

Back analysis of an ancient rockslide at Lake Fundudzi, Limpopo Province, South Africa

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Abstract A massive rockslide occurred in the Soutpansberg Mountains of the Limpopo Province ~20,000 years ago that blocked the course of the eastern flowing Mutale River forming Lake Fundudzi, which is the only known true inland lake in South Africa. The landslide occurred in the northwestward dipping and highly jointed Fundudzi Formation of the Soutpansberg Group. The failed slope was observed to be a homogenous jointed sandstone rock slope which is quartzitic in places. Cluster analysis, which uses fuzzy clustering in Rocscience Dips, revealed three major joint sets (JS1, S0 and JS3) in the rock mass with an average dip and dip direction of 85°/237°, 15°/283° and 73°/157°, respectively. The main failure plane best correlated with JS3, which dips towards the lake. The kinematic analysis also revealed that the mechanism of failure was planar which is in agreement with site observations. The strong rock material classification of the sandstone from a laboratory index and mechanical tests was one of the indicators that failure could have been mostly structurally controlled. The technique of back-analysis was employed using RocPlane in order to investigate the geotechnical conditions that may have resulted in failure. The results obtained from the sensitivity analysis in RocPlane showed

that the joint roughness coefficient (JRC) followed by the residual basic friction angle (ϕ_r) were the main influential shear strength parameters on the factor of safety, F , as opposed to the joint wall compressive strength (JCS) which has very little impact on F for the range of values considered. Results from back-analysis show a range of possible combinations of JRC and ϕ_r values under different groundwater and seismic conditions. Whilst the groundwater shows more impact than the seismic coefficient, the field evidence points to seismic activity as a more plausible trigger than groundwater. This study shows the usefulness of back-analysis techniques in assessing various conditions of failure that may have resulted in ancient rockslides. Although it may not result in a unique set of geotechnical parameters that may have resulted in failure, it, however, gives an insight into probable causes of failure and, thus, can be useful in mitigation of future failures.

Keywords Rockslide · Lake Fundudzi · Kinematic analysis · Back analysis · Planar failure

Introduction

Landslides are one of the major natural disasters and account for enormous property damage in terms of both direct and indirect costs (Dai et al. 2002). The annual costs of landslide-associated expenses in South Africa were estimated at approximately US\$20 million (Paige-Green 1989). Based on an annual standard inflation rate of 6%, the current associated expenses would be US\$90 million. In landslide studies, assessment is often made after failure has occurred. However, a proper assessment of a landslide can prevent future occurrence of landslides or minimize the hazards associated with such failures.

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Lake Fundudzi is a particular landslide site of great interest situated in the heart of the Soutpansberg Mountains in the Limpopo Province of South Africa. It is a 3-km-long and 1-km-wide inland freshwater body formed by a landslide event that blocked the course of the eastward flowing Mutale River. The landslide occurred ~20,000 years ago in the northward dipping quartzitic sandstone of the Fundudzi Formation of the Soutpansberg Group (Van der Waal 1997).

The main aim of this study was to investigate the geotechnical conditions that could have led to the landslide that formed Lake Fundudzi by employing the techniques of back-analysis based on the limit equilibrium procedures using RocPlane in Rocscience® Inc. software. An extensive discontinuity survey was carried out using a combination of scanline survey and cell or window mapping due to the hazardous nature of the terrain and the limited rock mass outcrop. The data obtained was used to carry out a kinematic analysis based on Markland's (1972) test using the Dips software in Rocscience in order to determine the type of failure mechanism that may have occurred. Laboratory tests were conducted on representative samples in order to geotechnically characterise the site and to obtain the relevant parameters for the analysis.

Location of study area

Lake Fundudzi is located in the Mutale River valley of the eastern Soutpansberg Mountains in the Limpopo Province (22°50'22.08"S and 30°18'36"E) near the Kruger National Park. The Limpopo Province is the northern-most province in South Africa and borders Mozambique, Zimbabwe and Botswana. The study area lies approximately 57 km north east of Louis Trichardt and 21 km north-west of Thohoyandou (Fig. 1).

Geology of the area

The geology of the study area is characterised by sandstone of the Fundudzi Formation. Barker et al. (2006) described the geology as sandstone and quartzitic in places, locally gritty or a conglomerate with interbedded basaltic lava, tuff, shale, agglomerate and siltstone (Fig. 2).

The slope of the landslide was observed to be a homogenous rock slope. The sandstone, which is quartzitic in places, was the only rock type mapped during fieldwork. This homogenous rock mass slope attains a maximum height of 320 m in the area from the lake's water level to the crest of the landslide. The sandstone is pinkish to reddish brown in colour, and medium- to fine-grained. The bedding is preserved and heavy jointing of the rock mass is evident.

Thin section microscopic analysis of the sandstone indicates that it is composed predominantly of quartz

(+95%) and minor amounts of secondary carbonate minerals. Distinctly visible also on stage rotation under crossed polarized light is an undulatory extinction of quartz grains which indicates that the quartz minerals had undergone deformation. Grain to grain contact was evident and almost interlocking, indicating recrystallization. The majority of the sediments show high sphericity (0.7–0.9) and roundness (0.5–0.9) of the mineral grains and, in general, the sorting is good. Quartz veins are also present. Silica cements around the quartz grains forming very hard sandstone.

According to Van Eeden et al. (1955), the Fundudzi fault running longitudinally through the lake has no relation to the formation of the landslide. Barker (1979) mapped two NW–SE and SW–NE intersecting faults which form the failure release surfaces (or buttresses). Their orientation coincides with other inferred faults mapped in the region. A lineament that was identified in this study from Google Earth satellite imagery is the one that coincides with the tension crack at the scarp of the landslide (Fig. 3).

This lineament extends far beyond either of the lateral release surfaces for over 500 m from each release surface side. Several minor rock slides and falls are still active and noticeable on each of the release surfaces, and were also noted by Janisch (1931). A 23-m-deep vertical tension crack, which is inactive, was also observed at the crest of the failure plane.

The dips of the beds on the fractured sides of the release surfaces are 30° north on Dongwe and 45° north on Chintanda (Fig. 3). The rock mass itself is heavily jointed.

Climate, drainage and geohydrology

The climatic conditions of the study area are characterised by summer rainfall, with an average annual precipitation of 1280 mm. The area has a subtropical climate which is hot and dry. Extreme cold and sharp frosts are exceptional and during June and July, the climate is very pleasant. Thunder showers occur during the summer months (i.e. October–March) only (Van Eeden et al. 1955).

The lake is fed by three streams draining a catchment of more than 6000 ha, i.e. Mutale 2480 ha, Godoni 2270 ha and Muiladi 1270 ha as shown in Fig. 4. The Mutale catchment is highest (up to 1438 masl) and receives rainfall which is >1200 mm per annum and accordingly has the greatest influence on the lake (Van der Waal 1997). The Mutale River is important because Lake Fundudzi was formed in it as a result of the large landslide. At the lower end of the lake, the water flows out through a mass of loose rock blocks (Van Eeden et al. 1955). At the time of this study, there was no detailed literature on the local geohydrological conditions. Also, no springs or any form of

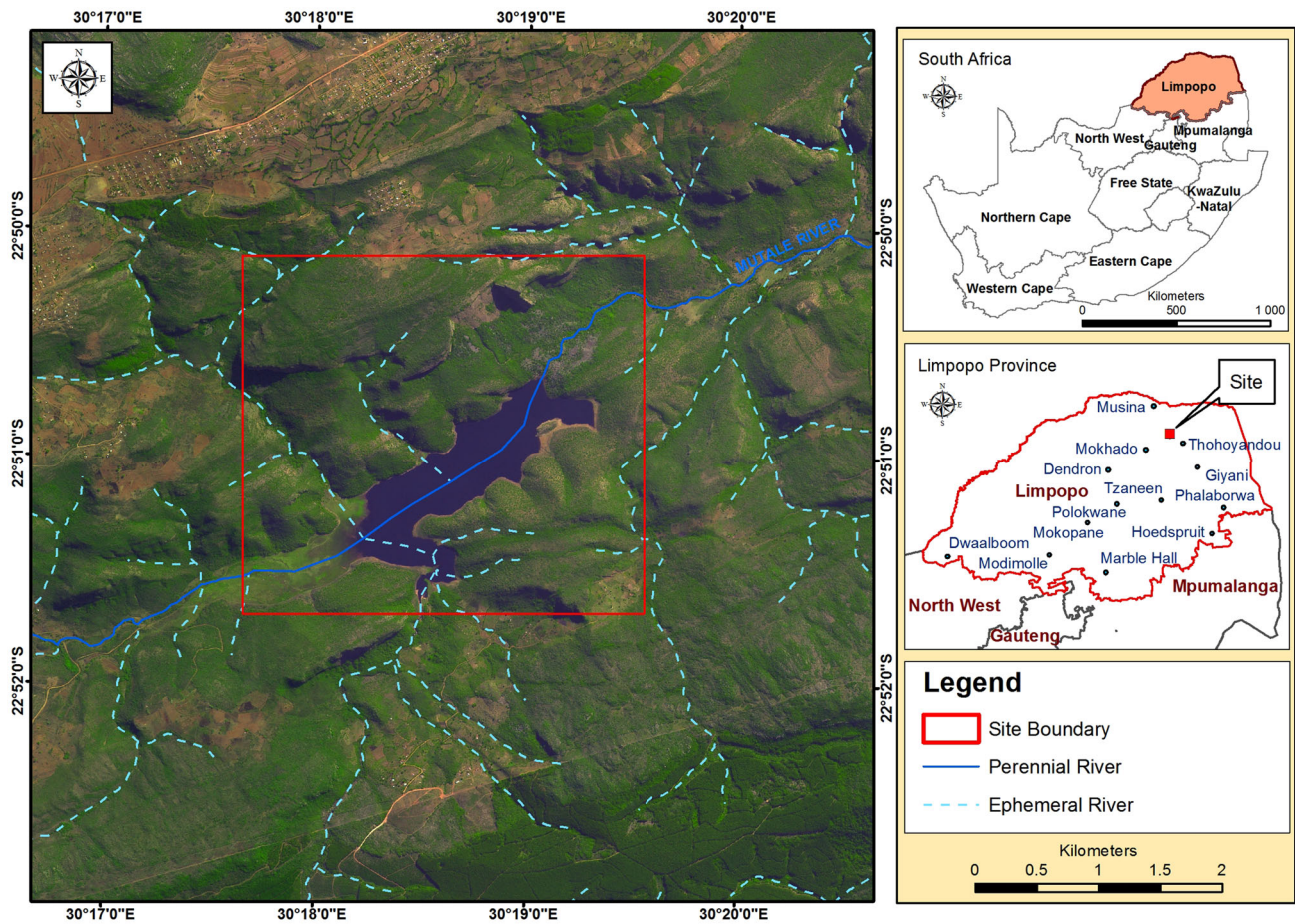


Fig. 1 Location map of Lake Fundudzi

water drainage features were observed through the landslide from the scarp down to the lake. Kristen et al. (2007) stated that in order to understand natural climate variability, it is important to have long, continuous and well-dated climatic records from the continents and oceans of both hemispheres. Such records are rare in the southern hemisphere, and especially in southern Africa. The longest and most continuous record of precipitation in South Africa is the sedimentary record from the Tswaing crater lake (formerly “Pretoria Saltpan”). The 1.13-km-wide Tswaing crater is located approximately 40 km northwest of Pretoria at an altitude of 1045 m above sea level and approximately 400 km from Lake Fundudzi. Dry and worst case groundwater scenarios were both used for determining the probable significant landslide trigger.

Description of Lake Fundudzi and the rockslide

The valley of Lake Fundudzi runs roughly NE–SW. It is long and narrow with high fringing mountains and on the northern side, which is where the failure plane is located,

the valley walls are steeper than 60° (Fig. 5). The eastern end is dammed by a landslide of immense proportion which has blocked the valley where a projecting spur from the south had narrowed it (Janisch 1931).

The shore of the lake is steep and rocky, made up of angular blocks and loose rubble lying at the angle of repose (~45°) and marginally stable, needing only a trigger to set it into motion. The landslide, which piled up its largest blocks against the highest part of the spur, consists of great angular blocks of sandstone, many of them exceeding 7 m in diameter, lying randomly distributed with great spaces between them making climbing extremely difficult. The landslide has obviously come from the northern mountain side where a great block of rock has slipped from between two high buttresses (release surfaces) which are about 500 m apart. The scar extends back for about 230 m from the faces of the very steep buttresses, leaving a bare and steep rock slope above the accumulation of rock debris (Fig. 5). The mountains on the northern shore of the lake are very high and steep. There is a high and steep escarpment which increases in height towards the east until it culminates in the two buttresses, Chitanda and Dongwe,

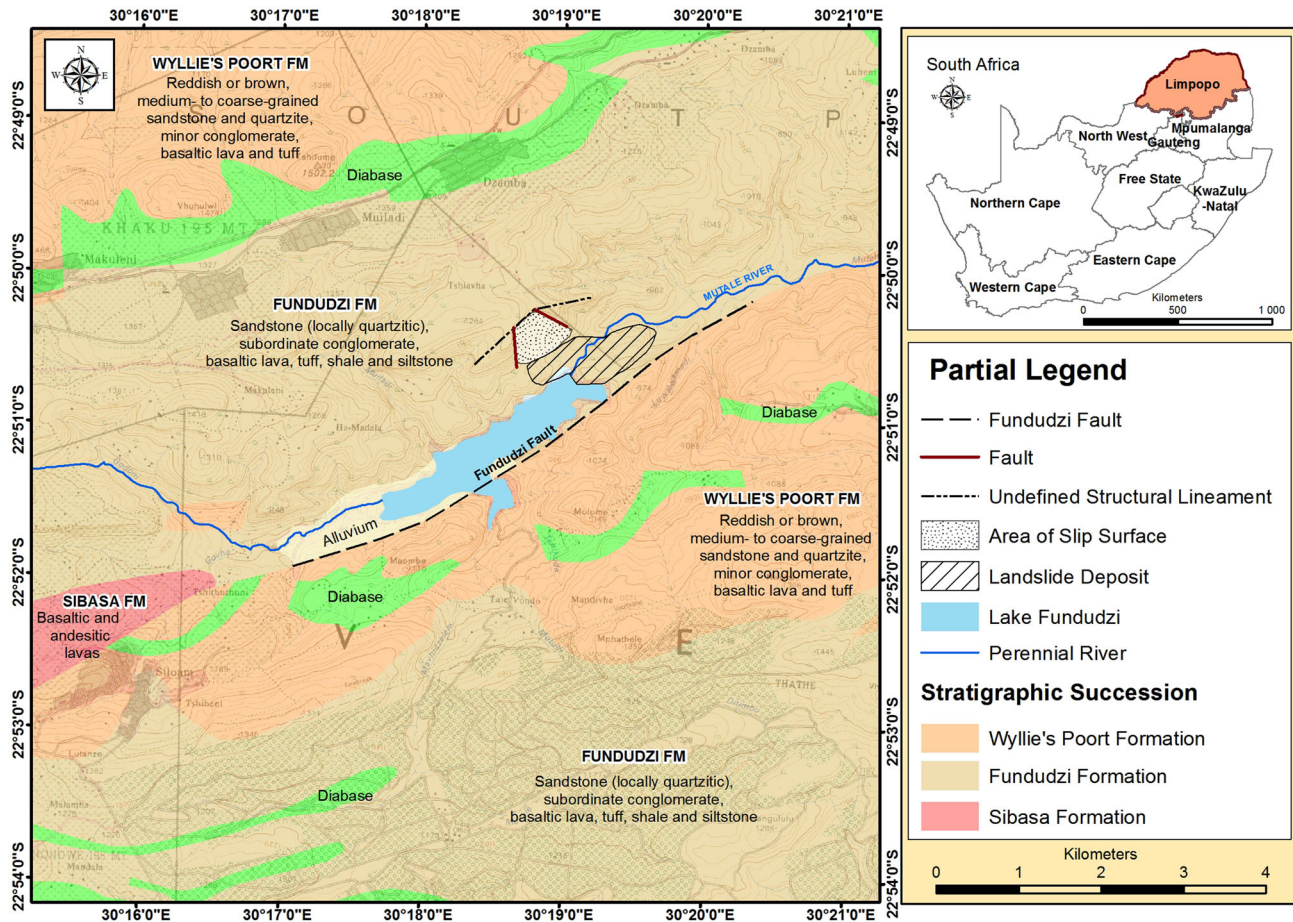


Fig. 2 Local geological map of the Lake Fundudzi landslide area

both at more than 300 m above the lake (Janisch 1931). Beyond them, the escarpment continues eastwards, but becomes lower, and ultimately disappears. The portion between the two buttresses which had slid down formed the central focus of this study (Fig. 5). The near vertical rock surfaces of these release surfaces, Chitanda and Dongwe, are over 120 m high in places from the failure surface. This bare slip surface that forms a smooth rock slope between the buttresses above the landslide debris and lake is at an angle of at least 60°. Several minor rockfalls and debris flows appear to have taken place recently on the eastern side of each buttress. The scars of the minor slide are characterised by fresh pink colouration of the sandstone whilst the scar of the main slide is greyish in colour. According to the Cruden and Varnes (1996) classification of landslides, the ancient landslide that occurred at Lake Fundudzi classifies as a rock slide.

Chiliza and Richardson (2008) also stated that the landslide is ~500 m in length, 350 m high above the lake, and has a failure surface area of ~22 ha. The calculated volume of rock that slid down the failure surface is

7–9 million m³. The farthest blocks, which are the largest, travelled a distance up to 700 m across the valley. The largest blocks have a diameter between 7 and 10 m (Chiliza and Richardson 2008).

The great powers of Lake Fundudzi have been kept a closely guarded secret by the Venda people, who have preserved it for generations. The lake is often associated with myth and legends of the Venda people and is believed to be protected by a python god, who has to be pacified annually with gifts of traditionally brewed beer (Khorombi 2000). Special permission must be granted by the Netshiava royal family to visit the lake.

Discontinuity survey and kinematic analysis

Discontinuities play a major role in slope stability as their presence affects the mechanical and hydrogeological properties of the rock mass (Bye and Bell 2001). Prominent jointing is evident continuously throughout the failed slope rock mass. Therefore, these discontinuities were studied in

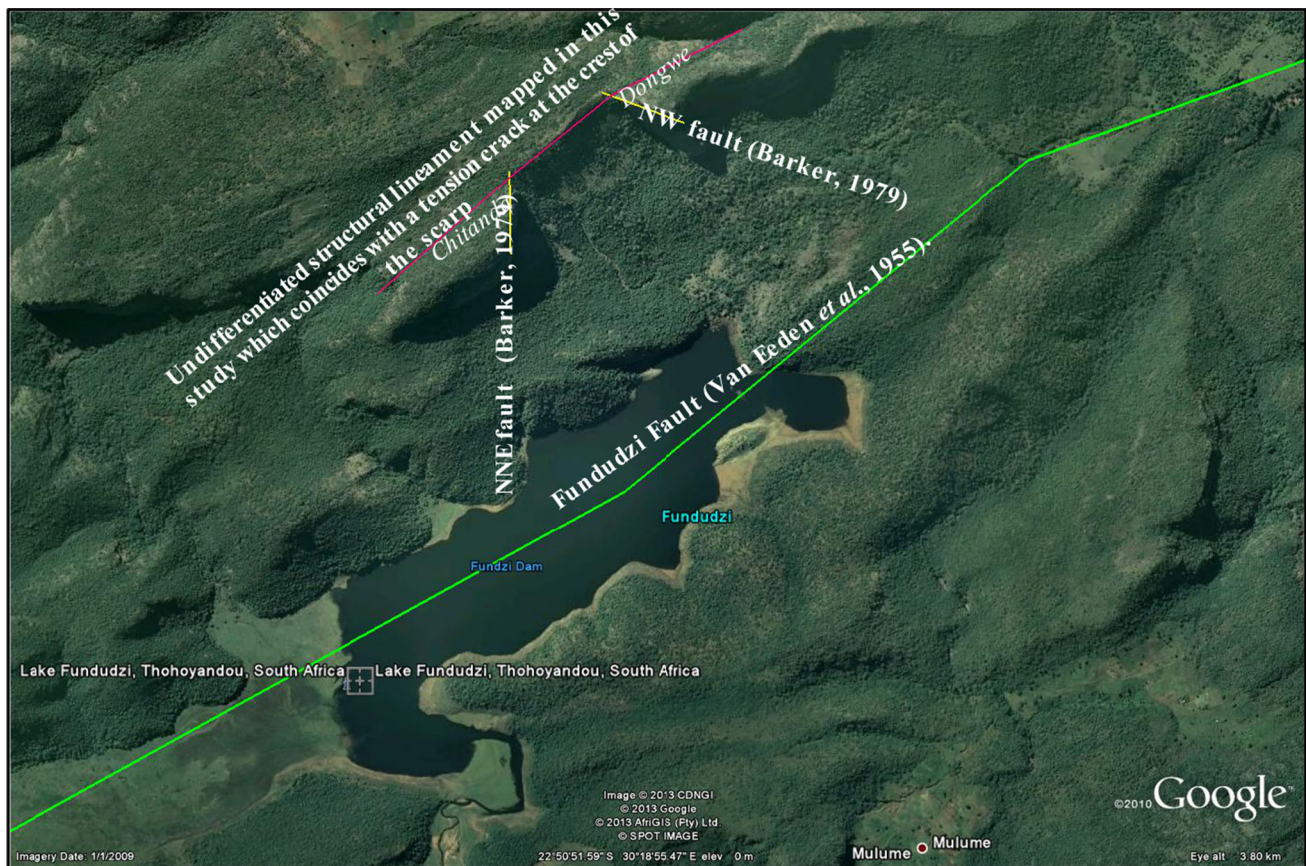


Fig. 3 A Google Earth[®] satellite imagery showing major structural features in the vicinity of the site

detail to assess the possible failure mechanisms using kinematic analysis as prescribed by Markland (1972).

The basic technique used in mapping surface or underground exposures is the scanline survey (Brady and Brown 2006). However, due to the hazardous nature of the terrain coupled with the limited rock mass outcrops, it was impossible to carry out a full-scale scanline survey. As a result, cell or window mapping was used to collect discontinuity data. Measurement of dip and dip direction of the discontinuities was done at outcrop locations where data was collected by making use of a Clar type geological compass manufactured by Breithaupt Kassel. Wherever possible, the conditions of the discontinuities such as spacing, roughness, water conditions and filling were recorded. Mathis (1987) suggested that sparsely scattered joint sets are not readily detected with scanline mapping, but are generally detected by cell mapping; hence, local variations in properties are easily detected, resulting in better knowledge of variability. Over 200 discontinuity datasets, where rock mass outcrop exposures permitted, were recorded.

It was observed during field investigation that the possible mode of failure of the landslide was planar. A kinematic analysis was thus undertaken in order to

confirm this. The data obtained from the discontinuity survey was analysed using Rocscience's Dips. The data was plotted as poles, and cluster analysis, which uses fuzzy clustering, was performed to determine the major joint sets. Subsequently, the mechanism of failure was determined by analysing the relationship between the major joint sets and the slope face by Markland's test as suggested by Hoek and Bray (1981). The dip and dip direction of the plane representing the pre-failure slope was found to be $80^{\circ}/130^{\circ}$ on the basis of topography and face angle of remaining buttresses on either side of the failure surface. Based on joint surface conditions, which were predominantly slightly rough to rough, a value of 35° was chosen for the frictional angle for the quartzitic sandstone as prescribed by Hoek and Bray (1981).

About 200 poles were plotted on an equal angle, lower hemisphere projection and the poles were contoured. Three sets of joints were subsequently determined: S0 (bedding), JS1 and JS3 (discontinuities; Fig. 6).

The maximum concentration of poles for S0 is approximated at $15^{\circ}/283^{\circ}$. In the case of JS1 and JS3, the maximum pole concentrations are approximated at $85^{\circ}/237^{\circ}$ and $73^{\circ}/157^{\circ}$, respectively.

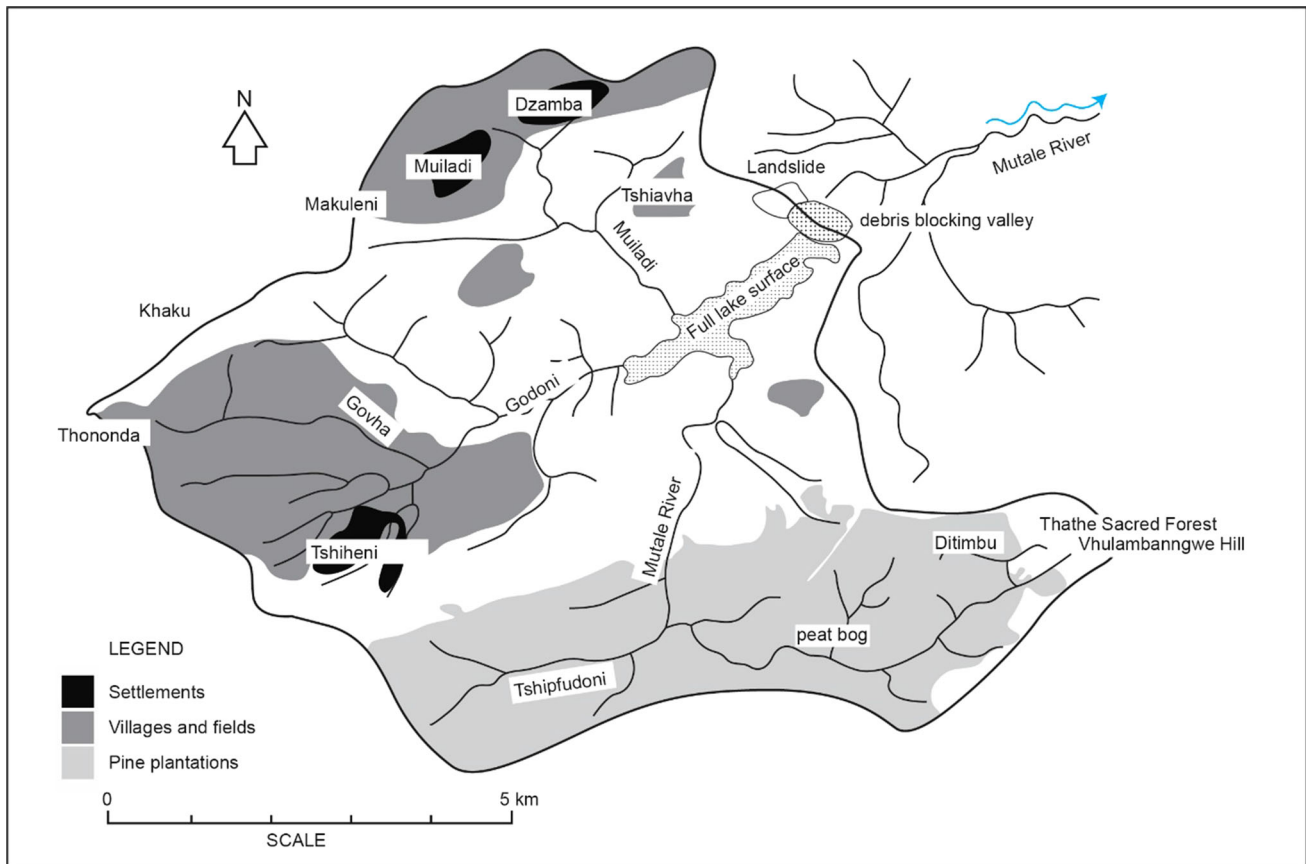


Fig. 4 Catchment area of Lake Fundudzi and associated land uses (modified after Van der Waal 1997)

In order to analyse for plane failure, the pole friction cone of 90° and 35° plunge dips was estimated to represent the friction of the rock mass (Fig. 7). No measurements were carried out on the lateral release faults (joints); however, from field observations, their dips and dip direction could be related to JS3 and JS1, respectively. These faults played a lateral release mechanism role to allow block sliding on a single plane. The daylight envelope is visible and marked with a black circle. According to Markland's test, any pole that falls outside the cone represents a plane that could slide if kinematically possible. Hence, there was no chance for S0 to slide because it falls within the friction cone area on the stereonet and dips into the slope. Based on the analysis, there is an indication that (JS3) poles represent planes which would slide and were susceptible to plane failure.

A crescent-shaped zone formed by the daylight envelope and the pole friction circle encloses the region of planar sliding.

Toppling and wedge kinematic analyses revealed probabilities of 8.45 and 31.89% for all planes, respectively. Though wedge sliding analysis showed the second highest percentage of probability, no wedges were identified in the field.

Rock mass classification

Determination of the strength of an in situ rock mass by laboratory testing is generally not practical. As a result, the rock mass strength is estimated from geological observations and from test results on individual rock pieces or rock surfaces which have been taken from the rock mass (Hoek 2007).

Rock mass rating (RMR) geomechanical classification was applied to characterize the slope rock mass, in conjunction with a slope mass rating (SMR) classification which is basically a modification from the basic RMR in order to adapt it for slope stability evaluation as defined by Romana (1993). The RMR and SMR systems are also comparable. The RMR classification incorporates six parameters, which are uniaxial compressive strength (UCS) of rock material, rock quality designation (RQD), spacing of joints, condition of joints, groundwater conditions and orientation of joints (Bieniawski and Orr 1976).

The rock mass class determined from the 5 basic parameters was 74, which according to Bieniawski (1989), falls in class II for good rock masses (Bieniawski 1989) before discontinuity orientation adjustment. Rating adjustment for unfavourable discontinuity orientation was

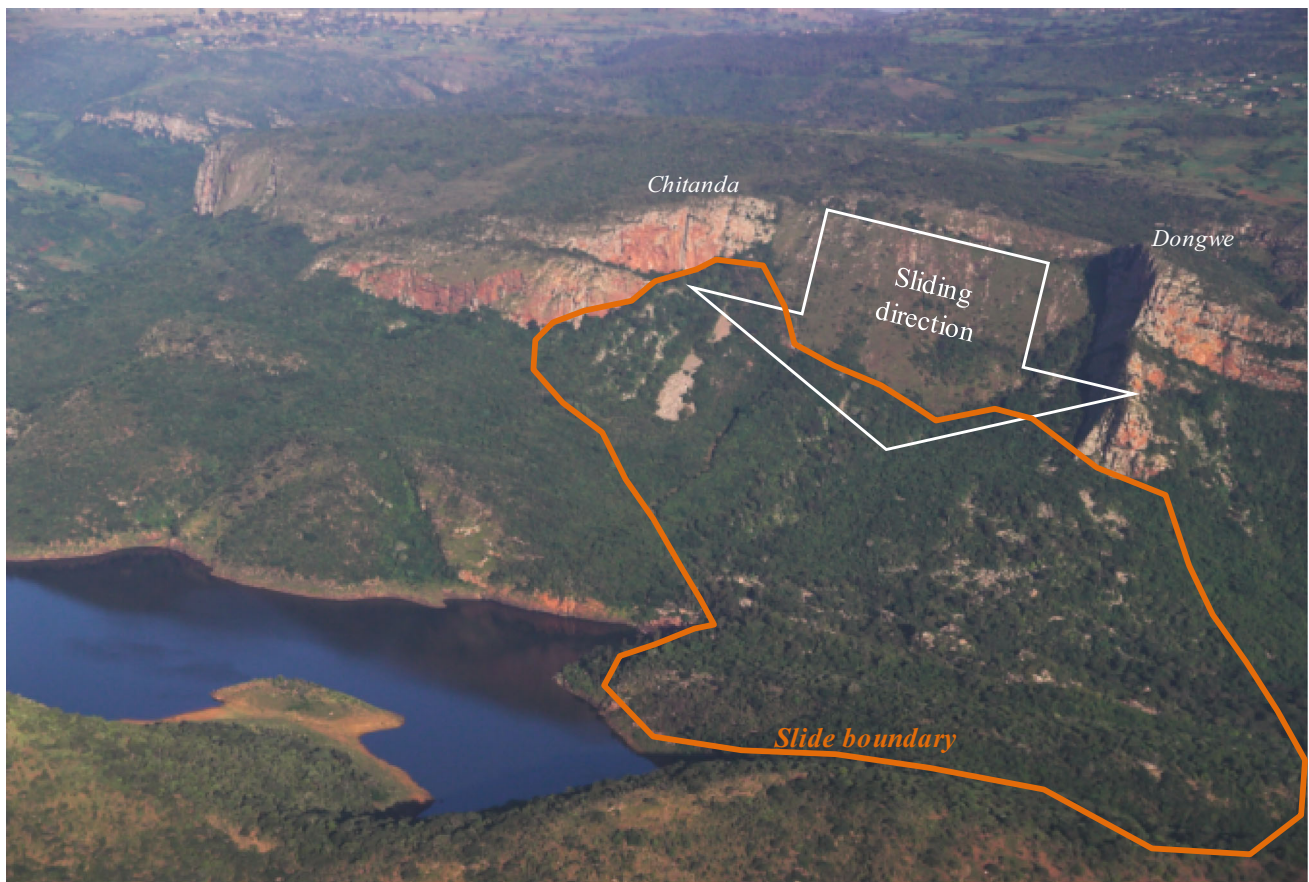


Fig. 5 Aerial view of the Lake Fundudzi landslide situated in the Soutpansberg mountains

set at (–50) for the slope due to unfavourable JS3 described in the previous section which strikes almost parallel to the slope face and dips out of the slope. Hence, the basic RMR after discontinuity orientation adjustment for application to the slope was $(74-50) = 24$. The total rock mass classification determined from the six total ratings was class IV for poor rock mass.

The SMR is defined as:

$$\text{SMR} = \text{RMR} + (\text{F1} \times \text{F2} \times \text{F3}) + \text{F4}, \quad (1)$$

where F1 ranges from 1.00 to 0.15 depending on parallelism between joints and slope face strikes; F2 ranges from 1.00 for joints dipping more than 45° to 0.15 for joints dipping less than 20°; F3 reflects the relationship between the slope face and joint dip; and the F4 adjustment factor for the methods of excavation is fixed empirically as +15 for natural slopes, which is the same as rating adjustment for discontinuity orientations in Bieniawski (1989).

Based on the SMR value obtained, the slope could be classified as class IV, which classifies the rock mass as ‘bad’. According to Romana (1993), such a slope will have a probability of failure of 0.2. Clearly, from the rock mass classification results using both RMR and SMR, the

presence of unfavourable major discontinuity orientations had an adverse effect on the stability of the slope and the rock mass strength.

Geotechnical characterisation of the site

The shear strength of joints is a key property in slope stability analysis and design of engineering structures. Determination of friction angles expressed as basic, residual or peak friction angle are fundamental to the understanding of the shear strength discontinuities in a rock mass (Hoek 2000). Determination of reliable shear strength values is a critical part of a slope stability analysis because relatively small changes in shear strength can result in significant changes in the safe height or angle of a slope. Rock slopes generally fail along existing geological defects, notably discontinuities (Bye and Bell 2001).

Representative samples of the major joint sets were collected for Golder shear box testing. The Golder shear box is designed for testing samples at normal loads of up to 2 MPa and the normal load is applied by means of a dead load system (Hencher and Richards 1989). The dead

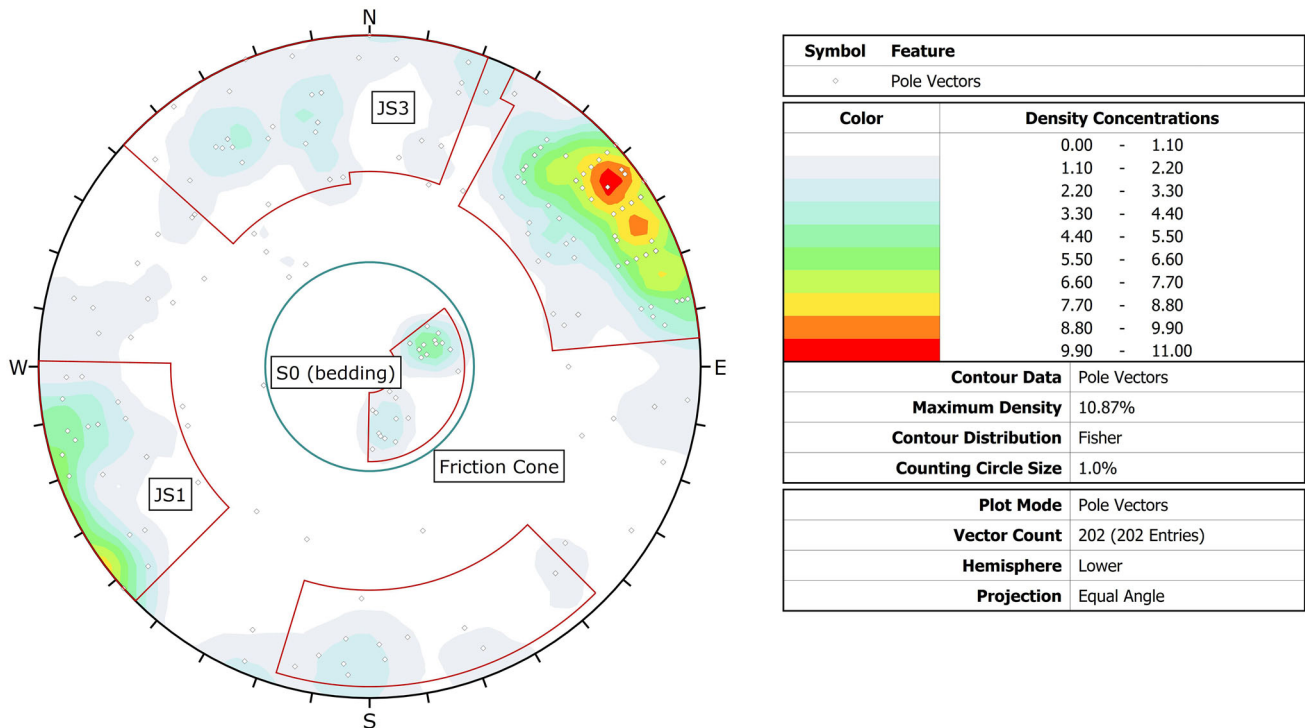


Fig. 6 Stereoplot showing the contoured plot of the poles and the three joint sets

load remains constant throughout the test and is, therefore, more sensitive than the portable shear box. The samples were tested at a normal stress range of 0.38–0.60 MPa. As pointed out by Barton (1976), the maximum effective normal stress acting across joints considered critical for stability lies in the range 0.1–2.0 MPa. Prior to testing, the joint roughness coefficient (JRC) of each sample was estimated based on Barton and Choubey's (1977) ten standard roughness profiles. Samples were also collected for index testing in accordance with the ISRM (1981)-suggested methods for rock characterization, testing and monitoring, in order to assess the geotechnical properties of the rock material on site. Tilt tests were also undertaken on samples to obtain an approximate value of the basic friction angle of the sandstone using a method similar to that described by Barton and Choubey (1977).

A summary of the geotechnical properties of the various tests conducted on the sandstone material collected is shown in Table 1. The Golder shear box testing at normal stresses ranging from 0.38 to 0.60 MPa and estimated JRC values ranging from 9 to 13 revealed peak shear strengths which range from 0.17 to 0.37 MPa. The average basic friction angle obtained was 27°, with a range of 25°–31° which fell within the range of values for sandstone (i.e. 26°–35°) as stated by Barton and Choubey (1977). As pointed out by Barton and Choubey (1977), the peak shear strength values

obtained from the Golder shear box tests and the results from the tilt tests provide a more reliable means of estimating the JRC by back-calculation, than comparison with the typical standard roughness profiles. The back-calculation of JRC can be useful in determining joint profiles because of the subjectivity in visually matching joint surface profiles with the ten standard roughness profiles whose JRC value ranges from 0 to 20 (Miller et al. 1989). The back-calculated JRC values using the Barton and Choubey (1977) empirical equation are in close proximity to the visually matched joint profile estimates for this study. Results from index tests showed that the rock has a generally medium to high strength (ISRM 1979) based on Schmidt hardness rebound, point load and unconfined compressive strength tests. It is evident from these results that the rock mass had to be affected by other geological factors such as joints, discontinuities and faults (i.e. structurally controlled) for the slope to have failed and not rock strength (i.e. rock material degradation).

Back-analysis of the rockslide using RocPlane

Back-analysis of the landslide was carried out in RocPlane in Rocscience to determine the shear strength parameters and the possible geotechnical conditions that may have led to failure.

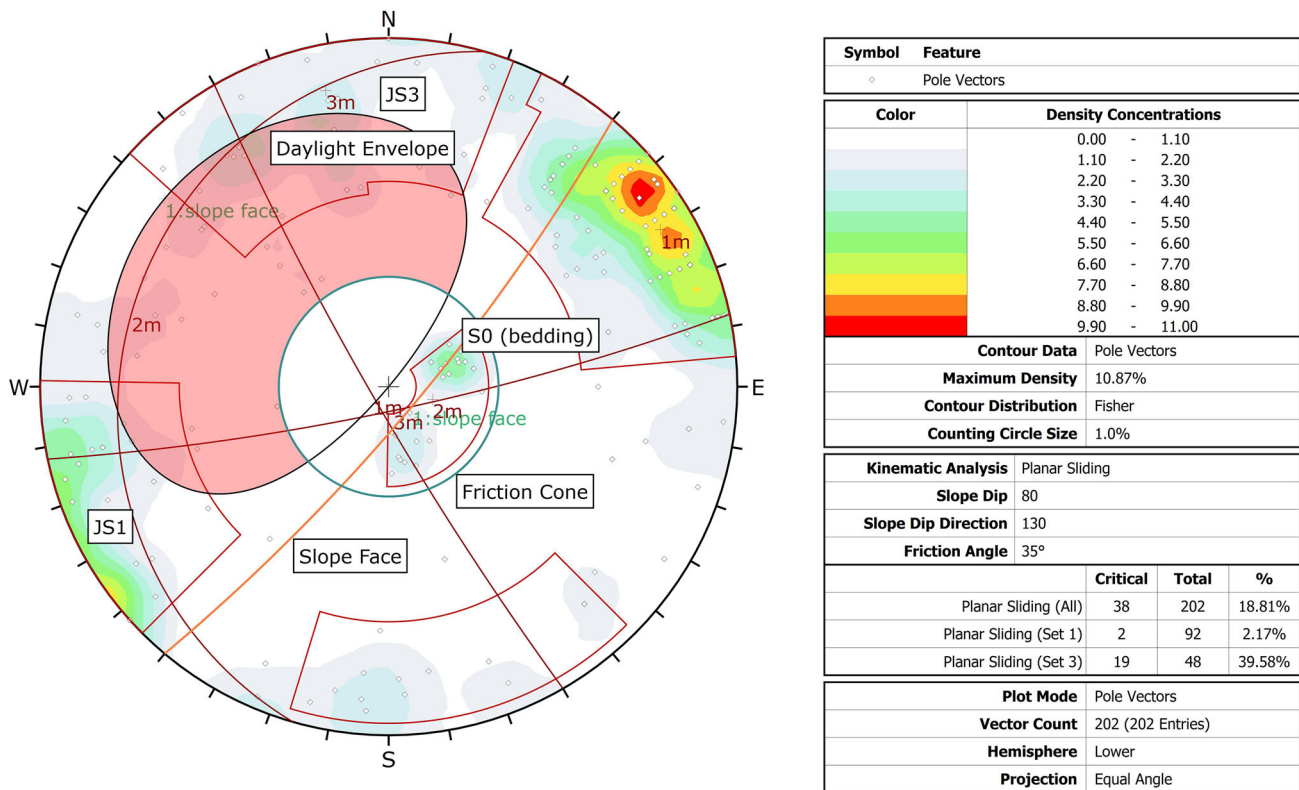


Fig. 7 Stereonet plot showing analysis of plane failure using a daylight envelope and friction cone

As failure was structurally controlled or occurred along a plane, the use of RocPlane was considered to be suitable for the analysis. Stead et al. (2006) and Sharifzadeh et al. (2009) stated that if the rock mass is competent with a good overall rock mass condition as in the case of the study area, its instability mechanism lies on a plane and can thus be analysed using the limit equilibrium method of analysis. Thus, the Janbu simplified method was used for the analysis. As stated by Fredlund et al. (1981), the advantage of using the Janbu simplified method is that it is less prone to convergence issues and tends to give a lower bound on the factor of safety, F . The Barton and Bandis (1990) criterion, which establishes the shear strength of a failure plane, was selected over the popular Mohr–Coulomb failure criterion (i.e. cohesion, c and friction angle, ϕ) which is satisfied by an infinite set of answers, which means that many possible (c , ϕ) combinations will give $F = 1.00$. The Barton and Bandis criterion results in a curvilinear failure envelope that more closely represents the actual physical characteristics of shear resistance. Another major advantage of the Barton–Bandis approach is the relative ease at which the shear strength parameters (i.e. JRC, JCS, σ_n and ϕ_b) can be established (Donnelly and Rigbey 2007).

In order to perform a back-analysis of the failed slope, it was necessary to survey the dimensions and location of sections through the slide. Therefore, a simple slide-block 2-D slope geometry was reconstructed that satisfied planar geometrical conditions. The cross-section was taken through the centre of the failure surface (Fig. 8) which represents the lowest sliding basal surface for landslide volume estimation. Oppikofer (2009) stated that volume calculations of ancient rockslide may be feasible through aerial photographs and topographic maps by surface reconstruction based on the contour line pattern outside or within the scar area. No pre-landslide geometry and aerial photographs could be obtained for the 20,000-year-old Lake Fundudzi landslide. This study made use of the slope height, failure plane, dip and slope angle data collected in the field, topographical maps and Google Earth® satellite imagery to determine the pre- and post-failure geometry. The slope shape used for the pre-failure condition was obtained by contour line smoothing of the topographical map with a 1:50,000 scale.

Figure 9 shows a reconstructed slope geometry of the failure surface in 2-D and the slope dimensions. This section was used in the back-analysis of the landslide. The slope geometry is considered to be realistic and valid,

Table 1 Results of geotechnical properties of the rock material obtained from laboratory testing

Type of test	Range	Average	Number of samples tested
Schmidt hardness rebound test			
Rebound values (R)			
Vertical	28–50	37	17
Horizontal	28–48	37	37
UCS by Miller (1965), (MPa)			
Vertical	50–135	82	17
Horizontal	50–130	79	23
Point load test			
Is (50)	1.37–12.28	5.51	29
UCS given by Broch and Franklin (1972), (MPa)	31–282	126	29
Direct unconfined compressive strength test (UCS) (MPa)	145–250	189	5
Unit weight, kN/m ³	28.5–28.6	28.6	5
Indirect tensile (Brazilian) strength test (MPa)	4.1–7.8	5.7	6
Tilt angle test:	25–31	26.5	15
$(\phi_b$ —basic friction angle), (°)			
Golder shear box			
Estimated JRC	9–13	10	7
Peak shear strength (MPa)	0.17–0.37	0.26	6
Residual shear strength (MPa)	0.10–0.25	0.18	6
Back-calculated JRC values	9.7–10.4	10	6

particularly for an analysis in RocPlane which requires upper face, slope face, failure plane and where present, a tension crack.

Selection of geotechnical parameters for analyses

Barton and Choubey's (1977) method was used in the study because failure occurred along a persistent structural discontinuity. Accordingly, the laboratory testing programme achieved parameters that satisfy Barton and Choubey's (1977) estimate of shear strength, i.e.:

$$\tau = \sigma_n \tan [\phi_b + \text{JRC} \times \log_{10}(\text{JCS}/\sigma_n)], \quad (2)$$

where JRC = joint roughness coefficient, JCS = joint compressive strength, τ = shear strength along the rock joint, σ_n = normal stress acting on the surface of the rock joint and ϕ_b = angle of frictional sliding of the planar surface (basic friction angle).

The values used for the analysis are based on the values shown in Table 1. El-Ramly et al. (2002) pointed out that the performance of a structure is often controlled by the average soil properties within a zone of influence, rather than soil properties at discrete locations. It is, therefore, logical to use average values for the geotechnical properties in a slope stability analysis as was the case in this study.

According to Bell (2007), the JCS is equivalent to the UCS of the rock if the joint is unweathered which was determined from laboratory testing. The JRC was

determined by back-calculation and peak shear strength. An average unit weight value of 28.5 kN/m³ was calculated for the quartzitic sandstone and was used in the analyses. This value was kept constant throughout the analyses as the variability of the unit weight on F is insignificant due to the fact that the variability in material density is small (Alonso 1976).

Also important to mention is that the use of ϕ_r in the adopted Barton and Bandis (1990) shear strength criterion is based on weathering of joints. For fresh, unweathered and slightly weathered joints, $\phi_r = \phi_b$ where ϕ_b = basic friction angle.

It should be pointed out that the laboratory tests measure only a small part of the landslide. Cornforth (2005) stated that whereas the laboratory tests measure only an infinitesimally small part of a landslide, back-analysis presents an attractive procedure as the scale is much larger and the parameters obtained reflect the strength of the entire landslide mass. However, laboratory testing of representative samples help in the geotechnical characterization of the site as it gives an idea of the likely values of the necessary geotechnical parameters of the different materials needed for the analysis.

Groundwater conditions for analyses

Determination of the groundwater conditions at the time of a landslide is important because the pore water pressure

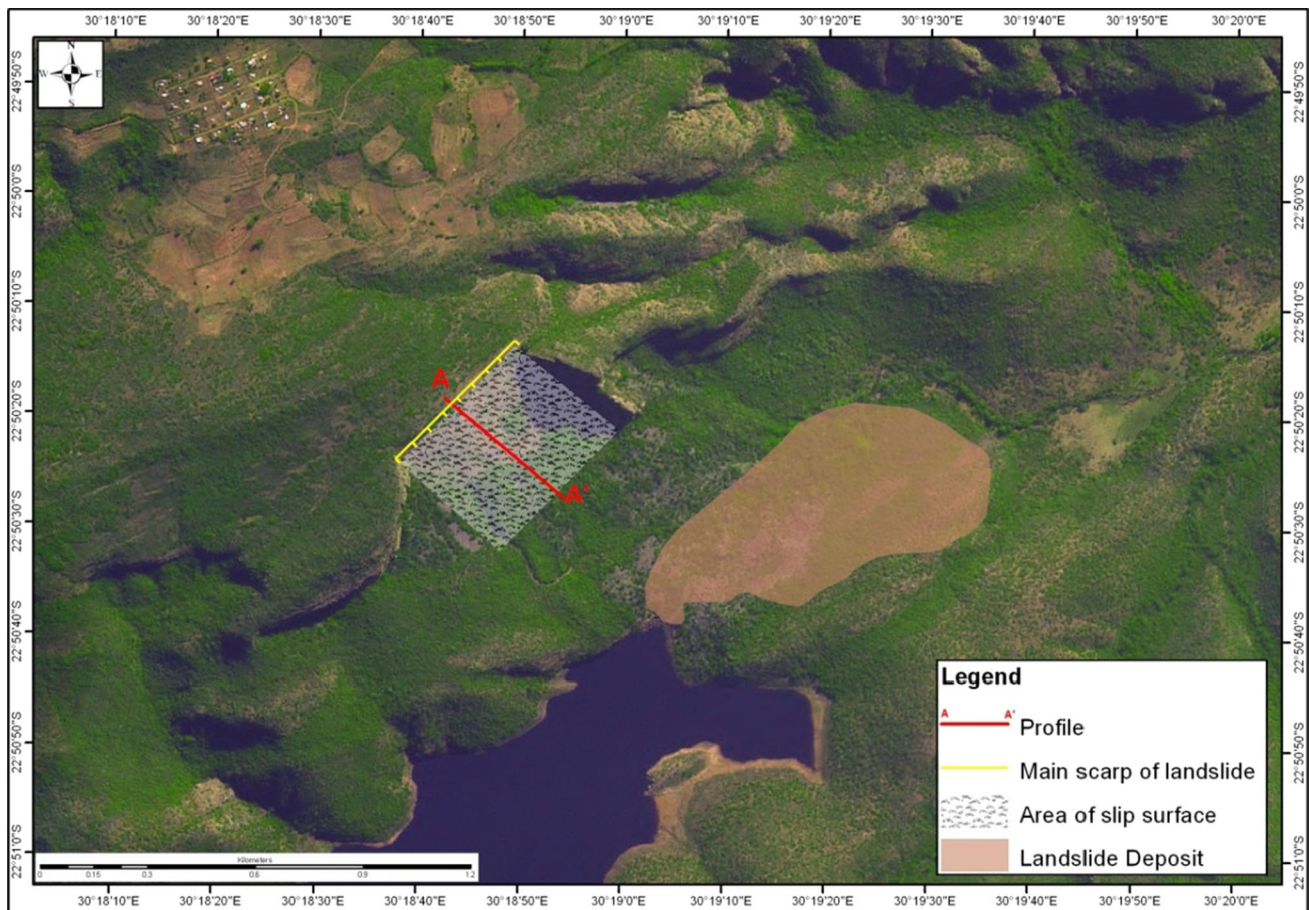


Fig. 8 Line of cross section (A–A') for the analyses

affects the effective stress acting on the failure plane (Duncan and Stark 1992). No water body or groundwater seepage was observed at the exposed sliding plane and across the landslide area which was hanging 350 m above the lake except for the Lake Fundudzi's vast water body at the bottom of the valley. The modelling of groundwater influence in the analysis considered the presence of water in the tension crack. The South Africa Weather Services (2009) records revealed that the region experiences moderate annual rainfall (1280 mm). Palaeoclimate interpretation by Kristen et al. (2007) revealed that the rainfall record peaks before the corresponding insolation maximum at ~ 47 Ka BP is reached, and it is relatively high at ~ 30 Ka BP, when austral summer insolation is at a minimum. Furthermore, it exhibits a broad minimum from the Last Glacial Maximum until the middle Holocene (~ 27 –5 Ka BP). It can thus be deduced that the period of 20,000 years ago was characterised by wet conditions. This also coincides with the last glacial maximum which was around 18,000 years ago. Due to the occurrence of a tension crack on the main scarp area of the rock slide, different groundwater scenarios in the pre-failure tension crack

which may have been open and filled with water were considered in the analyses. Therefore, three different scenarios of groundwater conditions in the tension crack were considered in the analyses. These were:

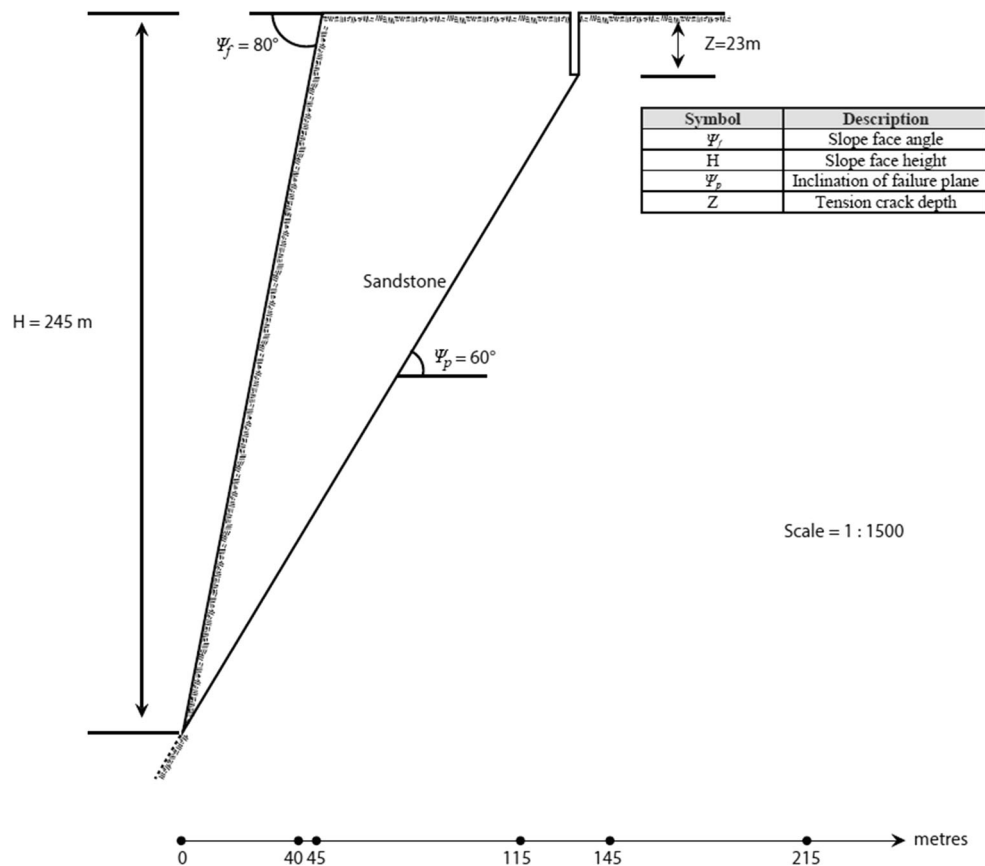
- 0% for completely dry tension crack
- 50% for partially filled tension crack
- 100% for full tension crack

These three conditions were assumed constant for each set of analyses conducted.

Seismic ground acceleration condition for analyses

Seismic ground acceleration was considered in the analyses in order to investigate whether seismicity could have been a triggering factor for the rockslide to have occurred. Data from the Council for Geoscience shows two seismic events of magnitude 5 on the Richter scale to have occurred in 1940 and 1970. The seismic hazard map of Southern Africa suggests that the area is located in a region with a peak horizontal ground acceleration of between 0.05 and 1.0 m/s^2 (Fernández and du Plessis 1992). Therefore, a

Fig. 9 Reconstructed slope of the Lake Fundudzi rockslide cross section A–A' shown in Fig. 7



pseudostatic approach which represents the most unfavourable condition for stability was used in this study in order to investigate whether seismic conditions could have triggered the rockslide. The Hynes-Griffin and Franklin (1984) method which recommended a seismic coefficient equal to $0.5 \times$ peak ground acceleration (PGA) for pseudostatic analysis, was adopted for the pseudostatic analysis in RocPlane. Thus, the seismic coefficient values shown in Table 2 were used in the analyses and included as seismic force in the analyses.

Sensitivity analysis

Prior to the back-analyses, sensitivity analysis was carried out to see the effect of the main input parameters (i.e. JRC, ϕ_r and JCS) on the factor of safety in RocPlane. Sharifzadeh et al. (2009) stated that in order to avoid any adverse effects of back-calculated parameters on further stability analysis, it is necessary to assess the influence of parameter variation on slope behaviour using a sensitivity analysis procedure. Figure 10 shows the data for the geometrical features that characterise the slope which were input into RocPlane. The pre-failure slope angle was 80° , the slope height is 245 m and the failure plane day-lights on the slope face with a dip of 60° . Results from the

sensitivity analysis, as shown in Fig. 11, show that the JRC has the most influence on the value of F and the impact of the JCS on F is almost negligible. It was thus logical to assume a constant value of JCS in the analysis and only the JRC and ϕ_r values were considered as variables in the back-analysis.

Procedures in the back-analysis

Back-analysis requires a known factor of safety, F , which at failure, is equal to unity (Sauer and Fredlund 1988). This eliminates one of the unknowns in the analysis. A “trial and error” procedure was used whereby the trial values of an unknown shear strength parameter is matched (i.e. this is done by varying one variable at a time, while keeping all other variables constant), until the best fit of shear strength parameters is obtained for a value of $F = 1.00$. Several scenarios were considered in the analysis to investigate the geotechnical conditions under which failure may have occurred, the probable cause or causes of failure, as well as a possible triggering mechanism. In the first instance, the average JRC and JCS values were set constant at 10 and 189 MPa, respectively, under dry conditions and with no consideration of seismic acceleration. The value of ϕ_r that would give a value of

Table 2 Seismic coefficient (α) values calculated from the available peak ground acceleration (PGA) values

	Published PGA values (adapted from Fernández and du Plessis 1992; m/s^2)	Calculated seismic coefficient (α) based on Hynes-Griffin and Franklin (1984): $0.5 \times \text{PGA}$
Minimum	0.05	0.025
Mean	0.075	0.0375
Maximum	0.10	0.05

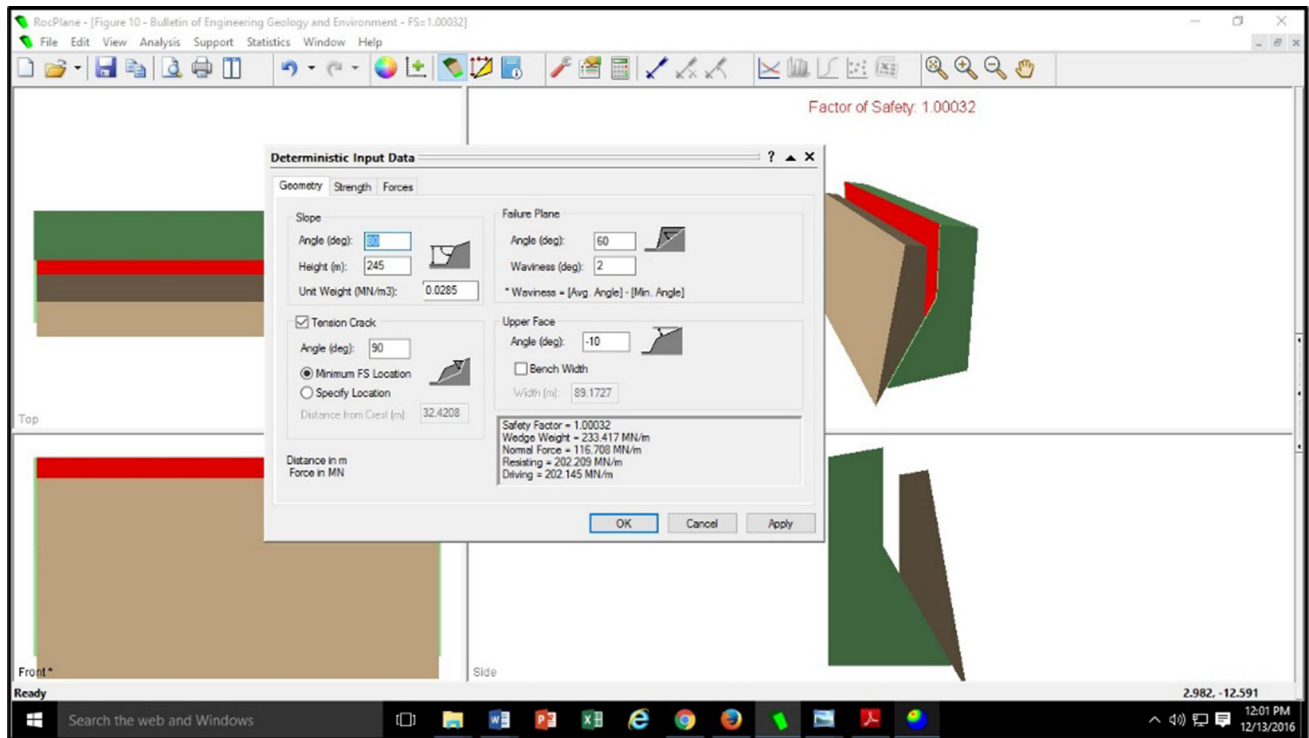


Fig. 10 Geometric slope parameters captured on RocPlane

$F = 1.00$ was then found. The analysis then investigated groundwater as a possible triggering mechanism. For each groundwater condition mentioned in the section, “Groundwater conditions for analyses” (i.e. completely dry, partially filled and completely filled tension crack), ϕ_r was back-analysed for a possible range of values of JRC. The analysis was then repeated for a possible range of typical values of ϕ_r for sandstone ranging from 35° to 40° to back-analyse the JRC values that would result in a value of $F = 1.00$. The analysis then considered seismic activity as a possible triggering mechanism for the rockslide. Back-analysis was undertaken for the three seismic coefficient scenarios mentioned in Table 2 in the first place, for ϕ_r for a possible range of values of JRC, and in the second case, for JRC for typical ϕ_r values of sandstone ranging from 35° to 40° . In the final case, based on the average values of JRC and ϕ_r , a back-analysis was undertaken to determine the seismic coefficient required to give a value of $F = 1.00$.

Results and discussion

The planar failure surface is inclined at 60° which is steeper than the back-analysed residual friction angle, which was 36.8° under dry static conditions. Also, the critical slope height for a slope containing a planar discontinuity dipping at an angle of 60° is ~ 21 m which makes the slope height at the project site about 12 times higher than the allowable critical slope height (Hoek and Bray 1981).

The back-analysed value of ϕ_r obtained under dry conditions with no consideration of seismic conditions was found to be 25.4° based on the average JRC value of 10. Table 3 shows the back-analysed ϕ_r values obtained for the assumed range of JRC values under different groundwater level conditions in the tension crack and Table 4 shows the back-analysed JRC values obtained for the typical range of values for ϕ_r under the different groundwater level conditions considered. Figures 12 and

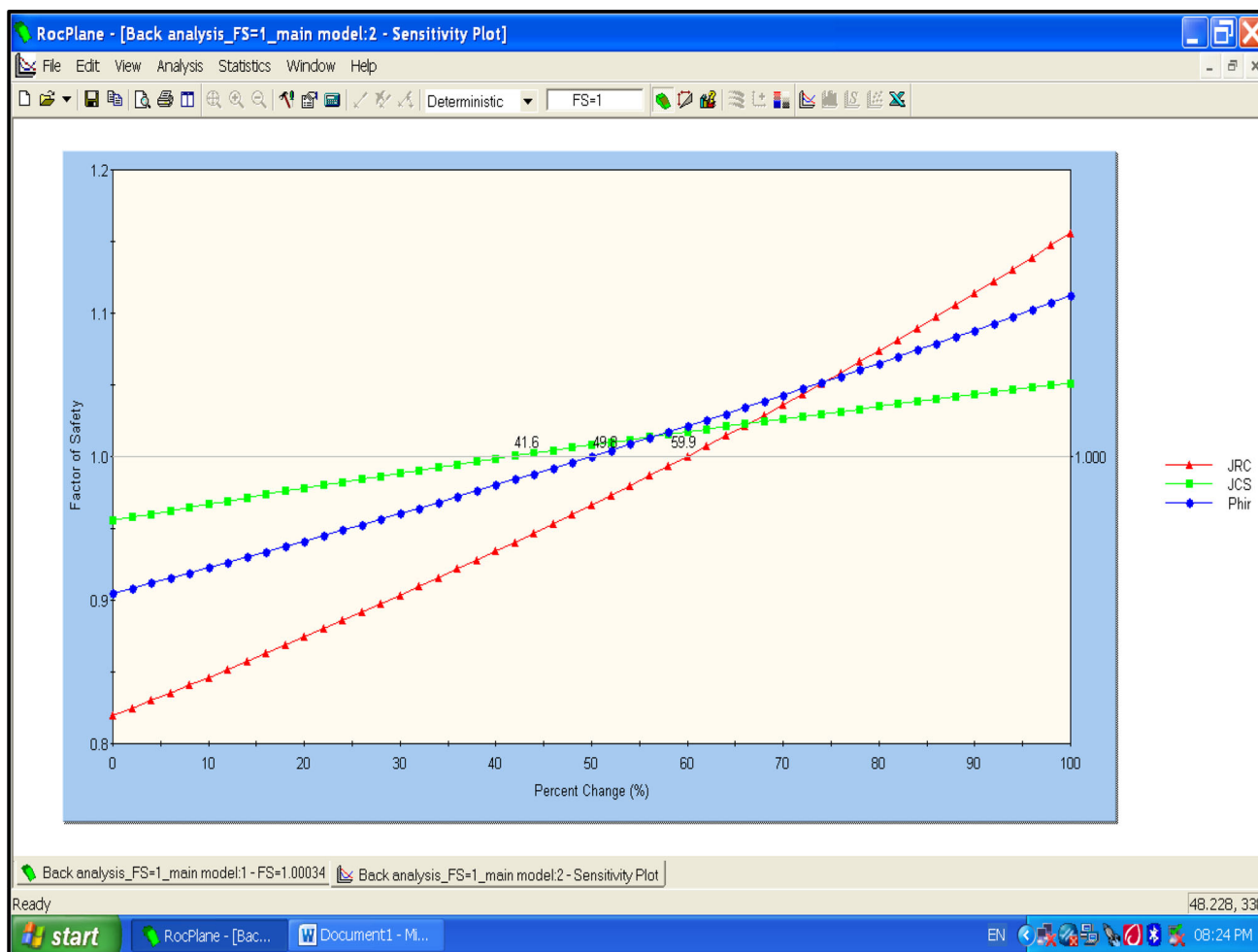


Fig. 11 Sensitivity analysis of back-calculated shear strength parameters

13 also show the plots for these two cases. From the plot, it can be seen that there is an inverse linear relationship between the assumed values and the back-calculated values. For instance, as the assumed value of JRC increases, the back-calculated value of ϕ_r decreases. Thus, as roughness increases, the friction angle required for failure decreases and vice versa.

For each of the assumed JRC values, the groundwater does not seem to have much impact on ϕ_r . For instance, at an assumed JRC value of 8.5 the value obtained for ϕ_r is 30.7°, 32.7° and 34.9° under completely dry, 50%-filled and 100%-filled tension crack groundwater conditions, respectively. This shows a difference of $\sim 2.0^\circ$ in the value obtained for ϕ_r from a completely dry tension crack to a 50%-filled tension crack and 50%-filled tension crack to 100%-filled tension crack, respectively. Within the range of 8.5–11 JRC values considered, the value obtained for ϕ_r for $F = 1.00$ ranges from 30.7° to 22.1, 32.7° to 23.9° and 35.0° to 26.0° under completely dry, 50%-filled and 100%-filled tension crack conditions, respectively. This gives a

difference of 8.1°, 8.8° and 9.0°, respectively, in the values obtained for ϕ_r .

It is evident from the results shown in Fig. 13 that for each of the assumed typical ϕ_r values for sandstone, there is minimal change (i.e. <1.0) in the back-analysed JRC values across the different groundwater scenarios considered in the analysis. For example, for an assumed value of 35° for ϕ_r , the required JRC value changes from 7.3 to 7.8 and from 7.8 to 8.5 under dry, 50%-filled and 100%-filled water level scenarios in the tension crack, respectively. The analysis shows that for the assumed values of ϕ_r ranging from 35° to 40°, the difference in the back-analysed JRC values are 1.5, 1.4 and 1.4 under dry, 50%-filled and 100%-filled water level scenarios in the tension crack, respectively. For an assumed ϕ_r value, the difference in JRC values obtained considering the three different groundwater conditions is very minimal as evidenced by the plots shown in Fig. 13.

Considering the range of values, it can be deduced that the groundwater conditions have very little impact on the

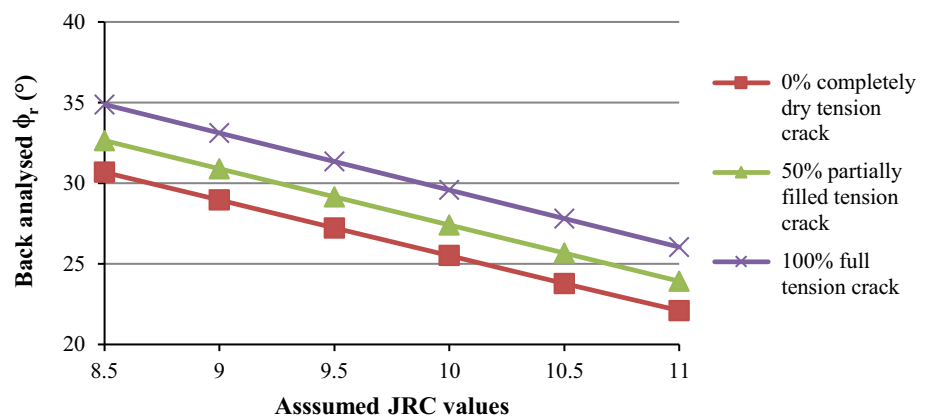
Table 3 Back-analysed ϕ_r values under different groundwater levels in the tension crack

Assumed JRC value	Back-calculated ϕ_r (°)		
	0% completely dry tension crack	50% partially filled tension crack	100% full tension crack
8.5	30.7	32.7	34.9
9	29.0	30.9	33.1
9.5	27.2	29.2	31.4
10	25.5	27.4	29.6
10.5	23.8	25.7	27.8
11	22.1	23.9	26.0

Table 4 Back-analysed JRC given ϕ_r under different groundwater levels in the tension crack

Assumed typical ϕ_r values for sandstone (°)	Back-calculated JRC		
	0% completely dry tension crack	50% partially filled tension crack	100% full tension crack
35	7.3	7.8	8.5
35.5	7.1	7.7	8.3
36	7.0	7.6	8.2
36.5	6.8	7.4	8.0
37	6.7	7.3	7.9
37.5	6.5	7.1	7.8
38	6.4	7.0	7.6
38.5	6.2	6.8	7.5
39	6.1	6.7	7.4
39.5	6.0	6.6	7.2
40	5.8	6.4	7.1

Fig. 12 Graph of assumed JRC values versus back-analysed ϕ_r values obtained from different groundwater conditions



back-analysed values for ϕ_r and JRC. This minimal impact could be attributed to the very high angle of the slope. In the case of ϕ_r , not much friction is kinematically required, whereas in the case of JRC, failure is more controlled by an increase in normal stress due to the very high angle of the slope than an increase in pore water pressure. Thus, even though the discontinuity is partially or fully saturated, it is

insufficient to have a significant impact on stability. The JRC, then, does not need to be significantly greater to cope with the change in saturation.

The results obtained from the pseudostatic analyses are shown in Tables 5 and 6 for the back-analysed ϕ_r and JRC values, respectively. Figures 14 and 15 also show the plots for the two cases, which show an inverse relationship

Fig. 13 Back-analysed JRC values for an assumed range of ϕ_r values under different groundwater levels in the tension crack

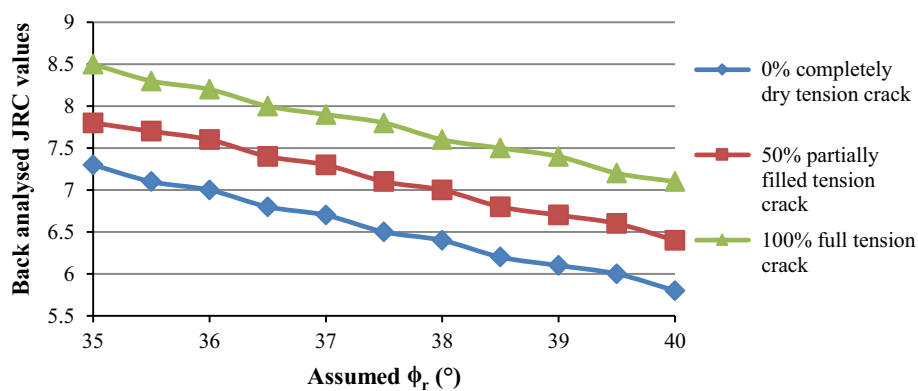


Table 5 Back-analysed ϕ_r values under different seismic coefficient scenarios

Assumed JRC values	Back-calculated ϕ_r (°)		
	Minimum seismic coefficient conditions = 0.025	Mean seismic coefficient conditions = 0.0375	Maximum seismic coefficient conditions = 0.05
8.5	33.2	34.5	35.7
9.0	31.5	32.7	33.9
9.5	29.7	31.0	32.2
10.0	28.0	29.2	30.4
10.5	26.2	27.4	28.6
11.0	24.5	25.7	26.9

between JRC and ϕ_r . For an assumed JRC value, the difference in the back-analysed value of ϕ_r obtained from the minimum seismic coefficient value to the maximum seismic coefficient value was found to be 2.5° . For the range of JRC values considered (i.e. 8.5–11), the difference in the range of ϕ_r value obtained is approximately 10° . It is also seen from Table 6 that for the range of ϕ_r values considered (i.e. 35° – 40°), the difference in JRC values obtained is ~ 1.4 in all three scenarios.

Based on the average JRC and ϕ_r values, a back-analysis was run to determine the value of the seismic coefficient that would give a value of $F = 1.00$. This was found to be 0.02 and is lower than the minimum published values for seismic coefficients for the area. If seismic activity could have been the triggering mechanism, then it implies that the slope was in such a marginal state of instability for failure to have been triggered by undetected seismic activity. Factors such as the geometry of the slope, unfavourable joint sets and poor RMR could have rendered the slope marginally stable.

The results from the back-analysis present a range of possible situations which could have resulted in a value of $F = 1.00$ and not a definite solution as to what could have led to failure. Although back-analysis usually yields a better estimate than laboratory test results, there are many uncertainties associated with the process. Some of these

uncertainties have been discussed by Leroueil and Tavenas (1981) as well as Duncan and Stark (1992).

Hussain et al. (2010) mentioned that some of the uncertainties that influence the back-calculated parameters are engineering properties of the other material in the cross-section, slope geometry at the time of failure, phreatic surface and pore water pressures present at the time of failure, location of failure surface and existence of tension cracks. These uncertainties can be reduced by investigating some of the unknowns such as undertaking a detailed field investigation in order to define the failure surface and slope geometry. Furthermore, the results obtained from back-analysis should be properly evaluated in close collaboration with field evidence. As pointed out earlier, the landslide occurred about 20,000 years ago and there was no factual record of events that prevailed or occurred during failure. However, the failure plane and the failure geometry are well-known; thus, it was possible to reconstruct the pre-landslide geometry which brought about a reduction in the uncertainty in the analyses. The use of selected seismic coefficient values determined on the basis of Hynes-Griffin and Franklin (1984) and the PGA record for the region further brought a reduction in the uncertainty. The groundwater conditions thus presented one of the major uncertainties in the analysis.

Table 6 Back-analysed JRC values given ϕ_r under different seismic coefficient scenarios

Assumed typical ϕ_r values for sandstone	Back-calculated JRC		
	Minimum seismic coefficient (α) conditions = 0.025	Mean seismic coefficient conditions $\alpha = 0.0375$	Maximum seismic coefficient conditions $\alpha = 0.05$
35.0	8.0	8.4	8.7
35.5	7.9	8.2	8.6
36.0	7.7	8.1	8.4
36.5	7.6	7.9	8.2
37.0	7.4	7.8	8.1
37.5	7.3	7.6	8.0
38.0	7.1	7.5	7.9
38.5	7.0	7.4	7.7
39.0	6.9	7.2	7.5
39.5	6.7	7.1	7.4
40.0	6.6	6.9	7.3

Fig. 14 Graph of assumed JRC values versus back-analysed ϕ_r obtained from different seismic ground acceleration scenarios

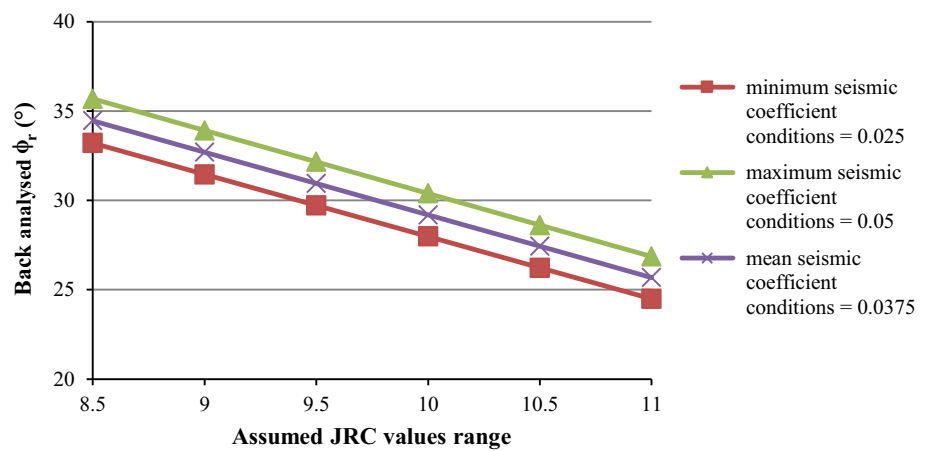
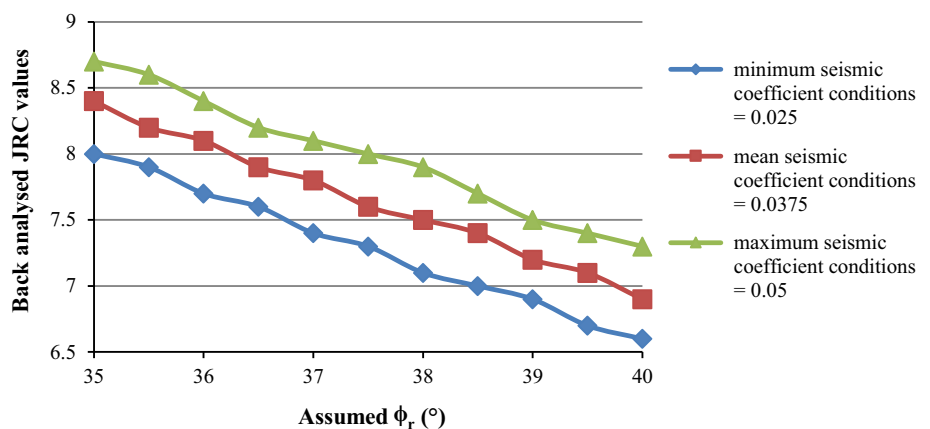


Fig. 15 Back-analysed JRC values versus assumed ϕ_r under different seismic ground acceleration coefficients



Whilst the analysis in this study shows groundwater to be of more impact than seismic activity for an assumed JRC value, the evidence on site would hardly suggest a scenario for a 100%-filled tension crack. Thus, there is uncertainty as to whether groundwater conditions or seismic activity could

have triggered the landslide. Cruden and Varnes (1996) pointed out that landslides can have several causes such as geological, morphological and anthropic, but only one trigger. However, as pointed out by Wiczorek (1996), in some cases, landslides may occur without an apparent trigger as a

result of a variety or combination of causes, such as chemical or physical weathering of materials, which gradually bring the slope to failure. Duncan and Wright (2005) further pointed out that when a slope fails, it is often not possible to determine a single cause that acted alone. In most cases, several causes exist simultaneously as has been demonstrated in this study. Therefore, attempting to decide which one finally triggered failure is not only difficult but also technically incorrect particularly so in this case for a landslide that occurred 20,000 years ago. The final factor is a trigger that sets a body of earth in motion that was already on the verge of failure. From the analyses conducted, the presence of either seismic ground acceleration or water pressure from water levels in the tension crack could probably have triggered the landslide. According to Hoek and Bray (1981), the critical height for a slope containing a planar discontinuity dipping at an angle of 60° should be ~ 21 m. The failed rock slope, which has a height of 245 m, is about 12 times higher than the required threshold height for it to have remained stable. Evidence from the rock debris across the valley floor shows a highly disrupted material which may perhaps point to a massive rock avalanche which has been caused by a seismic event. Keefer (1984) pointed out that seismic ground acceleration is probably required for very highly disrupted rock avalanches. Selby (1982) earlier pointed out that rockslides may be very large and catastrophic in mountainous regions where large available relief permits acceleration of rock debris to great velocity. In the case of the Lake Fundudzi landslide, the farthest blocks, which are the largest, travelled a distance of up to 700 m across the valley. Based on the field evidence, it can be deduced that a seismic event of very low magnitude could more likely have triggered the rockslide which was already in a marginal state of stability, whereby massive failure could have broken down the competent sandstone rock mass into debris of large rock blocks deposited across the valley floor.

Conclusions

Lake Fundudzi, the only true inland lake in South Africa, was formed due to a massive ancient rockslide that blocked the course of the eastward flowing Mutale River. The calculated volume of rock that slid down the failure surface was approximated to be 8 million m^3 . Analysis using the Rocscience Inc. Dips software based on Markland's test revealed three major joint sets, JS1, S0 and JS3, with an average dip direction of 237° , 283° and 157° , respectively. The rockslide was detached from a near vertical, south-facing, 23-m-deep scarp with an orientation that best coincides with JS3. The results further revealed that JS3 poles represent planes that were susceptible to plane failure. From the analysis, plane failure mode was determined

based on the orientation of the slope, the orientation of the discontinuities, and the internal angle of friction of the rock mass. This was found to be consistent with field observations. Failure could have been mostly structurally controlled, particularly by the unfavourable orientation of S0 such as the very steep angle of the failure plane (i.e. 60°) which daylighted to a very steep slope face (i.e. 80°).

Both the RMR and SMR classify the rock mass as poor after adjustments. This points out that the presence of unfavourable major discontinuities had an adverse effect on the stability of the slope and the rock mass rendering the slope to be in a marginal state of stability.

Sensitivity analysis shows that the JRC has the greatest impact on F followed by ϕ_r whilst the impact of the JCS on F was considered negligible.

Results from the back-analysis show a range of possible combinations of JRC and ϕ_r values under different groundwater and seismic conditions. The results, however, present an uncertainty as to whether groundwater or seismic activity could have triggered the landslide. Whilst the groundwater shows more impact than the seismic coefficient, the field evidence points to seismic activity as a more plausible trigger than groundwater.

Back-analysis is a useful procedure in investigating the various conditions of slope failure, particularly in this case of an ancient rockslide where so much uncertainty exists. However, it cannot provide a unique combination of geotechnical parameters but a range of possible parameters and possible scenarios that may have led to failure which may be useful in preventing future failure occurrence. The results, thus, have to be treated with good judgement and also evidence from field investigation as was done in this study.

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References

- Alonso EE (1976) Risk analysis of slopes and its application to slopes in Canadian sensitive clays. *Géotechnique* 26:453–472
- Barker OB (1979) A contribution to the geology of the Soutpansberg Group, Waterberg Group, Northern Transvaal. MSc thesis. University of the Witwatersrand, Johannesburg (unpublished)
- Barker OB, Brandl G, Callaghan CC, Eriksson PG, van der Neut M (2006) The Soutpansberg and Waterberg Groups and the Blouberg Formation. In: Johnson MR, Anhaeusser CR, Thomas RJ (eds) *The geology of South Africa*. Geological Society of South Africa, Johannesburg/Council for Geosciences, Pretoria, pp 301–319
- Barton NR (1976) The shear strength of rock and rock joints. *Int J Mech Min Sci and Geomech Abstr* 13(10):1–24
- Barton NR, Bandis SC (1990) Review of predictive capabilities of JRC-JCS model in engineering practice. In: Barton N,

- Stephansson O (eds) Rock joints. Proceeding international symposium on rock joints, Loen, Norway. Balkema, Rotterdam, pp 603–610
- Barton NR, Choubey V (1977) The shear strength of rock joints in theory and practice. *Rock Mech* 10(1–2):1–54
- Bell FG (2007) *Engineering Geology*, 2nd edn. Butterworth-Heinemann (Elsevier), Burlington, p 581
- Bieniawski ZT (1989) *Engineering rock mass classifications*. Wiley, New York
- Bieniawski ZT, Orr, CM (1976) Rapid site appraisal for dam foundations by the geomechanics classification. In: *Proc. 12th International Congress on Large Dams, Mexico*, pp 483–501
- Brady BHG, Brown ET (2006) *Rock mechanics for underground mining*, 3rd edn. Springer, The Netherlands
- Broch E, Franklin JA (1972) The Point load strength test. *Int J Rock Mech Mining Sci* 9(6):669–697
- Bye AR, Bell FG (2001) Stability assessment and slope design at Sandsloot open pit, South Africa. *Int J Rock Mech Min Sci* 38:449–466
- Chiliza SG, Richardson S (2008) Landslide incidence in the Limpopo Province, South Africa. In: *Proceedings of the first world landslide forum*. United Nations University, Tokyo, pp 100–103
- Cornforth D (2005) *Landslides in practice: investigation, analysis and remedial/preventive options in soils*. Wiley, New York, p 596
- Cruden DM, Varnes DJ (1996) Landslide types and processes. In: Turner AK, Schuster RL (eds) *Landslides investigation and mitigation*. Special report 247, Transportation Research Board, National Research Council, Washington, pp 36–75
- Dai FC, Lee CF, Ngai YY (2002) Landslide risk assessment and management: an overview. *Eng Geol* 64(1):65–87
- Donnelly CR, Rigbey SJ (2007) The assessment of sliding resistance beneath concrete structures. Conference Paper. Canadian Dam Association Conference, St. John's, Canada. doi:10.13140/2.1.2163.8728
- Duncan, JM Stark TD (1992) Soil strengths from back-analysis of slope failures. In: *Proc. stability and performance of slopes and embankments II*, ASCE, GSP 31, Berkeley, pp 890–904
- Duncan JM, Wright SG (2005) *Soil Strength and Slope Stability*. Wiley, Canada
- El-Ramly H, Morgenstern NR, Cruden DM (2002) Probabilistic slope stability analysis for practice. *Can Geotech J* 39:665–683
- Fernández LM, du Plessis A (1992) Seismic hazard maps for Southern Africa. Council for Geoscience, Geological Survey of South Africa, Pretoria
- Fredlund DG, Krahn J, Pufahl DE (1981) The relationship between limit equilibrium slope stability methods. In: *Proceedings, 10th international conference on soil mechanics and foundation engineering*, Stockholm, Sweden, vol 3. Balkema, Rotterdam, pp 409–416
- Hencher SR, Richards LR (1989) Laboratory direct shear testing of rock discontinuities. *Ground Eng* 22(2):24–31
- Hoek E (2000) *Rock engineering*. Course notes by Evert Hoek. Hoek's Corner. <http://www.rocsience.com>. Accessed 25 Aug 2014
- Hoek E (2007) *Practical Rock Engineering*. Rocscience, Toronto (**e-book**)
- Hoek E, Bray JW (1981) *Rock slope engineering*, 3rd edn. Institute of Mineralogy and Metallurgy, London, p 358
- Hussain M, Start TD, Akhtar K (2010) Back-Analysis procedure for landslides. In: *Proceedings of the international conference on geotechnical engineering*. Pakistan Geotechnical Engineering Society, Lahore, Pakistan, pp 159–166
- Hynes-Griffin ME, Franklin AG (1984) Rationalizing the seismic coefficient method. US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Misc. Paper GL-84-13
- ISRM (1979) Suggested methods for determining in situ deformability of rock. *Int J Rock Mech Min Sci Geomech Abstr* 16(3):195–214
- ISRM (1981) Rock characterisation, testing and monitoring: ISRM suggested methods. In: Brown ET (ed) *Published for the commission on testing methods, International society for rock mechanics*. Pergamon press, Oxford, New York, p 212
- Janisch EP (1931) Notes on the central part of the Soutpansberg and on the origin of Lake Fundudzi. *Trans Geol Surv South Africa* 54:152–162
- Keefer DK (1984) Landslides caused by earthquakes. *Geol Soc Am Bull* 95:406–421
- Khorombi M (2000) Towards a sustainable land-use plan for the Lake Fundudzi catchment area. M. Inst. Agr. thesis. University of Pretoria, South Africa (Unpublished)
- Kristen I, Fuhrmann A, Thorpe J, Röhl U, Wilkes H, Oberhänsli H (2007) Hydrological changes in southern Africa over the last 200 Ka as recorded in lake sediments from the Tswaing impact crater. *S Afr J Geol* 110:311–326
- Leroueil S, Tavenas F (1981) Pitfalls of back-analysis. In: *X International conference of soil mechanics and foundation engineering*, Stockholm, vol 1, pp 185–190
- Markland JT (1972) A Useful technique for estimating the stability of rock slope when the rigid wedge slide type of failure is expected. Imperial College Rock Mechanics Research Reprint, No. 19
- Mathis JI (1987) Discontinuity mapping—a comparison between line and area mapping. In: *Proceedings 6th international congress, International Society for Rock Mechanics*, Balkema. vol 2, pp 1111–1114
- Miller SM, McWilliams PC, Kerkering JC (1989) Evaluation of stereo digitizing for measuring rock fracture. In: *Rock mechanics as a guide for efficient utilization of natural resources*. Proceedings of 30th US symposium on rock mechanics, West Virginia. Balkema, Rotterdam, pp 201–208
- Oppikofer T (2009) Detection, analysis and monitoring of slope movements by high-resolution digital elevation models. Faculty of Geosciences and Environment Université de Lausanne (UNIL), Lausanne, p 183
- Paige-Green P (1989) Landslides: extent and economic significance in southern Africa. In: Brabb EE, Harrod BL (eds) *Landslides: extent and economic significance*. Balkema, Rotterdam, pp 261–269
- Romana M (1993) A geomechanical classification for slopes: slope mass rating. In: Hudson JA (ed) *Comprehensive rock engineering*. Pergamon Press, Oxford, pp 575–599
- Sauer EK, Fredlund DG (1988) Effective Stress, limit equilibrium back-analysis of failed slopes: Guidelines, landslides-glissements de terrain. In: Christophe Bonnard (ed). *Proceedings of the 5th international symposium on landslides*, Balkema, Rotterdam
- Selby MJ (1982) *Hillslope materials and processes*. Oxford University Press, Oxford
- Sharifzadeh M, Sharifi M, Delbari SM (2009) Back analysis of an excavated slope failure in highly fractured rock mass: the case study of Kargar slope failure (Iran). *Environ Earth Sci* 60:183–192
- South Africa Weather Services (2009). <http://www.weathersa.co.za>. Accessed 10 July 2014
- Stead D, Eberhardt E, Coggan JS (2006) Developments in the characterization of complex rock slope deformation and failure using numerical modelling techniques. *Eng Geol* 83:217–235
- Van der Waal BCW (1997) Fundudzi, a unique, sacred and unknown South African lake. *S Afr J Aquat Sci* 23(1):42–55
- Van Eeden OR, Visser HN, Van Zyl JS, Coertze FJ, Wessels JT (1955) The geology of the eastern Soutpansberg and the Lovveld to the north. Explain. Sheet 42 (Soutpansberg), Geol. Surv. S. Afr., 117 pages
- Wieczorek GF (1996) Landslide triggering mechanisms. In: Turner AK, Schuster RL (eds) *Landslides investigation and mitigation*. Special report 247, Transportation Research Board, National Research Council, Washington, pp 76–90