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A visco-plastic model for slope analysis applied to a mudslide in Cortina d'Ampezzo, Italy

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Abstract

This paper describes a visco-plastic model which, using recorded groundwater levels, is capable of simulating the velocity trend in landslides. It also deals with the phenomenon of shear strength regain that occurs in montmorillonitic clays constituting slopes during periods when landslides are stationary. The model has been validated with long-term observations of a clay slope affected by a mudslide.

Keywords: deformation, landslides, numerical models, pore pressure, shear strength

Introduction

Attempts to calculate the rate of displacement of numerous landslides in clayey materials on low-gradient slopes using the classical limit equilibrium method of stability analysis, give rise to displacements and velocities well in excess of those automatically recorded in field. In the classical approach, soil strength is regarded as a unique resisting force. Movement may be initiated when the equilibrium between driving and resisting forces is modified by a pore water pressure increase, whereupon the shearing resistance decreases and, as a consequence, the driving forces prevail. A finite difference between driving and resisting forces is then available. For a definite pore pressure increase, this nett force is constant and therefore the landslide would move subject to constant acceleration with an indefinitely increasing velocity. This accounts for very high calculated values of velocity very soon after the beginning of the movement.

The foregoing means that limit equilibrium stability analysis should only be applied to determine the factor of safety of a slope in the phase immediately preceding the movement ($F=1$) while a different kind of analysis should be used in the successive phase. The latter phase can be analysed by means of a visco-plastic model capable of taking into account an additional component of the resisting forces that, as commonly observed in the field, causes the velocity to reach a constant value soon after the start of the movement. In practice, a nett force becomes available (the difference between the driving force and the shearing resistance) and, as movement starts, the velocity increase is simultaneously accompanied by an increase in viscous resistance. Since the viscous resistance is proportional to velocity, the available nett force tends to decrease to zero and the velocity becomes constant. From a conceptual point of view this dynamic behaviour model is comparable with a Bingham's model. It provides a complete explanation for the relationship between the pore pressure variations along the slip surface of a landslide and its velocity.

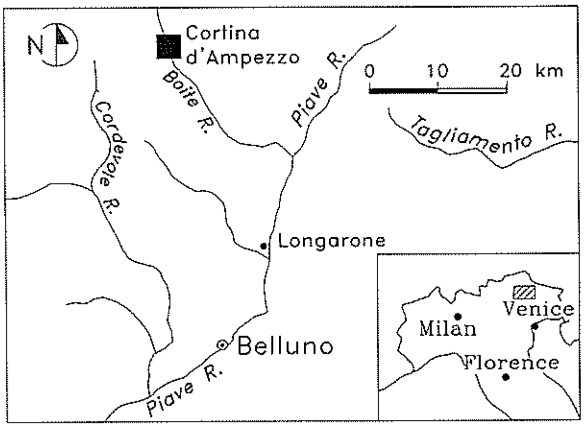

FIG. 1. Location map of the mudslide at Alverà, Cortina d'Ampezzo, Italy.

The Alverà Mudslide

From a numerical point of view, the model has been developed by considering an infinite slope (Skempton & De Lory 1957). Its validity has been verified against four years of displacement and groundwater level records. The mudslide chosen for the model validation affects a slope situated near Cortina d'Ampezzo, in the Eastern Dolomites, Italy (Figs 1 & 2). It has been described by

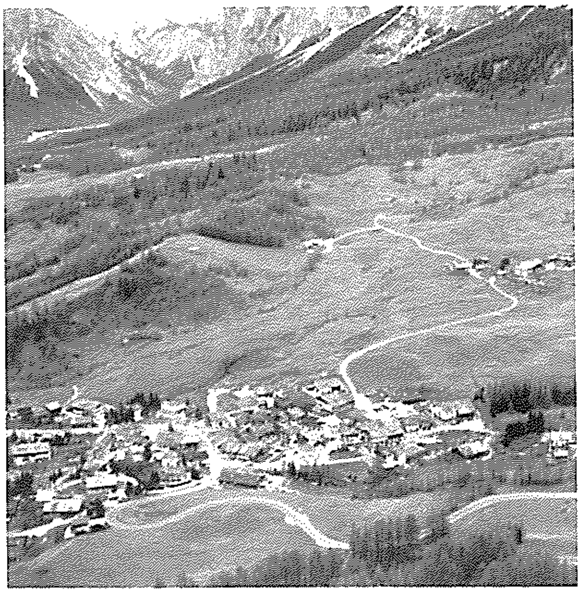

FIG. 2. Aerial view of the Alverà mudslide.

Angeli *et al.* (1991), Deganutti & Gasparetto (1991) and Gasparetto *et al.* (1994). The slope consists of clayey materials resulting from the weathering of the San Cassiano Formation, which mainly consists of alternating beds of sandstone, marl and clay that outcrop in the hillside (Fig. 3) above the landslide. The mudslide is about 1 km long with a slip surface in the lower part of the body, developed at a depth of about 5 metres. Analysis of borehole cores obtained from the lower part of the slope, permitted a distinction to be made between two separate layers.

The upper one (about 20 m thick), within which the landslide has occurred, consists of irregular poorly sorted blocks of the original rock dispersed in an argillaceous matrix. The material is widely affected by cracks. The lower layer consists of more consolidated homogeneous clays. A system of calcite filled fissures up to some centimetres thick within the lower layer suggests that there is abundant water movement from the overlying calcareous rocks (Fig. 3).

Observed hydraulic conditions indicate a groundwater table which is very close to the ground surface and which is subject to extremely rapid variations. A groundwater flow pattern parallel to the ground surface was assumed. A monitoring system consisting of inclinometer tubes and piezometric standpipes equipped with electric transducers for the measurement of the hydraulic head in the slope was installed in 1989. Each monitoring hole was also provided with a steel wire extensometer for the continuous measurement of the landslide displacements (Angeli *et al.* 1988, 1989). In addition, a network of benchmarks to provide topographic control allowed superficial movements to be detected. Besides all these instruments, a meteorological station was set up in order to record rainfall and snow-water equivalent precipitation values, air temperature and snow cover thickness. A sampling interval of ten minutes was chosen so that all the parameters involved in the landslide could be recorded, and the peak values occurring during critical events were certain to be measured.

Laboratory tests were carried out on samples from boreholes, drainage trenches excavations and trial pits. The results showed significant differences between samples collected at any depth in the slope and those taken from the slip surface in a trial pit dug in the lower part of the mudslide (Fig. 4). The values regarded as characteristic of the materials are listed in Table 1. Mineralogical analyses carried out on samples collected from the slip surface have shown that the material is montmorillonitic clay.

The visco-plastic rheological model

An attempt was made to compute the velocity which the landslide mass could achieve, once affected by known

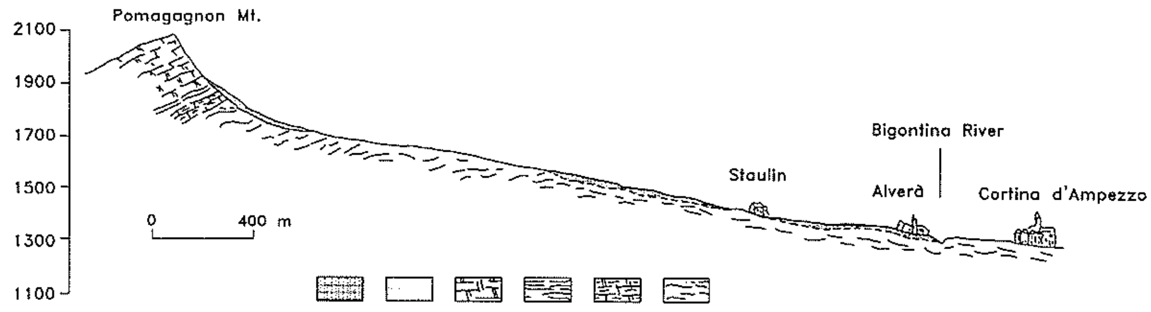

FIG. 3. Geological cross-section of the study slope: (1) Mudslide body; (2) Talus scree slope (Quaternary); (3) Cyclical dolomites (Dolomia Principale); (4) Alternating sequence of siltstones, marls and limestones (Raibl Formation); (5) Crystalline massive dolomites (Cassian Dolomite); (6) Prevalent overconsolidated clays with interbedded pelites, arenites and biocalcarenites (San Cassiano Formation) (From Angeli *et al.* 1991).

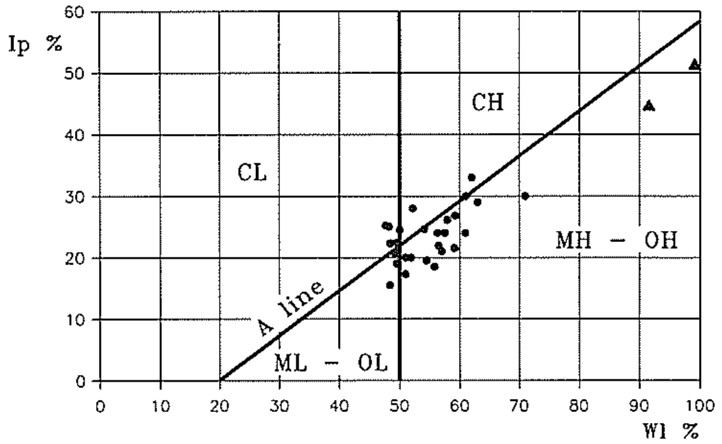

FIG. 4. Casagrande chart: the triangles indicate samples from the slip surface of the mudslide.

TABLE 1. *Geotechnical characteristics of the materials.*

Samples location	C.F. (%)	W_l (%)	I_p (%)	γ_{sat} (kN/m^3)	ϕ'_r (deg)
Any depth in the slope	30÷63	47.7÷71.0	15.5÷33.0	18.73	16.7÷25.5 (direct shear test)
Slip surface (2 samples)	68; 71	9.15; 99.1	44.5; 51.1	18.73	15.9 (ring shear test)

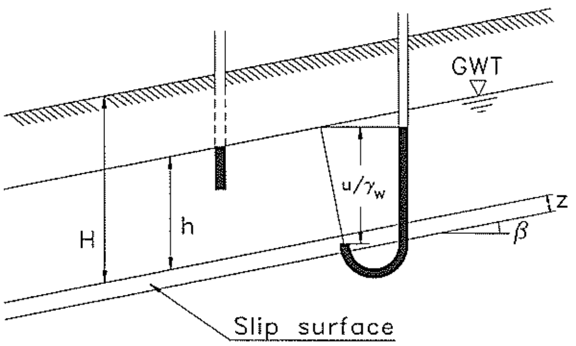

FIG. 5. An infinite slope with a viscous deforming layer (z).

piezometric changes. The balance of the driving and resisting forces along the slip surface in an infinite clayey slope (Fig. 5), which has undergone antecedent large movements (the intercept cohesion c' being assumed equal to zero) and without taking into account any other resistance parameter except for the internal friction angle provided the starting point. The analysis at critical equilibrium conditions, in terms of stresses, is given by

$$F=\frac{\tau_f}{\tau}=\frac{c'+(\sigma-u)\tan\phi'}{\gamma H \sin\beta \cos\beta}$$

where:
$\sigma=\gamma \cdot H \cos^2\beta$ (total normal stress)
$u=\gamma_w h \cos^2\beta$ (pore pressure)
$c'=0$ (cohesion intercept).
Equilibrium requires:
$\tau=\tau_f$
That is $\gamma H \sin\beta \cos\beta=(\gamma H-\gamma_w h)\cos^2\beta \tan\phi'$.

As soon as the pore pressure term exceeds the threshold value linked with the critical equilibrium, so that the shear strength decreases along the slip surface, the above equation becomes:

$\gamma H \sin\beta \cos\beta>(\gamma H-\gamma_w h)\cos^2\beta \tan\phi'$.

The difference between driving and resisting forces gives rise to an acceleration of the landslide mass:

$\gamma H \sin\beta \cos\beta-(\gamma H-\gamma_w h)\cos^2\beta \tan\phi'=m a/A$ (motion with constant acceleration), where A is the area of the slip plane portion corresponding to a sliding mass m. Since $a=\mathrm{d}v/\mathrm{d}t$ the value of the velocity can be easily calculated.

However this appears to be very high compared with measured values. Therefore, in order to achieve more realistic calculated values of velocity, some experimental approaches which take into account the viscous component of resistance were carefully examined (Pariseau & Voight 1979; Vulliet & Hutter 1988; Nieuwenhuis 1990; Van Genuchten 1990; Van Asch & Van Steijn 1991).

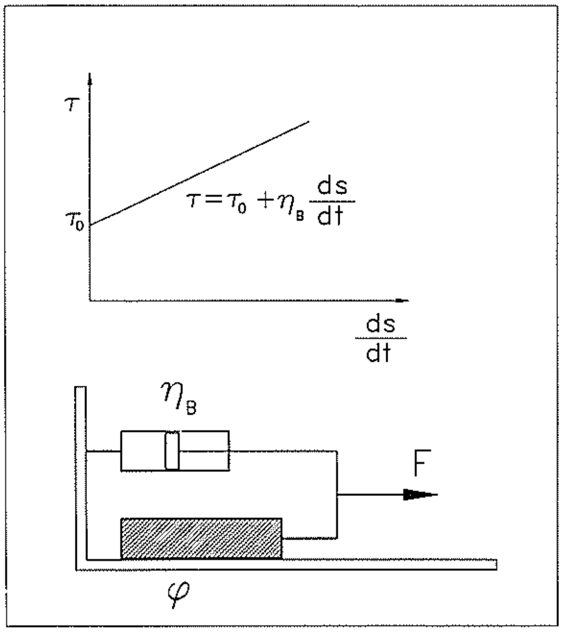

FIG. 6. Model of a Bingham body.

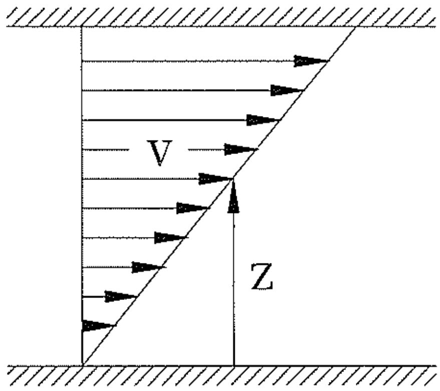

FIG. 7. Flow of a fluid induced by relative motion between two parallel flat plates.

These studies assume that the viscous resisting forces control the process of acceleration or deceleration of moving masses. They also describe some experimental relationships between velocity, viscosity and others parameters, obtained by means of the ring shear apparatus or of the viscometer (Locat & Demers, 1988).

Following the above suggestions, an attempt was made to describe the behaviour of this clayey soil with a Bingham's body model. This is best visualized in terms of a weight and a dash-pot joined in parallel (Fig. 6). Once the frictional resistance of the weight is overcome, it starts moving with a velocity controlled by the viscous resistance of the dash-pot $\tau=\eta_B \cdot v$, where η_B expresses the viscous characteristics of the oil in the dash-pot.

By analogy with fluid mechanics, the viscous resistance of the soil may be also expressed in terms of stresses as follows (Fig. 7): $\eta\,dv/dz$. Thus the overall resistance of a clayey material is expressed by $\tau_f+\eta\,dv/dz$, where $\tau_f=c'+\sigma'\tan\phi'$ as in the above equation for the shear strength of cohesive soil. The term $\eta\,dv/dz$ comes into play when the shearing stresses, inside a soil layer of thickness z (see also Fig. 5), exceed the yield shear stress τ_f and *laminae* of soil start moving at different velocities, parallel to the applied shearing stresses, sliding over each other.

At this time a resisting force directly proportional to the gradient of velocity and inversely proportional to z develops between the adjacent laminae. The factor of proportionality η, called the coefficient of viscosity, shows a large variability depending on the nature of the material.

Assuming a linear trend for the velocity profile along the z axis, as occurs in a Newtonian fluid, the term dv/dz can be substituted by v/z, where z corresponds to a well-defined thickness of deforming soil around the slip surface and v equals the velocity of the sliding mass. The difference between driving and resisting forces in terms of stresses now becomes:

$$\tau-\left(\tau_f+\frac{\eta\,v}{z}\right)=\frac{m\,a}{A}=\frac{m}{A}\frac{dv}{dt}$$

in which acceleration is a function of velocity.

A procedure of integration yields the following expression for the velocity:

$$v=\frac{[X-\exp(\ln a-Yt)]}{Y}$$

in which:

$$X=\frac{(\tau-\sigma'\tan\phi')A}{m} \text{ and } Y=\frac{A\,\eta}{m\,z}.$$

In order to apply the above velocity expression to the selected slope, an exact knowledge of geometry ($\beta=8°$; $H=5$ m), the character of the material ($c'=0$; $\phi'=15.9°$; $\gamma=18.73$ kN/m^3), the shear strength of the clays (τ_f), the forces acting (τ) and, finally, the parameters involved in the viscous resistance (η and z) were required. Due to the difficulty of determining separately the values of η and z with either laboratory or *in situ* tests, (any little variation of them strongly affects the calculated value of the velocity), they were included in the term $\eta_d=\eta/z$ called the coefficient of dynamic viscosity. As a consequence the contribution to resistance can be expressed as $\tau=\eta_d\,v$, which is analogous to the Bingham's expression for viscous resistance $\tau=\eta_B\,v$.

A first attempt value for η_d of 4.5×10^{11} N s m^{-3} was obtained in the laboratory by direct shear tests carried out on the fine-grained (passing No 40 sieve, 0.42 mm) matrix of the soil. The computer program, prepared for the velocity computation, progressively adjusted this

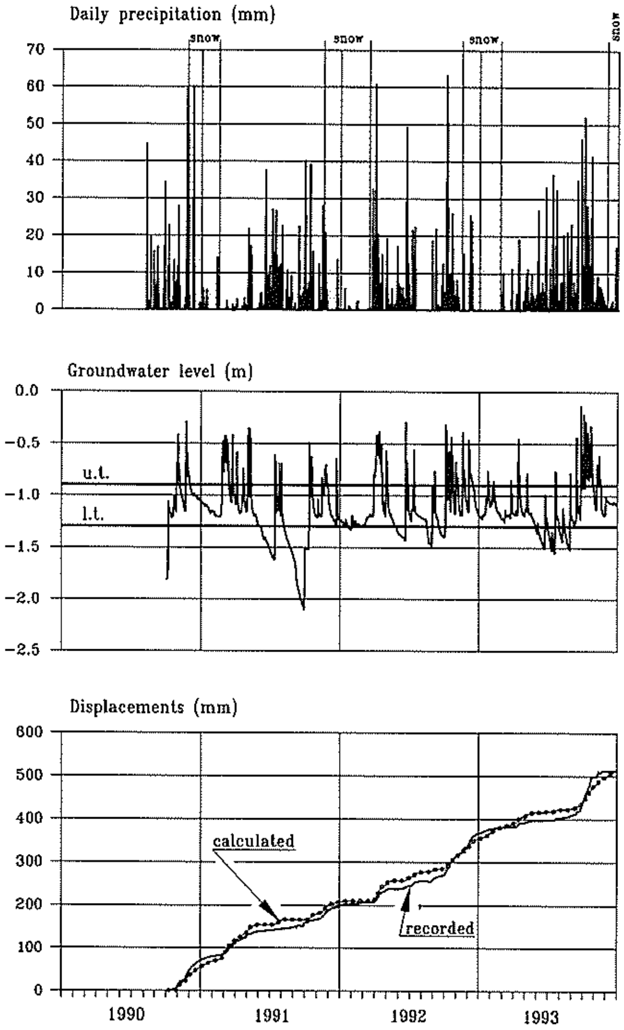

FIG. 8. Precipitation depth, groundwater level, recorded and calculated displacement in the Alverà mudslide.

value until the calculated values of velocity fitted the recorded ones. The final adopted value was $\eta_d = 7.8 \times 10^7\,\mathrm{N\,s\,m^{-3}}$. The reason this value is lower than the laboratory one can most probably be attributed to the presence in field of a certain amount of non-viscous and/or coarse-grained material.

In order to obtain better numerical results from the model, the computer program took also into account the presence of two different threshold values for pore pressures. These were detected when the complete time series of pore pressure and displacement data were analysed over the four-year period (Figs 8 & 9).

The lower threshold (l.t.) corresponds to a complete cessation of motion in the landslide, whereas the upper one (u.t.) appears to be connected with the reactivation of the movement. On the basis of the effective stress principle (Δh tan ϕ'), the interval of about 40 cm between the two thresholds corresponds to a shear resistance change of about 1 kPa. Hence, it is possible to infer that attaining of the upper pore pressure threshold determines a phase of landslide reactivation, leading to a decrease of the shearing resistance of about 1 kPa, whereas the attaining of the lower pore pressures threshold determines a phase of landslide rest leading to an increase of the shear strength of about 1 kPa.

This situation is outlined in Fig. 10 where the upper limit of shear strength variation ($\Delta\tau_f$) corresponds with periods of rest, whereas the lower limit of shear strength variation corresponds with periods of landslide activity. These two thresholds are inversely linked to the lower and upper thresholds (l.t. and u.t.) of the pore pressure curve. The model also takes into account when the pore pressure curve enters the area above the u.t.-line. According to the effective stresses principle, increasing the pore water pressure through this area, tends to decrease the shearing resistance, thus increasing the nett force available to movement or, in other words, increasing the velocity.

On the other hand, the model does not consider the increase in shear resistance when the pore pressure curve moves downward below the l.t.-line, because this represents an indefinitely stable field. This behaviour can be attributed to strength regain in the montmorillonitic clays during periods when the landslide is stationary (Gillot 1968; Zaruba & Mencl 1982; Skempton 1985; Nieuwenhuis 1990). Direct shear tests on material passing the No 40 sieve (0.42 mm) for normal stresses equal to field values, confirmed the existence of significant strength regain even after short periods of rest. The amount of regain tends to decrease with time of rest, suggesting that it becomes constant after a long time period (Fig. 11). The strength regain values obtained in the shear box tests were apparently slightly higher than the ones observed in field, the difference being due to the coarser grading in the latter case.

Using the pore pressure values recorded over the four year period for the study site, and taking into account strength regain by considering in the computer program two distinct pore pressure thresholds, it was possible to calculate values of velocity. These were then converted into displacements and compared with the recorded ones. The results are presented in Figs 8 & 9 which show very good agreement between the calculated displacement curve and the recorded one.

Discussion of the results and conclusion

The most important results obtained from the experimental analyses and case study on a low-gradient clayey slope can be summarized as follows.

The use of classical stability analysis on an infinite clayey slope gives an unrealistic landslide velocity

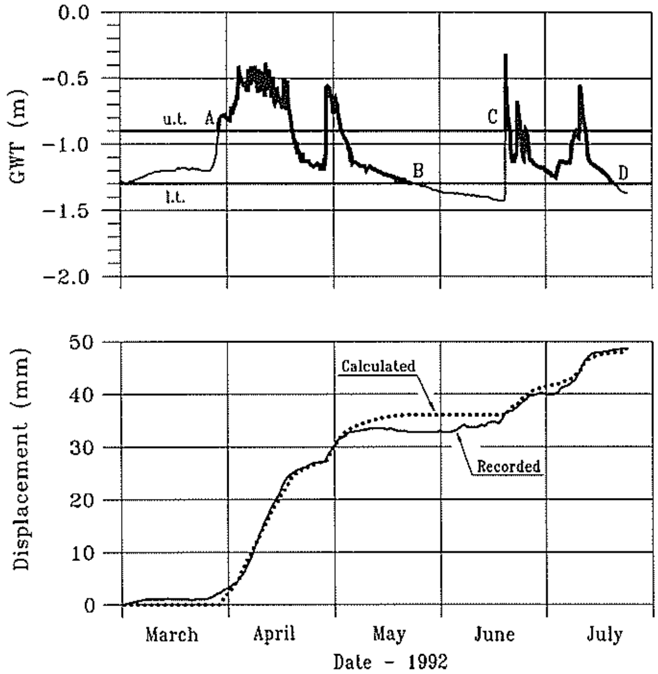

FIG. 9. A critical situation for stability: groundwater level, upper (u.t.) and lower (l.t.) piezometric thresholds, recorded and calculated displacement.

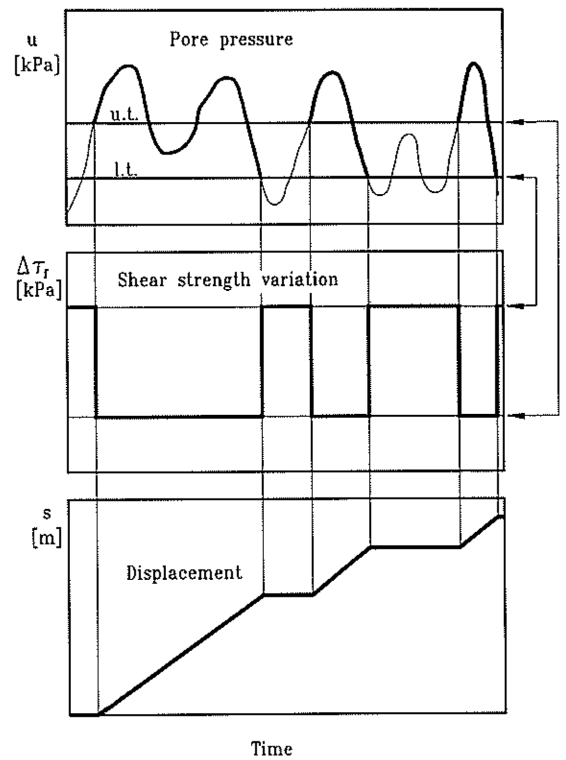

FIG. 10. Simplified scheme for the relationship between pore pressure thresholds, shear strength variation and movement.

development with an indefinitely increasing velocity (motion with constant acceleration), as soon as pore pressures become sufficient to induce any movement. This is in contrast to the results of monitoring real slopes where velocity tends to become constant soon after the initiation of movement.

The visco-plastic model in which the acceleration tends to decrease as the velocity increases, approaching to a steady state value with time suggested in the paper provides a more realistic model for velocity development. For this only a new pore pressure variation can produce a change in landslide velocity. This mechanism is supported by evidence from long-term recording carried out on the study slope.

The model takes into account the presence of two different pore pressure thresholds, as indicated by the analysis of a complete time series of pore pressure and displacement data. The physical effect of the two thresholds (instead of only one) is that after shear strength regain associated with periods of landslide rest, a higher pore pressure value is required to initiate movement than to maintain it. Typically once motion stops the pore pressure will be at the lower threshold while, at this point, the process of strength regain will restart. Hence the higher threshold must be exceeded for movement to occur again.

This type of behaviour is consistent with the mineralogical analyses of the clays involved. The presence of montmorillonite was confirmed in the clays concerned and such materials are well known for the phenomenon of thixotropic strength regain. Such a process was

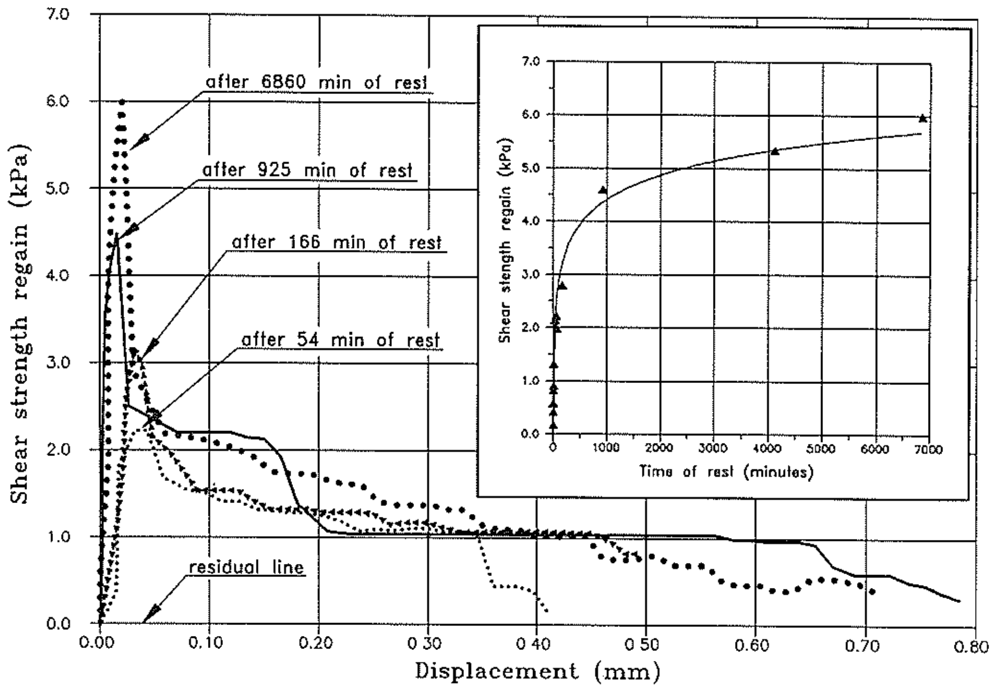

FIG. 11. Shear strength regain in direct shear tests after different times of rest (rate of movement = 0.0609 mm/min, $s_n = 71$ kPa).

confirmed by direct shear tests carried out on the fine-grained soil matrix of samples taken from the slip surface (Fig. 11).

The effects of the existence of two different pore pressure threshold values can be seen schematically in Figs 9 & 10. As shown the movement starts when the piezometric curve reaches points A or C and stops only when the curve definitely reaches points B or D, lying on the lower threshold. However, every time the piezometric curve falls into the zone between the two thresholds (l.t. or u.t.), but without reaching the lower one, movement continues. Thus the model takes into account contributions to movement which would not be explained if only one threshold value were being used.

The most important result of the model application is, however, the very good agreement between the calculated displacement curve and the recorded one (Figs 8 & 9). This correspondence is very significant since it is a first step in the prediction of the magnitude of landslide movements. Providing sufficient knowledge of the relationships between pore water pressures, rainfall and other climatic data is available, the displacements to be expected with particular precipitation intensities could be predicted.

In order to validate the results of the present research and to explore further its implications in terms of the prediction of landslide displacements, further research is needed. It is important that more case studies where strength regain has occurred coupled with carefully performed *in situ* large-scale shear box tests are carried out.

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References

ANGELI, M.-G., GASPARETTO, P., SILVANO, S. & TONNETTI, G. 1988. An automatic recording system to detect the critical stability conditions in slopes. *Proceedings, of the 5th International Symposium on Landslides, 10–15 July, Lausanne, Switzerland*, 1, 375–378.

——, ——, PASUTO, A. & SILVANO, S. 1989. Examples of landslides instrumentation (Italy). *Proceedings, of the 12th International Conference on Soil Mechanics and Foundation Engineering, 13–18 August 1989, Rio de Janeiro, Brasil*, 3, 1531–1534.

——, MENOTTI, R. M., PASUTO, A. & SILVANO, S. 1991. Landslides studies in the Eastern Dolomites Mountains, Italy. *Proceedings of the 6th International Symposium on Landslides, 10–14 February 1992, Christchurch, New Zealand*, 1, 275–282.

BROMHEAD, E. N. 1979. A simple ring shear apparatus. *Ground Engineering*, 25, 5, 40–44.

DEGANUTTI, A. & GASPARETTO, P. 1991. Some aspects of a mudslide in Cortina, Italy. *Proceedings of the 6th International Symposium on Landslides, 10–14 February 1992, Christchurch, New Zealand*, **1**, 373–378.

GASPARETTO, P., PANIZZA, M., PASUTO, A., SILVANO, S. & SOLDATI, M. 1994. Research in the Cortina d'Ampezzo area. *In:* CASALE, R., FANTECHI, R. & FLAGEOLLET, J. C. (eds) *Temporal occurrence and forecasting of landslides in the European Community (Final report). Science Research Development. European Commission Brussels*, 741–768.

GILLOT, J. E. 1968. *Clay in Engineering Geology.* Elsevier, Amsterdam.

LOCAT, J. & DEMERS, D. 1988. Viscosity, yield stress, remoulded strength, and liquidity index relationships for sensitive clays. *Canadian Geotechnical Journal*, **25**, 799–806.

NIEUWENHUIS, J. D. 1990. Seasonal strength variations on shear planes in shallow landslides. *Proceedings of the VI International Association of Engineering Geologists, Amsterdam*, 2257–2265.

PARISEAU, W. G. & VOIGHT, B. 1979. Rockslides and avalanches: basic principles, and perspectives in the realm of civil and mining operations. *In:* VOIGHT, B. (ed.) *Rockslides and avalanches*, **2**. Elsevier, Amsterdam, 1–95.

SKEMPTON, A. W. 1985. Residual strength of clays in landslides, folded strata and the laboratory. *Geotechnique*, **35**, 3–18.

——— & DE LORY, F. A. 1957. Stability of a natural slopes in London Clay. *Proceedings of 4th International Conference of Soil Mechanics and Foundation Engineering, London*, 378–381.

VAN ASCH TH. W. J. & VAN STEIJN, H. 1991. Temporal patterns of mass movements in the French Alps. *Catena*, **18**, 515–527.

VAN GENUCHTEN, P. M. B. 1990. On the viscous resistance in the movement mechanism of landslides in the French Alps. *Proceedings of the VI International Association of Engineering Geologists Congress, Amsterdam*, 1739–1745.

VULLIET, L. & HUTTER, K. 1988. Viscous-type sliding laws for landslides. *Canadian Geotechnical Journal*, **25**, 467–477.

ZARUBA, Q. & MENCL, V. 1982. *Landslides and their Control.* Elsevier, Amsterdam.

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