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A "Total Slope Analysis" methodology applied to an unstable rock slope in Washington, USA

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ABSTRACT

A "Total Slope Analysis" methodology, that combines several numerical techniques, is adopted to investigate an unstable rock slope in Washington State, USA. For this specific study, the distinct-element code UDEC is used to assess the stability and potential failure volume of the rockslide. Once the potential rockslide volume has been estimated and failure mechanism assessed, the runout path, distance and velocity are assessed using the dynamic or rheological flow model DAN3D. Site investigation and data reconnaissance plays an important role for both stages in the "Total Slope Analysis", including outcrop mapping, aerial photograph interpretation, scanline joint surveys and 3-D laser scanning.

The results of the "Total Slope Analysis" can be directly applied to assessment and mitigation of the landslide hazard, greatly aiding engineering judgment by providing key qualitative and quantitative insights into the risk analysis.

INTRODUCTION

A landslide risk analysis involves two components: a hazard analysis and an analysis of the consequences should the landslide occur. A key aspect of the hazard analysis is the *landslide characterisation*, which involves the prediction of the onset of extremely rapid failure (i.e. character of failure, location, size) and a quantitative estimate of post-failure motion ("runout"), including the travel path, travel distance and velocity (Hungr et al. 2005). These parameters provide important input into the consequence analysis because they define the potential damage to which an exposed structure or person may be subjected.

Despite considerable advances in understanding landslide mechanisms through numerical modelling simulations, accurate landslide characterisation remains difficult (Hungr et al. 2005). These difficulties are caused, in part, by the large degree of uncertainty regarding the physical properties of an unstable rock mass, including: the dimensions and geometry of controlling features; mechanical properties of the discontinuities and rock mass; and the influence of geologic factors such as groundwater and tectonic stresses. An analysis method that combines several existing techniques can be used to reduce this uncertainty by utilising the strengths inherent in each technique.

This paper presents a methodology that combines several numerical techniques, linking failure initiation processes to runout, termed "Total Slope Analysis" (e.g. Stead and Coggan 2005). The methodology is applied to an unstable rock slope investigation at the Afternoon Creek in Washington State, USA, with complex topographic and structural controls, both at the source area and along potential runout travel paths. The complexity of the case poses a challenge to the selection and design of an appropriate mitigation scheme to counter the hazard. For this specific study, the distinct element code UDEC (Itasca 2000) is first used to assess the stability state and potential failure volume. These results are then used to analyse the runout path, distance and velocity using the numerical dynamic or rheological flow code DAN3D (McDougall and Hungr 2004). This coupled assessment is first performed as a back-analysis of a recent rockslide event from the source area. In the second part of the analysis, the results from the back analysis are used to constrain forward predictions of the existing hazard. This is schematically illustrated in Fig. 1.

The paper also describes the data collection methodology and input required for both phases of the "Total Slope Analysis". This includes the use of outcrop mapping, aerial photograph interpretation, scanline joint surveys and 3-D laser scans. The Afternoon Creek case study is used to develop the methodology and is included as an example; all results are preliminary.

BACKGROUND

The "Total Slope Analysis" methodology presented here is developed and calibrated by means of an earlier rock avalanche at the study site. The event occurred in November, 2003 and involved approximately 750,000 m^3 of rock. Most of the rock avalanche debris travelled more than 300 m in elevation down a steep slope and landed in the narrow, steep-walled Afternoon Creek valley. The leading edge of the material flowed approximately 500 m in horizontal distance from the centre of the source area, but did not reach the nearby Washington State Route 20 (SR 20) highway. The dry, very-coarse granular slide deposit is composed of orthogneiss boulders ranging in size from one metre to 25 m diameter. The source area for the failure lies at the top of a steep ridge. This topographic control allowed lesser amounts of the rock avalanche debris to travel down the back side of the ridge into the Falls Creek and Falls Creek Chute (Fig. 2 and 3). This debris travelled more than 600 m in elevation down a steep slope and landed on SR 20, an important route through the North Cascade Mountains. Portions of the roadway and guardrail were destroyed and boulders up to 3 m in diameter were deposited on the road (Fig. 2). The slope continues to threaten SR20 due to potentially unstable material at the top ridge. This paper focuses on runout in the Afternoon Creek.

Field observations indicate that failure occurred in three stages. Initial collapse occurred in the lower portion of the failed zone. This material moved directly down slope into the Afternoon Creek. The event unloaded the toe of larger, more competent blocks. These slid on highly persistent planes initially parallel or oblique to the Afternoon Creek, forming the second, largest phase of failure. Rock fall and (or) toppling in the upper portion of the failure zone followed the removal of lateral support provided by the large blocks. This allowed some of the rock fall or topple material to travel down the west side of the ridge into the Falls Creek and Falls Creek Chute, where the much steeper descent path enabled the debris to reach the highway below (Fig. 3). It is not known whether the three failure phases occurred in a continuous sequence or independently.

BACK-ANALYSIS OF THE PREVIOUS ROCK AVALANCHE EVENT

Failure initiation

Phase one of the Total Slope Analysis involved the back-analysis of the most recent rockslide event using two methods: limit equilibrium (after Hoek and Bray 1991) and distinct element (UDEC; Itasca 2004) (Fig. 4). Stead et al. (2001) summarise the advantages and limitations inherent in these different methodologies, while Starfield and Cundall (1988) outline the proper application of numerical modelling to rock mechanics problems. The Afternoon Creek rock slope is a data-limited problem in the sense that not all relevant data can be obtained due to difficult access to rock-face outcrops. Therefore, exact models cannot be constructed. So rather than using the models to recreate the exact geometry and stress-strain scenario, they are used in the form of parametric studies, to better understand (1) the physical properties of the slope, and (2) the failure mechanism. The multi-directional movement of the failed mass in this case study warrants a three-dimensional analysis; however for the purpose of developing this methodology the analysis is attempted with two-dimensional methods.

The objectives of back-analysis of the failure are the following:

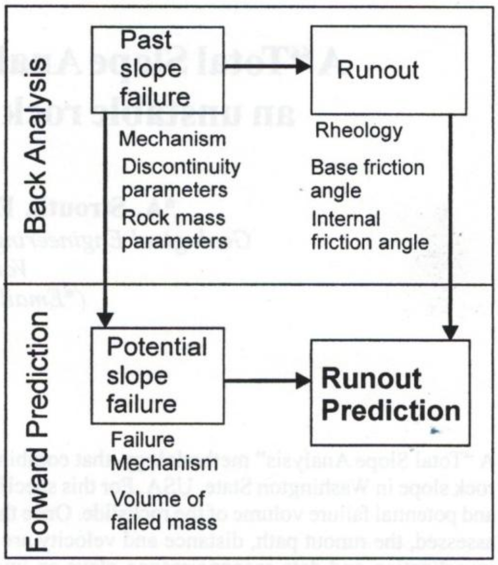

Fig. 1: "Total Slope Analysis" flowchart

1. Constrain estimates of the physical properties of the slope:
   a. Slope geometry,
      i. Attitude of discontinuities,
      ii. Location and orientation of weak zones,
   b. Mechanical properties.
2. Identify the failure mechanism:
   a. Determine how the mechanism affects the volume of failed material,
   b. Determine how the mechanism affects the post-failure movement of the material.

An understanding of the failure mechanism is important when deciding the type of runout analysis to perform and the methodology to be followed. For example, block toppling often does not involve the simultaneous failure of the entire unstable rock mass, but instead involves gradual, retrogressive, piecemeal disintegration; in contrast, translational rock block or wedge failure usually occur as single extremely rapid events (Hungr and Evans 2004). When slope failure occurs in two or more stages, the runout should be modelled in an equal number of stages - each stage involving a portion of the total volume initiating from the correct location in the source zone. Additionally, an understanding of the failure mechanism is important in the forward prediction phase of the "Total Slope Analysis". This knowledge helps the analyst to choose a suitable method for failure analysis and, given understanding of the slope geometry, provides upper and lower limits on the range of likely failed volumes.

Simplified stability analysis

A limit equilibrium analysis was initially performed to test the sensitivity of the slope problem to such parameters as joint cohesion and friction angle, and pore-water pressure.

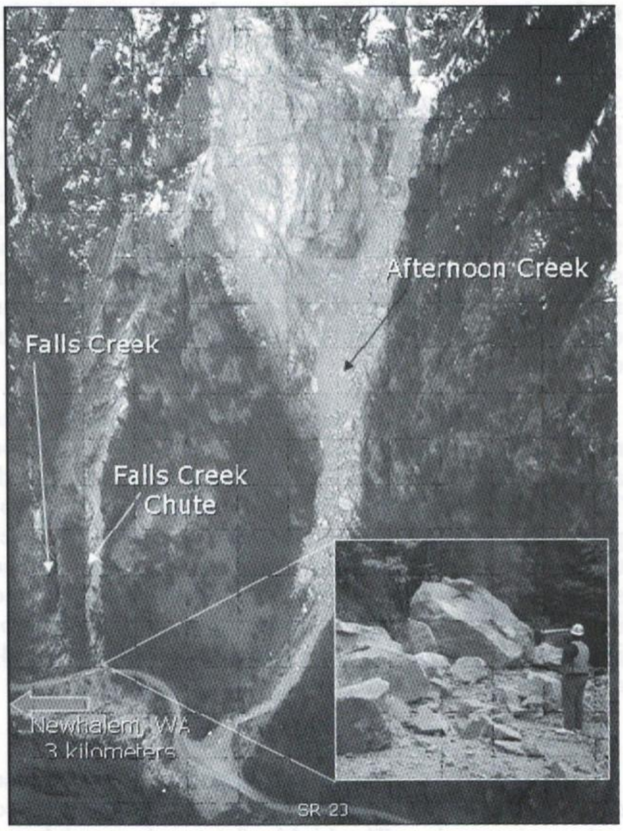

Fig. 2: Afternoon Creek rockslide above SR 20 near Newhalem, WA. Photograph provided by John Scurlock, Concrete, WA.

This analysis was based on the solution for planar failure described in Hoek and Bray (1991). The above noted parameters were systematically varied and the sensitivity to each parameter noted to control the subsequent development of a simplified distinct-element model.

The simplified or idealised distinct-element model built in UDEC, incorporated the topographic surface from the 'before' DEM, approximate joint sets and discontinuities that were visible from photographs, and roughly estimated material properties. The model was run several times, varying the discontinuity orientation and spacing, changing material properties, and adjusting the ratio of major and minor principal stresses. The purpose of this analysis was to identify shortcomings in the model design and procedure and to guide the plan for data collection by identifying which parameters have the most influence on the analysis. Two cross-sections of the slope were considered. Portions of the failed mass moved approximately parallel to each cross-section in the respective areas of the failure zone (Fig. 5).

This analysis showed that the shape of the failure surface is only mildly sensitive to changes in the in-situ stress state, but highly sensitive to changes in the shape and thickness of the highly-fractured zone. Also changes in dip of the steep, persistent joint set (plane B), strongly controls the volume of failed material. Therefore the shape and thickness of the highly-fractured zone and orientation of the plane B joint set became the focus for further data collection and parametric study.

Data collection

Data collection involved a review of existing information and field surveys. Digital Elevation Models (DEMs) defined the pre-failure topographic surface (i.e. the surface being modelled) and the post-failure surface after the November, 2003 rockslide. The DEM from before the rockslide was produced by the U.S. Geological Survey based on aerial photographs, whereas that from after the rockslide was created from aerial LiDAR flown one month after the event.

Due to the steep, inaccessible slope, a traditional statistical joint survey was not possible in the upper portion

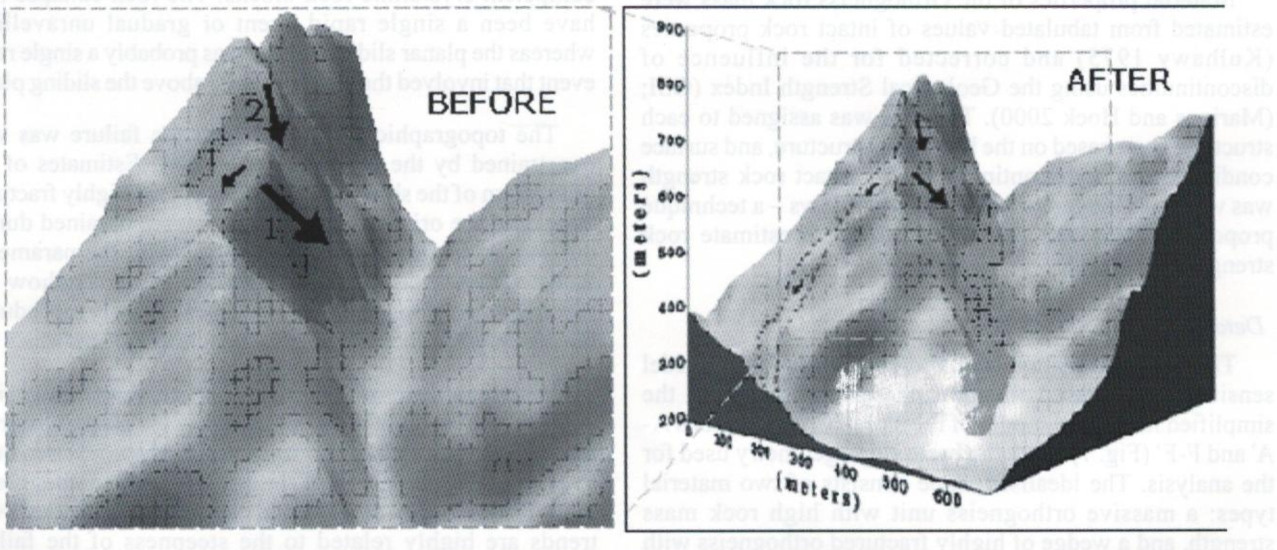

Fig. 3: Digital elevation model of the Afternoon Creek rockslide. Unstable mass is shown in gray. Arrows indicate direction of movement. Numbers indicate two phases of movement. Dotted blue line indicates the extent of path.

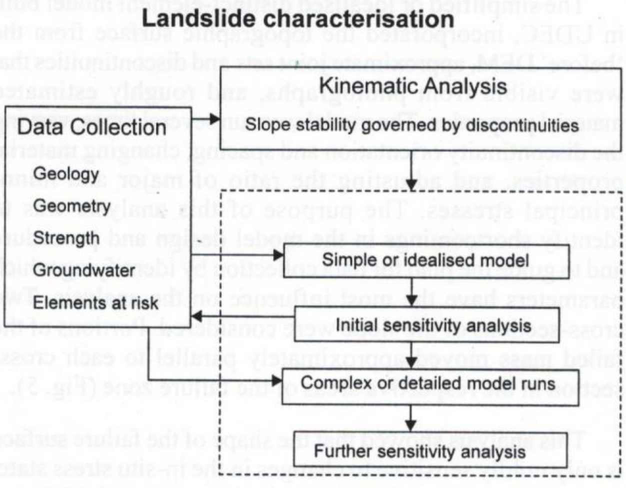

Fig. 4: Flow chart of the landslide characterisation process

of the rock mass where slope failure occurred. A scan line survey was completed in the lower portion of the slope and the orientation and spacing of the principal joint sets were extrapolated to the upper reaches of the slope. These estimates were roughly verified with photographs; however it is recognized that the jointing pattern in the lower section may not be relevant to the upper reaches of the slope. In order to counter these deficiencies, a 3-dimensional laser scanning survey was done. The result of each laser scan is a three-dimensional point cloud of the slope, with the average point spacing ranging from 5 cm to 50 cm (Fig. 6). Joint planes were automatically and manually identified in the point cloud with the software Split-FX 1.0. The orientation of the joint planes was then calculated by the software and displayed in the form of a stereonet. Two persistent, regularly spaced joint sets were recognised in the investigation and considered in the analysis.

Material properties of the Orthogneiss rock mass were estimated from tabulated values of intact rock properties (Kulhawy 1975) and corrected for the influence of discontinuities using the Geological Strength Index (GSI; (Marinos and Hoek 2000). The GSI was assigned to each structural zone based on the lithology, structure, and surface conditions of the discontinuities. The intact rock strength was verified by means of rock hammer blows – a technique proposed by Marinos and Hoek (2000) to estimate rock strength in the field.

Detailed stability modelling

The new information acquired during the initial model sensitivity and data collection phases was added to the simplified numerical model of the slope. Cross-sections A-A' and F-F' (Fig. 7) show the basic slope geometry used for the analysis. The idealised slope consists of two material types: a massive orthogneiss unit with high rock mass strength, and a wedge of highly fractured orthogneiss with relatively low rock mass strength. The shear zones that separate the two material types daylight and are visible at surface; however their orientation at depth is uncertain due to their undulating nature. In-situ boundary stresses were assigned so that vertical, horizontal, and out-of-plane stresses were initially equal. During this stress initialisation process, an elastic constitutive model was used, and the intact rock and joint properties were set high to prevent plastic yielding or failure along joint planes from occurring. After this stress initialisation step, all zones were changed to a 'Mohr-Coulomb' elasto-plastic constitutive model. The strength of the rock mass and discontinuities was then incrementally decreased to represent strength degradation due to weathering, and groundwater flow since deglaciation of the valley.

Key parameters were then systematically varied in the model until the reaction of the modelled slope and shape and location of the actual failure surface was reproduced. The model was run several times changing a range of parameters to identify potential failure mechanisms. After the optimal potential failure mechanism was defined the slope geometry was constrained. Parameters that were varied include: (1) in-situ state of stress; (2) dip and spacing of joint sets; (3) dip and location of bounding shear zones; (4) rock mass strength; and (5) discontinuity strength.

Results

Numerical analysis of the first stage of failure shows the development of a shear failure surface in the highly fractured orthogneiss wedge. The highly fractured material and competent blocks resting on this material displace towards the Afternoon Creek (Fig. 8). Following removal of this part of the rock mass, through failure, the toe of a highly persistent planar joint set is unloaded, allowing planar sliding to become kinematically feasible. In summary, the failure mechanism involved shearing and collapse of a highly fractured zone near the toe of the slope, followed by a second phase of planar sliding involving large joint-bounded competent crystalline rock blocks. The rock collapse may have been a single rapid event or gradual unravelling, whereas the planar sliding failure was probably a single rapid event that involved the entire volume above the sliding plane.

The topographic profile through the failure was well constrained by the aerial LiDAR DEM. Estimates of the orientation of the shear zones that bound the highly fractured zone, and the orientation of plane B were obtained during the data collection phase and verified with the parametric study. Cross-sections A-A' and F-F' (Fig. 7) show the reconstructed slope geometry that most-closely reproduced the observed failure surface in the numerical analysis.

With regards to rock mass properties, limit equilibrium analysis shows that the factor of safety for planar failure is most sensitive to changes in joint cohesion, less sensitive to changes in slope height, and insensitive to changes in joint friction angle for this slope geometry. These sensitivity trends are highly related to the steepness of the failure surface, the angle of which significantly exceeds that of realistic joint friction values. As such, joint cohesion,

Fig. 5: Map of rock slide area showing location of cross-sections and failed zone

provided in the form of intact rock bridges, must have played an important pre-failure role in terms of providing strength and stability. Further constraints on these properties were derived through the numerical modelling back-calculation exercise. The strongest rock mass properties, including both joint and rock mass cohesion and friction, for which failure could occur are given in Table 1, and represent the best estimates of the rock mass mechanical properties at the time of failure.

Failure runout

The numerical dynamic or rheological flow model DAN3D (McDougall and Hungr 2004) was used to provide understanding of material behaviour during runout and to analyse the runout path, distance and velocity of the rock avalanche. The model is based on depth-averaged, hydrodynamic theory. The inputs required for the dynamic runout analysis include the material properties along the runout path, the material properties of the runout material (dependent on the chosen rheology), and the volume of the initial sliding mass.

The objectives of the back-analysis of runout are the following:

1. Determine a rheology that approximates the behaviour of the flowing mass
2. Obtain reasonable estimates of relevant material parameters (depending on the rheology)
   a. Basal bulk friction angle
   b. Internal bulk friction angle

Model setup

The source zone and rock avalanche deposit were identified and delineated using aerial photograph analysis and field observation. The thickness, area and volume of the source zone and deposit, were quantified by comparing the DEMs from before and after the rock avalanche. The *before* DEM was subtracted from the *after* DEM to create a difference map; areas with decreased mass are part of the source zone, while areas with increased mass are part of the avalanche deposit. Verifying these zones through photographs and field observation, the deposit and source zones were individually outlined and the volume of each zone calculated. The volume of the source was approximately 641,000 m^3 while the volume of the deposit was approximately 868,000 m^3, corresponding to a bulking volume increase of 135%.

Input files for the DAN3D analysis were prepared from the DEMs. The necessary input files for this analysis include a DEM of the source (created from the difference map), and a DEM of the runout path (created from the *before* DEM). A 135% bulking factor was applied to the source to account for volume increase caused by fragmentation of the failed mass.

Based on previous experience (e.g. Hungr and Evans 1996), a dry-friction rheology was assumed for this analysis given the very-coarse granular nature of the material. As such, the only required material parameters for the associated constitutive model are the internal friction angle and a basal friction angle. The internal bulk friction angle is related to the angle of repose, which was measured in the field. The basal bulk friction angle is the average friction angle along the entire avalanche path at the interface between the path and the moving mass. This value depends on the hard-to-quantify effects of several parameters including composition of the path and moving mass, and the velocity of the moving mass; an iterative trial and error method is used during the modelling process to help determine this value (McDougall and Hungr 2004). Each friction angle was systematically varied until the results of the model most closely reproduced the runout distance, shape of the deposit, distribution of mass, and path extent of the actual slide.

One hypothesis mentioned earlier is that failure occurred in two major stages, followed by minor rock fall. To test this hypothesis the source was divided into two failure phases and the runout of each phase modelled sequentially. The total failed volume of the two-phase analysis is equivalent to the total failed volume of the single-phase analysis. The first failure phase was allowed to run out until the entire mass came to rest. Then the second phase was allowed to run out over the deposit of the first failure phase material.

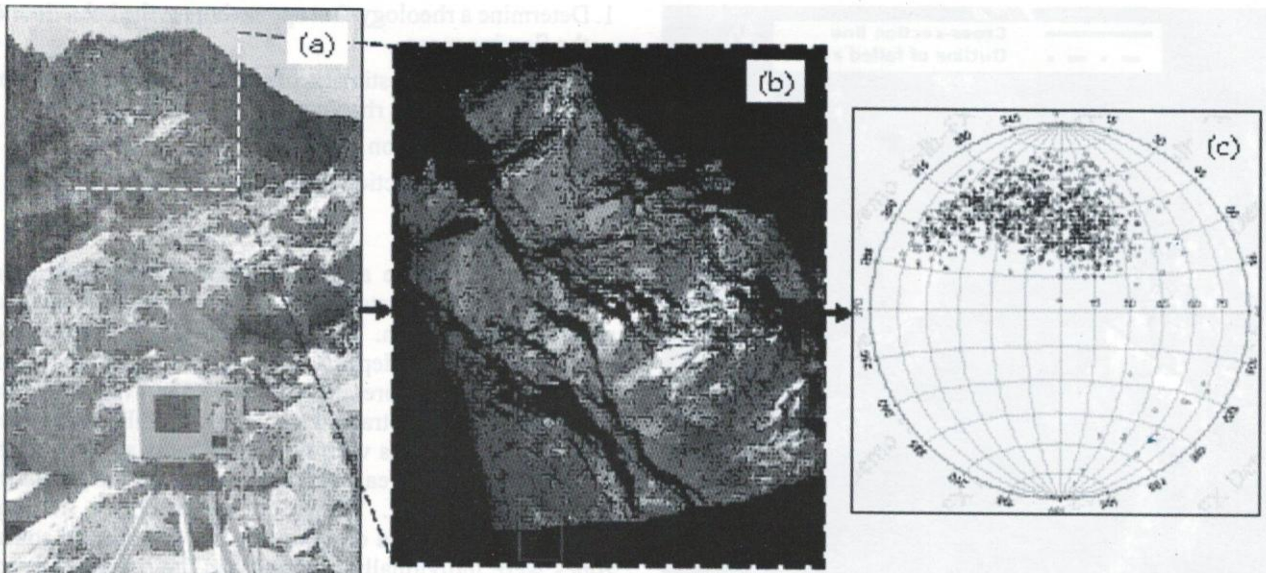

Fig. 6: Data collection with a 3-D laser scanner. (a) field set-up location, Afternoon Creek, WA. (b) Afternoon Creek point cloud. (c) Stereonet displaying automatically generated joint set orientation data from the point cloud. Point cloud and stereonet images developed using Split-FX software.

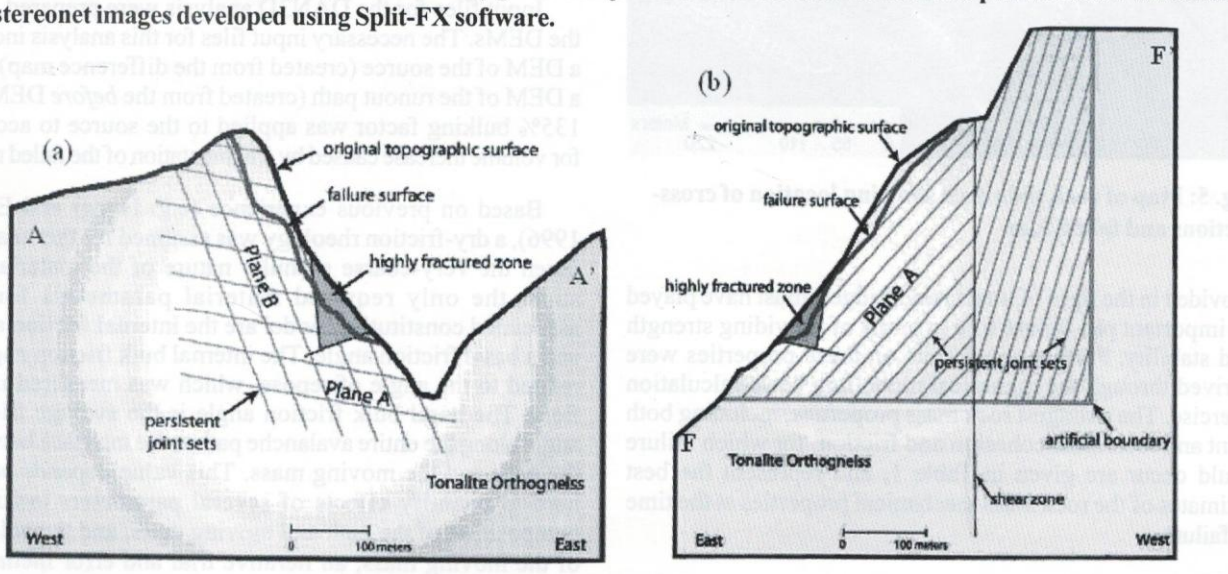

Fig. 7a: Cross-section A–A'; (b) Cross-section F–F'

The results of this two-phase analysis were compared to the single-phase analysis.

Results

The rock avalanche material is primarily blocky orthogneiss boulders ranging in size from less than 1 m to 25 m in diameter. There was no silt or clay and negligible sand-sized particles observed in the deposit. Cobble-sized material dominates near the canyon walls below the highly-fractured zone. The average angle of repose of the deposit is approximately 37°. This was the initial estimate for both the basal and internal friction angles. However, through back calculation, the frictional strength parameters that most closely reproduced the physical characteristics of the actual runout event (Fig. 9) are:

Basal Bulk Friction Angle: 37°

Internal Bulk Friction Angle: 40°

Although the extent and shape of the model and actual deposit are similar, there are several differences, including the following: (1) too much spreading after the debris exits the narrow Afternoon Creek canyon; (2) centre of mass has travelled too far; and (3) run-up on the opposite valley wall is too high. One reason for these differences is the assumption that the entire source volume fragments immediately when the model simulation is started. Therefore the internal pressures of the mass are much larger in the

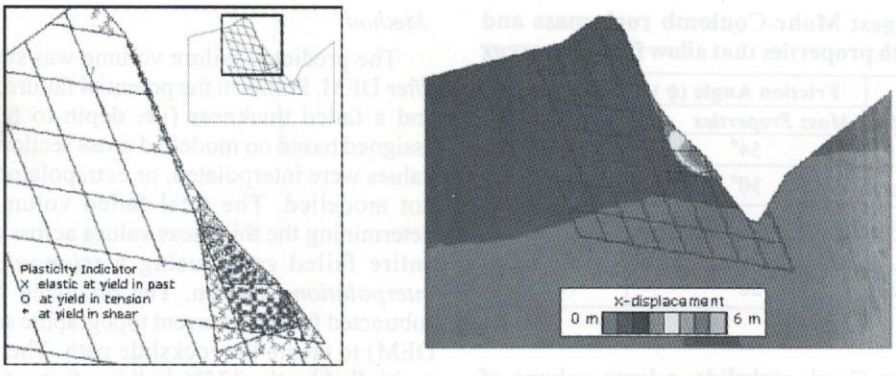

Fig. 8: UDEC numerical modeling results of cross-section A–A'

model than they would be in reality. These high pressures increase the momentum of the entire flowing mass causing the excessive run-up, excessive spreading, and excessive runout of the centre of mass.

A second explanation for these differences is that a single *average* value was used for both the basal and internal friction angles. In reality, these parameters vary along the entire path. For example, the basal friction angle may be at a maximum at the top of the runout path, in the failure zone, where it is approximately the same as the slope of the failure surface for the fragmented material (or the joint friction angle for the competent blocks that failed by planar sliding). The basal friction angle may decrease during runout. Varying the friction angle from a maximum value at the beginning of the path to a minimum value near the end of the path may produce better results.

When the runout is simulated in two phases using exactly the same friction parameters and rheology, the results of the model more closely resemble the real event. The key differences between the modelled and actual event, as previously listed (i.e. spreading, run-up, centre of mass travel distance), still exist in the two-phase simulation but to a lesser degree. These results support the hypothesis that failure occurred in two or more phases.

FORWARD PREDICTION

Runout analyses are usually employed to estimate the extent of the impact area and map the distribution of hazard intensity parameters, such as landslide velocity, flow depth, and depth of deposits (Hungr 1997). McDougall and Hungr (2004) explain that prediction of post-failure motion is an essential component of hazard assessment *when a potential source of a mobile landslide can be located* (McDougall and Hungr 2004). Back-analysis of similar runout events provides understanding of the rheology and reasonable estimates of material properties of the runout path and failed mass. Hungr (1995) notes that such back-calculated properties can be applied to the forward prediction of future events with reasonable confidence; however the runout analysis of the previous event does not provide an estimate of the source location or volume.

Taking these factors into consideration, numerical modelling and the lessons learned from the back-analysis are used to constrain the source location and initial volume of the potential rockslide.

Prediction of failed volume

The most important (and sometimes least constrained parameter) for runout prediction and hazard analysis is the volume of the failed mass. Further slope stability modelling was done to constrain necessary input of the predictive runout analysis. The objectives of further slope stability modelling are the following:

1. Identify potential failure mechanisms:
   a. Determine how the mechanisms affect the volume of failed material,
   b. Determine how the failure runout should be modelled.
2. Estimate the potential source location and volume of future rock failures

Method

Prediction of the failed volume began with identifying the portion(s) of the slope where further slope failure could occur. This volume may be in the crown of the slide, along the flank, or unstable material in the failed zone. Slope models of the potentially unstable areas were constructed.

All parts of the slope model from the back analysis phase were applied to the forward prediction phase of slope failure modelling. The current slope geometry (i.e. topography, discontinuities exposed after the previous slide, etc.), data collected from the literature and field surveys, and the results from the back analysis were combined to form the initial slope model. Parameters such as discontinuity strength, discontinuity orientation, and rock mass strength were systematically varied within the range of reasonable values determined during the back analysis. The depth, shape, and location of the failure surface for each scenario (where a failure surface did develop) were recorded and the minimum and maximum volume of failed material noted. The failure mechanism was also identified for each scenario. The failure mechanism is important because it controls how the runout initiates; for example the rate of failure and stages of failure.

Table 1: The Strongest Mohr-Coulomb rock mass and discontinuity strength properties that allow failure to occur

	Friction Angle (ϕ)	Cohesion
Rock Mass Properties		
Competent orthogneiss	54°	2.3 MPa
Highly-fractured wedge	30°	0.2 MPa
Discontinuity Properties		
Plane A	26°	0.4 MPa
Plane B and shear zones	26°	None

Results

For the Afternoon Creek rockslide, a large volume of potentially unstable material remains in the failed zone. This is a likely source for future rock avalanches, and is the focus for the remainder of the analysis (Fig. 10). Displacement of the highly fractured zone has left the toe of this zone unsupported. Planar failure along moderately dipping joints (shown in cross-section F-F') is likely. Hungr and Evans (2004) point out that motion of such translational sliding failures is usually extremely rapid. The entire volume above the failure plane, extending to the lateral and rear release surfaces is displaced in a single event. Therefore the runout of the rockslide should be modelled as a single phase involving the entire volume (this also coincides with the worst case scenario).

Limit equilibrium analysis of the planar failure mechanism shows that the ratio of resisting forces to driving forces (i.e. the factor of safety) decreases with increased slope height. Therefore, assuming that the joint cohesion and friction angle is equivalent for all joints, the deepest joint plane which enables kinematic freedom to the sliding blocks above represents that most likely to fail. However, the joint shear strength may not be equivalent for all joint planes. For example, discontinuities near the surface may have decreased shear strength due to de-stressing and physical and chemical weathering, allowing planar failure to occur in the upper reaches of the slope. As such, the upper limit for the failed volume involves that above the deepest kinematically feasible sliding plane, and the lower limit involves that above the shallowest persistent sliding plane (Fig. 11). The upper limit of the failed volume for this analysis is approximately 480,000 m^3.

Future failures in the crown and flank of the slide are also possible. Further investigations are planned to determine the likelihood of failure in these areas. Relatively small toppling failures and rock falls are likely to occur along the main scarp, but these do not pose a hazard to the highway below the Afternoon Creek side of the ridge where the blocks are most likely to fall.

Prediction of rockslide runout

DAN3D was used to predict the runout path, distance, and velocity of the potential failed volume. These results can be used for hazard analysis, and qualitative and quantitative risk analysis.

Method

The predicted failure volume was superimposed on the *after* DEM. Points in the potential failure zone were selected and a failed thickness (i.e. depth to failure plane) value assigned based on modelled cross sections. Failed thickness values were interpolated, or extrapolated to areas that were not modelled. The total failed volume was created by determining the thickness values across a grid covering the entire failed zone, using a *triangulation with linear interpolation* function. The predicted failed volume was subtracted from the current topographic surface (i.e. the *after* DEM) to create the rockslide path. The failed volume was multiplied by the 135% bulking factor to create the source volume input file.

The runout simulation was run with DAN3D for the upper limit of the predicted failed volume, corresponding to the worst case scenario of failure. The same rock avalanche rheology, basal bulk friction angle, and internal bulk friction angle that achieved the best results in the back analysis were used for these simulations.

Key differences between the back analysis runout model and the actual deposit were considered during the interpretation of the results; as noted during the back analysis, it is expected that the simulation will slightly over-estimate the run-up of the leading edge, run-out of the centre of mass, and spreading of the deposit.

Results

Runout of the largest volume simulated does not reach Washington state route 20 when the following parameters were used:

Rheology: Frictional
Basal Bulk Friction Angle: 37°
Internal Bulk Friction Angle: 40°

The path is confined within the Afternoon Creek valley walls until the material exits the canyon and spreads out onto the fan (Fig. 12). The maximum velocity reached during the simulation was approximately 40 m/s. The leading edge of the deposit came to an abrupt stop in the model (Fig. 12).

CONCLUSIONS

The results of the 'Total Slope Analysis' methodology, applied to the potentially unstable Afternoon Creek rock slope, predicts the runout path, distance, and velocity for a potential rock avalanche. The runout does not reach SR 20 for the worst-case scenario modelled in this analysis. Several numerical modelling programs are utilised to enable a more complete understanding of the problem. The method involves back analysis of the failure initiation and runout of a previous rock avalanche at the site that occurred in November 2003. The results of the back analysis provide input to the simulation of future slope failures, and help the geotechnical analyst to more accurately predict the volume

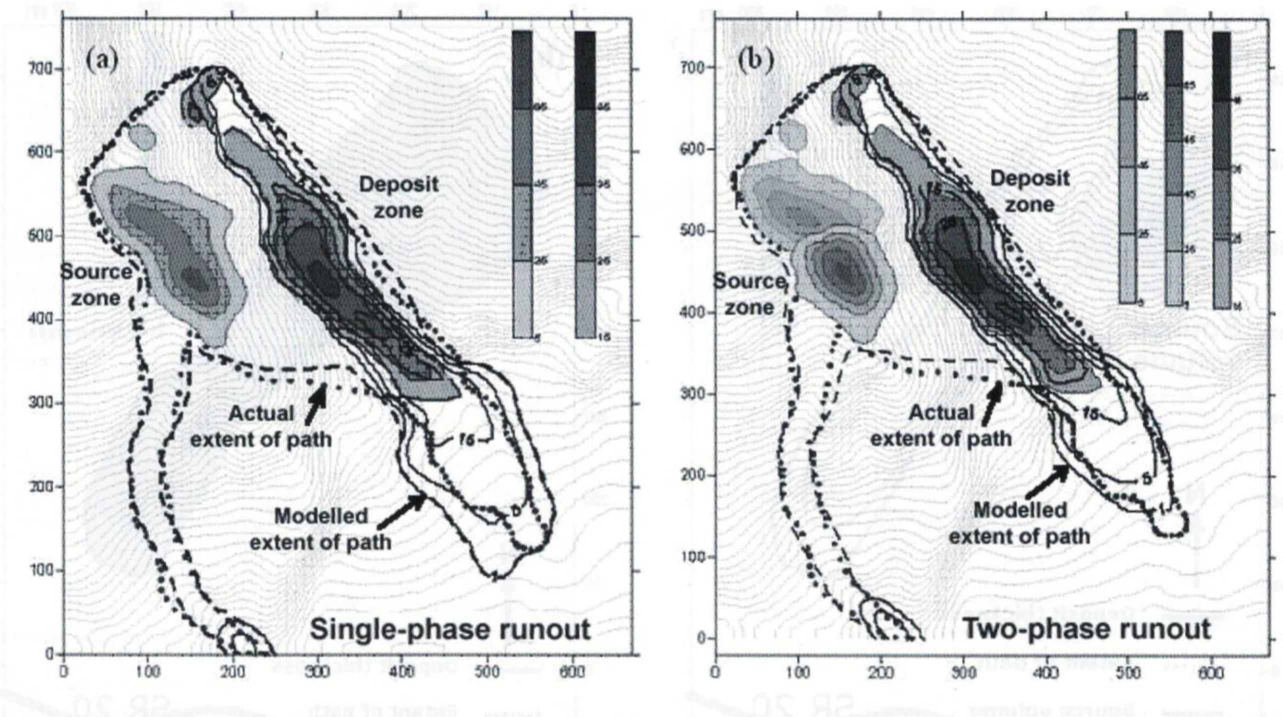

Fig. 9: (a) Dynamic runout analysis results. Filled contours indicate source zone (left) and actual deposit thickness (right). Dotted lines indicate actual extent of path. Black open contours indicate modelled thickness. Dashed line indicates modelled extent of path. (b) Runout is modelled in two phases.

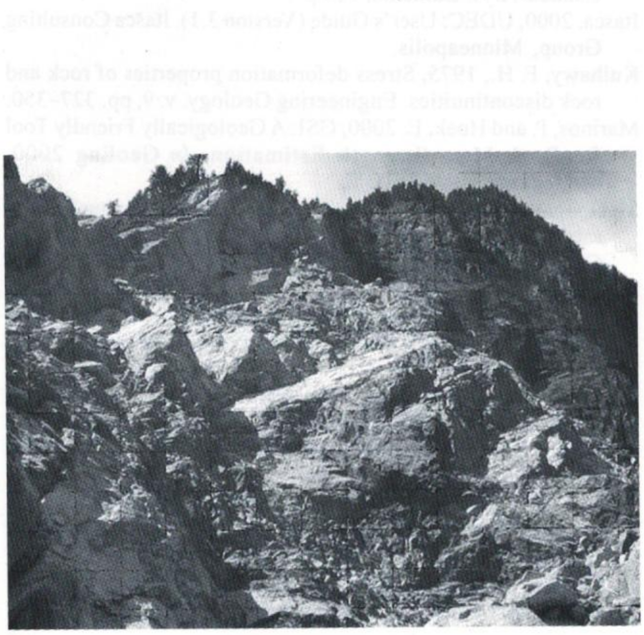

Fig. 10: Potentially unstable material in the failed zone

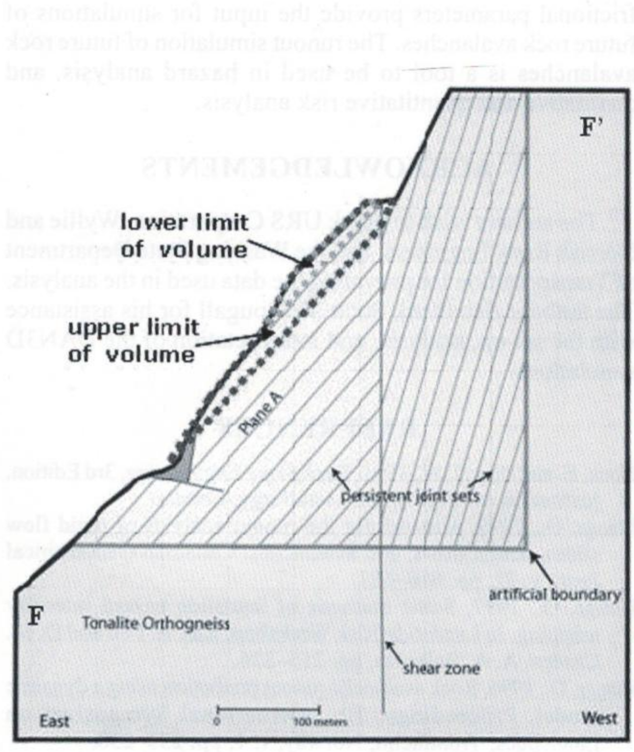

Fig. 11: Cross-section F–F' showing failed volumes that are kinematically feasible in the model

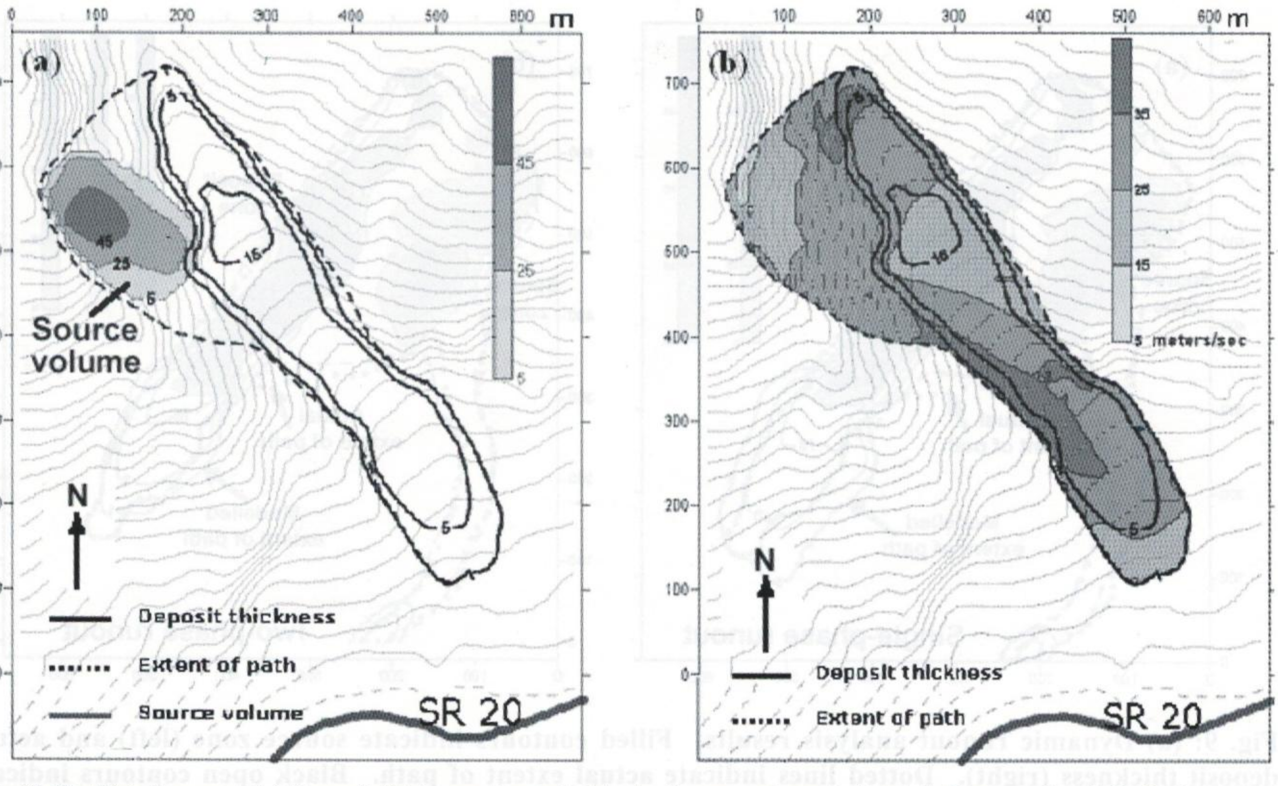

Fig. 12: Predicted runout. (a) Deposit thickness shown in black. Source volume is shaded. Extent of path dashed. (b) Maximum velocity plot.

of future failures. The predicted volume and predetermined frictional parameters provide the input for simulations of future rock avalanches. The runout simulation of future rock avalanches is a tool to be used in hazard analysis, and qualitative and quantitative risk analysis.

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