can be quite heavy (comparable to falling rocks) but the impact area is much smaller since the trees tend to slide down the slope and hit the bottom of the barriers with their tip. Another extreme load for rockfall barriers is the impact of snow slides in winter time. This was investigated by Margreth et al. [3] and led to the conclusion that it is possible to design flexible barriers in such a way that they can cope with falling rocks in spring/autumn and snow slides in winter.

As done with the snow slides, the extreme loads high-speed rockfalls and tree falls were tested in full-scale tests without extrapolation and then back-calculated by numerical modelling. It was investigated how to improve flexible rockfall protection systems in such a way that they are able to withstand these extreme loads.

[1] Geobrugg AG Protection Systems, Hofstr. 55, CH-8590 Romanshorn, Switzerland
[2] WSL Swiss Federal Institute for Forest, Snow and Landscape Research, Zuercherstr. 111, CH-8903 Birmensdorf, Switzerland

FLEXIBLE ROCKFALL BARRIERS SUBJECTED TO HIGH-SPEED ROCKFALLS

In certain cases rockfall protection is required below vertical rock faces [4] (Fig. 1). If these rock faces are high, the possible maximum velocities of falling rocks can become high as well. The problem of the so called bullet-effect becomes evident. The performance of barriers with velocities higher than 25 m/s was not investigated so far. To achieve a similar safety level for barriers impacted with high velocities, full-scale field tests were executed in 1999/2000 in Varen, Switzerland with velocities of more than 40 m/s (Fig. 1).
The following observations were made. High-tensile chain-link mesh stopped 50 kJ at v=43 m/s (52 kg at 96 m free fall) without damage. Standard chain-link or hexagonal mesh on top of ring nets got perforated from rocks of 20-30 kg at v=43 m/s (96 m free fall). Ring nets need to be covered with high-tensile chain-link in case of v > approx. 30-35 m/s. The energy absorption capacity of ring nets at v= 43 m/s is approx. 25% less than with v= 25 m/s. With this knowledge it is possible to determine under what conditions the ring net has to be reinforced and the secondary netting has to be replaced by high-tensile mesh.

FLEXIBLE ROCKFALL BARRIERS SUBJECTED TO TREE FALL

In steep, afforested terrain, being hit by a sliding tree is a frequently encountered hazard for people and infrastructure (Fig. 2). The tree impact can result from either forest operations, old or instable trees or storms. If trees start sliding and lose their limbs, their trunks can reach relatively high velocities. Very often flexible rockfall protection systems are installed between protective forest and infrastructure in order to protect it from rockfalls or are specifically installed before forest operations. In both cases impacts of trees can occur into flexible protection nets that were originally developed and designed for catching rocks.
For the systematic analysis of the interaction between flexible protection systems and impacting trunks a test series with a 1’000 kJ system was carried out in 2005 (Fig. 2). The tests were made at the WSL-test site in Walenstadt, Switzerland. The trunks were accelerated vertically by free falling into the installed protection system. Three tests were executed with 160 kJ (1’600 kg tree with 14 m/s impact velocity), 320 kJ (1’600 kg with 20 m/s) and 610 kJ (2’000 kg with 24 m/s) resp. The was no maintenance on the system in between the tests (Fig. 2).
At all the tests the tree trunk got caught and stopped by the protection system. The forces in the superstructure are comparable with a rockfall of the same energy level. The ring net has to withstand higher local forces due to the concentrated impact area. However the tree got pressed through the high-tensile ring net without damaging the net. The tests were further used for the validation of the FE-Software FARO that was developed by the Swiss Federal Institute of Technology ETH in Zürich especially for dynamic impacts [1] (Fig. 3). The calculated results show a high correlation with the 1:1 tests.

CONCLUSIONS

It can be concluded that it is possible to design flexible rockfall barriers to meet extreme loads as high-speed rockfalls, falling trees, snow slides or similar hazards based on the systems already approved according to the existing testing guidelines. But it is important to execute according field tests on a realistic level for mass, velocity and impact area and then validate the numerical model in order to design the systems to the specific site conditions. The work presented here allows doing so with high-speed rockfalls of velocities up to 45 m/s and falling trees with energies of up to 600 kJ.

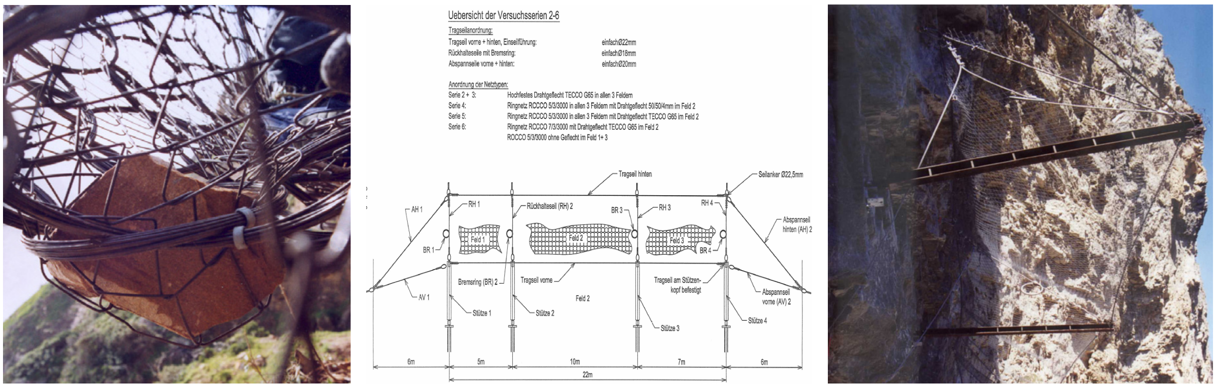


Fig. 1 High-speed rockfalls: Hazard in reality (left, [4]), drawing of the test setup (center) and installed test barrier at Varen (right)

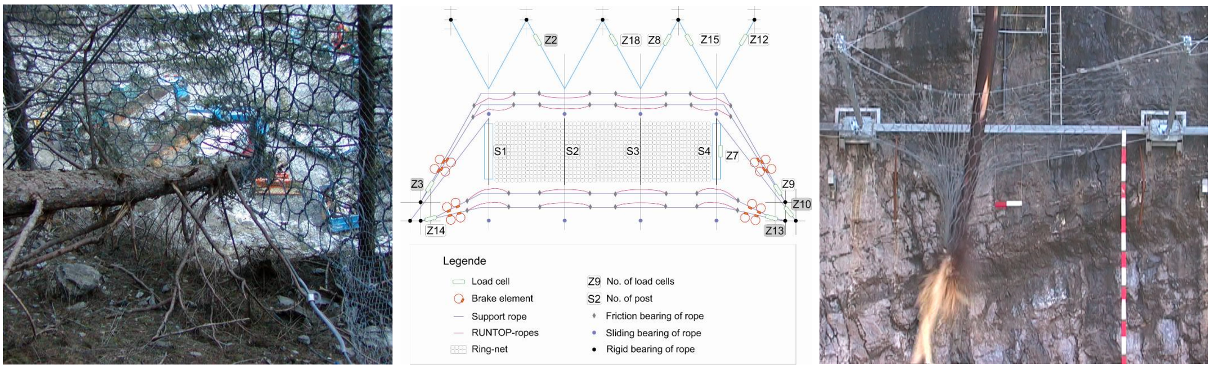


Fig. 2 Tree Fall: Hazard in reality (left), drawing of tested system (center) and impact test in Walenstadt (right)

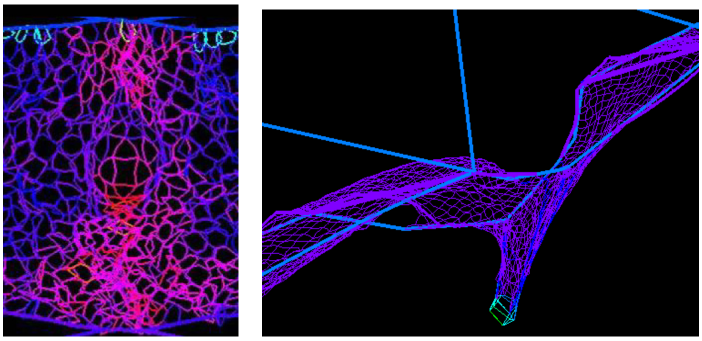


Fig. 3 Numerical modelling with software FARO of a high-speed rockfall (left) and a tree impact (right)

REFERENCES

[1] Anderheggen E., Volkwein A., Grassl H.: Numerical Simulation of Highly Flexible Rockfall Protection Systems, Fifth World Congress of Computational Mechanics, Vienna, 2002

[2] Gerber, W.: Guideline for the approval of rockfall protection kits. Swiss Agency for the Environment, Forests and Landscape (SAEFL), Swiss Federal Research Institute WSL, Berne, 2001

[3] Margreth, S., Roth, A.: Interaction of flexible rockfall barriers with avalanches and snow pressure, Cold regions science and technology, 2007

[4] Volkwein A., Melis L., Haller B., Pfeifer R.: Protection from Landslides and High Speed Rockfall Events – Reconstruction of Chapman's Peak Drive. Proc. Structures and Extreme Events, IABSE Symposium, Lissabon, 2005

TESTING OF FLEXIBLE BARRIERS - BEHIND THE GUIDELINE

Nils Wienberg[1], Hans Weber[2], Marco Toniolo[2]

As part of a test series for a rock slide safety net, a field test of a rock slide safety net of energy class 1000 kJ was conducted between 9/3/07 and 12/19/07 at the testing area of Isofer AG in Italy. Set up and vertical throw with the corresponding monitoring and evaluation occurred based on the requirements in the Swiss Federal Guideline "Guideline for the approval of rockfall protection kits" [1] and the European guideline "ETAG 27" [2]. Differing from the regulations listed, a system with a distance of 5 meter between the posts in the strike field area was used.

Keywords: testing, rockfall protection, guidelines, special load cases

TESTING OBJECTIVE

The Swiss and European guidelines [1,2] specify a rockfall event in the middle of a three field wide barrier with an impact velocity of 25 m/s as the standard rockfall testing event. The fields usually have a width of 10 m and also the minimum height of the barriers is described. But usually, the nature or the real world projects end up in special solutions which cannot be answered by above guidelines. E.g. what happens if a post or rope is being hit by the falling rock or how does the system behave due to an event into a periphal field.
Therefore, the test's objective was to measure the resistance of a rockfall protection system (Isostop 1000, planned working height, 5 m distance between supports) against the impact of a thrown object hitting the protective net at a certain speed and in different areas of the protective net. In detail, the seven partial tests given in Tab. 1 were conducted.

Tab. 1 Details of tests

Test No.	Mass	Release height	Impact velocity	Energy	Impact location
	[kg]	[m]	[m/s]	[kJ]	
1	3220	32	25	1000	Center of central field
2	3220	8	12.5	250	Post S3
3	1640	32	25	500	As test no. 1 but no repair after no. 2
4	3220	32	25	1000	As test no.1 with simulated repaired net
5	3220	8	12.5	250	Retaining rope at post S3
6	3220	32	25	1000	Center of peripheral field
7	3220	32	25	1000	As test no. 1 with only 3m barrier heigt

SETUP OF THE ROCK SLIDE SAFETY NET

The tested rockfall safety net is a rope net with diagonal mesh. Every second cross point is connected with a cross clip. The net is connected on the top and on the bottom rope with a joint cable. Various brake elements are installed in the construction. The test setup is described in detail in the technical documentation.

[1] BTU Cottbus' Department Chair for Soil Mechanics and Foundations/Geotechnology,
Konrad-Wachsmann-Allee 1• D-03046 Cottbus, +49 355 / 69-2602 (fax: -2566)
[2] isofer ag, protection systems, Industriequartier, CH-8934 Knonau +41 44 768 55 55 (fax: -30), info@isofer.ch

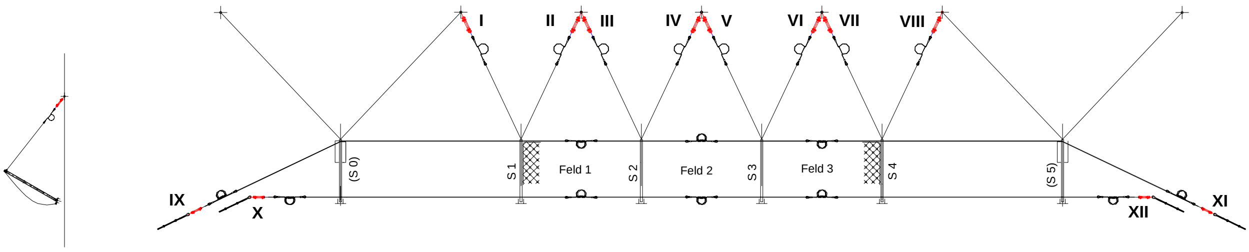


Fig. 1: Building plan of the tested system with arrangement of load cells I-XII

TEST EXECUTION

The test facility is located in an old quarry in Italy. The protection system is mounted with an inclination of 30° to an almost vertical rock face. Load cells measuring forces up to 400 kN and a sample rate of 1200 Hz are integrated in the retain ropes and at the anchorage of the support ropes. For the distribution of the load cells in the system see Fig. 1. The tests are additionally recorded by high speed video with 250 pictures/sec.
The test bodies are shaped as shown in Fig. 2 and made of reinforced concrete. They have the masses 3200 kg and 1640 kg and cube dimensions of 1.19 m and 0.96 m at a average density of 2700 kg/m^3.
After installation of the test setup, the safety facility and camera positions were measured by using an electronic theodolite. All brake elements were marked with colour spray in such a way that the rope movements could be determined after the test. The object thrown was lifted by a crane and centred above the strike field using a plumb line. Thereafter, it was released from a height defined by the necessary potential energy.
After the test, measurements of post positions and the rope centres of the load-bearing ropes above and below the strike field were repeated and the actual status of the system was evaluated. From the single pictures of the high speed video, the brake process with the timely decrease of the velocity and the brake distance is retrieved. It also allows the analysis of the barrier deformations.

RESULTS


Fig. 2: Post hit (test 2) and retain rope hit (test 5)

All tests were successfully performed. The test specimens were always retained. The residual height for the barrier after the tests lays between 68 and 86%. All brake elements were activated and the ropes deformed as planned and expected, resp. the net did not reveal any damages. Fig. 2 shows the deformed system after a post and a rope hit.

Tab. 2 Force Measurements

Test No	**1**	2	3	4	5	6	7	S02-3 1)
Position	max. Measurements in kN							
I	0	1	1	1	0	6	18	-
II	23	2	27	0	2	60	39	-
III	72	18	78	71	58	34	64	120
IV	51	21	53	49	41	57	-	-
V	60	53	13	3	37	11	50	108
VI	64	44	51	59	57	25	95	65
VII	22	24	24	34	39	3	19	32
VIII	10	2	5	11	10	3	15	24
IX	138	111	142	122	170	159	121	196
X	94	28	93	110	92	179	122	154
XI	119	48	83	97	100	102	153	177
XII	115	93	104	150	91	109	101	150

1) Type approval of safety for protection against rockfall, Test certificate No S02-3

SUMMARY

During the period 9/3/2007 to 12/19/2007 a large test series was conducted on a rock slide safety net model Isostop 1000 kJ by Isofer AG. Objects thrown of different mass and size were stopped by the system at various impact speeds and strike locations. The structure's resistance against the impacts brought to bear on it was demonstrated.
The tests show that a barrier system developed according to the actually effective guidelines is able to withstand exceptional load cases that are not explicitly considered in the guidelines even though such load cases will occur in nature.

REFERENCES

[1] SAEFL (2001/2006) Swiss Federal Guideline, "Guideline for the approval of rockfall protection kits", Bern
[2] EOTA (2008) GUIDELINE FOR EUROPEAN TECHNICAL APPROVAL of FALLING ROCK PROTECTION KITS, „ETAG 27", EOTA European Organisation for Technical Approvals

RISK ASSESSMENT ON ENCOUNTERING ROCKFALLS ALONG A 430KM PART OF THE CHENGDU-LHASA HIGHWAY IN TIBET

L. Q Zhang[1], Z. F. Yang[2]

The Chengdu-Lhasa highway is an important transportation corridors connecting Tibet with the inland of China. A 430 km long part of the highway is investigated for rockfall activities, and 19 slopes chosen for risk assessment. Firstly, rockfall frequency is estimated by combining possible knowledge about site investigation, experiences and theoretical analyses. Then, the Bernoulli formula is used to calculate annually averaged probability of encountering rockfalls under three situations, i.e., immobile vehicles, mobile vehicles and foot passengers. The probability of rockfall-induced casualties is assessed by multiplying the probability of encountering rockfalls and vulnerability of a highway user.

Keywords: the Chengdu-Lhasa highway, rockfalls, rockfall hazards, risk assessment

INTRODUCTION

Assessment on hazards and risks of rockfalls were conducted by a number of investigators, with a number of rockfall assessment methods achieved [1-5]. As rapid development of economy in China, a great number of highways in mountainous areas have been confronted with more and more rockfall events. Obviously, risk assessment on encountering rockfalls along these traffic routes is significant for zonation of rockfall hazards and optimum scheme of mitigation costs.

The Chengdu-Lhasa Highway starts at Chengdu of Sichuan Province, and ends at Lhasa of Tibet. A 430 km long part of the highway, between Paksho and Nyingtri in southeast Tibet, is investigated for rockfall activities. Situated within the suture zone created by the collision of the Indian and Eurasian tectonic plates, the region is characterized by deep valleys, steep slopes, highly fractured and heavily weathered rocks, extensive areas of debris accumulation, and locally high rates of geomorphological evolvement. As a result, rockfall events along the highway are usually unexpected, uncertain and frequent, and risk assessment of rockfall hazards are complicated by extensive distribution of dangerous blocks or boulders, lack of field data, complex geological structures, difficult site conditions and the high number of interacting factors. In this paper, a comprehensive and empirical method is utilized to estimate rockfall frequency in difficult mountainous conditions, probability of encountering rockfalls, and probability of rockfall-induced casualties.

ENVIRONMENTS OF ROCKFALL HAZARDS

The 430km part of highway under investigation belongs to the drainage area of the Palongzangbu River in Tibet. The undercutting of the river induces load relief-induced cracks, fissures and joints in the direction of the river, with steep cliffs being formed. Due to warm and

[1] Key Laboratory of Engineering Geomechanics, Institute of Geology and Geophysics, Chinese Academy of Sciences, +86-10-82998642 (fax: 86-010-62040574), zhangluqing@mail.iggcas.ac.cn
[2] Key Laboratory of Engineering Geomechanics, Institute of Geology and Geophysics, Chinese Academy of Sciences, +86-10-82998642 (fax: 86-010-62040574), yzf@mail.iggcas.ac.cn

wet airstreams from Bengal Bay, higher air temperature and richer precipitation occur in this stream area than in other areas of the Qinghai-Tibet Plateau. In addition, the drainage basin is situated in a seismic active belt southeast of the Qinghai-Tibet Plateau, having a seismic intensity IX. In a word, particular rock formations, complicated geological structures and steep mountainous landforms provide advantageous conditions for the occurrence of rockfalls, and frequent earthquakes, special climates and human activities are potential triggering factors of rockfalls. In addition, lower grade of road surface will increase the risk level of rockfall hazards. After site investigations, 19 hazardous rockfall zones are chosen for risk assessments, whose numbers are from Slope-1 to Slope-19.

ANNUALLY AVERAGED PROBABILITY OF ENCOUNTERING ROCKFALLS

Similar to some investigators [4~6], rockfall frequency (N_r) can be relatively estimated in terms of onlooking of rockfall events, impact traces of rockfalls, occurrence of screes, obvious unstable rock blocks or boulders, recordings of casualties, utilization of mitigating measures, etc.. In the 19 rockfall zones, annual frequency of rockfalls having a volume of larger than $5\times10^{-4}m^3$, were roughly estimated to be 12, 2.5, 4, 4, 0.08, 0.025, 9, 12, 36, 48, 0.1, 0.08, 0.015, 0.17, 0.07, 4, 2, 0.08 and 1, respectively. For a rockfall zone, human activities and potential rockfalls are supposed to be uniformly distributed in space and time along the highway, and Bernoulli formula can be used to compute probability of encountering rockfalls under three situations [6~8], i.e. immobile vehicles, mobile vehicles and foot passengers. Firstly, let us suppose that:

(1) l_v and l_p the averaged length (m) of vehicles and foot passengers, respectively; and, d_v and d_p are separation distances (m) between two adjacent vehicles and passengers, respectively;
(2) v_v and v_p are the averaged speeds (km/h) of mobile vehicles and foot passengers, respectively; and, d_s is the safety distance (m) necessary for a mobile vehicle to avoid static rockfalls on the road surface ahead;
(3) N_d is recurrence number of a situation per year, and T_d is duration time (h) for a situation.

For the situation of mobile vehicles, annual probabilities of being impacted during a rockfall event, can be calculated by following two formulas,

for uniformly distributed multiple mobile vehicales, $$p_A = \frac{N_s T_d (l_v + d_s)}{365\times 24(l_v + d_v)} \quad (1)$$

for a specific mobile vehicle (i.e., single target), $$p_A = \frac{N_s (l_v + d_s)}{1000\times 365\times 24 v_v} \quad (2)$$

For immobile vehicles and foot passengers, the annual probability of being impacted during a rockfall event can be obtained by the similar formulas to Equations (1) and (2). When passing the 19 rockfall zones with a number of rockfall frequency (N_r), annually averaged probability of encountering rockfalls can be calculated by means of the probability p_A and the Bernoulli formula, $P_A = 1-(1-p_A)^{N_r}$.

PROBABILITY OF ROCKFALL-INDUCED CAUSALTY

For immobile vehicles, mobile vehicles and foot passengers along the highway, rockfall-induced losses are limited only to human lives, and vulnerability analysis only to rockfall-induced casualties in the present paper. The 19 rockfall zones involved are characterized by steeper angles and higher locations of potential rockfalls (from several dozens to hundreds of meters high above the road surface), so it is possible for a quick falling block or boulder to penetrate into conventional jeeps, lorries, buses or cars. For the conveniences of risk assessment, the authors suppose that: (1) a rockfall with a volume of larger than $5\times10^{-4}m^3$, has an potential energy of penetrating into a vehicle; (2) for a vehicle impacted by rockfalls, the possibility of causing a user's casualty is from 20% to 80%, in views of potential impact energy, vehicle types, number and density of passengers in a vehicle, roadway conditions, etc..

Using rough statistical data in 2002, the probability of impact event induced by a rockfall, p_A, can be calculated by similar/same formulas to Eqs. (1) and (2), and the probability of encountering rockfalls, P_A, by the Bernoulli formula. By multiplying the probability P_A and vulnerability of a highway user, the annually averaged probability of rockfall-induced casualties can be calculated. As indicated in the calculated results, the probability of rockfall-induced casualties in rockfall zones Slope-1, Slope-4, Slope-7 to Slope-10, Slope-16, Slope-17 and Slope-19, are higher than those in other rockfall zones. The calculated results indicate that, 9 rockfall zones have a potential risk level of slightly higher than 1.0×10^{-6}, especially for those slopes with bigger rockfall frequencies.

CONCLUSIONS

By field investigations into rockfall characteristics, transportational conditions and human activities, the authors present empirical estimates of rockfall frequencies, probability of encountering rockfalls, vulnerability analysis of a highway user and the risk of rockfall-induced causalities. Compared with acceptable risk level (e.g., 1.0×10^{-6}) of geological hazards in developed countries or regions, the total risk of rockfall-induced causalties is slightly higher for a user of the 430 km part of the Chengdu-Lhasa highway. As a relative assessment of risk level, the present method may be feasible and helpful for risk zonation of rockfall hazards and optimization of mitigation costs along this part of the highway.

REFERENCES

[1] Franklin J A and Senior S A. The Ontario rockfall hazard rating system. Engineering Geology and the Environment, Marinos, Koukis, Tsiambaos & Stoumaras (eds) Balkema, Rotterdam, 1997, 647~656
[2] Abbott B, Bruce I, Savigny W, et al.. A methodology for the assessment of rockfall hazard and risk along linear transportation corridors. Paper presented at the 8th International IAEG Congress, Balkema, Rotterdam, 1998, 1195~1200
[3] Cancelli A and Crosta G. Hazard and risk assessment in rockfall prone areas. Risk and reliability in ground engineering, Thomas Telford, London, 1993, 177~190
[4] Hungr O, Evans S G and Hazzard J. Magnitude and frequency of rock falls and rock slides along the main transportation corridors of southwestern British Columbia. Can. Geotech. J., 1999, 36:224~238
[5] Abbott B, Bruce I, Savigny W, et al.. Application of a new methodology for the management of rockfall risk along a railway. Paper presented at the 8th International IAEG Congress, Balkema, Rotterdam, 1998, 1201~1208
[6] Bunce C M, Cruden D M and Morgenstern N R. Assessment of the hazard from rock fall on a highway. Can. Geotech. J., 1997,34:344~356
[7] McClung D M. The encounter probability for mountain slope hazards. Can. Geotech. J., 1999, 36:1195~1196
[8] Hungr O and Beckie R D, Assessment of the hazard from rock fall on a highway: Discussion. Can. Geotech. J., 1998, 35:409

Index of Authors

Index of Keywords