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UNDERSTANDING THE MECHANICS OF LARGE LANDSLIDES

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ABSTRACT

Our understanding of the mechanics of large landslides has improved considerably over the last decade with the development of new, innovative data collection methods, in conjunction with efforts to acquire unique data sets through detailed monitoring of several large rock slopes and the integration of these with increasingly sophisticated computer modelling techniques. In this paper the authors examine these recent developments in the context of three major issues that are considered fundamental to improved landslide characterization. Firstly we consider the role of damage mechanisms in large landslides, including what is meant by damage processes, the types of damage and the controls on damage distribution within the rock slope. Secondly we discuss the role of kinematics in the mechanics of large landslides considering the complex interactions that exist between the influence of scale, release surfaces and confinement, failure mechanisms, persistence and rock bridges, and groundwater pressures. Finally we discuss important advances in the modelling of large landslides and how this contributes to an improved mechanistic understanding of their spatial and temporal evolution. In particular, consistent with our discussion of damage in large landslides, we emphasise the developments of methods of modelling brittle fracture on rock slope damage.

Key words: *landslide mechanisms; brittle fracture; damage; rock bridges; kinematics; remote sensing; discrete fracture networks; numerical modelling; distinct element voronoi; lattice spring*

INTRODUCTION

During the last decade considerable advances have been made in our understanding of the mechanics of large landslides, yet considerable challenges still remain to characterize and model the complex mechanisms often involved. New measurement techniques, notably borehole televiewer tools, photogrammetry and LiDAR remote sensing, and real-time, high resolution InSAR monitoring, are contributing unprecedented amounts of data. Interpreting and applying this data, however, still remains largely subjective as geological complexity and uncertainty continue to pose major obstacles.

This paper will address these challenges by posing a series of questions focussing on what we perceive to be some of the critical issues in landslide and rock slope investigations. We will also summarize both the state-of-the-art and the advances made in recent years in data collection and geomechanical modelling drawing from our own research and also published case studies.

WHAT IS THE ROLE OF DAMAGE MECHANISMS IN LARGE LANDSLIDES?

DAMAGE IN ROCK SLOPES

Damage mechanics in landslides studies can be regarded as the characterization of rock mass and discontinuity damage, and the consideration of the evolution of the damage variables as a function of disturbances to the effective stress state. The damage variable, D, has been defined in numerous ways but perhaps one the earliest definitions considers the ratio of the amount of area of cracks/voids, ΔS_D relative to a selected area, ΔS, within the slope, i.e. $D = \Delta S_D/\Delta S$. Using this approach damage varies from zero for an undamaged material to 1 for a fully failed element. The development of damage results in a degradation of the rock mass modulus and strength parameters. Damage has been defined in numerous ways by considering the effects of damage such as reduced elastic modulus, hardness or velocity. If we consider the classical problem of a landslide where non-persistent joints are present with intervening rock bridges, RB (see Fig. 1), then the original percentage of rock bridges is given by:

$$RB\% = \frac{\Sigma RB}{L} x100 \qquad (1)$$

As the rock bridges gradually fail under the influence of downslope gravitational stress the damage increases until D=1 and ΔRB goes to zero, such that:

$$\frac{\Sigma \Delta L + \Delta RB}{L} = D = 1 \qquad (2)$$

This approach can be interpreted as combining a pre-existing tectonic damage state due to jointing of ΔL/L (Fig. 1c) superimposed by a gravitational induced damage ΔRB/L. Failure of a slope in this case is thus treated as sliding along a non-persistent shear surface area. In traditional slope stability problems the failure surface is considered fully persistent (Fig. 1a) with a uniform factor of safety. In practice the potential failure surface is often not fully formed and may require the failure of "patches" or small areas of rock bridges (Fig. 1b). These rock bridges may reflect the presence of proto-joints (as discussed by Hencher *et alii*, 2012), sedimentary structures on bedding surfaces, or undulations due to tectonic activity. Damage can similarly be characterized by considering the ratio of the cumulative failed rock bridge area to the total area of the potential failure surface.

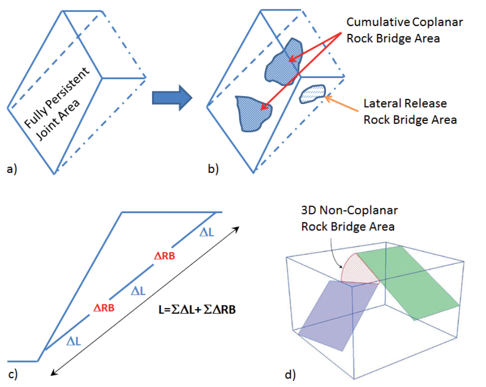

Fig. 1 - Characterization of rock slope damage related to non-persistent joints and presence of intervening rock bridges, with examples of: a) sliding surface coinciding with fully persistent plane of weakness; b) sliding surface comprised of non-persistent co-planar joints and cohesive rock bridge elements; c) classical definition and calculation of damage; and d) non-coplanar joints and rock bridges

CRACK PROPAGATION AND DAMAGE

Numerous recent studies in rock engineering have recognized the importance of crack initiation, crack damage and crack coalescence during brittle rock failure. These studies have incorporated different stress or damage thresholds to derive new failure criteria that are more applicable to brittle rock under low confinement. Such tri-linear or S-shaped criteria (Diederichs, 1999; Kaiser & Kim, 2008) have primarily been applied in underground rock engineering. Recent studies however have demonstrated their applicability to landslide (Leith, 2012) and large open pit slopes (Wesseloo & Dight, 2009; Eberhardt, 2009). Figure 2 shows that the conditions for brittle dominated failure mechanisms under high gravitational and low confining stresses may be particularly relevant in the toe (and crest) area of high rock slopes where stress-induced cracking may lead to high damage concentration zones. In the inter-

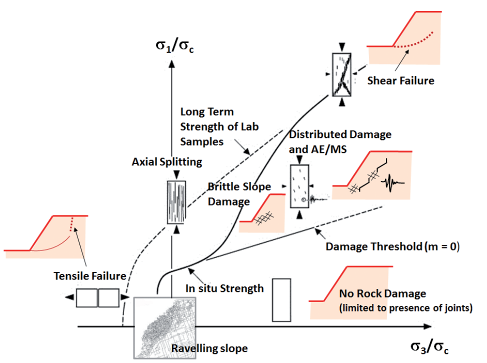

Fig. 2 - S-shaped brittle failure criterion and its application to common stress conditions found in rock slopes. Modified after Diederichs (1999)

nal zones within landslides damage may be dominated by shear localization under higher confinement. These processes agree with the observed early indicators of instability, with tension cracking at the slope crest and heave/bulging at the toe, in accordance with the concept of progressive failure advocated by early workers. The process of brittle failure through crack propagation in slopes is hence a fundamental damage dominated failure mechanism involving degradation of rock bridges, destruction of asperities and roughness along potential failure surfaces, and the development of through-going step path failure. Together, damage processes in landslides range from the initial microscale (intra/intergranular microcracking) through to meso- and macroscale fragmentation and comminution of the rock slope mass during global slope failure.

SPATIAL AND TEMPORAL DAMAGE CONTROLS

The evolution of damage in a slope varies both spatially and temporally. In consequence certain areas of a slope may be pre-disposed to increased damage either in relation to driving forces, water pressures or due to the existence of pre-existing tectonic damage. Numerous workers have shown the inter-relationship

between structures and rock mass quality where the latter essentially reflects pre-existing damage. However, the spatial distribution of damage in a rock slope or landslide has received very little attention to date. In numerous field-based landslide investigations, the authors have repeatedly recognized characteristic damage distributions associated with variations in:

- Slope topography
- Failure surface morphology
- Failure surface geometry
- Failure mechanism
- Lithological variations
- Geological structure

Such damage is evident in field observations, geophysical surveys, microseismic data and in the results from geomechanical models. Evidence of spatial distribution of rock mass damage in the field includes the location of tension cracks, back scarps, lineaments and troughs, compression induced bulging/heave, and outcrop fracturing/fragmentation.

Temporally, numerous processes may increase damage within a rock slope over time including:

- Tectonics - folds, faults, uplift, deformation phases
- Geologic processes associated with rock genesis (intrusion, metamorphism, alteration)
- Geomorphic processes – glacial erosion, glacial rebound, fluvial down cutting
- Earthquakes
- Precipitation and snowmelt events
- Long term creep

In classical damage mechanics the importance of cyclic loading and creep are both considered. The authors suggest that these processes acting over 100's to 1000's of years play a critical role in landslide development which has received comparatively little attention particularly when modelling landslide failure mechanisms. Cyclic processes result in fatigue of the rock slope and the gradual accumulation of long term damage until a "critical slope damage threshold" is attained. Such processes may be tectonic such as repeated earthquakes of widely varying magnitude gradually removing rock bridges and roughness on failure surfaces. They may also be geomorphic such as variations in topography (gravitational loading), water pressures, thermal stresses, freeze-thaw and uplift/erosion over time. Griffiths *et alii* (2012) emphasize the dynamic nature of landform evolution processes, an essential pre-requisite for understanding the mechanics of landslides and developing realistic geomechanical models. These landslide damage types reflect the varying stress paths that the slope is subjected to; it is this locally changing stress-path that is the underlying driving mechanism of slope failure and which must be considered in landslide models.

EXTERNAL AND INTERNAL EVIDENCE OF SLOPE DAMAGE

Figure 3 shows selected examples of rock slope damage based on surface observations, what we term here as "external slope damage". Yan (2008) and Tuckey (2012) have demonstrated the utility of LiDAR and photogrammetry surveys to map and assess external rock slope damage using fractography techniques. This includes the characterization of damage density and differentiation between (1) intact rock bridges, which describe intervals of intact rock separating non-persistent discontinuity tips; and (2) rock mass bridges, which describe larger intervals of jointed rock between major structures such as faults. LiDAR data, both terrestrial and airborne, provides frequent evidence of external slope damage including tensions cracks and geomorphic features. Moore *et* *alii (*2011) have shown through distinct element modelling how such landslide induced damage can cause amplification of earthquake waves resulting in seismically induced internal damage. An excellent example of this is the Madison Canyon rock slide (Hadley, 1964), which was triggered by the 1959 Hebgen Lake magnitude 7.5 earthquake in Montana, USA (Fig. 4). Pre-failure evidence of gravitationally induced fracturing was present, which could have served to amplify the earthquake effects.

Examples of "internal slope damage" include both borehole and three-dimensional (3-D) geophysical observations. Borehole logging using acoustic televiewers on Turtle Mountain, Alberta, Canada (Spratt & Lamb, 2005) showed evidence of damage and disturbance associated with previous rock slope instability. Geophysical surveys and microseismic monitoring have provided evidence of internal rock slope damage in the form of anomalous reductions in seismic velocities and other geophysical properties. At the site of the Randa rockslide, Switzerland, Green *et ali*i (2006) reported reductions in seismic velocity to less than 1500 m/s (compared to intact gneiss velocities of 3500-6500 m/s) representing an estimated 17% volume of voids, or damage, in the rock mass. Numerous microseismic surveys in open pit rock slopes have indicated rock slope damage associated with excavation progress and geological structures such as faults

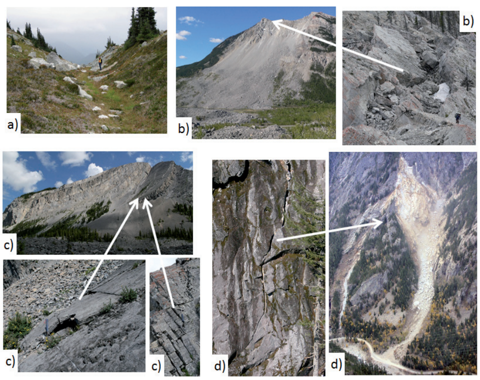

Fig. 3 - Examples of external rock slope damage: a) antiscarp associated with deep-seated gravitational displacement; b) tension cracks associated with head scarp of Frank Slide, Alberta, Canada; c) Palliser rockslide, Alberta, Canada and associated bucking and toppling damage; d) large, 50+ m long intact rock fracture damage associated with Newhalem rock slide, Washington, USA

INFLUENCE OF SLOPE TOPOGRAPHY ON ROCK SLOPE DAMAGE

Leith (2012) through field observations and geomechanical modelling showed the influence of exhumation induced brittle fracture (tensile jointing) along alpine valleys in Switzerland and the influence of this damage on both glacial U-shaped valley development and rock slide occurrence. Leith showed that the spatial distribution of the damage in two dimensions is related to the present day topography with clear bounding elevations of rock mass damage on the valley walls. Jackson (2002) reports a similar possible influence of glacial ice elevation and related damage on the lower slopes of Turtle Mountain, Alberta, which has received little attention in previous modelling studies. Deeply incised valleys may be. observed at the location of major rockslides including Vajont, Italy (Ghirotti, 2006) and Eiger, Switzerland (Oppikofer *et alii,* 2008) (Fig. 5). The influence of such major geomorphic processes has yet to be fully integrated into geomorphic stress-path modelling of landslide mechanisms. Eberhardt *et alii,* (2004) showed clearly through FEM, DEM and brittle fracture modelling (FDEM) the influence of convex glacially over-steepened topography on the spatial distribution of rock slope damage at the Randa rockslide. The mechanical influence of glacier retreat on landslide development has been the subject of some controversy over recent years particularly with regard

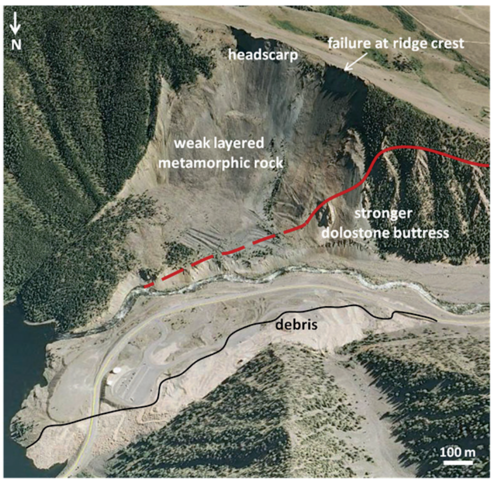

Fig. 4 - The 1959 Madison Canyon rockslide, Montana, USA, associated with earthquake triggering and seismic amplification damage

to the concept of glacier-debutressing (McColl *et alii*, 2010). Notwithstanding there is considerable evidence for the onset of slope instability following deglaciation. The authors suggest that the initiation of failure may result through a combination of changing kinematics (kinematic release) and slope damage in the valley walls due to glaciation related processes (high pore water pressures, oversteepening, brittle fracture, etc.).

Figure 6a shows the Mitchell Creek landslide in northern British Columbia, Canada, discovered during mineral exploration and described by Clayton *et alii,* (2013). This landslide is actively deforming and has been the subject of geotechnical investigations using cored drillholes, acoustic televiewers, field mapping and photogrammetry. It is currently being monitored using combined borehole, surface survey and remote sensing instruments. The slope damage mechanisms are complex involving components of topping and translation with rock mass degradation due to combined tectonic, glacial and gravitationally induced damage. A particularly interesting facet of this landslide is the ability to date the onset of the instability following recent glacial retreat using aerial photography, Fig. 6b. Numerous studies on deep-seated gravitational displacements have shown the characteristic external damage evidence such as antiscarps and grabens. Many of these landslides believed to have initiated following deglaciation. In engineering projects involving proximity to deep-seated gravitational displacements (e.g., hydroelectric and tunneling projects), internal evidence of slope damage has been noted.

An important feature recognized in large open pit slopes is the existence of a zone of relaxation and damage sub-parallel to the pit slope profile. This zone varies in width and is related to both slope geometry and blasting disturbance. It is important to realize that the geomorphic evolution process of a natural slope, which can be equated to the bench excavation sequence of an open pit slope, may similarly produce important damage generating stress relief-stress concentrations behind the slope face. A practical consequence of the existence of a zone of relaxation behind the rock slope is that the hydrogeology may change as fluid flow pathways open and others close, or the rock mass may be subjected to increased weathering and/or ravelling leading to surficial instability. It is in this area that we are often obliged to make estimations of the rock mass quality (e.g. GSI, RMR, etc.); clearly these estimates are liable to be lower bound values with rock mass quality improving with distance into the rock slope. It can generally be expected that the external surficial damage should be greater than the internal damage.

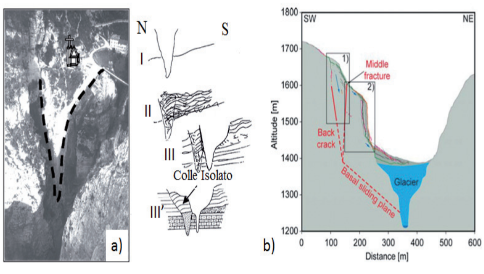

Fig. 5 - Geomorphic damage related to deeply incised valleys associated with: a) Vajont rockslide (after Ghirotti*, 2006); and b) Eiger rockslide (after* Opikoffer *et alii, 2006)*

DAMAGE RELATED TO FAILURE SURFACE GEOMETRY AND MECHANISMS

The failure surface geometry has an important influence on rock slope damage. Figure 7 shows conceptual examples of the damage processes associated with varied failure surface geometries, as they relate to different failure mechanisms. Where rock slope failures have moved parallel to pre-existing, highly persistent, planes of weakness (e.g., bedding, faults, etc.), the amount of induced damage during failure is reduced.

Wolter *et alii* (2013) and Massironi *et alii* (this volume) have investigated in detail the failure surface morphology at Vajont (Fig. 8). Clearly this failure surface does not represent the planar surface morphology shown in Fig. 7a but bears more resemblance to Figs. 7b-c. The undulations recorded on the existing eroded failure scar at Vajont are however

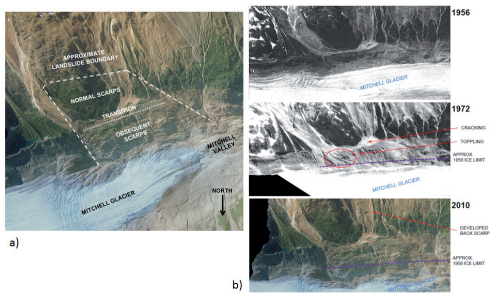

Fig. 6 - The Mitchell Creek Landslide, British Columbia, Canada, showing: a) slide boundaries, instability damage zones and location of valley glacier; and b) history of glacier retreat and development of landslide damage indicators derived from air photographs

3-D in nature and are formed by two generations of approximately N-S to NNW and E-W to WNW-ESE striking folds. The folds result in complex dome and basin to crescent and mushroom interference patterns. The nature of this folding in the failure surface varies spatially allowing the definition of failure surface morphological domains (Wolter *et alii*, 2013). The authors suggest that this variation in failure surface morphology may have been reflected variations in the induced damage preceding and during the failure at Vajont. Damage would have been associated with the translation of failure blocks during dilation of the rock mass as it overrode these fold undulations. The importance of the claybeds at Vajont has been discussed by numerous workers. We suggest that the true importance of the beds is a complex inter-relationship between the fold structure undulation amplitudes and wavelengths, and the claybed thickness. Using a similar concept of displacement relative to claybed thickness and roughness used in the Q rock mass quality system (Barton *et alii*, 1978)

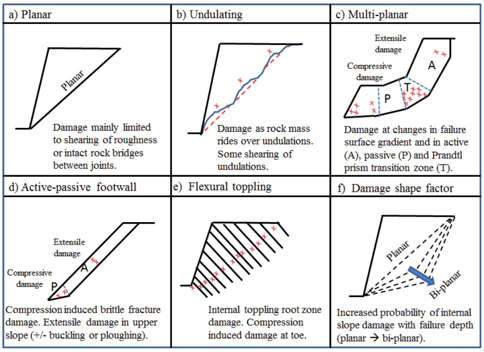

Fig. 7 - Failure surface morphology/shape and associated damage mechanisms

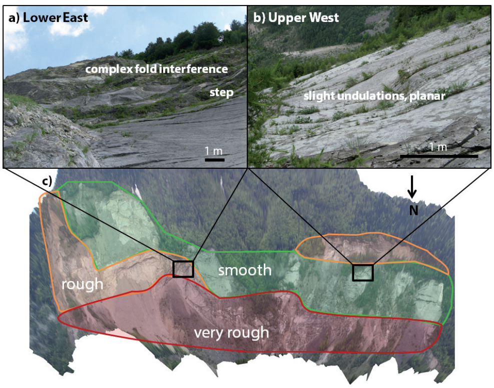

Fig. 8 - Failure surface morphology domains at Vajont, Italy, and relation to tectonic folding

we suggest that movement could have involved a stick-slip damage mechanism with movement along the claybeds locking up at the folded bed undulations (Fig. 9). High pore water pressures at the foot of the slope and progressive gravitational induce damage could allow the overriding of the fold undulations, producing alternating cycles of clay (slip) and roughness (stick) dominated displacement-damage mechanisms. These cycles may have been of short time duration, less than the displacement monitoring frequency, resulting in their combined expression in the overall recorded slope displacement.

Such movement might also be expected to be expressed in irregular microseismic activity. Clearly the failure surface morphology at Vajont varies significantly hence it might be expected that the displacement modes and damage mechanisms likewise vary in relation to structural morphological controls. At the Aknes rock slope, similar variations of the schistosity may also be present in the sub-surface topography both along dip and strike. Long term progressive damage may thus also be influenced by anisotropic roughness in three-dimensions which when considered along with varying kinematics may relate to the movement of the landslide as a series of blocks. Similar variations in the slide surface morphology and thickness have been recorded at other landslides

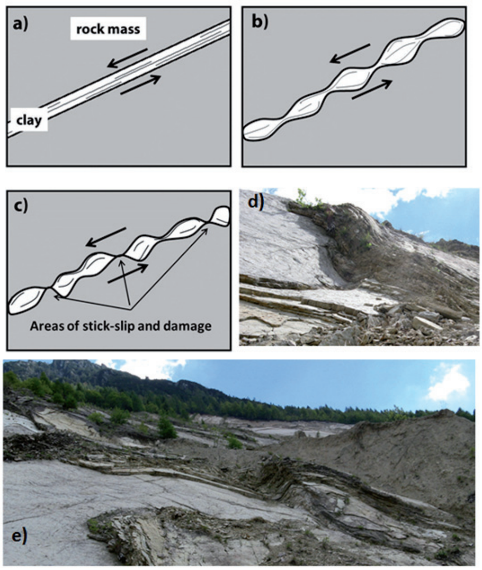

Fig. 9 - Hypothesized interrelationship between failure surface morphology and clay bed thickness, and folded scarp surface at the Vajont rockslide, Italy

exhibiting discrete block movements (Kalenchuk, 2010) The authors suggest that anisotropic failure surface morphology (or roughness) may be an important control on 3-D damage mechanisms (and progressive displacements) in landslides which requires further consideration in future geomechanical models.

STRUCTURAL CONTROLS ON ROCK SLOPE DAMAGE

Structural controls on rock slope failure have been emphasised by numerous authors, both with respect to rock mass quality and rock slope kinematics. Folds and faults have an important influence on pre-existing rock slope damage, often predisposing areas of a rock slope to react to gravitational stress through varied failure and damage mechanisms. At Vajont the presence of the Col Tramontin and Col delle Erghene Faults in the eastern half of the slide have resulted in a lower rock mass quality and predominantly shear type mechanisms of damage. In the western half of the Vajont slide the rock mass is less disturbed by faulting, and failure corresponds to a more active-passive nature due to the higher rock mass quality (Wolter *et alii*, 2012) (Fig. 10). Brideau *et alii* (2005, 2009) and Donati *et alii* (2012) describe the influence of faulting and shearing on rock slope stability at the Hope Slide. Here the failure debris is highly fragmented reflecting both the pre-existing damage and the gravitationally induced damage. Similarly, in a detailed study of Turtle Mountain, Alberta, site of the 1903 Frank Slide, Brideau *et alii* (2012) and Pedrazinni *et alii* (2008) correlate the rock mass quality (GSI) and the instability mechanisms with the influence of fold induced damage and joint sets. Figure 11a shows modelling interpretations of fold induced damage and its influence on the Frank Slide. Benko & Stead (1998) in early distinct element modelling clearly showed the importance of damage in the hinge of the anticlinal fold and thrusting at the toe on instability. Figure 11b illustrates the results of research by Pedrazinni *et alii* (2008) which emphasized the importance of structure on rock mass quality on the present day stability of Turtle Mountain. Badger (2002) also emphasized the importance of fold-related structures (and pre-existing tectonic damage) on rock slope failure mechanisms, Fig 11c. Figure 12 shows the influence of folding, faulting and foliation on induced damage for different engineered and natural rock slopes.

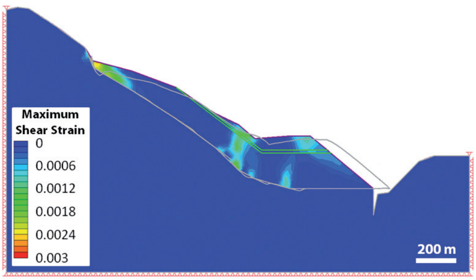

Fig. 10 Finite-element model simulating active-passive zones in the western section of the Vajont rockslide, Italy. After Wolter et alii (this volume)

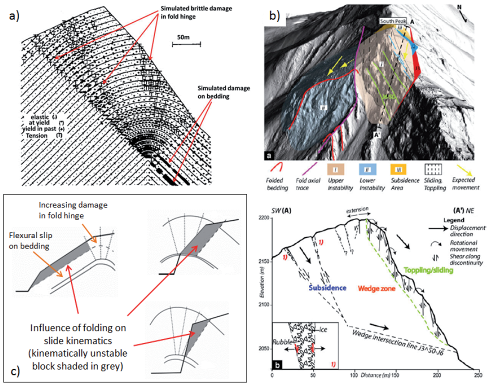

Fig. 11 Structurally induced damage: a) distinct-element model simulating damage in the fold hinge of the Frank Slide, Alberta, Canada (after Benko & Stead, 1998); b) damage and current instability at Turtle Mountain, site of the Frank Slide (after Pedrazinni et alii, 2008); and c) influence of folding on damage and kinematic (after Badger, 2002)

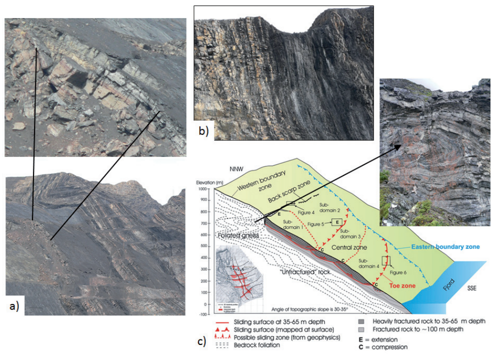

Fig. 12 - Damage associated with geologic structures: a) fold-related damage in in footwall slope in an open pit coal mine in eastern British Columbia, Canada; b) damage associated with a fault in coal measures; c) foliated structure of the Aknes rock slope, Norway (map after Kveldsvik et alii, 2008; photograph by D. Stead)

LITHOLOGICAL CONTROLS ON ROCK SLOPE DAMAGE

The lithology in a rock slope may have an important influence on damage mechanisms and the development of failure. It is suggested that some rock types may act to concentrate brittle rock mass damage, whereas other rock types may respond in a more ductile manner dominated by yield and shear mobilization. In rock slopes composed of principally brittle rock types, behavior may be characterised by sliding along structures and associated intact rock bridge damage. In other rock slopes involving highly fractured rock masses or more ductile rock types, damage and failure may be in the form of shear induced localization or "plastic yield" as modelled in most numerical codes. Benko (1998) clearly showed the importance of lithology in toppling rock slope failures in Coal Measures strata (mudstones, sandstones, conglomerates, coal) (Fig. 13). The influence of lithology on the failure mechanism at Vajont (conglomerate beds) was noted by Ghirotti *et alii* (1992) and is also investigated by Wolter *et alii* (this volume) The weaker mudstones and sandstones were observed to have yielded through shear damage whereas the stronger sandstone/conglomerate beds acted as concentrators for tensile damage, playing an important role in the overall slope stability. Jackson (2002) in a study of the factors controlling rockslides in the Canadian Rocky Mountains emphasised the role of landform evolution where rock slope instability is promoted by slope retreat of resistant rock masses overlying weaker rock masses. Erosion and yield of the weaker rock types is suggested to induce brittle damage in the overlying resistant rock masses leading to progressive and repeated instability. Potential damage mechanisms within a slope are hence a complex interaction of variations in lithology and geological structure.

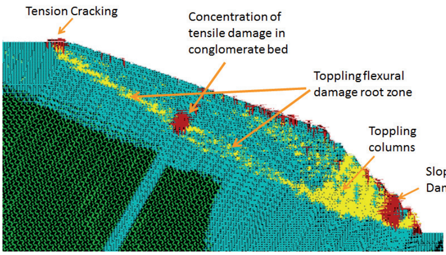

Fig. 13 - Influence of lithology on distinct element modelling of flexural toppling failure in coal measure rocks. Modified after Benko (1998)

TEMPORAL DAMAGE

When considering the progressive failure of a landslide it is important to consider not only spatial damage variations but also temporal variations in damage. Slide surfaces coinciding with persistent discontinuities with negligible intact rock bridges may fail in a rapid brittle manner. More complex undulating or multiplanar failures may involve variations in damage both spatially and temporally. Stead *et alii* (2007) describe the stages in the brittle failure of a rock slope with respect to failure processes as primary, secondary and tertiary. It is suggested that these stages also reflect similar changes in damage operating in a rock slope. Sullivan (2007) and Mercer (2006) describe changes in the deformation of a rock slope using time-dependent stages. Sullivan recognised five stages- elastic, creep, cracking and dislocation, collapse (failure) and post failure. Figure 14 modified after Dick *et ali*i (2013) shows failure divided into regressive, progressive and post-failure stages. The onset of failure occurs at the transition between the regressive and progressive stages at which time the rate of damage increases in the rock slope. Laboratory acoustic emission studies by the authors on varied rock types have shown similar stages in the acoustic emission activity associated with creep mechanisms. This behavior in a low confining stress environment can be interpreted as brittle damage dominated creep mechanisms. Landslide monitoring can be interpreted based on these slope deformation stages (or slope damage stages). Inverse velocity analyses (Fukozona, 1985; Rose & Hungr, 2008; Eberhardt, 2008) have been used to predict the time to failure of several unstable slopes based on displacement monitoring data. These authors however caution against their use in the prediction of failure where brittle failure occurs by sliding along discrete surfaces. Using the analogy of slope displacement and slope damage stages in brittle rock masses, it is suggested that acoustic emission and microsesimic data (combined with brittle fracture modelling) may be amenable to similar data processing methodologies for slope failure prediction.

Previous methods of displacement monitoring in slopes were based on point measurements, for example involving survey prisms. These methods indicate the regressive and progressive displacement stages at discrete points and hence localized external or surficial damage. With the recent introduction of InSAR and RADAR slope monitoring techniques the displacement is recorded over wide areas at pixels that can be displayed in point cloud format. Slope radar technologies allow improved temporal estimates of increasing/cumulative displacements (reflecting external damage) but also the ability to spatially correlate damage/displacements with rock mass quality variations, structures and lithology. Linear zones of displacement have for example been correlated with release structures indicating progressive damage on these features with time. Gischig *et alii* (2009, 2011) used InSAR to monitor slope displacements at the site of the Randa rockslide and were able to delineate release structures through such movements. Interpretation of RADAR/InSAR data may provide information on the spatial distribution of surficial/external slope damage in the form of slope damage maps arising from gravitationally induced displacements

The authors have developed discrete fracture network engineering techniques in an attempt to characterize damage associated with landslides both in situ and in particularly in geomechanical models. In this approach damage can be considered in terms of D_{10} (number of cracks/unit length), D_{21} (length of cracks/unit area) and D_{31} (length of cracks per unit volume). This approach is particularly useful in characterising damage during brittle fracture modelling of rock slopes.

WHAT IS THE ROLE OF KINEMATICS IN LARGE LANDSLIDES?

Appreciation of the kinematic controls is an important step toward understanding the mechanics of landslides. At their simplest rock slides are often analysed as two dimensional plane strain problems assuming elastic blocks and sliding along continuous planes. In practice the necessary conditions for plane strain analysis are rarely met with most landslides incorporating there-dimensional aspects. In this section of the paper we will consider the kinematic controls

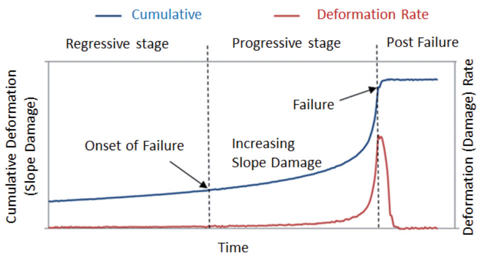

Fig. 14 - Stages of slope deformation and damage in rock slopes. Modified after Dick *et alii (2013)*

on rock slope failures and their inter-relationship with geological structure, topography, rock mass quality, geomorphology and time.

SCALE AND KINEMATICS

The scale of a rock slope predetermines the usefulness of simple stereographic techniques in kinematic analysis. The latest version of DIPS v.6.0 (Rocscience, 2013) incorporates an interactive method of determining the kinematic feasibility for planar, wedge, flexural and direct toppling modes of failure. The scale of the rock slope however usually limits the application of this method to smaller rock slopes where the likelihood of joint persistence being of sufficient size to enable kinematic release is possible; joint set persistence provides an effective cut-off for kinematic block size. As slopes increase in height over 50m, the applicability of kinematic analysis becomes more limited to rock slopes containing continuous

TEMPORAL DAMAGE

When considering the progressive failure of a landslide it is important to consider not only spatial damage variations but also temporal variations in damage. Slide surfaces coinciding with persistent discontinuities with negligible intact rock bridges may fail in a rapid brittle manner. More complex undulating or multiplanar failures may involve variations in damage both spatially and temporally. Stead *et alii* (2007) describe the stages in the brittle failure of a rock slope with respect to failure processes as primary, secondary and tertiary. It is suggested that these stages also reflect similar changes in damage operating in a rock slope. Sullivan (2007) and Mercer (2006) describe changes in the deformation of a rock slope using time-dependent stages. Sullivan recognised five stages- elastic, creep, cracking and dislocation, collapse (failure) and post failure. Figure 14 modified after Dick *et alii* (2013) shows failure divided into regressive, progressive and post-failure stages. The onset of failure occurs at the transition between the regressive and progressive stages at which time the rate of damage increases in the rock slope. Laboratory acoustic emission studies by the authors on varied rock types have shown similar stages in the acoustic emission activity associated with creep mechanisms. This behavior in a low confining stress environment can be interpreted as brittle damage dominated creep mechanisms. Landslide monitoring can be interpreted based on these slope deformation stages (or slope damage stages). Inverse velocity analyses (FUKOZONA, 1985; ROSE & HUNGR, 2008; EBERHARDT, 2008) have been used to predict the time to failure of several unstable slopes based on displacement monitoring data. These authors however caution against their use in the prediction of failure where brittle failure occurs by sliding along discrete surfaces. Using the analogy of slope displacement and slope damage stages in brittle rock masses, it is suggested that acoustic emission and microsesimic data (combined with brittle fracture modelling) may be amenable to similar data processing methodologies for slope failure prediction.

Previous methods of displacement monitoring in slopes were based on point measurements, for example involving survey prisms. These methods indicate the regressive and progressive displacement stages at discrete points and hence localized external or surficial damage. With the recent introduction of InSAR and RADAR slope monitoring techniques the displacement is recorded over wide areas at pixels that can be displayed in point cloud format. Slope radar technologies allow improved temporal estimates of increasing/cumulative displacements (reflecting external damage) but also the ability to spatially correlate damage/displacements with rock mass quality variations, structures and lithology. Linear zones of displacement have for example been correlated with release structures indicating progressive damage on these features with time. GISCHIG *et alii* (2009, 2011) used InSAR to monitor slope displacements at the site of the Randa rockslide and were able to delineate release structures through such movements. Interpretation of RADAR/InSAR data may provide information on the spatial distribution of surficial/external slope damage in the form of slope damage maps arising from gravitationally induced displacements

The authors have developed discrete fracture network engineering techniques in an attempt to characterize damage associated with landslides both in situ and in particularly in geomechanical models. In this approach damage can be considered in terms of D_{10} (number of cracks/unit length), D_{21} (length of cracks/unit area) and D_{31} (length of cracks per unit volume). This approach is particularly useful in characterising damage during brittle fracture modelling of rock slopes.

WHAT IS THE ROLE OF KINEMATICS IN LARGE LANDSLIDES?

Appreciation of the kinematic controls is an important step toward understanding the mechanics of landslides. At their simplest rock slides are often analysed as two dimensional plane strain problems assuming elastic blocks and sliding along continuous planes. In practice the necessary conditions for plane strain analysis are rarely met with most landslides incorporating there-dimensional aspects. In this section of the paper we will consider the kinematic controls on rock slope failures and their inter-relationship with geological structure, topography, rock mass quality, geomorphology and time.

SCALE AND KINEMATICS

The scale of a rock slope predetermines the usefulness of simple stereographic techniques in kinematic analysis. The latest version of DIPS v.6.0 (ROCSCIENCE, 2013) incorporates an interactive method of determining the kinematic feasibility for planar, wedge, flexural and direct toppling modes of failure. The scale of the rock slope however usually limits the application of this method to smaller rock slopes where the likelihood of joint persistence being of sufficient size to enable kinematic release is possible; joint set persistence provides an effective cut-off for kinematic block size. As slopes increase in height over 50 m, the applicability of kinematic analysis becomes more limited to rock slopes containing continuousstructures such as bedding and major faults. Kinematic methods are also generally limited to simple block shapes and through-going fractures. Complex failure mechanisms such as biplanar, multiplanar, step-path and buckling are not easily handled by simple daylighting oriented kinematic analysis techniques. Caution is required not to oversimplify the kinematics of landslide failure mechanisms to fit within the limitations of conventional kinematic analysis.

BLOCK SHAPE AND KINEMATICS

The shape of blocks related to landslides are fundamentally three dimensional. Even a simple mechanism such as planar failure, often depicted and analyzed in two-dimensions, involves a hexahedral block with varying orientation of lateral and rear release surfaces. In a traditional stereographic analysis, the kinematics of such a hexahedral block will be treated as both a daylighting planar failure along a basal surface and wedge intersections - perhaps not considering appropriately the block theory aspects of hexahedral failure. The routine kinematic and limit equilibrium analysis of rock slope wedge failures has been simplified over recent decades to consider all wedge failures as either tetrahedral blocks which may or may not be truncated in the upper part by a tension crack. Failure is normally considered to be by translation along the line of intersection or on a more adverse dipping plane. The rotation of wedges was considered in early rock wedge analyses but has largely been ignored in recent research. Hungr & Amann (2011) show the importance of considering rotational wedge movement of rock wedges. No routinely available wedge analysis software considers the kinematic case of a pentahedral wedge where the base of the tetrahedral block is truncated by a basal sliding surface such as bedding or a fault. Such a failure mechanism has been increasingly recognized in large open pits and is also prevalent in natural slopes and has been termed a "non-daylighting" wedge. In this case the plunge of the line of intersection would not daylight and the role of the basal surface is critical.

To treat complex block shapes, block theory has been used to clearly demonstrate its importance with respect to landslide kinematics and controlling instability. Brideau (2010) undertook block theory analysis in association with 3-D numerical modelling to investigate the role of block shape on instability. Discrete Fracture Network approaches have been used to indicate the stability of kinematic keyblocks in rock slopes (Rogers *et ali*, 2006; Merrien-Soukatchoff *et ali*, 2011) using codes such as Fracman and Resoblock, respectively. Recent work by Thompson (2011) and Elmoutiie & Poropat (2011) shows great promise in incorporating block shape, size and orientations into rock slope analysis with the ability to show the dynamic failure of rock slopes associated with progressive sliding and removal of unstable blocks. It is suggested that a combined approach to rock slide analyses using discrete fracture networks and geomechanical models will allow improved understanding of landslides failure mechanisms as data collection and monitoring techniques continue to evolve.

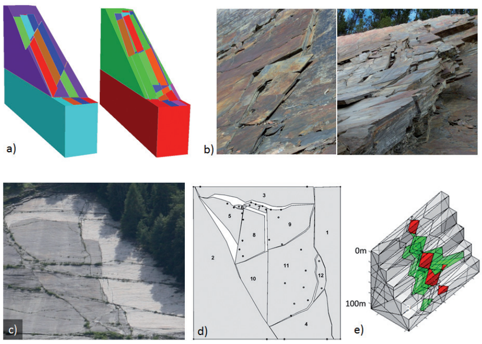

Fig. 15 - Rotational-translation failures and DFN based block removal kinematic analysis: a) simple 3DEC simulations of rotational-translation failure; b) rock slope with rotational-translational failure; c) joint controlled rotational-translational block failure at Vajont, Italy; d) block movements identified at Aknes, Norway (after Kveldsvik et alii, 2008); and e) DFN-based (Siromodel) block removal based on stability of fully formed blocks (after Elmouttie & Poropat, 2011)

ROTATION VS. TRANSLATION

Steeply plunging wedges may rotate out of the slope or topple. Rotational-translational planar failures are also commonly observed particularly where plunging folds are present within a rock slope. Simple 3DEC analyses (Fig. 15) clearly show the rotation of slope parallel blocks (and hence kinematically non-daylighting) out of a slope with a down plunge component of displacement. Such down plunge block movements are evident at Vajont on a small scale and may also have been important kinematically in the main slope failure. Rotation in slope failures has predominantly been considered out of the slope in the slope dip direction (+/- 20 degrees) for the case of toppling failure and can be considered as dip rotational mechanisms. Brideau & Stead (2011) use 3DEC to investigate the influence of the orientation of lateral release, rear release and basal surfaces on the kinematics of toppling failure mechanisms. Considerable differences in slope displacements and damage inflicted on the rock slope are associated with oblique dip rotation mechanisms. Brideau & Stead (2012) also performed a similar 3DEC kinematic modelling study on translational planar failure mechanisms showing a similar importance of kinematics in determining both failure mode and slope displacement. Few workers apart from Hungr & Amann (2011) and Steffen (Personal communication, 1981) have considered kinematic rotation about vertical axes or what might be considered kinematically as plan rotational failures. Hungr and Amann used a combined approach of analytical methods, limit equilibrium and 3DEC analysis while Steffen and other earlier workers used vector algebra limit equilibrium approaches.

The analysis of slope monitoring data frequently shows evidence of failure kinematics involving both plan and dip rotation. However our understanding of such failure mechanisms cannot be furthered through simplified 2-D models and requires the use of 3-D codes such as 3DEC, FLAC3D and Slope Model.

JOINT SPACING, PERSISTENCE AND KINEMATICS

The importance of considering the influence of joint spacing and persistence on rock slope kinematics has been demonstrated by Brideau (2010) and Brideau *et alii* (2012). Change in the spacing of joint sets has long been known to change the kinematics of rock slope failure and was demonstrated by Hencher *et alii* (1996) and in more recent modelling studies by Brideau (2010). The spacing and persistence of joint sets/ major structures also controls the number of blocks that are considered in the landslides. Research by Sitar & McLaughlin (1997) and Wolter *et alii* (this volume) has shown that the number of blocks considered during modelling of the Vajont landslide using DDA and DEM methods, respectively, influences the mobilized shear strength required at failure and the damage mechanism prior to failure. Internal distortion (dilation) during slope failure is also related to the blockiness of the rock mass and the shape of the failure surface and may be an important damage mechanism in large landslides. At the Revelstoke dam abutment in B.C., Canada, Corkum & Martin (2004) used 3DEC to clearly demonstrate the important role of blockiness (or number of blocks) in modelling rock slope displacement and dilation. At Vajont Mencl (1988) discussed possible internal distortion in the mid slope where the chair back meets the seat of the failure surface. Wolter *et alii* (this volume), Fig. 16, have shown clearly the influence of block size on potential internal slope damage (dilation). Internal dilation within slopes leads to zones of high fracturing throughout the slope. At the Aknes rock slope, major fractures are distributed along the slope reflecting significant damage and internal dilation. Modelling results using DEM codes have clearly shown the internal dilation associated with varied failure mechanisms (Brideau & Stead, 2012; Kinakin, 2005; Alz'oubi, 2009). It is suggested that internal dilation within a landslide is often an essential component of kinematic release that has received insufficient attention both in field studies and in geomechanical modelling.

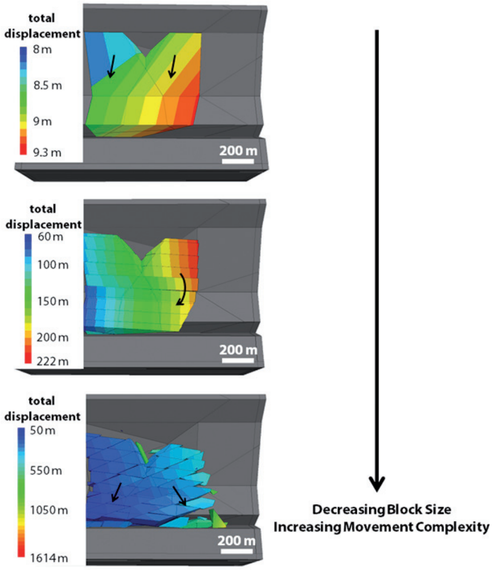

Fig. 16 - Influence of block size on failure mechanism and damage, Vajont rockslide, Italy. After Wolter et alii (this volume)

RELEASE SURFACES AND KINEMATICS

Release surfaces are an essential component of kinematic release for a landslide but are often ignored in field characterization studies unless 3-D numerical models are undertaken. Otherwise, they are assumed to be present but considered to be of negligible importance. What constitutes a "release surface" is in reality complex and often related to the scale of the instability. In small-scale rock slope failures, joint sets may provide adequate rear and lateral release. As the scale of the instability increases, release is less likely to be provided by discrete joints but may be provided by:

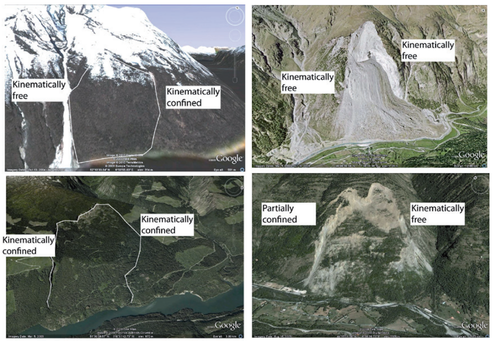

Fig. 17 - Lateral confinement mechanisms and major landslides. After Brideau (2010)

- Discontinuity step paths - along the dip or strike direction
- Major discrete structures - faults or shear surfaces
- Lithological contacts
- Combined jointing/intact rock fracture
- Slope excavation - i.e. anthropogenic
- Topographic features - e.g. gullies
- Previous adjacent instability.

Where closely spaced non-persistence joints exist, lateral release may be provided by stepping between sub-parallel joints. Release may thus need to overcome more resistance to sliding than that for continuous major structures. Release on faults will be influenced by the orientation of fault surface features such as slickensides and fault steps. Lithological contacts may form release surfaces where high angle bedding or boundaries between rock types of varying mechanical properties exist. Where non-persistent lateral release surfaces are present then failure may be associated with progressive damage of intact rock bridges and degradation of surface roughness. Rotational moments greater than the tensile strength of the rock may allow lateral release of wedges through extension of pre-existing joints and rock bridge failure (Hungr & Amann, 2011). In many cases, slope excavation or geomorphic processes may remove kinematic restraint. Such failures may be intimately linked with slope erosion through undercutting, gullying and drainage incision. Brideau (2010) examined several major rockslides in relation to their kinematic constraints and recognized three intuitive kinematic boundary conditions:

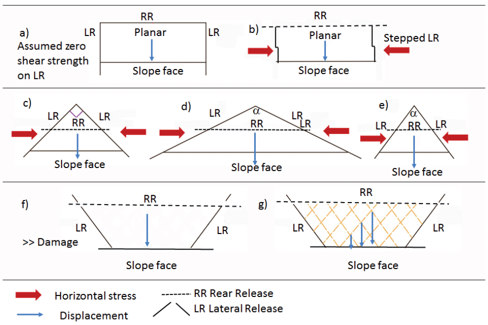

Fig. 18 - Plan view diagrams showing: a, b) Type I lateral release sub-perpendicular to strike of slope; c-e) Type II lateral release pentahedral wedges/hexahedral blocks with convergent lateral release surfaces; and f, g) Type III divergent lateral release surfaces and influence on damage associated with failure

- Failure confined on both sides - bounding structures required for lateral release
- Failure confined on one side (structure required) and unconfined on other side due to topographic feature (gullying)
- Failures unconfined on both sides – no structural release required (topographic promontory/nose).

The orientation of lateral and rear release surfaces with respect to landslide failure mechanism and damage has not been widely investigated or reported in the literature. Figure 17 shows a conceptual classification of landslides with respect to lateral and rear release orientation. Three main classes of landslide lateral release can be recognised:

- Conventional lateral release surfaces at right angles to the strike of the slope
- Convergent lateral release surfaces intersecting behind the slope crest
- Divergent lateral release surfaces with a notional intersection in front of the slope.

Figure 18a shows plan views of a Type I planar failure with vertical lateral release surfaces as defined by Hoek & Bray (1981). These may fail in a direction +/- 20 degrees of the dip direction of the slope as assumed in Wyllie & Mah (2004). This orientation minimizes frictional resistance on the lateral releases surfaces (if horizontal stresses are assumed to be zero). If the lateral releases are sub-vertical in dip or not parallel to the dip direction of the slope then the effect of gravitational stresses would be to mobilise an additional lateral release shear strength component. Tectonic stresses, if present, may result in elevated stresses in the horizontal direction acting normal to the lateral releases surfaces. It is also possible that lateral release surfaces may be stepped (Fig. 18b) and water pressures may act on lateral release surfaces in addition to the conventionally considered rear release or tension crack water pressures

Figures 18c-e, show plan views of Type II converging sub-vertical lateral release surfaces where the angle (α) between the release surfaces may play an important role in failure kinematics. If no tension crack or rear release surface is present, the geometry is that of a pentahedral non-daylighting wedge. The intersection between the lateral release surfaces may have a steep plunge with sliding and failure being determined by the dip and dip direction of the basal sliding surface. In practice the pentahedral wedge could vary from an over-hanging rotational (or toppling) wedge where the lateral release intersection plunges into the slope to an active-passive pentahedral wedge with a low plunge lateral release surface intersection and a daylighting basal release surface. The pentahedral wedges may be asymmetric or symmetric, and this would influence the failure mechanism (sliding/rotation). The angle (α) could have an important kinematic confinement effect; with obtuse angles of intersection between the lateral release surfaces, confinement would be reduced and horizontal forces if present may act to displace the wedge out of the slope (Fig. 18d). With an acute angle (α) between the lateral release surfaces there would be significant confinement acting on the pentahedral wedge and horizontal forces could act to clamp the wedge into the slope (Fig. 18e). The size of the angle (α) is analogous to the 'wedge angle' in conventional tetrahedral wedge analysis (Hoek & Bray, 1981) and the influence of horizontal forces in relation to the inter-lateral release surface angle is similar conceptually to the influence of wedge shape noted for roof wedges in tunnels (Goodman 1989). However, these effects have not been considered in the analysis of pentahedral non-daylighting wedges particularly with respect to the inter-relationship between failure kinematics and slope damage required for failure. It should also be noted that non-daylighting pentahedral wedges may also fail through composite basal surfaces involving failure of intact rock bridges. If a rear release surface or tension crack is present sub-parallel to the slope strike then this reflects the geometry of a hexahedral block failure sliding along the basal surface and involving varying components of frictional resistance due to the lateral release surfaces. Applying block theory, the blocks defined by the converging lateral release surfaces in Fig. 18 would be considered removable. The blocks bounded by the diverging lateral release surfaces, Type III (Fig. 18f,g), are not however amenable to block removal and failure could be associated with increased damage/intact rock fracture. This trend of release surfaces could result in a pentahedral failure mechanism controlled not by the lateral release surfaces but by the spacing of sub-parallel joint sets (Fig. 18g). It is suggested that failure might be more liable to occur by progressive unravelling of smaller pentahedral keyblocks allowing the constraints due to block theory imposed on the slope to be overcome.

This simple conceptual consideration shows the importance of considering the orientation of release surfaces in landslide investigations and not relying solely on conventional kinematic planar and tetrahedral (wedge) methods of analysis. Further work is ongoing to characterise major rockslides according to the lateral release geometry and to constrain rock slope failure mechanisms by three-dimensional geomechanical models such as 3DEC and Slope Model (Itasca 2013a, b) incorporating varied release geometries.

GROUNDWATER AND KINEMATICS

The importance of groundwater pressures in landslides has received considerable research in soil landslides but is less well understood in rock slope failures. Picarelli *et alii* (2012) provide an excellent summary of the state-of-the-art with respect to groundwater studies in landslides. Recent research in large open pits is of direct relevance to our improved understanding of landslides, particularly in rock. Beale (2009) and Dowling *et alii* (2011) provide an extremely useful review of the factors controlling groundwater flow and pressures in rock slopes in addition to state-of-the-art discussion of the importance of depressurisation of open pit slopes and the current trends in groundwater-geomechanical modelling. The importance of faults is highlighted in relation to their acting either as conduits for groundwater flow or impermeable barriers. Faults in the latter case may act to compartmentalize the groundwater within the slope; this effect plays an important role in faulted terrains (Bonzanigo *et alii*, 2007). Beale (2009) discusses the varied approaches to groundwater modelling in rock slopes and concludes that in most cases an equivalent porous medium approach has proved to be adequate for slope design.

In landslide studies, although groundwater pressures are routinely considered in two-dimensional limit equilibrium methods, their consideration in 3-D analyses is less common. Data uncertainty with respect to measured groundwater levels and model uncertainty related to complexities in the structural geology makes the realistic incorporation of groundwater pressures and fracture permeability in landslides a challenging area for future research. In particular, little work has been undertaken to consider the role of groundwater pressure variations on slope damage (a notable exception is Smithyman, 2007), or the incorporation of groundwater into the kinematics of block movement in 3-D distinct-element models. The relationship between damage/fracturing and groundwater pressures requires a coupled hydro-mechanical approach. The presence of faults and joints controls both the pathways for fluid flow and kinematic release, with elevated pore pressures contributing to localized fracturing and damage, which in turn creates new fluid flow pathways, redistributed pore pressures, and corresponding slope movements in response to these evolving changes in the effective stresses (Fig. 19). High pore water pressure may induce damage during landslide development and in contrast this damage/fracturing may eventually lead to a reduction in pore water pressure and a process of self-stabilization (albeit perhaps temporary). This process in a rock slope may occur in cycles of induced movement/stabilisation with changes in fluid pressure as observed at Vajont. Each cycle could lead to a cumulative increase in slope damage and catastrophic failure upon reaching a critical slope damage threshold.

HOW CAN WE MODEL DAMAGE AND KINEMATICS EFFECTIVELY IN LARGE LANDSLIDES?

A detailed description of the state-of-the-art in the numerical modelling of landslides is beyond the scope of the current paper. The authors would refer readers

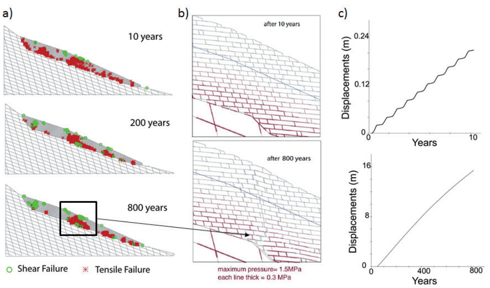

Fig. 19 - a) Progression of yielded elements with alternating seasonal high and low groundwater tables over a 800 year period for the Campo Vallemaggia creeping rockslide, Switzerland. b) Progressive development of an internal shear surface causing the slide to break into two halves and the corresponding redistribution of pore pressures. c) Modelled displacement vs. time plots showing the seasonal stick-slip portions of the landslide motion, most clearly identifiable at the resolution of 10 years. After Smithyman et alii (2009)

to Stead & Coggan (2012) for a more detailed discussion. In this paper we propose to focus on the potential use of modelling to characterize the principal subjects discussed so far, that is, damage and kinematics.

As discussed earlier, rock bridges and discontinuity persistence are a fundamental component of understanding the mechanisms of failure of large landslides. Simulating the influence of rock bridges in rock slope failure can be undertaken implicitly or explicitly. Limit equilibrium methods consider rock bridges implicitly by assuming an apparent cohesion and friction associated with the percentage of rock bridges along a potential failure surface. Continuum mechanical models also treat rock bridges implicitly by incorporating rock bridge degradation through an equivalent (although hard to define) reduction in rock mass properties. Conventional discontinuum methods assume continuous through-going or interconnected joints and hence also use a scaled apparent cohesion and friction along the joints to allow for rock bridges. Explicit treatment of rock bridges has received considerable attention during the last 10 years. In the simplest variants of the explicit method, discrete non-persistence fractures are assumed and methods of simulating intact rock/rock mass fracture between joints are applied to incrementally model the development of a continuous failure surfaces. The most recent variants of the explicit method of rock bridge simulation incorporate discrete fracture networks within a slope and then simulate rock bridge fracture between the variously oriented discontinuities. These methods allow two- and three-dimensional simulation of fracture in rock slopes using a synthetic rock mass approach. In the following section, we will present a brief overview of numerical codes and their application to modelling damage, rock bridge failure and failure kinematics.

MODELLING DAMAGE AND KINEMATICS

Modelling of brittle fracture or damage associated with rock slope failure prior to 2000 was largely undertaken using simple boundary element (displacement discontinuity) methods, Scavia *et alii* (1996). This early modelling produced some valuable insights into the role of rock bridges in rock slope instability. More recently, there has been a significant increase in the number of studies that have considered brittle fracture in landslides and rock slopes. Today a wide variety of codes can be used to simulate brittle fracture allowing further insight into the mechanics of large landslides. It is emphasised that considering our current state of knowledge, brittle fracture methods should be used as one component of a toolbox approach including conventional continuum and discontinuum codes.

Three principal methods have been used to model 2-D brittle fracture in landslides in both recent research and practice:

• Distinct element (UDEC) Voronoi "Damage" models
• Particle Flow Codes (PFC)
• Hybrid finite-discrete element methods (FDEM)

Two-dimensional UDEC Voronoi methods have been used for almost 20 years however their application to rock slopes is quite recent. Yan (2008) used a combined toolbox approach of UDEC Voronoi, FDEM, boundary element and continuum FEM methods to simulate rock bridge failure in rock slopes. Franz (2009) and Alzo'ubi (2009) in perhaps the most detailed works to date used UDEC Voronoi to successfully simulate both natural rock slope failures and open pit mine slopes. Figure 20a shows UDEC Voronoi models of a high engineered rock slope incorporating rock support and realistic inclusion of joints sets. This work showed the importance of the degradation of the tensile strength of the rock mass on slope instability. UDEC Voronoi methods conventionally discretize the rock mass into polygons a process

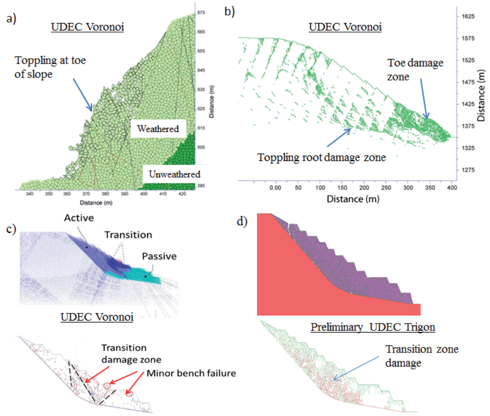

Fig. 20 - Simulated damage: a) UDEC Voronoi model of rock slope at Checkerboard Creek, British Columbia, Canada (after Alzo'ubi, 2009); b) UDEC Voronoi model of rock slope at Highland Valley Copper Mine, British Columbia, Canada (after ALZO'UBI, 2009); c) UDEC Voronoi model of active-passive rock slope showing influence of rock bridge content in the transition zone (after Tuckey, 2012); and d) preliminary UDEC Trigon model of active-passive transition zone (after Gao, 2013

known as tessellation. Properties can then be given to the polygonal boundaries to represent either joints or intact rock. Tuckey (2012) and Tuckey *et alii* (2013) used UDEC Voronoi to simulate the influence of varied percentages of volumetric rock bridge content in the transitional zone between an active-passive wedge (similar to the situation in the western section of the Vajont slide) (Fig. 20b).

A recent development in the UDEC Voronoi method has been developed by Gao (2013) where the polygons are divided into triangular blocks and the properties are then given to each face of the triangle. This method appears to provide a much more deformable rock mass which is less influenced by the polygonal shape of the blocks.

Two dimensional Particle Flow Code models have been used successfully to model surface mine slope failure mechanisms, natural rockslides and runout. They have the advantage over 3-D PFC codes in being able to accommodate larger problems but arguably do not realistically model kinematics as realistically. Recent developments have seen the incorporation of a smooth joint model to simulate joints (Lorig *et alii*, 2009) and more recently a flat joint model (Potyondy, 2012), with the latter allowing the simulation of the correct compressive to tensile strength ratio of the simulated rock. The strength of the rock mass in a PFC model is simulated by the bonds between circular particles. Breakage of the bonds allows realistic simulation of brittle fracture and damage. Figure 21 shows examples of PFC2D in the simulation of rock slope failures.

The Finite-Discrete Element code, ELFEN, has

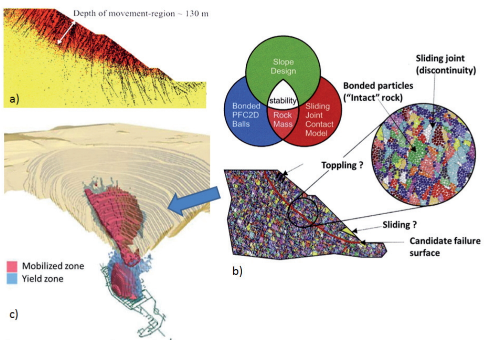

Fig. 21 - Rock slope modelling using Particle Flow Code, PFC, and Synthetic Rock Mass, SRM: a) PFC2D model of rock slope (after Lorig et alii, 2009); b) SRM approach (after Mas Ivars et alii, 2011); and c) FLAC 3D model of open pit slope using SRM derived data (after Sainsbury et alii, 2008)

been used successfully by several researchers in simulating both natural and engineered slopes. Eberhardt *et alii* (2004) describe the use of the ELFEN code in successfully simulating the stages of damage and failure during the Randa Rockslide, Switzerland (Fig. 22a) and illustrate the importance of brittle rock mass damage on instability. Figure 22b shows the use of ELFEN in simulating the damage associated with footwall failures common in high mountain ranges and surface coal mines. The zones of extensile induced damage in the upper slope and the compressive induced damage in the active-passive toe are clearly visible and agree with both continuum and other brittle fracture codes. Stead *et alii* (2006) summarize the applications of ELFEN to varied natural failure mechanisms and Yyazmensky *et alii* (2010) illustrate the integrated use of a Discrete Fracture Network (DFN), Fig 22d, and ELFEN in the successful simulation of a 900 m high failure at the Palabora mine in South Africa (Fig. 22e). The use of ELFEN to date however has been limited to 2-D brittle fracture modelling of rock slopes. Recent work by Hamdi *et alii* (2013) has shown the successful simulation of 3-D damage at the laboratory scale; the extension of the code to 64 bits within the near future may allow simulation of larger scale three-dimensional brittle fracture problems.

The development of 3-D brittle fracture-damage modelling methods for large open pit slopes has seen considerable advances over the last five years as part of the Large Open Pit (LOP) Project. A Synthetic Rock Mass (SRM) approach was developed using PFC3D models and incorporating a DFN derived from field data (Mas Ivars *et alii*., 2011). This model was used to investigate the scale effects and anisotropy of rock mass strength for different lithological units. Samples from the laboratory scale to 80 m in height were tested in uniaxial, triaxial and tensile conditions to simulate the rock mass strength. These synthetic rock mass strengths were subsequently used to develop FLAC3D models incorporating directional weakness in a strain-softening ubiquitous constitutive model. FLAC3D models with strengths constrained against the SRM modelling were then used to successfully model mine scale problems at the Palabora and Northparkes block caving mines (Sainsbury *et alii*, 2008).

The most recent development in the 3-D brittle fracture/damage modelling of rock slopes has been the introduction of the Lattice Spring code, Slope Model (Itasca, 2012). This code allows realistic modelling of brittle fracture in large rock slopes. The code in essence replaces the spheres in a PFC3D model with nodes. The bonds between the PFC spheres are replaced by springs, Fig. 23a. A discrete fracture/joint network can be incorporated within the lattice spring model and brittle fracture damage/cracking is simulated by breakage of the springs. Figure 23b shows a Slope Model simulation of a non-daylighting wedge as discussed previously in the paper. It is possible to simulate progressive failure through the failure of

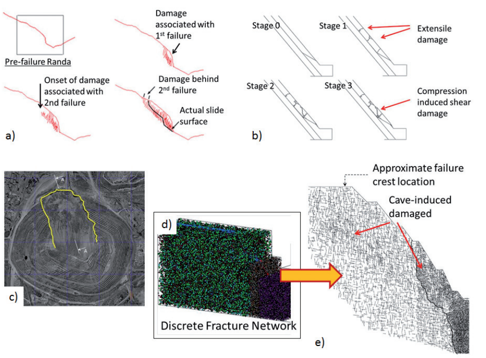

Fig. 22 - a) FDEM ELFEN simulation of Randa rockslide showing stages in slope damage and failure; b) ELFEN simulation of a high mountain/footwall slope showing zones of extensile and compression-induced rock slope damage (after Havaej et alii, 2013); c) massive, 900 m high, open pit rock slope failure at Palabora, South Africa; d) constructed DFN for Palabora model simulation; and e) FDEM-DFN ELFEN simulation of Palabora pit slope failure (after Vyazmensky et alii, 2010)

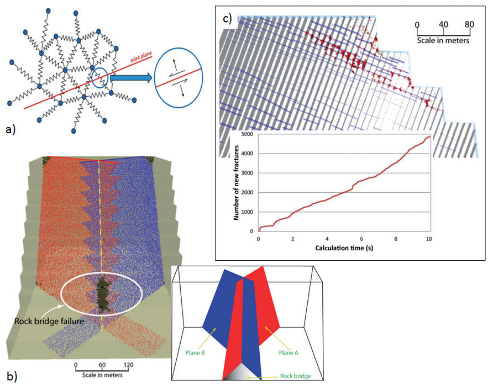

Fig . 23 a) Lattice Spring treatment of stiffnesses and strengths between particles; b) Slope Model lattice spring simulation of a non-daylighting wedge failure; and c) simulation of toppling failure with plot showing number of new cracks generated as a function of calculation time steps

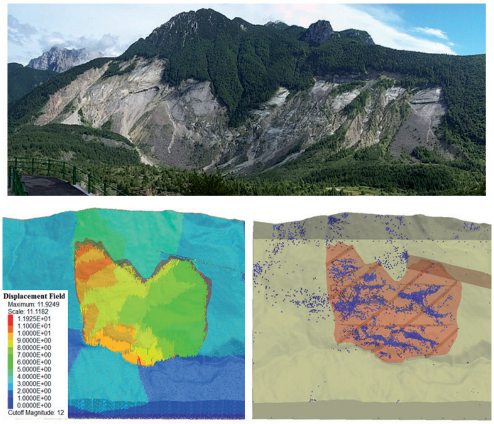

Fig. 24 Preliminary Slope Model lattice spring simulation of the Vajont rockslide. After HAVAEJ *et alii (2013)*

rock bridges either as patches along the failure surfaces or as failure between joints, Fig. 23c. Figure 24 shows a preliminary Slope Model simulation of the Vajont rockslide. As discussed by WOLTER *et alii* (this volume), the cracking within the rock slope is concentrated in the zone between the active-passive blocks in the western half of the slide. This preliminary work shows the accumulation of internal damage within the Vajont Slide with numerical time. Future work will investigate the influence of discrete fracture networks and incorporate groundwater pressures. (Slope Model is able to couple brittle fracture propagation and groundwater pressures in a fracture rock mass).

A recent development by GAO (2013) has been to incorporate the Voronoi Trigon logic into 3DEC. This has allowed the use of 3DEC to simulate brittle fracture at both the laboratory and to a lesser extent the field scale. Although computing overhead may currently limit full scale 3DEC Voronoi models in size this development allows the possibility to consider large scale block movements, with fracture enabling kinematic mobility in critical areas of the slope.

Kinematic analysis of rock slopes by definition is predominantly a 3-D technique which has hitherto often been limited to simple stereographic methods. The use of 2-D models can only be used in simple cases to examine failure surface kinematics. The use of 3-D distinct-element models is an area for future research in assessing the combined roles of damage, groundwater pressure, and structural control on instability. The 3DEC code has been shown by SAINSBURY *et alii* (2007), YAN (2008), STROUTH & EBERHARDT (2009), FRANZ (2009), BRIDEAU (2010), and others to be particularly useful in kinematic rock slope studies. The inter-relationship between 3-D fracture permeability, groundwater pressures and kinematics has received little attention and is an interesting avenue for future research. The 3DEC code assumes through-going joints although a persistence factor has been used to indirectly investigate the influence of non-persistence on rock slope instability, BRIDEAU *et alii* (2012). The influence of non-persistent joints and groundwater on rock slope kinematics is an important area for future research which could be approached with a combine 3DEC – Slope Model approach. The influence of seismic amplification on rock slope stability (both damage and kinematics) is also an area where three dimensional distinct-element models may find important use building on the results of 3-D continuum models.

DISCUSSION: CAN WE BETTER OPTIMIZE FIELD DATA COLLECTION?

Detailed descriptions of the state-of-the-art in monitoring methods for landslides have been presented in several recent works (EBERHARDT & STEAD, 2011; STEAD *et alii*, 2012). These touch on several critical parameters and important areas that require further data to improve our understanding of the mechanics of landslides:

- Structural geological mapping
- Discontinuity persistence and rock bridges
- Failure surface morphology
- Rock mass quality, block size and shape
- Groundwater and fracture permeability
- Engineering geomorphology

The importance of building an accurate structural geological map has long been recognized in landslide studies, as rock slope failure mechanisms are intimately related to the structural geological history. The influence of tectonic deformation on rock mass quality, damage and kinematic controls, together with the stress-path to which the rock slope has been subjected, has been shown to be important in geomechanical modelling. The usefulness of considering rock slope fractography in the stability of exfoliation domes has been discussed by TUCKEY (2012) using both field mapping and ground-based LiDAR.

TUCKEY (2012) also presents an excellent summary of historic rock bridge research and highlights both the variation in assumed rock bridge content in previous publications, Table 1. Numerous workers have consid-

LANDSLIDE	MEASUREMENT	ROCK BRIDGE CONTENT (%)	REFERENCE
Palliser slide (Alberta, Canada)	Length %	2 to 3	STURZENEGGER (2011)
Aknes slide (Norway)	Length %	1 to 3	GRØNENG ET AL. (2009)
Highway road cuts (Idaho, USA)	Length %	0 to 3	RISTAU (1994)
Gypsum mine (Arizona, USA)	Length %	16 to 36	LEBARON (2011)
Diavik A154 Pit (NWT, Canada)	DFN; length %	>5 25 3 to 45	KARAMI ET AL. (2007) MOFFITT ET AL. (2007) ELMO ET AL. (2011)
Rockfall (French Sub-Alps)	Failure scar mapping; area %	0.2 to 5.0	FRAYSSINES & HANTZ (2006)
Failed roof slab above road	Failure scar mapping; area %	26	PARONUZZI & SERAFINI (2009)

Tab. 1 - Summary of selected case studies quantifying intact rock bridge content

ered rock bridges in the geomechanical modelling of rock slopes including GRØNENG *et alii (*2009, 2010) at the Aknes rock slope in Norway, and STURZENEGGER & STEAD (2012) at the Palliser rockslide in Canada. TUCKEY (2012) clearly demonstrates the advantages, but also the challenges, in using remote sensing techniques (LiDAR and photogrammetry) to provide estimates of both discontinuity persistence and rock bridge content in a rock slope at several field study sites. These involve the integrated use of Discrete Fracture Network Engineering approaches to characterise rock bridge content and persistence distributions (STURZENEGGER, 2010; TUCKEY, 2012; TUCKEY *et alii*, 2013). TUCKEY (2012) also provide detailed characterization of step-path geometry, while STURZENEGGER *et alii* (2011) also discuss the challenges in the production of DFN's from remote sensing data.

LiDAR and photogrammetry also present a valuable means to collect data, post failure, on the characteristics of failure surfaces in large rockslides. STURZENEGGER & STEAD (2012), WOLTER *et alii* (this volume), DONATI *et alii* (2012) have all shown clearly the utility of remote sensing methods in failure surface characterization at the Palliser (Canada), Vajont (Italy) and Hope (Canada) landslides, respectively. Excellent examples of the use of geophysics and monitoring in characterizing sub-surface shear surfaces/zones have been presented by GANEROD *et alii* (2008), WILLENBERG (2008) and KALENCHUK (2011) for the Aknes (Norway), Randa (Switzerland) and Downie (Canada) rockslides, respectively. The ability to import failure surface morphology such as fold undulations and step-path geometry into geomechanical models means that there is no longer a need to over-simply or assume failure surface geometries. Clearly, it is also important to use engineering judgement and not to add unnecessary detail to the modelling process.

The ability of remote sensing techniques such as LiDAR and photogrammetry to cover large and inaccessible areas of the rock slope means that methods of using this data to complement rock mass characterization (GSI, RMR, etc.) should continue to be developed. To-date remote sensing methods have largely been used to acquire discontinuity orientation data; it is essential that the scope of data collected remotely be broadened to maximize the use of the data. This data should ideally provide input for geomechanical models such as block size and shape. KALENCHUK *et alii* (2010) describe methods for treating block size distributions and classifying block shape. The collection of data required for Discrete Fracture Network generation (orientation, trace length, intensity and termination) should be undertaken where possible if more sophisticated SRM based numerical models are to be used. The consideration of geostatistics when characterising the rock mass strength using systems such as GSI has also been shown to be an important issue in geomechanical modelling, JEFFERIES *et alii*, (2006). Continued laboratory testing studies are required to provide improved estimates of parameters used in rock slope modelling including joint normal and shear stiffness as well as rock mass dilation. Future developments in constitutive criteria incorporating brittle fracture under low confinement may also require a shift in emphasis on laboratory testing procedures toward characterization of damage and associated developments in geomechanical models.

Collection of groundwater data for landslides requires piezometer installations. Multiple vibrating wire piezometers (bentonite isolated or grouted-in) are commonly used in open pit slopes. Alternatives include multiple piezometer installations such as the Westbay system. Characterization of groundwater in major natural rock slopes is often subject to significant data and model uncertainty due to the complexity of the fracture system controlling flow and pore pressure distributions, limitations in access, and cost. Remote sensing can be used to characterize seepage in natural and open pit slope. VIVAS *et alii* (2013) shows the successful use of both LIDAR and thermal imaging in the characterization of seepage in natural and open pit rock slopes and proposes a seepage intensity method for characterizing seepage.

Finally, the authors stress the importance of undertaking engineering geomorphological mapping as part of a landslide investigation. Such mapping not only provides information on past geomorphic process that may have influenced the instability, but also emphasizes the importance of considering landform evolution. A landslide may have been influenced by a geomorphic stress-path whereby processes such as river incision, tectonic uplift and glaciation, glacier retreat, weathering, etc., serve to bring the current slope to a critical damage threshold. At Vajont, Wolter *et alii* (this volume) clearly show the importance of considering engineering geomorphology and how geomorphic processes can be used to constrain geomechanical models.

CONCLUSIONS

In this keynote paper the authors have deliberately made no attempt to provide a detailed discussion of all aspects of landslide failure mechanics, but instead have concentrated on the important role of rock slope damage as a major driver of rock slope instability. Our paper introduces the concept of rock slope damage and then attempts to place it in context with the wide array of controls in which damage is manifested in a slope. Our intention is to stimulate a different way of viewing landslides which will hopefully lead to an improved understand of the complexity of landslide mechanics. The kinematics of a landslide is emphasized here as being a critical element in any landslide study, which is all too often simplified to fit with simplistic models. Our discussion on kinematics is intended to emphasise that landslides are invariably three dimensional entities with three–dimensional structure, rock mass quality, topography and groundwater characteristics. Numerical modelling has advanced to include sophisticated two and three –dimensional codes – it is however in the hands of the user to choose the correct code and to treat the problem in its correct kinematical manner.

The development of geological, hydrogeological, structural, rock mechanics and geotechnical models (as used in the open pit mining sector) is crucial to understanding the mechanics of landslide. Recent developments in data collection are also intrinsically three-dimensional providing point clouds of 3-D data which must be optimised if their benefits are to be maximised in understanding the mechanics of landslides. Finally we end by emphasising that our future understanding of the mechanics of landslides will require a more effective way of visualising the hidden interior of a rock slope. State-of-the-art remote sensing methods now provide rock slope surface data, and must in turn be integrated with surface and subsurface monitoring data, 3-D geomechanical models, and most importantly future developments in geophysical techniques, if our understanding of the mechanics of complex landslides is to continue to improve. A major obstacle to improving our understanding of the mechanics of major landslides is the high level of existing model and parameter uncertainty. The use of a toolbox approach in forensic investigations of major landslides is an essential step in reducing model uncertainty whereas integrated geomechanical modelling-rock slope characterization and modelling will allow a reduction in data uncertainty.

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