Analysis of design choices for a slope stability scenario in the humid tropics

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Prediction of the conditions under which landslides may occur is essential for designing sustainable risk-mitigation measures and cost-effective geotechnical structures. Slope stability analyses typically account for slope geometry, soil mechanical properties and groundwater conditions to determine the performance of a slope with respect to a specified factor of safety. These properties vary both spatially and over time; geotechnical design codes require the use of factored design parameter values to account for possible worst-case conditions. Furthermore, standard geotechnical analyses typically exclude the dynamic hydrological processes of rainfall infiltration and loss of matric suction that often trigger landslides. Slope stability assessment is particularly challenging in developing countries with limited resources for acquiring slope data, meeting conservative design standards and mitigating landslide risk. This study applied a combined slope hydrology and stability model to address these issues for a residual soil slope in the tropics. This paper presents a method for maximising stability information from limited data, disaggregating the effects of three different design parameter sets and factor of safety threshold choices, as well as diagnosing the dominant geotechnical and dynamic landslide-triggering factors. This modelling approach provides a more transparent basis for sustainable slope-management decisions.

Notation

c'	cohesion intercept (kPa)
d	vertical depth of weathering layer (m)
F	factor of safety
K	hydraulic conductivity (m/s)
K_{sat}	saturated hydraulic conductivity (m/s)
γ_d	dry unit weight (kN/m ³)
γ_{sat}	saturated unit weight (kN/m ³)
γ'_ϕ	partial factor of safety for the angle of friction
θ_s	saturated moisture content (m ³ /m ³)
σ'	effective stress (kPa)
τ	shear strength (kPa)
ϕ'	angle of friction (degrees)
ϕ'_d	design angle of friction (degrees)
ϕ'_k	characteristic angle of friction (degrees)
ψ	matric suction (m)

1. Introduction

1.1 Disaster risk reduction: engineering sustainability context

Developing countries are disproportionately affected by natural hazards and their consequences and the process of development can generate new disaster risk (UNDP, 2004). In 2015, both the UN Sendai Framework for Disaster Risk Reduction and the Sustainable Development Goals (SDGs) reiterated the need for increased resilience to disaster risk as an integral part of sustainable development (UN, 2015a, 2015b). Two of the SDGs directly relate

to the built environment and call for more resilient, sustainable and safe infrastructure, housing and cities. A particular challenge for civil engineers, planners and disaster risk-reduction practitioners in developing countries is that the pressure for new housing and infrastructure often means that informal construction of housing outpaces regulation (Satterthwaite *et al.*, 2007) and even relatively high-volume roads may be constructed with limited resources for slope design or stabilisation (Hearn and Hunt, 2011). This brings the interactions between development and landslide risk – and, hence, the social, economic and environmental sustainability of related engineering practice – into sharp focus.

Despite these challenges, engineers may still be able to make informed decisions about hazard and risk reduction. Examples include a community-based urban landslide hazard reduction framework that combines local knowledge with slope hydrology and stability modelling to identify surface water drainage solutions (Anderson and Holcombe, 2006) and ongoing international development funding programmes that have developed guidelines for low-cost road construction in mountainous regions (e.g. TRL, 1997).

Professional engineering institutions have formally established the principles of sustainable engineering in their policies. The Institution of Civil Engineers' sustainable development charter (ICE, 2003: p. 73) calls for members to develop '... solutions which strike an informed balance in terms of cost, benefits, sustainability and acceptability ...' in order to support economic growth, protect the

environment and improve quality of life. The American Society of Civil Engineers has established principles of sustainability in Policy 418 (ASCE, 2016), which call for civil engineers to commit to ‘Do the Right Project ... Do the Project Right ... Perform Life Cycle Assessment from Planning to Reuse ... Use Resources Wisely ... Plan for Resiliency ... and Validate Application of Principles’.

Against this background, a slope stability assessment method for a typical data- and resource-limited scenario in the humid tropics is presented. This paper demonstrates how uncertainties related to parameter value choice and dynamic subsurface hydrology can be investigated so that engineers can make more informed and sustainable design choices.

1.2 Rainfall-triggered landslides

Rainfall-triggered landslides are one of the main natural hazards exacerbated by development activities such as urbanisation and road construction. The occurrence and impacts of such landslides are increasing globally, particularly in rapidly developing countries in the humid tropics (Alcántara-Ayala, 2002; Smyth and Royle, 2000). The humid tropical climatic region is defined according to the Köppen–Geiger climate classification (e.g. Peel *et al.*, 2007). Widespread landslide events, such as those caused by Hurricane Mitch in 1998 in Central America, can damage lifeline road networks and reverse economic growth (Cepal, 1999), whereas ‘everyday disasters’, associated with small-scale frequent slope failures, tend to affect the most socio-economically vulnerable communities, leading to risk accumulation and suppressed economic development (Bull-Kamanga *et al.*, 2003). Landslide hazard assessment from regional to site-specific scales is the first step in identifying, reducing and managing this disaster risk. These assessments require information on slope topographic, geotechnical and hydrological properties – each of which can be highly spatially and temporally variable and therefore inherently uncertain. Further uncertainties can arise from limitations in the analytical or modelling approaches used to assess slope stability. In engineering terms, such uncertainties may be regarded as part of the risk associated with a slope. However, for clarity, this paper separates the term ‘uncertainty’ from the UN International Strategy for Disaster Reduction terminology for disaster risk, where risk is a function of the probability of an event (the hazard) and the consequences of the occurrence. The way in which these uncertainties are dealt with has a bearing on the diagnosis of a slope as safe or unsafe (i.e. the level of hazard and risk), the identification of the processes dominating slope instability, the confidence that can be assigned to that diagnosis, decisions on how to mitigate the risk and the cost-effectiveness of any engineering works. The identification of the driving cause of failure can aid sustainability (particularly in low-resource areas) by allowing remediation measures to be better targeted.

1.3 Factors of safety

1.3.1 Lumped factors of safety

The standard engineering approach to slope stability assessment is from a performance-based design perspective in which a slope

must exceed a particular factor of safety, F (i.e. non-failure), over its design life. Where high uncertainty about geotechnical conditions coincides with a high potential for damage or loss of life, then higher factors of safety will typically be required (Hearn *et al.*, 2011). Hencher (2012: p. 280) suggests that where the consequences of failure are likely to be high, using $F > 1.4$ is ‘an acceptable number to guard against failure’. Terzaghi and Peck (1948: article 51) suggest a range of 1.3–1.5. Uncertainties may arise from variable slope properties, potential invalid assumptions (e.g. regarding failure mechanisms), operator bias and unknown human factors.

These parameter uncertainties are hidden from end users as the lumped factor of safety is used to account for the variation of many different sources of uncertainty. Limit-state design seeks to rectify this to some degree by splitting the total F value into components – for example, in Eurocode 7 (CEN, 2004), factors may be applied to actions, material properties and resistances. Factored model parameters therefore aim to represent potential worst-case conditions, before being input into a stability analysis model (cf. Bolton, 1989; McMahon, 1985; Simpson *et al.*, 1981).

1.3.2 Reliability-inspired geotechnical engineering

Some geotechnical uncertainties can be more explicitly accounted for by using reliability methods – if sufficient data are available and the focus is on either parameter uncertainty or performance data (Vardanega and Bolton, 2016). While traditional deterministic analyses allow uncertainties to be lumped, in the case of reliability methods, information is required about the mean and the tails of the parameter distributions. However, in both cases, the analyses often assume hydrostatic conditions. This assumption means that the dynamic hydrological processes by which rainfall can trigger landslides are not always accounted for.

1.3.3 Hillslope hydrology methods

Hillslope hydrologists and landslide hazard scientists have developed a range of physics-based deterministic and stochastic models for assessing slope stability at different spatial scales (Dai *et al.*, 2002). In these modelling approaches, the geotechnical parameters are not usually factored, and a factor of safety of $F < 1$ is taken to indicate failure. Dynamic soil hydrological processes (such as rainfall infiltration and subsurface flows) and the resulting moisture content and pore pressures are often represented over time to determine the probability of failure for a given rainfall event.

1.3.4 Landslide mechanisms and mitigation measures

It must also be noted that landslide mechanisms can vary depending on the material involved (from fine and coarse soils to rock – see Varnes (1978)) and factors such as the degree of saturation and the land cover of a particular slope. Landslides in soils can be broadly separated into rotational mass movements or translational near-surface failures (e.g. Craig, 2004: p. 347).

Rotational movements have a curved failure surface and rotate about an axis perpendicular to the slope. Such failures can be deep-seated where slope materials are deeply weathered and relatively homogeneous. Near-surface failures can include highly mobilised events such as mudflows or lahars. These failures are generally shallow and move down the slope parallel to the surface. Typical landslide mitigation measures may involve reducing the destabilising forces through slope regrading or improved subsurface drainage or increasing the resisting forces by retaining or reinforcing the slope material or loading the toe of the slope (Hearn *et al.*, 2011). In many cases, the likelihood of translational slope failures can be lowered by the presence of vegetation. The reinforcing properties of roots and the interception of rainwater by plants can both contribute significantly to an increase in slope stability (e.g. Hubble *et al.*, 2013; Wu *et al.*, 1979).

Sánchez-Silva and Rosowsky (2008: p. 196) argue that '[b]y adopting safety standards specified in codes of practice in developed countries, less developed countries are spending more than what can be economically justified based on competing objectives of economic development and safety.' Risk management in engineering works should not simply be a function of 'cost overruns, safety and reliability' but should be expanded to include environmental sustainability considerations (MacAskill and Guthrie, 2013: p. 188).

1.4 Research questions

In this research, a combined slope hydrology and stability model was used to investigate the implications on sustainable landslide hazard assessment and mitigation of using performance-based engineering approaches. The following questions are considered with respect to the predicted stability of a study site and the impact on potential slope design and hazard mitigation decisions. (a) What is the influence of different choices of design parameter values and safety factor thresholds? (b) What are the implications of assuming static against dynamic hydrology? This paper focuses on a specific site to demonstrate a methodology that provides greater transparency in dealing with uncertain and dynamic slope properties and processes. It assesses how sustainable slope stability standards could be developed for sustainable geotechnical design in data- and resource-limited locations. This assessment methodology is designed to assist engineers working in developing countries where sophisticated laboratory characterisation of the local soils is rarely (if ever) able to be carried out.

2. Model parameters

2.1 Mechanical properties (soil)

The inherent stability of a soil-mantled slope is a function of preparatory factors such as the geometry of the slope (i.e. its height and angle), the mechanical properties of the soil and the groundwater conditions. The limit equilibrium method for stability analysis is widely used to determine the slope factor of

safety, F , based on these slope properties, for a potential failure surface within the soil mass. The stabilising moment or stress is a function of the shear strength of the slope material (τ : kPa), which is commonly expressed in terms of the Mohr–Coulomb failure criterion (e.g. Powrie, 2014: p. 247)

$$1. \quad \tau = c' + \sigma' \tan(\phi')$$

where σ' is the effective stress (kPa), ϕ' is the effective friction angle (degrees) and c' is the cohesion intercept (kPa). However, there is uncertainty associated with the values of the cohesion intercept and friction angle parameters used in Equation 1. The natural structure and fabric of the soil can significantly influence c' and, in particular, ϕ' (e.g. Wesley, 1990).

To design against long-term failure of slopes, it may not be advisable to rely on the presence of cohesion in the soil or on the peak friction angle (to guard against possible progressive failure) (see the centrifuge modelling results reported by Take and Bolton (2002, 2011) and the review of various numerical modelling approaches reported by Rouainia *et al.* (2009)).

2.2 Hydrological properties

The conventional limit equilibrium methods used in geotechnical engineering practice do not always account for changes in subsurface hydrology due to rainfall (e.g. Fan and Chang, 2015). In unsaturated soils, the presence of negative pore water pressure creates matric suction in the soil voids, which increases the effective stress (Terzaghi and Peck, 1948: article 12). This in turn increases the shear strength, hence increasing the slope stability. However, the infiltration of water into a slope during a rainfall event leads to a reduction in the matric suction in the soil mass. Such hydrological variations can significantly affect the stability of a slope over time and the majority of landslides in the humid tropics are triggered by the high-intensity or high-duration rainfall that is characteristic to these regions (e.g. Guidicini and Iwasa, 1977; Larsen and Simon, 1993; Larsen and Torres-Sánchez, 1998; Lumb, 1975). Studies of tropical residual soils have shown that pore pressures respond rapidly to rainfall and that the occurrence of multiple shallow landslides correlates with 24 h rainfall totals (Brand *et al.*, 1984; Lambe, 1996). Thus, conventional static stability analyses can result in such suction-controlled slopes being diagnosed as safe when they may in fact be prone to failure as a result of subsequent rainfall infiltration (Anderson, 1983; GCO, 1982).

Various physics-based models exist that represent dynamic slope hydrological processes over different timescales and at regional, catchment and site-specific spatial scales (e.g. Anderson and Howes, 1985; Baum *et al.*, 2010; Iverson, 2000; Michaelides and Wilson, 2007; Montgomery and Dietrich, 1994). In these dynamic models, changes in matric suction are directly related to the hydraulic conductivity (K : m/s) of the soil. In unsaturated conditions, the hydraulic conductivity is lower than that in

saturated conditions due to the presence of air bubbles in the soil voids. Therefore, unsaturated hydraulic conductivities and pore water pressures must be iteratively calculated over time and throughout the soil mass based on the soil moisture characteristic (or suction–moisture) curve and groundwater seepage conditions. The inclusion of these dynamic hydrological processes in slope stability analyses can provide important information for geotechnical engineers by capturing potentially critical drops in effective shear strength as pore water pressures vary over time. Without this type of analysis, it is possible for engineers to misidentify the greatest potential source of failure, leading to the possibility of specifying mitigation measures that are less effective than expected.

Figure 1. Typical residual soil slopes in the study site area – Castries, Saint Lucia, Eastern Caribbean (photograph: E. A. Holcombe, 2011)

3. Model and study site

The combined hydrology and stability model (Chasm) was used to investigate the different effects of soil parameter values, safety factor choices and slope stability process representation (i.e. inclusion of dynamic hydrology) on the determination of whether or not a slope is safe. Modelling was carried out for a specific study site in Saint Lucia, Eastern Caribbean, for which the stability response to various rainfall events is known, as it has been previously studied (Almeida *et al.*, 2017; Holcombe *et al.*, 2012, 2016). Typical weathered residual soil slopes, similar to those studied, are shown in Figure 1.

3.1 The combined hydrology and stability model

Chasm is a physically based, hydrologically dynamic model that simulates the effects of rainfall on the stability of a slope through the analysis of changes in pore pressure and the implementation of limit equilibrium method stability analysis. Input files define the slope cross-sectional geometry, hourly rainfall conditions for the duration of the simulation, the boundaries for an automated slip surface search and the soil properties. The full set of model equations and an extended explanation of the methodology can be found in the study by Wilkinson *et al.* (2002). An account of the original development of the model is given by Anderson and Howes (1985).

In Chasm, slope cross-sections are represented as two-dimensional meshes of $1\text{ m} \times 1\text{ m}$ cells, each with a specified material type defined by its mechanical and hydraulic properties. This allows for the creation of weathering and layering patterns in the modelled slope. Figure 2 illustrates a typical slope cross-section in Chasm and indicates the simulation parameters required. The initial groundwater conditions for a simulation are set by specifying an estimated water table location. A 7 d period with zero rainfall is typically implemented at the start of the simulation to allow these initial groundwater conditions to

Figure 2. Representation of Chasm cross-section and parameters

equilibrate and reach a steady state. The hydrological system of the slope is modelled by a forward explicit finite-difference scheme, which updates the moisture content and pore water pressure at the end of each time step (typically 10–60 s). Chasm simulates both saturated and unsaturated flow regimes using Darcy's law (Darcy, 1856) and Richards' equation (Richards, 1931), respectively. For unsaturated cells, the hydraulic conductivity is updated each time step by using the Millington–Quirk procedure (Millington and Quirk, 1959) and the soil moisture characteristic curves defined for each material type. Rainfall infiltration into the modelled slope is determined by the hydraulic conductivity of the top cell of each column. Rainfall that does not infiltrate the slope is assumed lost as run-off and removed from the calculations.

The stability assessment is performed at the end of each hour of the simulation and incorporates the pore water pressures calculated by the hydrological model component. In this study, Bishop's circular limit equilibrium method (Bishop, 1955) is used. This is an appropriate assumption for representing the rotational failures that are commonly observed in tropical residual soils. A user-defined slip search grid provides the centres of rotation for an automated search algorithm in which the failure circle with the minimum F is identified for that simulation hour (Figure 2).

Chasm has been used previously to predict the critical conditions of specific slopes (e.g. Thiebes *et al.*, 2014), the impact of vegetation (Collison *et al.*, 1995) and urbanisation (Holcombe *et al.*, 2016) on the stability of slopes and to develop design charts for cut slopes in the humid tropics (e.g. Anderson and Lloyd, 1991; Anderson *et al.*, 1997). It is parsimonious with respect to data availability in resource-limited environments and has been shown to classify correctly 77% of failed slopes and 68% of stable slopes in back-analysis of observed landslide-triggering rainfall events in Hong Kong (Anderson, 1990).

3.2 The study site

The site selected for this analysis is the lower slope of an urbanised hillside in Castries, Saint Lucia. This slope section has an overall angle of approximately 28° and is mantled with an average of 4 m of residual soil (weathering grades V–VI using the Geotechnical Engineering Office (Geo, 1988) classification system) overlying a similar depth of decomposed bedrock (weathering grades III–IV) that overlies volcanic bedrock.

In 2012, a study of this site was undertaken as part of surface-water drainage works to improve slope stability (Holcombe *et al.*, 2012). At the time, estimates of soil strength properties were made based on a combination of direct shear tests of similar local soils (Holcombe, 2006) and comparison with volcanic residual soil properties in Hong Kong (GCO, 1982). The Geotechnical Control Office (GCO, 1982) information was based on numerous triaxial and permeameter tests from which the resulting shear strength and hydraulic conductivity mean values and ranges were stated with respect to weathering grade. Weathering depth

estimations were based on local expert knowledge of the soils and geology of the area and interviews with community residents who had carried out excavations while constructing their houses.

4. Design of simulations

Rainfall was simulated in the model after a 7 d long dry 'equilibrating period'. Soil properties were classified in accordance with the Hong Kong weathering grades (GCO, 1982) due to their similarity to the undisturbed, tropical, volcanic soil of the modelled site. The ranges of parameter values that informed the simulations during this study are listed in the Appendix. The values of c' and ϕ' assigned to weathering grades I and II preclude failures in these layers (the values are on the upper end of the values given by Hoek and Brown (1997)), thus limiting the failure surfaces to the upper two strata, matching qualitative field observations of similar locations in the region. The general arrangement of the weathering grades as used in this study is shown in Figure 3. In this study, $F = 1.4$ was adopted as a typical conservative 'engineering safety limit' for an urban location. The aim of this section is to identify the driving failure process in the modelled slope.

4.1 Design cases (mechanical properties)

In this section, the effect of different design parameter values on whether the system passes or exceeds different safety thresholds is determined (research aim (a)). Characteristic values ('cautious estimates') are required when performing geotechnical design tasks under the auspices of Eurocode 7 (CEN, 2004). In this study, a set of characteristic values based on collected data from the literature will be determined as the base design case (see the Appendix).

- Case A represents the modelled slope behaviour when 'characteristic parameters' are used.
- Case B models the removal of vegetation that commonly occurs due to urbanisation processes. In this case, the value of ϕ' would remain unchanged from that of case A, but the loss of tree roots and grasses would result in a loss of cohesion in the upper strata. Also, a slight reduction of c' that is assigned to the second layer allows for the very real scepticism with which many geotechnical engineers view the cohesion parameter, which is obtained by a linear curve fit to a curved failure surface (e.g. Parry, 2004: p. 57; Powrie, 2014: pp. 246–247; Terzaghi and Peck, 1948: pp. 95–97).
- Case C applies the typical Eurocode 7 (CEN, 2004) partial factor of safety to the friction angle (see Equation 2: based on CEN (2004) clause 2.4.6.2)

$$2. \quad \tan(\phi'_d) = \frac{\tan(\phi'_k)}{\gamma'_\phi}$$

where ϕ'_d is the design angle of friction (degrees), ϕ'_k the characteristic angle of friction (degrees) and γ'_ϕ is the partial factor of safety for the angle of friction, set to 1.25, in accordance with Eurocode 7 (Bond and Harris, 2008: p. 183; CEN, 2004: Table A.2, p. 129). This design case could be

Figure 3. Hydraulic conductivity values and model design

described as a ‘worst credible’ design scenario (e.g. Bolton, 1989; Simpson *et al.*, 1981).

The soil parameters used for cases A, B and C are shown in Table 1.

4.2 Modelled hydrological variations

The implications of imposing different rainfall patterns’ hydrological and K_{sat} values on the calculated slope stability (F) (research aim (b)) were then assessed for the modelled slope. The hydraulic conductivity profile in the slope consisted of a ‘weathered’ layer and a ‘bedrock’ layer, between which the saturated hydraulic conductivity (K_{sat}) value decreases by an order of magnitude (as in GCO (1982)). The variation in the spatial pattern of K_{sat} was simulated by separating the weathered layer into three blocks, each measuring 43 m horizontally, named the top, middle and toe blocks. Each of the blocks would in turn become the variable block (Figure 3). The 24 h storm intensities used in this study (Table 2) were based on a report

by Klohn-Crippen (1995). Klohn-Crippen analysed the return period of the 1994 tropical storm Debbie and, by extension, the probable maximum precipitation the dam in question would likely experience. This study applied design storms of 1–24 h and return periods of 10, 20, 50, 100, 200 and 500 years to the slope, the rainfall values for which were taken from the intensity–duration–frequency graph in Klohn-Crippen’s report. As discussed later in this paper, scenarios where a ‘double event’ occurs were also modelled.

5. Results

For the modelled slope shown in Figure 3, the mechanical soil properties of case A were used when investigating the following phenomena, as case A represents the characteristic soil parameters for the model slope in question without the application of partial factors to the mechanical soil properties

- critical depths of the weathered material (grades III–VI)
- location of the critical hydraulic conductivity zone
- effect of rainfall intensity
- effect of rainfall duration
- effect of antecedent rainfall.

Table 1. Geotechnical design cases

	Symbol	Case A: characteristic parameters	Case B: design parameters	Case C: factored parameters
Grades V and VI	ϕ' : degrees γ_{dr} , γ_{sat} : kN/m ³ c' : kPa θ_s : m/s	25 17, 19 8 0.413	25 17, 19 0 0.413	20.46 17, 19 0 0.413
Grades III and IV	ϕ' : degrees γ_{dr} , γ_{sat} : kN/m ³ c' : kPa θ_s : m/s	45 21, 23 45 0.413	45 21, 23 30 0.413	38.66 21, 23 0 0.413
Grades I and II	ϕ' : degrees γ_{dr} , γ_{sat} : kN/m ³ c' : kPa θ_s : m/s	50 25, 27 30 0.413	50 25, 27 30 0.413	43.63 25, 27 0 0.413

Table 2. Storm design (24 h)

Return period	Depth: mm	Intensity: mm/h
1 in 10 years	216	9.00
1 in 20 years	252	10.50
1 in 50 years	288	12.00
1 in 100 years	336	14.00
1 in 200 years	360	15.00
1 in 500 years	432	18.00
Hurricane Tomas >500 years ^a	533	22.21

^a Hurricane Tomas (2010) delivered over 500 mm of rainfall in 24 h in Saint Lucia (see Anderson and Holcombe, 2013); according to Klohn-Crippen (1995) this is greater than a 500-year return period

5.1 The effect of weathering depths and saturated hydraulic conductivity

Four depths of each of the top two material strata (grades III and IV and V and VI) were tested to investigate the effect of uncertainty in these stratum depths on the factor of safety. The depths (d) tested were 2, 4, 6 and 8 m for each layer, such that the total weathered material depth (grades III–VI) became 4, 8, 12 and 16 m deep, respectively. The results showed that the factor of safety reduced significantly between $d = 2$ m and $d = 4$ m, from an average value of 3.2 to an average of 1.8. A drop in the F value was also observed as d was increased to 6 m and then to 8 m; however, this drop was much less pronounced. This result makes sense in that the slope will become more stable with a lower fraction of the slope occupied by the weaker soils associated with a higher weathering grade. The range of the computed values of F throughout the simulation is also interesting. Between $d = 2$ m and $d = 4$ m, the range increased, but for values of d above 4 m, the range then decreased. For the set of simulations where the bedrock $K_{\text{sat}} = 1 \times 10^{-7}$ m/s and weathered material $K_{\text{sat}} = 1 \times 10^{-6}$ m/s, the range changed from 0.03 at $d = 2$ m to 0.2 at $d = 4$ m and back to 0.03 at $d = 8$ m. The remaining simulations discussed in this study all use the slope section where $d = 4$ m.

5.2 Critical hydraulic conductivity zone

Once the depth of the layers of the ‘weathered material’ had been chosen for the simulations, the effect of changing the magnitude and spatial pattern of the hydraulic conductivity was tested. The critical slip surface was found to be the same for all weathered slopes where $d = 4$ m, regardless of where the variable block was located, the magnitude of the hydraulic conductivity or the

applied rainfall event. The radius of the slip circle was calculated to be 48 m with a centre of (116, 67) (see Figure 3 for the location of the origin). By studying the effect of this slip surface location on the factor of safety, it is clear that any variation in the hydraulic conductivity will be most influential when it occurs at the toe of the slope, as illustrated in Figures 4–6. Additionally, it is worth noting that the slopes with a higher hydraulic conductivity are more sensitive to a rainfall event, because high infiltration rates result in a rapid increase in soil moisture and, consequently, a reduction in stabilising matric suction. The variation due to the location of the more permeable zone does not have a great influence on the stability of this slope configuration, but it must be noted that the effect may be more pronounced for other slopes.

5.3 Effect of rainfall intensity

The first seven 24 h rainfall events of increasing intensity (Table 2) were applied to the slope model. The results from this set are illustrated for a slope with bedrock $K_{\text{sat}} = 1 \times 10^{-6}$ m/s and weathered material $K_{\text{sat}} = 1 \times 10^{-5}$ m/s in Figure 7. Other bedrock K_{sat} values were also tested, and the effect on the results was minimal. Up to the point at which the rainfall event begins ($t = 168$ h), there is no difference in the rate at which the F value increases, as all seven simulations are identical in approaching steady-state seepage during this phase. During the rainfall events, the rate of and magnitude of the decrease in F values increase with the intensity of the storms. This is due to the increase in the volume of water infiltrating into the slope as storm intensity increases (controlled by hydraulic conductivity) and the subsequent reduction in the matric suction at higher rates for higher rainfall intensities.

Figure 4. Factor of safety variation for a variable toe block (case A – soil parameters)

Figure 5. Factor of safety variation for a variable middle block (case A – soil parameters)

5.4 Effect of storm duration

Five storm events were simulated (with return periods of 50 years), each of increasing duration, for the three configurations of K_{sat} . Only the results for a slope of bedrock $K_{sat} = 1 \times 10^{-6}$ m/s and weathered material $K_{sat} = 1 \times 10^{-5}$ m/s are included in this study (Figure 8) as this configuration best illustrated the variation in F values. However, a slope with a bedrock $K_{sat} = 1 \times 10^{-7}$ m/s and

weathered material $K_{sat} = 1 \times 10^{-6}$ m/s and another with a bedrock $K_{sat} = 1 \times 10^{-8}$ m/s and weathered material $K_{sat} = 1 \times 10^{-7}$ m/s were also tested. These exhibited the same pattern as that shown, but with less variation in the slope stability response. The values of F stay in the same range as previous simulation results (Figures 4–6). However, as would be expected, the storm intensity controls the rate of change. Also included in Figure 8 is a

Figure 6. Factor of safety variation for a variable top block (case A – soil parameters)

Figure 7. Variation in F value for a variety of 24 h rainfall events of increasing return period (case A – soil parameters)

comparison of the rainfall intensity and the infiltration capacity of the uppermost layer of the slope (i.e. the cells on the slope surface).

5.5 Antecedent rainfall

Having investigated the response of the study site to intense rainfall events, the effects of antecedent rainfall were then considered.

As well as the tropical cyclonic and convective storms characterised by short periods of intense rainfall, slopes in the tropics are subjected to weather systems that deliver longer periods of lower-intensity rainfall (such as low-pressure systems and tropical waves) that may last several days and recur throughout the rainy season (Larsen and Simon, 1993; Lumb, 1975). This is a

Figure 8. Variation in F for a series of 1-in-50-year rainfall events of increasing intensity, with the infiltration capacity of the uppermost layer of the slope compared to the rainfall intensity (case A – soil parameters)

concern for engineers when assessing slope stability over project lifetimes. The effect of these long-duration low-intensity rainfall events on the slope stability was tested by applying a 15 d long cumulative antecedent rainfall of either 100 or 200 mm, followed by a 24 h storm (event 1). This pattern was found by Lumb (1975) to be a reliable predictor of multiple landside events such as those observed during the rainy seasons in such climates. The same 15 d

of rainfall followed by the same storm (event 2) was then again immediately applied to the slope to create a double event. As a control, a series of simulations was also run with no antecedent rainfall. Throughout this set of simulations, the mechanical soil properties of case A were applied to the slope. Figures 9–11 illustrate the effect of double events on a slope with varying antecedent rainfall levels (0, 100 and 200 mm).

Figure 9. Variation in F for 0 mm antecedent rainfall (case A – soil parameters)

Figure 10. Variation in F for 100 mm cumulative antecedent rainfall (case A – soil parameters)

Figure 11. Variation in F for 200 mm cumulative antecedent rainfall (case A – soil parameters)

While the application of antecedent rainfall did reduce the factor of safety somewhat, the maximum change due to this remained less than 0.2. The lowest value of F observed was 1.36 at $t = 942$ h, for the slope of bedrock $K_{sat} = 1 \times 10^{-6}$ m/s and weathered material $K_{sat} = 1 \times 10^{-5}$ m/s, with the 200 mm cumulative antecedent rainfall. While this may be considered too low a factor of safety for engineering design (i.e. less than the aforementioned threshold of $F = 1.4$), it can be agreed that the slope at this point was still far from failure.

5.6 Effect of geotechnical design cases

Examination of Figure 12 shows that use of the soil properties in cases A, B or C gives identical curve shapes, with only the magnitude of the factor of safety changed. The effect of hydrology (as explained previously) is constant throughout and only the hydraulic properties of the soil, once the simulation has started, can influence the changes in F values. This reflects the ability of Chasm to capture dynamic analyses as opposed to classic static ones. It is

Figure 12. Comparison of the F curves for cases A, B and C: two cycles of 15 d antecedent rainfall of 0.28 mm/h (i.e. 100 mm cumulative total) applied, followed by 24 h storms of increasing intensities (the engineering safety limit of $F = 1.4$ is shown, and $F = 1.0$ is taken to sensibly represent the collapse limit)

not until a high-return-period (200 or 500 years or more) event is considered that slopes modelled using case B reach the collapse limit, albeit during event 2. However, at no stage does design case B or C ever yield an F above the generally used engineering safety limit. Case B and the case with zero cohesion are arguably credible design assumptions that a geotechnical engineer may make. Simply using the ‘assumed average’ soil properties, case A along with the dynamic analysis may lead one to believe that this modelled slope is essentially immune to failure.

6. Discussion

6.1 Application of partial factors from codes of practice

The methodology applied to the site in this study is transferable to other slopes. This approach provides a more transparent way for engineers in developing countries to deal with the selection of design parameter values and safety factors and to represent the dynamic hydrological processes that can trigger landslides (particularly in ‘data-poor’ jurisdictions), such that remediation methods may be focused on the most critical factors. Engineers who rely on static analyses for slope stability problems often ignore the dynamic effects of a rainfall event (e.g. Fan and Chang, 2015) and use an estimated (worst-case or average) water table position. The implication of this traditional approach is that a single value of F is assumed to represent the slope at all times. With the initial estimation of a typical water table position, all the slopes in this study begin the simulation with a value of F above 1.0, suggesting that the slope would be stable, if not ‘safe’ – even when case C (worst credible) is used. However, this value changes during the simulations, causing some slopes, which would have been considered stable, to ‘fail’. Furthermore, the application of the factored ϕ' values of case C to case A would make even these slopes close to borderline, and they would almost certainly cross the $F = 1.4$ line when exposed to large storms.

6.2 Appropriate safety standards

This raises an interesting conversation on the safety of not only slopes, but of construction in the developing world in general. The condemnation of this slope according to the safety standards of developed countries is simply not sustainable in less developed countries. Sánchez-Silva and Rosowsky (2008) argue that the level of acceptable risk a country adopts should be related to its level of development: for less developed countries, higher levels of risk are necessary for economic development. This is a sensitive statement as it is linked to societal tolerance of risk, perceptions of safety and the Alarp principle (in which risk should be reduced to ‘as low as reasonably practicable’), which requires a value to be placed on human life at individual and societal levels.

These results are pertinent in a discussion of appropriate design parameters for slope stability in less developed countries. The application of the safety standards of developed countries to this slope would conclude that it was unsafe and would recommend the instigation of stabilising measures. However, this is costly, and these measures are unlikely to be the most efficient or sustainable use of the resources of a less developed country.

6.3 The effect of vegetation on slope stability

The mechanical strength of the soil itself is not the sole indicator of the stability of a slope. In this study, the potential stabilising (or destabilising) properties of plants and trees were ignored, and all the strength of the slope was assumed to originate from the soil. However, this may yield values of F significantly lower than demonstrated in reality for vegetated slopes. A systematic investigation of the stabilising effects of tree roots on Australian hillslopes concluded that the presence of root networks in the soil can increase modelled cohesion by up to 10 kPa (Hubble *et al.*, 2013). Even introducing grass cover to slopes has been found to increase stability (e.g. Glendinning *et al.*, 2009; Holcombe *et al.*, 2016; Hughes *et al.*, 2009). The presence of roots in the slope can improve shear strength by inhibiting soil movement and by providing additional cohesion. It is reasonable for engineers to consider including this additional cohesion when assessing slope stability in general and in developing nations in particular. The inclusion of even small amounts of cohesion where it would traditionally have been disregarded could aid in avoiding unnecessary implementation of stabilisation measures, so long as the stabilising vegetation is preserved. The reduction in infiltration of water into the slope due to vegetation cover was also found to give significant increases in F . Therefore, when assessing the safety of slopes, particularly those in less developed countries, engineers should be aware that the soil properties are not solely responsible for stability. The presence of vegetation cover providing root reinforcement of soils (e.g. Holcombe *et al.*, 2016) may justify the use of small values of c' being utilised in slope assessments.

7. Conclusions

A series of dynamic simulations mapping the change in the factor of safety of a slope before, during and after rainfall events has been presented. The simulations investigated the effects of changes in hydraulic and in mechanical soil properties, namely, K_{sat} , c' and ϕ' , and their interaction. For the studied slope, changes in K_{sat} did not have much influence on the overall value of F (this may be not be so for other slope geometries). Using different assumed soil properties, in particular c' , can have severe consequences for the assessed safety of a slope. For this slope, the elimination of c' in the soil, as is often assumed for design (particularly when no site-specific data are available), can mean that the computed factor of safety drops from well above the minimum allowable value ($F = 1.4$) to just above the borderline of failure ($F = 1.0$). The use of sustainable remediation measures (e.g. increasing vegetation cover (Holcombe *et al.*, 2016)) may be explored as part of the design and assessment process.

Future work should explore the use of more systematic methods for dealing with uncertainty. In developing countries, where resources are limited and engineering solutions are often constrained by the existence of informal construction, it may be wiser to focus engineering efforts on targeted geotechnical and hydrological data acquisition (accepting the initial increased financial outlays associated with this) as part of the nation’s infrastructure-management efforts, rather than attempting to conform to impractical and unsustainable safety standards, or worse by ignoring them completely.

8. Data availability

The data used to plot Figures 4–12 in this study can be downloaded from the University of Bristol – Research Data Repository (Shepherd *et al.*, 2017).

Acknowledgement

CJS was funded by a University of Bristol Interdisciplinary Research Internship in 2015 when this study was initiated.

Appendix

Table 3 shows the ranges of parameter values used in the simulations.

Table 3. Ranges and sources of parameter values used in the simulations (based on data ranges also stated in the online supplement to Holcombe *et al.* (2016))

	Symbol	Reference	Comments	Units	Range	Number of data points	Selected parameter (case A)
Grades V and VI	ϕ'	(a) Dearman <i>et al.</i> (1978) (b) GCO (1982)	(a) Large database collected from the literature by Dearman <i>et al.</i> (1978) for undisturbed granitic and gneissic soil, predominantly triaxial (drained or undrained) and shear tests (b) Triaxial tests on colluvium, decomposed granites and decomposed volcanics	Degrees	20–42	17	25
	γ_d, γ_{sat}	Irfan (1996)	From ranges of dry densities of Hong Kong granites; grade boundaries ‘approximate’; Irfan (1996) found dry density a good means of granite weathering classification in the UK	kN/m ³	12–18	Not stated	17, 19
	c'	(a) Simon <i>et al.</i> (1990) (b) GCO (1982)	(a) Range of values for diorite and volcanoclastic soils from ten representative slopes in Puerto Rico; values given as quantile measures, the lowest value being a 25th quantile and the highest a 75th quantile; data from borehole shear tests (b) Triaxial tests on colluvium, decomposed granites and decomposed volcanics	kN/m ²	1–14	(a) 6	8
	θ_s	White <i>et al.</i> (1998)	Regolith samples from Puerto Rico from the top of Guaba ridge, elevation: 680 m; measurements made on coarse samples by measuring the weight difference after heating	m ³ /m ³	0.35–0.46	12	0.413
Grades III and IV	ϕ'	Rahardjo <i>et al.</i> (2004)	Sample of residual soils from the Bukit Timah granitic and Jurong sedimentary formations in Singapore, from ‘boreholes using a Mazier sampler’; evaluated using the Mohr–Coulomb failure envelope (found to be linear)	Degrees	42–51	4	45
	γ_d, γ_{sat}	Irfan (1996)	From ranges of dry densities of Hong Kong granites; grade boundaries ‘approximate’. Irfan (1996) found dry density a good means of granite weathering classification in the UK	kN/m ³	17–25	Not stated	21, 23
	c'	Rahardjo <i>et al.</i> (2004)	Samples of residual soils from the Bukit Timah granitic and Jurong sedimentary formations in Singapore, from ‘boreholes using a Mazier sampler’; evaluated using the Mohr–Coulomb failure envelope (found to be linear)	kN/m ²	35–55	4	45
	θ_s	White <i>et al.</i> (1998)	Regolith samples from Puerto Rico from top of Guaba ridge, elevation: 680 m; measurements made on core samples by measuring the weight difference after heating	m ³ /m ³	0.35–0.46	12	0.413
Grades I and II	ϕ'	GCO (1982)	High values chosen for modelling purposes to reflect observations that landslides most frequently occur in weathered and soil layers, not bedrock	Degrees	—	—	50
	γ_d, γ_{sat}	Irfan (1996)	From ranges of dry densities of Hong Kong granites; grade boundaries ‘approximate’; Irfan (1996) found dry density a good means of granite weathering classification in the UK	kN/m ³	25–26	Not stated	25, 27
	c'	GCO (1982)	High values chosen for modelling purposes to reflect observations that landslides most frequently occur in weathered and soil layers, not bedrock	kN/m ²	—	—	30
	θ_s	White <i>et al.</i> (1998)	Regolith samples from Puerto Rico from top of Guaba ridge, elevation 680 m; measurements made on core samples by measuring the weight difference after heating	m ³ /m ³	0.35–0.46	12	0.413

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