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An integrated numerical analysis approach applied to the Randa rockslide

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ABSTRACT: Conventional limit equilibrium and numerical modelling slope analysis techniques have distinct advantages and disadvantages inherent in their respective methodologies. The analysis technique chosen will depend on the site conditions and the potential mode of failure identified. The engineer must be aware of all existing slope analysis tools and be cognizant of their limitations. Knowledge of these different methods is essential in view of the potential variation in the input parameters required and in the subsequent interpretation of the generated results. In complex cases the required analysis methodology may not involve the use of a single technique, but may require the integrated use of several conventional and numerical methods. One such example is the 1991 Randa rockslide in the southern Swiss Alps, where failure occurred in two distinct episodes, involved several complex mechanisms relating to geological, mechanical and hydrological processes, and for which no clear triggering mechanism can be asserted.

1 INTRODUCTION

To evaluate the potential hazard relating to an unstable rock slope, it is essential to understand the processes and mechanisms driving the instability. These mechanisms are often complex and act at depth, making the investigation and characterization of contributing factors difficult. This poses a problem in the analysis stage of the investigation as uncertainties arise concerning the analysis technique to be employed and what input data is required. Today, a vast range of slope stability analysis tools exist for both rock and mixed rock-soil slopes; these range from simple infinite slope and planar failure limit equilibrium techniques to sophisticated coupled finite-/distinct-element codes.

Conventional limit equilibrium and numerical modelling slope analysis techniques have specific advantages and disadvantages inherent in their respective methodologies. The analysis technique chosen will depend on the site conditions and the potential mode of failure identified. In complex cases the required analysis methodology may not involve the use of a single technique, but may require the integrated use of several conventional and numerical methods.

One such example is the 1991 Randa rockslide in the southern Swiss Alps, where failure occurred in two distinct episodes, involved several complex mechanisms relating to geological, mechanical and hydrological processes, and for which no clear triggering mechanism can be asserted. The complexity of the slide geometry presents significant limitations with respect to the applicability of two-dimensional solutions and the uncertainty with regard to the underlying failure mechanisms requires consideration of both continuum and discontinuum techniques. In addition, elements of brittle fracture development and progressive failure require the implementation of specialized state-of-the-art numerical modelling codes.

Using the Randa rockslide as a working example, this paper demonstrates the integration of several conventional and numerical techniques and presents an overview of the possibilities that exist with current commercial slope analysis packages. Emphasis will be placed on how the advantages of the different modelling tools can be maximized to provide optimal results with respect to visualization and comprehension of the processes and mechanisms contributing to instability.

2 THE 1991 RANDA ROCKSLIDE

The 1991 Randa rockslide (Fig. 1) involved the failure of approximately 30 million m^3 of rock in two separate events three weeks apart. Situated in southern Switzerland, 10 km north of Zermatt, the slide mass was made up of massive gneisses alternating with mica-rich paragneisses (Fig. 2) belonging to the Penninic St.-Bernhard nappe. The sliding rock mass is presumably limited by shallow dipping joints (Wagner 1991) and sets of steep near-vertical joints. Tension cracks on surface behind the 1991 slide scarp tend to preferentially open parallel to steeply dipping natural joints.

The slide occurred in two stages with the first slide occurring on April 18, 1991 and the second failing on May 9, 1991. Although no clear triggering mechanism could be resolved from the seismic and precipitation records, it was noted that failure coincided with a period of heavy snowmelt (Schindler et al. 1993). However, examination of the snow height and temperature records shows that this was not an exceptional event and that heavier snowmelts had been recorded in previous years (Eberhardt et al. 2001).

Figure 1. The 1991 Randa rockslide in the Canton Valais in southern Switzerland.

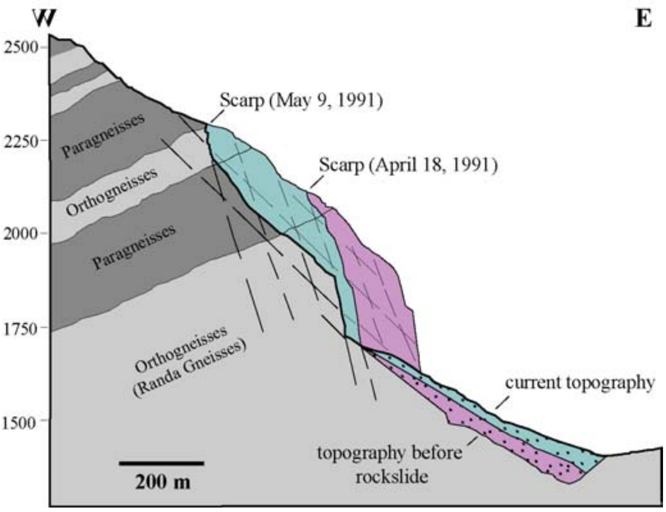

Figure 2. Cross-section showing the geology and geometry of the 1991 Randa rockslide before and after the two failure events (after Wagner 1991).

3 CONVENTIONAL METHODS OF ANALYSIS

3.1 Kinematic and limit equilibrium techniques

Conventional methods of rock slope analysis can be generally broken down into kinematic and limit equilibrium techniques (Tab. 1). Kinematic methods concentrate on the feasibility of translational failures due to the formation of daylighting wedges or planes. As such, these methods rely on the detailed evaluation of rock mass structure and the geometry of existing discontinuity sets that may contribute to block instability. This assessment may be carried out by means of stereonet plots and/or specialized computer codes which focus on wedge formation and key block theory. For example, the program DIPS (Rocscience 2001) allows for the visualisation and determination of the kinematic feasibility of rock slopes using friction cones, daylight and toppling envelopes. It is essential that the user is aware that such approaches do not consider failure modes involving multiple joints/joint sets or internal deformation and fracture. Discontinuity data and joint set intersections can, however, be imported into companion limit equilibrium codes (e.g. SWEDGE - Rocscience 2001) to assess wedge feasibility and the factor of safety against sliding.

Limit equilibrium techniques are routinely used in the analysis of landslides where translational or rotational movements occur on distinct failure surfaces. In general, these methods are the most commonly adopted solution method in rock slope engineering, even though many failures involve complex internal deformation and fracturing which bears little resemblance to the 2-D rigid block assumptions required by the analyses. However, limit equilibrium analyses may be highly relevant to simple block failure along discontinuities or rock slopes that are heavily fractured or weathered (i.e. soil-like).

All limit equilibrium techniques share a common approach based on a comparison of resisting forces/moments mobilized and the disturbing forces/moments. Methods vary, however, in the assumptions adopted in order to achieve a determinate solution. Considerable advances in commercially available limit equilibrium computer codes have taken place in recent years, including those that integrate:

- finite-element groundwater flow analyses (e.g. SEEP/W and SLOPE/W - Geo-Slope 2000);
- three-dimensionality (e.g. CLARAW - Hungr 1992);
- probabilistic analysis;
- ground support and reinforcement;
- unsaturated soil shear strength criteria; and
- greatly improved visualization, and pre-/post-processing graphics.

Table 1. Conventional methods of rock slope analysis (after Coggan et al. 1998).

Analysis Method	Critical Parameters	Advantages	Limitations
Stereographic and Kinematic	Critical slope and discontinuity geometry; representative shear strength characteristics.	Relatively simple to use; give initial indication of failure potential; may allow identification and analysis of critical key-blocks using block theory; links are possible with limit equilibrium methods; can be combined with statistical techniques to indicate probability of failure.	Only really suitable for preliminary design or design of non-critical slopes; critical discontinuities must be ascertained; must be used with representative discontinuity/joint shear strength data; primarily evaluates critical orientations, neglecting other important joint properties.
Limit Equilibrium	Representative geometry and material characteristics; soil or rock mass shear strength parameters (cohesion and friction); discontinuity shear strength characteristics; groundwater conditions; support and reinforcement characteristics.	Wide variety of commercially available software for different failure modes (planar, wedge, toppling, etc.); can analyse factor of safety sensitivity to changes in slope geometry and material properties; more advanced codes allow for multiple materials, 3-D, reinforcement and/or groundwater profiles.	Mostly deterministic producing single factor of safety (but increased use of probabilistic analysis); factor of safety gives no indication of instability mechanisms; numerous techniques available all with varying assumptions; strains and intact failure not considered; probabilistic analysis requires well-defined input data to allow meaningful evaluation.
Physical Modelling	Representative material characteristics; appropriate scaling factors.	Mechanisms clearly portrayed and results of analysis are a useful constraint for numerical modelling; centrifuge models able to investigate effects of time on failure mechanisms.	Simplistic groundwater simulation especially in rock; techniques do not allow for the effects of scale and *in situ* stress; centrifuges can be expensive.
Rockfall Simulators	Slope geometry; rock block sizes and shapes; coefficient of restitution.	Practical tool for siting structures; can utilize probabilistic analysis; 2-D and 3-D codes available.	Limited experience in use relative to empirical design charts.

3.2 Randa limit equilibrium back-analysis

Limit equilibrium analyses are best employed to provide either a deterministic factor of safety or a range of shear strength parameters at failure (i.e. a back-analysis). To demonstrate this, a back-analysis was performed for the Randa rockslide using the program SLIDE (Rocscience 2001). A 2-D slope geometry was assumed based on a section taken through the centre of the slide mass and along the line of failure. A water table was assumed based on observations at the time of failure and data collected from recent borehole drillings.

Given the complexity of the Randa rockslide failure surface and rock mass characteristics, the application of a conventional 2-D circular analysis (e.g. Bishop's simplified method) may be considered inappropriate. Figure 3 shows the results from the back-analysis performed using a non-circular slip surface block search routine (whereby the slide mass is divided into active and passive blocks). Back-calculated values provide a simple means of deriving initial shear strength estimates to be used in other more advanced modelling analyses. Results from this analysis show that for a failure surface closely matching that of the 1991 Randa failure, and assuming a rock mass unit weight of 27 kN/m^3, a cohesion of 1.5 MPa and friction angle of 40° produces a factor of safety of 0.99.

It should be stressed that the limit equilibrium analyses undertaken provide a range of c-ϕ values which would maintain limiting equilibrium along a predetermined structurally controlled failure surface within the rock mass. They do not represent the c-ϕ which would be involved in the generation of these failure surfaces. The relatively high values of cohesion may be indicative of the need for fracture across rock bridges on a micro-scale. Given the

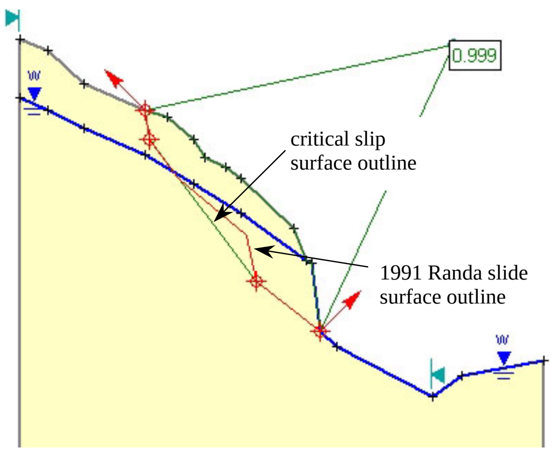

Figure 3. Limit equilibrium analysis performed using a non-circular slip surface block search routine (γ = 27 kN/m^3, c = 1.5 MPa, ϕ = 40º).

large number of assumptions incorporated into these analyses, and where it is necessary to include the stress state within the rock mass and/or the influence of complex deformation and brittle fracture mechanisms, numerical modelling techniques must be used.

4 NUMERICAL METHODS OF ANALYSIS

Many rock slope stability problems involve complexities relating to geometry, material anisotropy, non-linear behaviour, *in situ* stresses and the presence of several coupled processes (e.g. pore pressures, seismic loading, etc.). Advances in computing power and the availability of relatively inexpensive commercial numerical modelling codes means that the simulation of potential rock slope failure mechanisms could, and in many cases should, form a standard component of a rock slope investigation.

Numerical methods of analysis used for rock slope stability may be divided into three approaches: continuum, discontinuum and hybrid modelling. Table 2 summarizes these methods.

4.1 Continuum techniques

Continuum approaches used in slope analysis include the finite-difference and finite-element methods. Continuum modelling is best suited for the analysis of slopes that are comprised of massive intact rock, weak rocks, or heavily fractured rock masses. Most continuum codes incorporate a facility for including discrete fractures such as faults and bedding planes but are inappropriate for the analysis of blocky mediums. Numerous 2-D and 3-D commercial codes are available, which often include a wide choice of constitutive criteria including elastic, elasto-plastic, strain-softening and visco-plasticity.

Although rock slope stability problems rarely meet the conditions relevant to that of a continuum, continuum modelling has the advantage of being able to efficiently model progressive and time-dependent failure mechanisms. These factors must be considered if rock slope stability analyses are to evolve beyond current phenomenological-based practices and move towards more mechanistic-based approaches. Experiences at Randa and elsewhere in Switzerland have aptly demonstrated the need for this change (Eberhardt et al. 2001). To do so, natural rock slope failures should be viewed as the response to the progressive accumulation of events with time that act to degrade the equilibrium state of the slope (e.g. heavy rainfall event, spring snowmelt, etc.), with each event bringing the slope nearer to failure. Limit equilibrium analysis and other phenomenological approaches only provide a snapshot of the conditions at the moment of failure, and as such they provide a simplified answer as to why the slope failed, but not within the context of time as to “why now?”.

Table 2. Numerical methods of rock slope analysis (after Coggan et al. 1998).

Analysis Method	**Critical Parameters**	**Advantages**	**Limitations**
Continuum Modelling (e.g. finite-element, finite-difference)	Representative slope geometry; constitutive criteria (e.g. elastic, elasto-plastic, creep, etc.); groundwater characteristics; shear strength of surfaces; *in situ* stress state.	Allows for material deformation and failure (factor of safety concepts incorporated); can model complex behaviour and mechanisms; 3-D capabilities; can model effects of pore pressures, creep deformation and/or dynamic loading; able to assess effects of parameter variations; computer hardware advances allow complex models to be solved with reasonable run times.	Users must be well trained, experienced and observe good modelling practice; need to be aware of model and software limitations (e.g. boundary effects, meshing errors, hardware memory and time restrictions); availability of input data generally poor; required input parameters not routinely measured; inability to model effects of highly jointed rock; can be difficult to perform sensitivity analysis due to run time constraints.
Discontinuum Modelling (e.g. distinct-element, discrete-element)	Representative slope and discontinuity geometry; intact constitutive criteria; discontinuity stiffness and shear strength; groundwater characteristics; in situ stress state.	Allows for block deformation and movement of blocks relative to each other; can model complex behaviour and mechanisms (combined material and discontinuity behaviour coupled with hydro-mechanical and dynamic analysis); able to assess effects of parameter variations on instability.	As above, user required to observe good modelling practice; general limitations similar to those listed above; need to be aware of scale effects; need to simulate representative discontinuity geometry (spacing, persistence, etc.); limited data on joint properties available (e.g. jk_n, jk_s).
Hybrid/Coupled Modelling	Combination of input parameters listed above for stand-alone models.	Coupled finite-/distinct-element models able to simulate intact fracture propagation and fragmentation of jointed and bedded rock.	Complex problems require high memory capacity; comparatively little practical experience in use; requires ongoing calibration and constraints.

4.1.1 Randa stress and progressive failure analysis

Two-dimensional continuum modelling was performed using the finite-element code Visage (VIPS 2001) to examine the natural stress distribution at Randa due to topography as well as different possible stages of stress-induced progressive failure. Although progressive failure in brittle crystalline rock masses like Randa involves processes more related to the failure with time of intact rock bridges between existing natural joints or asperities between interlocked joint surfaces, continuum modelling can be used to examine the evolution of stresses, strains and plastic yielding within the rock mass (i.e. joint and intact rock behaviour modelled as one). As such a preliminary set of 2-D models were run to simulate strength degradation with time leading to failure.

The finite-element model incorporated 833 9-noded quadrilateral elements (3465 nodes). Stresses were initialized assuming gravity loading (i.e. $\sigma_H / \sigma_V = 0.33$) and a homogeneous, isotropic rock mass ($\gamma = 27$ kN/m^3, E = 25 GPa, $\nu = 0.25$). Pore pressures were not included in the analysis, although they will be in future stages of the study. Similarly, complexities relating to material heterogeneity and anisotropy based on field investigations and *in situ* testing will be incorporated in later stages.

Following the initialization of the *in situ* stress state, an elasto-plastic constitutive criterion was assigned to the slope materials assuming a Mohr-Coulomb yield criterion. Strength values were initially set to those for a generally coherent granitic-gneiss rock mass (c = 20 MPa, $\phi = 40°$, $T_o = 1$ MPa). The bulk rock mass cohesion was then gradually decreased to simulate the progressive degradation of rock mass strength with stress and time due to brittle fracturing (Fig. 4; Eberhardt et al. 1999). Modelling stages correspond to reductions in rock mass cohesion of 50%, 75% and 90%. Figure 5 demonstrates the transition of stable slope conditions to those of shear failure by showing the evolution of shear strains over the last two stages of progressive failure. Similar to results derived from the limit equilibrium analysis, failure occurs when the cohesion decreases to 1.0 MPa (i.e. 90% of intact strength). Examination of the 2-D finite-element results with respect to the actual shape of the failure surface show that although the lower section of the yield surface agrees closely, the upper section appears to extend further back beyond the observed failure back-scarp (Fig. 6). Interestingly, this upper region of the Randa rock slope has shown signs of instability and is the subject of an on-going investigation (see this volume Willenberg et al. 2002).

Progressive failure can thus be depicted through these models as the gradual decline in rock mass strength, leading to plastic yielding along a shear zone delimiting the failure surface. It should be emphasized, however, that it is unlikely that large sections of the rock mass would uniformly experience a 90% strength degradation, and that these results represent a simplified representation of the rock mass degradation that correlates with conditions that were probably occurring in the vicinity of the failure surface.

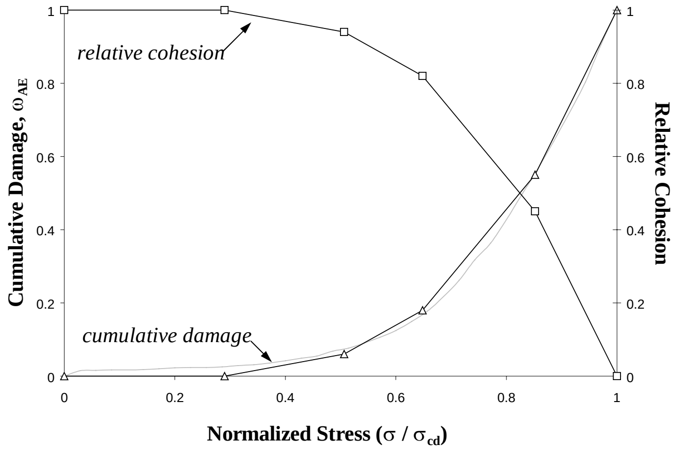

Figure 4. Cumulative damage model showing decrease in material cohesion due to increasing stress and stress-induced brittle fracturing (after Eberhardt et al. 1999).

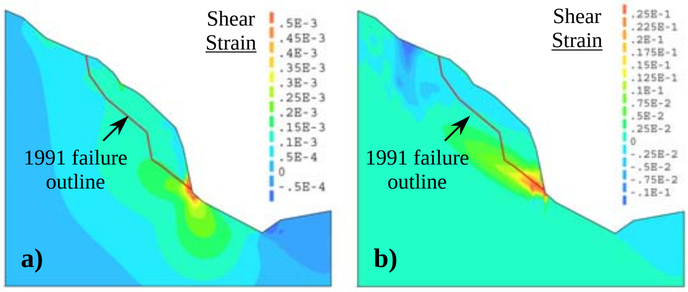

Figure 5. Evolution of shear strains over two stages of progressive intact strength degradation leading to slope failure: (a) 75%; (b) 90%.

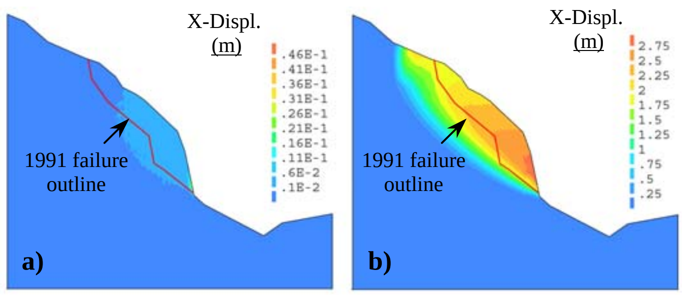

Figure 6. Evolution of horizontal displacements over two stages of progressive intact strength degradation leading to slope failure: (a) 75%; (b) 90%.

4.2 Discontinuum techniques

Discontinuum methods treat the rock slope as a discontinuous rock mass by considering it as an assemblage of rigid or deformable blocks. The analysis includes sliding along and opening/closure of rock discontinuities as controlled by the joint properties (normal and shear stiffness, cohesion, friction, etc.). Discontinuum modelling constitutes the most commonly applied numerical approach to rock slope analysis, the most popular method being the distinct-element method. Distinct-element codes such as UDEC (Itasca 2001) use a force-displacement law specifying interaction between the deformable joint bounded blocks and Newton's second law of motion, providing displacements induced within the rock slope. UDEC is particularly well suited to problems involving jointed media and has been used extensively in the investigation of rockslides.

4.2.1 Randa discontinuity-controlled failure analysis
Figure 7 presents the results of an initial UDEC discontinuum analysis performed for the Randa rockslide. In the analysis, discontinuities are assumed to be fully persistent and interconnected. Material properties for the deformable bocks are the same as those used in the continuum models presented in the previous section. The difference between the two analyses (i.e. continuum and discontinuum) is that in the case of the discontinuum model, failure is controlled by pre-defined discontinuities.

If the Randa failure is modelled as having occurred along a set of interconnected discontinuities, this would require the presence of a set of fully persistent joints dipping at an angle of approximately 50º out of slope. This corresponds to the dip of the failure surface and key discontinuities reported by Wagner (1991) as having promoted failure (Fig. 2). Pre-failure stability, however, would require an exceptionally high friction angle of more than 50º along these discontinuity planes. Furthermore, geometrical inspection suggests that the persistence of

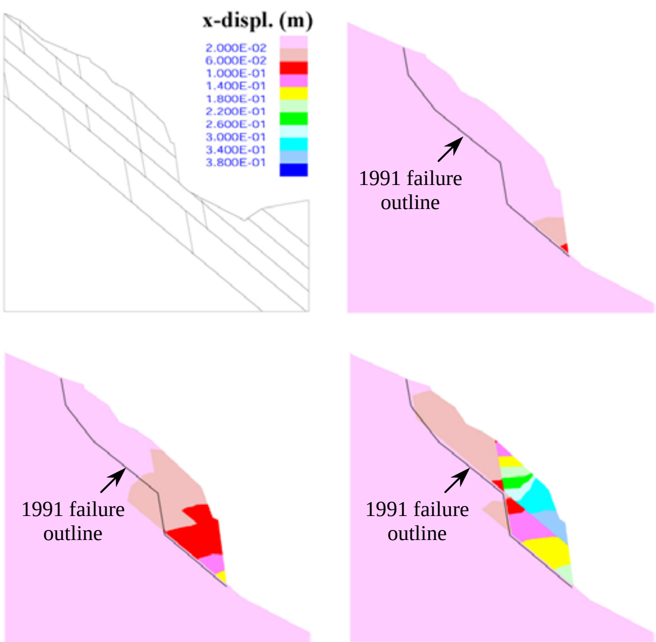

Figure 7. Evolution of horizontal displacements and progressive failure over several stages of discontinuity strength degradation (i.e. through joint cohesion reduction).

these discontinuities would have to be in excess of 500 m. Field studies conducted as part of this study indicate that highly persistent discontinuities are rare or are difficult to ascertain. This suggests that failure was more likely associated with a shear surface that developed in a step-path fashion along discontinuities of more limited persistence, separated by rock bridges (intervals of intact rock providing elements of cohesive strength) i.e. failure by progressive development of a shear surface.

Based on this conceptualization, the UDEC analysis was carried out assuming a joint friction of 40º and joint cohesion of 5 MPa (representing the strength provided by intact rock bridges). Models then simulated progressive failure through gradual reduction of joint cohesion using a similar methodology to that described for the continuum models. Results show that failure initiated when joint cohesion was reduced to 1 MPa, catastrophic failure occurred at 0.1 MPa (Fig. 7). Further analysis is planned in later stages of this research work to more accurately portray discontinuity data sets collected in the field in terms of spacing, persistence and connectivity.

4.3 Hybrid techniques

Although both continuum and discontinuum analyses provide useful means to analyze rock slope stability problems, complex failures like Randa involve mechanisms related to both existing discontinuities and the brittle fracturing of intact rock. Coupled finite-/distinct-element codes are now available which allow for the modelling of both intact behaviour and the development of fractures through adaptive remeshing techniques (ELFEN 2001). These methods use a finite-element mesh to represent either the rock slope or joint bounded block coupled together with discrete elements able to model deformation involving joints. If the stresses within the rock slope exceed the failure criteria within the finite-element model a crack is initiated. Adaptive remeshing allows the propagation of the cracks through the finite-element mesh to be simulated.

Preliminary results for Randa using 2-D hybrid modelling techniques are shown in Figure 8. Current models have begun by using both the topography and the observed failure plane as part of the input geometry. As more field data and laboratory testing are performed to constrain the modelling input, work will be extended towards modelling the progressive development of the failure plane, in 2-D and 3-D. These models will work towards understanding how existing discontinuities and stress-induced brittle fracturing work together to promote rock slope instabilities. Furthermore, through such hybrid techniques, modelling can now and will be extended to model the complete failure process from initiation, through transport to deposition.

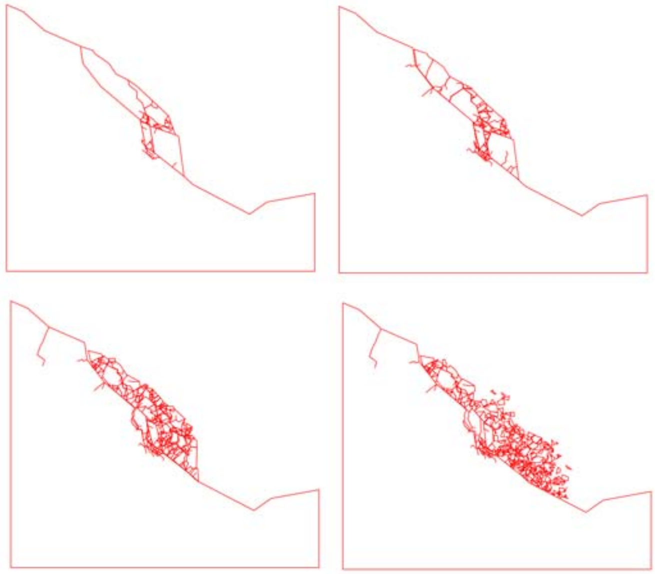

Figure 8. Hybrid finite-/discrete-element rockslide analysis showing several progressive stages of brittle failure.

5 FUTURE DEVELOPMENTS

The analysis of complex rockslides can now be undertaken routinely using state-of-the-art numerical modelling codes on desktop computers. If the benefits of these methods are to be maximized then it is essential that field data collection techniques are more responsive to advances in design capabilities. Current data collection methodologies have changed little over the last decade and are aimed towards limit equilibrium analysis. Data including rock mass characteristics, *in situ* deformation and pore pressures must be collected in order to allow more realistic modelling of rock slope failure mechanisms.

The next decade holds enormous potential in our ability to model the complete failure process from initiation, through transport to deposition. This will provide a far more rigorous understanding on which to base risk assessment. Practitioners and researchers must make the effort to think beyond the use of stand-alone computers and embrace the rapidly developing technology of parallel computing. The advent of virtual reality programming will allow the engineer to convey the results of simulations in a powerful and graphically efficient manner. It is essential however that quality/quantity of both input data and instrumentation data for modelling purposes be improved concomitantly in order to provide the requisite validation.

Although only in its initial stages, this study and the modelling results presented demonstrate the benefits of integrating conventional and numerical modelling techniques in order to efficiently capitalize on the strengths of the different methodologies available for slope stability analysis. As such it is vital that good modelling practices be observed and followed. This then means that not only must consideration be given to integrating different numerical techniques, but integrating numerical modelling with site investigation, laboratory testing and *in situ* monitoring campaigns as well (e.g. Table 3).

Table 3. Integration of slope instability investigation methods.

Investigation Method	Parameters Investigated
Desk Study	Previous investigations, literature review, available data.
Site Investigation	Field mapping, scanline surveys, observations of instability, hydrogeological observations.
Laboratory Testing	Determination of rock mass strength and material behaviour including discontinuity shear strength evaluation.
Conventional Stability Analysis	Kinematic feasibility, deterministic limit equilibrium (i.e. Factor of Safety), probabilistic sensitivity analysis.
Numerical Modelling	Simulation of slope deformation and stability, analysis of progressive failure and shear surface development.
Field Monitoring	Monitoring of 3-D deformations, groundwater and microseismicity.

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