LESSONS LEARNT FROM FIELD TESTS IN SOME POTENTIALLY UNSTABLE SLOPES IN SWITZERLAND

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abstract

Rain-induced slope instability is a significant natural hazard in Switzerland, Slovenia and elsewhere in Europe. This contribution was prepared especially for the 12th Šuklje Symposium, and recognises that landslides occur both in mountain regions as well as in lowland regions during and following extreme-rainfall conditions. The Institute (and Professorship) for Geotechnical Engineering at the Swiss Federal Institute of Technology (ETH Zürich) has been engaged over several years in projects concerned with the characterisation, monitoring and modelling behaviour of slopes in mainly granular porous media across the full range of altitudes in Switzerland. A link is made to the doyen of the Šuklje day and then three case histories are presented and discussed to demonstrate the principal reactions to seasonal rainfall. A small slip was released in two of these cases and the "triggering" factors have been investigated and are discussed in this contribution. It transpires that the mode of inslope drainage influences the way in which the ground saturates and hence the volume of the potentially unstable ground. Simple stability analyses using limit equilibrium and soil parameters that have been amended to account for unsaturated soil behaviour were found to function well for slopes in largely granular media.

keywords

rain-induced landslides, slope stability, case histories, monitoring, characterisation, modelling

1 PROF. DR. LUJO ŠUKLJE: ON LANDSLIDES AND HIS CONTRIBUTIONS

Academician Professor Dr Lujo Šuklje (Fig. 1; [1]) was the pioneer of Slovenian Soil Mechanics. He was appointed to a full Professorship at the University of Ljubljana in the year of the first author's birth and died in the year that she came to ETH Zurich at the same professorial grade. He was reputed to have been a strict, yet caring teacher, who left his mark on the development of conceptual soil mechanics, particularly on the rheological behaviour of soils [2], including creep, anisotropy,

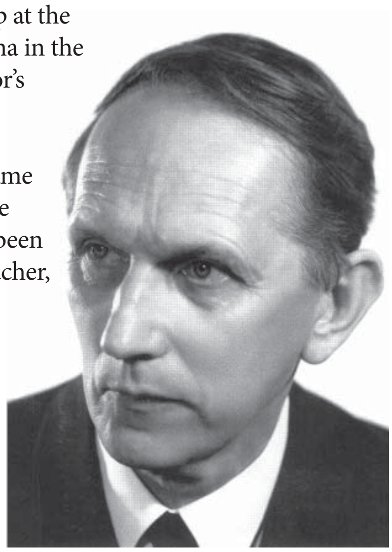


Figure 1. Academician Professor Dr. Lujo Šuklje [1].

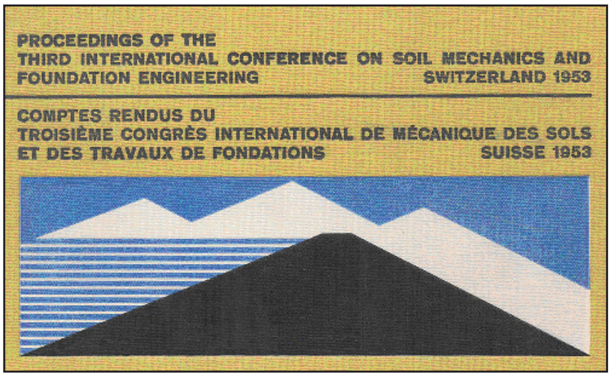


Figure 2. Proceedings of the third International Conference of Soil Mechanics and Foundation Engineering, Switzerland 1953 [3].

viscoplasticity and consolidation. Slope stability is treated too, in his highly regarded text book 'Rheological Aspects of Soil Mechanics' [2], with 4 chapters in Section 5. He insists that the laboratory testing of specimens should be conducted in the direct shear and the triaxial apparatuses following consolidation. This theme is taken up later in this paper, which is dedicated to celebrating 101 years after his birth in Jelsa, Hvar Island, in September 1910.

Interestingly, records of the discussion on Session 4 concerning 'the foundations of buildings and dams, bearing capacity, settlement observations, regional subsidences' held at the 3rd International Conference of Soil Mechanics and Foundation Engineering (Fig. 2; [4a]) that was held in Zürich in 1953, report written discussions from Mr L. Šuklje, which were presented in French. In the first, he took issue with a Mr S.J. Button concerning his treatment of foundation stability with a ϕ=0 assumption, when shear resistance changes with depth. He mentioned that the position of the most unfavourable slip zones could also include thin bands of lower shear resistance and were also affected by the presence of rigid or flexible foundations. Interestingly, both themes were investigated in recent doctoral theses in the first author's group [5,6]. He also makes some fascinating comments on using either a grease or slimy mud to represent soft soils subjected to rigid foundation loads in physical models [7], initially at the Building and Public Works Laboratory in Paris and later at the Soil Mechanics Laboratory at the ETS of Ljubljana. These resulted in satisfactory analytical solutions by applying circular failure surfaces, again an area of interest to the authors.

The second written discussion [4a] presented tests demonstrating the effect of secondary settlement and a means for calculating the duration thereof, by using laws of similarity to apply test results to full-scale experiments. Professor Šuklje considered the micromechanics by explaining that the grains slid into more stable positions during this process and he either agreed or took issue with the General Reporter (Mr M. Buisson) and the ISSMFE-Harvard-MIT hierarchy (Profs. Terzaghi, Casagrande, Taylor).

The last discussion [4b] was an oral one in the session on 'stability and deformations of slopes and earth dams, research on pore-pressure measurements, groundwater problems'. He again referred to the adoption of a ϕ=0 approach by a former doctoral researcher at ETH Zürich, who was then Director at the NGI in Norway, Dr Laurits Bjerrum. He was probably thinking of the Zalesina landslide, which is discussed below, and pointed out that this approach could not be used in Yugoslavia since most of the landslides occurred in heterogeneous clay soils and that effective stresses should be used at all times. It was also of great importance to include the influence of water pressures due to standing and flowing water. These last two are central tenets, which will form the backbone to this paper.

'En général, les expériences acquises montrent que l'eau souterraine peut en réalité réduire à rien la stabilité des talus, non seulement par la dissolution des forces de cohésion (adhésion), mais aussi par l'effet mécanique de la sous-pression ou de la pression du courant....... Le calcule montrerait une sécurité qui n'existe pas....... Il faudra, sous le même point de vue, reconsidérer également l'application de la méthode ϕ=0.' [4b].

Fifth Technical Session: Influence of Ground-Water on Slope Stability
Friday morning, 24 September, 1954

Session 5/1

LANDSLIDE ZALESINA
by
ERVIN NONVEILLER and LUJO ŠUKLJE

Figure 3. Extract from the Fifth Technical Session of the Conference on Stability of Earth Slopes hosted in Stockholm, with photograph of the Zalesina landslide [8].

Professor Šuklje's contribution to the understanding of the cause of landslides, and his proposals for remediation, bear further mention and consideration in the context of this invited lecture. In conjunction with his former student Ervin Nonveiller (Fig. 3; [8]), he contributed to the Fifth Technical Session of the Conference on Stability of Earth Slopes hosted in Stockholm. The paper was entitled 'Influence of Groundwater on Slope Stability' and concerned the Zalesina Landslide, then within Yugoslavia and now in the Republic of Croatia, due south of Obcima Kostel in southern Slovenia.

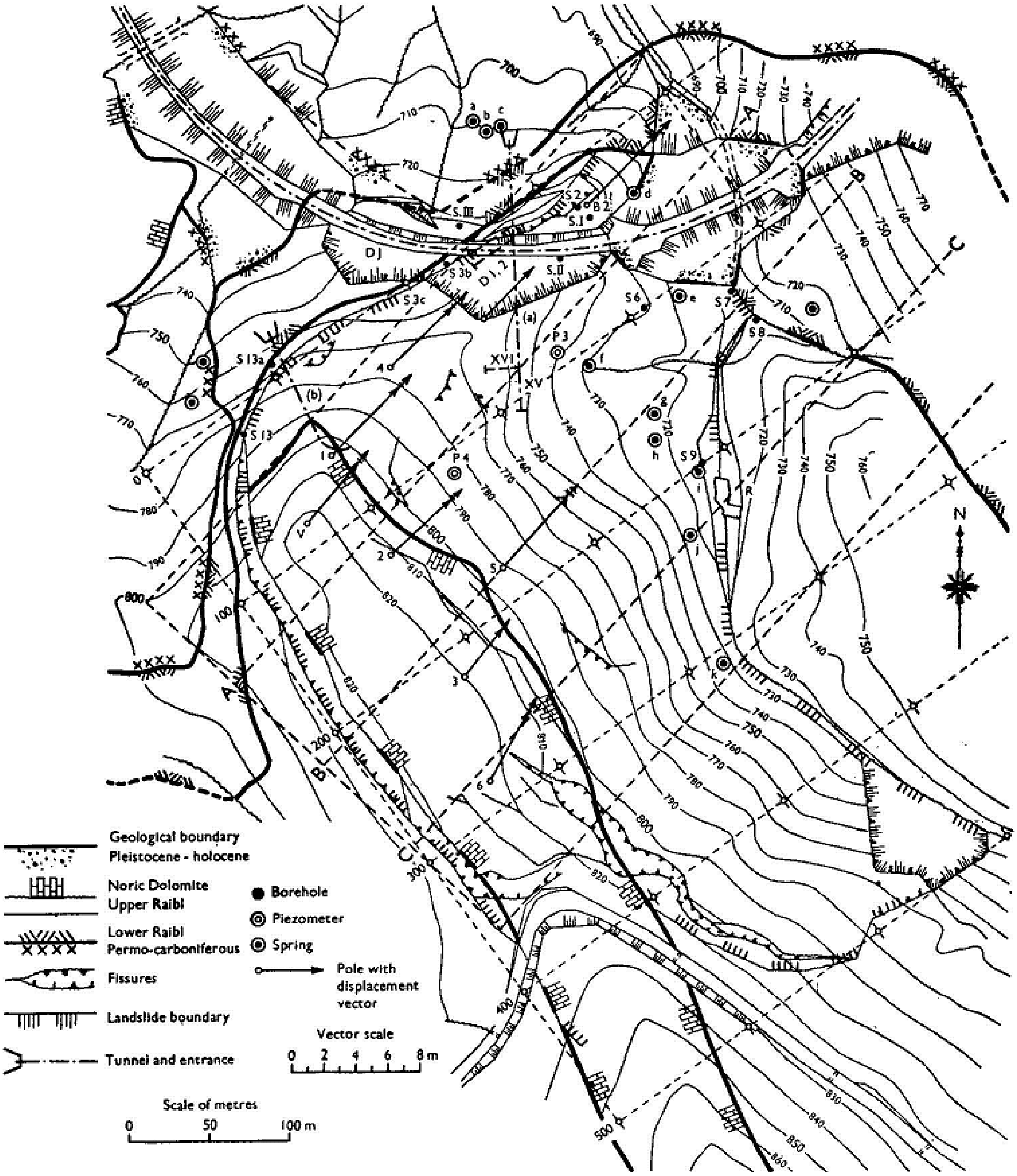


Figure 4. Mapping the Zalesina Landslide, Croatia, (Republika Hrvatska) [8].

A slope above a railway line demonstrated 'a spontaneous natural phenomenon' (Fig. 4; [8]), with a maximum length of the front being 500 m, and an average of 300 m. This was caused by a combination of 'tectonic processes (dinaric and transversal faults), long-term erosive creek action and internal erosion', while newly developed springs emerging from the slope played a major role as well, with one delivering 200 litres/min [8]. After a drought year in 1950, in which a continuous crack was observed running along a fault line over the railway line down to the creek, 2 m of net downslope movement was accompanied by an average of 6-mm/day rainfall over 6 months at the end of 1951 into early 1952. This comparison between the rainfall and the movement also showed that a cumulative total of more than 1000 mm of rain fell before the masses came to a more stable position [8].

Initiated at the contact between two strata between the Upper Raibl formation (siliceous or dolomitic sandstone with various forms of schisty shale, slate and limestone) and the overlying Noric Dolomite, cracks developed mainly perpendicular to the main direction of movement and played a crucial role in enhancing macro-permeability. A combination of large, deeper slides, older slides and secondary, comparatively shallow landslides in the weathered upper layers and steep slopes were probably caused by uplift and seepage pressures, through exfiltration from the bedrock, which could be treated effectively by remedial drainage. Some exploration tunnels were excavated to reveal the extent of the location of the shear zone.

The authors concluded a number of points that remain wholly relevant to today and were, at this time, still somewhat visionary.

1. An effective stress analysis must be conducted based on parameters derived from consolidated specimens in a ring-shear apparatus and from consolidated, undrained (quick and slow) triaxial tests.
2. Friction is more important than cohesion in tectonically disturbed beds, which govern the sliding conditions; a statement that would be "music to the ears" of one of the first author's mentors, Professor Andrew Schofield FRS FREng [9].
3. Uplift and seepage pressures play a major role in slopes during heavy rainfall and must be accounted for in the stability calculations.
4. The sliding may occur over a broad zone, here approximately 7 m, but the whole mass slides mainly as a solid body.
5. Monitoring of the groundwater level should be achieved using piezometers and geoelectrics, the latter being a method that is now finding considerable favour.
6. The method of potential slip surfaces can still be applied (assumed here to be limit equilibrium calculations), notwithstanding the approximations made, likewise an approach dwelt on briefly in this contribution.
7. Partial natural stabilisation was achieved, which could be improved by:
   a. removal of mass from the top to the foot of the slide;
   b. draining the sliding masses with a drainage system;
   c. a combination of a) and b).

SURFICIAL RAINFALL INDUCED LANDSLIDES

Surficial rainfall-induced instabilities triggered in unsaturated slopes have been reported over many decades (e.g., [10-15]). Systematic studies of the most important triggering effects have been pursued as Pan-European multidisciplinary projects in recent years (e.g., MOUNTAIN RISK, TRAMM, SAFELAND).

Lateltin et al. [16] commented that periods of extreme rainfall, combined with a long humid winter and a cool late spring, as well as rising groundwater due to infiltration from snowmelt, have proved to be highly significant in causing landslides in Switzerland. This is supported by evidence from historical, Swiss meteorological data calibrated against landslide events derived from dendrochronology [17].

Colleagues in SE Asia report other leading studies (e.g., [18-22]), which have contributed data to earlier opinions on whether 2-3 weeks of steady rainfall is more dangerous [e.g., 11,12] than localised short, sharp, intense rainfall events [13,23]. Sometimes, there is a critical combination of antecedent rainfall over a specific duration followed by a shorter and more intense storm [24]. However, no one has been able to achieve a universal link between the intensity and duration of rainfall despite the best efforts of a generation of researchers from [25] through to [26] (see Fig. 5).

It is clear from Fig. 5 that rainfall-induced landslides present a significant natural hazard. Consequently, the field monitoring of slopes has been carried out by numerous researchers [e.g., 18-22, 31-39] and the statistics of rain-induced landslides have been presented for a series of storms in Switzerland in 2005 [40] and in neighbouring Austria [41] from 1950 to 2000. However, uniform conclusions have not emerged, largely due to the wide variety of sites, instrumentation as well as different levels of sophistication and investment in the characterisation,

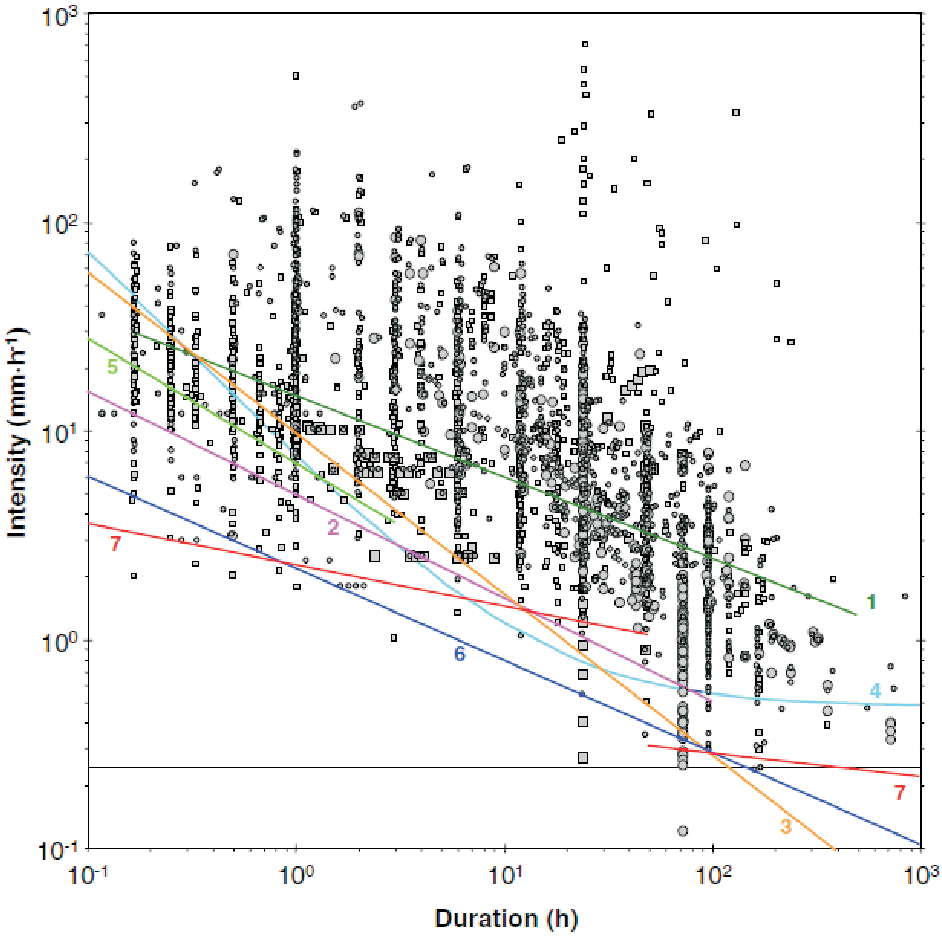


Figure 5. Global Intensity-Duration thresholds based on Guzzetti et al., [26] with published global (worldwide) Intensity-Duration rainfall thresholds proposed by: 1 [25]; 2 [27]; 3 [28]; 4 [29]; 5 [30]; 6 Inferred from this data; 7 thresholds inferred rom the probability estimates of the rainfall conditions, for two different rainfall periods (D < 48 h, D ≥ 48 h). The horizontal line shows a 0.25-mm/h rainfall intensity.

monitoring and modelling. This contribution attempts to add to the database obtained from past field tests and endeavours to provide a structure with which to accord a typology. It focuses on slopes that are predominantly in granular media and is accompanied by a very simple mode of analysis that would have common roots with Professor Šuklje's work.

Key factors affecting the likelihood of a mass movement being triggered are largely due to the location and origin of the slope (geological, geomorphological, anthropogenic factors), which are more or less constant with time in comparison with the more variable environmental effects, largely due to meteorology, hydrological, altitude and specifically precipitation [42] affecting infiltration [43] and water regimes, vegetation and temperature. Some impacts and outcomes that bear consideration are listed below, although they may not be the sole arbiters of whether a slope will remain stable or not:

1. topographical-soil engineering influences, particularly the slope angle related to the friction angle of the relevant strata derived from the original geology, in terms of deposition modes, anisotropy, macro--micro permeability with soil layering and underlying bedrock (in terms of the depth and the shape of bedrock surface);
2. precipitation-infiltration into the ground (soil) [44], groundwater flow processes, weathering at the soil--rock interface with attendant influence on permeability, exfiltration from the rock into the overlying soil layers and drainage into the underlying rock (e.g. [45-50]);
3. other factors affecting stability, such as the reinforcing effects of vegetation [51,52] and biological [51] or chemical [53] cementation;
4. triggering mode (extreme rainfall, changing groundwater and thermal regimes, seismicity and volcanic effects etc.; e.g. [54];

5. surface area, volume, aspect and mobility of failing debris, which have been found to be due largely to the initial void ratio as well as the availability of water to fill, or enlarge, the voids to form debris flows.

The locations of the three monitoring field sites (Table 1) discussed in this paper are given on a topographical map of Switzerland (based on [55]; Fig. 6;), showing the Alps (south) and Jura (north) mountains running approximately WSW to ENE, with the more highly populated areas (in pink) mainly located around the cities and in the 'Mittelland', between both mountain ranges. Also shown are cartoon pictures of the ground and a brief description of the main components. Some characteristics are extracted from each of the sites to enable subsequent analyses to be conducted and conclusions to be drawn.

Two main mechanisms were observed. These were a function of the inhomogeneity and permeability of the soil and rock layers as well as possible drainage channels in the soil and rock forming the slope. These will be described as top-down or bottom-up saturation, with the attendant influence on the volume of debris mobilised in the failure mode and hence the risk entailed by failure. Infiltration into and exfiltration from the bedrock are found to be key factors as well.

The interaction between the air and the pore water in the soil pores played a major role as did the cyclical response of the slope to rainfall and groundwater flow, as it became saturated or drained. For example, Petkovŝek et al. [69] have measured this for unbound base and sub-grade layers for several roads in Slovenia and the case histories discussed here rely on the instrumentation listed in Table 1.

Determining the soil state close to failure is dependent upon the stress path followed by the soil elements undergoing saturation. This is significant. Some laboratory stress path tests are presented and discussed for two of the sites to represent rainfall infiltration, together with data from a variety of direct shear box tests.

GRUBEN

The field data from the Gruben field test (Table 1) demonstrate that the fine-grained component of the matrix was significant at 15.9% < 63μm, and with < 2% at <2μm (from two representative surface samples

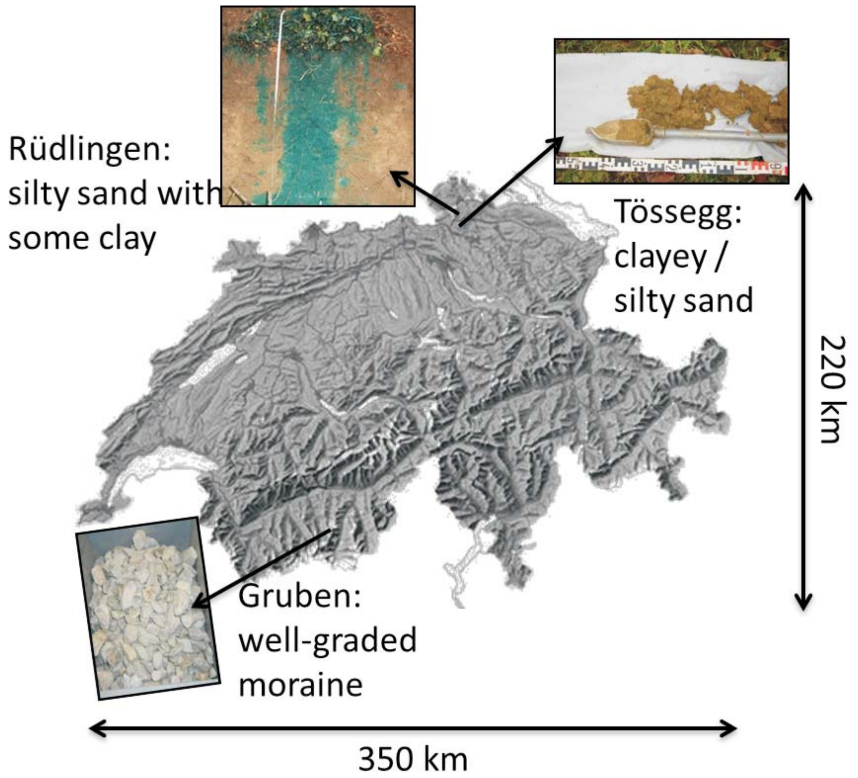
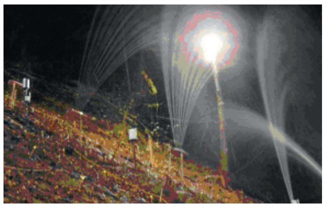


Figure 6. Location of three field-monitoring and test sites on a topographical map of Switzerland [55], which are described more fully in Table 1.

Table 1. Summary of field test sites in Switzerland.

Test field & Canton	Gruben Valais	Tössegg Zurich	Rüdlingen Schaffhausen
Dates	1998-2000	2004-2007	2008-2009
Picture			
Geology	scree slope, glacial debris /moraine, (Fletschhorn crystalline; Pretriassic Bernhard Plate of muscovite-rich gneiss / slate with some albite, chlorite, biotite)	quarternary outwash gravels mixed with finer grained deposits overlying horizontally layered sandstone (upper salt-water marls) above the lower freshwater marls	landslide debris (e.g. blocks & fine-grained soils) covering horizontally layered marls and sandstones in steep wooded areas
Soil layers	well graded angular granular soil particles with 10% fines < 60 μm	grassy surface with clayey sand overlying silty sand until rock at 1-2 m depth	roots in loose layer silty sand with %clay increasing with depth
Preferential infiltration & drainage systems	dense soil, voids e ~0.3, vertical & probably slope parallel flow with surficial run-off	channels from roots & animals in clayey sand, sometimes slope-parallel flow in silty sand, with little run-off and some infiltration into bedrock	primarily vertical drainage in pores (e=0.9) & along roots, infiltration into & exfiltration from bedrock
Test area & masl§	12 m (B) x 8 m (L)=100 m^2 and 5.2 m (B) x 10.5 m (L)=55m^2 2750 masl	15 m (B) x 13 m (L)=195 m^2 367 masl	8 m (B) x 35 m (L)=280 m^2 385 masl
Slope α	31° (did not fail) & 42° (failed)	27° lower slope to 17° upper slope (did not fail)	38-42° (2008: did not fail; Mar. 2009 failed)
Instrumentation*	MS, TDR, MP, JFT, FO	MS, TDR, MP, JFT, ST, UV, G-ERT	MS, TDR, MP, JFT, EDZ, P, SG, INC, PG, ST, AS, G-ERT (UNIL)
Friction angle φ'	39° (triaxial) & 41° (direct shear)	31° & 39° clayey & silty sand respectively in direct shear	30-32° (triaxial and direct shear)
Sponsors	Canton Wallis; BAFU	HazNETH, CCES / TRAMM	CCES / TRAMM
Dissertations	[56,57#]	[58,59#]	[60,61#]

§ masl metres above sea level, B breadth, L length

* MS Meteorological data such as air temperature, rainfall, humidity etc., plus UV Ultraviolet radiation, Measurement of volumetric water content: TDR Time Domain Reflectometry, MP Moisture Point, Suction: JFT Jetfill Tensiometer, pore-water pressure: P Piezometer, Earth pressure: EDZ earth pressure cell with P, ST soil temperature, Deformations: FO fibre optic cables, SG strain gauges, INC inclinometer, PG photogrammetry (plus AS Acoustic Sensors (STEP/ETHZ), G – Geophysical monitoring through ERT – Electrical Resistance Tomography); UNIL – University of Lausanne, BAFU - Federal Office for Environment; HazNETH – Natural Hazards Network ETHZ, CCES - Competence Centre for Environmental Sustainability / TRAMM – Triggering of Rapid Mass Movements.

These are doctoral dissertations. Additional semester or Masters' dissertations conducted on the Rüdlingen landslide include [62-68].

at up to 1 m depth, which were taken adjacent to Field 1; [70-72]. These fine grains and interstitial capillary space controlled the permeability and some aspects of the shear strength, in particular relating to the development of suctions. Furthermore, full saturation was not achieved and hence suctions were not dissipated completely, even after almost one week of intense rainfall in 1999 with a minimum baseline (after the first 2 days) of 10 mm/h.

In total, 1.9 m of rainfall was applied artificially (averaging 13 mm/h over the week in 1999, Fig. 7). The Moisture Point readings (Fig. 7, [70]), which averaged the volumetric water content over a given length, indicated that 'full' saturation was being approached only over the top 15 cm, following a gradual increase in the second half of the week. Even then, Sr was only 92% at the end of the experiment in 1999. Interestingly, there was a strong response to the raised rainfall intensity to > 20 mm/h on the morning of the 12th of July, in which little change was observed in the top 15 cm, but there seems to have been a consistent 'breakthrough' suddenly within a couple of hours to all sensors monitoring 15-30 cm, 30-60 cm, 60-90 cm and 0.9-1.2 m. Recovery to lower degrees of saturation occurred somewhat variably as the rainfall intensity dropped to 10 mm/h for the ensuing 16 hours.

Both test fields (2000/1 & 2000/2: slopes of 31° and 42° respectively) were subjected to a lower rainfall intensity in 2000, (8-22 mm/h: 0.76 m and 0.7 m of cumulative artificial rainfall on Fields 1 and 2 respectively). Fig. 8 shows the calibration of saturation data from 1999 to 2000 from using the same Field 1 (but with slightly less instrumentation), although this is only for ~2 days of rainfall, when there was a surficial failure in the steeper Field 2 (42°) and the tests were stopped. After failing to measure suctions in any instrument in 1999 due to the challenging environment [40], this was achieved in 2000 on Field 1 and the results were consistent with the measurement of water content, showing the initial and latest responses in terms of loss of suction at the shallowest and the deepest tensiometers, respectively. A summary of the net loss is shown in Fig. 8 [71].

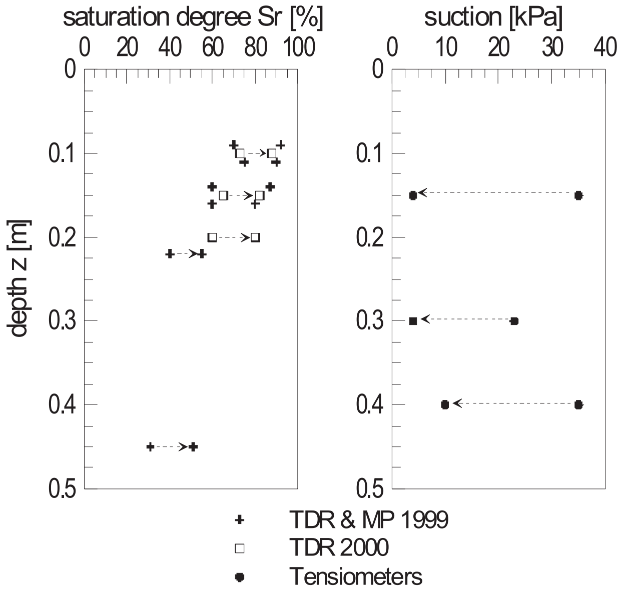


Figure 8. Gruben: summary of immediate pre- and post-rainfall event data showing increase in saturation combined with loss of suction as infiltration progresses (a) saturation degree determined from TDR and MP probes: Fields 1/1999 & 1/2000 (31° slope), (b) Suctions determined from tensiometers: Field 1/2000 (31° slope), Gruben [71].

TÖSSEGG

The Tössegg site is located in Canton Zurich on the banks of the river Rhine. It was selected due to a series of extreme events in May 2001 in which 42 surficial landslides occurred nearby following 10cm of rainfall in

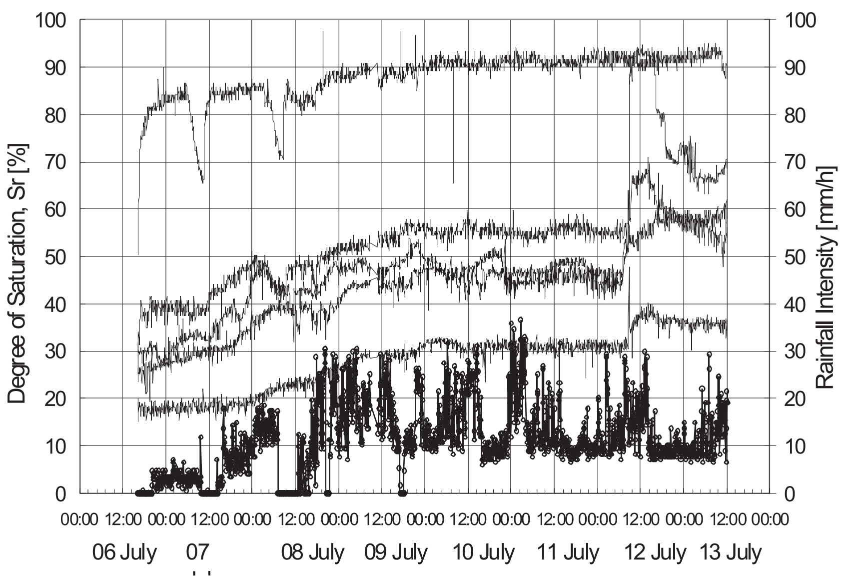


Figure 7. Gruben: Rainfall intensity and Moisture Point (MP) measurements in terms of saturation degree with time: Field 1/1999 (31° slope) [70].

40 minutes [73]. The slope was investigated and characterised using geophysical, geological and geotechnical methods [58,74-76]. A clayey sand was revealed below the grassy humus layer, with a silty sand overlying sandstone-marl bedrock at between just less than 1 m to nearly 2 m depth.

The electrical resistivity tomograms shown in Fig. 9 were developed for measurements made over the extent of the grassy slope in 8/2003 and 7/2004 [75], whereby resistivity depends on porosity, degree of saturation and mineral content. The rainfall differed significantly in the summer 2003 and summer 2004, in which 2003 was a very dry summer, reflected in the higher surface resistivities determined in Fig. 9. Variations in resistivity can be explained by changes in saturation, since other parameters remain more or less constant. The saturation of the layer below the topsoil, and up to about 1 m depth, on top of the underlying bedrock is greater in July 2004 in the lower part of the slope, where the instrumentation was ultimately installed for the monitoring experiment [74].

Table 2. Tössegg test field: layers at location of upper / lower / right / left instrument clusters (after [58]).

Layer thicknesses (cm)	Upper left	Bottom left	Upper right	Bottom right
Topsoil	25	20	30	25
Clayey sand	60	135	0	100
Silty sand	10	45	150	45
Depth to bedrock (cm)	95	200	180	170

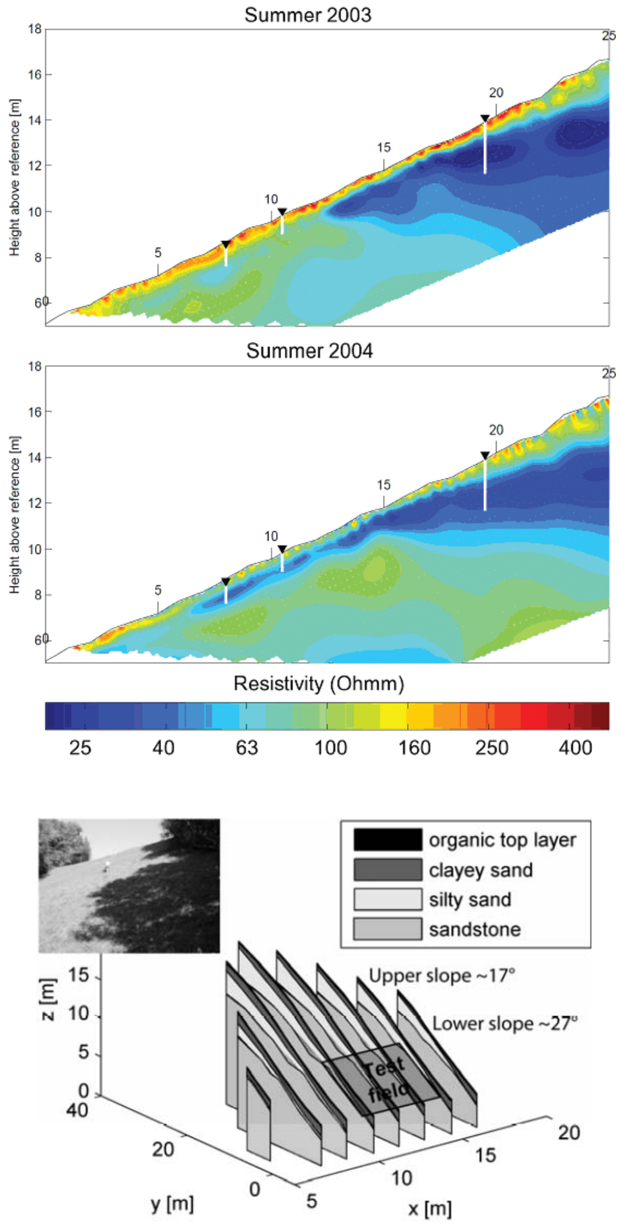


Figure 9. Tössegg: Comparison of electrical resistivity tomograms taken in August 2003 and in July 2004 [75] combined with ground model and location of the test site [74].

The reactions to summer (mid-August 2005) and winter (April 2005) rainfall events in terms of volumetric water content θ are shown with depth before, during and after the rainfall event for instruments installed at the four corners of the test field in Fig. 10, with soil conditions recorded in Table 2. The highest saturation was reached in winter (Fig. 10a) over the top 60 cm, mainly in the clayey sand, after 24 hours of rainfall with a greatest change in volumetric water content up to 0.075. There was some saturation of the underlying silty sand layer at about 1.2 m depth, on the right-hand side of the field, indicating percolation and a short-term increase in saturation in the more permeable layer. The saturation degree reverts to the original state four days after the rainfall event.

The maximum saturation for the summer event (Fig. 10b) was increased over the top 45 cm only, from a relatively dry state by increments of θ up to 0.15. The volumetric water content was generally lower prior to the summer event, than when fully saturated in winter [58]. Most of the rainwater had infiltrated with low runoff and there was little reduction in saturation over 48 hours after the rainfall stopped because both the temperatures and the evapotranspiration rate were low for this season during the days after the rainfall event.

Small-scale sprinkling experiments, covering a circular area of 1 m^2, were used after the monitoring period to

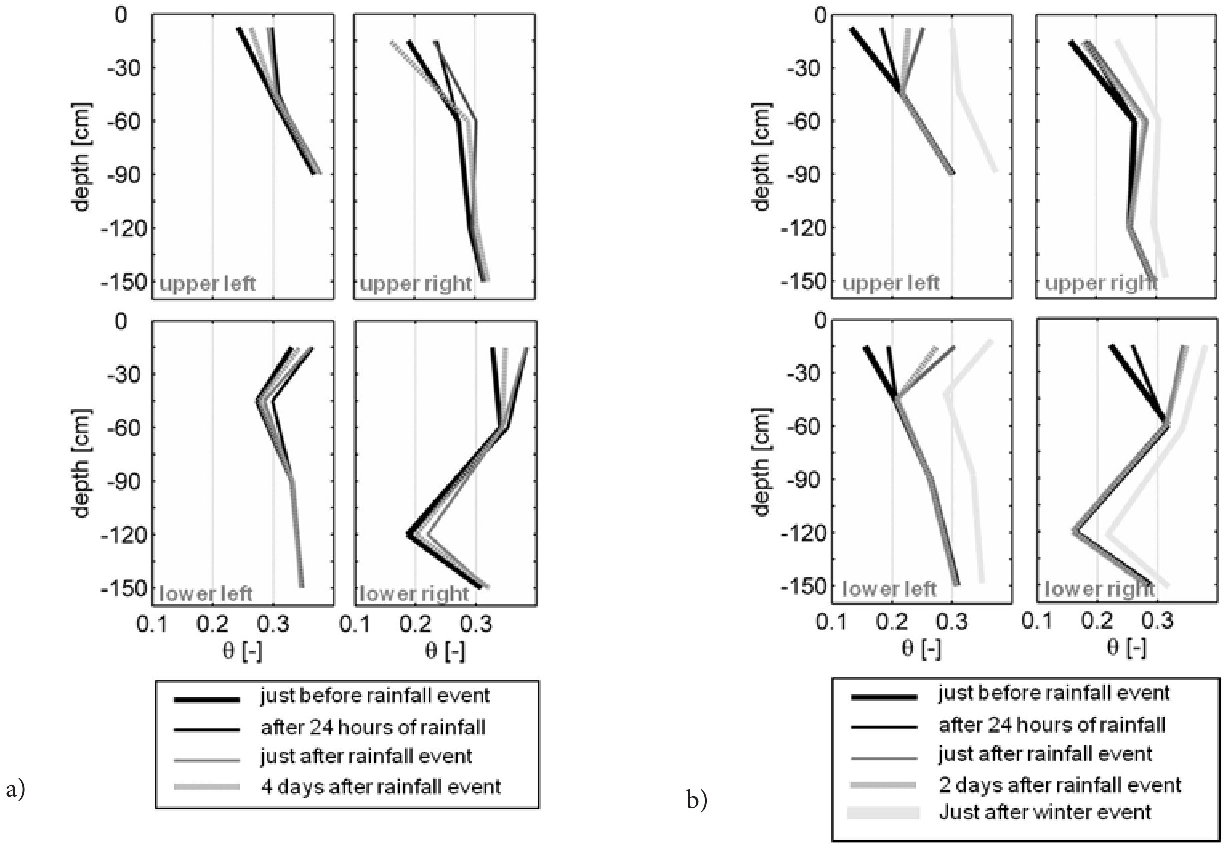
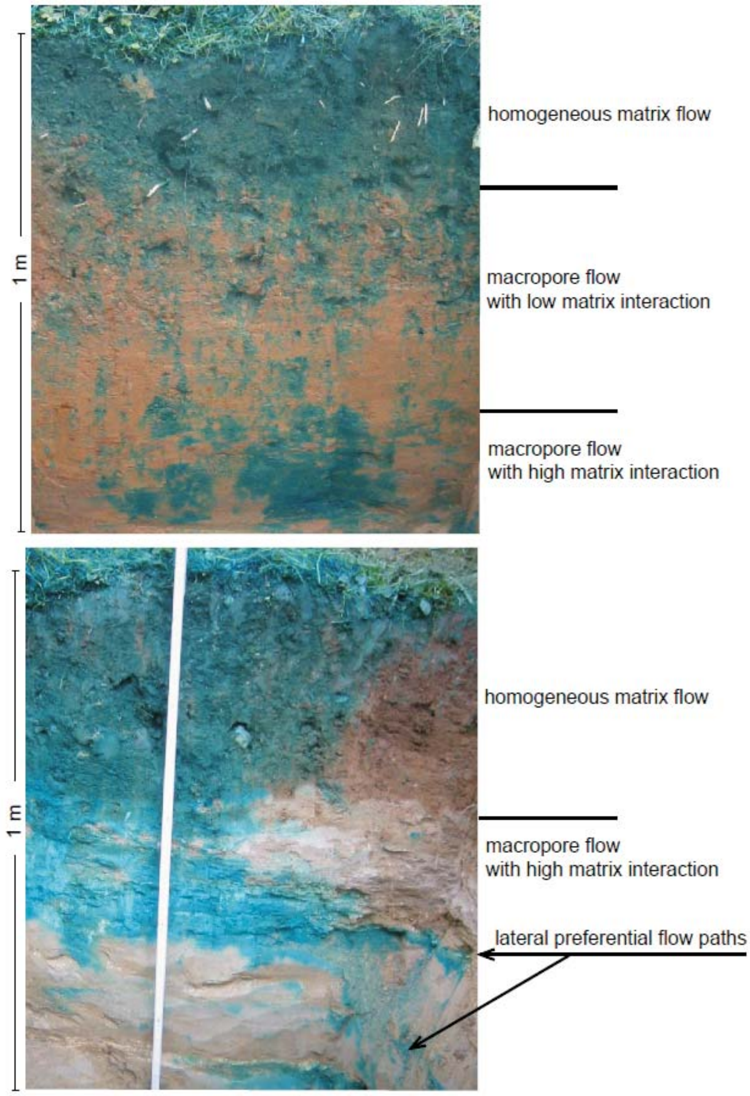


Figure 10. Volumetric water content with depth for rainfall events in
(a) winter (April 2005) (b) summer (mid-August 2005) for locations upper / lower / right / left referring to instrument clusters [76].

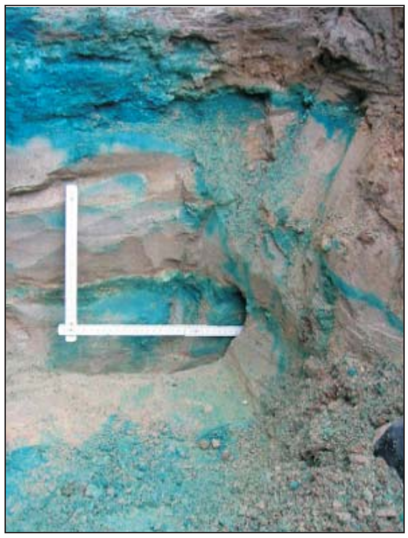
investigate the infiltration characteristics [77]. Brilliant blue FCF food dye [78] was added as a tracer (4 g/l) to water to visualise the flow paths and sprinkled at an intensity of 60 mm/h over a period of 2 hours. Runoff was collected from the bottom of the ring and measured with a tipping bucket. Vertical soil sections were excavated to a depth of 1 m on the day after sprinkling to reveal a high infiltration capacity with the topsoil stained homogeneously to a depth of about 30 cm, and with no overland flow (Fig. 11a). Preferential flow occurred below, with little matrix interaction showing clearly the small-scale variability of the infiltration flow paths. Lateral flow was detected at the transition to weathered bedrock (silty sand) and deeper percolation into the weathered sandstone was revealed by several stained flow paths and fractures below (Fig. 11b). Thus, perched saturation might be expected at the transition between the topsoil and the subsoil, as well as at the transition to bedrock. Such a formation of perched water tables can be assumed to support instability. Finally, the excavations revealed a fissure in the bedrock that was extremely effective in preventing groundwater-table rise at this location (Fig. 11c).

RÜDLINGEN

The forested Rüdlingen experimental site with dimensions of 35 m upslope x 8 m width (Table 1) is located about 2 km upriver from Tössegg on the west bank of the Rhine [79]. The slope gradient was 38-42°. Extensive characterisations of the soil [62,63,79,80] and root distributions (Figs. 12a-c; [52]) were conducted in test pits (and on specimens extracted in the laboratory) for a determination of the strata and the key soil properties prior to failure. Colluvium (silty sand with some clay) overlay sandstone and marlstone, which were located at a depth of between 0.5 m to more than 5 m. Figures 12a-c and Colombo [62] show that the macro-permeability between the peds was significantly greater than the micro-permeability within them, aided by the presence of roots that promoted drainage of the coloured water down to the bedrock (Fig. 12a). A high infiltration capacity, with homogeneous matrix flow, was revealed in combined sprinkling and dye-tracer tests. No overland flow was observed during 60 mm/hr sprinkling for 5 hours over an area of 1 m^2 [76]. Some sections remained unsaturated though, as can be seen in Figures 12b-c.



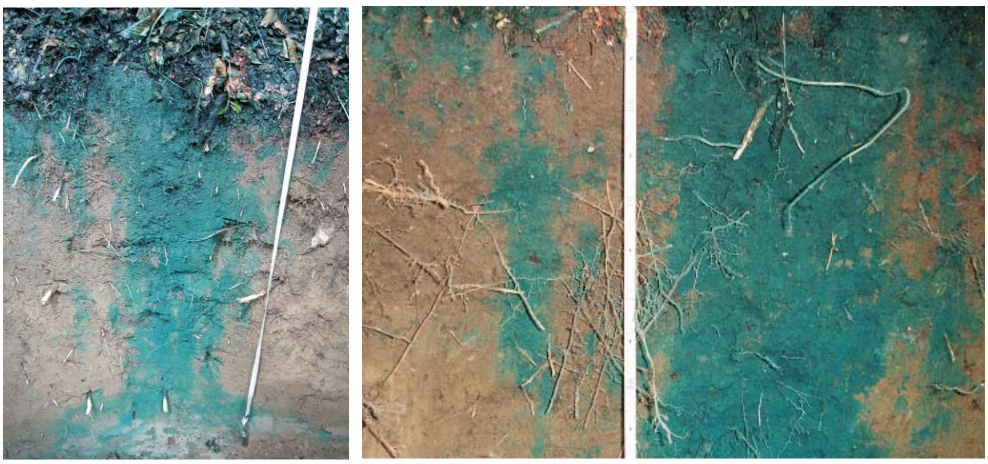
a) b)



c)

Figure 11. Infiltration experiments at Tössegg [77]

(a) topsoil and clayey sand,
(b) topsoil, clayey sand, silty sand, weathered sandstone
(c) showing same preferential lateral flow paths as in (b) with underlying fissure in sandstone bedrock (photographs and annotations: Dr. Peter Kienzler).



a) b) c)

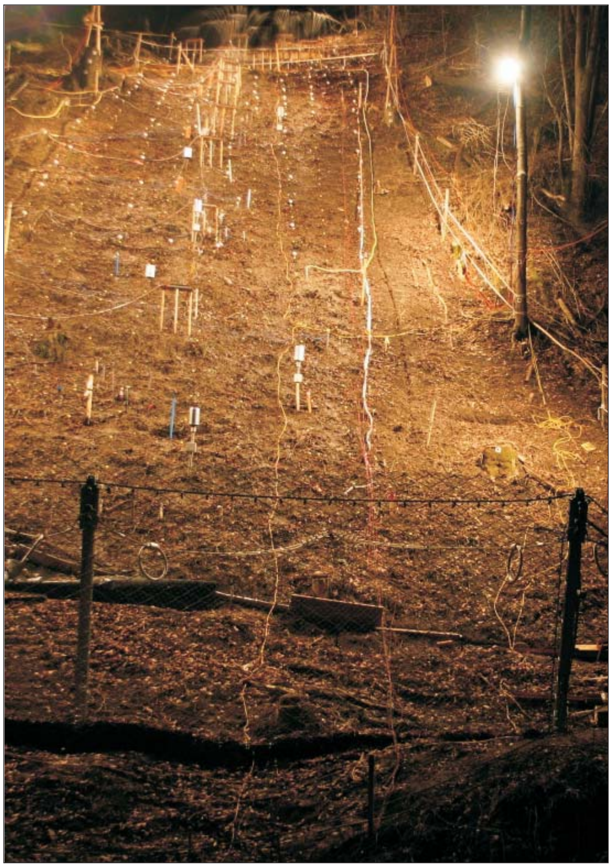
Figure 12. Rüdlingen: excavation of pits following infiltration tests (a) primarily vertical flow with lateral spreading above rock [79], (b) indications of partially saturated flow (c) root matrix (photographs: Dr. Peter Kienzler).

Beck [64,65] and Askarinejad et al., [81] also reported intermediate-scale laboratory infiltration tests to examine the influence of the soil structure on the saturation and hydraulic conductivity.

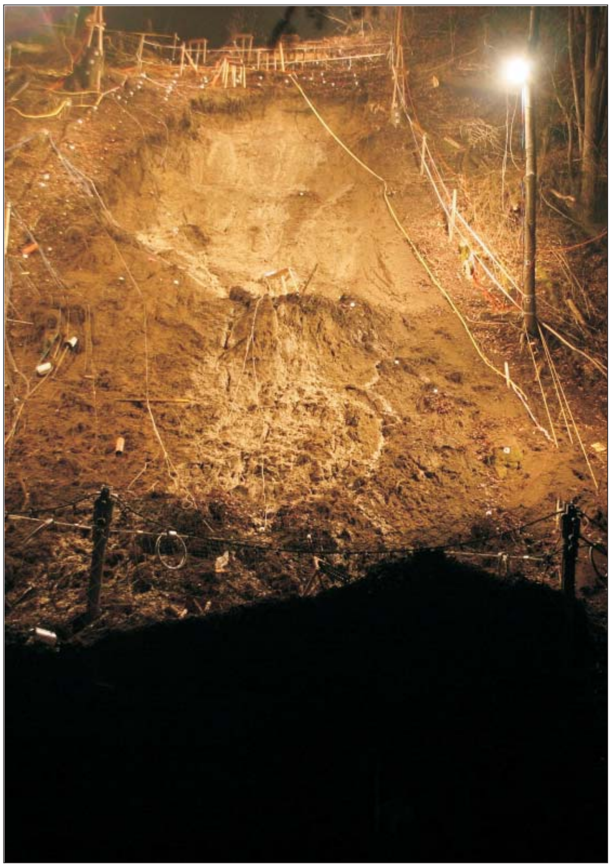
Lateral flow and perched saturation are observed above the bedrock (Fig. 12a).These results hinted strongly at the vulnerability of the slope to failure with the possibility for a water table to build up above the rock and with expectations of a failure above the transition to bedrock rather than a more surficial failure that is typical for many rainfall-induced landslides [71]. However, some stained fractures below the subsoil also implied that substantial percolation into the bedrock might occur, which could prevent complete saturation and failure of the instrumented slope (Fig. 13; [79]). This was indeed noted by Brönnimann et al., [70] in permeability tests in the lower part of the slope just prior to the first of two sprinkling experiments that were conducted to represent an extreme rainfall event, in October 2008 and March 2009. She observed that 100 litres were poured into the borehole over a few minutes, which drained immediately without developing a groundwater table. The ERT geophysical characterisation [82] also indicated the presence of saturated fissures in the bedrock in the lower half of the slope.

Over 1.3 m of rain (370 m^3) was applied in the first event over almost 5 days in October, largely directed towards the lower half of the slope. The increase in water table was minimal with a maximum value measured as an almost negligible excess pore pressure of 2 kPa. Movement was measured by photogrammetric methods in the upper-right part of the slope [83], but no failure developed.

Much less rainfall (232 mm; 65 m^3) was applied in the second (March) experiment, focused mainly at the top of the slope. 130 m3 of debris were mobilised 15 hours after the rainfall started. Movements on the centimetre scale were noted about 2 hours before failure occurred, initially near the surface and subsequently at 1.3 m depth [84]. Strains accelerated about 30 minutes before failure, as observed by three inclinometers that had been developed in house [85].



a)



b)

Figure 13. Rüdlingen: picture of the instrumented test field a) prior to and b) post failure (Photographs: Amin Askarinejad).

Suctions reduced (Fig. 14; [77]) following the start of sprinkling and positive pore pressures developed almost immediately at a depth of 60 cm in the top cluster (3). Saturation was slightly slower in the middle (2) and bottom (1) clusters until pore pressures reached 5-10 kPa at a depth of 1.5 m at the moment of failure. The data show a higher water table at the top of the slope, which is also supported by piezometer measurements. Water flow out of the back scarp of the failure scar immediately following failure contributed additional pore pressures, in the form mentioned by Professor Šuklje in relation to the Zalesina landslide. These were assumed to have triggered the destabilisation from the top, which took about 48 seconds from observation of the formation of a tension crack at the top of the slope to reach the protection net at the base of the slope. A scar was formed of dimensions 17 m downslope, approximately 7.5 m wide and approximately 1.3 m deep, on the left side, and 0.5 m deep, on the right.

SATURATION AND DESATURATION MECHANISMS

Several mechanisms are presented to describe the saturation of slopes. Infiltration is dominant in the first case and the saturation develops from the surface downwards ‚top down' (Fig. 15a, and in the summer regime for Fig. 15b). The wetting front increases the saturation in the upper layer, decreasing the suctions, and hence resistance induced by partial saturation. Unsaturated soil mechanics must be included in the analysis of slope stability. Slopes with an inclination α steeper than a critical friction angle φ'_{cv}, as for example in Gruben (Table 1a), will most probably slip surficially (see next section).

An overlying layer (1) was mainly fully saturated in the second case, for example for the winter regime in Tössegg (Fig. 15b). The underlying, and more permeable, layer (2) was saturated from the ‚bottom up' from groundwater flow along the bedrock in Tössegg (Fig. 15b) and at the top of the slope in Rüdlingen (Fig. 15c). The shear surface would be at a depth of more than 1 m, causing more debris material to be released in an unstable situation. In this case, the evaluation of slope stability should follow methods based on 'saturated' soil mechanics. Analyses for both cases are presented in the next sections.

SHEAR PARAMETER DETERMINATION THROUGH TRIAXIAL STRESS PATH TESTS

Infiltration processes cause an increase in the pore-water pressure, which decreases the mean effective stress leading to failure of the slope. It is essential to model the effective stress paths followed in the field in the laboratory when determining shear parameters (e.g., [4,87]), which can be done mainly in two ways:

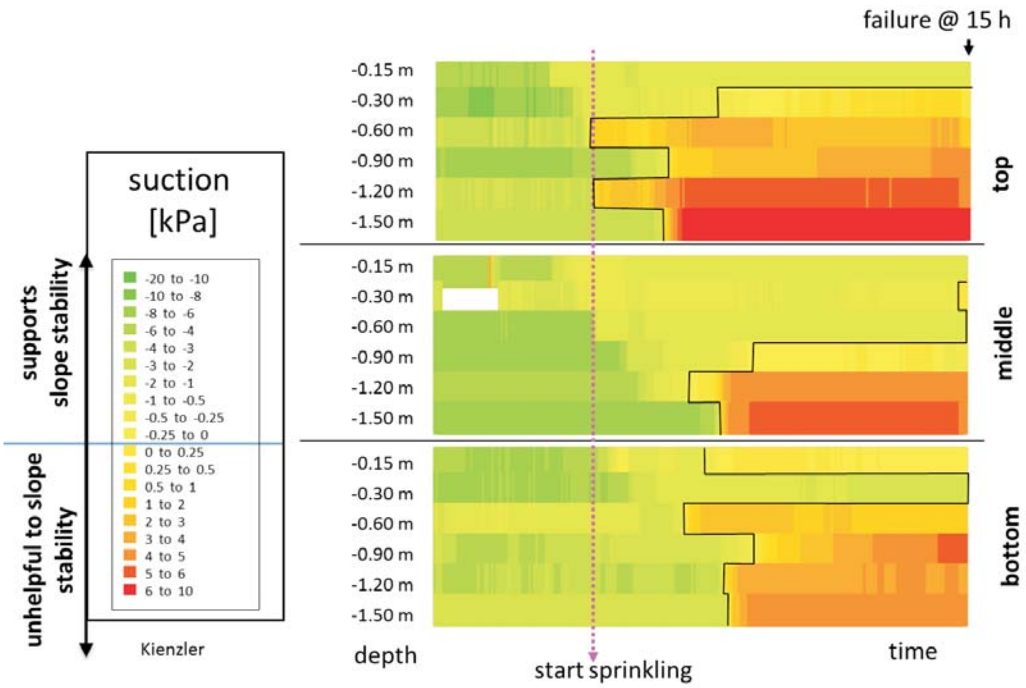


Figure 14. Rüdlingen: suctions and pore pressures measured at each of the 3 clusters from the top to the bottom of the slope, for the 15-hour duration of the sprinkling experiment in March 2009 (after [44, 77]).

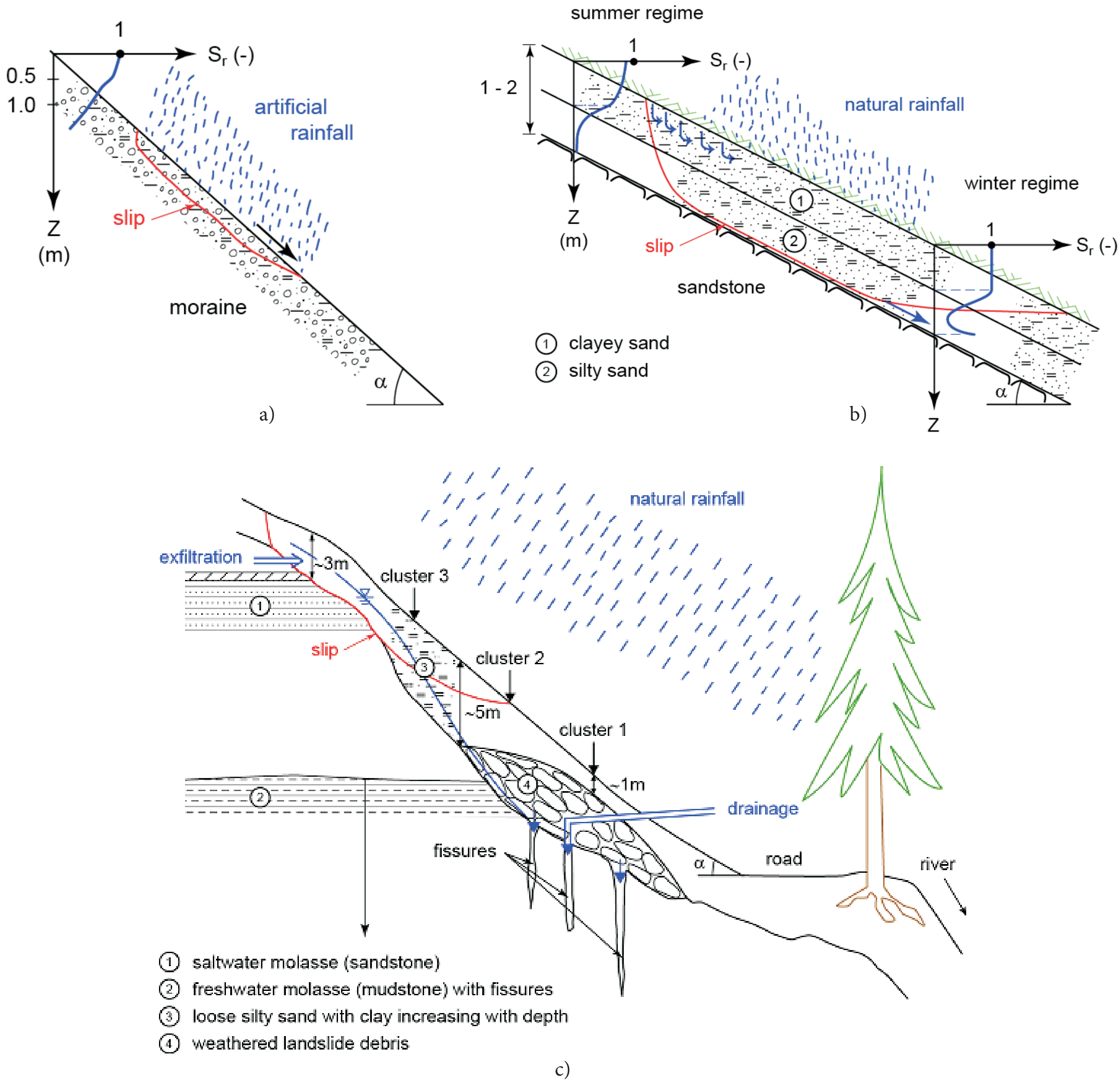


Figure 15. Saturation mechanisms after Springman [86] with a) ‚top down' saturation through infiltration, typically moraine as in Gruben [57,71]; b) in summer regime ‚top down', in winter regime additional ‚bottom up' through run-off above bedrock, as for Tössegg [76]; c) exfiltration from and infiltration into underlying rock at Rüdlingen (sketch interpreted from [60]).

1) maintaining a constant pore-water pressure and decreasing the total stress accordingly (e.g., [71] after [35,88,89]);
2) maintaining a constant total stress and increasing the pore-water pressure (e.g., [80]).

The first was adopted for the Gruben site (Fig. 16) as principal effective stresses σ'_1 and σ'_3 (where σ'_1 and σ'_3 are the axial and radial effective stresses), were reduced at the same rate (0.5 kPa/h) until reaching failure, as Constant Shear (deviator) stress Drained (CSD) paths in a large triaxial device [71]. Details of the sample preparation and consolidation paths for three CSD tests on 250-mm diameter, reconstituted, medium dense, moraine specimens, with a height of 500 mm and a maximum particle size up to 45 mm are described fully in [57] and [71]. Volumetric dilation and a slight hardening develop towards failure, with the gradient of the critical state line drawn as 1.59 in q-p' space ($q = \sigma'_1 - \sigma'_3$; $p' = (\sigma'_1 + 2\sigma'_3)/3$ for this stress path), through the failure state of the three tests, and leading to a value of ϕ' = 39° at failure (Fig. 16). This is slightly lower than that deduced from the shear box data (41°) that are presented in the next section.

Another approach was adopted by Casini et al. [80] for the Rüdlingen silty sand (Fig. 17) to model the expected

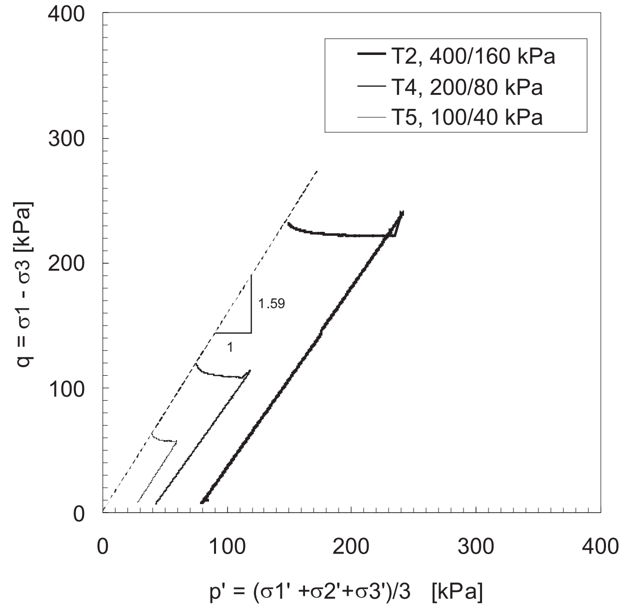


Figure 16. Constant Shear Drained (CSD) triaxial stress path tests on reconstituted Gruben moraine: stress path data shown in *q-p*' space with consolidation to σ_1'/σ_3' as shown in the legend, prior to shear (after [71]).

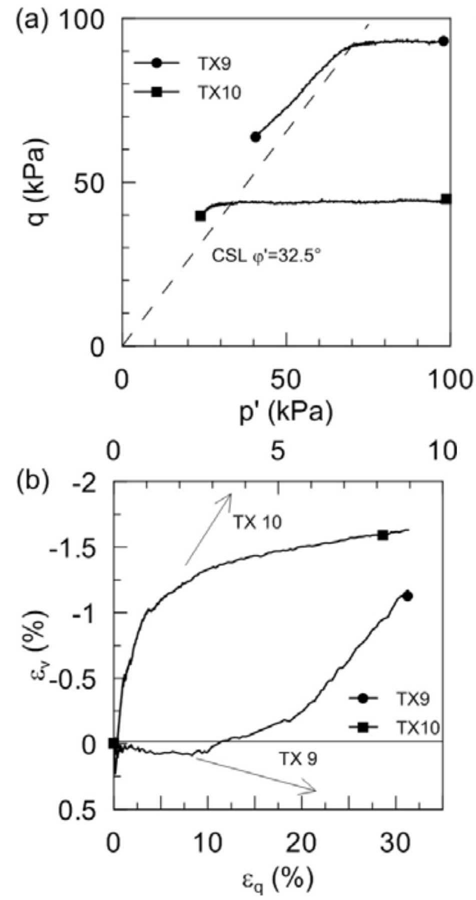


Figure 17. Anisotropic consolidation drained shear under constant axial load (CADCAL) triaxial stress path tests on reconstituted Rüdlingen silty sand: (a) stress path data shown in *q-p*' space; (b) ε_q-ε_v space, with strain axes at the top of TX10 and bottom for TX9 [80].

loss of p' due to a net increase in pore pressure without a change in the total stresses. Specimens were reconstituted and consolidated anisotropically before being subjected to drained shear under constant axial load (CADCAL). The mean effective stress was decreased by increasing pore-water pressure at a rate of 1 kPa/h for all drained stages, with the top and bottom drainage lines connected to a pore-pressure controller. The applied vertical load and radial stress were held constant throughout the tests [80].

Both specimens reached an ultimate failure stress ratio lying above a critical state line interpreted at an inclination of 1.3, representing ϕ'= 32.5°, and reaching values of $q/p' = \eta = 1.4$. Both specimens also demonstrated dilatant modes of failure; however, there had been concern that this soil might exhibit instability through shear strain acceleration towards failure, increase in plastic volume, accompanied by a significant loss in deviator and mean effective stress [80]. The behaviour of the soil was not properly unstable in the stress paths analysed (e.g., [90]), but temporary instability, denoted by strain acceleration at constant external loading rate, triggered the soil failure in any case [80]. Unsaturated conditions and these transitional characteristics of the soil behaviour should be modelled, including the typical volumetric behaviour of fine-grained soils, strain-induced anisotropy, and time dependence. Possibly, the migration of the fines content during shear led to time-dependent, high compressibility while limiting the potential unstable mechanisms as the matrix developed a more stable orientation, much as proposed by Professor Šuklje in 1953 for one-dimensional secondary creep. The dilatant mode of failure observed in these CADCAL tests might be a possible explanation of drops of measured piezometric levels about 1 hour before the failure in the second Rüdlingen test [91]. The time of occurrence of these drops is comparable with the time at which the acceleration of the bending strains is measured by inclinometers, which can support this hypothesis.

SHEAR PARAMETER DETERMINATION THROUGH DIRECT SHEAR TESTS

A simple slope stability analysis is based on the assumption of an infinite slope and a suitable constitutive model, such as the Mohr Coulomb criterion, with some flexibility over the choice of appropriate parameters, including a cohesion c' as well as an internal friction angle φ'. For saturated soils with a shear strength τ_f along a shear plane:

$$\tau_f = c' + (\sigma_n - u_w)\tan\varphi' \qquad (1)$$

(where σ_n is the normal stress, u_w is the pore water pressure). The shear strength at failure along a base failure plane parallel to the slope surface (Eqns. 2 &3) is enhanced by an apparent cohesion (here as c^*) for unsaturated conditions, which can also be related to the suction s by Gens [92]:

$$\tau_f = c'_{total} + (\sigma_n - u_a)\tan\varphi', \quad \text{where} \quad c'_{total} = c' + s/(\cot\varphi' + (\frac{s}{c^*})) \qquad (2a)$$

or as a function of the saturation degree S_r (e.g. [93]) from a Water Retention Curve (WRC) (e.g., [94,95]):

$$\tau_f = (\sigma_n - u_a)\tan\varphi' + (u_a - u_w)S_r\tan\varphi' \qquad (2b)$$

Equation 2b can be written, as follows, with S_r and suction ($u_a - u_w$), which were determined from in-situ measurements (Fig. 18; [71]):

$$\tau_f = c^* + \sigma_n \tan\varphi' \qquad (3)$$

whereby for $u_a = 0$, with $c^* = (-u_w)\, S_r \tan\varphi'$, the apparent cohesion c^* can be presented as a function of saturation degree, as in Fig. 19, for carrying out simple stability analyses. The intercept from Fig. 18 can be taken from the test series at different values of S_r, and plotted in Fig. 19 as a guide to selecting a suitable value of c^* for Equation 3. This data was derived from in-situ field direct shear tests [57,71], after the method developed by Fannin and Wilkinson [96]. This response is consistent with preceding experimental data (e.g., [97-98]).

The natural soil samples tested with in-situ direct shear tests exhibited marked dilatancy at failure for the applied stress levels [71]. Moreover, the TDRs installed in the vicinity of the eventual failure surface showed decreases in the degree of saturation at approximately 6 and 1.5 hours before the slip in the sprinkling experiment in 2000. This observation can be attributed to the dilative behaviour of the soil, which was measured in these series

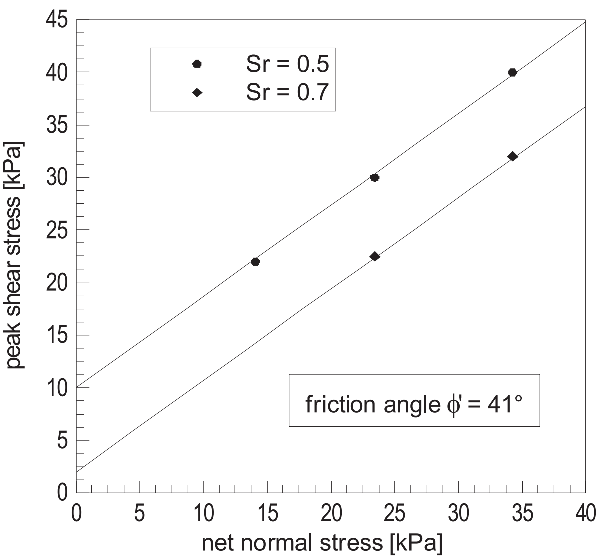


Figure 18. Gruben: shear-strength envelopes mobilised as a function of saturation degree: Peak shear stress v. net normal stress [71].

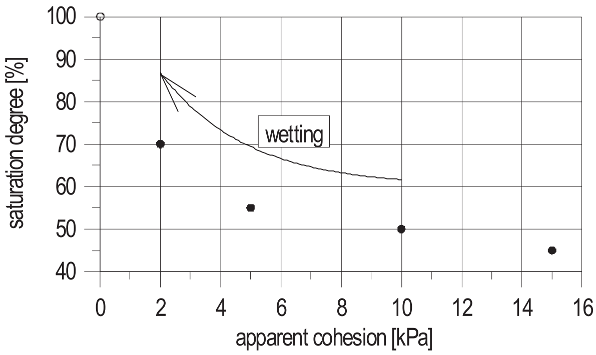


Figure 19. Relationship between saturation degree and apparent cohesion for alpine moraine in direct shear tests in the field [71].

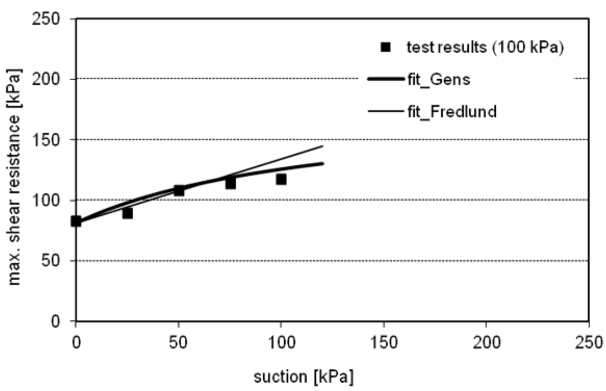


Figure 20. Tössegg: Shear strength envelopes for suction-controlled direct shear tests on silty sand with peak shear stress mobilised as a function of suction [after 58].

of in-situ direct shear box tests, or to the piping of fine-grained soil particles [91].

Thielen [58] used Gens' [92] simple criterion to represent the data for shear box tests conducted at the University of Catalunya on both the silty and the clayey sand in an apparatus designed to impose specific values of suction (Fig. 20). She reported values of ϕ' of 31° and 39° for clayey and silty sand, with c^* of 300 kPa at σ_n = 40 kPa and 1000 kPa at σ_n = 100 kPa for clayey sand and c^* of 100 kPa at σ_n = 100 kPa for silty sand. Also fitted against Thielen's [58] data was Fredlund et al.'s [99] criterion, although this is not pursued further in this paper.

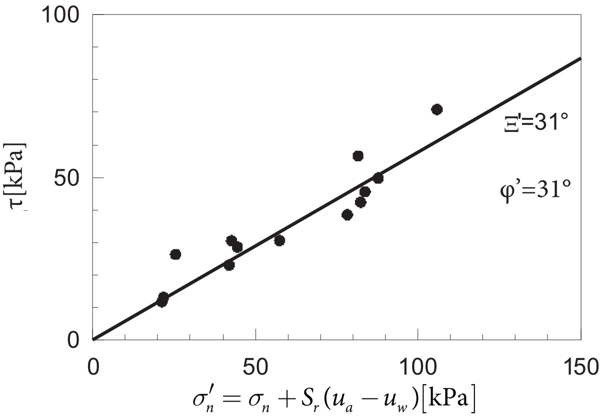


Figure 21. Rüdlingen: Interpretation of direct shear tests at constant water content to predict the critical state angle of friction for the reconstituted silty sand [100].

Shear data derived from direct shear tests conducted on unsaturated reconstituted Rüdlinger silty sand soil at constant water content [63], with a known value of water in the voids ew compared to the total voids e, with the degree of saturation $S_r = e_w/e$ [100]. The results (Fig. 21) were interpreted from Equation 2b by allowing for the effects of suction derived from the WRC (e.g., [71,93,101,102]), which led to a lower bound of φ' = 31° for these reconstituted specimens.

LIMIT EQUILIBRIUM CALCULATIONS

Infinite slope conditions

Unsaturated conditions on the base shear plane

A global factor of safety SF against failure can be guaranteed to be considerably over 1.0 for many slopes in their natural states through the enhancement to shear resistance caused by suction. Stability calculations may be simplified to infinite-slope conditions in some cases, whereby the shear zone at depth z is shallow relative to the width of the slope, which is assumed to be infinitely long. Balancing the force acting on each slice, assuming that the phreatic surface lies below the section concerned (e.g., after [71,103]), the following equation can be deduced for the factor of safety SF:

$$SF = \frac{\{(c^* / \cos^2 \alpha) + \gamma z \tan \varphi'\}}{\gamma z \tan \alpha} \tag{4}$$

where γ is the unit weight of the soil (kN/m^3) and α the slope inclination. The necessary value of c^* for SF = 1.0 can be determined through Equation 5.

$$c^* = \gamma z \cos^2 \alpha \{\tan \alpha - \tan \varphi'\} \tag{5}$$

Saturated conditions on the base shear plane

A more traditional approach with a positive pore-water pressure was placed in Equation 6 based on an infinite saturated slope, with the groundwater level at a depth of z_1 and a shear surface at a depth of $z_1 + z_2$, in cases where the saturation developed from 'bottom up' (after Figs. 15b & c, although the failure surface is rather deep compared to the length), given by:

$$SF = \frac{c' + (\gamma z_1 + \gamma' z_2) \cos^2 \alpha \tan \varphi'}{(\gamma z_1 + \gamma_{sat} z_2) \sin \alpha \cos \alpha} \tag{6}$$

where γ_{sat} is the unit weight of the soil in saturated conditions and $\gamma' = \gamma_{sat} - \gamma_w$.

Two and simplified three dimensiona approaches

Approximations can be made to adopt an inclined channel, with α as the slope angle, for cases in which the depth of the shear zone, z, is significant in comparison with the width, d, of a failure mechanism. The shear strength can be determined to include unsaturated conditions for homogeneous ground along the sides and base of the channel, if the shear surfaces are above the groundwater level [e.g. 63,100] and root reinforcement in the form of an additional cohesion c_r (e.g., [104]) also along the sides and base as may be required. An estimation of the relevant earth-pressure coefficient K was necessary to determine the effective stresses acting horizontally over the sides with depth. Amending c_r to account for changes with depth is indicated in Equation 7, in which a combined apparent cohesion can be input to the requisite equation to calculate the factor of safety for a channel of a specific width with a base shear plane at depth z:

$$c^* = S_r(u_a - u_w)\tan\varphi' + c_r(z) \qquad (7)$$

Askarinejad et al., (2012b) present some two and three dimensional limit equilibrium solutions for these cases.

Application to Gruben

Why slopes become unstable during or shortly after intensive rainfall can be explained for a case history from a 42° steep moraine slope in Gruben (2000/2) with insitu unit weight γ = 20.2 kN/m^3 and φ' =39°. Although saturation only reached 95% at a depth of z < 0.5 m, a surficial failure occurred at this depth. Infinite slope conditions were assumed since the extent of the zone (Area > 50 m^2) was large relative to the depth of failure (< 0.5 m).

The minimum factor of safety (Eqn. 4) was calculated to be smaller than 1.0 without a contribution from an apparent cohesion. Although the failure in Gruben was only surficial, the value of c^* < 1 kPa was deduced at depths z <0.5 m, which agreed well with the results from the direct shear tests in the field (40% < S_r < 100%), (Figs.18 & 19), 0.2 < c^* < 0.5 kPa for 0.9 < S_r < 0.95 (z~0.2 m). Although some of these assumptions cannot represent the failure mechanism exactly, at least they help to explain why a failure surface at a depth of <0.5 m is possible for a 42° slope, when S_r = 0.95.

Application to Tössegg

Although the shape of the critical failure surface is transitional towards a slip circular form at Tössegg [58], the outcomes are briefly described here. Thielen [58] used coupled thermo-hydraulic (TH) finite-element analyses within the Geostudio code VADOSE/W [105] both to model the slope response during the monitoring period at Tössegg, using data from one year to calibrate the model and then to insert the parameters obtained in the second year of data to complete a validation. This was quite successful and then the relevant parameters and state in the ground were input to the parallel two-dimensional limit equilibrium SLOPE/W model [106] to find the critical failure mechanism. This was selected by the programme itself from the most critical case according to the approaches proposed by Morgenstern and Price [107], Spencer [108], Bishop and Morgenstern [109] or Janbu [110].

Calculations were carried out at monthly intervals for 2005 (Fig. 22). The lowest (2D) factor of safety was 1.74 on 1.2.2005, reflecting the saturated state of the clayey sand in the winter months and partial saturation in the underlying silty sand layer (see Figs. 11a&b; 15b). This is also conservative because side friction is ignored. It is unlikely that the factor of safety will approach unity given a lowest angle of friction of 31° and a steepest slope angle of 27° unless significant artesian pressures develop in the silty sand.

Application to Rüdlingen

Fig. 23 summarises a simplified stability analysis using a channel of infinite length and the dimensions shown as a slope inclination α = 40°, width of d = 8 m (Figs. 23a-d). A void ratio e = 0.9, a lateral earth pressure of $\sigma'_h = K(\sigma_v + S_r(u_a - u_w))$ and $K = 1 - \sin\varphi'$, with the water table at depths of z = 0.8 m, 1.2 m, 4 m, and φ'=31° were chosen as representative values for the field conditions in Rüdlingen.

The contribution from c_r was varied with depth (Eqn. 7) showing, as was deduced from the first experiment in October 2008, that the presence of roots and a deep water table was essential for stability (Fig. 23d). This is entirely consistent with the position described due to bottom

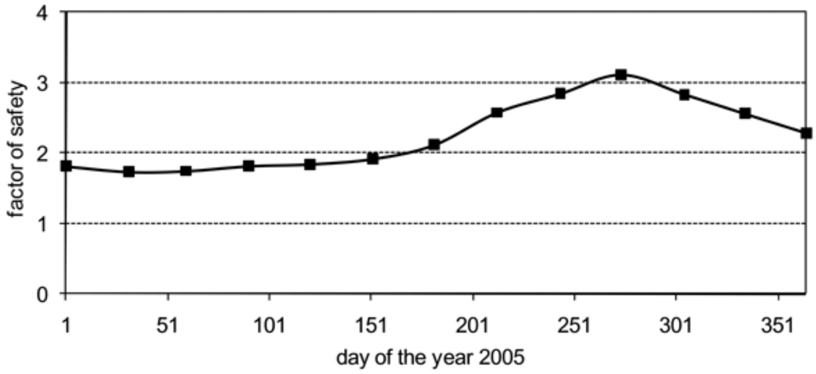
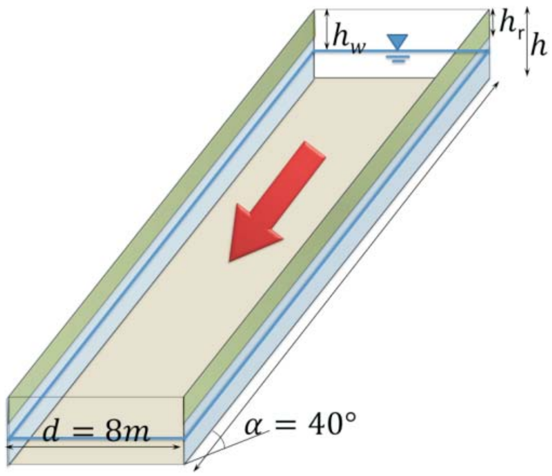


Figure 22. Tössegg: seasonal variation of factor of safety in 2005 against slope failure calculated using SLOPE/W after coupling with suctions and pore pressures obtained from TH Modelling with VADOSE/W (after [58]).

up saturation to build a water table above the bedrock (e.g. Fig. 15c) as the SF drops and with the critical shear surface at the boundary with bedrock as the water table rises from 4 m (Fig. 15d) to 1.2 m (Fig. 15c) to 0.8 m (Fig. 15b). The failure body (Fig. 13b) bottomed out on rock even though it has not followed a channel form with the base shear surface parallel to the surface and vertical sides.

A constant value of c_r = 5kPa along the sides and the base contributes to the stability in comparison with the case in which there is no contribution from the roots, or merely a local effect close to the surface (two other models with c_r constant at 5 kPa over the top 0.2 m of soil or with c_r reducing from 3 kPa over the top 0.2 m, decreasing by 1 kPa each additional 0.2 m).

a)



b)

c)

d)

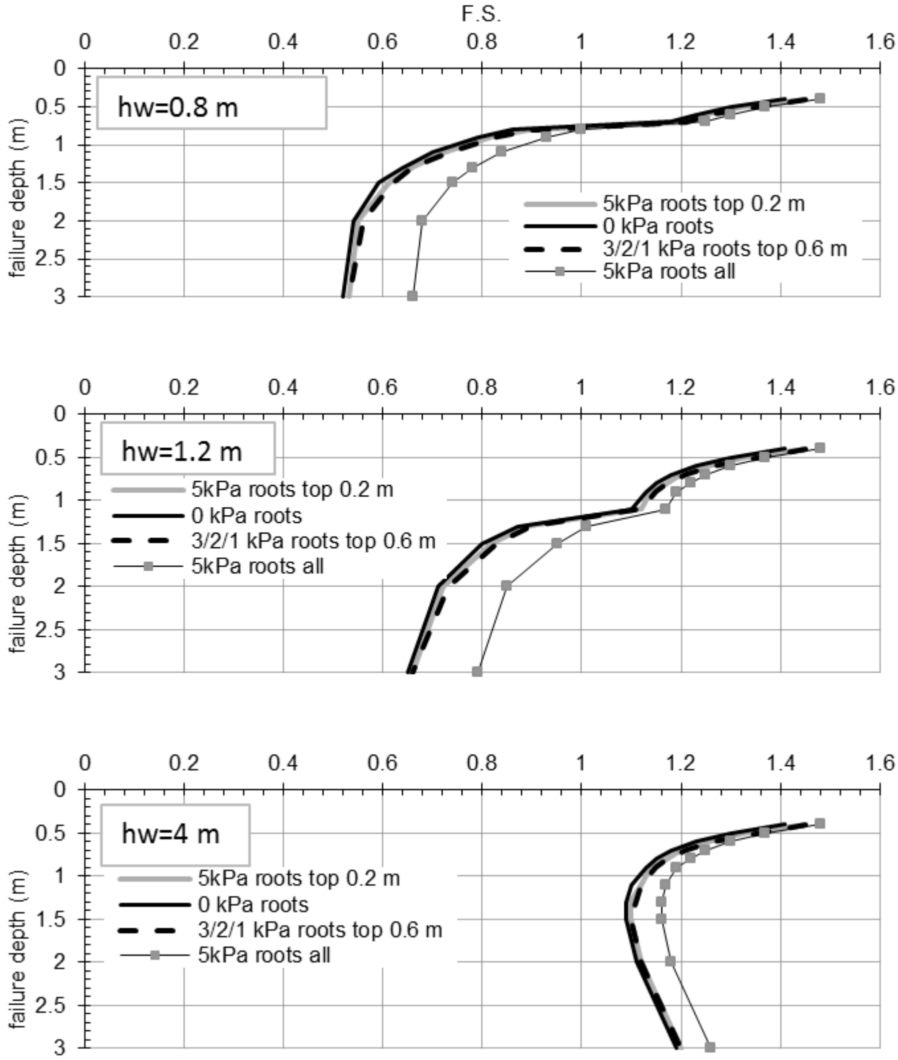


Figure 23. Two-dimensional limit equilibrium stability calculations for a channel geometry, including suctions (when above the water table) and root reinforcement on the sides and base: (a) channel dimensions with parameters appropriate for Rüdlingen; results for various values of c_r and water tables at (b) z = 0.8 m (c) z = 1.2 m (d) z = 4 m.

Fig. 23d shows the effect of suction combined with roots, in which the critical depth for such a failure mechanism with the lowest SF occurs between 1.2 to 1.4 m, depending upon the degree of root reinforcement. This was not too dissimilar from the values obtained, notwithstanding the differences between the geometry assumed in Fig. 23a and that observed in Fig. 13b.

Subsequently, Bischof [66], Malecki [67] and Askarinejad et al. [111] have investigated the development of failure mechanisms using more advanced coupled finite-element analyses, and further work is underway.

A PRACTICAL APPLICATION: CONDITION INDICATORS

'Condition Indicators' were adopted for the use of the observational method (Peck, 1969) to determine safe access to the test site in Rüdlingen (after Fig. 24) during the artificial rainfall experiments. The critical combination of S_r and suction s (= u_a – u_w) in an infinite slope stability analysis, as described in Equations 4 above with $c^*= S_r(u_a - u_w)\tan\varphi'$, was applied to the Ruedlingen slope. These provided the basis for an alarm system and were supported by observational markers: green (s > 20 kPa) – no restrictions on access or behaviour; orange (20 kPa > s > 7 kPa) – warning, within the uncertainty envelope for failure, restrictions to access on, below or within 10 m to the sides and above the slope; red (s < 7 kPa) - danger - whereas several activities could be permitted or limited, to reduce any risk to the participants and public.

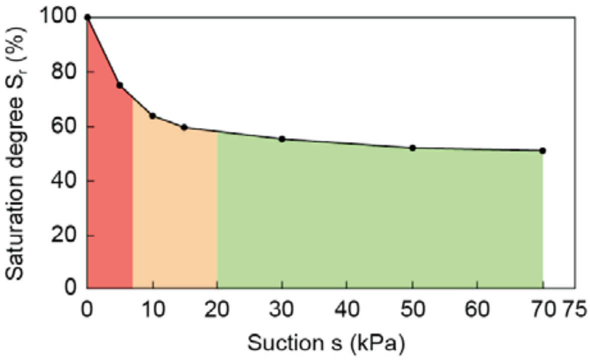


Figure 24. Relationship between saturation degree and suction of Rüdlingen silty sand in the laboratory based on the wetting branch of the Water Retention Curve [80] superimposed with alarm levels [86].

SUMMARY

Leroueil [113] observed that 'the failure of natural slopes constitutes an important geotechnical problem that involves a variety of geomaterials in a variety of geological and climatic contexts, and which has a major socio-economic impact in many countries.' This paper reviews the state of knowledge for rainfall-induced landslides in primarily granular materials, examining the influence of partial saturation, infiltration and saturation regimes on slope failure.

The infiltration of rainfall has led to surface instability in slopes steeper than the internal angle of friction, just prior to full saturation being reached in three field sites in Switzerland that have been well characterised and instrumented. Each slope has contributed significant learnings and a generic approach to categorising the saturation of slopes has been proposed. This has been accompanied by several calculation methods that have been presented in this short contribution for adoption in making a preliminary judgement of slope stability.

Triaxial, field and laboratory shear box data have been used to determine the shear parameters for the mobilised shear strength at failure using a simple Mohr-Coulomb analysis and assuming the effect of suction is modelled by an apparent cohesion, which depends on the saturation degree. Alternatively, suction may be estimated from the saturation degree at various depths and included in a shear-strength envelope, which is dependent on suction and saturation degree. Prediction of this 'apparent cohesion' term has been made, by adopting a factor of safety equal to unity under simple limit equilibrium, infinite-slope stability analysis with an extension to a simplified three-dimensional case where necessary.

An apparent cohesion, depending upon saturation degree, offers a simple option for modelling the significantly more complex unsaturated behaviour in a slope, whereas the modelling benefits greatly from understanding of the slope hydrology and the way in which the cyclical saturation and drainage processes develop with time. Despite the simplicity of the analyses and extreme heterogeneity of the moraine, it was found for the moraine slope in both cases that the factor of safety reduced almost to unity at depths of ≤ 0.5 m for the 42° slope, as had been observed from the field test.

Despite modern computational capacity and highly advanced, multi-parameter, constitutive models, a case is made that the use of simple models is still as valid today as they were in Professor Šuklje's time. The presence of water in slopes, as discussed by Professor Šuklje, will always challenge the calculation of stability. However, understanding the mechanisms and applying simple and robust models and good engineering judgement can often lead to the ability to make challenging decisions about slope safety.

ACKNOWLEDGEMENTS

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