

Problems in predicting the mobility of slow-moving landslides

Th.W.J. van Asch ^{*}, L.P.H. Van Beek, T.A. Bogaard

Faculty of Geosciences, P.O.Box 80-115, 3508 TC Utrecht, The Netherlands

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Abstract

This paper discusses problems in predicting the mobility of slow-moving landslides. Three case studies are presented here where research has been carried out by the Utrecht University: The La Valette landslide complex in the French Alps, the La Mure landslide in the French pre-Alps near Grenoble and the Hau landslide in Switzerland.

To predict field velocities of these slow-moving landslides the viscosity parameters of the material of these landslides were determined by strain-controlled tests in a ring shear apparatus based on Bishop's design at Utrecht University. The viscosity parameters from the laboratory proved to be 10 to 1000 times lower than viscosities obtained from back analyses on the observed velocities in the field. This discrepancy may be explained by the development of negative pore pressures when the plastic material slides over a rigid, wavy slip surface and/or by convergent flow effects. The associated gain in strength results in a higher apparent viscosity.

A more detailed analysis is made for the movements of the La Valette landslide. Observed velocities at the La Valette landslide are difficult to describe by one parameter set as the response to a change in groundwater level is not the same during a rise or fall in the piezometric level. This deviation may be explained by rapid changes in total stresses and consequently changes in pore pressure under (partly) undrained conditions. The emerging hysteresis with local and temporal variations in pore pressure makes it difficult to predict in detail the moving pattern of landslides.

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1. Introduction

There are many problems in predicting the behaviour of slow-moving landslides. Their hydrology can be complex due to the presence of small-scale anisotropy and large-scale heterogeneity in the permeability of intact and fissured material (Corominas et al., 1999; Malet et al., 2005). Consequently, large landslides have shown an erratic and complex response to rainfall

(Noverraz et al., 1998; Corominas, 2000; Malet et al., 2005). Deformation within the landslide body also affects the hydrological and mechanical properties (Nieuwenhuis, 1991). During movement both tension and compression zones in fine-grained landslides develop. Where material extends, fissures develop leading to rapid infiltration and drainage. Under compression, excess pore water pressures as a result of undrained loading may generate sudden surges (Bonnard et al., 1995, Baum and Fleming, 1996, Caron et al., 1996). In rest, consolidation of the shear band may lead to strength regain, attenuating the likelihood of reactivation (Nieuwenhuis, 1991, Angeli et al., 2004).

^{*} Corresponding author. Tel.: +31 2532763.

E-mail address: t.vanasch@geo.uu.nl (T.W.J. van Asch).

In this paper two problems related to the prediction of the behaviour of slow-moving landslides are discussed:

- The validity of laboratory tests on soil viscosity for the prediction of landslide velocities in the field.
- The hysteresis in movement in relation to observed groundwater fluctuations.

Three case studies are evaluated which were investigated by PhD students of the Utrecht Uni-

versity: The La Valette landslide complex in the French Alps (Van Beek and Van Asch, 1996), the La Mure landslide in the French pre-Alps near Grenoble (Van Genuchten, 1989, Nieuwenhuis, 1991) and the Hau landslide in Switzerland (Van Beurden, 1997).

2. The viscosity model

Several flow models have been proposed to describe the mobility of slow-moving landslides. Flow models based on Bingham's law are frequently used (Yen, 1969,



Fig. 1. Aerial view to the North of the La Valette landslide, near Barcelonnette (French Alps). AB: the line where velocities were measured.

Van Asch and Van Genuchten, 1990, Angeli et al., 1996):

$$\frac{dv}{dz} = \frac{1}{\eta}(\tau - \tau_0) \quad (1a)$$

$$\tau_0 = (\sigma - u)\tan\phi'_r \quad (1b)$$

where v is the velocity (m/s), z is depth, η is the dynamic viscosity (kPa s), τ is the shear stress (kPa) and σ the total normal stress (kPa), u the pore pressure (kPa) and ϕ'_r the residual friction angle of the material (the residual cohesion is assumed to be zero).

For a proper calculation of the velocity profile, Eq. (1a) has to be integrated over the depth z , which results in a non-linear velocity profile. However, for narrow shear bands (<5 cm) one can assume a straight velocity profile and thus a linear relation between the velocity and z . Thus, one can write for the velocity of a landslide moving over a thin shear band:

$$v = \frac{h_m}{\eta}(\tau - \tau_0) \quad (2)$$

in which h_m is the thickness of the slip surface (m).

For larger ranges of excess stress ($\tau - \tau_0$) many authors found a non-linearity of v with τ , both in laboratory tests as in the field (Bertini et al., 1986; Cartier and Pouget, 1988; Salt, 1988; Bracegirdle et al., 1991; Cornforth and Vessely, 1991). This can be expressed as:

$$v = \frac{h_m}{\eta_0}(\tau - \tau_0)^b \quad (3)$$

The exponent b can be interpreted as a variation of the viscosity with the applicable excess shear stress, while η_0 can be considered as the intrinsic viscosity of the material. According to Vulliet and Hutter (1988), the form of Eq. (3), seems to describe landslide velocities better than other, more empirical laws.

3. Description of the selected landslides

The three investigated landslides represent slow-moving landslides of near constant activity that developed in different kind of materials and of different size. Movement is concentrated in a distinct shear band with a viscous character and the speed seems to be controlled by varying groundwater levels. According to Hung et al. (2001), they can be classified as mudflows.

3.1. The La Valette landslide

The mass movement complex of La Valette near Barcelonnette in the French Alps amounts to a volume

of 3.6 million m³ (Van Beek and Van Asch, 1996; Maquaire et al., 2003). It consists of a rotational block (50 m in depth) incorporating in-situ Flysch and Terres Noires rock material, which failed in 1982 in the upper part of the slope (Fig. 1).

The middle part consists of meta stable weathered marl (*Terres Noires*) and argillaceous morainic material. This part became active in 1986. Rapid slides in the middle part transformed into mudflows which built up the lower part of the landslide complex. The upper scarp of the landslide complex is situated at a height of 2000 m while the toe lies at 1450 m.

The analyses focus on the movement in the middle part where the material moved over a depth of 20 m with a slope of 18°. The type of movement can be characterised as a translational slide. Displacements at the surface were monitored by topometry along a survey line perpendicular to the direction of movement. The data sets analysed here cover the period from 9 September 1988 until 10 September 1991. The material in the middle part of the landslides consists of strongly remoulded Terres Noires mixed with morainic deposits. The matrix is a sandy silt with a considerable clay fraction in which larger fragments (stones, gravel) are incorporated. Peak strength measured with triaxial tests on undisturbed material amounts to $\phi_p = 26.8^\circ$ ($n = 10$, $R^2 = 0.98$), the peak cohesion is relatively low at 2 kPa. After failure strain softening was observed. With a strain of 16% the incomplete residual strength has a residual friction value of $\phi_r = 24.5^\circ$ ($n = 4$, $R^2 = 0.95$). The residual cohesion is assumed to be zero (see Table 1).

3.2. The La Mure landslide

This landslide near La Mure (French Alps) developed in varved clays. It has a semicircular and slightly concave form and its width and maximal length are 50 and 70 m respectively. Large fissures with up to 2 m-high scarps divide the slide into large blocks, which became progressively active from 1960 (Fig. 2).

The mean depth of the slip surface is 4.1 m and the inclination of the slip plane decreases gradually from 20° to 12° in a down-slope direction (Van Genuchten,

Table 1

An overview of the strength characteristics of the material for the three landslides (tr = triaxial test, ds = direct(re) shear test, rs = ring shear test)

Landslide	ϕ_p	c_p kPa	ϕ_r	c_r kPa	Texture
La Valette	26.8 (tr)	2 (tr)	24.5 (rs)	~0 (rs)	Sandy silt
La Mure	23 (ds)	3 (ds)	18.7 (ds)	1.6 (ds)	Clay/silt
Hau	21.6 (tr)	9.5 (tr)	21.5 (rs)	~0 (rs)	Sandy clay

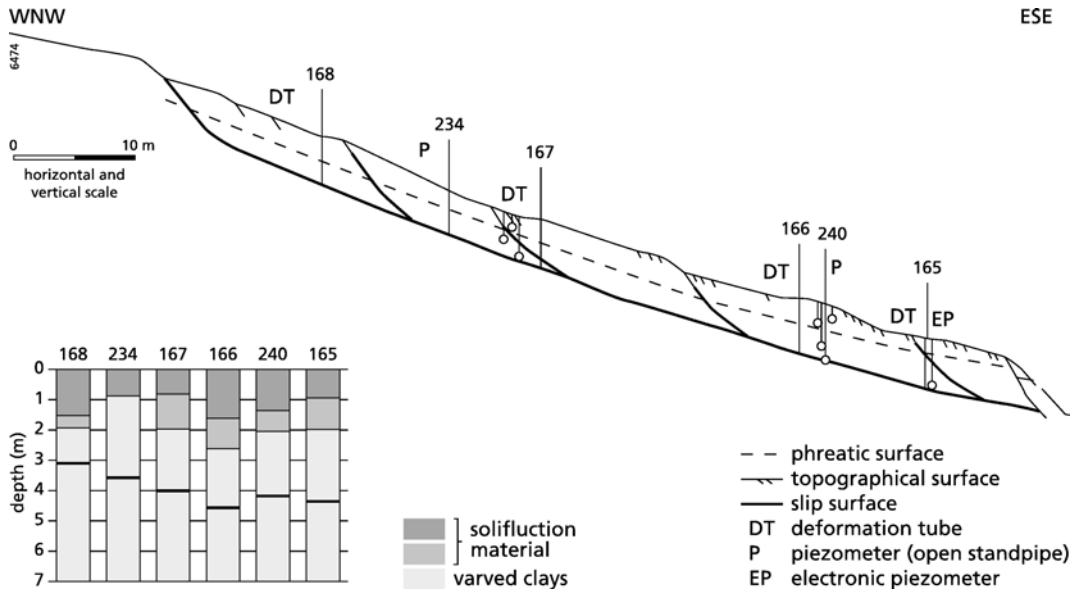


Fig. 2. A cross section of the La Mure landslide (after Van Genuchten, 1989).

1989; Nieuwenhuis, 1991). Displacement measurements at the surface and in depth have been carried out in the period 1979 until 1988. The observed displacements show a convergent pattern in a downslope direction. Moreover, the observations reveal that 80% of the total displacement is concentrated in a small shear band of 15 mm while the remaining 20% is attributed to slower creep deformations in the soil above the shear band with a maximum depth of 1.8 m (Van Asch and Van Genuchten, 1990).

The varved clays are clearly over-consolidated with OCR values varying between 10 and 20 (Van Genuchten, 1989). The clays consist of lamina of silt ($d_{50}=25\ \mu$) and pure clay ($d_{50}=2\ \mu$) with LL (Liquid Limit)=32–38% and PI (Plasticity Index)=13–18. Peak strength measured with direct shear and triaxial tests varies between $\phi_p=23^\circ\pm 2$ and $c_p=3\pm 2$ kPa. The residual strength was estimated by Nieuwenhuis (1991) from different residual strength envelopes obtained by re-shear tests in the direct shear box and ring shear tests under different normal stresses ($\phi_r=18.7^\circ$ and $c_r=1.6$ kPa; see Table 1).

3.3. The Hau landslide

The Hau landslide is a slow-moving landslide, which is active from at least 1955. The flow has a divergent character from the source area to the toe. The depth is 6 m while the slip surface is inclined at 14° . Van Beurden (1997) described the activity of the landslide in detail. The landslide consists of weathered bedrock material consisting of an alternation of sandstone and

marls, resulting in a sandy clay to clay-loam texture, (LL=31%; IP=10%). Measured peak strength varies between $\phi_p=21.6\pm 2.2^\circ$ and $c_p=9.5\pm 2.3$ kPa. The residual strength parameters obtained from ring shear tests amount to $\phi_r=21.5^\circ$ whilst $c_r\approx 0$ kPa (see Table 1).

4. Predictions for viscous behaviour of landslide materials

4.1. Laboratory vs. field results

The viscosity parameters of Eq. (3) were determined by strain-controlled ring shear tests for the three landslides considered. The ring shear apparatus was based on Bishop's design and is described in detail by Nieuwenhuis (1991).

Ring shear tests on material from the La Valette landslide were carried out under an effective normal stress of 86 kPa (Van Beek and Van Asch, 1996). Under normal stress the sample was consolidated until no further settlement was observed. An aperture was applied between the upper and lower ring of 300 μ . The resulting shear stress was recorded at strain rates between 0.5 and 26.2 cm per day. This covers the range of velocities observed at the La Valette landslide.

For the La Mure landslide, the ring shear tests were used to determine the viscosity of the narrow slip surface in which 80% of the total deformation took place (Van Genuchten, 1989; Nieuwenhuis, 1991). These strain-controlled ring shear tests were carried out under

a normal stress of 65 kPa and the resulting shear stress were recorded at strain rates between 0.5 and 26 cm per day.

The strain-controlled ring shear tests on the sandy clay material of the Hau landslide were carried out under a normal stress of 195 kPa with strain rates varying between 0.5 and 20 cm per day (Van Beurden, 1997).

Using Eq. (3), η_0 and b were optimised for τ_0 values that were calculated by means of Eq. (1b). Van Beurden (1997) determined the thickness of the shear band, h_m in Eq. (3), by filling two vertical cylindrical holes of 5 mm diameter that passed the sample with red sand and observed that more than 98% of the deformation occurred in a narrow shear band of 3 mm.

Table 2 shows the fitted parameters η_0 and b of Eq. (3) for the three different landslide materials. Optimisation involved maximizing the correlation coefficient between the measured and calculated velocities. The morainic material of the La Valette shows the highest viscosity and the highest sensitivity to changes in excess shear stress (higher b value). The viscosity of the materials measured in the laboratory seems to be rather low compared to the viscosity usually used to predict landslide velocities in the field. This can be verified for the three cases by comparing the laboratory values against field-scale estimates obtained from back-analyses.

Field-scale estimates of the viscosity parameters η_0 and b were calculated from the observed velocities using Eq. (3). At any time, the excess shear stress, $\tau - \tau_0$, was calculated using the infinite slope model given the residual strength values and the height of the groundwater. It was assumed that the thickness of the shear band in the field was the same as that measured in the laboratory by Van Beurden (1997) for the La Valette and Hau landslides (3 mm). In case of the La Valette landslide, the back-analysed η_0 -values are about three orders larger than the laboratory value (Table 2). Still, the best fit obtained for the back-analysis shows a rather

Table 2

A comparison between values of the viscosity characteristics of three different landslide materials obtained from ring shear tests and back analyses in the field

Landslide	Laboratory ring shear results				Back analysed from field velocities			
	h_m (m)	η_0 (kPa s)	b	R^2	h_m (m)	η_0 (kPa s)	b	R^2
<i>La Valette</i>	0.003	7.1E+04	4.7	0.99	0.003	1.23E+07	4.6	0.63
<i>La Mure</i>	0.003	5.1E+03	2.3	0.99	0.015	1.3E+07	2.2	0.82
					0.015	4.9E+06	2.2	0.81
<i>Hau</i>	0.003	9.2E+03	2.2	0.92	0.003	1.2E+05	2.1	–

Figures in italic are back analysed, assuming convergent flow.

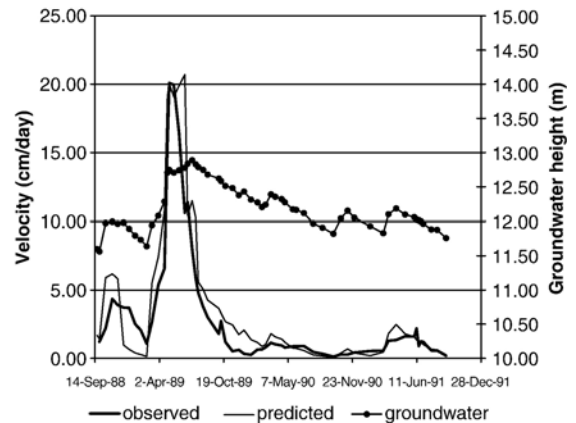


Fig. 3. Observed and predicted velocities and groundwater level of the La Valette landslide. Predicted velocities were calculated using constant values for η_0 and b calibrated over the total observation period.

low correlation coefficient. ($R^2=0.63$; Fig. 3). The single, optimised parameter set cannot predict all the surges during the observed period, a phenomenon that will be treated in more detail in the section below.

The back-analysed values of η_0 and b of the La Mure landslide were based on the available dataset of cumulative displacements and groundwater fluctuations for the period March 25th until April 7th 1987. The excess shear stress was calculated using the infinite slope model for two sections, one with a mean slip plane angle of 17° and the other of 12° . These sections determine respectively 86% and 14% of the stability (Nieuwenhuis, 1991). Again, the intrinsic viscosity, η_0 , is much higher than the value obtained in the laboratory (Table 2). Also, Nieuwenhuis (1991) calculated with a plastic flow model the extra contribution of convergent flow to the stability of this landslide. He found that the convergent flow effect increased the residual strength τ_0 with 8% of the total driving force. The increase in stability due to convergent flow results in a lower estimate of the viscosity but it still differs considerably from the laboratory results (Table 2). The observed versus calculated cumulative displacements, including the stabilizing effect of convergent plastic flow, are shown in Fig. 4.

The laboratory values of η_0 and b also overestimate the observed displacements at the Hau landslide. Field data on displacements are scarce at this site and yearly displacements were estimated varying between 0.35 and 1.6 m with groundwater levels varying between 0 m and 1 m below the ground surface (Van Beurden, 1997). The back-analysis used a constant groundwater level of 1 m below the surface throughout the year and a maximum total displacement of 1.6 m. Again, this returns a much higher viscosity than determined from the ring shear tests (Table 2).

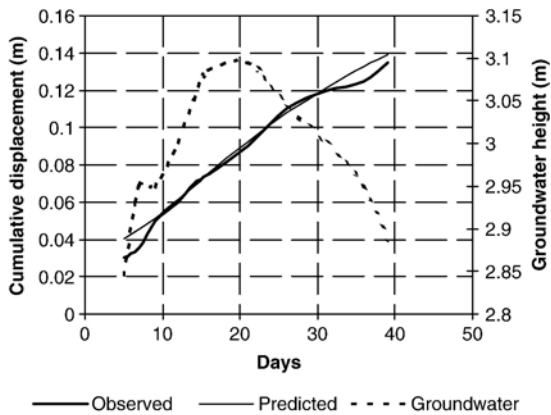


Fig. 4. Observed and predicted cumulative displacements and groundwater levels of the La Mure landslide, for the period 25th March until 7th of April 1987.

It should be noted that for the La Valette landslide and the Hau landslide the assumed thickness of the shear band, h_m , was only 3 mm, as measured in the ring shear apparatus. This value is probably too low in the case of an actual landslide and one may expect this thickness to be at least 5 to 10 times higher. This would put it in the order between 1.5 to 3 cm as observed for the La Mure landslide by Van Genuchten (1989). Consequently, the back-analysed intrinsic viscosity for the La Valette and the Hau landslide would increase proportionally, thus further distancing the laboratory values from the field-scale ones.

4.2. Discussion

The discrepancy between laboratory-scale and field-scale viscosity parameters observed here is not new; in a literature review Van Asch (1993) established that field-scale velocities appeared to be a hundredfold higher than laboratory-scale ones. This discrepancy may be explained by the following factors:

First, there may be errors in the calculation of the excess shear stress. In reality, the stability may be higher than calculated with the 1D infinite slope model, resulting in viscosities that are on the high side. We pointed already to the stabilizing effect of convergent flow for the La Mure landslide. Its incorporation resulted in a decrease in the intrinsic viscosity (Table 2). Notwithstanding, the corrected field-scale, viscosity still remains higher than the laboratory-scale one.

Moreover, the excess shear strength may not be constant over the landslide complex and the 1D infinite slope model is therefore too simplistic. Compression and extension that result from variations in the excess shear strength may result in additional uncertainty in the field-scale velocity.

Second, viscosity may comprise other components than pure viscosity. Apparent viscosity can be generated by negative and positive pore pressures and confound the truly intrinsic viscous behaviour of the material. Keefer and Johnson (1983) contributed strength development during movement for a hundred percent to pore pressure effects developed by a porous elastic solid sliding over a wavy rigid surface, completely disregarding the pure viscosity. Positive pore pressures may develop on the proximal side of the bumps of the wavy slip surface where material is in compression while negative pore pressures may develop at the distal side where the material is in extension. Velocities are then largely controlled by the saturated hydrologic conductivity of the material. Using this theory, Van Beurden (1997) was able to reach the observed displacements of 0.35 to 1.6 m/year using wave length bumps of the slip surface around 15 m with an amplitude around 0.6 m.

Third, Ter Stepanian (1963) suggests that the intrinsic viscosity η_0 increases linearly with the effective normal stress σ' . However, since the ring shear tests were carried out at the normal stress levels approaching those experienced in the field, this mechanism cannot explain the observed discrepancy between field and laboratory values.

Fourth, the thickness of the shear band in which the majority of the displacements occur, as observed at the La Mure landslide, is most times insufficiently known and cannot readily be reproduced under laboratory conditions. Bogaard (2001) determined viscosity values by means of stress controlled direct shear tests on undisturbed material, that was not pre-sheared as in the ring shear test. His laboratory viscosities could predict observed displacements measured on a slow-moving landslide in the field near Salins (French Jura), assuming a shear band of 15 cm.

Van Genuchten (1989) carried out stress controlled direct shear tests on pre-sheared and undisturbed material. The tests on pre-sheared samples delivered viscosity values in the same range as measured with the ring shear tests (Nieuwenhuis, 1991). However, direct shear tests on not pre-sheared varved clays material, delivered much higher viscosities that could predict the observed creep deformations in the 1.8 m deep zone above the shear band (Van Asch and Van Genuchten, 1990).

5. Temporal analyses of the mobility of the La Valette landslide

As a further evaluation, possible factors that influence the mobility of landslides are presented here.

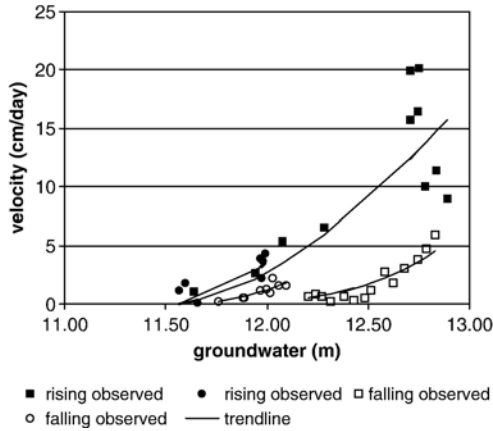


Fig. 5. The relation between velocity and groundwater height in the La Valette landslide, for respectively two periods of rising and falling limbs of the groundwater level.

Different responses of the velocity to the rise and fall in the groundwater level were already observed by Bertini et al. (1986) and this mechanism is evaluated for the La Valette landslide, for which the observed displacements of the La Valette landslide cannot be fully described by a single parameter set for η_0 and b as mentioned before (Fig. 3).

For the La Valette landslide, the change in velocity was analysed for two rising and two falling limbs of the groundwater level; rising limbs with sufficient data points to fit Eq. (3) is available in the periods from September 28th 1988 until November 24th 1988 and from February 22nd 1989 until July 13th 1989, in which period the highest groundwater level occurred. Similarly, falling limbs cover the periods from July 14th 1989 until January 12th 1990 and April 8th 1991 until September 10th 1991.

The different response of the velocity to the groundwater is shown in Fig. 5. Given the groundwater level, velocities and the rate of change are higher for the rising limb than for the falling one. To test whether this difference is consistent with the theory of Bertini et al. (1986) we fitted with Eq. (3) η_0 and b on the measured curves of Fig. 5. The results are shown in Table 3.

The findings show that during rising limbs the material seems to have lower intrinsic viscosities (η_0) and a lower dependency on excess shear stress (b) than during falling limbs of the groundwater level. Thus, material behaviour approaches that of a Bingham fluid during the rising limb and that of a plastic solid during the falling limb.

Alternatively, the findings could be interpreted in terms of changes in the pore pressure rather than in

Table 3

Back analysed apparent viscosity characteristics of the landslide of La Valette during rising and falling limbs of the groundwater level

Period	Piezometric level	η_0 (kPa s)	b	R^2
09-28-88 till 10-24-88	Rising limb	1.5E+05	1.6	0.86
02-22-89 till 07-13-89	Rising limb	2.1E+05	1.6	0.72
07-14-89 till 01-12-90	Lowering limb	1.1E+07	3.3	0.85
04-08-91 till 09-10-91	Lowering limb	8.1E+05	2.2	0.87

terms of intrinsic material properties. During movement excess pore pressure, both positive and negative, may develop due to compression and extension and this may result in apparent viscosity before they dissipate (Fig. 6):

If one assumes the cohesion is zero and the material at imminent failure, the threshold shear stress τ_0 can be calculated as follows from Eq. (3):

$$\tau_0(\sigma - u_{\text{normal}}) \tan \phi' \tag{4}$$

Following the conventional assumption of the infinite slope model, u_{normal} is calculated here for slope-parallel seepage:

$$u_{\text{normal}} = \gamma_w h_w \cos^2 \alpha \tag{5}$$

where γ_w the bulk unit weight of water, h_w the vertical height of the groundwater level and α the slope.

If the pore pressures during movement were to differ from slope-parallel seepage as a result of external

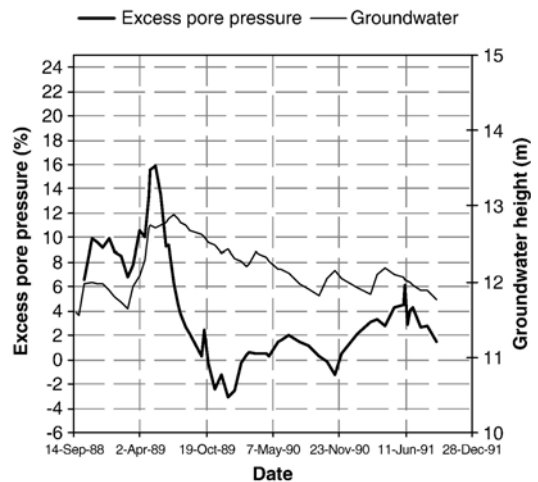


Fig. 6. Back analysed excess pore pressure in relation to groundwater height for the La Valette landslide from 14 September 1988 to 28 December 1991.

loading, the total denoted here as u_{mobile} , Eq. (5) can be rewritten as:

$$\tau_0 = (\sigma - u_{\text{mobile}}) \tan \phi' \quad (6)$$

The excess pore pressure can thus be defined as a ratio of the normal pore pressure as:

$$Ru_{\text{excess}} = \frac{u_{\text{mobile}} - u_{\text{normal}}}{u_{\text{normal}}} 100\% \quad (7)$$

At any moment u_{mobile} can be resolved provided one has values for η_0 and b , which are assumed to be intrinsic material properties in this case. The values of these parameters were determined for a period during which the velocities and the changes therein were low. As during this period external loading was of no concern, the pore pressure was taken to be equal to u_{normal} , i.e., slope-parallel seepage applies. The period from February 26th 1990 until June 25th 1990 was selected in this case. For this period the fitted parameters η_0 and b amount to $4.2 \cdot 10^6$ kPa s and 2.4 [–] respectively ($R^2=0.83$). Subsequently, these values were used to resolve u_{mobile} for the entire record of observed field velocities by combining Eqs. (3) and (6). The corresponding value of u_{normal} was calculated from Eq. (5). The resulting ratio of the excess pore pressure over the observation period is depicted in Fig. 6. As a general trend, excess pore pressure is positive and fluctuates with the groundwater level before the maximum groundwater level was reached. When the groundwater level recedes, the excess pore pressures decrease rapidly and even attain a level between the bench mark groundwater levels of parallel seepage calculated with Eq. (5). After this period of fluctuations, external loading seems to affect the pore pressure in the shear band less clearly and the excess pore pressures vary around zero.

5.1. Discussion

The theory of Ter Stepanian (1963; see above) states that the intrinsic viscosity of the material is a function of the effective stress. Thus, when the cohesion is zero, this intrinsic viscosity can be expressed as:

$$\eta_0 = \frac{\sigma'}{\lambda} \quad (8)$$

in which λ is a constant, which is called the flow factor. It shows that η_0 decreases with a decrease of σ' (rise in groundwater level) and increases with an increase of σ' (fall in groundwater level). This implies that the intrinsic viscosity is not necessarily constant over time as suggested in the Fig. 5 and Table 3. The effective stress, however, over the period February 22nd 1989 until July 14th 1989, varied

by 8% whereas the back-calculated intrinsic viscosity changed fifty-fold during that period (Table 3). So, the dependency of the intrinsic viscosity with the effective normal stress, as suggested in Eq. (8), is not as strong as the effect of external loading on the pore pressure. A fifty-fold increase seems unrealistic given the evidence that relationship of the intrinsic viscosity with the effective normal stress is generally weak (e.g., Bogaard, 2001).

The increase in viscosity with a rise in groundwater may be attributed to changes in the soil skeleton of the shear band or an overall increase in the thickness of the shear band. The rapid decrease in viscosity after the groundwater peak is harder to explain.

Sassa (1988) showed for debris flows that the pore pressure ratio increases with increasing velocities because of the vibration of debris flow particles and the lack of vertical drainage. He found that at velocities of about 1 m/s created an excess pore pressure of 50% (see Eq. (7)). For the La Valette landslide, we calculated an excess pore pressure of 15% for an observed velocity of 20 cm per day. However, it seems unlikely that the mechanisms of pore pressure increase, described by Sassa (1988) play a role in the La Valette landslide. The velocity is too low and the stress conditions too high in the La Valette landslide to generate these effects. Sassa (1988) found also that the excess pore pressure is maintained during a certain time even when the mass comes to rest. In our case however an immediate drop in pore pressure occurred during the drop in velocity.

It seems more feasible that the temporal pattern of movement is explained by rapid changes in total stresses and consequently changes in pore pressure under (partly) undrained conditions as suggested by Giusti et al. (1996). The La Valette landslide as described above is a complex landslide. It cannot be considered as a rigid moving body but there may be zones of compression and extension during movement (Picarelli et al., 1995). In certain zones the compression state may alternate in time with a state of extension. If this alternation is more or less synchronic with respectively a rise and drop in groundwater one can expect a hysteresis pattern as shown in Fig. 5, which was also mentioned by Leroueil et al. (1996).

Keefer and Johnson (1983), mentioned other effects influencing the pore pressure. They explained that the generation of negative pore pressures behind the distal sides of bumps in the slip plane creates a feed back mechanism during movement. Van Genuchten and Van Asch (1988) found also these feedback mechanisms in intermittently sliding blocks of the La Mure landslide. The movements of the blocks showed a stepwise character pointing to a “slip–stick” mechanism. Movements were generated by a rise in pore pressure due to

infiltrating precipitation. A stop of the movement did not coincide with a lowering of the groundwater. It was ascribed to irregularities in the slip surface (large stones, boulders) forming cavities during movement behind these boulders, creating large but very local suctions reducing the mean pore pressure in the slip surface.

6. Conclusions

Laboratory-scale viscosity parameters from strain-controlled ring shear tests are inconsistent with field-scale findings from back-analyses; for three slow-moving landslides with discrete shear bands where research was carried out by Utrecht University the laboratory values proved to be a 10 to 1000 times lower than those from the field. Viscosities derived from strain-controlled direct shear tests over a smaller strain range on not disrupted material, i.e., simulating a broader shear band, proved to be more apt to describe visco-plastic creep movements in a broader zone above the discrete shear band.

Apart from the difficulty of measuring the real viscosity in the shear band, the prediction of displacement rates in the field remains very difficult due to other field factors. The local geometry of landslides and the elasticity of their material generate an uneven distribution of the excess shear stress along the landslide. Zones of compression and extension create positive and negative excess pore pressures that result in a hysteresis in the apparent viscosity which renders it extremely difficult to predict the mobility of slow-moving landslides in detail.

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