EUROCK 2002 – ISRM International Symposium on Rock Engineering for Mountainous Regions

25-28 November 2002
Funchal, Madeira
pp. 145-153

NUMERICAL ANALYSIS OF PROGRESSIVE FAILURE IN NATURAL ROCK SLOPES

EBERHARDT, ERIK
Engineering Geology, Swiss Federal Institute of Technology (ETH), Zurich, SWITZERLAND, erik@erdw.ethz.ch

KAISER, PETER K.
Geomechanics Research Centre, Laurentian University, Sudbury, Ontario, CANADA, pkaiser@nickel.laurentian.ca

STEAD, DOUG
Earth Sciences, Simon Fraser University, Vancouver, B.C., CANADA, doug_stead@sfu.ca

ABSTRACT

The 1991 Randa rockslide in the Swiss Alps involved several complex mechanisms relating to geological, mechanical and hydrological processes for which no clear triggering mechanism can be ascertained. This paper investigates the concept of progressive failure and the numerical modelling of rock mass strength degradation in natural rock slopes using the Randa rockslide as a working example. Results from continuum (i.e. finite-element) modelling are presented to illustrate an interpretation suggesting that initiation of a progressive rock mass degradation process, ultimately leading to failure, was initiated following deglaciation of the valley. Discontinuum (i.e. distinct-element) modelling is then applied to investigate the underlying mechanisms contributing to the episodic nature of the rockslide. Finally, the use of hybrid methods that combine both continuum and discontinuum techniques to model fracture propagation are discussed in the context of modelling progressive slip surface development linking initiation and degradation to eventual catastrophic failure.

1. INTRODUCTION

In many rock slope stability analyses, the failure surface is assumed to be structurally-controlled and is predefined as a continuous plane or a series of interconnected planes. The reasons for this are partly due to post-failure observations where fully persistent discontinuities are fitted to the failure surface to explain its origin in a geological context, and partly due to the constraints of the analysis technique employed, many of which require the input of fully persistent discontinuities (e.g. limit equilibrium wedge or planar analysis, distinct-element method, etc.). Such assumptions are often valid, but only in cases where the volume of the failed block is relatively small (e.g. 1000's of m^3) or where major faults and/or bedding planes are present.

In massive natural rock slopes and deep engineered slopes (e.g. open pit mines), it is unlikely that such a network of fully persistent natural discontinuities forming a complete 3-D outline of the unstable mass exists. Terzaghi (1962), Jennings (1970), Einstein et al. (1983) and others suggest that the persistence of key discontinuity sets is limited and that a complex interaction between existing natural discontinuities and brittle fracture propagation through intact rock bridges is required to bring the slope to failure. Eberhardt et al. (2001) argue that such processes must be considered to explain the temporal nature of massive natural rock slope failures. For example, in small engineered slopes, the rock mass may be continuously disturbed by blasting and fully persistent discontinuities may be exposed/daylighted during excavation enabling kinematic feasibility. However, natural rock slopes do not experience such rapid changes to their kinematic and have stood relatively stable over periods of several thousand years. This is not to say that in a natural rock slope, a system of natural discontinuities may not be interconnected forming a significant portion of what will eventually be the failure surface, but that a component of strength degradation with time must also occur within the rock mass.

As such, massive rock slope instability requires the progressive degradation of cohesive elements, for example intact rock bridges and interlocked joint asperities, to bring the slope to catastrophic failure. This paper explores the concept of progressive failure and examines the inclusion of strength degradation in the modelling of massive rock slope failures, focussing on the example of the 1991 Randa rockslide in southern Switzerland.

2. PROGRESSIVE FAILURE IN NATURAL ROCK SLOPES

The concept of progressive slope failure was originally introduced to explain discrepancies between average shear stresses back-calculated along failed shear surfaces in overconsolidated clay slopes and shear strengths of the same clay material in laboratory testing. Bjerrum (1967) summarized in his Terzaghi lecture that failure in such cases must be preceded by the development of a continuous sliding surface through the progressive development of a shear surface along which shear strength is reduced from peak to residual values (Fig. 1). This model implies a gradual development of the eventual failure plane.

In rock slopes, Terzaghi (1962) emphasized that most rock masses contain discontinuous joints varying in persistence, such that both their pressure-conditioned shearing resistance (i.e. the frictional strength component) and the cohesion of intact rock bridges between discontinuous joints (referred to as “*effective cohesion*” along the shear surface) help act to resist shear failure. Progressive failure in brittle rock slopes would therefore involve the failure of individual rock bridges as their shear strength was exceeded. Whereupon stresses ahead of the shear plane would increase and subsequent intact rock bridges fail in a chain reaction until the surface of failure extended to the point where kinematic release became possible.

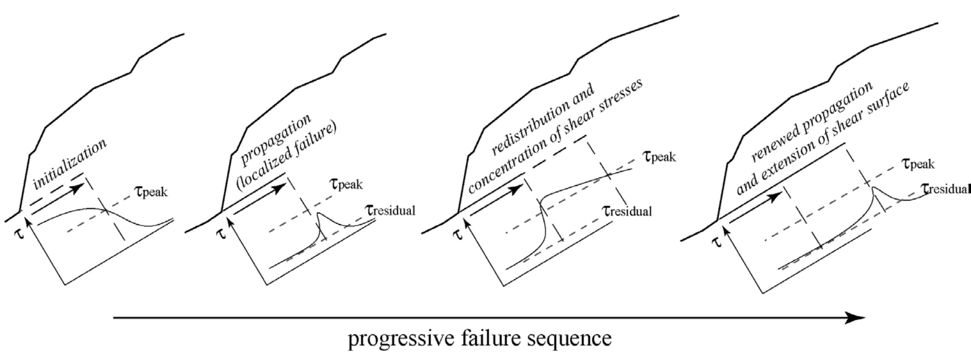

Figure 1. Shear surface development by progressive failure (after Bjerrum 1967)

Under such conditions, failure should initiate at the toe of the rock slope where the stresses are highest and propagate upwards to the surface. Such a failure mode would be considered as being predominantly brittle with very little internal deformation required to accommodate failure. Yet in many cases surface tension cracks appear at the top of the slope long before catastrophic failure occurs, suggesting that failure can also be partly driven through internal deformation mechanisms (e.g. cumulative brittle damage, elasto-plastic yielding, creep, etc.). Therefore, as shown in Figure 2, the primary controls contributing to massive rock slope failure can be viewed as strength degradation in the form of shear plane development (i.e. progressive failure) and strength degradation manifested through internal slide mass deformation (i.e. brittle-ductile yielding or strain-softening). The latter component being most dominant in situations where the failure surface is non-planar as at Randa. It is particularly important in situations where internal shear surfaces develop (as reported by Martin & Kaiser 1984).

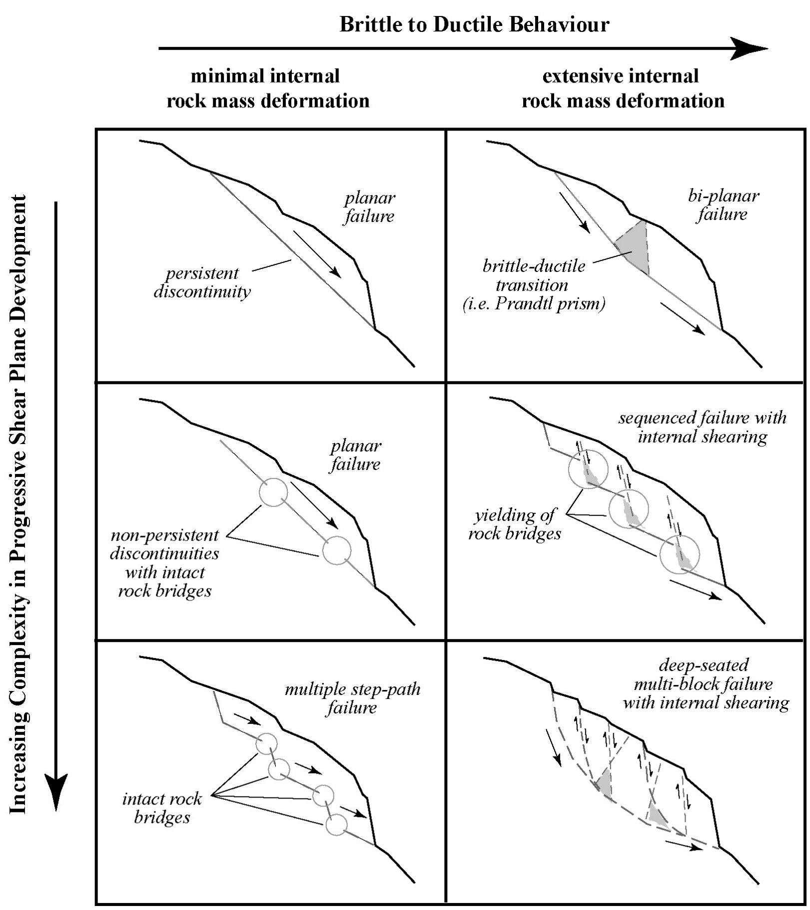

Figure 2. Massive rock slope sliding mechanisms as controlled by progressive shear plane development and internal rock mass deformation/damage

Depending on the complexity of the rock mass geology and subsurface structure, these controls will vary in their influence on the overall instability state. In cases where adversely oriented discontinuities are relatively persistent and/or failure only requires the propagation of a shear plane through intact rock bridges, the failure mechanism will kinematically require very little internal deformation for release (Fig. 2, left-hand column). Examples in the Alps include the 1806 Goldau slide where failure occurred along a shear surface parallel to planar bedding contacts, or the 1881 Elm slide where failure was initiated by undermining of the slope (Fig. 3).

If kinematic release does not occur along one or more relatively planar features, failure requires internal deformation of the rock mass whereby the slide body undergoes a brittle to ductile transition as damage accumulates, localized slip occurs along joints and rock mass strength degrades (Fig. 2, right-hand column). Alpine examples of these types include the 1963 Vaiont slide, where Mencl (1966) proposed the development of a Prandtl wedge (i.e. damage zone) to explain failure along bi-linear slip surfaces (Fig. 4). Limit equilibrium techniques have long recognized the importance of internal shears (e.g. Sarma 1979). However, these methods do not consider the gradual, deformation dependent loss of cohesion and mobilization of frictional strength along these internal shears (e.g. Hajiabdolmajid & Kaiser 2002a).

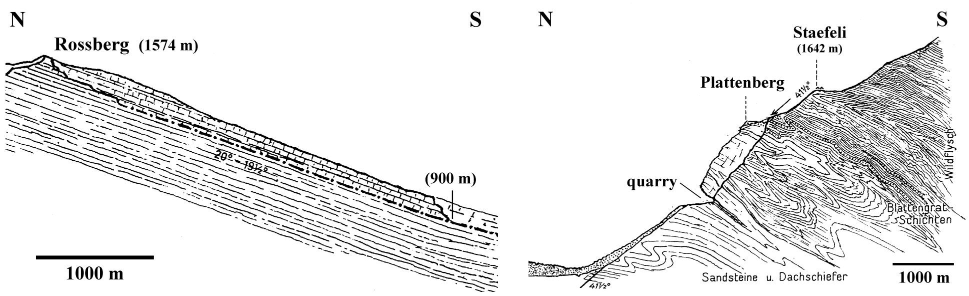

Figure 3. Schematic cross-sections of the Goldau (LEFT) and Elm (RIGHT) rockslides (after Heim 1932)

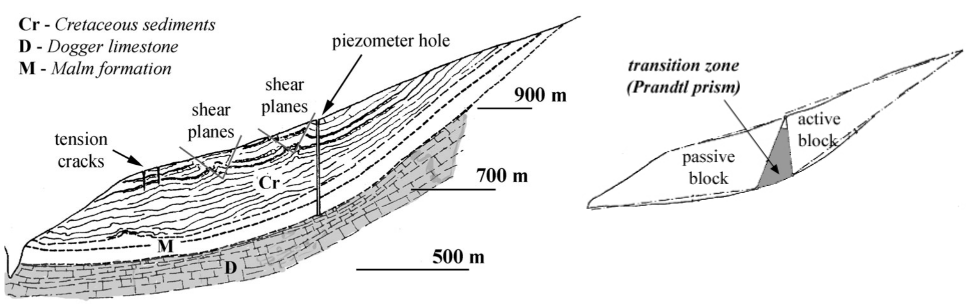

Figure 4. Schematic cross-section of the Vaiont slide showing transitory damage zone development in the form of a Prandtl prism (after Mencl 1966)

3. NUMERICAL MODELLING OF PROGRESSIVE FAILURE – RANDA ROCKSLIDE

The Randa rockslide involved the failure of approximately 30 million m^3 of massive crystalline rock in two distinct episodes (Fig. 5). As foliation dips favourably into the slope, Schindler et al. (1993) suggested that failure occurred along extensive shallow dipping stress-relief joints parallel to the surface. These persistent joints can be observed along the sliding surface but are more limited in persistence when encountered in surface outcrops (Willenberg et al. 2002). Cross-cutting faults were also proposed as dividing the slide mass into smaller units (Schindler et al. 1993, Götz & Zimmermann 1993), presumably to explain the episodic nature of the slide.

Analysis of climatic and regional seismic data, however, showed no clear indications of a triggering event (Schindler et al. 1993). Eberhardt et al. (2001) instead suggest that time-dependent mechanisms relating to brittle strength degradation and progressive failure are likely the contributing factors that brought the slope to failure. As such, the failure mechanism can be viewed as the progressive accumulation of damage with time (i.e. through heavy rainfall, snow melt and freeze-thaw cycling events) that acts to degrade the strength and equilibrium state of the slope, with each event bringing the slope nearer to failure.

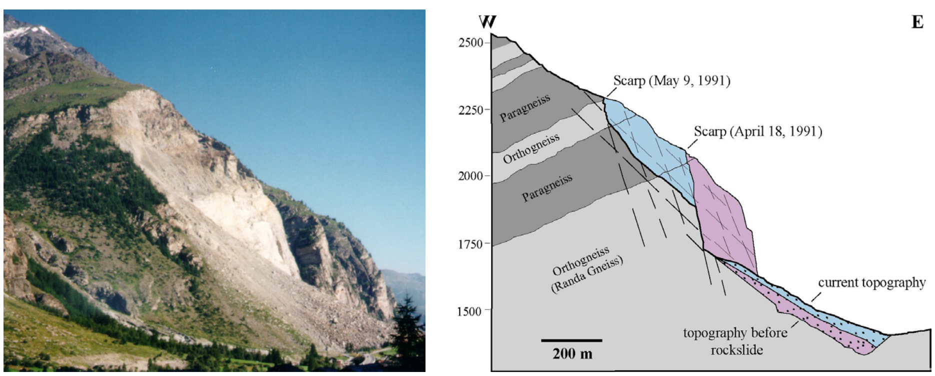

Figure 5. The 1991 Randa Rockslide illustrating the two key sliding events (after Schindler et al. 1993)

3.1 Initiation

The initiation of progressive failure requires the presence of a plane of weakness, which acts as a stress concentrator through which fracture propagation begins. Given the discontinuous nature of most rock slopes, these stress concentrators are likely to take the form of natural joints. In the case of Randa, the dominant joint set forms parallel to the foliation which dips into the slope sub-perpendicular to the failure surface (Fig. 5). However, field observations also suggest the presence of stress-relief joints parallel to topography and coinciding with the failure surface.

Finite-element modelling was used to investigate the role glacial processes may have played in initiating the failure process. Glacial unloading was simulated assuming that the glacier filled the valley (Fig. 6). Material properties were based on those for glacial ice (γ = 0.009 MN/m^3, E = 10 GPa, ν = 0.3) and unweathered gneiss (γ = 0.026 MN/m^3, E = 30 GPa, ν = 0.3). To model damage initiation, a Mohr-Coulomb elasto-plastic yield criterion was used (c = 10 MPa, ϕ = 30°, To = 0.5 MPa). This approach allows for an investigation of damage initiation and is only valid as long as related stress redistribution processes do not dominate.

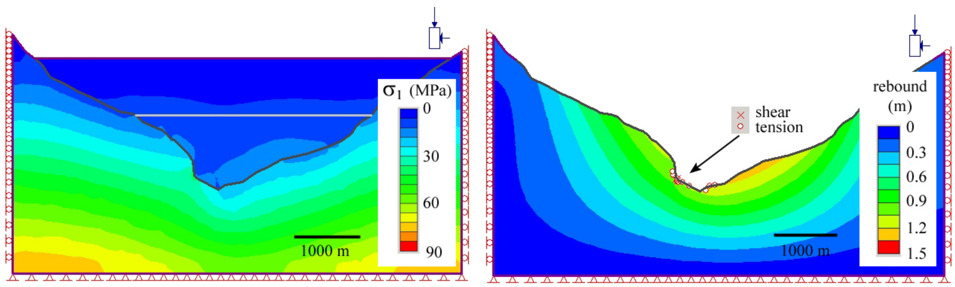

Figure 6. Finite-element continuum modelling of initial loading conditions due to glaciation (LEFT) and initiation at toe of Randa slope due to glacial rebound (RIGHT)

Results indicate that damage initiation could have been both a function of glacial rebound and oversteepening of the valley slopes. Sensitivity analysis showed that tensile damage developed at the toe of the slope for tensile strengths below 1 MPa (Fig. 6). The addition of pore pressures worked to increase the extent of tensile damage considerably, not only at the slope's toe but also within the upper slope as well. Ice height had only a minor effect, but was critical with respect to the development of tensile damage as opposed to shear damage, which was seen to be more a function of slope oversteepening.

3.2 Shear Plane Development

Modelling results showing the development of tensile zones with glacial rebound combined with stress concentrations induced through oversteepening of the valley walls, provide a likely initiation source for progressive shear plane development (as well as of rock mass degradation near the surface, above the eventual toe of the failed slope). Furthermore, Eberhardt et al. (2002) showed that the boundary of the 1991 Randa failure followed a zone approximately parallel to shear stress and plastic strain contours.

The modelling of shear plane development and progressive failure, however, is a more difficult task as the model has to incorporate strength degradation (as shear strains evolve) and ideally, both fracture propagation and time. On a practical level, strength degradation can be easily implemented as a function of strain through the use of continuum techniques. Hajiabdolmajid & Kaiser (2002a) demonstrate this by modelling cohesion loss as a function of plastic strain (i.e. accumulated brittle fracture damage) in a back analysis of the Frank Slide (Canada). As cohesion is destroyed, the frictional component of strength begins to mobilize. The influence of rock mass brittleness on deformation-controlled slope failure processes is further investigated in a companion paper by Hajiabdolmajid & Kaiser (2002b).

Figure 7a provides an example of a similar type of analysis and shows the evolution of a shear plane for the Randa slope as cohesion decreases. In this sequence, the *in* situ stress state is initialized, a Mohr-Coulomb elasto-plastic yield criterion is implemented and cohesion is progressively decreased as the shear plane evolves. Figure 7a demonstrates the transition of stable slope conditions to those of shear failure by showing the evolution of shear strains over two stages of progressive failure.

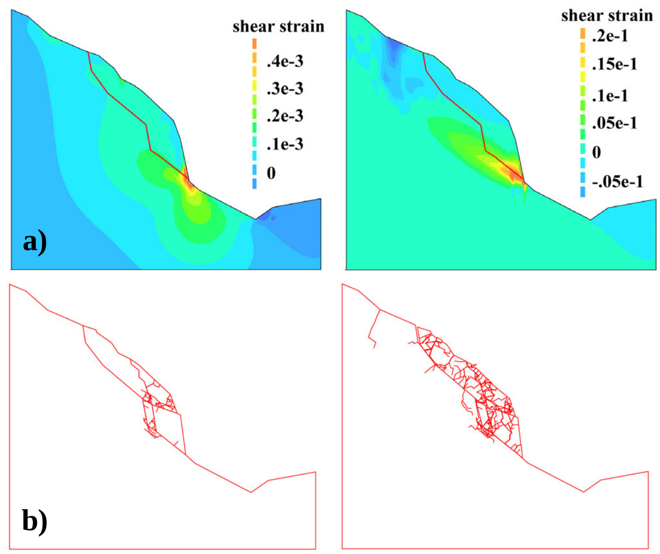

Figure 7. Numerical modelling of progressive shear plane development through: (a) continuum techniques showing shear strain development during cohesion loss; (b) hybrid continuum/discontinuum techniques showing evolution of brittle fracture network development (after Eberhardt et al. 2002)

Continuum modelling can be used to examine the evolution of stresses, strains and plastic yielding within the rock mass, but cannot easily incorporate jointing and propagating fractures. Studies are currently ongoing examining the use of hybrid continuum and discontinuum codes which allow for the modelling of both intact rock behaviour and the propagation of fractures (Eberhardt et al. 2002). These methods use adaptive remeshing routines to model the propagation of brittle fractures through a finite-element continuum (Rockfield 2001). Pre-existing discontinuities and induced fractures are in turn represented by discrete-elements. Preliminary results for the Randa rockslide are shown in Figure 7b. Ongoing work aims at improving this progressive failure model by incorporating shear plane propagation along pre-existing discontinuities and through intact rock bridges, using these techniques.

3.3 Internal Deformation and Strain Softening

Finally, modelling was performed to explore the role of internal rock mass deformation in contributing towards the Randa rockslide. In this case, the problem was modelled assuming the presence of a bi-planar failure surface that approximates the 1991 failure surface. Discontinuum techniques were employed in this situation using the distinct-element code UDEC. However, emphasis was not placed on yielding along the failure surface but within the intact slide body. Terzaghi's effective cohesion relationship was used to simulate the presence of intact rock bridges along the eventual failure surface.

By minimizing slip along the failure surface, a strain-softening criterion was used to simulate damage in the slide mass as a function of plastic strains. Similar to analyses performed in the previous section, intact rock strength was degraded as damage and plastic strains accumulated. Input values were approximated from brittle fracture thresholds for weak intact granite (Eberhardt et al. 1999) as listed in Figure 8.

ε_p	c (MPa)	ϕ (°)	To (MPa)
0	10	5	1
0.001	1	20	0.1
0.002	0.1	40	0
0.005	0.1	40	0

Figure 8. Distinct-element strain-softening model showing development of Prandtl yield zone at base of slide surface and propagation of tensile damage upwards through intact slide mass, dividing the slide mass along the contact between first and second Randa rockslide events

These models assist in explaining the episodic nature of the two main events that constituted the 1991 Randa rockslide. Figure 8 shows that a zone of yield due to shear damage develops near the base of the eventual slide surface and transforms into a tensile failure zone as straining occurs. This zone of tensile damage continues through the intact slide mass dividing the rockslide into two distinct blocks, approximating the contact between the first and second Randa events. This would suggest that the episodic nature of the slide could be explained by brittle tensile fracturing that developed over time due to stress concentrations and strength degradation. The tensile nature of failure in this area also supports the strain-dependent frictional strength development concept introduced by Hajiabdolmajid & Kaiser (2002a).

4. CONCLUSIONS AND FUTURE DEVELOPMENTS

To better understand the temporal nature of massive rockslides, stability analyses must incorporate elements of progressive failure and rock mass strength degradation. Various numerical models have been used here to demonstrate the evolution of failure in massive natural rock slopes as a function of shear plane development and internal slope deformation.

Numerical modelling results incorporating these concepts show, that in the case of the 1991 Randa rockslide, pre-existing fully persistent geological structures are not necessary to explain the failure. Instead, finite-element results show that the failure process could have initiated through the development of rock mass damage at the slope's toe following deglaciation of the valley and slope rebound. Stress concentrations resulting from oversteepening of the slopes would cause the damage zone to propagate and develop throughout the rock mass leading to a progressive failure starting from two or more damage initiation centres. Specifically, the distinct-element models show that deformation of the slope mass due to strength degradation would result in a brittle-ductile transition zone developing at the base of the slide mass, and development of a major sub-vertical tensile fracture zone dividing the slide mass into two blocks. This agrees with the observed episodic nature of the 1991 failure.

REFERENCES

Bjerrum, L. 1967. Progressive failure in slopes of overconsolidated platic clay and clay shales. Journal of the Soil Mechanics and Foundations Division, ASCE 93: 1-49.

Eberhardt, E., Stead, D., Coggan, J. & Willenberg, H. 2002. An integrated numerical analysis approach to the Randa rockslide. In J. Rybár et al. (eds), Proceedings of the 1st European Conference on Landslides, Prague: 355-362. Lisse: A.A. Balkema.

Eberhardt, E., Stead, D. & Stimpson, B. 1999. Effects of sampling disturbance on the stress-induced microfracturing characteristics of brittle rock. Canadian Geotechnical Journal 36(2): 239-250.

Eberhardt, E., Willenber, H., Loew, S. & Maurer, H. 2001. Active rockslides in Switzerland - Understanding mechanisms and processes. In H.H. Einstein et al. (eds), International Conference on Landslides - Causes, Impacts and Countermeasures, Davos: 25-34. Essen: Verlag Glückauf.

Einstein, H.H., Veneziano, D., Baecher, G.B. & O'Reilly, K.J. 1983. The effect of discontinuity persistence on rock slope stability. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 20(5): 227-236.

Götz, A. & Zimmermann, M. 1993. The 1991 rock slides in Randa: Causes and consequences. Landslide News 7: 22-25.

Hajiabdolmajid, V. & Kaiser, P.K. 2002a. Modelling slopes in brittle rock. In 5th North American Rock Mechanics Symposium, NARMS-TAC, Toronto: 331-338. University of Toronto Press.

Hajiabdolmajid, V. & Kaiser, P.K. 2002b. Slope stability assessment in strain-sensitive rocks. In Proceedings of the EUROCK Symposium, Madeira, Portugal: in press.

Heim, A. 1932. Bergsturz und Menschenleben. Zurich: Fretz and Wasmuth Verlag.

Jennings, J.E. (1970). A mathematical theory for the calculation of the stability of slopes in open cast mines. In P.W.J. VanRensburg (ed), Planning Open Pit Mines, Proceedings, Johannesburg: 87-102. Cape Town: A.A. Balkema.

Martin, C.D. & Kaiser, P.K. (1984). Analysis of a rock slope with internal dilation. Canadian Geotechnical Journal 21: 605-620.

Mencl, V. 1966. Mechanics of landslides with non-circular slip surfaces with special reference to the Vaiont slide. Geotechnique 16(4): 329-337.

Rockfield (2001). ELFEN 2D/3D Numerical Modelling Package (Version 3.0). Swansea: Rockfield Software Ltd.

Sarma, S.K. (1979). Stability analysis of embankments and slopes. Journal of the Geotechnical Engineering Division, ASCE 105: 1511-1524.

Schindler, C., Cuénod, Y., Eisenlohr, T. & Joris, C.-L. 1993. Die Ereignisse vom 18. April und 9. Mai 1991 bei Randa (VS) - ein atypischer Bergsturz in Raten. Eclogae Geologicae Helvetiae 86(3): 643-665.

Terzaghi, K. 1962. Stability of steep slopes on hard unweathered rock. Géotechnique 12: 251-270.

Willenberg, H., Spillmann, T., Eberhardt, E., Evans, K., Loew, S. & Maurer, H. 2002. Multidisciplinary monitoring of progressive failure processes in brittle rock slopes - Concepts and system design. In J. Rybár et al. (eds), Proceedings of the 1st European Conference on Landslides, Prague: 477-483. Lisse: A.A. Balkema.