THEME 3

Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movements

Experiences with corrective measures applied to slope movements

Evaluation de l'efficacité des mesures de stabilisation par rapport aux conditions géologiques et aux types des mouvements du terrain

Expériences acquises par l'exécution des mesures de la stabilisation

General report / Rapport général

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ASSESSMENT OF THE EFFECTIVENESS OF CORRECTIVE MEASURES IN RELATION TO GEOLOGICAL CONDITIONS AND TYPES OF SLOPE MOVEMENT

EVALUATION DE L'EFFICACITÉ DES MESURES DE STABILISATION PAR RAPPORT AUX CONDITIONS GÉOLOGIQUES ET AUX TYPES DES MOUVEMENTS DU TERRAIN

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Summary:

The General Report consists of three parts. In Part 1, the various types of corrective measures are briefly reviewed. Attention is then concentrated on the two, most used measures; modification of the slope profile by excavation and filling, and drainage. An analysis is made of the optimum positioning of corrective cuts or fills, making use of the influence line concept, borrowed from structural engineering. In this way a neutral point, neutral line and neutral zone are defined for circular and non-circular landslides and for various values of $\bar{B}$ with respect to the applied change in total stress. Drainage is then discussed in more detail, with particular attention being given to horizontal drains and to trench (and counterfort) drains. Performance data for trench drains in the U.K. are then reviewed and analysed. From this a tentative basis for design is developed.

In Part 2 the papers contributed to Theme 3 of the Symposium are reviewed. The clogging of drainage systems by siltation or by geochemical effects, is also discussed. Finally, in Part 3, some suggestions are made as to the desirable directions of future research. An extensive list of references is provided.

Résumé:

Le Rapport Général se compose de trois parties. Dans la première partie les différentes mesures de stabilisation sont brièvement examinées. L'attention est ensuite portée sur les deux mesures les plus courantes: la modification du profil de la pente par excavation et remblayage et par drainage. Une analyse est faite de l'emplacement optimal des tranchées ou des remblais de stabilisation qui utilise le concept de la ligne d'influence emprunté au génie structural. De cette manière un point neutre, une ligne neutre et une zone neutre sont définis pour les glissements de terrain circulaires et non-circulaires ainsi que pour des valeurs diverses de $\bar{B}$ en tenant compte du changement apporté à la tension totale. Le drainage est ensuite discuté de façon plus approfondie en insistant plus spécialement sur les drains horizontaux et les drains tranchés (et contreforts) en particulier. Les résultats d'essais sur l'utilisation de drains tranchés dans le Royaume Uni sont présentées et analysées. A partir de ces données une tentative de mise au point de drains tranchés est developpée.

Dans la deuxième partie les contributions écrites du Sujet 3 sont discutées, ainsi que le colmatage des systèmes de drainage par dépôt de limon et par effets géochimiques. Enfin, dans la troisième partie sont présentées des suggestions sur l'orientation à donner aux futurs projets de recherches. Une liste détaillée de références est donnée.

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Introduction

In view of its context, this General Report concentrates on the stabilisation of slopes consisting of soils and rocks in their natural state, in either natural slopes or cuttings, and excludes ones that comprise chiefly fills.

The theme chosen by the Organising Committee is a good one. It directs our attention specifically towards the assessment of the efficacy of stabilization measures, which a reading of the literature shows to have been a matter largely neglected hitherto. This situation has doubtless arisen partly from a natural desire to close the file on a job and partly from the reluctance of the client or owner to accept continuing expenditure on long-term monitoring.

In accordance with the wishes of the Organising Committee, the General Report consists of three parts. The first reviews the main aspects of the theme in the light of the present state of knowledge, the second evaluates the contribution of the submitted papers while in the third, suggestions are made as to the nature of the still unsolved problems and the direction of future research.

1 - REVIEW

General

Many general reviews of methods of slope stabilization have been made. Some of the more recent are by Root /1953, 1958/, Baker & Marshall /1958/, Brawner /1959/, Mehra &Natarajan /1966/, Záruba & Mencl /1969/, Duncan /1971, 1976/, Schweizer & Wright /1974/, Smith /1974/ and Broms /1975/. A useful review of methods particular to rock slopes is provided by Peckover & Kerr /1976/.

The main methods of stabilization used are summarised briefly below. They may be employed singly or in combination.

1. Excavation &filling

a/ Excavate at toe until stability is attained : a crude method which relies on stimulating retrogression of the slide until its average slope is sufficiently gentle to be readily maintained. Large quantities of excavation are generally involved. The classic application of the method is in the Culebra reach of the Gaillard Cut on the Panama Canal /Lutton & Banks,1970/.

b/ Remove and replace slipped material: either wholly by free-draining material /Symons,1970/ or, more economically, by recompacted slip debris provided with drains /Newman,1890, Duncan,1971/. The method is obviously applicable only to slips of modest size. A variant of this method is to destroy pre-existing shear surfaces at shallow depths by digging out, remoulding and recompacting the excavated material /Weeks, 1970/.

c/ Excavate to unload slope: either by a general flattening, with or without berms /Baker & Marshall,1958, Broms,1969/, or locally at the head of a slide /Peck & Ireland,1953; Lutton & Banks,1970/. As discussed later, it is important that such excavations are correctly positioned.

d/ Filling to load slope: generally by means of berms, possibly combined with other gravity structures, such as a gabion or reinforced earth walls, at its toe /Viner-Brady, 1955; Broms, 1969; Early & Skempton,1972; Záruba & Mencl,1976/. Again the correct positioning of stabilising fills is of great importance, as is their proper drainage. The disturbing effects of embankments that have, unavoidably, to be placed in destabilizing positions can be reduced by the use of light-weight fills, such as fly-ash.

2. Drainage /Cedergren,1967, 1975, Rat,1976/

a/ Lead away surface water: this should generally be done immediately /Záruba &Mencl,1969/.

b/ Prevent the build-up of water in tension cracks: this should also be attended to straight away /East,1974/. Attempts to seal such cracks against surface inflow usually fail, as any seal will tend to be broken by the slightest further movement. It is better, therefore, to make arrangements to drain cracks.

c/ Blanket slope with free-draining material, with filters as necessary: This combines measure 1 /d/ with drainage /Root, 1958/ and is particularly effective in the case of slopes exposed to rapid drawdown /Skempton,1946, Finzi & Niccolai,1961; Cedergren,1967; Klengel et al. 1974/.

d/ Trench drains: these are generally narrow* and aligned directly downslope /Early & Skempton ,1972/, thus largely avoiding the risk of reactivating the landslide being treated. They are sometimes supplemented by shallower drains laid in a chevron or herring-bone pattern /Duvivier,1940/. An earlier version of the trench drain is the counterfort drain. In this the invert is located in firm ground beneath the slip surface so that, in addition to reducing ground-water pressures, the drains also provide some mechanical support /Gregory,1844; Collin,1846/. Open or gravel-filled drain trenches running cross-slope are sometimes built above the crest of a slip or slope, when they are usually termed interceptor or cut-off drains /Toms & Bartlett,1962; Smith,1964/. Shallow ones merely intercept surface run-off; deeper ones are intended to intercept ground-water flowing towards the slope. A deep cut-off trench, extended downwards by drain holes into a drainage gallery, was constructed at the head of the colluvial slope being stabilized at Weirton /D'Appolonia et al.,1967/. Care must be taken to avoid siting cut-off drains so that they could act as a tension crack in any future landslide.

e/ Horizontal drains: usually drilled into a slope on a slightly rising gradient and provided with perforated or porous liners /Smith & Stafford,1957; Root,1958; Robinson,1967; Henke, 1968; Nonveiller, 1970; East,1974; Tong & Maher,1975; Brandl,1976/. The maximum practicable length of such drains is generally around 100 m, though one 231 m long is reported by Záruba & Mencl /1976/. Lengths of up to about 60 m are more common. In slides of large scale, horizontal drains can be used to advantage in conjunction with vertical drainage shafts /Nat. Conf. Landslide Control,1972/, with trench drains /La Rochelle et al., 1976/, or with galleries /see 2f below/. In cold climates it may be necessary to prevent the outlets of horizontal drains from freezing /Golder, 1971/.

f/ Galleries: expensive, but can be appropriate to use in the treatment of very large slides /Viner-Brady,1955; Kezdi,1969; National Conf. Landslide Control,1972; Rico et al.,1976/. Supplementary drainage borings can be made through the sides, floor or roof of the galleries as required /Taniguchi & Watari,1965; Rodriguez et al.,1967; Záruba & Mencl,1969; Hoek & Bray,1974, Nilsen & Lien,1976/. For galleries running parallel to the slope face, Sharp /1970/ has made a study of the optimum locations for various ratios of horizontal to vertical permeability, using a variable resistance analogue.

g/ Vertical drains: these may discharge by gravity through horizontal drains or adits /well-drains/ /Seaton, 1938; Palmer et al. 1950; Sherrell,1971; Rat, 1976/, by siphoning, within the normal limitation of depth /Root,1958/, or by automatically activated pump /National Conf. Landslide Control,1972; Hoek & Bray, 1974/. Alternatively, the water may be blown out of the wells at intervals by compressed air. Under favourable hydrogeological conditions it is sometimes feasible to discharge downwards into an underlying aquifer at lower piezometric pressure /Parrott,1955; Wilson,1961/. In some cases, however, such measures have led to fresh stability problems associated with the under-draining stratum /Záruba & Mencl,1969; Lefebvre & Lafleur,1976/. Vertical drains may also be used as relief wells, discharging upwards, to lessen artesian groundwater pressures at depth. The use of sand drains in this way, to stabilize a slope of quick clay, is described by Holm /1961/.

* Wider drains, in which a bulldozer can operate, are used in the U.S. /Root,1958/.

h/ Electro-osmosis: used in the drainage of low permeability soils, even some clays. Water migrates from anodes towards cathodes, whence it is removed, with or without pumping /Casagrande et al.,1961; Bjerrum et al.,1967, Mitchell,1970; Wade,1976/.

j/ Vegetation: acts chiefly through reduction of pore pressures by evapo-transpiration. Záruba & Mencl /1969/ point out that in this, and other respects, deciduous trees are superior to conifers. There is also some contribution at shallow depth from the strength and binding action of the roots /Toms,1948; van der Burgt & van Bendegom,1948; Mehra & Natarajan,1966; Záruba & Mencl,1969; Gray,1970, 1974; Brown & Sheu,1975; Colas et al.,1976/. In addition, a vegetation cover effects the amount of infiltration /Hills,1971; Rodda et al.,1976/. The role of vegetation in controlling surface erosion is mentioned in 5b/ below.

3. Restraining structures

The use of rigid restraining structures is generally less appropriate than that of methods involving drainage or reshaping of the slope. Numerous cases of failure of such structures are reported by Root /1958/ and Baker & Marshall /1958/. When properly engineered, however, they can have a useful role, particularly where space is restricted.

a/ Retaining walls, founded beneath the unstable ground: some indication of the great variety of designs used is given by Root /1958/, Baker & Marshall /1958/, Záruba & Mencl /1969/ and Costa Nunes /1969/. Bujak et al. /1967/ describe the stabilisation of a rock slide by means of a concrete retaining block, held down by prestressed rock anchors and supporting a toe fill. In this case the efficacy of the corrective measures was checked by monitoring.

b/ Piles: a wall of continuous or closely spaced driven cantilever piles can be effective in stabilizing shallow slides /Toms & Bartlett,1962; Záruba & Mencl,1969/. More deep-seated slides have been successfully stabilized by, for instance, anchored sheet or bored pile walls /D'Appolonia et al.,1967; Brandl, 1976, Záruba & Mencl 1976/ or by large diameter cylinder pile retaining walls, generally of cantilever type /Andrews & Klasell,1964; De Beer,1969; Wilson,1970; De Beer & Wallays, 1970/. A general discussion of this type of structure is given by North-Lewis & Lyons /1975/. A particularly massive design, employing anchored foundation piers 30 to 35 m deep and 13 m in diameter at 24 m centres to stabilize a rock slope in Italy, is described by Baldovin & Fattore /1974/. In slopes of soft clay, there is a danger that the displacements and excess pore-water pressures induced by pile-driving will trigger a landslide /Bjerrum & Johannessen,1960; Broms & Bennermark,1968/.

c/ Soil and rock anchors, generally pre-stressed: these are employed either in conjunction with retaining structures, as in 3a/ & b/, or alone to reduce the driving forces of a landslide and to increase the normal effective stresses on its slip surface. For a planar slide, it can readily be shown that, for a given anchor force P, the maximum improvement of the factor of safety of the slide is achieved when P is inclined towards the slope at an angle with the slip surface equal to $\tan^{-1}\left(\frac{\tan \phi'}{F}\right) = \theta'_{mob}$. This result is independent of the magnitude of c', and of the ground-water pressures obtaining.

Anchors have been perhaps most commonly used on translational rock slides /Záruba & Mencl,1969; Hoek & Bray,1974; Baldovin & Fattore,1974; Lang,1976/. They can also be used to stabilize soil slides /Cambefort,1966; Costa Nunes 1966/. Allowance must then be made for short-term excess pore-water pressures and subsequent consolidation under the anchor pads with concomitant losses in anchor stress. Anchors are also coming into increasing use in open pit mines, particularly in America, for example to permit semi-permanent slopes to be cut more steeply /Barron et al.,1971; Littlejohn et al.,1977/. Another application is in the securing of large boulders, for instance in slopes of residual soil /Costa Nunes & Velloso,1963/. Cables or chains may also be used for this purpose /Bjerrum & Jorstad,1968/. Rock bolts are used for stabilisation of shallow instability in rock faces /Lang,1961; Price & Knill,1967/, together with other measures /Fookes & Sweeney,1976; Peckover & Kerr,1976/. Information on the long-term behaviour of pre-stressed anchors, especially with regard to stress losses through creep and corrosion, is sparse.

4. Miscellaneous methods*

a/ Grouting: a classical use of this is to reduce the permeability of the ground in order to reduce the ingress of ground-water to a landslide /Mitchell,1970; Matsubayashi,1972/. Cement grouting, usually aerated, has also been shown to be effective when injected into the slip surface of slides in cutting slopes of stiff clay /Ayres,1961/ and in other materials /Duncan,1971; Záruba & Mencl,1969/. If done without care it can, of course, trigger a slide.

b/ Chemical stabilization: by ion exchange and other processes. In the present connection it is deep soil stabilization rather than surface treatment that is relevant. Handy & Williams /1967/ claim to have stabilized a slide by the introduction of quick lime into the sliding zone through lime wells, partly by pozzolanic effects and partly through drying. Moum et al. /1968/ question the efficacy of lime wells but suggest that the diffusion of various salts through wells may be a practicable stabilization method. For the quick clay that they investigated, potassium chloride had the best overall effect. An application of potassium chloride diffusion in practice is described by Eggestad & Sem /1976/. Some theoretical background is given by Mitchell /1976/.

c/ Suppression of natural electro-osmosis: a method of slope stabilization described in several papers /e.g. 1968, 1973/ by Veder. Under certain conditions it is claimed that ground-water pressures can be reduced by the insertion of „short circuit electrodes", which suppress an inherent, naturally occurring electro-osmosis in the ground. Several landslides in Austria and Italy are stated to have been stabilized in this way: the method does not appear to have been evaluated yet in the U.K. or the U.S.A. Veder /pers. comm./ points out that the principle is evidently inapplicable to cases where the ground-water pressures are controlled by infiltration under gravity through very permeable layers.

d/ Electro-osmotic anchors: a technique suggested under the name „electro-osmotic tie-backs" by Casagrande & McIver /1971/ for improving the stability of tailings dams. It may also be of more general application in fine-grained materials.**Impressive increases are reported in the pull-out resistance of thin steel rods pushed into the ground and then used as electrodes in a direct current system for 1 to 2 weeks. In the soils tested both electrodes showed an increase in pull-out resistance, though this was naturally more marked at the anodes.

e/ Freezing: expensive and rarely used. The best known application is in temporarily stabilizing a flow of silt during construction of the Grand Coulee Dam /Gordon,1937/. Use of the method in stabilizing a slide in a tunnel roof is described by Gunleiksrud /1968/. A general review of the technique is given by Sanger /1968/.

f/ Heating: in the course of railway construction in 19th century England, it was a fairly common practice to stabilise the slopes of clay cuttings by „burning". Wide counterfort drains were dug and filled with alternate layers of clay spoil and coal. The coal was then ignited, thus baking the intermixed and the adjacent clay /see, for example, Copperthwaite,1880/. Although particularly advantageous when applied to clays and shales of high carbonaceous content, which were virtually self-firing, the technique was also used in non-carbonaceous clays such as the London Clay. More recently, unstable slopes of loess and

* Some of these may still be regarded as being in an experimental stage.

** The technique may be of use in the construction of reinforced earth walls with clayey backfills.

of clay have been stabilised by passing hot gasses through a system of tunnels or boreholes /Hill,1934; Beles & Stanculescu, 1958; Litvinov et al.,1961; Záruba & Mencl,1969; Mitchell, 1970/, Litvinov et al. also mention a technique combining heating with chemical treatment, which they term thermo-ohemical stabilisation.

g/ Blasting: a controversial and unreliable technique, intended to disrupt a slip surface or to improve drainage /Root,1958; Baker &Marshall, 1958; Záruba &Mencl, 1969: Mitchell, 1970/.

h/ Bridging: a technique occasionally used to carry a road over an active or potential landslide. It is more often employed on steep slopes affected by translational slides of moderate or small width /Root, 1958; Baker & Marshall, 1958; Záruba & Mencl, 1969; Costa Nunes, 1969/.

5. Erosion Control

The close link between mass movements and erosion is a basic element in the geological cycle of denudation. It follows, therefore, that the control of erosion, in both the general and the specific sense, is fundamental to the prevention of landslides. This point has been emhpasised by Bjerrum et al. /1969/ and by Hutchinson /1973/. Furthermore, once a landslide has been stabilized it is important to prevent any further worsening of its condition through erosion. The potential eroding agent is usually water, tough erosion by wind and other agencies can occasionally be significant /.e.g. Colenutt,1928/.

a/ Control of toe erosion: in cases where the toe of a landslide is situated in the sea, a lake, a reservoir or a river, it is of prime importance to prevent erosion at this most critical point. Measures commonly used include concrete or crib walls, rip-rap and other revetments, groynes and spur dikes /Viner-Brandy,1955; National Conf. Landslide Control,1972/.

b/ Control of surface erosion on slopes: generally achieved through proper attention to the design of drains for surface water /Mehra & Natarajan,1966/ and the encouragement of suitable vegetation cover /Barata,1969/. The latter was done originally by topsoiling and seeding, sometimes in combination with jute netting to prevent soil erosion while the vegetation was becoming established /Mehra et al.,1967/. More recently various techniques of hydraulic seeding, in which a mulch containing the plant seeds is sprayed onto the slope, have been developed /Schiechtl,1965/. In these the necessary interim erosion protection is often provided by asphalt emulsion, or other chemical or plastic admixtures. A layer of soil-cement has sometimes been used as a semi-permanent control of surface erosion /Costa Nunes &Velloso,1963/.

c/ Control of seepage erosion: this type of erosion occurs when the seepage drag of ground-water discharging at a free face is large enough to dislodge and remove individual soil particles. The resultant back-sapping tends to undermine the superincumbent strata and eventually to cause their collapse. This form of failure is quite common in both natural and cut slopes, and can be very damaging. Soils in the coarse silt to fine sand range are particularly prone to it /Terzaghi,1950; Hutchinson,1968/. The free face may also, under certain conditions, migrate into the slope in a localized manner, leading to piping by subsurface erosion /Terzaghi & Peck,1967 ; Sherard et al.,1972/. The chief method of controlling seepage erosion is by placing inverted filters over the area of discharge /Ward,1948; Terzaghi & Peck,1967/, or by intercepting the seepage at some distance back from the face with wells or sand drains /Anon,1965/.

From this brief review of types of corrective measure, two points emerge. The first is that, while most of the papers suggest that the measures applied were effective, at least in the short term, it is rather rare for the efficacy of the corrective works to be properly assessed by appropriate monitoring. The second is that drainage appears to be the most generally used stabilization measure, with the modification of slope profile by cutting and filling also frequently employed. Thus, in the remainder of this paper, attention is concentrated on these two corrective methods.

Modification of slope profile by excavation and filling

General remarks

The respective merits of removing the head of an actual or potential slide, of flattening the slope uniformly or benching it, or of building a berm at its toe, are discussed by Baker & Marshall /1958/, Root /1958/ and Mehra & Natarajan /1966/. In general, such cuts and fills would appear to be most effective when applied to deep-seated landslides, in which the slip surface tends to fall steeply at the head and rise appreciably in the region of the toe. Clearly, however, there is also a scale effect, so that the influence of a given cut or fill on the overall factor of safety will diminish with the size of the landslide being treated. For example, in the case of the large deep-seated coastal landslides at Folkestone Warren the toe-weighting constructed as part of the stabilization works /Viner-Brady,1955/, although well positioned, improves the overall long-term factor of safety by only about 3.5 to 5%*.

It is, of course, very important to ensure that neither cuts nor fills trigger the existing or potential slide that they are designed to stabilize, nor generate fresh slides local to themselves. It should also be borne in mind that a fill is a rather specific measure and while it may deal satisfactorily with the particular slide that it was designed to stabilize /abc on Fig. 1/, it may be quite ineffective against an almost equally serious „over-rider" slide /on abd/. The danger is greatest in the case of translational landslides, as illustrated in Fig. 1: the geometry of rotational slides makes them less prone to, though not necessarily safe from, this type of failure.

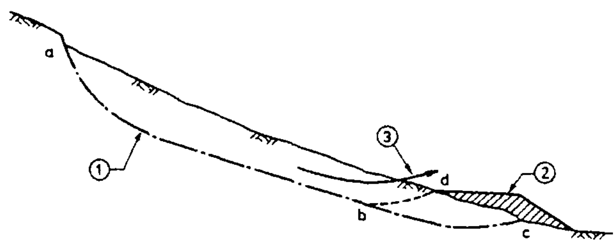


Fig. 1 : Translational slide stabilized by a toe fill, showing the danger of a potential over-rider slide.
(1) Slip surface. (2) Toe fill. (3) Over-rider slide.

A point frequently neglected in the literature on slope stabilization is that cuts and fills, depending on the ground condition and the speed of their construction, comprise more or less undrained unloadings and loadings respectively. The matter is clearly presented by Bishop and Bjerrum /1960/. Thus in the case of a fill, the value of the factor of safety, F, will generally be less in the short-term than in the long-term. The opposite will usually apply in the case of cuts. In both cases it is good practice to check both the short- and long-term values of F. An important advantage of a corrective fill is that, once successfully placed, its stabilizing effect will tend to increase until the ground beneath it is fully consolidated and thereafter, unless by some accident the fill is removed, its contribution to stability will be a permanent one** As discussed later, this reliability in the long-term is generally less assured in the case of drainage measures.

The „neutral line" concept

The efficacy of a corrective cut or fill is controlled by its location, weight and shape and by the characteristics of the actual or potential landslide that it is intended to stabilize. These factors are all known, or can be determined, at the design stage. It follows, therefore, that while it is advisable to check their performance by appropriate monitoring during and after construction, the efficacy of cuts and fills should first be determined by analysis before they are carried out.

* Calculated for sections W6 and W8 of Hutchinson /1969/.

**Provided that proper drainage of the fill has been arranged, for example by an underlying drainage blanket and by appropriate surface drains.

In structural engineering design a familiar device is to consider that an „influence load" of some convenient magnitude passes over a structure, for instance a bridge, and to determine the resulting „influence lines", typically for bending moments and shear forces at any point, as a function of the position of this load on the structure. The same idea may be applied to a landslide, to determine influence lines for changes in its factor of safety produced by an influence load moving across it.

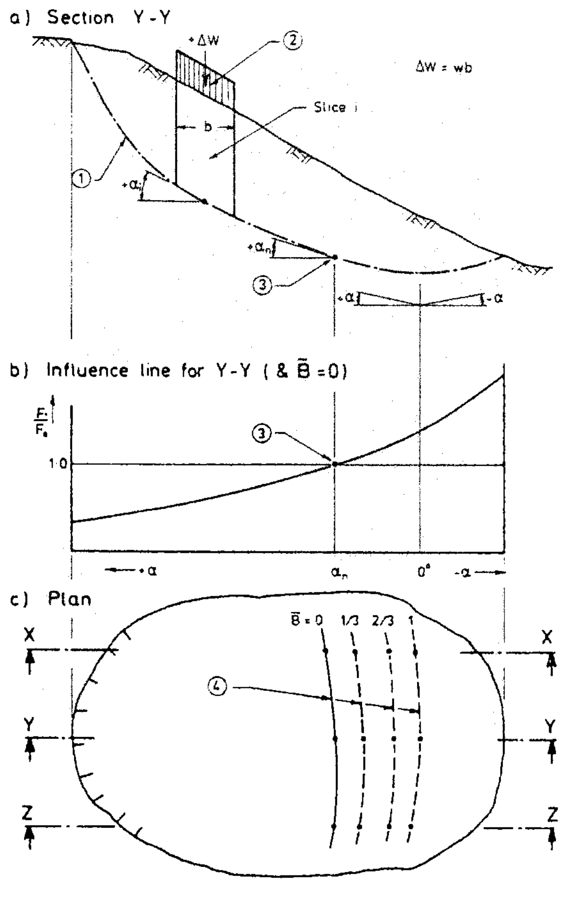


Fig. 2 : a) Cross-section of a landslide, with an influence load ΔW acting.

b) Influence line (diagrammatic) for the effect of ΔW on the overall factor of safety of the landslide.

c) Plan of the landslide, showing diagrammatically the positions of the neutral lines for different $\bar{B}$ values.

(1) Slip surface. (2) Influence fill. (3) Position of neutral point (drawn for $\bar{B} = 0$). (4) Positions of neutral line, for different $\bar{B}$ values.

Consider, for simplicity, a slope that can be represented two-dimensionally by the cross-section in Fig. 2a. If a convenient, arbitrary influence load, ΔW, is now assumed to act, in turn, at successive positions between the head and toe of the landslide, we can derive an influence line for the resulting effect on the overall safety factor, as indicated in Fig. 2b. The ratio

$$\frac{F_1 (= F_0 + \Delta F),\text{ the overall F with } \Delta W \text{ acting}}{F_0,\text{ the original value of F}}$$

is taken as a convenient measure of this effect.

An influence load representing fill, with ΔW positive and acting downwards, will of course tend to decrease the existing factor of safety, F_0, when it acts in the vicinity of the head of the slide and to increase this when it acts near the toe. Of particular interest is the point where $\Delta F = 0$, or $F_1/F_0 = 1.0$, termed the NEUTRAL POINT, which indicates where an influence load will have no effect on F_0. The trace of the neutral points in plan is termed the NEUTRAL LINE /Fig. 2c/ and this forms the boundary between the area of the landslide on which a fill /or cut/ would improve its stability and the area for which the reverse would hold.

Determination of the location of the neutral point

Let the slip surface /Fig. 2a/ be of general shape, with effective shear strength parameters c' and ϕ'. Let the influence load, in the first instance, represent a uniformly distributed fill, of intensity w per unit of horizontal area, acting on a slice, i, of horizontal width b. Thus $\Delta W = wb$. Now in the general case, application of this load will set up undrained pore-pressures on the subadjacent slip surface, of magnitude $\Delta u = \bar{B} \cdot \Delta\sigma_1 = \bar{B} \cdot w$. If, following Skempton & Hutchinson /1969/, we apply the Conventional Method to this non-circular slip surface, and let

$$F_0 = \frac{\Sigma \text{ available resisting forces}}{\Sigma \text{ driving forces}} = \frac{\Sigma R_0}{\Sigma D_0}$$

the new factor of safety F_1 will then be

$$F_1 = \frac{\Delta N_i' \tan\phi' + \Sigma R_0}{\Delta W \sin\alpha_i + \Sigma D_0}$$

where α_i is defined in Fig. 2a and $\Delta N_i'$ is the change in effective normal force produced on the base of slice i by the influence load ΔW, given by

$$\Delta N_i' = \Delta W (\cos\alpha_i - \bar{B}\sec\alpha_i)$$

Hence

$$\frac{F_1}{F_0} = \frac{\Sigma D_0[\Delta W (\cos\alpha_i - \bar{B}\sec\alpha_i)\tan\phi' + \Sigma R_0]}{\Sigma R_0(\Delta W \sin\alpha_i + \Sigma D_0)}$$

Now, at the neutral point,

$$\frac{F_1}{F_0} = 1 \qquad \text{and } \alpha_i = \alpha_n \text{ (Fig. 2a)}$$

The position of the neutral point is thus given by

$$\tan\alpha_n = (1 - \bar{B}\sec^2\alpha_n)\frac{\tan\phi'}{F_0} \qquad (1)$$

Alternatively, the Bishop Simplified Method /Bishop,1954/ may be used as the basis for the analysis. Then, with the symbols used in that paper,

$$F_0 = \frac{1}{\Sigma W \sin\alpha}\Sigma\left\{[c'b + \tan\phi'(W - ub)] \cdot \frac{\sec\alpha}{1 + \frac{\tan\phi'\tan\alpha}{F_0}}\right\}$$

Hence, putting

$$[c'b + \tan\phi'(W - ub)] = J,\text{ and } \frac{\sec\alpha}{1 + \frac{\tan\phi'\tan\alpha}{F_0}} = M_0$$

$$F_0 = \frac{1}{\Sigma W \sin\alpha}\Sigma(JM_0)$$

Then the new factor of safety, with ΔW acting on slice i, is

$$F_i = \frac{1}{\Delta W \sin\alpha_i + \Sigma W \sin\alpha}[\Delta W \tan\phi'(1-\bar{B}) M_{1_i} + \Sigma(JM_1)]$$

where

$$M_1 = \frac{\sec\alpha}{1 + \frac{\tan\Phi'\tan\alpha}{F_1}},\text{ and } M_{1_i} = \frac{\sec\alpha_i}{1 + \frac{\tan\Phi'\tan\alpha_i}{F_1}}$$

Hence

$$\frac{F_1}{F_0} = \frac{[\Delta W \tan\varphi' (1 - \bar{B})\, M_{1_i} + \Sigma(JM_1)]\, \Sigma W \sin\alpha}{(\Delta W \sin\alpha_i + \Sigma W \sin\alpha)\, \Sigma(JM_0)}$$

To find the position of the neutral point, put $\frac{F_1}{F_0} = 1$ and $\alpha_i = \alpha_n$ as before.

Then $\sin\alpha_n = (1 - \bar{B})\, M_{i_n} \dfrac{\tan\varphi'}{F_0}$

but at the neutral point $M_{i_n} = M_{o_n}$, therefore

$$\sin\alpha_n = \frac{(1 - \bar{B})\sec\alpha_n}{1 + \frac{\tan\phi' \tan\alpha_n}{F_0}} \cdot \frac{\tan\phi'}{F_0}$$

which reduces to

$$\tan\alpha_n = (1 - \bar{B}\sec^2\alpha_n)\frac{\tan\phi'}{F_0} \quad \text{, as before /eqn (1)/.}$$

This is a quadratic in $\tan\alpha_n$. The negative root gives high negative values of α_n, which doubtless chiefly reflect the known anomalies of both the Conventional and the Bishop Simplified Methods in cases of steeply rising slip surfaces /Whitman & Bailey, 1967; Turnbull & Hvorslev, 1967/. These negative values of $\alpha_n (\varphi'_{mob} - 90$ and steeper) are not of practical importance and are therefore neglected.

Equation (1) is a general solution which applies for any value of $\bar{B}$ to positive or negative influence loads, i.e. to fills, if assumed to be of zero strength, and to cuts if the complications of changes in slide geometry and the breakdown of possible negative pore-pressures are neglected. It also holds for slip surfaces of circular or general shape and, in principle, regardless of whether these are pre-existing or potential. It would obviously be questionable, however, to apply the idea to a potential slip surface which had no physical reality. It will be noted that the position of the neutral point is independent of ΔW and c'.

Two special cases can be distinguished:

(i) for $\bar{B} = 1.0$, when

$$\tan\alpha_n = (1 - \sec^2\alpha_n)\frac{\tan\varphi'}{F_0} \qquad (2)$$

which is satisfied by $\alpha_n = 0$ (3)

and (ii) for $\bar{B} = 0$, when $\tan\alpha_n = \dfrac{\tan\varphi'}{F_0} = \tan\varphi'_{mob}$ (4)

i.e. $\alpha_n = \phi'_{mob}$

In the general case of $1.0 > \bar{B} > 0$, the neutral point will occupy positions intermediate between those of cases (i) and (ii) above, in accordance with equation (1). The solution of this equation is shown graphically in Fig. 3.
While B is an undrained parameter, the case of $\bar{B} = 0$ may also conveniently be regarded as equivalent to the fully drained condition with respect to a corrective earthwork of any initial pore-water pressure response*. In principle this idea can be applied to any $\bar{B}$ value.

For circular slip surfaces, the result for case (i) confirms the self-evident fact that for $\bar{B} = 1.0$ the position of the neutral point is situated vertically below the centre of the slip circle /Fig. 4a/. In comparison, the solution for case (ii) shows that the position of the neutral point for $\bar{B} = 0$ is displaced towards the slope by a horizontal distance equal to the radius of the approximate friction circle, i.e.

$R \dfrac{\sin\varphi'}{F_0}$ /Fig. 4a/

* The long-term effect of the earthworks on the original steady state pore-water pressures in the landslide as a whole is neglected.

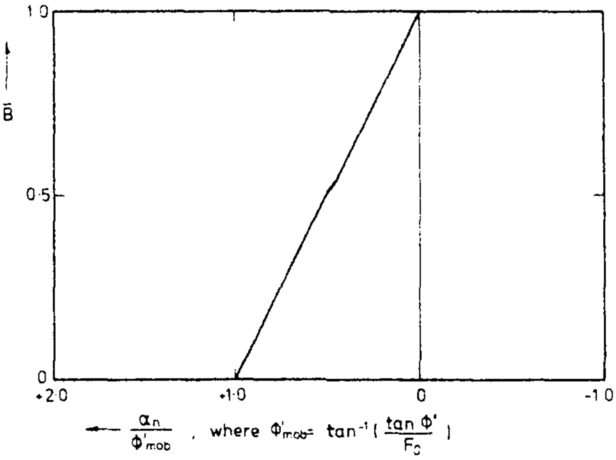


Fig. 3 : Approximate variation of α_n/φ'_{mob} with $\bar{B}$. The relationship is not perfectly linear, but the divergencies are insignificant within the normal range of α_n/ϕ'_{mob} values.

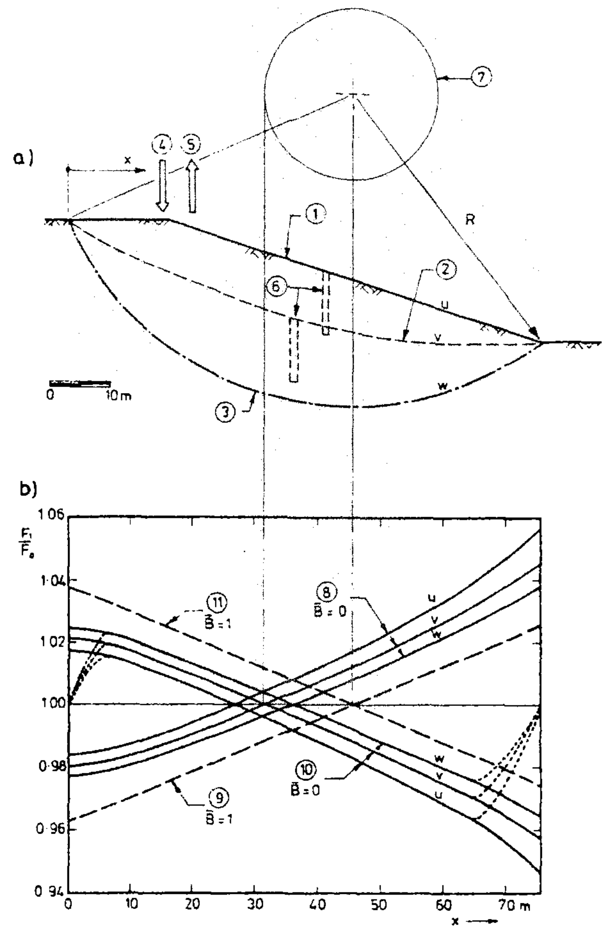


Fig. 4 a) and b)

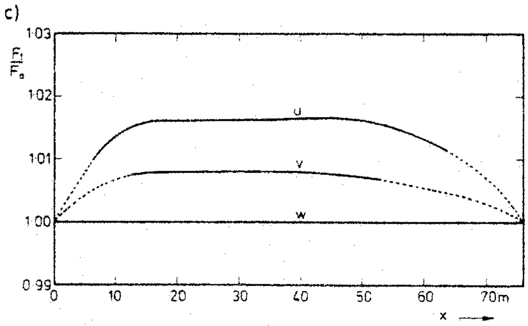


Fig. 4 : Influence lines for F_1/F_0 for a typical circular slide: –

a) Cross-section of landslide.

b) Influence lines for an influence "cut" and an influence fill for $\bar{B}$ = 1.0 and $\bar{B}$ = 0 and for various original piezometric lines.

c) Influence lines for an influence drainage, for various original piezometric lines.

(1) Ground surface and piezometric line u ($\bar{r}_u$ = 0.49).

(2) Piezometric line v ($\bar{r}_u$ = 0.29). (3) Slip surface and piezometric line w ($\bar{r}_u$ = 0). (4) Influence fill. (5) Influence "cut". (6) Influence drainage. (7) Friction circle with radius = R sin ϕ'_{mob}. (8) Influence fill with $\bar{B}$ = 0. (9) Influence fill with $\bar{B}$ = 1.0. (10) Influence "cut" with $\bar{B}$ = 0. (11) Influence "cut" with $\bar{B}$ = 1.0.

NB. Curves (9) and (11) are independent of the original piezometric line.

In addition it is clear that for non-circular slip surfaces which include a planar section, the neutral line will widen to form a NEUTRAL ZONE should the inclination of this section to the horizontal coincide with the value of α_n for the particular conditions obtaining.

Discussion

Partly as a check on the above results for the positions of the neutral point and partly in order to define the chosen influence lines in their entirety, a number of computer analyses have been run for a constant slice width b. The effects of various influence loads acting on a typical circular slip surface have been determined using Bishop's Simplified Method /1954/ and similar investigations for a typical non-circular slip surface have been made by the Morgenstern & Price Method /1965/. In all these analyses an arbitrary influence load of 2o.39 tonne f /200 kN/ on a 1 metre width /measured horizontally in a valleyward direction/ has been used in the circular cases and the same load acting on a 2 metre width in the non-circular ones. In all cases the section of landslide analysed is 1 m thick in a cross slope direction.

The complete influence lines for each case considered are given in Figs. 4a, b, 5, 6, and 7a, b. As will be seen from the plots of F_1/F_0 against α in Figs. 5 and 6, the computations confirm the theoretical predictions for the positions of the neutral points.

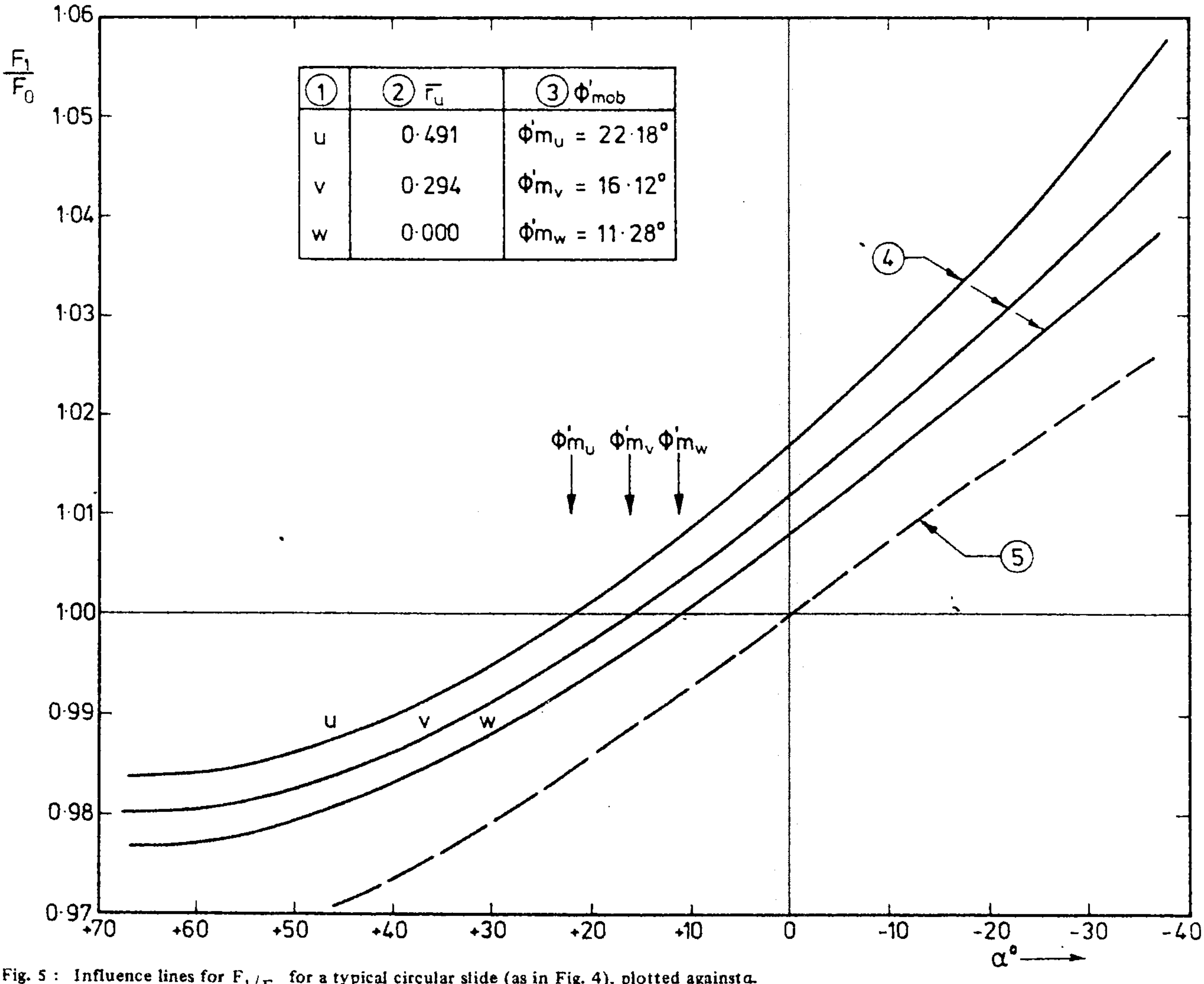


Fig. 5 : Influence lines for F_1/F_0 for a typical circular slide (as in Fig. 4), plotted against α.

(1) Original piezometric line. (2) Average pore pressure ratio. (3) Mobilised effective angle of shearing resistance. (4) Influence lines for an influence fill with $\bar{B}$ = 0 and various original piezometric levels. (5) Influence line for an influence fill with $\bar{B}$ = 1.0, which is independent of original piezometric level.

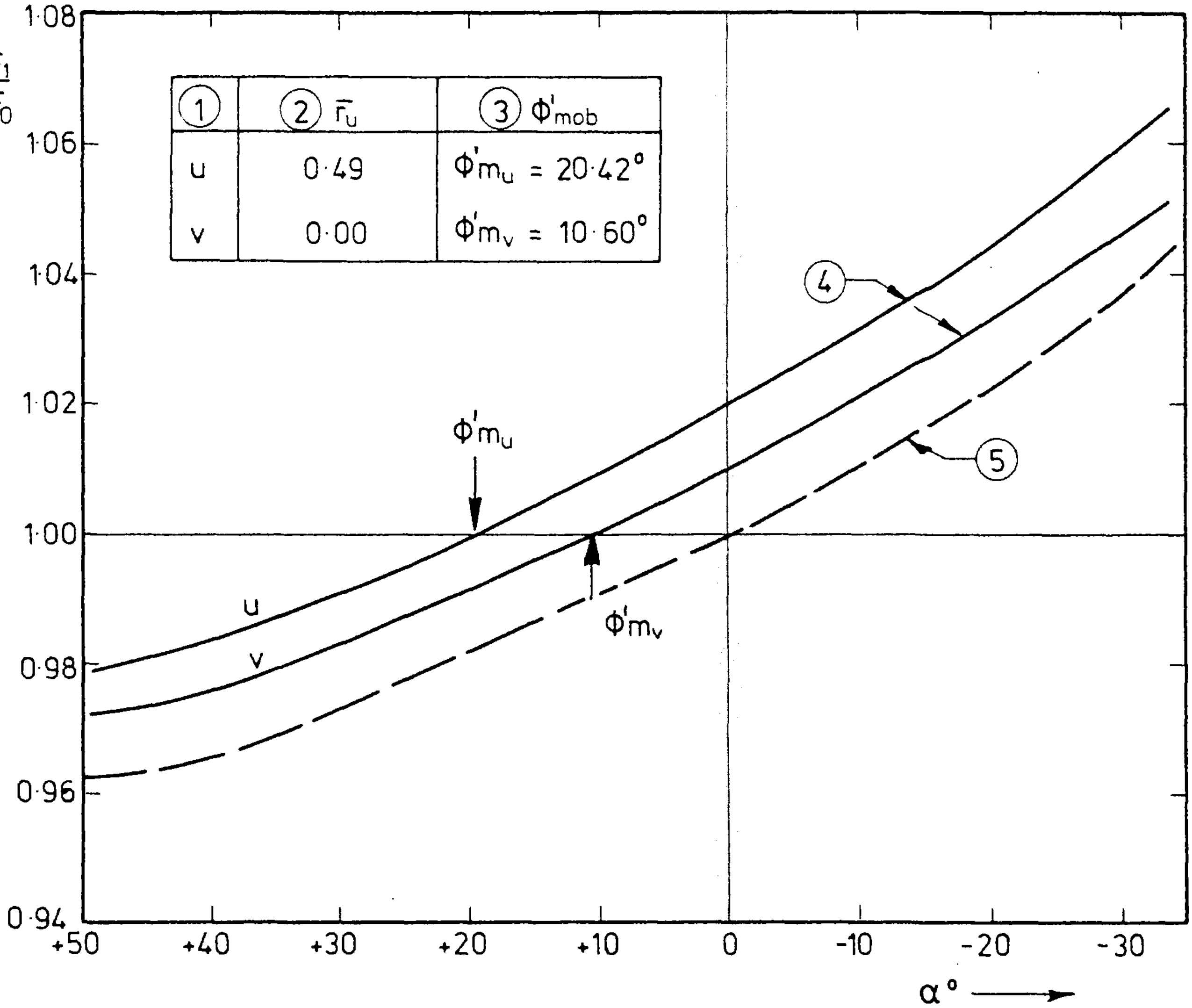


Fig. 6 : Influence lines for F_1/F_0 for a typical non-circular slide (as in Fig. 7), plotted against α. (1) to (5) are as defined for Fig. 5.

A check, for the circular case with $\bar{B}$ = O, on the sensitivity of the results to the magnitude of a positive influence load, indicates that the values of F_1/F_0 - 1 are proportional to this load up to a value of at least twice the maximum total vertical stress acting on the slip surface considered, that is, well beyond any practical limit. This proportionally does break down, however, if the influence load is increased greatly, for instance by a factor of 100.

A pre-requisite for application of these ideas is, of course, determination of the location of existing or likely potential slip surfaces. As this information is needed in any case, however, to permit piezometers to be installed in their proper positions, this is not seen as a disadvantage. The examples considered all comprise single slips. In practice, complex assemblages of slips are frequently encountered and then a set of neutral lines /as in Fig. 2c/ may need to be defined separately for several, or all, of the component single slips.

It follows from equation (1) that the positions of the neutral lines, for any conditions other than that represented by the special case of $\bar{B}$ = 1.0, will tend to shift valleywards as the initial factor of safety, F_0, is increased towards F_1 by the stabilization works. The practical significance of this will depend partly on the shape of the slip surface and partly on the designed improvement in F_0. In principle it seems better to work as far as possible with the final value of F.

It should perhaps be emphasised that the values of $\bar{B}$ in the foregoing analysis refer strictly to the undrained change in pore-water pressure associated with the applied cut or fill, and the conclusions reached are thus independent of the „background" drainage conditions in the slip as a whole, except insofar as these determine F_0. Thus, for example, the neutral line for an influence loading with $\bar{B}$ = 1.0 will be defined by $\alpha_n = 0^0$ regardless of whether the slip itself is in the short-term, intermediate or long-term condition. For the first two of these conditions, however, the continuing change in pore-water pressures in the slip itself, as these move towards equilibrium, must also be considered, in relation to its effect on both the overall long-term factor of safety and the long-term position of the neutral point.

In the past, vectors of surface movement in an active landslide have sometimes been used as a guide to the proper location of corrective earthworks, for instance by limiting the extent of a counterweight berm to the area which has shown a tendency to rise /K.R. Early, pers. comm./. On the assumption that the ground surface movements closely reflect the shape of the underlying slip surface, we now see that this method is accurate for a fully undrained fill but on the conservative side for a fully drained one.

Some engineers hold the view that, in effect, the neutral line is located vertically beneath the centre of gravity of the slip mass. From the foregoing, however, it is clear that, in general, this is not correct. In the landslides examined so far, two of which are shown in Fig. 8, the vertical through the centre of gravity of the

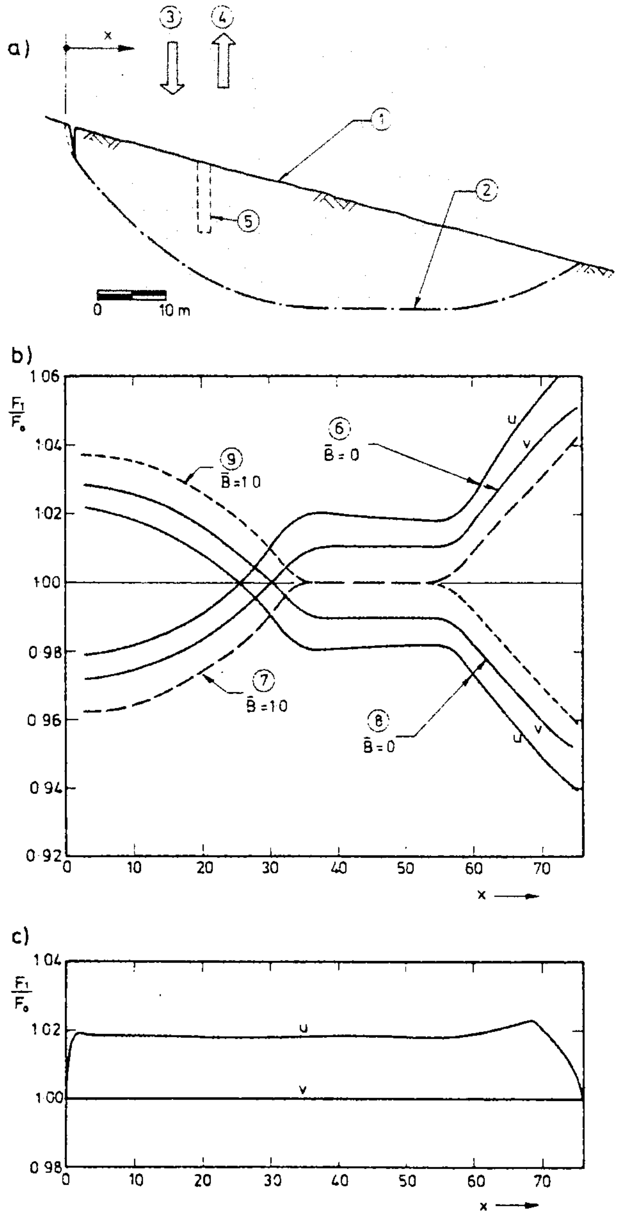


Fig. 7 : Influence lines for F_1/F_0 for a typical non-circular slide: –

a) Cross-section of landslide.

b) Influence lines for an influence "cut" and an influence fill for $\bar{B}$ = 1.0 and $\bar{B}$ = 0 and for various original piezometric lines.

c) Influence lines for an influence drainage for various original piezometric lines.

(1) Ground surface and piezometric line u ($\bar{r}_u$ = 0.49).

(2) Slip surface and piezometric line v ($\bar{r}_u$ = 0).

(3) Influence fill. (4) Influence "cut". (5) Influence drainage.

(6) Influence fill with $\bar{B}$ = 0. (7) Influence fill with $\bar{B}$ = 1.0

(8) Influence "cut" with $\bar{B}$ = 0. (9) Influence "cut" with $\bar{B}$ = 1.0

NB. Curves (7) and (9) are independent of original piezometric level.

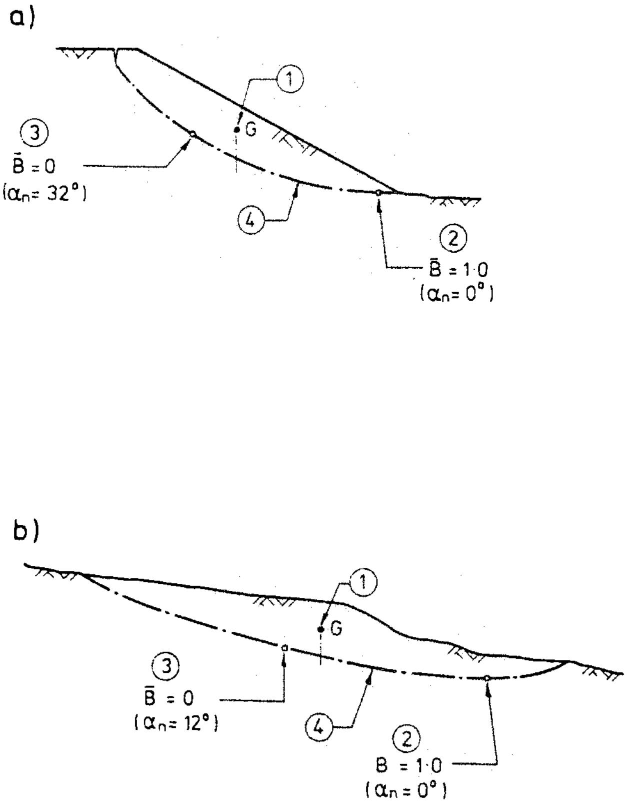


Fig. 8 : Examples of the relationship between the extreme positions of the neutral point and the centre of gravity of a slide.

a) Circular slide at Selset (after Skempton & Brown 1961)

b) Non-circular slide at Bury Hill (after Hutchinson, et al. 1973)

(1) Position of the centre of gravity. (2) Neutral point for $\bar{B}$ = 1.0. (3) Neutral point for $\bar{B}$ = 0. (4) Slip surface.

slip mass falls between the $\bar{B}$ = 1.0 and the $\bar{B}$ = 0 positions of the neutral point. This would seem to be generally the case, but may not invariably be so.

It is anticipated that the chief value of the work presented above will be in the initial stages of a design, when a knowledge of the positions of the neutral line for various values of $\bar{B}$ /e.g. Fig. 2c/, should be of help in choosing, for instance, the optimum route for a road to cross an existing landslide or the best location for corrective earthworks. The influence lines may also be used to make preliminary quantitative estimates of the F_1/F_0 ratio produced by a given earthwork. A final check on this should, however, always be made by running orthodox stability analyses.

Drainage

General remarks

The neutral line concept is clearly inapplicable in this case, as drainage of any part of a landslide is always beneficial. Influence lines for the effect of an „influence drainage", producing a reduction in piezometric pressure of 10.2 tonne f/m^2 /100 kN/m^2/ on successive 1 m wide slices for three different piezometric lines in a typical circular slip, are given in Fig. 4c. Similar influence lines for a piezometric reduction of 10.2 tonne f/m^2 on successive 2 m wide slices for two different piezometric lines in a typical non-circular slip are given in Fig. 7c. From these examples we see that the effect of a given drainage is rather constant throughout, except for a slight rise at the toe. The rapid reduction in effect at both head and toe of the slides results from the neglect of negative pore-water pressures once the reduced piezometric line falls below the level of the associated part of the slip surface.

Some indication has been given earlier of the great variety of drainage measures that have been used for slope stabilization. Of these, two will be treated in more detail here.

Horizontal drains

This term refers to small diameter pipe drains that are installed within a slope, usually by helical auger or rotary drill*, on a rising gradient of typically 2% to 20% so as to discharge by gravity. While the origin of this type of drain is obscure, much of the early development work was carried out in California, where horizontal drains have since been widely used /Smith & Stafford,1957; Root,1958/. More recently, they have been employed to stabilize slopes in many other countries, including Japan /Taniguchi & Watari,1965/, Britain /Ayres,1961; Ronbinson,1967/, Germany /Henke,1968/, Czechoslovakia /Záruba & Mencl,1969/, Yugoslavia /Nonveiller, 1970/, New Zealand /East, 1974/, Hong Kong /Tong & Maher,1975/, Canada /La Rochelle et al.,1976/ and France/Cambefort,1966; Rat,1976/.

Horizontal drains are usually between about 5 and 2o cm in diameter. In most cases they are spaced between 3 and 2o m apart and, as mentioned above, have lengths of about 30 to 100 m. They may often with advantage, be installed from several elevations. Discharges from a single drain have varied from about 175,000 litres/day /Brawner ,1971/ to zero. Early installations generally consisted of perforated steel pipes without filters and were prone to both corrosion and siltation. Smith & Stafford /1957/ recommended that the 6 m length of drain nearest to the outlet should be galvanised and nonperforated, to slow down corrosion and hinder choking of the pipe by roots. More recently plastic pipes have been used, with filters formed of porous concrete /Robinson ,1967/, resin bonded sand /Nonveiller,1970/ or synthetic filter fabrics. In jointed rock and residual soil masses, Choi /1974/ recommends the use of drains with impermeable inverts.

The use of horizontal drains is most appropriate in the case of slopes where the ground-water lies too deep to be reached by trench drains. Such conditions are frequently associated with relatively steep slopes and deep-seated slip surfaces /Nonveiller, 1970/. Most of the slopes treated have been between about 15^0 and 45^0 in inclination, though horizontal drains have been installed in Hong Kong in slopes of weathered igneous rocks of up to 70^0 inclination /Tong & Maher,1975/. From a practical point of view it is important that the rock or soil involved is readily drillable and does not cave, and it is an advantage if the necessary drain length does not exceed about 90 m /Smith & Stafford,1957/. As with any form of drainage, it is helpful if the mass to be treated contains previous layers or zones, but this is not essential.

It can be difficult to install horizontal drains in a slide that is still moving, as the drill strings may jam. Such an operation was successfully carried out, however, on an active slide on the São Paulo to Santos highway /Teixeira & Kanji,1970/. The slide had an area of about 20 hectares and was moving at between 2 and 5 m/month. Because of this high movement rate, the horizontal drains were installed in two stages. In the first a preliminary stabilization was attained by installing drains about 40 m long. It was then possible to install the second stage drains in lengths up to 120 m. After 5 years the slide had shown no significant further movement.

Until recently, horizontal drains have been installed entirely on an empirical basis, with the quantity of water discharged as the main criterion of success. Latterly both Nonveiller /1970/ and Kenney et al. /1976/ have criticized this approach and re-emphasised that the primary aim of such drainage is to reduce pore-water pressures which, in clay slopes, may be achieved with a very small yield of water. Both Teixeira & Kanji /1970/ and, to a more detailed extent, Nonveiller /1970/ make use of flow nets to estimate the reduction in pore-pressure that a given horizontal drain installation will achieve. A very thorough theoretical study of the stabilizing effects of horizontal drains has more recently been made by Choi /1974/.

Kenney et al. /1976/ make a three-dimensional model study of the drainage of homogeneous 3:1 slopes by horizontal drains, and produce design charts which are believed to apply with reasonable accuracy to homgeneous slopes with inclinations of between 2.5 and 3.5:1. Two depths to an impervious lower boundary are considered. As the authors state, the design charts have yet to be calibrated against field experience.

As indicated above, monitoring of horizontal drain systems has generally been confined to measurement of drain discharge. In some cases the effect of drainage on the movements of the slide being treated have also been reported /Teixeira & Kanji,1970/. The effects of horizontal drainage installations on ground-water pressures have, as yet, been measured rather rarely. A brief report is given by Brawner /1971/ of reductions of between 9 and 12 m in cleft-water pressures in a 137 m high rock slope, occurring within 30 days of installing four trial horizontal drains. A fuller description of the stabilization by horizontal drains of slides in a 15 m high, 3:1 cut slope of silts and sands in New Zealand is provided by East /1974/. Fifty-five drains of 30 m nominal length and 3 m spacing, in three rows, were installed in the space of ten days during the winter of 1972. Within a further 5 days, piezometric levels had dropped by between 1.0 and 2.5 m /1.7 m on average/. A check in the winter of 1974 showed that the average depression of piezometric levels was then 2.0 m. No further movements of the slides were recorded in the two years following installation of the drains. It is now clear that this installation was over-designed. It was, however, an emergency operation to save a high-tension electricity pylon threatened by the slides and as such was both expeditous and successful. Various divergencies from the idealized slopes explored by Kenney et al. /1976/ prevent this case record from serving as a check on their design charts.

There is a great need for case records of well instrumented and monitored horizontal drain installations in various soils and rocks.

Trench and counterfort drains

General remarks

As indicated above, the term counterfort is used here to describe drains which penetrate into solid ground beneath a slip surface, and therefore also provide some mechanical buttressing to the slope, while the term trench drains is reserved for those which do not thus penetrate, and so contribute to stability only through their drainage action.

The counterfort drain seems to have been the first of these types to be used as a principal stabilization measure. While the origins have not been excplored, it is clear that drains of this type were being widely used in France and England during the first half of the 19th century, in both embankments and cuttings /Gregory, 1844; Collin,1846/. Collin, in particular, developed his designs to a considerable degree of sophistication, arranging, for example, the width of the counterforts to increase stepwise down-slope so as to improve the stability of the clay mass lying between them. To the same end, in remedying a landslide, the soil between the counterforts was often excavated successively and then, if sufficiently dry, replaced and compacted in thin layers /Collin,1846 p.71/. In contemporary English practice /R. Stephenson reported by Dockray,1844; Gregory,1844/ this operation was omitted, as indeed it is nowadays.

At that time it seems to have been generally believed that drains that did not penetrate beneath the seat of sliding were of little or no use /e.g. Whitely,1880/. The basis for such a view was removed by Terzaghi's enunciation of the principle of effective stress in 1925, but it took about a further three decades before trench drains took their proper place as a stabilization measure in the U.K.

Approximate theory

Up to now, the design of trench drains for slope stabilization in the U.K. has proceeded on a semi-empirical basis. Typically, the average lowering of piezometer level on the slip surface required to produce the desired increase in factor of safety has been calculated: the trench drains needed to effect this average lowering have then generally been dimensioned on the basis of experience.

* A case where horizontal drains were installed by hydraulic jacking is described by La Rochelle et al. /1976/.

There is an extensive literature on ground-water flow to drains, especially in the fields of soil physics and agricultural engineering /reviewed, for example, by van Schilfgaarde 1970, 1974, van Horn 1974/. This is largely concerned, however, with the prevention of water-logging of crop roots and thus with the determination of the phreatic surface between horizontal drains of relatively shallow depth /often 1 to 1.5 m/. Drainage for slope stabilization, on the other hand, requires a knowledge of the piezometric levels at depths of typically up to 8 or 10 m. The problem has therefore been approached through the use of flow nets, as was suggested by Henkel /1957/.

Even in the long-term, the actual three-dimensional trench drain problem, with intermittent recharge from infiltration, a variable inflow of ground-water from upslope, and non-homogeneous and isotropic permeability, varying with effective stress, involves non-steady saturated and partially saturated flow, and is highly complex*. Furthermore, the stress release occasioned by the excavation of the drains generally produces a significant short-term modification of the original pore-water pressures, which is followed by an intermediate stage of consolidation and possibly swelling, before the long-term condition is reached. The long-term condition is of chief interest in the context of slope stabilization, however, and the approximate treatment of this, described below, has therefore been developed.

The initial assumptions are that both the ground surface and the original piezometric surface are horizontal, that the permeability of the ground is homogenous and isotropic and that the trench drains are of rectangular cross-section, and parallel to each other. The effects of anisotropic permeability are explored subsequently. This arrangement and definitions of the various symbols used are shown in the key cross-section of Fig. 9. The drains are also assumed to be of great length L upslope compared with their spacing S, so that a two-dimensional approach will have some validity. A photograph of such drains being excavated /for which B = 0.6 m, D = 5 to 6 m and A = 11 to 12 m/ is shown in Fig. 10.

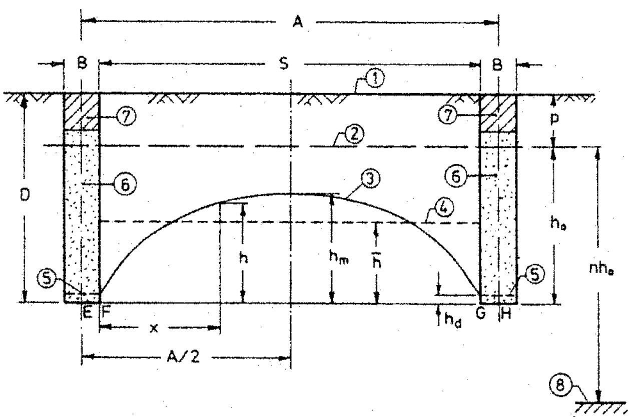


Fig. 9 : **Key diagram: cross-section of typical trench drains.** (1) Ground surface. (2) Original piezometric level on plane EH. (3) Piezometric levels on plane FG after drainage. (4) Mean piezometric level on plane FG after drainage. (5) Mean piezometric level on drainage inverts, after drainage. (6) Trench or counterfort drain. (7) Clay seal. (8) Impermeable boundary at depth.

Flow nets, derived from finite element analyses by E.N. Bromhead, have been drawn for an S/D ratio of 2, for two depths to an impermeable layer /defined by depth ratios, n, of 1.0 and 4.5** and for assumed ponding of water at ground level, i.e. h_o = D /Figs. 9 and 11/. From these, the piezometric levels on planes at any depth can readily be determined. For simplicity, however, attention is concentrated on the plane EH /Fig. 9/ at drain invert level. Piezometric surfaces, with respect to this plane, are plotted for the two depth ratios and various values of S/D in Fig. 12, on the assumption that

Fig. 10 : **Photograph of trench drain being dug in a till slope at Low Worsall, Yorkshire, April – May 1977 (with acknowledgements to the Northumbrian Water Authority).**

the permeability ratio $R_k = \frac{k_h}{k_v}$ is 1.0. For any given S/D ratio, the same plots, of course, also yield the piezometric surfaces for varying values of R_k through scale transformation.

It is now assumed that, in effect, a horizontal ponded surface exists at some depth p = D - h_o below ground level. This approximation, which is probably not unreasonable for clay soils in temperate climates, particularly as k often decreases rapidly with depth, enables the approach to be made more general by substituting h_o for D in Fig. 12. The approximation is, of course, also implicit in the theoretical curves in Figs. 13, 20 and 21.

From Fig. 9, the following definitions and relationships can be drawn with respect to plane EFGH after long-term drainage:

/i/ The average piezometric head, $\bar{h}$, on the plane FG between drains can be expressed as = $f_s\,h_m$, where f_s is a factor reflecting the shape of the relevant piezometric surface /3 in Fig. 9/ and h_m is the piezometric head mid-way between drains.

/ii/ The average piezometric head, h_{av}, on the whole plane EH, is given by $h_{av} = \dfrac{h_d B + \bar{h} S}{A}$

where h_d is the piezometric head at the drain inverts. In general, in the rest of the paper, h_d is assumed to be zero, when

$$h_{av} = \frac{\bar{h}\,S}{A}.$$

The expressions under /i/ and /ii/ above apply to horizontal ground. For ground of inclination β, with trench drains of high L/S ratio,

* In addition, the solutions are generally a function of the parameter q/k, the ratio of the infiltration and the permeability, which is not readily determined very closely.

**Increase of n above this value appears to have little effect on the flow net within the drain depth.

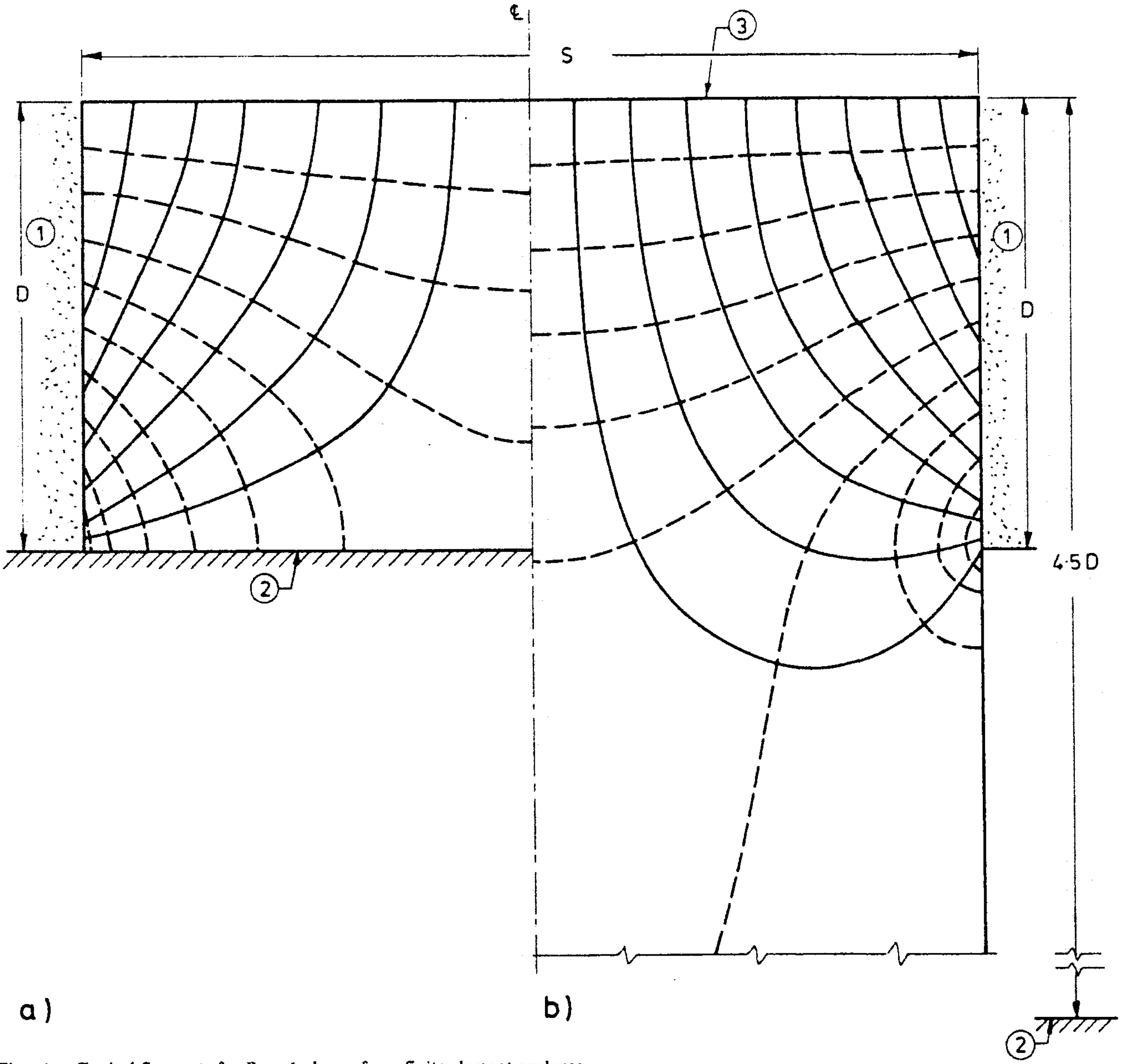


Fig. 11 : Typical flow nets for $R_k = 1$, drawn from finite element analyses;
a) for fully penetrating drain (n = 1)
b) for partially penetrating drain with n = 4.5.
(1) Trench drain. (2) Impermeable lower boundary. (3) Ponded upper boundary.

constructed down the line of steepest slope, the modified values of piezometric head will be given approximately by $h_o' = h_o \cos^2\beta$ $\bar{h}' = \bar{h} \cos^2\beta$, $h_{av} = h_{av} \cos^2\beta$, etc.

The data on Fig. 12 have been used to show how the ratios h_m/h_o and $\bar{h}/ho$ vary with S/ho, for n values of 1.0 and 4.5 and $R_k = 1.0$ /Fig. 13/.

As $\dfrac{\bar{h}'}{h_o'}$ $\dfrac{\bar{h}}{h_o}$ and $\dfrac{h_m'}{ho'} = \dfrac{hm}{ho}$,

this plot is valid for $\beta \geqslant 0$.

The efficiency of trench drains on a given cross-section is defined, with respect to the intervening mass of soil, as

$$\eta = \frac{h'_o - \bar{h}'}{h'_o} = \frac{h_o - \bar{h}}{h_o}$$

while the overall efficiency of the drains, on any given cross-section, is

$$\eta_{av} = \frac{h'_o - h'_{av}}{h'_o} = \frac{h_o - h_{av}}{h_o}.$$

Plots of $\bar{\eta}$ or η_{av} against S/ho can readily be made from the curves in Fig. 13, as shown later for $\bar{\eta}$. It will be noted that both $\bar{\eta}$ and η_{av} are independent of β.

Although, for convenience, the ensuing discussion is carried mainly in terms of $\bar{\eta}$ it should be borne in mind that the stabilizing effect of the actual trench drain installation will be a function of the true overall efficiency of the drains, which will depend on the variation of η_{av} over the site.

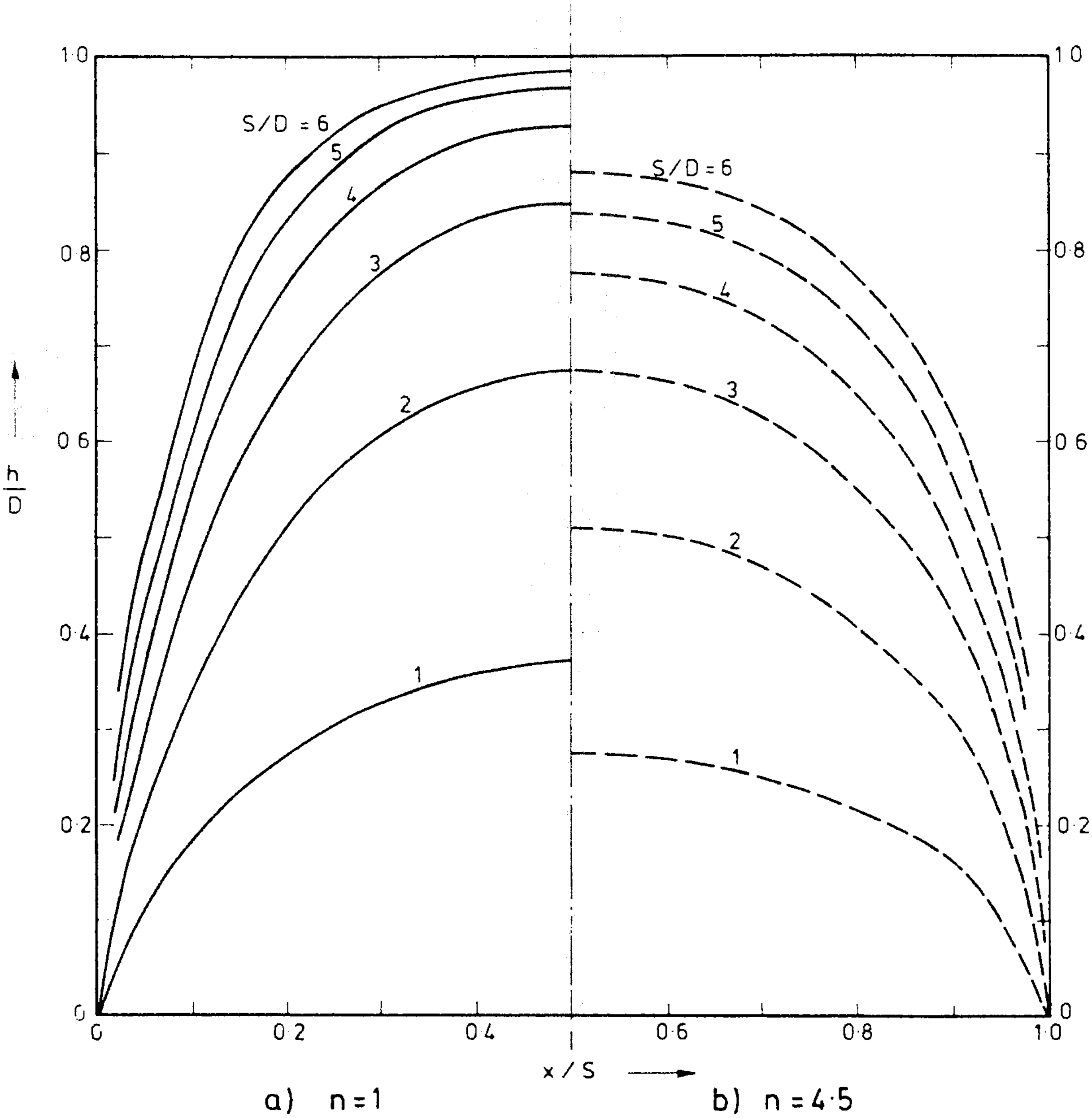


Fig. 12 : Curves showing the variation of piezometric level (at drain invert level and assuming $h_d = 0$) between trench drains, for R_k = 1 and various values of S/D.
a) For fully penetrating drains (n = 1)
b) For partially penetrating drains (n = 4.5)
Note. The ratios S/D and h/D are also taken to be approximately equivalent to S/h_o and h/h_o respectively. The effect of R_k 1.0 can also be obtained from these curves by the appropriate scale transformation.

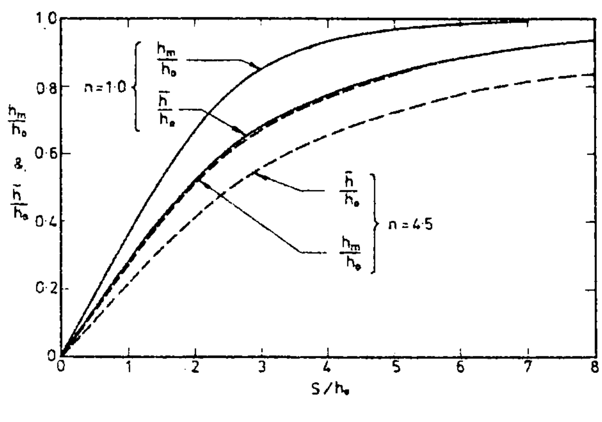


Fig. 13 : Curves showing the relationship between h_m/h_o and S/h_o, and between $\bar{h}/h_o$ and S/h_o, for fully penetrating drains (with n = 1.0) and partially penetrating drains, with n = = 4.5. R_k is taken as 1.
NB. The note appended to the caption for Fig. 12 also applies here.

Case records

Details of the performance of a number of trench and counterfort drain installations are given in Table 1. Only natural slopes have been included because, as Vaughan & Walbanke /1973/ have de monstrated for the London Clay, pore-pressures in cuts in over consolidated clays can be strongly depressed by the associate unloading for many years after the completion of excavation Indeed, the installation of trench drains in such a cut might wel be counter-productive initially, in accelerating the equilibration o pore-water pressures although, as the long-term condition is approa ched, they should, if sufficiently deep, hold the maximum piezo metric levels reached to lower values than would otherwise hav been the case. All the examples in Table 1. involve overconsolidate clays, or colluvium derived largely from these, and have piezome ters with their tips at or near to the plane of the drain inverts

One of the earliest well-documented descriptions of a counter fort or trench drain installation is that at Sevenoaks, Kent /Week 1969, 1970; Symons & Booth, 1971/. A downslope section of th site, showing the extent of a trial drainage installation, is show in Fig. 14a. The effect of the installation, on a section mid-wa between the drains, is shown in Fig. 14b. An example of the rea sonably long-term piezometric levels reached on a cross-sectio between these drains is given in Fig. 15. The material draine at Sevenoaks contains much coarse chert debris from the Hyth Beds and is probably the most permeable of all those listed i Table 1. Weeks /1969/ reports k values for the soliflucted lobe an sheet materials of about 4 x 10^{-6} cm/sec., measured in constar head tests on piezometers. On the other hand, the flow of groun water from upslope is probably also greatest at this site when directly to the north, a large area is occupied by the unconfine aquifer of the Hythe Beds /Fig. 14a/.

Fig. 14b is of interest in showing how the efficiency of drain can vary significantly up and down the slope, and how misleadin it could be to rely upon one monitored cross-section. In additio it should be noted that the effect of the drains is not fully fe until a point about 3.5 times the centreline drain spacing, A, belo their head. From this point /V/ the average effect of the drains tenc to increase to a point W, about 0.5 A before the lower end of th drains is reached, from where the effect rapidly dies out to zer at a distance of 0.3 A past the end of the drains. These values wi be particular to this site: the trend of these measurements is c general interest, however, and indicates that drains should b carried somewhat beyond the area needing to be stabilized, pa ticularly in the uphill direction. An alternative, which may b equally effective at the head of the unstable area, would be to e tend the drains less, but to provide them with „Y" or „T" shape extensions in plan, to act as interceptors.

Examples of other monitored cross-sections between trench drai are given in Fig. 16 and 17. Fig. 16 refers to a drainage installatic used, in conjunction with other measures, to stabilize a landslic which occurred during the construction of the M4 motorway Burderop Wood, near Swindon, Wiltshire /Skempton 1972, an forthcoming paper in Geotechnique by A.W. Skempton and J.G.l Dawson/.

The lowering of the piezometric line, for drain invert level, achiev in the first 1.6 years after completion of the drains is shown. In th case the material drained consists largely of a colluvium consisti of small lumps of Gault clay and fragments of Greensand in a m trix of completely remoulded Gault. Fig. 17 shows comparab data for the trench drain installation at Stag Hill, Guildford, Surr /largely after Simons, 1977/. This of special interest being, at 10 years after drain installation, the longest record available. In th case the drained material is colluvium consisting partly of littl disturbed brown London Clay and partly of similar material th has been much disintegrated to form a mudslide fabric. Duri trench excavation there was much overbreak where the disintegrat material predominated but little or none otherwise. The locatio of the trench sides is therefore uncertain* and they are shown wi a broken line in Fig. 17. There is also, as indicated, a degree uncertainty as to the position of the set of piezometers relative the flanking drains. Constant head permeability tests on piezomete from 1.5 to 12.5 m deep at this site, carried out by the author December 1965 and January 1966, indicate a permeability whi reduces linearly with depth, on a $\log_{10}k$ - depth plot, from abo 10^{-7} cm/sec at ground level to 5 x 10^{-10} cm/sec at a depth of 6

* This could apply, of course, on many other sites too. Tills tend stand well, however, /see Fig. 10/ except where there are maj inclusions of cohesionless material.

Site				Drain details					Drain performance							
Ref.No.	Name	Approx. slope	Principal material	B	D	S	h_o	S/h_o	h_m	$\bar{h}$	h_{av}	h_m/h_o	$\bar{\eta}$	η_{av}	T years	Source
1	Bredon Hill /Ur. Slip/	14°	Ur. Lias	.8	3.0	11.3	c.2.6	4.35	.90	.72*	.67*	.35	.72*	.74*	1.2	Skempton and Henkel /1958/
2	Sevenoaks /Lobe D/	7°	Lr. Greensand over Wealden	.9	4.6	17.4	3.1	5.61	1.60	.95	.90	.52	.69	.71	2.2	Weeks /1969/
3	Guildford /P402-4/	7°	London Clay	.8	5.0	15.3	3.97	3.85	2.36	1.86	1.77	.59	.53	.55	10.5	Largely Simons /1977/
4	Hodson /H1-2/	5°	Gault clay	.6	3.7	8.5	3.55	2.39	c.2.2	c.1.62	1.51	.62	.54	.57	1.5	Skempton /1972/
5	Hodson /H3-4/	5°	Gault clay	.6	3.7	8.5	3.55	2.39	c.1.5	c.1.12	1.05	.42	.68	.70	1.5	"
6	Burderop Wood /J6-9/	6°	Gault clay	.6	3.7	11.6	3.32	3.49	2.28	1.56	1.48	.69	.53	.55	1.6	Skempton /1972/
7	Burderop Wood /K9-14/	6°	Gault clay	.6	3.7	25.0	2.90	8.86	c.2.35	1.93	1.88	.81	.33	.35	1.6	"
8	Boulby /N2-3/	14°	Till	.5	5.2	11.0	4.86	2.26	2.52	2.02*	1.93*	.52	.58*	.60*	5.8	Cleveland Potash Ltd and Author's files
9	Boulby /T2/	15°	Till	.5	5.5	6.0	2.9	2.07	1.0	.80*	.74*	.34	.72*	.74*	5.9	
10	Barnsdale /D7-9/	11°	Ur. Lias	.7	3.5	11.3	2.77	4.08	1.35	.88	.83	.49	.68	.70	0.7	Chandler /in press/
11	Barnsdale /D22-24/	7°	Ur. Lias	.7	5.0	11.3	3.43	3.29	3.0	2.41	2.27	.87	.30	.34	0.4	"

Notes. All dimensions in metres.

* = estimated values, using an f_s value of 0.8.

TABLE 1. Details of the performance of some trench and counterfort drains in natural slopes.

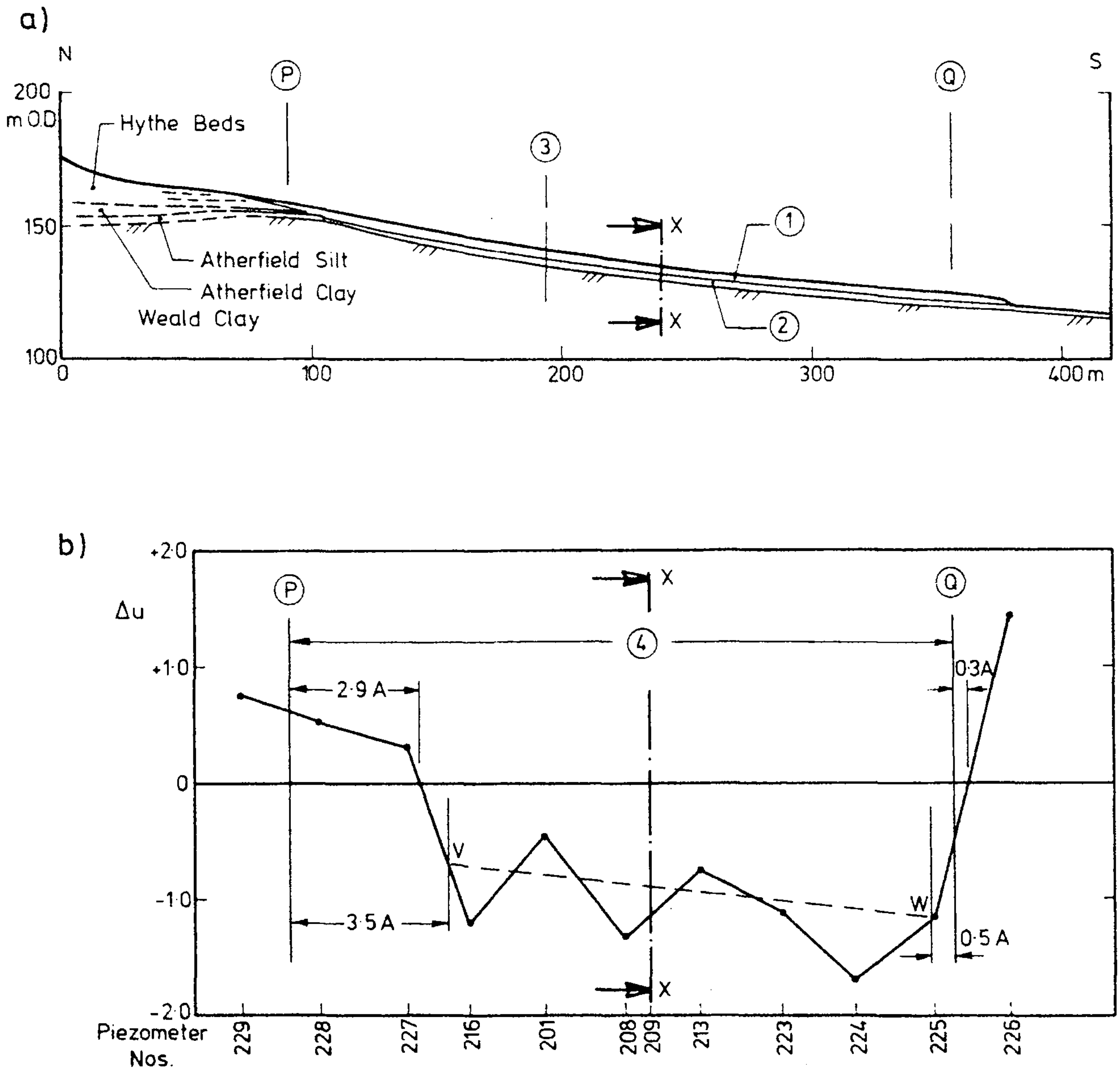


Fig. 14 : Counterfort drain installation at Sevenoaks, Kent (after Weeks,1969, 1970, & Symons & Booth,1971):
a) Longitudinal section.
b) Change in piezometric level, Δ u, at 4.9 m depth on centreline between drains as a result of the drainage (measured c. 1 month after installation).
(1) Upper solifuction lobe. (2) Lower solifluction sheet. (3) Centreline of proposed road, later rerouted. (4) Extent of counterforts (L ≙ 14.5A), 9.1 m to each side of the piezometer line.

In contrast to Sevenoaks, this site is on the flank of an isolated knoll, with probably a relatively small inflow of ground-water from upslope.

A long-term record for piezometer P404 at Guildford, kindly provided by Prof. N.E. Simons /1977/, is compared with the predrainage piezometric levels briefly recorded by a nearby piezometer /P103/ in Fig. 18. Unfortunately the original piezometer was destroyed during drain construction and not replaced for some time, so there is no record of the immediate drawdown produced at P103, nor of the behaviour in the succeeding 3 years. The long-term readings from March 1969 onwards to the present are reassuringly steady however.

A shorter, but more continuous record is provided by a drainage installation in a slope of till, with some sand and laminated clay layers, at Boulby, Yorkshire. This is given in Fig. 19, which is based upon data kindly provided by Cleveland Potash Ltd. Again the period for which the pore-pressures were monitored before installation of the drains is shorter than desirable, but the very rapid initial drawdown, diminishing as the depth of piezometer tips below trench invert level increases, is well shown for all three piezometers. Of great interest is the somewhat disturbing fact that during the first winter after installation of the drains, the piezometric levels rose strongly, almost to their pre-drainage values. This is particularly striking at piezometer N3*, where the pre-drainage artesian piezometric level of 1.68 m above original ground level was brought down to at least 1.3 m below this ground level in the September following

* A layer of laminated soils and clays was noted in the borehole for N3, in the vicinity of the piezometer tip. Dr P.R. Vaughan suggests that on cutting the drain trench through this layer the silt layers would wash out, allowing the clay laminae to fold or slump down, thus effectively sealing off the more permeable laminae.

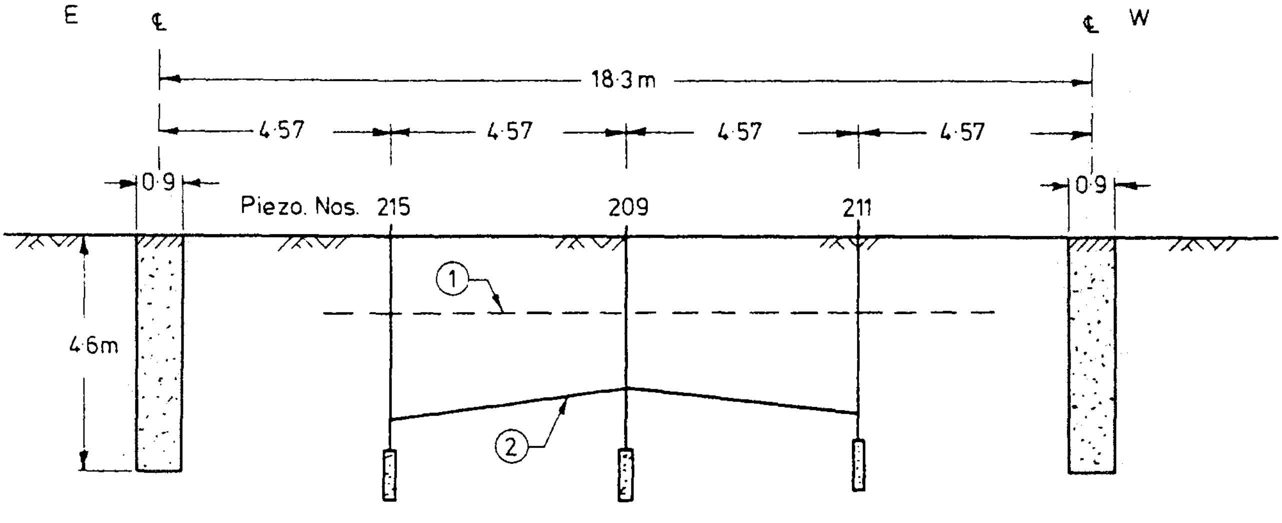


Fig. 15 : Cross-section of counterfort drain installation at Sevenoaks, Kent, showing the effect of drainage (after Weeks,1969, 1970).
Note. In this paper all such cross-sections are drawn looking downslope. (1) Approximate piezometric level before drainage. (2) Piezometric levels after drainage.

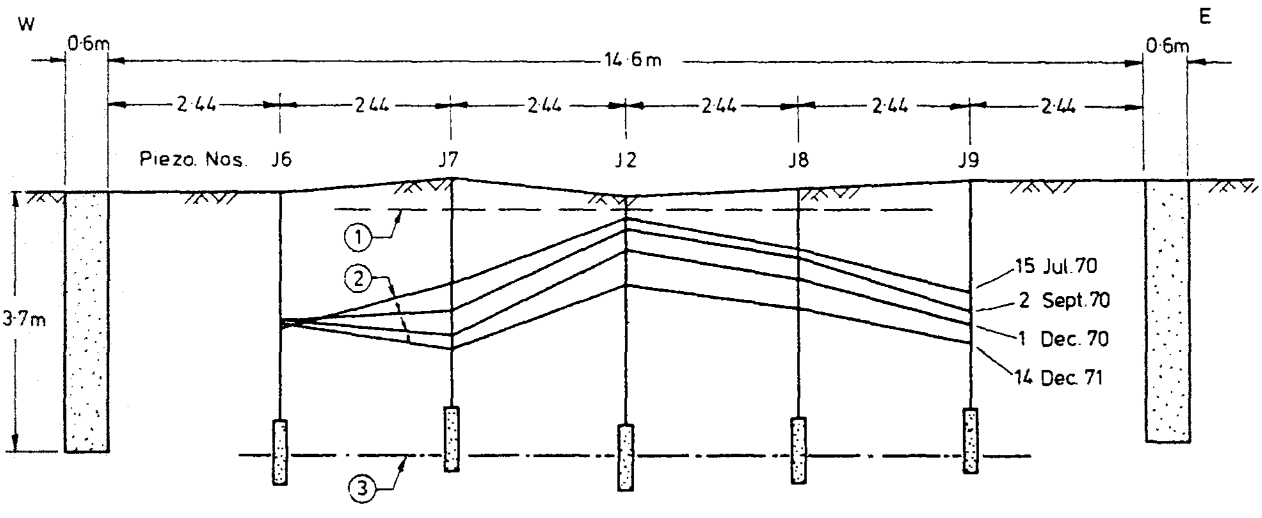


Fig. 16 : Cross-section of trench drain installation at Burderop Wood, Wiltshire, showing the effects of drainage at various times after installation (after Skempton,1972).
(1) Approximate original position of the piezometric surface. (2) Positions of the piezometric surface following drainage (the drains were constructed between 1 and 3 July 1970). (3) Approximate position of the slip surface.

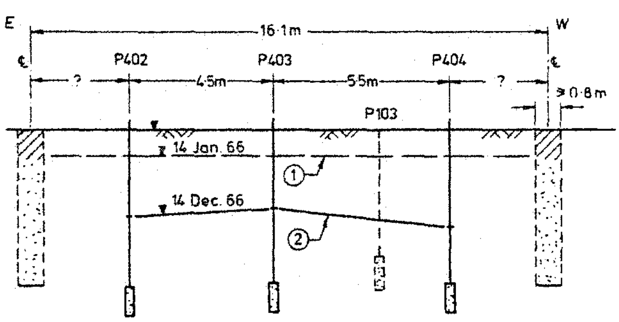


Fig. 17 : Cross-section of trench drain installation at Guildford, Surrey, showing the long-term effect of drainage (largely after Simons, 1977). (1) Approximate piezometric level before drainage, based on piezometer P103. (2) Position of the piezometric surface about 10.5 years after installation of the drains.

construction of the drains in July 1971, only to return to an artesian pressure of + 1.37 above the same datum in the following winter of 1971-72 /i.e. 90% of the September draw-down was temporarily lost/. By the next winter of 1972-73, however, the drains seem to have taken hold, and this situation has been satisfactorily maintained up to the present.

Although data are sparse, there are indications from other sites that this strong rise in piezometric levels during the first winter after drain installation may be a fairly general phenomenon. Some lesser, but significant, seasonal rise may even occur in the second winter after installation, but after that the present evidence suggests that seasonal rise is strongly damped by the presence of the drains and that around the year reduction in piezometric leveš is achieved. Whether this can be regarded as permanent is of course questionable, as discussed subsequently. An important lesson to be drawn from this delayed effect of drains is that some additional corrective measure, such as a toe fill, may be required to ensure stability, not only during the period of initial drawdown and consolidation but also during the first one, or possibly two, winters.

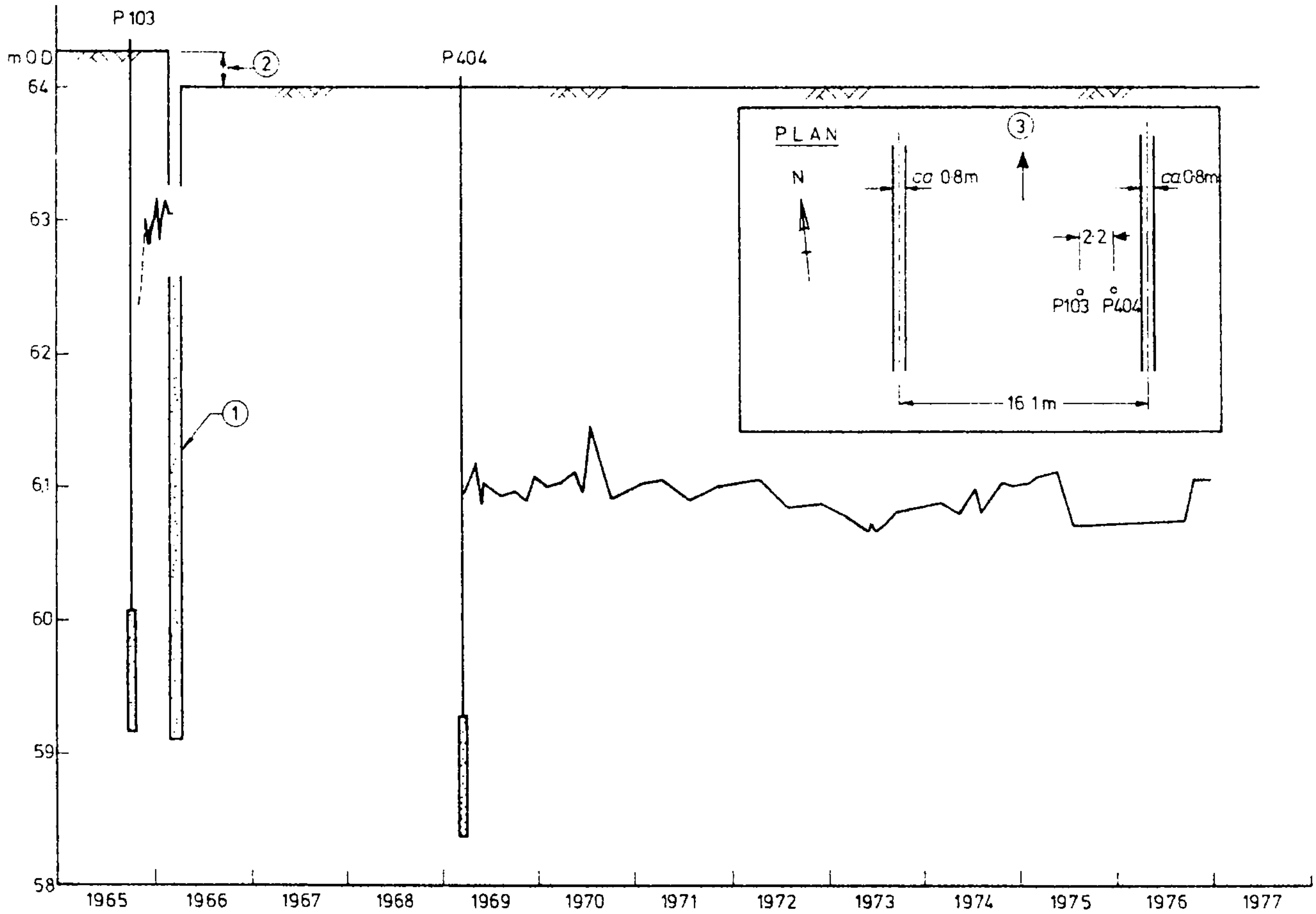


Fig. 18 : Long-term comparison of piezometric levels before and after drainage at Guildford, Surrey (largely after Simons,1977). (1) Trench drain. (2) Small lowering of ground level during construction of drains. (3) Downslope.

Henkel /1957/ remarks that the rate of drop in pore-water pressures following trench or counterfort drain construction is much more rapid than would be expected from a calculation based upon the coefficient of consolidation obtained from oedometer tests. More recently Chandler /in press/ has monitored this process for a trench drain installation in the Upper Lias, and concludes that the observed rate is at least 4 times as fast as that occurring in one-dimensional consolidation in the field, predominantly through stress relief effects.

Dr. M. Hamza, of Imperial College, is currently examining the immediate, undrained pore-pressure changes that are caused by the stress release consequent upon excavating drain trenches. Using a non-linear finite element programme, he finds, for trenches excavated in saturated, overconsolidated clay, with S = 15 m, D = 5 m, K_o = 1.0, B = 1.0 and h_o = 4.5 m, that the average immediate reduction in h_o, for the central three-quarters of the mass between the drains, is about 1.0 m for a = 0, and 1.5 m for a = -0,12* /a is as defined by Henkel 1960/. This stress release effect is clearly an important component of the rapid drawdowns observed soon after drain installation /e.g. Fig. 19/: given favourable boundary conditions, a further component will come from the following consolidation stage.

Comparison of theory with field behaviour

From the approximate theory developed earlier, the families of curves in Figs. 20 and 21 are readily derived. These can be compared with the field data summarised in Table 1.

The aim of Fig. 20 is to determine values of the shape factor f_s, for the piezometric levels between drains. Thus h/h_m is plotted against x/S for a typical drain installation with S/h_o = 3, for n values of 1.0 and 4.5 and R_k values of 1 and 5. As can be seen, the various theoretical curves do not differ greatly.

Against this background are plotted the available field observations. All these are from natural slopes, except those from the cutting at Cumnor Hill. In general there is an appreciable scatter, with the field observations falling on or below the theoretical curves. The theoretical values of f_s, for the curves shown, range from 0.76 to 0.80. It is suggested that in practical cases where only the mid-point piezometric height, h_m, is known, an estimate of the value of $\bar{h}$ can be made by taking this as = $f_s h_m$, with f_s = 0.8. This approximation is probably slightly conservative.

In Fig. 21a, theoretical curves for the relationship between h_m/h_o and S/h_o are shown for n values of 1.0 and 4.5 and R_k values of 1 and 5. With two exceptions /Bredon Hill and Sevenoaks/ these bracket the available field data. Finally, in Fig. 21b, theoretical curves for the variation of long-term drain efficiency $\bar{\eta}$ are given, for similar values of n and R_k. A comparison of the available field data with these curves leads to the following conclusions:

/i/ With the exception of point 11 for Barnsdale, which is rather short-term and whose efficiency is probably still increasing, none of the field points lies below the curve for n = 4.5 and R_k = 1 /curve G/.

/ii/ Up to an S/h_o ratio of 5, the bulk of the field data lies between curve G and the curve for n = 1.0 R_k = 5**/curve H/. If site average are considered, all the data, with the exception of point 1 for Bredon Hill, fall between curves G and H and lie generally in the upper half of that space.

* Equivalent to a value of the pore-pressure parameter A of + 0.16.

**No relevant measurements of the ratio R_k are available, but 5 has been chosen as a likely upper limit for the materials involved. Mitchell /1956/ provided some guidance for this choice.

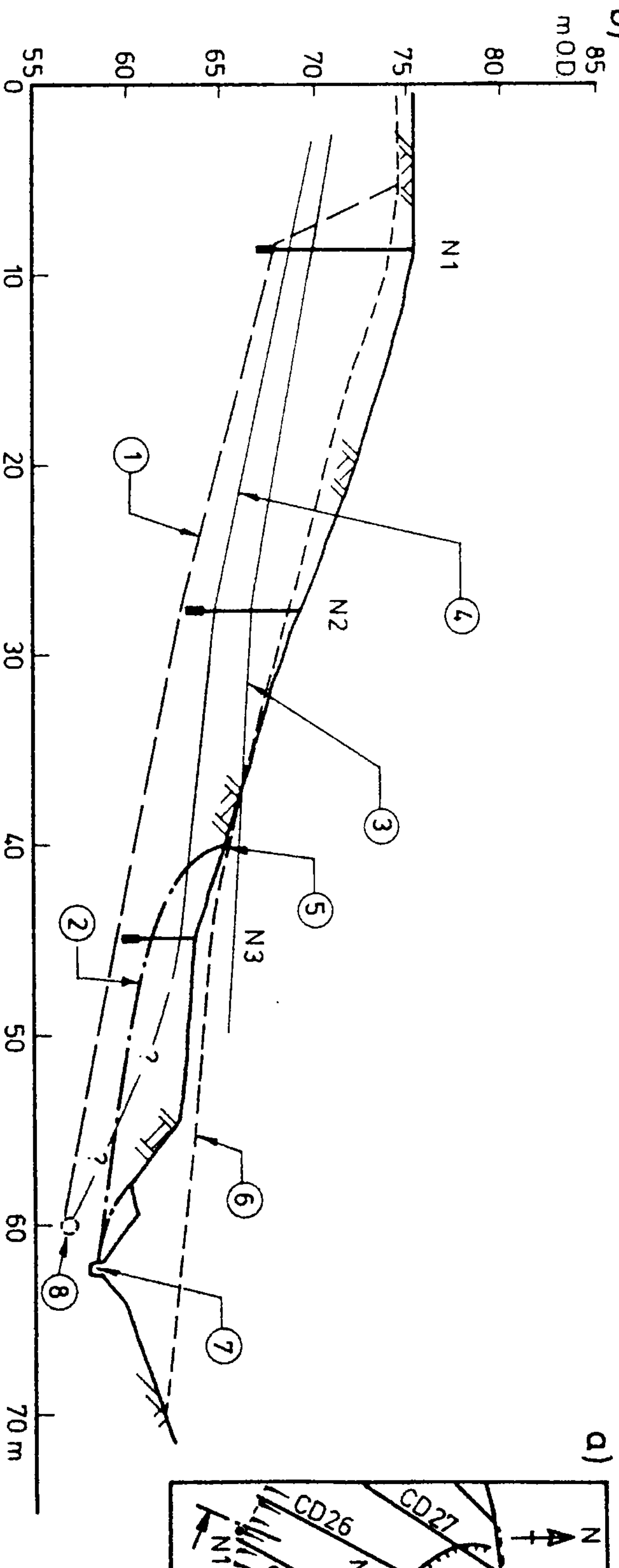


Fig. 19 a, b)

It is suggested that, until further observations become available, the curves H and G in Fig. 21b can be regarded as tentative upper and lower bounds for the design of trench or counterfort drains in overconsolidated clay slopes under temperate climatic conditions, up to an S/h_o ratio of 5 or 6. Higher ratios than this are unlikely to be under consideration. Curve G is likely to be a conservative lower bound, unless in a given case the n and R_k values or the hydrology are particularly unfavourable.

All the drainage installations in natural slopes that are listed in Table 1 appear to have been successful as a stabilizing measure. The efficacy of such drains in cut slopes is less assured, and several examples of failures subsequent to the installation of counterfort drains in railway cuttings in stiff fissured clays are given by Cassel /1948/ and by Ayres /1961/. This is probably due in part to the generally steeper slopes and shorter lengths of drain involved.

It is hoped to make a more detailed examination of trench and counterfort drains, including an examination of the necessary criteria for preventing the occurrence of slips between such drains and the effects of underdrainage, or of artesian pressures at depth, in a forthcoming papert with Mr E.N. Bromhead.

Clogging of drains

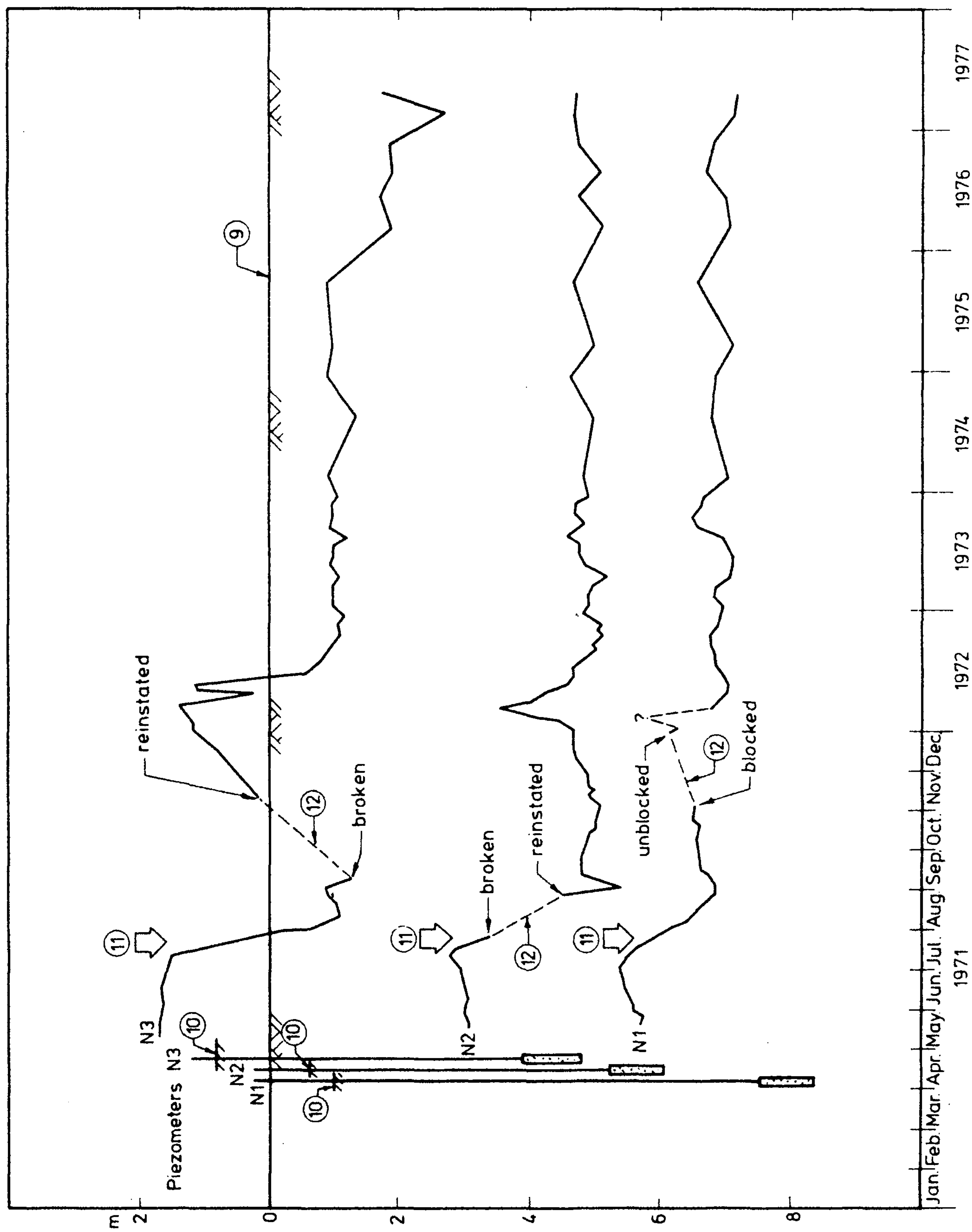
Drainage, as mentioned earlier, is one of the most effective and most used of stabilizing measures for slopes. At the same time, there is generally associated with such drainage installations some degree of doubt concerning their long-term performance.

Clogging of both pipe and permeable aggregate drains probably occurs most commonly by the ingress and lodging of silts and fine sands. Much attention has been devoted to this problem, particularly to the design of filters, and there is a correspondingly large literature on this subject, briefly summarised, for example, by Spalding /1970/ and Cedergren /1975/.

The matter is not yet fully resolved, however, and as in practice both specifications and workmanship, frequently fall below a good standard, cases of clogging by siltation are probably quite common, as indicated by Keene /1951/. Well detailed case records of this type of failure are less so, but there are indications that permeable aggregate drainage systems without proper filter protection may tend to silt up after a working life of about 10 to 20 years.

In the case of some bored horizontal drains in Derbyshire, the following information has been kindly provided by Mr C.V. Underwood, the County Surveyor /pers. comm./. Fifteen perforated PVC drains, of internal diameter 40 mm and lengths between 30 and 40 m, were installed without filters in 1971 at an angle of 5^0 above the horizontal through laminated silty clays. By 1975 all the drains had become inoperative, largely through siltation. The earlier horizontal drain installation at Otley in Yorkshire, however, constructed in 1964-67 using galvanised iron drains provided generally with pre-cast porous concrete filters, still appears to be working satisfactorily /G.R. Forrester, pers. comm./.

Some other ways which drains become clogged are listed by Keene /1951/. Nevertheless, after siltation it is probable that geo- and biochemical factors pose the greatest threat to the satisfactory operation of drainage systems, yet this subject has been largely neglected by civil engineers. There is a fairly extensive literature on the subject, however, in the fields of agricultural engineering and water supply. One of the most common geochemical effects is the precipitation of hydrated ferric oxide /iron ochre/. Useful studies of iron /and manganese/ ochre in near-surface agricultural drains have been made by Alcock /1973/ and by Thorburn & Trafford /1976/. The latter authors distinguish two main groups of ochre forming soils; peats and slightly organic marine sediments and pyrite-bearing rocks. A detailed discussion of measures adopted to prevent ochre deposition occurring in a deeper, civil engineering drainage system in Antwerp is given by Brand /1968/. In the earth dam context, problems arising from the geochemical and biochemical precipitation of iron compounds are discussed in a pioneering paper by Infanti & Kanji /1974/. They present a photograph showing a sand filter ce-



c)

Fig. 19 : Long term comparison of piezometric levels before and after drainage at Boulby, Yorkshire (with acknowledgements to Cleveland Potash Ltd):

a) Site plan (CD = counterfort drain).

b) Cross-section through piezometers N1 to N3.

c) Variation of piezometric levels with time.

(1) Invert of drains. (2) Approximate slip surface. (3) Max. piezometric levels measured before drainage. (4) Max. recorded piezometric levels in the period from 1 to 6 years after drain installation. (5) Slip scarp. (6) Regraded profile. (7) Stream. (8) Culvert. (9) Original ground level. (10) Regraded ground levels. (11) Installation of trench drains. (12) Piezometers temporarily out of action.

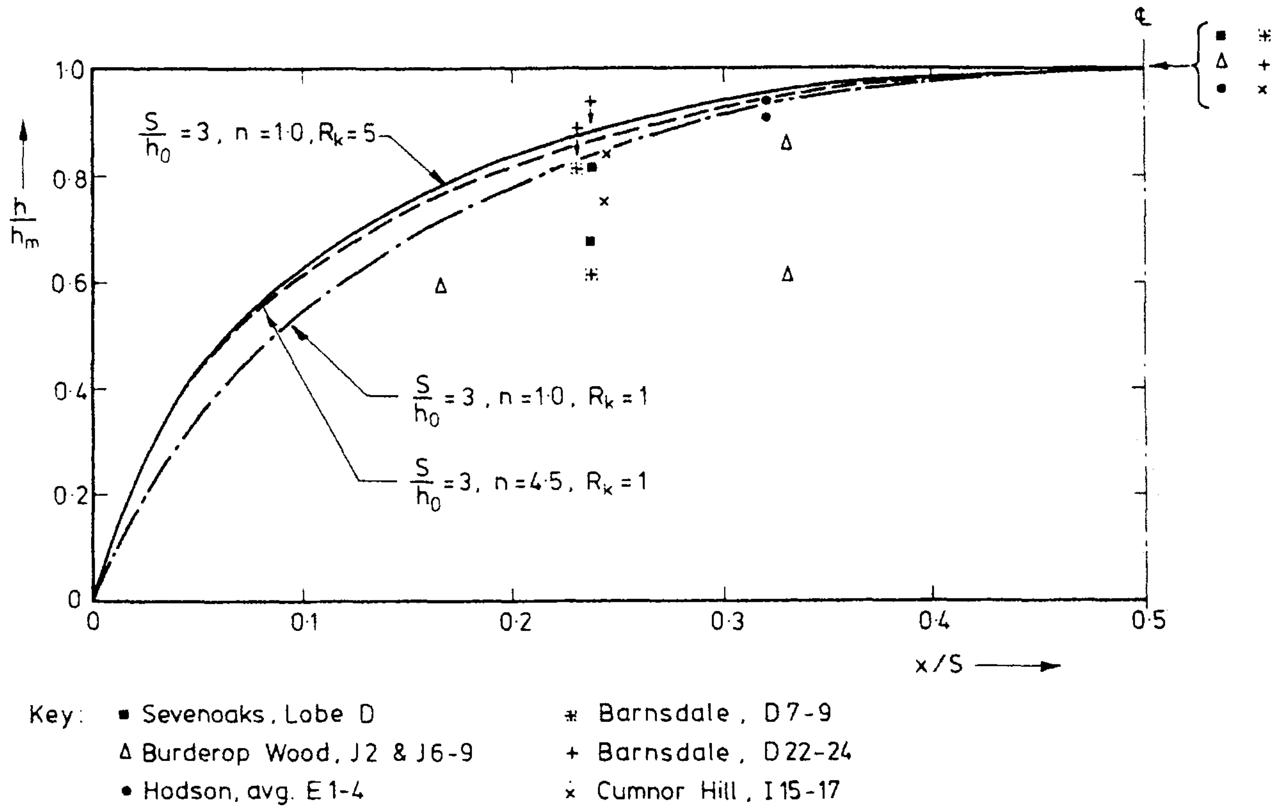


Fig. 20 : Computed relationships between h/h_m and x/S for R_k = 1 and 5 and n = 1 and 4.5, compared with the available field data.

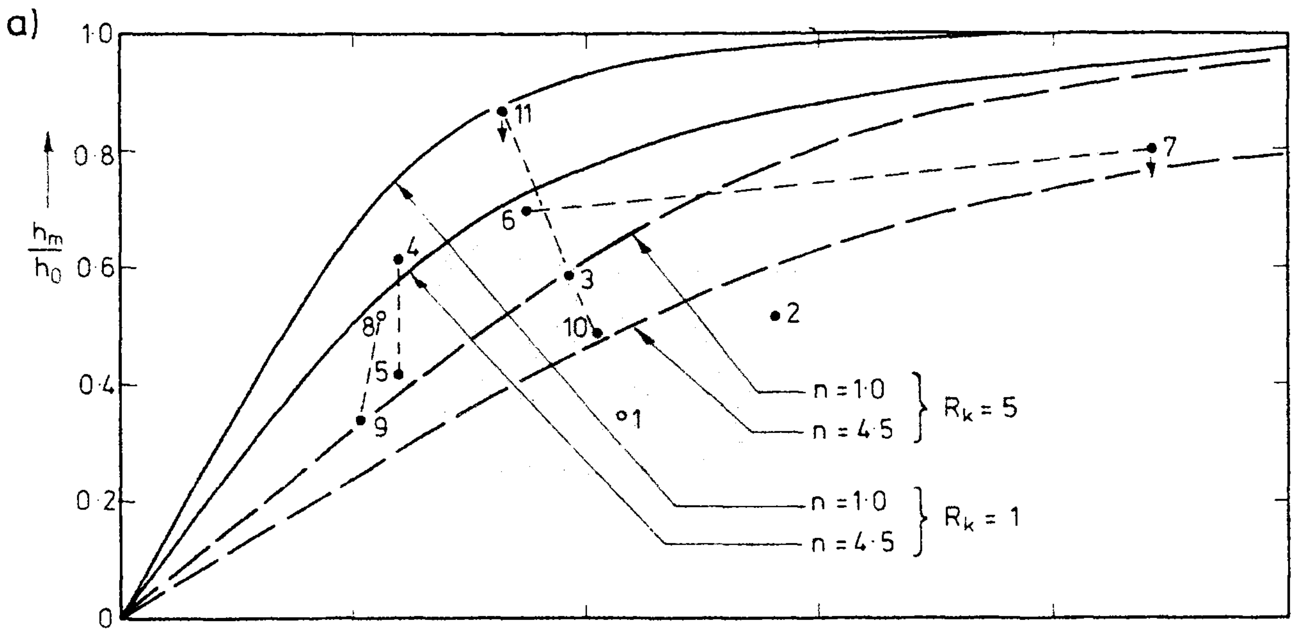


Fig. 21 a)

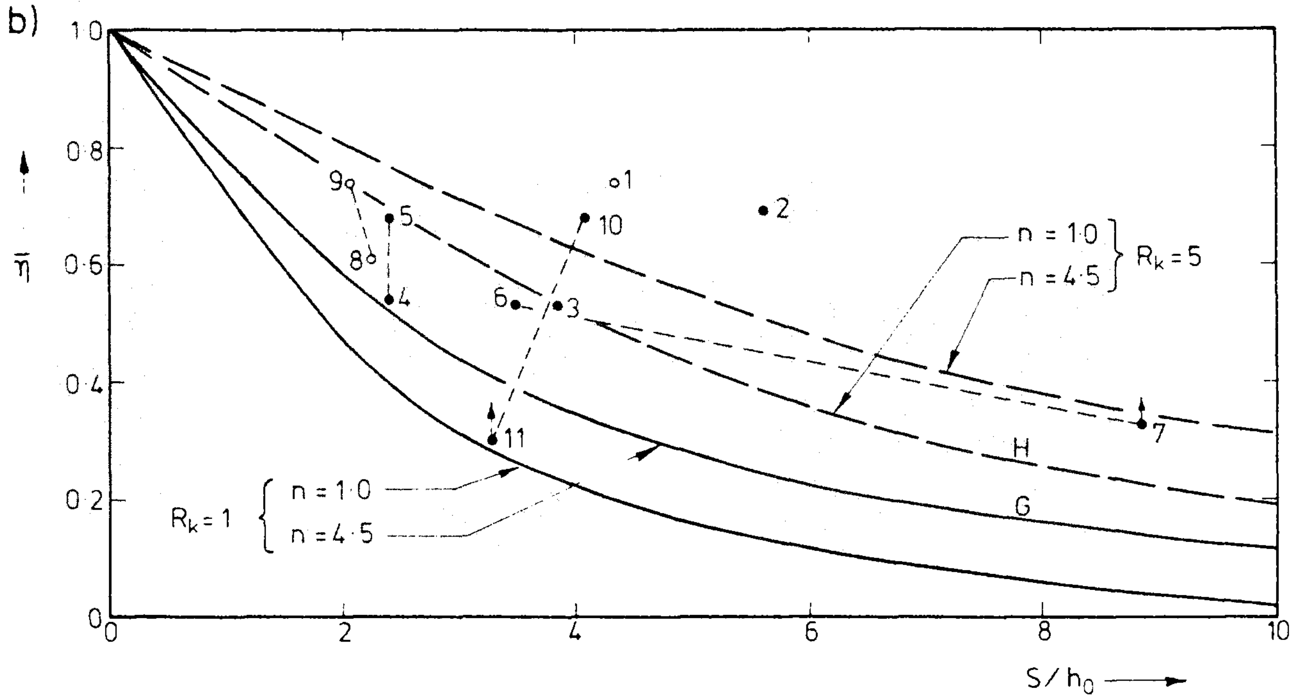


Fig. 21 : Comparison of approximate theory for trench drains with the field data summarised in Table 1.
a) Computed relationshp between h_m/h_0 and S/h_0 for R_k = 1 and 5 and n = 1 and 4.5, compared with the field data.
(For key to the numbering of the points (in Figs. 21a) and b)), refer to Table 1. Solid circles indicate the more reliable data).
b) Computed relationship between $\bar{\eta}$ and S/h_0 for R_k = 1 and 5 and n = 1 and 4.5, compared with the field data.
(In both Fig. 21a) and b), points from the same site are connected by a broken line.)

mented up by such processes, which are probably akin to those which form iron pans in natural ground. There is also some experience of well screen clogging through bacterial and other activity /M.S. Eglinton, pers. comm./.

Cases of clogging through the precipitation of other materials seem to be less common but blockage of drains, flowing partly full, by encrustations of calcium carbonate have been reported from limestone areas.

Drains made from artificial fabrics are currently being developed for use in civil engineering /e.g. Healy & Long,1972, Long & Healy, 1977/. These seem promising and do enable the manufacture of the filter element to be closely controlled under factory, rather than site, conditions. Preliminary tests however, indicate that clogging by silt is still a problem /Hoogendoorn & van der Meulen, 1977/. Further development and continuing testing of these materials is clearly necessary. The scope of the testing should also be widened to include geochemical and biochemical factors.

2 - SUBMITTED PAPERS

The 10 papers on Theme 3 cover the following main topics: site investigation, regional landslide studies, various slope stabilization works and dynamic effects.

The paper by **Yamaguchi** describes the use of repeated electrical resistivity surveying on a recent landslide in order to guide in the location of drainage wells and to check their efficacy. Wells are sited in zones of low apparent electrical resistivity, considered to correlate with an abundance of ground-water, or greatest slide danger. The first such well gave a strong discharge and a subsequent repeat of the resistivity survey showed that the associated zones of low apparent resistivity had disappeared. The method seems a very indirect way evaluating ground-water pressure distributions but is clearly much cheaper and quicker than the conventional one using piezometers. It could add to the value of a future trial of this method if the pre- and post-drainage situations were to be checked with piezometers.

The paper by **Bazyński & Frankowski** describes the investigation and stability analysis of a landslide affecting a railway cutting at Sadowie. The slide originated in 1934 and involves loess overlying Miocene clays. In the investigation, Dutch penetrometer tests were much used, but it is not clear to what extent these were useful. In the stability analysis of this slide on pre-existing slip surfaces it is disturbing to find no mention of residual strength, and that the strength testing and stability analyses are carried out in terms of total stresses.

An interesting, though tantalisingly brief, description of landslide problems on about 600 km of mountain roads in N. Bengal and Sikkim, is given by **Chopra**. The complexity of mass movements in such a region, and the importance of good road location, surface drainage and the control of erosion, are well brought out.

In his paper, **Fujita** reviews the occurrence of slides around reservoir margins in Japan, with particular reference to the influence on these of variations in reservoir water level. The paper gives a number of case records of slides occurring both on flooding as well as on drawdown. Much of the work done at Vaiont /e.g. Kenney,1967/ is relevant to this topic and one would also have expected to find reference to the concept of critical pool level.

Mencl, Papoušek & Paseka report on a case where an unstable slope of marly Neogene clays NW of Brno was successfully built upon. Despite complex geological and hydrogeological conditions, useful backcalculations of the stability of some of the landslides are made. As often found elsewhere, at comparable stress ranges, the laboratory value of residual strength lies 2 or 3 degrees below the values obtained by back analysis. The slopes were stabilised mainly by a combination of an anchored pillar wall and some horizontal drains and by minimising the size of cuts for roads and sewers. No monitoring of these measures is reported.

The paper by **Flimmel** deals with horizontal drain and pile wall installations in a more general way. Useful practical experience, gained on numerous landslide stabilisation schemes in Czechoslovakia, is given for both the above types of corrective measure. Some guidance is also given on the drain discharges and lowerings of ground-water level achieved. No mention is made of filters being provided on the horizontal drains.

A brief account of the stabilisation of an 80 m high rock slope in N. Bohemia, mainly by the use of pre-stressed, horizontal cable anchors, is provided by **Zajíc**. An outline is given of the comprehensive joint survey made and of the anchor design. It is to be hoped that long-term monitoring of this installation, which is currently being carried out, will be undertaken.

The final three papers all deal with dynamic aspects of slope stability, with particular reference to blasting. **Musaelyan, Kochurov &Lavrusevitch** describe field experiments in which the response of loess slopes to blasting was examined. The blasting was not carried out until the settlement of the loess, following preliminary wetting had been completed. The seismicity factor measured, for which the slope movements are known, is compared with those associated with natural earthquakes. **Dvořák** discusses the technique of removing parts of a landslide by blasting, for instance as part of the stabilization of a rock mass. He gives an example of a rock slide unintentionally initiated by blasting and suggests an approach whereby the appropriate size of blasting charge, that will not endanger the overall stability, can be determined. Finally, in a rather general paper, **Shahunants & Fedorenko** discuss principles of slope stabilization and management in folded mountain regions. These include, in addition to normal slope stabilization measures, the provision of dykes to deflect or retain landslide and mudflow debris, the provision of overflow channels around dams formed by large natural landslides and the use of conventional and nuclear explosion to shake down threatening masses. In the latter connection it is pointed out that reliable methods of predicting the run-out of slides are not yet available.

3 - RESEARCH NEEDS

Development of our knowledge of the efficacy of slope stabilization measures depends primarily on comprehensive and long-term monitoring of the performance of the various methods in use. In general, the essence of such monitoring is the carrying out of basic operations, such as the measurement of movements, stresses, pore-water pressures and drain discharges, in a well planned and thorough manner. In other words, although there is obviously room for its further development, sufficient technology already exists; it is interest and determination that are crucial if a valuable body of detailed case records is to be built up.

In the case of corrective cuts and fills, the main pre-requisite of success is a proper investigation of the landslide itself. The monitoring of movements, at the ground surface and preferably also with depth, should be an invariable practice and the values of $\bar{B}$ during construction should be measured by suitable piezometers. The main research needs here concern the difficulties of assessing the degree of stability of the existing slopes or landslides and of deciding what is an appropriate degree of improvement. In this connection, more knowledge of stress-strain conditions throughout the whole landslip mass especially in the approach to failure is required. This advance beyond the constraints of the limit equilibrium approach will also need to be accompanied by a re-examination of the usual ideas of „factor of safety".

With respect to drainage measures, there is a great need for properly monitored performance records of all the methods. The initial ground-water conditions should first be established for at least one full season, and preferably longer. In addition, measurements of the variation of permeability with depth, and the permeability ratio k_h/k_v, are highly relevant. As drainage can take a considerable time to become effective, long-term monitoring of its progress is essential. This needs to cover not merely the consolidation phase but also the succeeding steady state phase, in order to discover any deterioration through clogging or other causes.

In most drainage systems, the danger of clogging is usually present. Much effort is required here, not only in a research context but also to devise means of protection against clogging which will be satisfactory under site conditions. It is also important for this research to cover clogging through geo- or bio-chemical processes as well as that which can arise by siltation.

Concerning the considerable variety of other corrective techniques, only three research needs will be touched upon. In the case of permanent soil or rock anchors, efforts should be made to explore more fully their long-term behaviour, particularly with regard to stress loss and corrosion. For other corrective structures, and particularly for heavy cantilevered or anchored pile restraining structures there is a need for more measurements of deflections and earth pressures, and study of the relation between these. There is also room for further exploration of the extent to which geophysical investigation techniques can be of help in stabilization work.

Finally, in view of the close link between landslides and erosion, it would seem advisable to direct more attention towards methods of controlling the latter, on both the local and the regional scale.

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