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THE VISCOUS CREEP COMPONENT IN SHALLOW CLAYEY SOIL AND THE INFLUENCE OF TREE LOAD ON CREEP RATES

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ABSTRACT

Creep tests carried out on silty-clay samples of a 'terres noires colluvium' show that even in shallow soils (1 m in depth) stress conditions can be high enough to initiate measurable creep displacements by the mechanism of continuous (viscous) creep. It is therefore not necessary to think exclusively of a shrink–swell mechanism (seasonal creep) in order to explain these slow movements in the soil. The creep tests also show a negative correlation between effective normal stress and the viscosity of the soil material. Therefore in shallow soils removal of an external load (forest clearing) might increase the creep rate at lower slope angles and therefore decrease the stability of the slopes. At higher slope angles removal of external load decreases the creep rate.

KEY WORDS Continuous creep Bingham's model Soil viscosity Forest clearing Slope stability

INTRODUCTION

Creep in shallow regoliths has been attributed to several mechanisms. The best well-known theory is that of the shrink–swell mechanism first introduced by Davison (1889), related to wetting and drying, and/or freezing and thawing of the soil (Kirkby, 1967).

The creep process was also considered to be a random diffuse soil movement (Culling, 1963, 1983; Flavell, 1987).

Little attention has been given to the fact that an important creep component in shallow top layers might be a viscous flow related to a slow continuous deformation of the soil under the influence of gravitational and externally applied loads. This process of continuous creep has already been discussed by Terzaghi (1950), and he stated that this mechanism of 'continuous' creep occurs lower than the depths of seasonal changes in moisture and temperature. Obviously higher stress levels at greater depths are needed in order to initiate this type of creep (see a.o. Ter Stepanian, 1965; Kojan, 1968; Yen, 1969; Van Asch, 1984). However, few authors have discussed the possibility that also at a shallow depth, where the greatest changes in temperature and moisture content occur, the viscous component has to be taken into consideration. An important study in this field was carried out by Fleming and Johnson (1975). Field measurements in montmorillonitic silty clay were taken and they assumed that during wet conditions in the soil, downslope displacement occurred by viscous flow in an active creep zone with a depth of about 1 m.

The objective of this paper is to discuss the occurrence of a continuous (viscous) creep component in shallow clayey soils by means of laboratory experiments. It will be shown that this viscous component in these shallow top soils plays an important part in the response of creep rate to external loads in case the surcharge of a tree cover on slopes.

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PHYSICAL SETTING OF STUDY AREA AND SOIL PROPERTIES

Soil samples were taken from slopes which have developed in the so-called 'terres noires' in the basin of Barcellonette (French Alps). The 'terres noires' consist of black marls which date from the Jurassic age, which form the autochthonous complex in this basin, and are overlain by allochthonous naps of limestone and sandstone formations (Flysch a Helmintoides).

The slopes which vary in angle from 15°–30° are covered by a colluvial cover consisting mainly of weathered material of 'terres noires' with small components of morainic material.

The soils have a clay component varying from 25–30 per cent and a silt component varying from 50–60 per cent. Atterberg limits show a plasticity index varying from 5–10 per cent. The peak strength of the material, measured with triaxial tests, shows a cohesion $c'_{peak} = 14 \text{ kNm}^{-2}$ and an angle of internal friction $\phi'_{peak} = 27{\cdot}4°$. The dry bulk density of the soils ranges from 15 kNm^{-3} to 19 kNm^{-3} and the wet bulk density from 17 kNm^{-3} to 21 kNm^{-3}. The clay mineral content consists mainly of chlorites and illites. The soil is slightly overconsolidated.

METHOD OF INVESTIGATION

A number of stress–strain experiments were carried out in order to determine the viscous creep parameters of the material. Creep models were evaluated to determine viscous creep potentials in shallow soils. For this purpose undisturbed soil samples were cut out of soil pits with an ironbox (dimension 5 × 10 × 20 cm).

The samples were allowed to become saturated by partly submerging the blocks in a water bath for several days. Next the samples were trimmed to a height of 2 cm and placed between two wetted porous stones in which metal strips were inserted in order to maintain a high friction between the surface of the sample surface and the stones (Figure 1). The stones and samples were constantly wetted and sealed by plastic foil in order to prevent the drying of the sample. The samples were placed under a slope angle, to induce a particular combination of effective normal stress (σ') in kNm^{-2} and shear stress (τ) in kNm^{-2} (see e.g. Yair and De Ploey, 1979). The shear strain over time was measured with a dial gauge which had a sensitivity range of 0·01 mm. The shear strain rate is defined as ds/dt measured over the depth dy of the sample, where ds is the displacement measured with the dial gauge (Figure 1). Thirty-two tests were used for analyses of the creep rate parameters with normal stresses of 11·1, 9·6, and $7{\cdot}2 \text{ kNm}^{-2}$ and shear stresses varying from 4·2 to

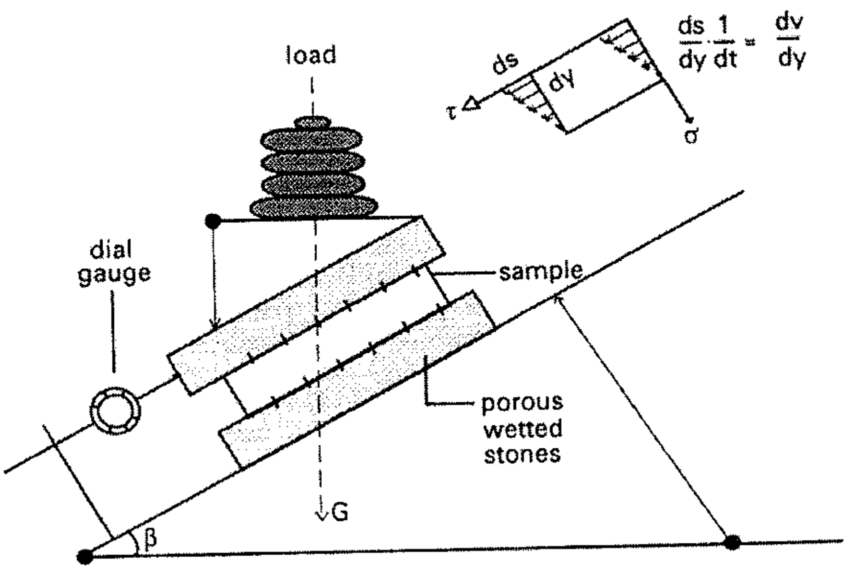

Figure 1. The experimental set up of the creep tests. For an explanation of the symbols, see text

6·6 kNm^{-2}. The tests were run for several days. The experiments show a decrease of the creep velocity over a period of time which is typical for the primary stage of creep deformation (a.o. Singh and Mitchell, 1968; Vyalov *et al.*, 1976; Tavenas *et al.*, 1978).

ANALYSES OF CREEP POTENTIAL OF SHALLOW SOILS BY MEANS OF BINGHAM'S MODEL

Creep velocity models which in the past have been evaluated for comparison with field measurements were based on a Bingham's rheological behaviour of the soil (a.o. Ter Stepanian, 1965; Yen, 1969). The Bingham's model can be described as follows:

$$\tau = \tau_0 + \eta \frac{dv}{dy} \tag{1}$$

where τ is a particular shear stress in a direction x in the soil, τ_0 is a threshold shear stress value below which no creep is considered to take place, η is a viscosity parameter and dv/dy is the creep rate dv in the direction x over the distance dy perpendicular to x.

In this model it is assumed that at a constant stress level the soil flows at a constant rate (η = constant) which in fact is not true, particularly during the first creep stage. It has further been assumed that below a certain stress level (τ_0) the amount of creep displacement is negligible. This last assumption has proved to be rather bold since many triaxial tests and more realistic experimental set-ups have shown that in fact there is no creep threshold value (a.o. Singh and Mitchell, 1968; Tavenas *et al.*, 1978; Yair and De Ploey, 1979). This might be of importance especially to the analyses of long-term geomorphological processes. Therefore we will discuss later a creep model without a threshold value. However, in this first analysis we assume an apparent threshold value in order to make sure that if stress conditions in shallow soil exceed this threshold value, creep flow is measurable.

In order to determine this threshold value a mean creep velocity was calculated for each creep test by presuming a constant creep velocity after a certain period of time. This means that a linear fit was made from the time-displacement (strain) graph for the period 24 hours after having started the creep experiment. We assume that after 24 hours the creep is in the second steady stage, while in fact the graphs still show a slight decreasing tendency of the velocity.

For each normal stress σ', a linear fit was made between the applied shear stress τ and the calculated mean creep velocity such as prescribed by the Bingham's Equation 1. The intercepts of these graphs (Figure 2) yield different τ_0 values for different normal stresses (σ'). The correlation of τ_0 with σ' delivers a linear relationship which can be described in terms of a Coulomb (yield) strength equation for the creep process:

$$\tau_0 = c_c + \sigma' \tan \phi_c \tag{2}$$

where τ_0 is the threshold value for creep (kNm^{-2}) and c_c (kNm^{-2}) and ϕ_c(°) are two strength parameters in terms of cohesion (c_c) and angle of internal friction (ϕ_c), which determine this creep threshold value. In our experiments the c_c value = 0.9 kNm^{-2} and the ϕ_c = 25.6°. Some authors (a.o. Yen, 1969; Nelson and Thomson, 1977) have suggested that the c_c and ϕ_c value (which are threshold values in our creep experiments) in Equation 2 correspond to the residual cohesion (c_r) and angle of internal friction (ϕ_r) of overconsolidated clays respectively, which can be determined by ring shear tests. In such a ring shear test the sample can be sheared infinitely in one direction after a shear plane has developed. This delivers so-called residual strength parameters of the material which are lower than the peak strength parameters. According to Skempton (1964) the residual cohesion of overconsolidated clays is practically zero whereas the residual angle of internal friction lies slightly below the peak friction angle of the material. If we look at the yield strength parameters (c_c = 0·9 kNm^{-2}; ϕ_c = 25·6°) which were determined by the creep tests (see above) and compare these with the peak strength parameters (c_{peak} = 14 kNm^{-2} and ϕ_{peak} = 27·4°), determined in the triaxial apparatus, then we might conclude that these yield strength parameters might be considered as residual strength parameters of the material.

The yield strength parameter in Equation 2 of the material gives the possibility to determine in a simple way the creep potential for shallow soils for different groundwater conditions. The following equation gives on the

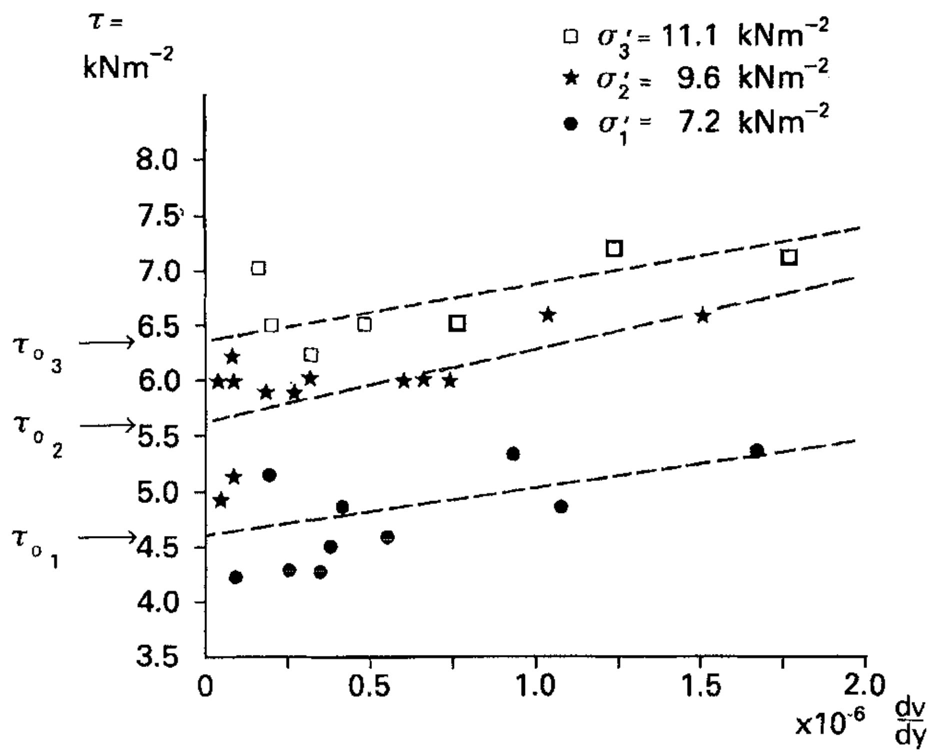

Figure 2. The relation between the mean shear strain rate and shear stress (τ) for three different effective normal stresses (σ')

bases of the infinite slope stability model a general relation between the critical slope angle for failure and the strength parameters c and ϕ.

$$\gamma_s z \tan \beta_{crit} - \frac{c}{\cos^2 \beta_{crit}} = (\gamma_s - m\gamma_w) z \tan \phi \tag{3}$$

in which γ_s and γ_w are respectively unit weight of wet soil and of water (kNm^{-3})°, c (kNm^{-2}), and ϕ (°) are strength parameters; $z(m)$ is the depth of the regolith (measured vertically); m is the relative height of the groundwater which runs parallel to the slope with respect to depth z of regolith, and β_{crit}, the critical slope angle (°) for failure.

If we substitute for c an ϕ in Equation 3 the peak strength values ($c'_{peak} = 14$ kNm^{-2} and $\phi'_{peak} = 27{\cdot}4°$) we can determine for different groundwater conditions the critical slope angle (β_{crit}) for rapid failure of the regolith by rupture along a slip plane. If we substitute for c and ϕ in Equation 3 the creep threshold values ($c_c = 0{\cdot}9$ kNm^{-2} and $\phi_c = 25{\cdot}6°$) we get for different groundwater conditions critical slope angles (β_{crit}) for the creep process. These threshold slope angles are far lower than the critical slope angles for rupture failure.

Given the measured threshold values and a thickness of the regolith of 1 m, the critical slope angle for creep is at maximum pore pressure conditions ($m = 1$) 16·0° and at zero pore pressure 28·5°. At zero pore pressure conditions the moisture content of the soil must still remain high, otherwise creep flow may decrease rapidly (Höwing, 1984). The determined threshold values and stability calculations show that detectable creep movements can be expected under wet conditions on steeper slopes in shallow regoliths. The most important reason is that —as has been shown in the yield strength equation for creep, which can be written as a Coulomb equation (Equation 2)—the cohesion term is practically zero. This means that the initiation of creep only depends on slope angle and pore water pressures and not on soil depth.

ANALYSES OF CREEP POTENTIAL IN SHALLOW REGOLITHS ON THE BASIS OF AN EMPIRICAL FIT OF STRESS-STRAIN RATE RELATIONSHIPS

It is generally accepted that creep rate is related to stress level and time as was also shown in the previous paragraphs. Furthermore it is generally believed that at least in the first creep stage there exists a negative linear relationship between the logarithm of creep rate and the logarithm of time (Singh and Mitchell, 1968; Vyalov *et al.*, 1976; Tavenas *et al.*, 1978). However, the relationship with stress level in terms of the principal stresses is rather complicated as was shown by the triaxial tests carried out by a.o. Tavenas *et al.* (1978). For the purpose of our next analysis we again assumed that the creep rate was constant in the latter stage of our creep experiments and we tried to find the best fit between the mean creep rate in the latter stage of each experiment (see the section, *Method of investigation*) and the applied shear stress τ and normal stress σ'. The best fit on our experimental results yields a linear multiple correlation between the logarithms of the creep rate and the stress parameters τ and σ'. A semilogarithmic relation between creep rate and deviation stress was found in many laboratory experiments (a.o. Singh and Mitchell, 1968; Tavenas *et al.*, 1978). The multiple regression carried out for the creep tests on these 'terres noires' colluvium clay samples yields the following equation:

$$\frac{\mathrm{d}v}{\mathrm{d}y} = e^{A\tau + B\sigma' + C} \tag{4}$$

in which A, B, and C are material constants which in the 'terres noires' colluvium are respectively 0·81, −0·36, −17·1.

The effect of the normal stress (σ') on shear strain (creep) rate (and therefore soil viscosity) is still being discussed (a.o. Ter Stepanian, 1965; Yen, 1969; Tavenas *et al.*, 1978; Höwing, 1984; Van Asch and Van Genuchten in prep.). Therefore it is interesting to note that our creep experiments show an inverse relation between the effective normal stress (σ') and creep rate.

Equation 4 can be used to develop a one-dimensional creep model for an infinitely long slope in order to assess the magnitude of creep flow in shallow soils and the effects of ground water seepage and external loads. In Figure 3 the x–y coordinate axes are drawn parallel and perpendicular respectively to an infinitely long slope with slope angle β. The depth of the regolith $= h$(m) and the depth of the groundwater measured from the soil surface $= y_w$(m) (Figure 3). Both are measured perpendicular to the slope surface. An external load representing a surcharge of trees, with a normal stress p (kNm^{-2}) and a shear stress s (kNm^{-2}) along the slope is also depicted in Figure 3. The shear stress τ at depth y is given by:

$$\tau = \gamma_s\, y \sin\beta + s \tag{5}$$

in which s is the unit weight of a given wet soil (kNm^{-3}).

Assuming a parallel seepage of groundwater along the slope the effective stress can be expressed by the following equations:

$$\sigma' = \{\gamma_s y - \gamma_w (y - y_w)\} \cos\beta + p \quad \text{for} \quad y \geq y_w \tag{6a}$$

$$\sigma' = \gamma_s y \cos\beta + p \qquad \text{for} \quad y \leq y_w \tag{6b}$$

The velocity at depth y for $y > y_w$ can be calculated by substituting Equations 5 and 6a in Equation 4 and by integrating this equation from y to h, assuming a boundary condition that $v=0$ for $y=h$. This yields the following equation for the velocity $v(y)$ at depth y ($y \geq y_w$)

$$v(y \geq y_w) = \frac{1}{k_1}\, \mathrm{e}^{k_2} (\mathrm{e}^{k_1 h} - \mathrm{e}^{k_1 y}) \tag{7}$$

in which

$$k_1 = A\gamma_s \sin\beta + B(\gamma_s - \gamma_w)\cos\beta$$

$$k_2 = B\gamma_w y_w \cos\beta + As + Bp + C$$

The velocity at a depth y for a point lying above the groundwater table ($y < y_w$) can be calculated by substituting Equation 5 and 6b into Equation 4 and integrating this equation from y to y_w with the boundary

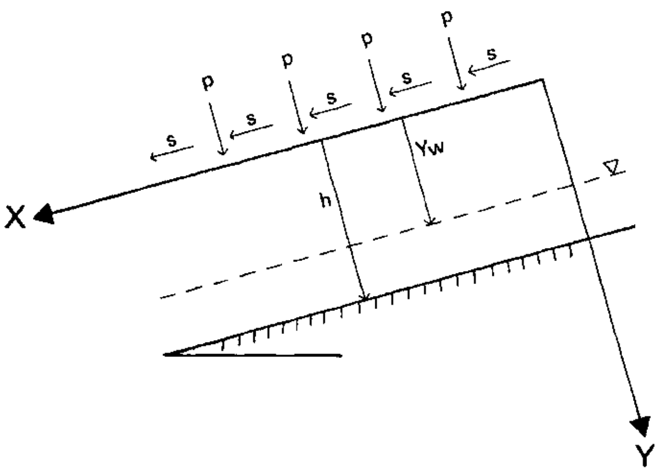

Figure 3. A one-dimensional creep model for an infinitely long slope. For an explanation of the symbols, see text

condition that for $y = y_w$ the velocity is $v(y_w)$ as calculated according to Equation 7. This yields for $(y \leq y_w)$

$$v(y \leq y_w) = v(y_w) + \frac{1}{k_3} e^{k_4} \{e^{k_3 y_w} - e^{k_3 y}\} \tag{8}$$

in which $k_3 = \gamma_s(A \sin \beta + B \cos \beta)$;

$$k_4 = As + Bp + C$$

and $v(y_w)$ the velocity at depth y_w calculated according to Equation 7. Thus the creep velocity profile is not a continuous one if there is a phreatic surface in the soil, because of a discontinuous increase of the effective stress σ' in a downward direction through the phreatic surface (compare Equation 6a and 6b).

Table I shows some creep velocities calculated by means of the model presented above and the material parameters (A, B, and C) which are determined by means of the creep tests. Creep velocities are calculated for points at the surface of a shallow regolith 1 m thick and for different slope angles and two groundwater depths. A surcharge from trees was also taken into consideration with a mean vertical stress of 3·5 kNm^{-2}. This was estimated from density measurements of trees (pinus sylvestrus) in the investigated area. The predicted creep rates are probably too high. The effect of tree logging is also to be seen in Table I. The model equations show that the effect may be an increase in creep rate at slopes with an angle below $\pm 25°$, while at steeper slopes, tree logging may cause a decrease in creep rate.

DISCUSSION AND CONCLUSIONS

The creep test carried out on silty-clay samples from the colluvium of 'terres noires' material show that even in shallow soils measurable creep displacements may occur due to a mechanism of continuous (viscous) creep. Therefore this mechanism must be considered when describing the process of so-called seasonal creep of shallow top soils. A Bingham's approach towards the rheological behaviour of the soil gives apparent yield strength values for creep (above which we can expect measurable creep rates), which probably are related to the residual strength parameters of the soil. The cohesion term in this yield strength equation is practically zero. In this case the initiation of creep only depends on a critical slope angle and a height of the phreatic surface and not on soil depth.

Table I. Calculated surficial creep velocities (mm per day) for different slope angles, under positive porewater and zero porewater conditions and for forested c.q. deforested slopes

Groundwater depth (Y_w in m)	Slope angle 15° trees	15° bare	20° trees	20° bare	25° trees	25° bare	30° trees	30° bare
0	3·1	5·1	9·7	12·0	33·2	31·3	121·5	87·3
1	0·7	1·2	1·6	2·0	4·0	3·8	12·4	9·0

A creep model which was based on the best correlation between measured creep rates and applied shear and normal stresses in the experiments gives surprisingly high creep velocities in shallow soils. An explanation for these high creep rates may be the dimension of the sample in the test. The relative high free face of 2 cm in relation to the length (20 cm) of the sample may influence the rate of the shear deformation. Creep tests carried out on varved clay materials with a direct shear apparatus (Van Asch and Van Genuchten, in press) resulted in considerably lower creep rates because the free face is partly supported by the stiff ring. However, at a larger shear deformation the creep process stopped (after about one day) probably due to the obstruction of the ring.

The creep rate figures in Table I assume fully wet conditions in the profile, even when there is no positive pore water pressure. One can imagine that fully wet conditions over the total profile are of short duration and that negative pore pressures prevail most of the time. Therefore it is important to assess the relationship between creep rate and negative pore water pressure in the soil. This is a field which has to be explored in order to get a more realistic estimate of creep rates caused by the process of continuous creep in shallow soils.

Theoretically we can expect an increase in effective stress and an increase in cohesive bonds between the clay particles (decrease of the C-factor in Equation 4) and therefore a decrease in creep rate. This has been confirmed by the field measurements of Fleming and Johnson (1975). The assessments of creep rate curves, which are related to cyclic periods of drying and rewetting over a longer period is also important to the study of creep flow in shallow soils. Fleming and Johnson (1975) for instance suggested that a drying period of a (montmorillonitic) soil is essential to the continuation of creep flow, because the rearrangement of particles during the drying period causes a relatively low resistance to creep deformation in the next wet period. They measured high creep rates at the start of a wet period with a rapid decrease during this wet period, which suggests a restart of the creep process at the beginning of the first stage of the creep process. Experiments carried out under laboratory conditions, with different types of clay minerals, should assess at which stage the creep process restarts after rewetting of the soil.

External loads on the slope have, particularly in shallow soils, a relatively large effect on the effective normal stress (σ') and thus on creep rates. This might be of importance for soil conservation practices. The model showed us a negative effect of forest clearing on slopes with angles lower than $\pm$ 25° and with shallow soils susceptible for creep. The removal of this external tree load results in an increase of creep rate (due to a decrease of the effective stress). This increase in creep rate might cause an increased rate of strength softening of the material (reduction of the peak strength values). Accelerated creep and failure by rupture are possible in this case on slopes, which were considered as stable with respect to their peak strength values, but which have become unstable due to reduction of this strength by the accelerated creep process. On steeper slopes ($> \pm 25°$) forest clearing may decrease the amount of creep rate and may therefore increase the stability of the slopes.

REFERENCES

Culling, W. E. H. 1963. 'Soil creep and the development of hillside slopes', *Journal of Geology*, **71**, 127–161.
Culling, W. E. H. 1983. 'Rate process theory in geomorphic soil creep', *Catena Suppl.* **4**, 191–214.
Davison, C. 1889. 'On the creeping of the soil cap through the action of frost', *Geological Magazine*, **6**, 255.
Flavell, W. S. 1987. 'Field investigation of a stochastic theory of soil creep', in Gardiner, V. (Ed.), *International Geomorphology 1986, part I*, John Wiley and Sons, 512–526.
Fleming, R. W. and Johnson, A. M. 1975. 'Rates of seasonal creep of silty clay soil', *Quarterly Journal of Engineering Geology*, **8**, 1–29.

Höwing, K. D. 1984. 'Das Kriechsverhalten gefüllter Gesteinstrennflächen und dessen Auswirkung auf die Langzeitstabilität von Felsböschungen', *Bochumer Geologische und Geotechnische Arbeiten*, **13**, Institut für Geologie, Ruhr-Universität-Bochum, 163 pp.
Kirkby, M. J. 1976. 'Measurement and theory of soil creep', *Journal of Geology*, **75**, 359–378.
Kojan, E. 1968. 'Mechanics and rates of natural soil creep', *Fifth annual Engineering Geology, Soils Engineering Symposium, Pocatello, Idako*, 233–253.
Nelson, D. J. and Thomson, E. G. 1977. 'A theory of creep failure in over consolidated clay', *Journal of the Geotechnical Engineering Division. Proceedings of the ASCE, SMI*, 21–44.
Singh, A. and Mitchell, K. J. 1968. 'General stress-strain time functions for soils', *Journal of the Soil Mechanics and Foundations Divisions. Proceedings of the ASCE, GTII*, 1281–1294.
Skempton, A. W. 1964. 'Long-term stability of clay slopes (4th Rankine Lecture)', *Geotechnique*, **14-2**, 77–102.
Tavenas, F., Leroueil, S., La Rochelle, P., and Roy, M. 1978. 'Creep behaviour of an undisturbed lightly overconsolidated clay', *Canadian Geotechnical Journal*, **15**, 402–423.
Ter Stepanian, G. 1963. 'On the long-term stability of slopes', *Norwegian Geotechnical Institute*, **52**, 1–14.
Terzaghi, K. 1950. 'Mechanism of landslides', in *Application of Geology to Engineering Practice*, Geological Society of America, Berkely, 83–123.
Van Asch, Th. W. J. 1984. 'Creep processes in landslides', *Earth Surface Processes and Landforms*, **9**, 573–583.
Van Asch, Th. W. J. and Van Genuchten, P. M. B. in press. 'A comparison between theoretical and measured creep profiles'.
Vyalov, S. S., Maslov, N. N., and Karanlova, Z. M. 1976. 'Laws of soil creep and long-term strength', *Proceedings of the XIIth Conference on Soil Mechanics and Foundation Engineering Paris*, vol. **I**, 423–431.
Yair, A. and De Ploey, J. 1979. 'Field observations and laboratory experiments concerning the creep process of rock blocks in an arid environment', *Catena*, **6**, 245–258.
Yen, B. C. 1969. 'Stability of slopes undergoing creep deformation', *Journal of the Soil Mechanical Foundation Division, American Society of Civil Engineers*, **95SM4**, 1075–1093.