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## An experimental and numerical study of a landslide triggered by an extreme rainfall event in northern Australia

**Abstract** In February/March 2007, an extreme rainfall event occurred in the Jabiru region of the Northern Territory of Australia. Rainfall of 784 mm fell in a 72-h period. This rainfall event resulted in 49 separate landslides occurring in the adjacent, but remote and inaccessible region of Arnhem Land. The landslides were extensively mapped and characterised. A common feature of the landslides was their relatively surficial nature. This paper reports on laboratory and field tests to characterise the material properties of the slide material and the underlying, more competent material. One particular, large and relatively accessible landslide was chosen for detailed investigation. The experimental data are used to carry out seepage and slope stability analyses, taking account of changes in the degree of saturation (and thus the negative pore water pressure or suction) in the slope material during the rainfall event in question. Using a parametric study in which various material parameters were varied around the measured mean values, it is shown that the failure of this particular slope could have been predicted using relatively straightforward seepage and limit equilibrium slope stability analyses, coupled with the relevant rainfall data, as long as the contribution of matric suction to the engineering characteristics of the slope material was accounted for. The work also highlights the importance of in situ conditions at the time a particular rainfall event (particularly an extreme event such as that considered in this paper) occurs. If the slope has a relatively high degree of saturation, manifested as a low initial in situ suction, it is more susceptible to rainfall triggering a slope failure. Although this observation is not novel, the investigation described in this paper confirms the importance of ambient in situ conditions and provides an indication of how the likelihood of landslide occurrence at this particular site may in the future be quantified, i.e. by focussing on antecedent rainfall history.

**Keywords** Rainfall-induced landslides · Shallow failures · Suction · Antecedent precipitation · Seepage · Slope stability

### Introduction

Rainfall for the 15 years prior to 2010 in northern Australia (known as the ‘Top End’) was consistently above average, which is in turn consistent with the observation that annual rainfall in this region alternates between sub-decadal to multi-decadal sequences of below average and above average rainfall (Erskine et al. 2011). In the Alligator Rivers Region (ARR) (Fig. 1) of the ‘Top End’, which is a 28,000-km<sup>2</sup> area that includes the drainage basins of the East, South and West Alligator Rivers, as well as the Wildman River, these rainfall trends are particularly well developed (i.e. have been observed over an extended period of time; in excess of 50 years (Carter 1990; Erskine et al. 2011)). The most intense storm on record in the ARR occurred in March 2007 (Saynor et al. 2012; Erskine and Saynor 2012). This

rainfall initiated a series of landslides, which were confined to slopes where olivine dolerite was exposed; surrounding slopes of exposed sandstone did not experience any slope failures. These landslides occurred approximately 20 km east of Jabiru Airport (Fig. 1) and, for this paper, will be referred to as the Jabiru slopes.

Landslides are often triggered by extreme rainfall (Berti and Simoni 2005; Casagli et al. 2006; Liao et al. 2010; Capparelli and Tiranti 2010) and the response time of slope materials is largely dependent upon the hydraulic conductivity and thickness of surface soils. Granular soils tend to respond quickly to intense rainfall of short duration but clay-like materials are normally more sensitive to prolonged rainfall with moderate intensity (Casagli et al. 2006). Shallow landslides are often triggered during rainstorms when pore water pressure builds at the contact between the surface soils and the underlying bedrock (Salciarini et al. 2006). The increase of pore water pressure decreases effective stress and thus reduces soil shear strength, eventually resulting in slope failures (Terzaghi 1950; Reid 1994; Wang and Sassa 2003). It is not always necessary for positive pore water pressures to develop; the reduction of matric suction (which otherwise increases shear strength) to below a critical level may be sufficient to trigger a landslide. This mechanism of slope failure due to rainfall infiltration has become widely acceptable as a plausible explanation for such failures. It is discussed in detail by Rahardjo et al. (2000) and Collins and Znidarcic (2004), amongst others. Given the vast available literature on the topic, the failure mechanism will be discussed only very briefly. The mechanism is as follows: infiltration from rainfall penetrates the surface of the soil slope causing an increase in pore water pressure. This does not necessarily mean that a positive pore water pressure results (although in some cases, it might). Rather, it results in a reduction in negative (relative to atmospheric) pore water pressure (also known as suction); the final pore water pressure may still be negative when slope failures occur. However, it is less negative than when a rainfall event began. This increase in pore water pressure (even it is still negative) results in a decrease in effective stress and thus a reduction in available shear strength (see de Campos and Menezes (1991) and Rahardjo and Fredlund (1995) for more detailed discussion of this effect). The reduction in shear strength may be sufficient to trigger a slope failure. Given that the rate of change of pore water pressure is related to the amount of water that penetrates a soil slope, aspects such as rainfall duration and intensity are critically important and must be accounted for.

As explained by Collins and Znidarcic (2004), in order to evaluate the likelihood that a particular rainfall event will trigger a landslide, it is necessary to characterise the hydraulic and shear strength properties of the soil profile at the site in question, after

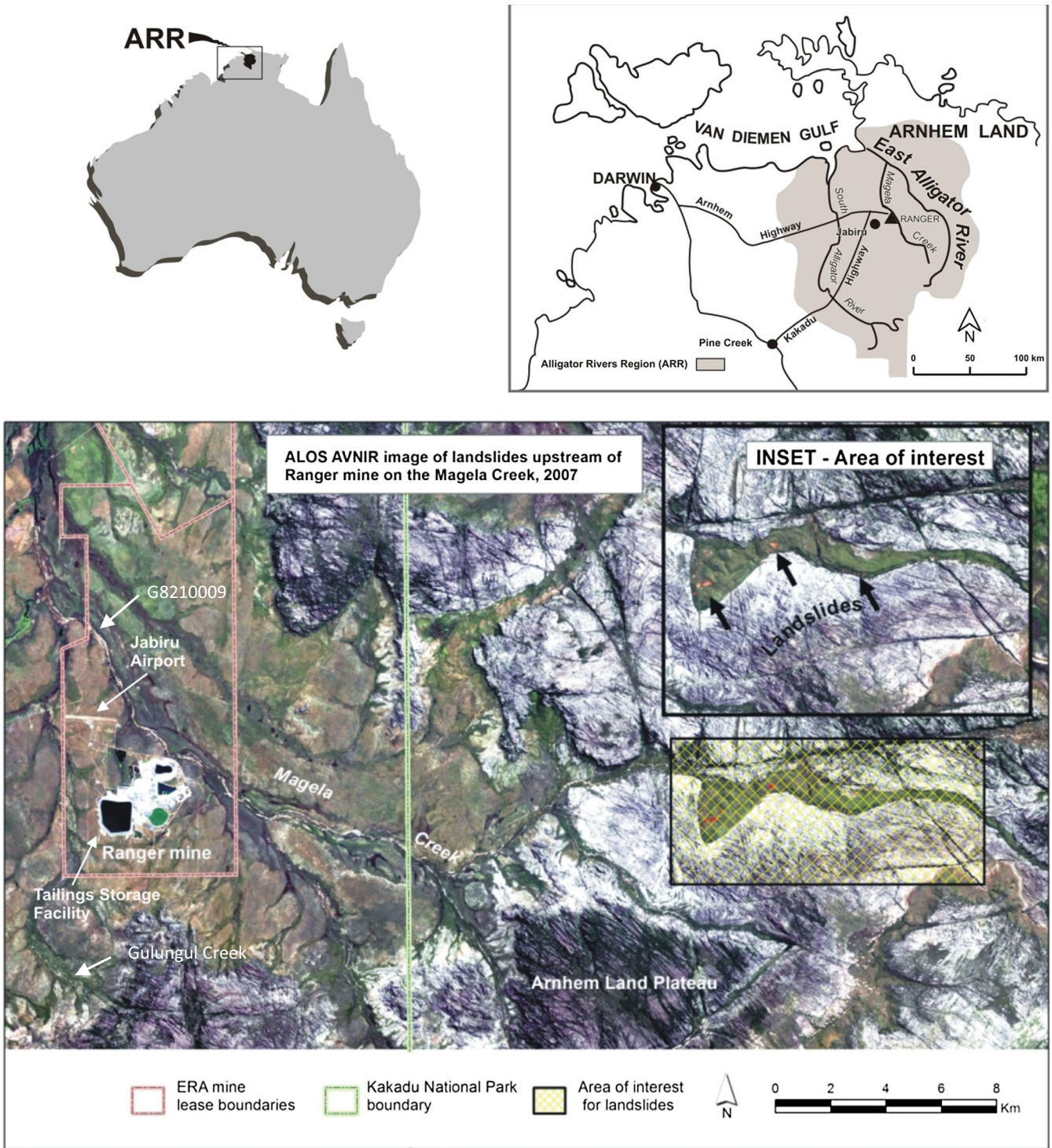


Fig. 1 Location of the area where the landslides of March 2007 occurred

which standard seepage and slope stability analyses, which are coupled, can be used to investigate the likelihood of a slope failure occurring, using representative rainfall data. This is the approach that was used in this paper.

This paper thus describes the rainfall event, the slope failures that followed, and the results of field and laboratory testing to determine appropriate parameters for use in an evaluation of the particular slide that was studied. The paper also describes the

results of numerical modelling using seepage and slope stability software and proposes a mechanism for the slope failure.

### Details of rainfall event

The closest rain gauges to where the landslides occurred are 20 km away at Jabiru Airport and G821009 at Magela Creek (see Fig. 1). Rainfall data from these rain gauges are very similar, as shown in Fig. 2. The Jabiru Airport gauge provided more complete rainfall data (particularly hourly recorded data) than the G821009 gauge, so rainfall data from the Jabiru Airport was considered to provide satisfactory representation for the analyses of a rainfall-induced landslide in this region. Although these data cannot perfectly represent rainfall at the particular site in question, this is unavoidable given the remoteness of the area (plus of course, with 49 different landslides having occurred, no single rainfall data set would be perfectly appropriate). Furthermore, Chiew and Wang (1999) showed good correlation between rainfall data from Jabiru and that from Oenpelli, which is 40 km NE of Jabiru. To consider the rainfall event of 2007 and whether it could have been predicted using currently available techniques, use of the Jabiru Airport data is considered suitable. However, it is acknowledged that the rainfall at the site in question would have probably not been exactly the same as that at Jabiru; ideally there would have been a rainfall station at the landslide in question, but the site is remote and uninhabited. We therefore believe using the Jabiru data is the best option available.

For the 72-h period between 5 pm on 27 February and 5 pm on 2 March, 784 mm of rainfall was recorded at the Bureau of Meteorology's Jabiru Airport gauge (Bureau of Meteorology et al. 2012). Hourly rainfall for this period for the Jabiru Airport station is shown in Fig. 3. During the 4 days prior to 27 February, there had been short periods of intense rainfall, with a total of 140.4 mm falling during this period. The probable maximum precipitation (PMP) for a 72-h duration event is 2200 mm (Bureau of Meteorology et al. 2012), and the return periods for the storm of 2007 for durations between 48 and 72 h exceeded 1:1000 years (Bureau of Meteorology et al. 2012). The highest 24-h total of 398 mm is the largest recorded in the region. The storm also resulted in the highest flood in Magela Creek and Gulungul Creek since gauging commenced in 1971 (Saynor et al. 2012). It is once again emphasised that the rainfall data shown in Figs. 2 and 3 do not necessarily represent the exact rainfall that occurred on the slope failure that was studied in this paper, it is the best data available and there is no way of reasonably estimating how the actual extreme event may have varied from that shown in Figs. 2 and 3.

### Landslide details

The existence of landslides triggered by this rainfall event only became apparent when a helicopter pilot reported seeing them in late March 2007. A field investigation carried out by Saynor et al. (2012) indicated that shallow slip surfaces had occurred on some soil slopes having inclinations of  $\pm 19^\circ$  and surficial soils dominated by cohesive soils having low permeability. A total of 49 landslides were identified in this region and 17 occurrences of all these landslides were delineated by combined field work, aerial reconnaissance and remote sensing, while the rest of the incidences were solely identified from ALOS AVNIR-2 images (Saynor et al. 2012). These landslides occurred in an area of about 2900 km<sup>2</sup> and were confined to surface outcrops of Oenpelli Dolerite, mainly olivine

dolerite, which is surrounded and unconformably overlain by sandstone. Saynor et al. (2012) estimate the eroded sediment mass from all landslides in the 2007 event was about 135,000 tonnes. They mapped 15 landslides in detail and found that all landslides were long and narrow with length/mean width ratios always less than 9.3. No landslides occurred on slopes less than  $17^\circ$  and the smallest headscarp was 1.0 m high. It should also be mentioned that helicopter pilots had never previously reported the occurrence of any landslides and there had not been any prior reports of pulses of red water in the Magela Creek. As mentioned later, the rainfall event of February/March 2007 produced significant 'red pulses' in this creek. There had thus been no sign of landslides prior to this rainfall event and the 49 landslides that did occur are considered to be attributable to the extreme rainfall event that occurred. This paper suggests that, at least for the particular slope failure studied in some detail, the particular mechanism causing the failure was rainfall-induced pore water pressure increase (or suction decrease). We cannot say with any certainty whether the other landslides were due to the same effect as there was no opportunity to study more than one.

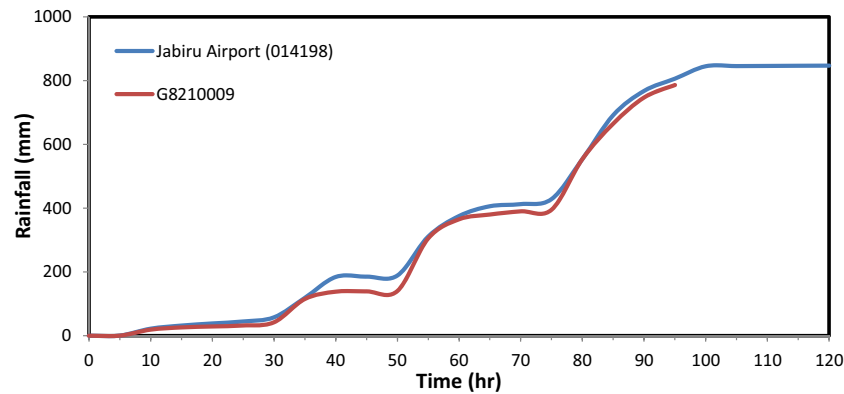
The area in which the landslides occurred is in a remote region of Arnhem Land and has restricted access because there are no roads or tracks and the only access was by helicopter. The field-work required all equipment (including water for the hydraulic conductivity testing) to be carried from the helicopter landing area, a distance of approximately 1 km. Of all the slope failures that occurred, the slope failure used as a case study here is one of the largest, with a volume of eroded sediments of 9,825 m<sup>3</sup> and a mean depth, width and length of 2, 80–90 and 70 m, respectively, as shown in Fig. 4. The geometry of the slope where the landslide occurred has an angle of  $19^\circ$  and overall height of 23 m as shown in Fig. 5. This particular landslide was chosen for more detailed study due to its large size and slightly easier access. It was the only large landslide where a moderately acceptable landing area for the helicopter was available. Field observations indicated that a surface soil layer with an average thickness of 2 m overlies an intact bedrock layer having very low permeability (designated 'impermeable' to reflect boundary conditions assumed later in modelling exercise). The failure surface was irregular but approximately parallel to the ground slope surface. It is shown in Fig. 5 as planar for simplicity.

### Field and laboratory investigation

Field and laboratory investigations were carried out to determine soil properties required in the analysis of slope instability. There are two main soil properties used in the analysis, hydraulic and shear strength properties, while other index properties such as the Atterberg Limits and specific gravity as well as bulk density were required for general assessment of the soil characteristics and classification. Soil samples were collected at the site and returned to the University of Western Australia laboratory for testing. Soil sampling and field testing were done at several positions at the site as described below, to get a good representation of soil characteristics.

### Methods of soil sampling and field tests

Both soil sampling and field tests were conducted at the intact slope (next to the landslide scar) and within the landslide scar.



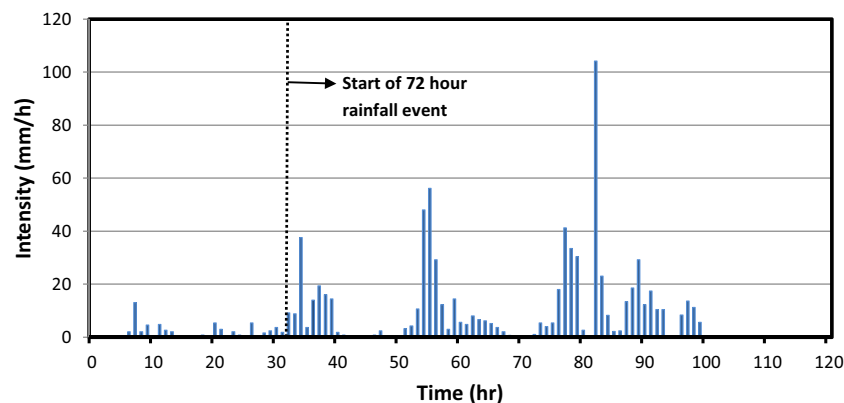
**Fig. 2** Mass curves of rainfall obtained from two rain gauges: Jabiru Airport (014198) and G8210009 from 26 February to 2 March 2007 (Erskine and Saynor 2012)

Undisturbed and disturbed samples at each location were taken at three depths: ground surface, mid-depth (1 m deep) and the base of the landslide (2 m deep). Undisturbed samples were obtained by gently driving steel cylinders (with one end chamfered) into the soil. These samples were used for water retention tests. Disturbed samples were taken for basic, index and direct shear tests. The sample site locations are shown in Fig. 6 and coordinates given in Table 1. Eight undisturbed samples were collected at four points (two samples at each point), at S1 and S3 (at the slope surface), S4 (at the mid-depth of the landslide) and S5 (at the landslide base). Only one sample was taken at the landslide base as the material was generally extremely hard, intact and impossible to penetrate with a sampling ring. This observation was consistent with the fact that movement of the slope occurred along the interface between the overlying soil and the intact material below. Disturbed samples were collected from five points: S1, S2 and S7 (at the slope surface), S4 (at the mid-depth) and S5 (at the landslide base). In situ hydraulic conductivity tests were also conducted at five points at the slope surface (S1, S2, S3, S6 and S7), one point at the mid-depth (S4) and one point within the landslide base (S5).

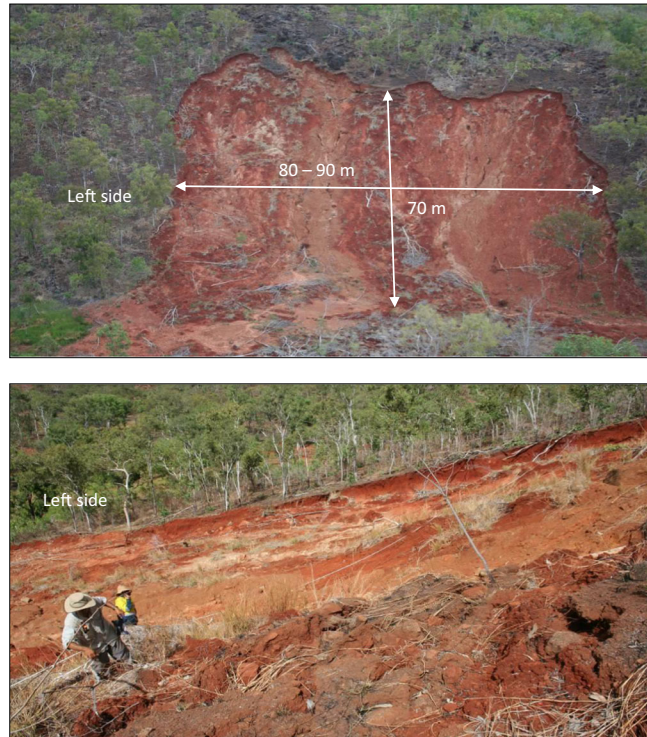
Saturated hydraulic conductivity was determined using a tension infiltrometer (CSIRO 1988). We used a 1988 CSIRO Disc Permeameter, which is relatively easy to use in the field and is also easily moved from one site to another, which was essential given the access limitations.

### Laboratory tests

Besides hydraulic conductivity, the soil water characteristic curve (SWCC) is another hydraulic property required in slope stability analyses. Water retention tests were conducted to identify the relationship between the change of water content and matric suction using the method described in detail in ASTM International (2008). This relationship is usually illustrated as a curve called the SWCC and is very useful in investigating the movement of water in unsaturated soils. Water retention tests were performed using a pressure plate with applied pressures up to 800 kPa. The undisturbed soil samples were tested using the pressure plates, and each pair of samples was tested in two different pressure plates, differentiated as B and W in the results section. Shear box tests were conducted to determine the effective stress shear strength parameters (i.e. apparent cohesion,  $c'$ , and angle of internal friction,  $\phi'$ ). Both dry and saturated samples were used in these tests, using a standard direct shear box (Australian Standard 2000). Undisturbed samples for strength testing could not be obtained from the site, so all shear box tests were carried out on samples prepared at the relevant field dry density. Index tests (Atterberg limits, bulk density and specific gravity) were also conducted in the laboratory using Australian Standard methods (AS 1289.3.9 Liquid Limit test using Cone Penetrometer; AS 1289.3.2.1 Plastic Limit test and AS 1289.3.5.2 specific gravity test using Pycnometer). Wet sieving



**Fig. 3** Hourly rainfall data obtained from Bureau of Meteorology's Jabiru Airport (Station 014198) from 26 February to 2 March 2007



**Fig. 4** Frontal (*upper photo*) and side view (*lower photo*) of the landslide at Jabiru that was studied in detail

and hydrometer analyses were carried out using British Standard (1990) to determine the particle size distribution.

## Results

### In situ hydraulic conductivity

The results of tension infiltrometer testing using the disc permeameter are presented in Table 1. The hydraulic conductivity ( $k_{sat}$ ) values decreased towards the base of the slope.

### Index and particle size distribution tests, and soil classification

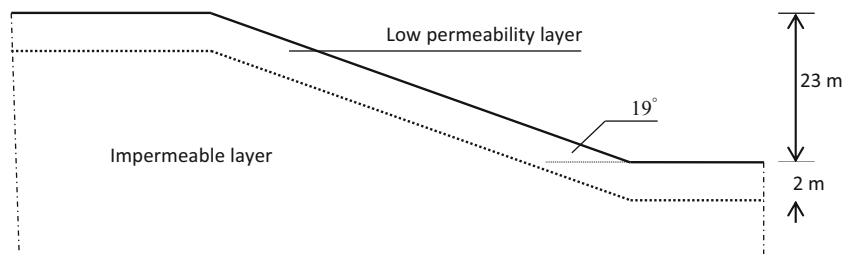
The results of the index tests, particle size distribution tests and the resulting soil classification according to the Unified Soil Classification System (USCS) are shown in Table 2. The specific gravity and bulk density of the soil varied from 2.6 to 2.9 and from 1.4 to 1.5 g/cm<sup>3</sup>, respectively. Soil samples S2, S4 and S7 indicate well-graded soil (Folk and Ward 1957; Folk 1974; Craig 1997). On the other hand, soil sample S5 was more uniformly sized. From five samples taken at the site, all were indicated as sandy or gravelly silt and clay with fine particles (<75 μm) ranging from 39 to 72 %.

### Water retention tests

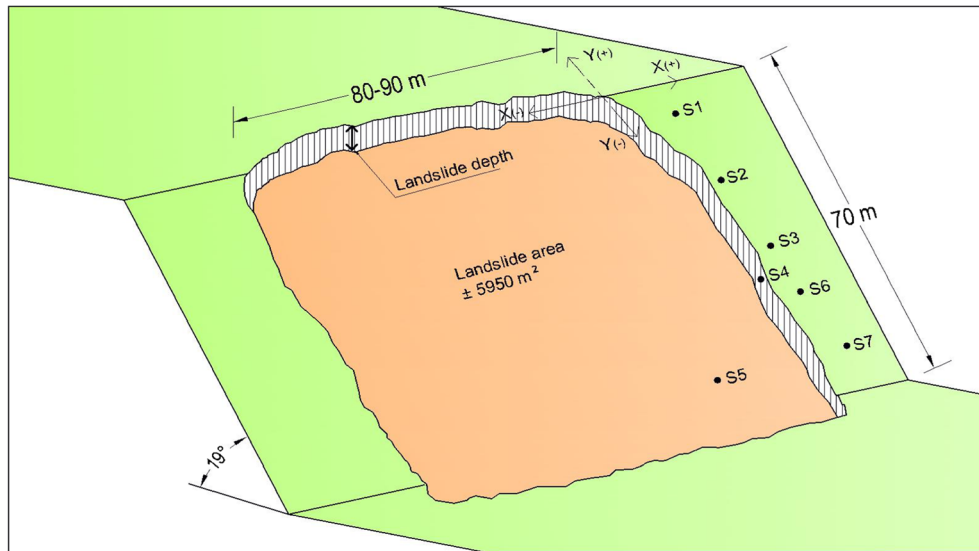
The SWCCs were plotted using data collected from the water retention tests, then refined using the Fredlund and Xing (1994) method and are shown in Fig. 7. The following formula was used to plot the best-fit curve of the SWCC which relates matric suction ( $\psi$ ) and volumetric water content ( $\theta$ ).

$$\theta = \theta_s \left[ \frac{1}{\ln \left[ e + \left( \frac{\psi}{a} \right)^n \right]} \right]^m \quad (1)$$

where  $\theta_s$  is saturated volumetric water content;  $a$ ,  $m$  and  $n$  are parameters determining the shape of the SWCC; and  $e$  is the natural number. The average curve was also derived from all collected data for the representation of the SWCC data used in subsequent stability analyses. The average SWCCs for the upper layer, base and entire layer were termed as  $A_{vu}$ ,  $A_{vl}$  and  $A_v$  respectively. It should be noted that only drying tests were done and these were used in the seepage modelling. Although some



**Fig. 5** Simplified geometry of the Jabiru slope, for use in seepage and stability modelling



**Fig. 6** Locations of sample points for soil sampling and field tests at the Jabiru slope

hysteresis in the SWCC is likely, this was not taken into account as it would also require knowledge of intermediate ‘scanning’ curves that are intermediate between a full drying and a full wetting curve. The key findings presented later in the paper, dealing with stability under near-saturated conditions, are less sensitive to the exact shape of the SWCC curve than factors such as saturated hydraulic conductivity and shear strength parameters. Future work will investigate in more detail the impact of using drying, wetting and intermediate SWCCs.

#### Direct shear tests

Shear strength parameters from the direct shear tests and hydraulic conductivity values from infiltrometer tests are summarised in Table 3.

#### Modelling

Having established values for the parameters required for use in an evaluation of slope stability at the Jabiru site, coupled seepage and stability analyses were carried out using the software SVFLUX (Thode and Gitirana 2008) for seepage analyses and SVSLOPE (Fredlund et al. 2008) for the slope stability evaluations.

#### Seepage analyses

SVFLUX was used to simulate seepage in the slope exposed to rainfall. The geometry was simplified, assuming a planar contact between the overlying, weathered dolerite and the unweathered bedrock below. Although in reality this surface was slightly undulating, visual inspection of the failure scar indicated a planar surface was a good approximation.

In the modelling, rainfall was applied as a boundary flux at the ground surface during rainfall; once a rainfall event was over, this boundary condition was altered to a free surface to allow seepage to continue unhindered (in the software used, this required the ‘review by pressure’ boundary condition to be invoked). ‘Zero flux’ was applied at the base of the dolerite and ‘review by pressure’ was also applied at the left and right boundaries to allow free flow of water parallel to the soil layer. Boundary conditions are illustrated in Fig. 8. No groundwater table was found during the site investigation (undertaken during the dry season), although the potential for a perched water table to develop during the wet season cannot be discounted. Suction was set as an initial condition of the slope, based on SWCC data and in situ water contents. Although there will invariably be some variation of suction within a slope, an

**Table 1** Data from the disc permeameter test

Testing point	Coordinate <sup>a</sup> (m)	$k_{spt}$ m/s
S1	4; -5	$7.6E^{-7}$
S2	1; -14	$8.7E^{-6}$
S3	2; -27	$1.2E^{-6}$
S4	0; -30	$2.5E^{-6}$
S5	-18; -48	$2.5E^{-6}$
S6	3; -35	$6.1E^{-8}$
S7	4; -50	$6.9E^{-8}$

<sup>a</sup> Refer to Fig. 6

**Table 2** Index properties and soil classification

Sample	SG	$\rho$ g/cm <sup>3</sup>	w %	CP %	FP %	LL %	PL %	PI %	Soil type
S1	2.82	1.45	33	61	39	50	37	13	Gravelly silt (MG)
S2	2.68		32	38	62	54	35	19	Sandy silt (MS)
S4	2.86	1.49	26	28	72	56	30	26	Clay (C)
S5	2.89	1.42	30	57	43	41	30	11	Sandy silt (MS)
S7	2.75		32	47	53	53	37	16	Sandy silt (MS)

SG specific gravity,  $\rho$  bulk density, w natural water content, CP coarse particles, FP fine particles, LL liquid limit, PL plastic limit, PI plasticity index (PI=LL-PL)

initially uniform value was chosen to simplify subsequent interpretation. Unsaturated hydraulic properties were used in these analyses. The modified Campbell (1974) estimation, using the measured SWCC curves, was applied to take account of unsaturated hydraulic conductivity in this analysis. Eventually, this seepage analysis model was solved by Flex partial differential equation (PDE) and the results visualised with this application and ACUMESH.

**Slope stability analyses**

Unsaturated shear strength was utilised in the slope stability analyses to include the contribution of matric suction as proposed by Fredlund and Rahardjo (1993).

$$\tau = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b$$

where  $\tau$ =shear strength,  $c'$ =apparent cohesion,  $(\sigma_n - u_a)$ =net normal stress,  $\sigma_n$ =total normal stress,  $u_a$ =pore-air pressure,  $\phi'$ =angle of internal friction,  $(u_a - u_w)$ =matric suction,  $u_w$ =pore water pressure and  $\phi^b$ =angle indicating the rate of increase in shear strength relative to the matric suction ( $\phi^b = 0.5 \phi'$  was used in these analyses, based on Rahardjo et al. 2010).

**Modelling results**

It has been found that rainfall intensity and duration, as well as hydraulic and shear strength parameters play key roles in the stability of slopes exposed to rainfall (Tsaparas et al. 2002; Rahardjo et al. 2007). A parametric study was thus performed to investigate the effect of these parameters on the prediction of likely rainfall-induced landslides at the Jabiru site. All the controlling parameters were varied, based on data collected from the field and laboratory tests and previous investigations. This was done to

ascertain whether the failures at Jabiru (i.e. a factor of safety less than or equal to unity) could have been predicted using software such as SoilVision, together with site-specific parameters and known rainfall conditions. The slope geometry used in this study had an angle of 19° and was about 70 m in length as identified from the field observations (Fig. 4). The depth of the surficial soil layer was assumed to be 2 m, based on observations made at the site of the landslide.

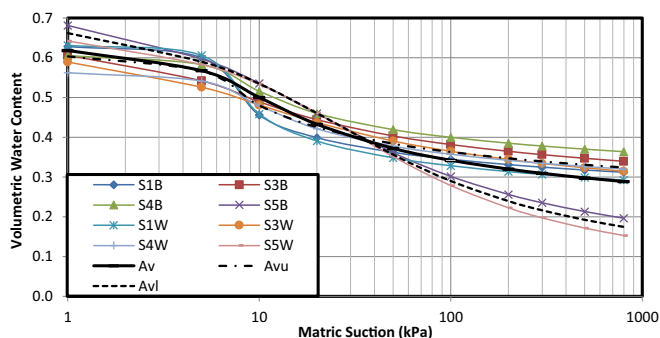
Controlling parameters used in this study are summarised in Table 4. With regard to rainfall data obtained from the landslide occurrence in 2007, the maximum rainfall amount was recorded as 784 mm during 3 days (Saynor et al. 2012). Variability of rainfall intensity and volume was applied in this study based on these numbers. Field and laboratory measurements (Table 3) show saturated hydraulic conductivity ranges from 0.22 to 31.33 mm/h, and shear strength parameters vary between 0 and 6.6 kPa for cohesion ( $c'$ ) and 25° to 39° for internal friction angle ( $\phi'$ ). Therefore, hydraulic and shear strength parameters used in this parametric study were varied around these values with other (deemed plausible based on experimental results) values as shown in square brackets (Table 4).

The results of the parametric study are discussed in conjunction with the effect of each controlling parameter on the slope stability.

**The effect of rainfall event**

For this parameter, both rainfall intensity and volume were varied as shown in Table 4 (cases 1-a and 1-b) and the rest used the rainfall data, shown as case 1-c. The remaining parameters were applied as mean values,  $k_s=8$  mm/h,  $\psi_i=42$  kPa,  $c'=3$  kPa and  $\phi'=32^\circ$ . The numbers shown in the square brackets in this table indicate parameters that were varied while all others were kept constant. Results are summarised in Fig. 9. The factor of safety gradually decreases once rainfall begins, then shows a rapid reduction when the surface layer becomes fully saturated, until reaching the lowest value ( $F_{min}$ ). The minimum factor of safety ( $F_{min}=1.2$ ) was achieved when a sufficient amount of rainwater ( $V_{min}=500$  mm) falls on the slope. It remains constant until the end of rainfall, and then slowly recovers as the soil dries. None of these analyses show a factor of safety (FoS) dipping below unity, a value that is considered to represent instability of the slope. Conditions under which a FoS less than unity occurred in the modelling exercise are presented later in the paper.

Rainfall intensity has a significant effect on the rate at which the factor of safety decreases. The higher the intensity of rainfall, the higher the rate of reduction of safety factor. However, if the rainfall



**Fig. 7** Soil water characteristic curves (SWCC) for undisturbed samples

**Table 3** Summary of controlling parameters obtained from field and laboratory tests

Testing points	Hydraulic conductivity $k_s$ (mm/h)	Shear strength parameters	
		$c'$ (kPa)	$\phi'$ (deg)
S1	0.27	0.0	39
S2	31.33	6.6	25
S3	4.24	N/A	N/A
S4	8.98	2.0	34
S5	8.98	4.7	32
S6	0.22	N/A	N/A
S7	0.25	3.6	30

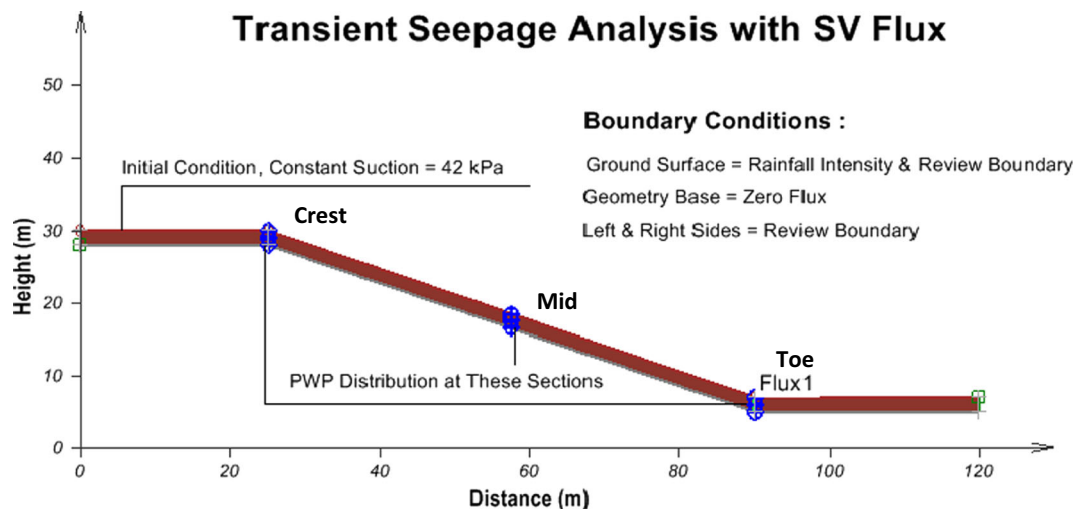
intensity is higher than the saturated hydraulic conductivity ( $k_{sat}$ ) of the surface soil, some of the rainfall becomes runoff (evaporation was not considered in these analyses). At rainfall intensities higher than 8 mm/h, increases in intensity have a reduced impact on the rate of decrease of the factor of safety. The reason that intensities higher than  $k_{sat}$  result in earlier decline of the factor of safety than occurs when the intensity is equal to  $k_{sat}$  is that when rainfall starts, the in situ suction provides a hydraulic gradient that is higher than simply that due to gravity alone; hence, infiltration rates can be higher than that equivalent to  $k_{sat}$ . None of the analyses predicted a factor of safety of unity.

Even rainfall intensities less than  $k_{sat}$  may result in slope instability in some slopes, as long as the duration of rainfall is long enough because it reduces the matric suction contribution to shear strength to below the value required to ensure stability. However, seepage through slopes with high hydraulic conductivity, such as many slopes in Hong Kong and Singapore (Brand et al. 1984; Pitts 1985), where rainfall intensity is much lower than  $k_{sat}$  has a negligible effect on the stability of the slope exposed to it. In such cases, rainwater infiltration quickly drains due to the porous nature of the soils, and the impact on suction reduction is minimal. At the Jabiru site, the surficial soils have lower hydraulic conductivities (than most in Hong Kong for instance;  $k_s \leq 10^{-6}$  m/s at Jabiru). At Jabiru therefore, shallow failure mechanisms are more dependent on volume than intensity of rainfall because at high intensities

(above the infiltration capacity) runoff occurs. Thus, prolonged rainfall is usually required to trigger slope failures at the Jabiru site. This observation is in line with those made at slopes containing clay-like materials that are normally more sensitive to rainfall with a long duration and moderate intensity (Cannon and Ellen 1988; Berti and Simoni 2005; Casagli et al. 2006).

The amount of rainwater infiltrating into a soil slope determines whether or not it becomes fully saturated and whether it reaches  $F_{min}$  for the given shear strength parameters. As shown in Fig. 9b, rainfall with a volume of 600 mm ( $\text{mm}^3/\text{mm}^2$ ) or more causes the most significant reduction in factor of safety, reaching  $F_{min}$  (for volumes of 600 to 1000 mm). In contrast, when the volume of infiltration is less than 600 mm, the slope never reaches the same  $F_{min}$  as it does for the larger volumes and the slope remains partially saturated. Consequently, the factor of safety essentially remains steady although it has not reached the lower  $F_{min}$  because rainwater infiltration and drainage have reached equilibrium.

An additional analysis was done, using the measured rainfall of about 800 mm in 3 days. The fluctuation of intensity that typically occurs with natural rainfall could produce a different amount of infiltration into subsurface soils than an assumed constant intensity of similar volume. High resolution of rainfall (e.g. with hourly recorded data) may produce lower predicted infiltration than low resolution data, as indicated by Hearman and Hinz (2007), which

**Fig. 8** Slope geometry and boundary conditions applied in the seepage analysis



**Table 4** Summary of various parameters applied in parametric study

Scenario	Rainfall pattern		Sat. hydraulic conductivity $k_s$ (mm/h)	Initial suction $\psi$ (kPa)	Shear strength parameters	
	Intensity (mm/h)	Volume (mm)			Cohesion, $c'$ (kPa)	Int. friction angle, $\phi'$ (deg)
1-a	[4, 6, 8, 10, 12]	800	8	42	3	32
1-b	8	[200, 400, 600, 800, 1000]	8	42	3	32
1-c	Rainfall data (Fig. 3)		8	[75, 42, 10]	3	32
2	8	800	[80, 20, 8, 2, 0.8]	42	3	32
3	8	800	8	[95, 18, 8, 3]	3	32
4-a	8	800	8	42	[0, 3, 6, 9]	32
4-b	8	800	8	42	3	[24, 28, 32, 36, 40]
4-c	8	800	8	42	[S1=0, S2=6.6, S4=2, S5=4.7, S7=3.6]	[S1=39.7, S2=25.3, S4=34.6, S5=31.8, S7=30.3]

in turn leads to predictions of delayed potential slope instability. The  $F_{min}$  in our analyses using the available high resolution data was achieved later than that using the average intensity (low resolution) for the same amount of rainfall, indicating good agreement with the argument of Hearman and Hinz (2007). This can be attributed to periods of extremely high intensity rainfall during the Jabiru storm (see Fig. 3), which were much higher than  $k_{sat}$ , meaning that some runoff must have occurred during these high intensity (albeit very short duration) bursts of rainfall. Figure 9c shows the results of these analyses, using three different values of initial suction (where the suction value was assumed constant throughout the slope). The results show a minimum factor of safety value of 1.2, with the time to reach this minimum value increasing as the value of initial suction increases. This is of course to be expected, as a higher initial suction means the soil is drier and has more storage space in the voids. Therefore, more rainfall is required to reduce the suction to critical values. Although this observation is perhaps intuitive, it illustrates the extreme importance of the state of a soil slope at the time an extreme rainfall event occurs. Immediately after a period of prolonged, light rainfall, a slope may be susceptible to landslide triggering even when exposed to a relatively minor storm event, whereas after a prolonged dry period, an extreme storm event may be required to trigger instability.

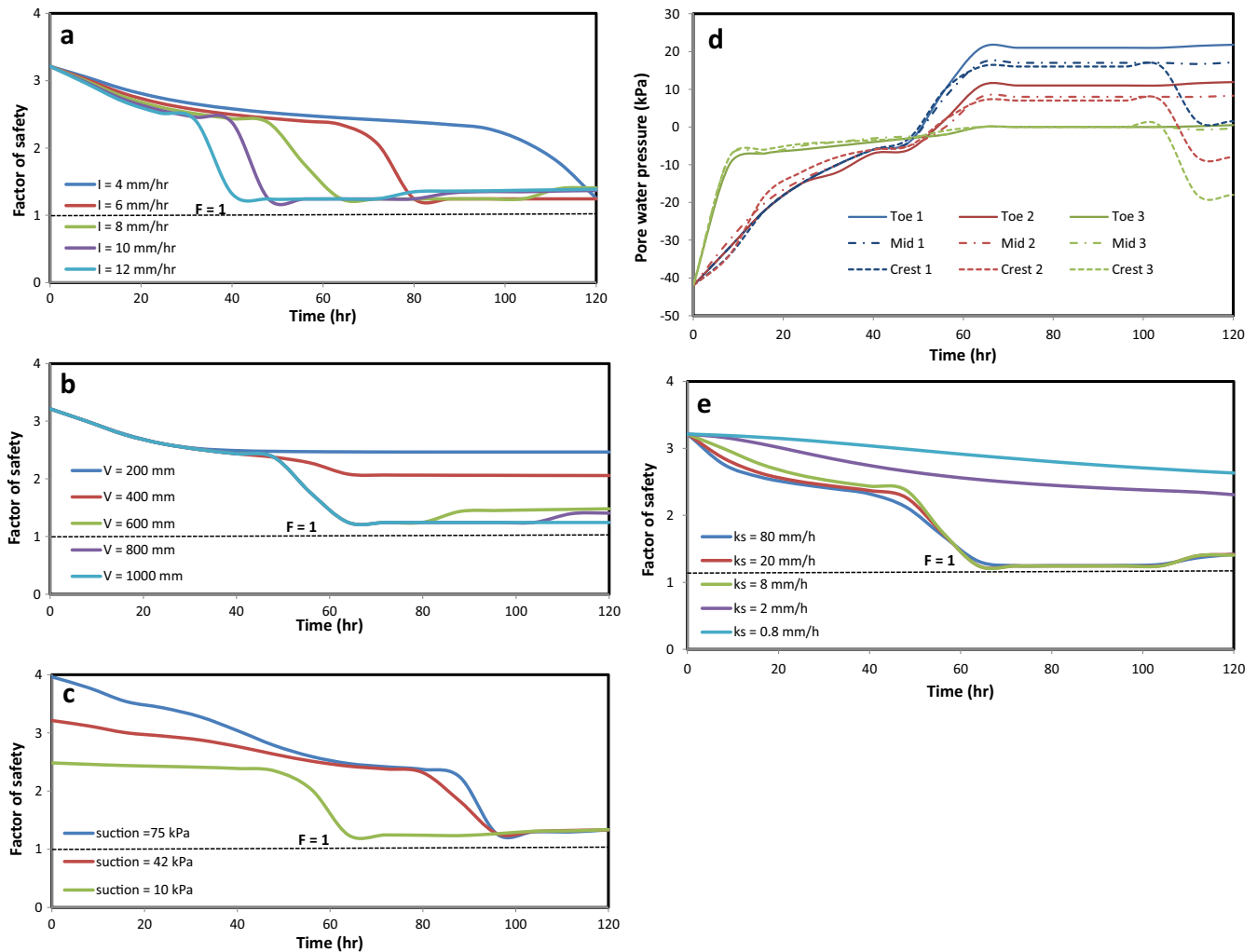
For the baseline suction value of 42 kPa, the time to reach  $F_{min}$  is about 96 h, which is much higher than the majority of times to reach  $F_{min}$  shown in Fig. 9a, b, confirming that using high resolution rainfall data in this case delays the time at which failure may have been triggered.

Generally, pore water pressure rises (becomes less negative) once rainfall starts and infiltration begins, and decreases after rainfall stops. However, the change of pore water pressure is not uniform over the entire soil layer, as illustrated by the modelling results in Fig. 9d, which shows the variation of pore pressure at three depths at each of three locations (the crest and toe of the slope as well as midway between) with time. At the soil surface (indicated as position 3 in Fig. 9d), pore water pressure increases

once rainfall begins and slows down when the soil becomes nearly fully saturated. The pore water pressure remains constant after the entire soil layer becomes fully saturated. Meanwhile, pore water pressure at some depths (positions 1 and 2) increases gradually after rainfall begins, followed by a sharp increase when the soil layer is nearly fully saturated. Thereafter, it remains constant until the end of rainfall. The pore water pressure remains constant even after the end of rainfall at all slope positions except at the slope crest (top) where it drops rapidly. Generally, the pore water pressure generated at the slope toe is higher than at the upper parts of the slope, for the same depth below ground surface. This is because rainwater infiltration drains from the top of the slope once rainfall stops, but seepage into the lower part of the slope (from the upper part) continues even after rainfall ceases. The lower the hydraulic conductivity of the slope soils, the longer the time required to start draining the soil layer at the lower part of the slope, particularly at the slope toe. The results in Fig. 9d also show that the pore water pressure values almost reach hydrostatic values during the rainfall event, and certainly become positive within most of the slope. This is because the low permeability interface at 2 m restricts downward drainage, forcing infiltrating water to begin flowing parallel to this interface, which, when coupled with the low hydraulic conductivity, results in positive pore pressures developing. As noted earlier, the field tests indicated that for similar depths, the soil near the toe of the slope has a lower hydraulic conductivity than higher up the slope. Although not modelled in this paper, this heterogeneous hydraulic conductivity would further impede drainage of water from the slope, increasing the likelihood of slope failure during a particular rainfall event. It was beyond the scope of this paper to investigate this hypothesis.

**The effect of soil properties**

There are two key soil properties relevant to rainfall-induced slope failures: hydraulic and shear strength properties. To investigate the effect of these properties, each parameter was varied while the remaining parameters were kept constant. Figure 9e summarises results for simulations where the hydraulic conductivity was

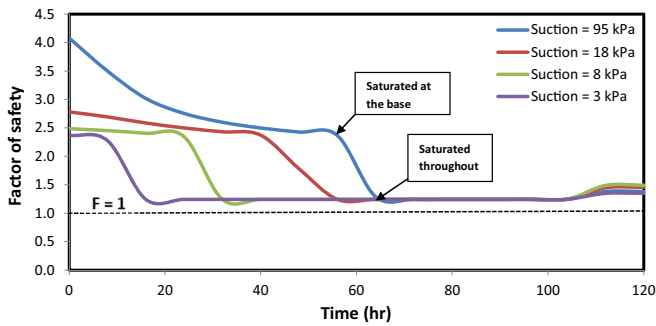


**Fig. 9** Factor of safety vs time curves with different rainfall events (all other material parameters kept constant). **a** Various rainfall intensities; **b** various rainfall volumes; **c** rainfall data with various initial suctions; **d** variation of predicted pore water pressure with time at three locations on the slope (crest, midway and toe) and at three different depths (positions 3 at the soil surface, 1 at the base of surface soils and 2 at midway between positions 1 and 3); **e** factor of safety vs time curves with different hydraulic conductivities (all other material parameters kept constant)

varied either side of the measured values. Rainfall applied to a slope with hydraulic conductivity roughly equal to and/or higher than rainfall intensity ( $k_s \geq I$ ) can significantly reduce the factor of safety of the slope. In this case, all rainwater can infiltrate into the slope, as little or no runoff occurs. Contrarily, rainfall applied to a slope with much lower conductivity than the intensity of the rainfall ( $k_s < I$ ) results in a relatively small effect on the slope stability, as shown in Fig. 9e, because only a small amount of rainwater infiltrates into the slope.

The initial suction in the soil profile affects the rate of reduction of the factor of safety of a slope exposed to rainfall and the time to reach  $F_{min}$ , which potentially triggers slope failure, as shown in Fig. 10, in which the initial (assumed uniform) suction was varied in either side of the measured in situ values. The higher the initial suction in a slope, the faster the rate of reduction in the factor of safety during a rainfall event. The factor of safety of a slope with a relatively high initial suction initially decreases very rapidly, due to the high hydraulic gradient produced by the suction, but of course

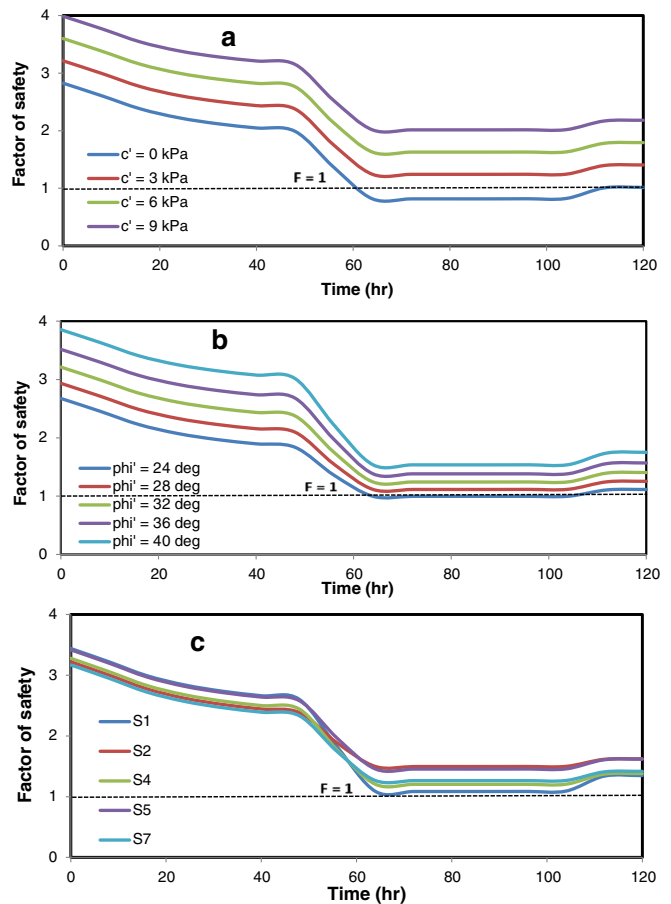
takes much longer to reach  $F_{min}$  because a greater volume of water is needed to bring the soil close to a state of saturation (and thus to a state of low suction). The results summarised in Fig. 10 illustrate a critically important aspect of slopes susceptible to rainfall-induced landsliding, which is that the likelihood of slope failure being triggered by a particular rainfall event is governed to a large degree by the ambient conditions in the slope at the time a particular rainfall event occurs. As can be seen in Fig. 10, if there had previously been sufficient rainfall to reduce the suctions in the slopes at Jabiru to 8 kPa or less,  $F_{min}$  would have been reached in only 32 h (or less). Any risk management strategy for slopes of the type discussed in this paper would therefore clearly have to account for this factor in some way. The risk of landsliding can thus not be quantified solely in terms of the engineering properties of the slope and the rainfall characteristics, but needs to take account of the state (defined in terms of in situ suction or water content) at the time rainfall occurs. The time at which the bottom of the upper, more permeable soil layer at the toe of the slope becomes



**Fig. 10** Predicted variation of factor of safety with time for different initial suction values (all other material parameters kept constant)

saturated is shown in Fig. 10 for the condition of 95 kPa initial suction; also shown is the time when the entire soil layer becomes saturated.

Shear strength parameters  $c'$  and  $\phi'$  have an important effect on the ability of a slope to withstand a particular rainfall event, with the apparent cohesion  $c'$  playing a somewhat disproportionate role because it is not dependent on the prevailing effective stress (which is low near the ground surface), unlike the frictional component of strength (Fourie 1996). Although these effects are well known, results for the Jabiru site with varying strength parameters are summarised in Fig. 11. They illustrate behaviour consistent with the above discussion. They also illustrate another key point. All the analyses of the Jabiru site presented so far, even those using actual rainfall data that resulted in the slope failures that occurred in 2007, did not predict factors of safety less than unity, i.e. apparently did not predict the failures. However, decreasing the apparent cohesion from the measured mean value of 3 kPa, to zero, results in a factor of safety well below zero, as shown in Fig. 11a. Although not shown in Fig. 11a, even with a finite apparent cohesion (of 1.5 kPa), the factor of safety still reached unity. It has been suggested (Harr 1987) that the coefficient of variation of apparent cohesion is of the order of 40 %, so an assumption of 1.5 kPa is not unreasonable for the Jabiru site. Given the acknowledged variability of apparent cohesion, and the sensitivity of factor of safety to this parameter, it would be prudent to assume zero apparent cohesion if stability analyses of the Jabiru slopes had been carried out prior to the failures occurring. Such analyses were not carried out, primarily because there had previously been no reported incidences of slope failures, plus the slopes are in a very remote part of Arnhem Land in the Northern Territory of Australia, and there are no communities living in the area, so risks to people are minimal. Contrary to the results for apparent cohesion, to reduce the factor of safety to less than unity by decreasing the angle of internal friction, a value of  $\phi'$  of 24° is required, i.e. a reduction of 25 %. However, according to Harr (1987), a coefficient of variation of only 7 to 12 % is usual for  $\phi'$ , implying a lower bound value of 28° would be appropriate for the measured mean value of 32°. The implication of adopting an overly optimistic value for apparent cohesion is thus clearly evident; it may predict a slope has a satisfactory factor of safety when in fact it does not. For example, for apparent cohesion values of zero and 3 kPa, there is a difference in predicted factor of safety of more than 50 % for time after start of rainfall in



**Fig. 11** Factor of safety vs time curves with different shear strength values (all other material parameters kept constant). a Various apparent cohesions; b various internal friction angles; c various shear strength values due to different testing points

excess of 60 h. The angle of internal friction has less of an impact on predicted factor of safety. Additionally, it commonly has a low coefficient of variation.

As a final step in the parametric study, analyses were carried out using shear strength parameters measured in each of the individual tests (Table 3), keeping all other parameters equal. The initial factor of safety of the slope resulting from the stability analysis using shear strength parameters of sample S1 is relatively high compared to those from the other samples. It then reaches the lowest  $F_{\min}$  during rainfall and even indicates near failure ( $F_{\min}$  approaches 1). This can be attributed to the relatively high internal friction but zero apparent cohesion applied in this analysis.

From several scenarios of the parametric study above, the minimum factor of safety is less than the value of 1.3 commonly prescribed in engineering practice. Additionally, either by neglecting the contribution of apparent cohesion or by taking account of curvature of the Mohr-Coulomb failure surface at low effective stress values (as suggested by Noor and Hadi (2010)), a factor of safety less than or equal to unity could have been predicted for the Jabiru slope when subjected to the rainfall event of February/March 2007.

## Discussion

The rainfall that occurred from 27 February to 2 March 2007 in the Jabiru region of the Northern Territory, Australia, provided a unique illustration of the concept of a threshold event. Until the time of the 2007 rainfall event, which was equivalent to an event with a recurrence interval in excess of 1:1000 years (Erskine and Saynor 2012; Saynor et al. 2012), there had been no reports of landslides occurring in the study region. However, the 2007 event triggered 49 documented landslides. One particular slide was chosen for more detailed study, based on its relatively easy access by helicopter and large size (almost 10,000 m<sup>3</sup> slide volume).

Analysis of the stability of this particular landslide was carried out using strength data from routine shear strength tests, together with results from in situ measurements of hydraulic conductivity and water retention tests on undisturbed specimens recovered from positions adjacent to and below the slide scar. Using this data in conventional (but coupled) seepage and limit equilibrium slope stability analyses yielded results which indicate that this particular failure that occurred at Jabiru could have been predicted, had the rainfall data been available prior to the failures occurring. The benefits of this finding is that for analyses of slopes of the kind described in this paper, which are characterised by a finite depth of highly weathered material overlying more competent and intact underlying bedrock, conventional limit equilibrium analyses are acceptable, as long as good quality material characteristics are available, and account is taken of the unsaturated nature of the material (particularly the contribution of suction to shear strength and the dependency of hydraulic conductivity on suction).

The rainfall event that triggered the Jabiru slope failures was an extreme event. Figure 12 shows Bureau of Meteorology intensity-frequency-duration (IFD) curves for the Gulungul Creek (located about 20 km from the landslide area) gauging station. IFD curves are commonly used to represent rainfall data for a particular site. The various lines shown in Fig. 12 are for different return periods

(also referred to as recurrence intervals). Much of engineering practice is based on assessments of probability of occurrence; thus, for example, many mining facilities (such as tailings storage facilities) have an operational design ‘storm’ being the 1:100-year, 72-h event. It enables the required capacity of stormwater structures to be calculated.

For the IFD curves relevant to Jabiru, the rainfall event is shown in bold, and the PMP shown by diamonds. The rainfall event produced the highest level recorded at this gauging station since gauging commenced in 1971. From Fig. 12, it can be seen that for durations more than 6 h, this rainfall event exceeded the 100-year return period, and as discussed by Erskine and Saynor (2012), for durations over 48 h, the return period exceeded 1000 years. Figure 9b predicts that for the slope studied, rainfall of more than 400 mm would have been necessary to reduce the FoS to a minimum value. For a rainfall volume of 600 mm, the minimum value was reached. Taking 400 mm as a lower bound value, according to Fig. 12, even a 48-h 1:100-year event would not have been sufficient to cause the slope to reach a minimum FoS. A 72-h, 1:100-year event would likely have produced sufficient volume of rainfall (about 432 mm) to achieve a minimum FoS. However, given the nature of the rainfall in this part of the Northern Territory, where intensities of 60 mm/h, or more, occur (see Fig. 3), for the measured in situ saturated hydraulic conductivity of approximately 8 mm/h, a 72-h storm with an intensity in excess of 1:100 would likely be necessary to trigger slope failure; an event such as occurred in 2007.

Considering the rainfall pattern shown in Fig. 3, and remembering that the 72-h period in question began at the time shown by the dashed vertical line, it seems likely that the slope only failed when the burst of rainfall at around 100 h occurred (which is about 60 h from the start of the intense rainfall event). This implies that relatively little rainfall fell after the slope failures occurred and that any discolouration of the stream water (see Erskine and Saynor

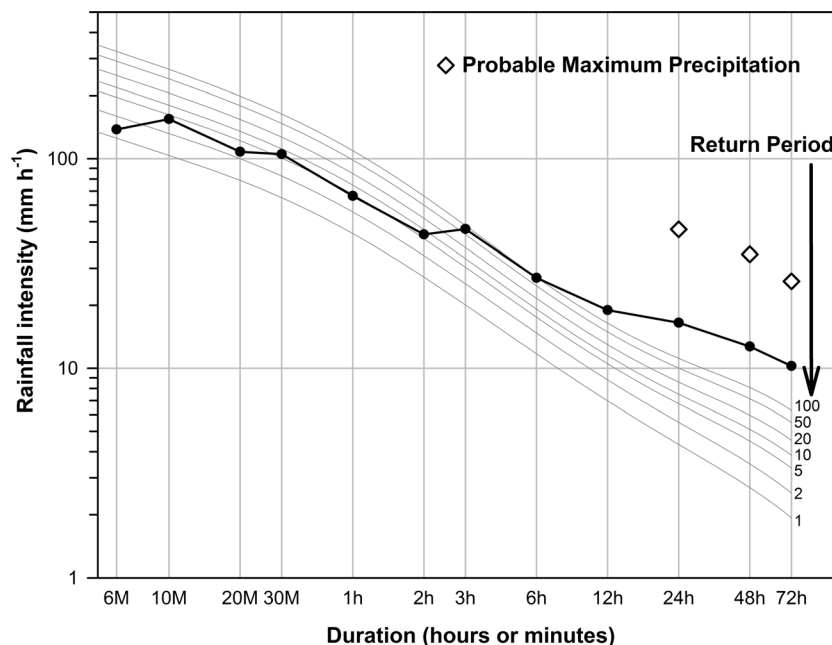


Fig. 12 Rainfall intensity-frequency-duration curves for the Gulungul Creek gauging station in 2007 (after Moliere et al. 2007; Erskine and Saynor 2012)

2012) was lost or diluted because there was so much water flowing down the creek. As mentioned before, existence of the landslides only became apparent when a helicopter pilot reported seeing them in late March 2007. In January 2008, staff of the Supervising Scientist in the Northern Territory undertaking routine helicopter monitoring trips around the nearby Ranger mine reported seeing 'red pulses' in Magela Creek. When the red water was followed upstream, it led directly to the area of the landslides. As discussed by Erskine and Saynor (2012), these red pulses were attributed to localised storms occurring directly over the landslide zone. The fact that these pulses were not noticed at the time the landslides occurred is attributed to the suggestion that the failure(s) occurred towards the very end of the rainfall event and there was subsequently not sufficient rainfall to erode soil from the failed mass. It is also likely that during the year or so between the failures and the 'red pulses', soil from the failure scars would have softened (i.e. become more erodible) during subsequent rainfall events, thus becoming more susceptible to erosion when a significant storm next occurred. Although it is conjecture, it is suggested that but for the rainfall burst at around 100 h (in Fig. 12), most, if not all, of the landslides that occurred would not have occurred. As reported by Saynor et al. (2012), field and aerial inspections together with stereoscopic vertical air photographs and ALOS satellite image interpretation failed to find any evidence of older landslides, except for occasional rockfalls along sandstone escarpments. The event of 2007 was thus certainly extreme, resulting in 49 separate landslides, where before there had not been any (certainly not for a very long time). This is consistent with the observation mentioned before that the 2007 event was equivalent to a return period in excess of 1000 years.

## Conclusions

This paper describes results of a series of analyses of the effect of an extreme rainfall event (in excess of 750 mm rainfall in 72 h) on natural slopes having a finite thickness of highly weathered residual dolerite. Focussing on one particular slope failure, the following conclusions may be drawn from the work:

- Failures occurred in slopes having an inclination of around 19°, despite the angle of internal friction of the soil averaging 32°. The profile of the slopes in question having a veneer about 2 m thick of weathered dolerite overlying intact sandstone probably resulted in the development of near-hydrostatic pore pressures in the slope during the rainfall event, which in turn caused the factor of safety to reduce to unity, triggering shallow slope failures.
- The failure that was studied in some detail occurred probably because not only did the negative pore pressures (suctions) increase to zero, but positive pore pressures developed. The rainfall event was long enough for this condition to develop.
- In situ conditions, specifically suction, at the time a rainfall event occurs have a critical influence on the response of a slope to rainfall. The initial suction value may in fact be the difference between a failure occurring and not occurring. Once rainfall ceases, assuming failure was not triggered, the factor of safety tends to increase again, as the pore water pressures begin to decrease. This observation points to the value of monitoring antecedent conditions, although developing a correlation between antecedent conditions and a

threshold rainfall event that will trigger a landslide is not a trivial exercise.

- The work confirmed the critical contribution of the apparent cohesion to preventing shallow slope failures. Given the extremely variable nature of apparent cohesion, the oft-touted, conservative approach of ignoring apparent cohesion appears to be sensible, in the case of the Jabiru slope studied. In analyses in which the apparent cohesion was reduced to zero (or even to only 1.5 kPa), a factor of safety of unity (or less) was predicted. When the mean value of apparent cohesion obtained from laboratory tests, together with other measured parameters, was used in stability analyses, the FoS was typically around 1.2. Thus, if the parameter  $c'$  is to be included in evaluations of the likelihood of shallow slope failures occurring, the validity of the chosen value of  $c'$  must be guaranteed.

The slope failure that occurred in Jabiru in 2007 provided an opportunity to compare an actual failure that was triggered by an extreme rainfall event with predictions made using conventional seepage and limit equilibrium stability analyses. Essential to making a meaningful and accurate evaluation of the impacts on slope stability of an extreme event was quantification of shear strength and hydraulic parameters. The ability of the approach outlined in this paper to predict a failure of the type that occurred in 2007 makes it possible to predict the potential impact of future extreme events with increased confidence. However, we acknowledge that only 1 of the 49 landslides that occurred due the 2007 extreme event was studied in detail and we cannot simply extrapolate our findings to the other 48 failures.

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