SOME ASPECTS OF THE MORPHOLOGICAL AND GEOTECHNICAL PARAMETERS OF LANDSLIDES, WITH EXAMPLES DRAWN FROM ITALY AND ELSEWHERE

I am honoured to have this opportunity to interact with young Italian research workers in applied geology, particularly because, to judge from recent international conferences, your country is among the most active in this area. My contribution is based partly on recent work, chiefly published in the 5th and 6th International Symposia on Landslides in Lausanne and Christchurch, and partly on new material.

KEY WORDS: Landslide, methodology, stability analysis.

PAROLE CHIAVE: Frana, metodologia, analisi di stabilità.

Landslide classification

It was argued by Hutchinson (Lausanne 1988) that both morphological and geotechnical classifications of landslides are needed and proposals for both these were given. Under the heading of geotechnical classification, particular attention was drawn to the importance of any pre-existing shears which may be present. In many cases these have remained unhealed for 10,000 years or more and with low residual shear strengths, commonly in the region of $\phi_r = 9$ to $14^\circ$, with $c_r \approx 0$, they often continue to dominate slope stability. In some geochemical environments, healing of cementation of former slip surfaces has taken place, reducing or removing their destabilising influence. An example of such cementation by calcium carbonate in the Ferrandina landslide, Basilicata, is currently being studied with Professor M. del Prete.

Pre-existing shears generated by processes other than earlier landsliding present a particularly subtle hazard, as there is then generally little or no surface evidence from which the presence of these can be inferred. The most common sources of such slip surfaces are flexural shear (bedding plane slip), resulting from tectonic activity, and periglacial solifluction. Examples of these, and other, phenomena are given by Hutchinson (1988; 1991; 1992).

In the 1988 paper, a theoretical treatment of flexural slip was also provided. It was emphasised that shears of this type are much more widespread and important than is generally realised, a point also made by Cloos (1964). Further examples of flexural shears are given by Busk (1929), Billings (1954) and De Sitter (1956), while some instances of their major effects on slope stability are provided by Henkel & Yudhbir (1966), Binnie et alii (1967), Skempton & Petley (1967), Stimpson & Walton (1970) and James (1992).

In Italy, it appears that some formations, for example the scaly clays of the Apenines, are so highly tectonised that they exhibit a general, chaotically sheared structure rather than distinct individual shear surfaces or shear zones (A.G.I., 1979).

Earlier landslides are generally discernible in the field or from stereoscopic aerial photographs, and the associated pre-existing shears should not, therefore, come as a surprise. Such shears are difficult to anticipate, however, when the old slides are mantled by subsequent deposits, for example of loess. Cases of this nature are described by Tybár (1971) and by Záruba & Mencí (1976). The extensive ancient landslides in schist occupying the flanks of the Cromwell Gorge, New Zealand, are also partially disguised by mantles of loess, colluvium and other slope debris (Gillon & Hancox, 1991).

Abandoned cliffs, both on the coast and inland, form an important component of many landscapes and, particularly where extensive clayey strata are involved, can present serious stability problems (Hutchinson, 1967; 1973). With regard to the morphological/engineering geological classification of these, a useful development since the Lausanne paper is the recognition of three main types of abandoned clayey slopes, namely the normal (Hadleigh, England) type, those with a deficient accumulation zone suffering toe erosion, (as at Ok Ma, Papua New Guinea) and those with an accumulation zone (of Limnpe, England, type). In addition, each of these types, but most commonly the first, may be modified by subsequent periglacial solifluction (or other superficial processes). Details of all these types of abandoned slope are given by Hutchinson (1992).
Influence line approach

Cuts and fills

The influence line approach was introduced in 1977 in connection with the stabilisation of landslides by cuts and fills and by drainage (Hutchinson, 1977; translated into Italian 1980). Further treatment of this approach with regard to the effects of cuts and fills was given by Hutchinson (1984). For convenience, typical influence lines, undrained and drained neutral points and neutral lines, together with some inferences that can be made from these, are summarised in Fig. 1.

(*) However, if a trench drain, for example, were to be constructed in a stratum in which negative ground-water pressures existed, it may hasten the equilibration of these and hence promote instability.

Drainage

It is evident that drainage, producing a reduction in ground-water pressures, will always improve slope stability*. As a result, no neutral points arise in the case of stabilisation of landslides by drainage. However, influence lines may be derived which will indicate the optimum location of sub-surface drainage measures (Hutchinson 1984).

A simple starting point is provided by the infinite slope analysis. Fig. 2 shows the variation of $F_t/F_o$, the ratio of the factor of safety, $F_t$, with drainage to the original one, $F_o$, with no drainage, with slope angle, $\alpha$. The slide thickness, $z$, is taken as 10m and the ratio $\gamma_d/\gamma$ as 1/2. The original piezometric surface is assumed to coincide with the ground surface (G.L.); a uniform nominal lowering of 1m in the piezometric surface is considered. The variation in $F_t/F_o$ arises

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(*) Line of influence, points neutrals (a), (b) and neutral lines (c) for cuts and fills on a typical landslide (a). After Hutchinson (1977, 1984).
because, for a given slice width, the total slice weight is independent of \( \alpha \) but has to be resolved to give the normal total force on the slice base. The magnitude of the water pressure is also independent of \( \alpha \), but the corresponding water force on the base of the slice, being of course the product of this pressure and the area of the base, increases with \( \alpha \) until eventually the normal effective stress, \( \phi \), on the slide base falls to zero. Thus, as seen from Fig. 2, the effect on \( F_i/F_0 \) of a given drainage increases as \( \phi \) diminishes, particularly beyond about \( \alpha = 30^\circ \).

For finite landslides with curved slip surfaces, the effects of drainage are naturally more complicated. Fig. 3 illustrates, for a circular slip surface, influence lines for \( F_i/F_0 \) against \( x \) calculated by Bishop’s Simplified method, considering a nominal influence drainage (of \( h = 2m \)) for each slice in turn. It is of interest that, away from the ends of the slip surface, the minimum value of \( F_i/F_0 \) is located in the vicinity of the drained neutral point for cuts and fills, where the slope of the slip surface, \( \alpha = \phi_{\text{mob}} (23.15^\circ \) in this case). The maximum value of \( F_i/F_0 \) is located in the passive region at the toe, in the vicinity of the point where \( \alpha = \phi_{\text{mob}} \) in the particular case studied. Similar indications, for a non-circular slide, are given by Fig. 7 of Hutchinson (1977). Further work is in progress to improve the accuracy and check the generality of these preliminary results. From a practical point of view, it will be noted that the differences in \( F_i/F_0 \) are not large. The inevitable diminution in the stabilising effect of drainage near the points where the slip surface outcrops is normally made good by the slip surface replacement effect of, for example, trench drains, which become counterforts as the slip surface shallows (Hutchinson, 1977).

**Slip surface replacement**

Slip surface replacement is normally implemented through the construction of counterfort drains through the slip surface into stable ground. Thus, over the area of the trenches, the slickensided clayey slip surface is replaced by granular material. A local increase in \( \phi \), typically from 10 to 15\(^\circ\) on the pre-existing slip surface to 30 to 35\(^\circ\) in the granular drain fill, is commonly achieved (i.e. \( \Delta \phi = 20^\circ \)). The stabilising effect of such slip surface replacement forms a useful supplement to that deriving from the associated drainage which is, moreover, independent of any future clogging of the drains.

An approximate influence line for the effect of slip surface replacement is a shallow non-circular
The influence lines for the drainage of a circular rotational landslide ($c' = 10$ kPa, $\phi' = 30^\circ$, $\gamma = 20$ kN/m$^3$), the original $\phi_{mob}$ with no drainage = 23.15$^\circ$. The influence drainage is represented by $h = 2$ m).

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A line of influence for the drenchage of a front for scorrimento rotazionale ($c' = 10$ kPa, $\phi' = 30^\circ$, $\gamma = 20$ kN/m$^3$, $\phi_{mob}$ originario, senza drenaggio = 23.15$^\circ$. L'influenza del drenaggio è rappresentata da $h = 2$ m).

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Anchors

Influence lines may also be drawn for vertical or inclined anchor loads (Hutchinson, 1984). Influence lines showing the effect of varying $\theta$, the angle between an anchor of fixed position and a planar slip surface, are shown in Fig. 5 (after Menkiti, 1987). It may be noted that the locus of optimum $\theta$ values for the various anchor loads follows the relationship $\tan \theta_{opt} = (\tan \phi')/F$ (Hutchinson, 1977; Hoek & Bray, 1981). Fig. 6, also after Menkiti (1987), shows influence lines for the stabilising effect on a circular landslide of an anchor of fixed absolute angle but varying location from the slip toe. No load is allowed for in Figs 5 & 6.

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Fig. 4 — Influence line for slip surface replacement in a shallow, non-circular landslide ($c' = 0$, $\Delta \phi' = 20^\circ$ and $\Gamma = 20$ kN/m$^3$).

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In practice, neither of the above indications is generally followed. It is usually most economical to install anchors at slopes of between 15 and 30° to the horizontal (sometimes more steeply), for ease of drilling, anchor insertion and grouting (Xanthakos, 1991). With regard to the position of anchors relative to the slope toe, it is important to ensure that they will not be over-ridden by a smaller slide in the upper slope or undermined by such a slide below them. It is frequently desirable, for these and other reasons, to install several rows of anchors.

Neutral Zones

Attention is drawn by Hutchinson (1977) to cases where the failure surface of a landslide includes a planar section which slopes valleywards at the appropriate angle (i.e. \( \alpha = 0^\circ \) for fully undrained conditions, \( \alpha = +\phi_{mob}^* \) for fully drained conditions, or intermediate values for intermediate drainage conditions) for it to constitute a neutral zone rather than a neutral point. An example of an undrained neutral zone appears in Fig. 7 of that paper.

Of particular interest are planar slides where virtually the whole failure surface is inclined at \( \alpha = +\phi_{mob}^* \) and thus constitutes an extensive drained neutral zone. As a result, cuts and fills will have negligible effects on the stability of such slides except at the immediate toe and head. Their stabilisation is therefore generally effected by drainage, sometimes in combination with anchors and/or restraining structures. The geometry of these slides is also such as to encourage more rapid slide movements. The slide of 1986 at Timpone Hill, Senise, Basilicata (Fig. 7) provides an example of this type (Del Prete & Hutchinson 1988).

Undrained unloading & loading

Both these concepts are described in principle, and illustrated for artificial cuts and fills, by Bishop and Bjerrum (1960).
Undrained unloading

An important early field observation of the slow recovery of piezometric levels following largely undrained unloading in clays was made by Lutton & Banks (1970), who noted that piezometric levels in the Cucharacha Formation (a Tertiary clay shale with a high overconsolidation ratio and montmorillonite content) below the Culebra Cut of the Panama Canal were still as much as 9 to 10 metres below canal water level, 50 to 60 years after completion of the excavation (although further unloading was effected by the subsequent landsliding of the cut slopes). Furthermore, Lutton & Banks (1970) were able to show that this slow rate of pore-pressure recovery was consistent with the drainage paths in the Cucharacha Formation and its very low coefficient of swelling ($c_s$) values ($c_s$ 0.1 m$^2$/year) measured in the laboratory.

In 1973, Vaughan and Walbanke demonstrated, in an elegant paper, the generation of depressed pore-water pressures by undrained unloading in London Clay railway and road cuttings, and the subsequent slow equilibration of these pressures to release the well-known delayed failures of such cuttings. Again this field behaviour was linked theoretically with the low $c_s$ values for the London Clay of 0.8 to 3.2 m$^2$/year.

Against this background, it was evident that the possibility had to be considered that the deep-seated coastal landslides in rapidly eroding parts of the London Clay coastline, particularly at Warden Point, Sheppey, Kent, England, were occurring under partially undrained conditions (Hutchinson 1973). This was demonstrated to be the case through a thorough piezometric investigation of the Warden Point cliffs by Bromhead & Dixon (1984). Subsequently, the likely pattern of pore-water pressures and water contents just offshore, beneath the freshly eroded shore platform (Fig. 8), was inferred by Hutchinson (1986) and the practical implications of this discussed.

Undrained Loading

Undrained loading produced by the construction of artificial fills, particularly on soft clay foundations, has been much studied, as noted above. In the field of natural slopes, undrained loading by the accumulation of slope debris from upslope was invoked by Hutchinson (1970) to explain the otherwise
anomalous movements of the accumulation zone of the Belinge mudslide, on the London Clay cliffs of North Kent. This concept was checked and confirmed through well instrumented field studies by Hutchinson & Bhagdari (1971) and the importance of undrained loading was recognised in a wide range of both sub-aerial and sub-aqueous mass movements.

**Combined undrained loading and undrained unloading**

Some landslides are suffering both undrained loading at their rear and undrained unloading at their toe. For example, the coastal landslides at Folkestone Warre, Kent, experience undrained loading at the rear from time to time through the fall of masses of chalk from the rear scarp. One of the largest such falls, at Steady Hole in 1915, caused a local seaward movement of the landslide of about 50m, despite the fact that it was recurring on a non-brittle, pre-existing slip surface (Hutchinson et alii, 1980). The toe of these slides is now protected by a continuous concrete sea wall groynes and, in place, a concrete encased toe-loading structure. The effects of the coastal erosion which occurred before these sea defences were constructed are, however, still being felt. The consequent undrained unloading of masses of low c, Gault clay embedded in the slip debris in the vicinity of the landslide toe depressed the associated pore-water pressures, which are still sub-sea-level and equilibrating only very slowly (c. 0.15m/yr in one case) (Hutchinson et alii, 1980). These continuously rising pore-water pressures are tending to destabilise the seaward parts of the slipped masses, with evident results in one area. Vertical erosion of the unprotected shore platform may be contributing to this.

**Direct and indirect seismically induced failures**

Seismically induced landslides may be divided into direct failures, which occur synchronously with the seismic shock and indirect failures, which take place some hours, or even days, after this. Dynamic analyses of stability are needed for direct failures: in indirect failures, static analyses which allow for the indirect, delayed effects of the seismic shock, typically changes in ground-water conditions, are appropriate.

**Direct failures**

The velocity, V, period, T, and wavelength, λ, of seisms waves are related by the expression \( \lambda = VT \). For such waves, the periods and velocities associated with strong ground movements vary typically from 0.5 sec and 60 m/sec in loose soils to 0.1 sec and 400 m/sec in compacted fills and to 0.05 sec and 1200 m/sec in hard rocks. The corresponding wavelengths are thus of the order of a few tens of metres.

It is evident from the wave form that the average synchronous acceleration (simultaneous seismic coefficient) which will be applied to a slope or landslide of horizontal length \( L_h \) in a valleyward direction will reach a maximum when \( L_h = \lambda/2 \). It will diminish for values of \( L_h > \lambda/2 \), as one or more accelerations of opposite sign will then be introduced (Fig. 9) (Ambraseys, 1977). For the values presented above, a maximum landslide length, \( L_h \), for which earthquake forces will all act in the same direction, of between 15 and 30m is indicated (Hutchinson, 1987).

The implication of this argument is that direct seismically induced failures are likely to be of restricted dimensions in a valleyward direction (less than about 30 m). There is some field evidence which supports this hypothesis, though more work is desirable. For example, in the Izu-Oshima-Kinkai earthquake of 1978 in Japan, Okusa & Anma (1980) reported that on the east coast cliffs, composed mainly of andesitic lavas and pyroclastics, direct failures, chiefly rockfalls, were of modest size, \( L_h \) ranging from 20-30 m, the lengths along-slope from 50-100 m, the depths from 2-10 m and the slide volumes from 200-5000 m³. Similarly in the 1980 Irpinia earthquake in Italy, D’Elia et alii, (1985) report that the only direct failures were 5 relatively small rockfalls. These were followed, over the succeeding few days, by 11 larger, indirect failures of various types involving clays, referred to below.
Indirect failures

The majority of seismically induced landslides are indirect failures involving generally clayey strata. A careful statistical study is likely to show that, of these, a large majority have involved renewals of movement on pre-existing slip surfaces, as in the November 1980 failures of landslides and mudslides reported by D’Elia et alii (1985) and by Cotecchia & Del Prete (1984). As would be expected from the above hypothesis, these indirect failures are all considerably larger, particularly in the L, dimension, than the relatively few direct failures which occurred, and ranged generally from 0.2 to 3.0 km² in surface area. The most likely mechanisms to be bringing about such indirect, delayed failures are ground-water changes induced in or near the landslide by the earthquake, through, for example, faulting or cracking, or the generation of excess pore-water pressure by cyclic loading in or near the slip surface with subsequent migration (Lemos et alii, 1985).

Three-dimensional stability

All landslides are three-dimensional and yet the great majority of slope stability analyses are carried out on a two-dimensional basis. It is thus appropriate to explore the divergences between the three-dimensional and two-dimensional factors of safety, F₁ and F₂, and the parameters controlling these, conveniently in terms of the ratio F₁/F₂.

Undrained Conditions

For the undrained analysis of homogeneous cohesive slopes in terms of cylindrical failure surfaces, Gens et alii (1988) have provided a comprehensive treatment, essentially the three-dimensional version of Taylor’s (1937) curves. In summary, this shows that for slopes of inclination 60° to 90°, values of F₁/F₂ start to exceed 1.10 (i.e. an error of 9.1% in F₁) when the ratio of the overall, cross-slope length of the slide to the slope height falls below about 5. It is noteworthy that the four case records reported by Gens et alii (1988) in this category have F₁/F₂ values ranging from 1.15 to 1.30. For slopes of 30° to 45°, F₁/F₂ values begin to exceed 1.10 when the above ratio falls below about 5 for a depth factor of 1 and below 7.6 for a depth factor of 2. Two of the six case records reported have F₁/F₂ values in excess of 1,10, with a maximum of about 1.24. Approximate estimates of F₁/F₂ for non-circular, purely cohesive slip surfaces can readily be obtained using the approach quoted by Hutchinson & Del Prete (1985).

Drained conditions

Three-dimensional stability analyses of slopes with curvilinear failure surfaces under drained (effec-
tive stress) conditions are presented, for example, by Hungr et alii (1989). However, it is not immediately clear from these treatments at what geometrical proportions of the failing body, and at what values of the in situ horizontal earth pressure coefficient, \( K (= a_r / a_p) \), the errors of a purely two-dimensional analysis will become unacceptable.

This question is explored here for the simple case of a rectilinear block slide. Its dimensions and the ground-water conditions assumed are shown in Fig. 10. Shear strength parameters of \( c'_r = 0 \) and \( \varphi'_r \) are assumed to obtain on both the basal and the end slip surfaces. The three-dimensional factor of safety, \( F_3 \), is also assumed to be the same on all these slip surfaces. Highly simplified ground-water conditions are envisaged: a partly water-filled tension crack is taken at the rear of the potential slide block and the water pressure at the base of this crack is also assumed to obtain along the whole of the basal slip surface, with \( \text{parallel flow} \) throughout the whole block. In cross section (1-1 on Fig. 10) the lateral water pressures are thus taken as linear from the assumed phreatic surface (level with the water surface in the tension crack) to the constant basal water pressure defined above.

![Diagram](image1)

**Fig. 10** — Plots of \( D/B \) (and \( B/D \)) against \( K \) for \( F_3/F_2 = 1.10 \), in both the \( \text{dry} \) and \( \text{fully wet} \) cases, for \( \alpha = 10^\circ \) and \( 30^\circ \).

— Diagrammi di \( K \), per \( F_3/F_2 = 1.10 \), in relazione a \( D/B \) e \( (B/D) \) nei casi \( \text{secco} \) e \( \text{totalmente sатуре} \) per \( \alpha = 10^\circ \) ed \( \alpha = 30^\circ \).

The average unit weight, \( \gamma_a \), of the potential slide block is assumed to be constant, whatever the ground-water conditions.

On this basis, general equations are derived for \( F_3 \), \( F_2 \) and \( F_1/F_2 \). These are given in Tab. 1, where the special cases of a \( \text{dry} \) and a \( \text{fully wet} \) slide are also presented. The results are expressed graphically for slope angles, \( \alpha \), of 10 and 30\(^\circ\) and \( K \) values ranging from 0.5 to 2.5. Fig. 11 covers the dry case, Fig. 12 gives the results for the fully wet case and Fig. 13 shows the relationship between \( K \) and the depth to breadth ratio, \( D/B \) (and \( B/D \)), of the slide block for a fixed value of \( F_3/F_2 = 1,10 \). These curves provide guidance on the slide proportions and \( K \) values at which it becomes necessary to use a three-dimensional analysis, if unacceptable errors in a two-dimensional analysis are to be avoided. For example (Fig. 13), \( F_3/F_2 \) values begin to exceed 1,10 (corresponding to a
Three-dimensional stability analysis

Hendron & Patton (1985) insist correctly that, to be meaningful, stability analyses of the Vaiont slide must be carried out three-dimensionally. As no measurements of in situ stresses have been made in the vicinity and, moreover, the topography is very rugged, it is difficult to estimate the shear resistance on the upstream slide boundary. The method adopted by Hendron & Patton has the merit of simplicity, but may under-estimate this resistance.

Rockfalls from the landslide toe

Collapses of the steep front face of the Vaiont slide mass into the reservoir occurred in November 1960 (Müller, 1964) and possibly also in the early moments of the catastrophic failure of October 1963. As discussed by Hutchinson (1984, see Fig. 7), these unloaded an area well downslope of the relevant neutral points where the basal slip surface of the slide was nearly horizontal. As shown by Fig. 1, unloading of a slide toe between the drained and undrained neutral points causes an initial rise in factor of safety, followed by a reduction to below its original value. If unloading occurs downslope of the undrained neutral point, that is where the slip surface is inclined into the slope, an immediate reduction in factor of safety is produced. Either or both of these phenomena could have contributed to the initiation of the slide. Furthermore, as the basal slip surface at Vaiont generally cropped out well above the bottom of the Vaiont gorge, the destabilising effects of this mechanism would not have been counteracted by the build-up of debris in front of the slide toe.

Internal brittleness

Renewals of movement in landslides on pre-existing shears at residual strength are usually characterised by slow and relatively limited displacements, because of the essentially non-brittle nature of the slip surface (Skempton & Hutchinson 1969). The Vaiont slide of 1963 was a failure on a pre-existing shear and yet was one of the most violent failures known. How did this come about? Several possibilities are discussed by Hutchinson (1987). The most important mechanism, implicit in Hendron & Patton's work, is that of internal brittleness, outlined below.

A non-circular, non-planar slide, such as Vaiont, is generally unable to move until the slide mass is broken up by internal shears to convert it into a kinematically admissible mechanism. This applies even when its apparent factor of safety with respect to the bounding slip surface alone is at or below unity. For Vaiont, this phenomena was manifested very stron-
gly, as shown by the plot of Fig. 14 (Hutchinson 1987). Slide movements began when impounding of the reservoir reached an elevation of about + 600 m above sea level. Preliminary calculations show that, for the Vaiont slide, critical pool level in the reservoir was above top reservoir level. Thus, as the reservoir level rose above + 600 m, the apparent threedimensional factor of safety of the slide, on its boundary shears, was falling, as indicated approximately by the dotted line in Fig. 14, to a value in the region of 0.9 near top water level. However, despite this, the movements during this period were relatively modest (about 4 m between mid-1960 and 9th October 1963, from the area under the curve in Fig. 29 of Hendron & Patton, 1985), as indicated. This remarkable behaviour is ascribed to the action of an internal brake on movement, arising from the continuing ability of the rocks within the potential internal shear zones to resist the forces increasingly placed upon them. Eventually the internal shears failed. Because of the high brittleness of the Cretaceous limestones, marly limestones and cherts which form the great bulk of the slide mass, the strongly non-circular shape of the boundary slip surface in section and the low residual strength of that surface, this failure was abrupt. It transformed the slide mass into a kinematically admissible mechanism and produced a sudden drop, of the order of 10% (c. 1.0 — c. 0.9), in the three-dimensional, overall factor of safety. This, in turn, led to a sudden acceleration of the slide mass, which provided the impetus for the exceptionally rapid movements which followed. These are discussed briefly below.

**Brittleness on pre-existing shears produced by negative rate effects**

A further, important type of brittleness may be induced on pre-existing slip surfaces by rapid shearing. Lemos et alii (1985) have shown in rapid ring shear tests that, after an initial, peak, most clays exhibit a positive rate effect with increase of displacement rate, while others show a negative rate effect. The latter are, naturally, of particular concern.

**Tab. 2 — Values of the ratio B/D below which the value of F3/F2 exceeds 1.10.**

<table>
<thead>
<tr>
<th>K</th>
<th>α = 10° dry</th>
<th>α = 10° fully wet</th>
<th>α = 30° dry</th>
<th>α = 30° fully wet</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,0</td>
<td>10,3</td>
<td>10,6</td>
<td>13,3</td>
<td>16,3</td>
</tr>
<tr>
<td>2,0</td>
<td>20,6</td>
<td>21,2</td>
<td>26,7</td>
<td>32,5</td>
</tr>
</tbody>
</table>

— Valore del rapporto B/D al di sotto del quale il valore di F3/F2 supera 1.10.
Recent rapid ring shear tests on gouge from the slip surface of the Vaiont slide (Tika-Vassilikos & Hutchinson, in prepn) reveals that this exhibits a high negative rate effect. Fig. 15 illustrates the results for Sample 3 ($w_p = 30\%$, $w_a = 49\%$, C.F. ($< 2\mu$) = 27\%) which indicate that the residual strength at displacement rate of 2600 mm/min is about 60\% below the slow residual value, after a shear displacement of less than 1 metre. The equivalent $\phi'$ value is little over 4\%. This effect, if representative, is sufficient to explain the high speed and run-out of the Vaiont slide without the need to invoke slip surface heating.

Of the several materials reported as exhibiting negative rate effects in residual shear, that from the Vaiont slip surface is the only one so far to be associated with a catastrophic landslide. More such materials should be sought. The headward areas of analogues to Vaiont, or wurststroms which commenced with a rock slide rather than a rockfall, should provide promising sites for such a search.

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SOMMARIO

Sono onorato di avere questa opportunità di incontrare i giovani ricercatori italiani di Geologia Applicata, soprattutto perché, a giudicare dalle recenti conferenze internazionali, il vostro paese è tra i più attivi in questo settore di ricerca. Il mio contributo è basato in parte sull'attività recente, i cui risultati sono stati pubblicati negli Atti del 5° e del 6° Siimposio Internazionale sulle Frane (Losanna 1988 e Christchurch 1992), ed in parte su materiale inedito.


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