# The In Situ Shear Behaviour of Fissured Soils

## Le Comportement sous Cisaillement In Situ de Sols Fissurés

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SYNOPSIS In many transported clay soils there is a pattern of jointing consisting of vertical cracks due to shrinkage and inclined fissures due to shrinkage and swelling. The 'line survey' technique for measuring all the important joint parameters is described, and typical values are given. A full-scale test to failure of a vertical bank showed that the pre-existing pattern of fissures initiates distress and a particular failure mechanism. Overall stability of the mass is probably governed by the residual strength along the joints.

## INTRODUCTION

Many clay soils exist which, while being stiff, are weakened by cracks or fissures. These inherent defects have been the main cause underlying the occurrence of landslides affecting housing schemes, or roads, or high embankments, as well as the sudden failures in deep excavations. The engineering behaviour of a soil mass, in particular that connected with shear, depends to a large extent on these joints in the material and yet the existing theories, methods of testing or design rules cannot always be safely applied. These problems have been recognised by Skempton (1966), Bishop (1966) and others, and for well-known materials, such as the London clay, guidelines have developed for the analysis of slope stability. However, no universally applicable design methods are available for predicting the overall strength of a large fissured clay mass.

## JOINTS IN CLAYS

The pattern of cracking in certain residual soils, such as weathered shales, has been explained by the inherited jointing along bedding planes or by cross-jointing plus subsequent shrinkage cracking. In many transported soils little information exists on the definite modes of origin or occurrence of such joints and the abundant inclined slickensides such as those shown in Figure 1.

There are a number of different types of joints found in these clay profiles, several of which can readily be explained by such simple modes of origin as tensile failure during shrinkage on drying causing vertical cracks, or shear failure in a shrinking mass at a depth where the major principal stress may be compressive or positive in a vertical direction, but the minor principal stress in the horizontal direction reaches a negative value sufficient to cause failure. On occasion shear failure may have caused such jointing during past ground movement in landslides, or some other tectonic effect forming striated slick surfaces. Some chemical effect may also have occurred causing changes in volume and distortions within the mass. However, the relatively frequent presence of many fairly large planar fissures, which are very smooth, or have a shiny surface and are termed 'slickensides', is not clearly explained in the literature.

Fig. 1 Abundance of inclined slickensides

Many possible reasons have been listed for the origin of such fissures and joints as follows :

(i) During consolidation, or deposition of additional load (Casagrande, 1949)
(ii) During swelling, or rebound from decrease in overbuden pressure (Casagrande, 1949)
(iii) Syneresis, a colloidal phenomenon (Eide, 1967)
(iv) Chemical changes - shrinkage (Kallstenius, 1963)
(v) Tectonic movements and landslides (Skempton, 1966)
(vi) Inherited from bed-rock (Fookes, 1965)
(vii) During drying out (Corte and Higashi, 1964)
(viii) Large lateral stresses (Aitchison, 1953; Terzaghi, 1961)

The last explanation is thought to apply to the phenomenon of slickensiding in most South African soils and some work in confirmation of this has been reported by Blight and Williams (1971).

Observations of profiles in stiff clay throughout South Africa yield a clear pattern of jointing. Beginning at the surface there is a layer of variable thickness of 'topsoil' which is granular in nature, either being a cover of windblown sand, or a 'mulch' formed by the breakdown of clayey material under the moisture extremes at the surface. This mulch often masks any pattern of surface cracking. The next horizon exhibits many vertical cracks which can be several centimetres wide near the surface and extend to depths of one metre or more. The vertical cracks generally have rough surfaces, though, on occasion, exhibit the formation of a shiny clay skin. The third major horizon then contains a system of fissuring or fairly planar inclined joints, which are often so shiny as to be called slickensides. At first sight these fissure inclinations do not appear to be concentrated around any single value, but have dip angles between about 30° and 60°. A typical soil profile of this type is given in Figure 2. The spacing between fissures appears to increase with depth, until, in the case of the transported clay at Vereeniging, the pebble marker and water table are encountered at about 7 m depth.

It has been found that slickensides occur in materials which have a Plasticity Index greater than 30 and also a clay fraction (less than 2 um diameter) greater than 30 per cent. In fact, slickensides apparently occur in clays with an Activity greater than 0,7 or in any material which is classified as having 'very high' potential expansiveness, or perhaps even 'high' expansiveness when plotted on the Activity chart (Williams, 1958). Further, the clay minerals present consist largely of those with an expansive lattice. The occurrence of slickensides has been recorded to depths of more than 15 m, usually above the water table. These materials are often underlain by residual

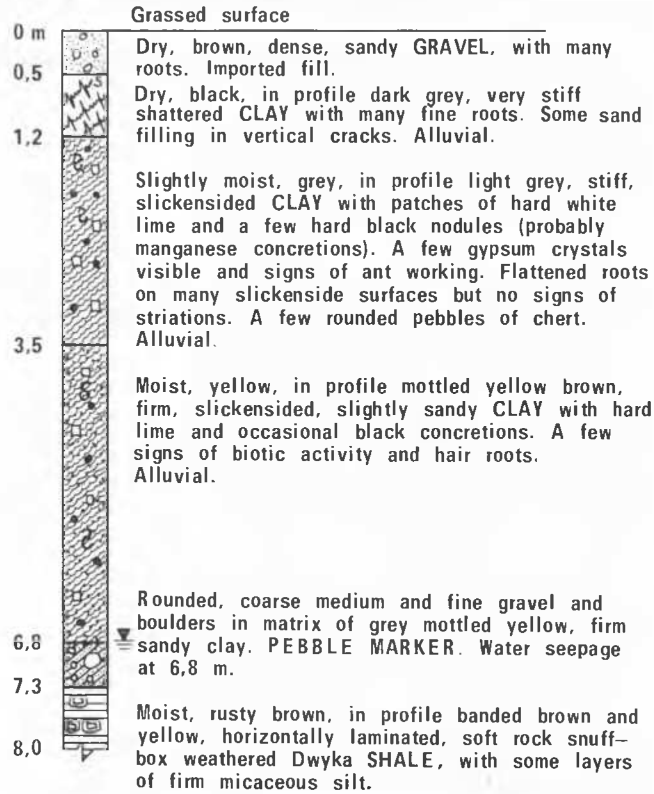


Fig. 2 Soil profile at Vereeniging

formations which have a consistency of soft rock and become harder with depth.

In reviewing the structural patterns developed in a number of different soil profiles a strong tendency has also been revealed for the incidence of slickensides to be influenced by activity of the soil type and by climatic environment. Where there is an active soil profile and a climate of a particular and appropriate seasonal moisture variation the slickensides are abundant, as confirmed by most of the profiles studied in the relatively high altitude areas in South Africa. Where one of these two factors is absent there may be very few slickensides at inclined angles of dip, although other cracks or fissures may exist. For example, in Mariental, South West Africa, the dry environment has resulted in almost no slickensided joints although there are many shrinkage cracks.

In summary, therefore, these general observations referring to stiff fissured clays lead to the conclusion that in South Africa the jointing is due either to tensile stresses in the upper horizons, or to large stress differences in the lower horizons causing shear failure. If active clays occur in a particular climatic environment where there is a possibility of reversal of movement, then slickensides are formed.

## FIELD SURVEYS OF JOINTING IN CLAYS

The technique for collecting information by means of a 'line survey', as described by Piteau (1970) and used in the study of jointed rocks, was successfully applied to fissured soils. For each joint measurements were recorded of dip angle, dip direction, joint length, spacing, waviness and surface texture. Any other special features were also recorded. Data from the study of block samples did not allow assessment of joint lengths and joint spacing. The technique finally developed for an exposure was first to select a particular horizon in the soil profile which could be viewed as a 'structural region', i.e. an exposure within which the joints appeared to be statistically similar. A line was marked along this horizon and a geological pick was used to remove, with a picking and plucking action, several centimetres or more in depth of the clay on the face so as to expose the joint within fresh material. The exposure was about half a metre in width. A tape was then stretched along this newly exposed strip and measurements were taken on each joint which would have intersected the tape, had part of the soil mass not been removed. A standard field sheet was developed for convenient recording of the position of intersection of each joint on the tape and all the other joint parameters and notes mentioned above.

The advance work of clearing the way and exposing all the joints was carried out by one man, while another followed a little way behind, using a geological pick to clear joints sufficiently for use of a compass/ inclinometer, and to make the observations. In this way it was possible to carry out a successful 'line survey' in much the same way as applied to rock faces, although care was required not to allow too long a period of time between the first man's exposures and the second man's measurements, as the surfaces were soon altered by desiccation in the hot and arid atmosphere. Experience indicated that a certain amount of bias was inevitable if one man both exposed and measured the joints, since there appeared to be a degree of selectivity for the larger fissures, or attitudes of joints, particularly those allowing easier measurements. The two-man team was found to be a better proposition. Wherever possible work was planned so that some line surveys were taken at right angles to others along the faces exposed in the excavation. The statistical analysis of the large amount of data was undertaken using computer programs developed for the purpose.

In the analysis of all joint data which has been collected so far it has been found that the distribution of values of the dip angle followed a normal, or Gaussian distribution. In Figure 3 a histogram is given for all the data collected on mottled yellow brown clay at Vereeniging and superimposed on this is the theoretical Gaussian frequency distribution curve. The equation for this normal

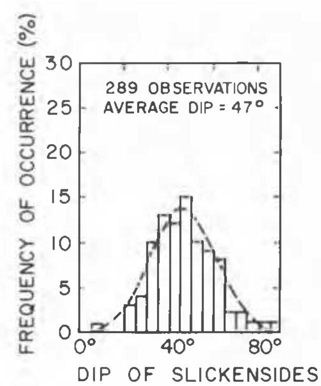


Fig. 3 Normal frequency distribution of dip angles

curve gives values evenly distributed about the overall mean value of the observations and the figure indicates a fairly good fit on either side of 47°. The coefficient of variation, however, is fairly large at about 0,3 and this explains why it is difficult to estimate by eye any typical value of joint inclination from a limited viewing of any exposure, particularly in a small test pit revealing only a small number of joints.

An analysis of many observations of dip direction has led to the conclusion that the orientation of slickensided joints is quite random and an example of a histogram is given in Figure 4. The slight bias is introduced by there being more observations along one particular line.

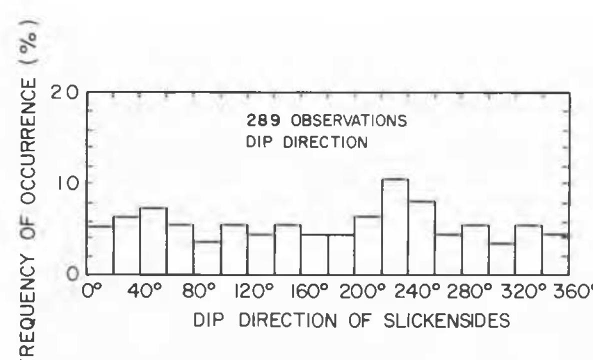


Fig. 4 Random frequency distribution of dip directions

When considering the histograms for the data on joint lengths there appears to be a clear modal value on the low side of the mean, in other words there is a distinct positive skewness. While there is thus a characteristic length of joint and very long lengths are uncommon, the possibility of encountering a single major joint, which could govern stability, should not be forgotten. It was found that a fairly good fit to the experimental values was obtained with a logarithmic normal distribution, in other words, the logarithms of the lengths were normally distributed. In Figure 5 the frequency distribution curve computed on this basis has been superimposed on the histogram for the Vereeniging mottled yellow brown clay horizon. It will be seen that the log normal distribution gives a fairly good fit in this typical case. A similar distribution has been found to fit fracture data by Bridges (1975) who used the word 'continuity' to describe extent or size of fracture.

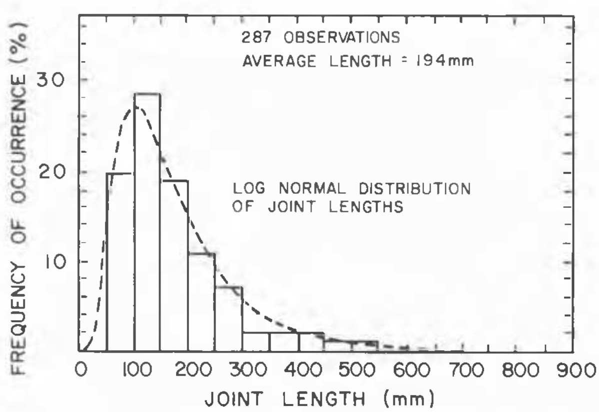


Fig. 5 Log normal frequency distribution of joint lengths

While a fair amount of information appears to have been published on joint lengths and dip, little is available on the spacing of joints, except mean values that are quoted from consideration of the total length of survey and total number of joints intersected. Following the work of Hudson and Priest (1975) a negative exponential distribution was considered for the Vereeniging data. One of the histograms given by these data for spacings is represented in Figure 6, on which the theoretical distribution based on the mean joint spacing is also superimposed, and it will be noted that the fit is fairly good.

## ESTIMATES OF STRENGTH

A suitable site for study of the in situ shear behaviour of fissured clay was found at Vereeniging in the alluvial clay at the time of construction of the new Civic Centre, a building complex with several large structures requiring the excavation of deep basements.

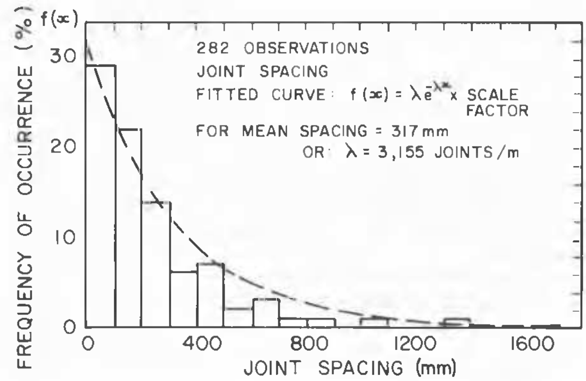


Fig. 6 Negative exponential frequency distribution of joint spacings

The general profile at this site is described in Figure 2 and the in situ moisture profile is given in Figure 7. The Plastic Limit and Liquid Limit down the profile are also given and it will be noted that the moisture content was very close to the Plastic Limit. The bulk density of the clay was about 1915 kg/m$^3$ and the degree of saturation was in the region of 90 per cent down the major portion of the profile.

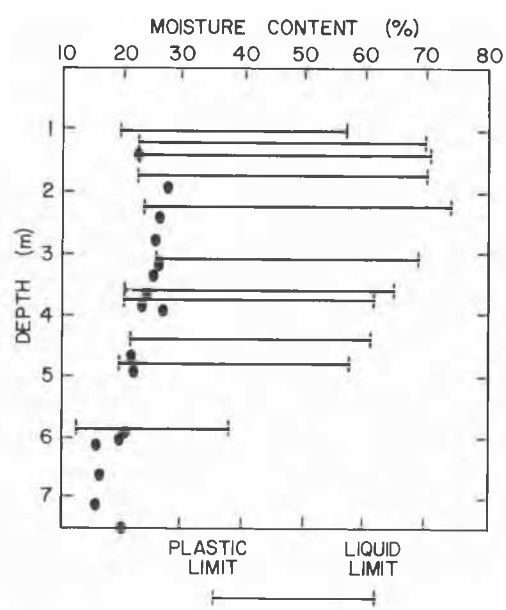


Fig. 7 Moisture content profile at Vereeniging Civic Centre

A large number of in situ shear tests were carried out using a Swedish vane of 30 mm diameter and 60 mm height. Both fairly rapid (undrained) and very slow (drained) tests were conducted at rotational speeds giving about 94 mm/min and 0,04 mm/min at the periphery. The results of some slow tests are shown in Figure 8. The peak values occurred at about 4 mm of displacement at the periphery of the sheared cylinder in the clay. The lower values shown are measurements of the remoulded, or perhaps residual, strength which was obtained after about 50 mm displacement. The fast tests gave peak values about 30 per cent higher but there was no discernible difference in residual values.

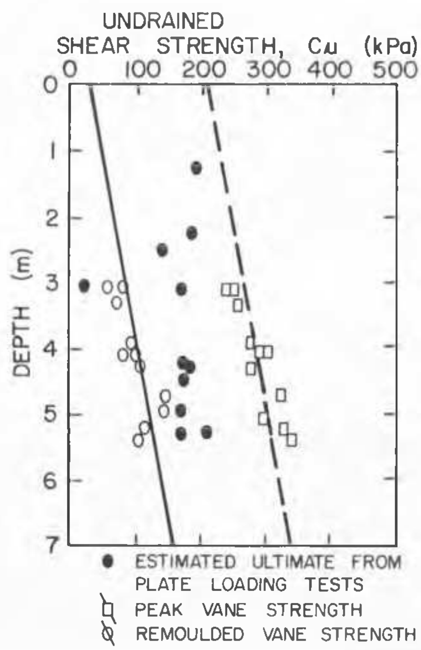


Fig. 8 Shear strength estimated from *in situ* tests

Loading tests on 150 mm diam. plates were also conducted at various depths in unlined boreholes. The ultimate bearing capacity obtained allowed an approximation to be made of the undrained strength.

These values of strength are also shown on Figure 8 and lie between the peak vane values, which presumably give the strength of intact material and the remoulded vane values, which probably reflect the strength along an existing shear plane. In other words the bearing capacity may be reduced because of the existing fissures in the clay.

During these tests cycles of unloading and reloading were introduced to allow determination of the modulus of deformation. A correction factor was applied for the depth effect. Although there was considerable scatter in results, an overall mean E-value was about 150 MPa.

An attempt was also made to measure the tensile strength of the fissured clay mass, using plates of different diameters to give some idea of the scale effect. The plates were cemented to carefully trimmed clay surfaces exposed in the bank, using a quickset epoxy paste, and a direct pull was applied. Failure always occurred suddenly and fracture surfaces through some intact material could be discerned as well as separation along inclined fissures - see Figure 9. The highest strength recorded on 45 mm diameter plates was 18 kPa while a typical value on 200 mm diameter plates was 8 kPa.

Fig. 9 Failure surface after tension test *in situ*

In the laboratory direct shear tests were carried out, under drained conditions, on saturated undisturbed samples of 76 mm diameter. These tests yielded peak parameters of $c' = 38$ kPa and $\phi' = 23^{o}$ while residual values after repeated reversals of shearing were $c'_r = 5$ kPa and $\phi'_r = 12^{o}$ (See Figure 10). Several attempts were made to measure the shear strength along an existing slickenside, by careful trimming of a sample to include such an existing plane. Shear failure occurred at small strains given by a few millimetres of displacement only and these results are also included in Figure 10. Perhaps the most important finding from this series of tests was that the shear strength along the plane of an existing slickenside was close to the residual value.

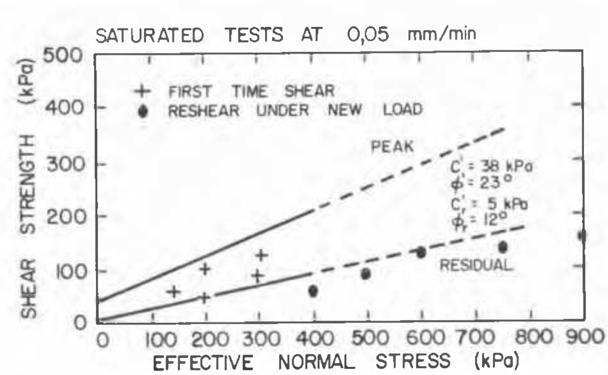


Fig. 10 Shear box test results on existing slickensides

SPECIAL SLOPE FAILURE TESTS IN FISSURED CLAYS

To study the possible mechanism of failure in a fissured clay mass and to assess the viability of various methods of stability analysis, a full-scale experiment was conducted in Vereeniging. The behaviour of a vertical bank was observed while excavation proceeded to a depth at which failure was induced. At each of two test sites pegs and targets were installed in the pattern shown on Figure 11 to allow measurement of vertical and horizontal displacements. Sleeved rods connected to depth points allowed for measurement of vertical movements within the soil mass. The horizontal movements within the mass were measured by means of an Invar steel tape and an optical plummet for sighting targets which were founded at depths of 2,0 and 3,5 m in boreholes lined with thin plastic casing. The bank was made using a front-end loader to excavate a vertical face about 8 m wide. The operation took several days, allowing time for precise surveys to be made at convenient stages of depth.

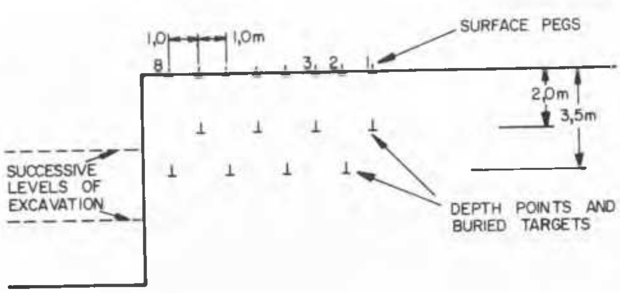


Fig. 11 Layout of surface pegs, depth points and buried targets

The first visual signs of any distress were the appearance of tension cracks parallel to the face when excavation had reached a depth of about 4 m. Special attention was paid to monitoring movements at the top edge, both to gauge general performance and particularly to afford a warning about safety. Figure 12 indicates the progress of movement of the peg at the top edge of the bank, and when there was a sudden increase in the rate of horizontal movement the operator of the excavating machine was withdrawn from the scene of the excavation. Fifteen minutes later the major failure occurred.

The vectors of movement are shown in Figure 13 and it is apparent that the actual mechanism of failure was not in the form of a classical deepseated circular slip failure. There were a number of stages in the development of distress:

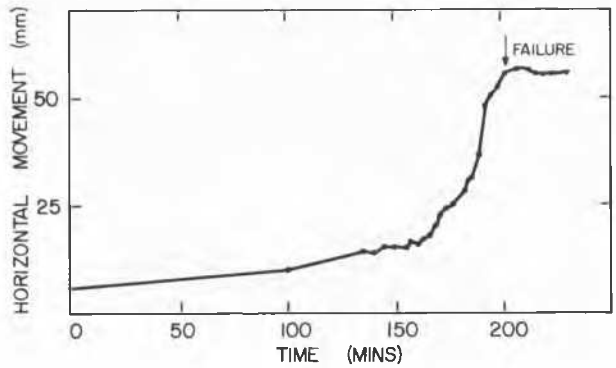


Fig. 12 Rate of movement towards excavation for peg near top edge of bank

(i) The first signs were tension cracks parallel to and about 1 m back from the top edge.
(ii) As the depth appeared to reach a critical limit of between 6 m and 7 m the face began to spall and loose pieces fell out of the vertical cut from a depth of about 1 m to 3 m below the top edge.
(iii) With further removal of material from the base of excavation there was a sudden widening of the tension cracks and an increase in spalling until a sudden major fall occurred.
(iv) The slope then remained 'alive' for some time with large pieces of debris disintegrating and spilling down the slope. This process of continual crumbling eventually resulted in the spoil appearing to consist of large granular clods of about 50 mm size.
(v) The final shape of the bank which then remained stable for many days indicated a vertical section, which terminated in an inclined surface, on which sliding had probably occurred, and then the lower portion of the excavation was covered by a heap of loose debris. (See Figure 14).

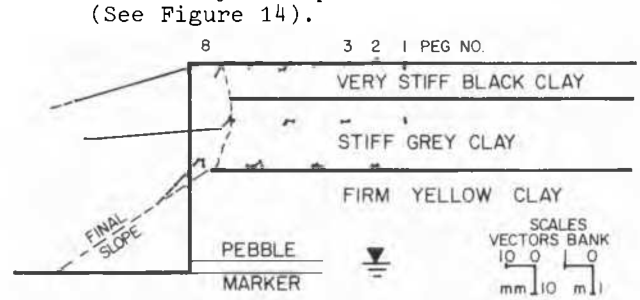


Fig. 13 Vectors of movement on north bank and shape of failure

It is clear that the pre-existing pattern of fissures in the mass has had a major influence in initiation of the particular form of distress. The propagation of this pattern of distress by the imposed conditions (or changes in stress) has then led to instability of the mass and final failure through a mechanism which differs from that which would probably have developed in an intact material. The mechanism which developed is like that of a sliding block type and this can be analysed by considering conditions of limiting equilibrium. In principle, therefore, the final analysis of slope stability is much the same in fissured clays or intact clays, but the crucial difference lies in the mechanism of failure initiated, which could give very different end results. In this particular case preliminary analysis indicates that the overall stability of the mass is most probably governed by the residual strength along the joints.

Fig. 14 East bank after failure showing slope debris

FINITE ELEMENT ANALYSIS OF BEHAVIOUR

There were several features of the final behaviour of the trial banks which were not in line with the usual form of slope failure. In particular, the following aspect deserved clarification: the fact that final failure occurred in both cases when the height of the bank was approaching 7 m, but the actual slip surface uncovered was only at a depth of about 3,5 m.

Because details of the actual failure mechanism could not be reconstructed with any confidence, after the very rapid ultimate movements, it was thought useful to attempt a modelling method for explaining the mechanisms involved.

After surveying a few finite element method computer programs available locally, that developed by C. St John (1972) of Imperial College was selected for use. This program assumes the linear elastic behaviour of the 'solid' elements of material, but allows the inclusion of 'joint' elements for which non-linear, strain-softening behaviour can be prescribed. The program was actually designed to simulate the complex behaviour of jointed rock systems and allows for excavation, or loading in a number of steps.

Previous stress analyses, in which excavation has been simulated by the finite element method, have been reported by Duncan and Dunlop (1969). These authors showed that high initial horizontal earth pressures in over-consolidated clays could result in high shear stresses which would aggravate progressive failure in stiff, fissured soils. A recent analysis of the movements occurring in an excavation about 20 m deep in London Clay has been made by Cole and Burland (1972) using a linear elastic analysis, assuming a linear decrease with depth of $K_o$, the in situ coefficient of earth pressure at rest. The displacements actually observed allowed back-figuring of the 'equivalent E' for London Clay which was found to increase significantly with depth. This result compared well with independent finishings.

The Vereeniging soil profile was simulated by a finite element mesh using vertical 'joint elements' with strain-softening behaviour in the black clay, but the dip angle for the other materials was taken from results of all the line surveys and used as the orientation of the 'ubiquitous joints' which can be incorporated in the 'solid elements'. E-values of 100 to 200 mPa have been used for the clays and Poisson's ratio was taken as 0,35. A number of analyses were made with several changes to details of the input data and a number of interesting features were revealed, such as :

(i) The sequence of excavation shows that it is at a particular depth, or perhaps the third step in excavation to 3,6 m, that instability may start to develop. This confirmed observations in the field where it became obvious that instability developed suddenly, beyond a certain depth of excavation.

(ii) It appeared that approximation of the condition in practice by a series of successive steps in excavation is more valid than the one step excavation process. This is an important aspect when simulating the behaviour of jointed materials by the finite element method.

(iii) A comparison of several outputs revealed the effect of the in situ earth pressure coefficient which was varied from 0,5 to 2,0. The best fit with actual displacements measured in the field trials was with a $K_o$ value of 1,0. See Figure 15 for typical output.

(iv) The characteristic fall-out of pieces in the second layer of grey clay was compatible with the zone of tensile stresses developed below the much stiffer layer of black clay.

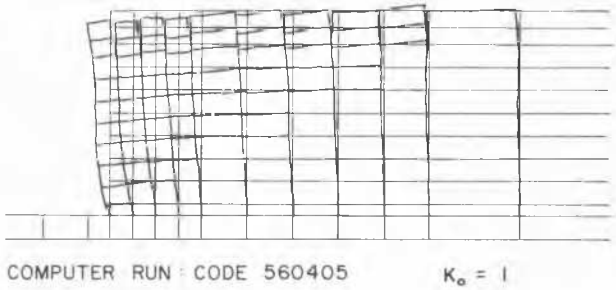


Fig. 15 Fourth and final step in excavation of vertical bank

GENERAL FINDINGS

A clear pattern of jointing can be associated with the typical South African profile in stiff clay, consisting of vertical or steeply dipping cracks near the surface due to shrinkage, but with increasing depth, through any active layers above the water table, the fissures are inclined at smaller dip angles consistent with passive shear failure on swelling. The dip direction of the joints is generally random. The lengths of slickensides which can be clearly observed have mean values of between 120 and 200 mm, depending on soil type maximum values are often up to 750 mm), while the spacing between these joints along any line is commonly in the range of 150 to 500 mm (although large gaps of 6 m or more may occur between concentrations of joints).

The shear strength along a slickenside surface appears to be near the residual value which may be determined in a shear box after repeated reversals of strain.

The mechanism of failure in stiff fissured clay differs from that in soft materials because of the pattern of jointing. In a vertical bank the initiation of a distress pattern by existing fissures produces a sliding wedge or block-type of movement, rather than the classical circular slip in intact clay. Indications are that the overall stability of the mass, or the final mechanism whereby failure is propagated, is governed by the residual strength along pre-existing joints. In these respects the stability of the mass should be analysed with the aid of theories in fracture mechanics, as already applied to major rock slopes (Jennings, 1970).

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