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Role of lateral stress in slope stability of stiff overconsolidated clays and clayshales

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ROLE OF LATERAL STRESS IN SLOPE STABILITY OF STIFF OVERCONSOLIDATED CLAYS AND CLAYSHALES

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Role of lateral stress in slope stability of

stiff overconsolidated clays and clayshales

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GENERAL INTRODUCTION

Definitions and Introduction

The failure of a soil mass in a slope is called a slide. It involves a downward and outward movement of the entire soil mass. A slide may occur in almost every conceivable manner, slowly or suddenly, with or without any apparent provocation. Usually, slides are due to excavation or undercutting the toe of the slope (Terzaghi and Peck, 1967). Slope failure is a widespread natural catastrophe, often occurring in conjunction with earthquakes or floods. Although an individual slope failure generally is not so spectacular or so costly as an earthquake, major flood, or tornadoes, the direct and indirect total financial loss due to slope failures has been estimated exceeds one billion dollars per year in the United States (Krohn and Slosson, 1976, Schuster, 1978).

A survey conducted by the Federal Highway Administration indicates that approximately 50 million dollars is spent annually to repair major landslides on the federally financed portion of the national highway system (Schuster, 1978). Various studies have shown that most damaging landslides are human-related; it is estimated that reduction of 95% to 99% in landslide losses can be obtained by means of preventive measures that incorporate thoroughly preconstruction investigation, analysis, design, and one followed by careful construction procedures (Schuster, 1978).

Stability analysis is used to analyze the condition of a slope to see whether it is stable or not, based on principles of soil mechanics. The goal is to achieve a reliable assessment of the stability of slopes, as well as the need for controlling and corrective measures (Huang, 1983). Geological studies emphasize origin, course, and resulting landforms with movement phenomena considered as a natural process (Huang, 1983). A combination of engineering and geological approaches provides the best overall picture.

Among different kinds of soil and rock materials, the stability of overconsolidated clays and clayshales is a special problem. The reason why it is of interest to civil engineers, engineering geologists, and applied geomorphologists is because of the abnormal behavior of the soils compared with other soil and rock materials. In addition to a tendency to slide under the influence of gravitational and other forces such as an increase of pore water pressure within a slope, there may be a progressive decrease in shear strength, and dissipation of recoverable strain energy due to weathering (Chowdhury, 1978).

This thesis contains two major objectives, firstly, to collect and analyze the available information concerning the slope stability of overconsolidated clays and clayshales, where the questions to be discussed are: (1) the reasons for slope failure, (2) what kind of slope movements are involved in the slope failure, and (3) the mechanisms for slope failure. The second objective is to study and evaluate the effects of lateral stress on slope stability and define approaches toward a better prediction of slope failure.

Basic Concepts

Time-rate

The change of stability conditions upon excavation of a slope and a comparison between the behavior of a weak, normally consolidated clay, and a stiff, overconsolidated clay are illustrated in Figure 1 (Bishop and Bjerrum, 1960). During rapid excavation of an initially saturated slope pore pressures decrease in response to rebound and the changes in total stress. Total stress analyses can be performed to estimate stability at the end of construction. After excavation, the soil mass swells as the pore pressures increase to those governed by seepage conditions. If the ultimate pore pressures increase after excavation, then the factor of safety declines to its lowest value when steady seepage conditions prevail (Morgenstern, 1977).

The concepts shown in Figure 1 have been supported by several studies (Kenney and Uddin, 1974) including data from a slope failure of the Kimola Canal analyzed as changes in the factor of safety as conditions changes from undrained to steady seepage. Also, the data collected by Kankare (1969) were so complete that the calculation of the changes in factor of safety with time can be made. The analyses correctly predict the time of the failure and confirm that there was a steady tendency for the pore water pressure conditions to become more critical (Morgenstern, 1977). It is of interest to note that on the basis of vane tests, a total stress analyses gave a factor of safety of 1.5 to 1.7 for the end-of-construction conditions but that this was not adequate to avoid failure (Morgenstern, 1977).

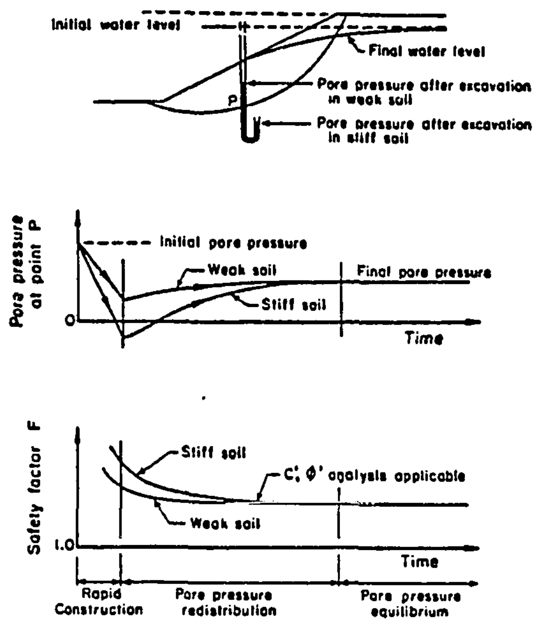

Figure 1. Changes in pore water pressure and factor of safety during and after the excavation of a cut in clay (after Bishop and Bjerrum, 1960)

For the comparison of normally-consolidated and over-consolidated behavior, at a given consolidation stress the undrained strength of an over-consolidated clay is greater than for a comparable normally consolidated clay. However, the A value for over-consolidated clay is less and the reduction in pore water pressure upon excavation therefore is greater, and the decline in factor of safety with pore water pressure redistribution is accordingly greater (Morgenstern, 1977). However, the decline in factor of safety may be related to progressive failure for normally consolidated clay or slopes with no excessive pore water pressure.

Classification

By a practical geotechnical view point, clay can be classified to three groups: soft intact clays, stiff intact clays, and stiff fissured clays (Terzaghi, 1936). The majority of clays fall readily into one or other of these groups, though there are some transitional cases (Skempton and Hutchinson, 1969). Terzaghi (1936) distinguished between soft and stiff clays primarily by liquidity index. The criteria for distinguishing between soft and stiff clays are summarized in Table 1. This classification recognizes the equal importance of strength and structure. Intact clays, in their own words, are free from joints and fissures. In contrast, a clay which is fissured contains a network of structural discontinuities comprising one or more of the following types: fissures, joints, slickensides, and laminations (Skempton and Hutchinson 1969).

The liquid limit of clays provides the best single index property as it reflects both the amount and the nature of the clay minerals present (Skempton and Hutchinson, 1969). Clays can also be classified by their origin or mode of formation which are: clays produced by rock-weathering *in situ*, sedimentary clays, glacial clays, periglacial clays, and clays transported by landsliding (Skempton and Hutchinson, 1969).

Clayshales composed of fine-grained, inorganic sedimentary materials are the predominant sedimentary rock in the earth's crust and thus are of great engineering significance (Morgenstern and Eigenbrod, 1974). However, diverse opinions exist in regard to their classification and identification. The term shale has been used by some to designate all argillaceous sediments including claystone, siltstone, mudstone, and marl (Ingram, 1953, Underwood, 1967). Whereas Twenhofel (1939) designated the larger group as mudstone or mud rock group and defined the shale as a member of this group. The boundaries between unindurated, indurated, and incipient metamorphic materials are of little primary interest to geologists, and therefore they have not been clearly defined (Morgenstern and Eigenbrod, 1974). For engineers, however, this distinction is important because of the sensitivity of the geotechnical properties to the nature and degree of induration as materials range from a soil to a rock (Morgenstern and Eigenbrod, 1974).

Table 1. Criteria for distinguishing between soft and stiff clays (after Terzaghi, 1936)

Clay Tpye	Liquidity Index	Consolidation State
Soft	$\geqslant 0.5$	normal or lightly overconsolidated
Stiff	≈ 0.0	heavily overconsolidated

Shales have been classified to two groups as shown in Figure 2 (Mead, 1936), who suggested that there are no sharp lines of demarcation between the two types of materials. The ambiguity in classifications has resulted in the use of terms such as "clay-shales" (Morgenstern and Eigenbrod, 1974).

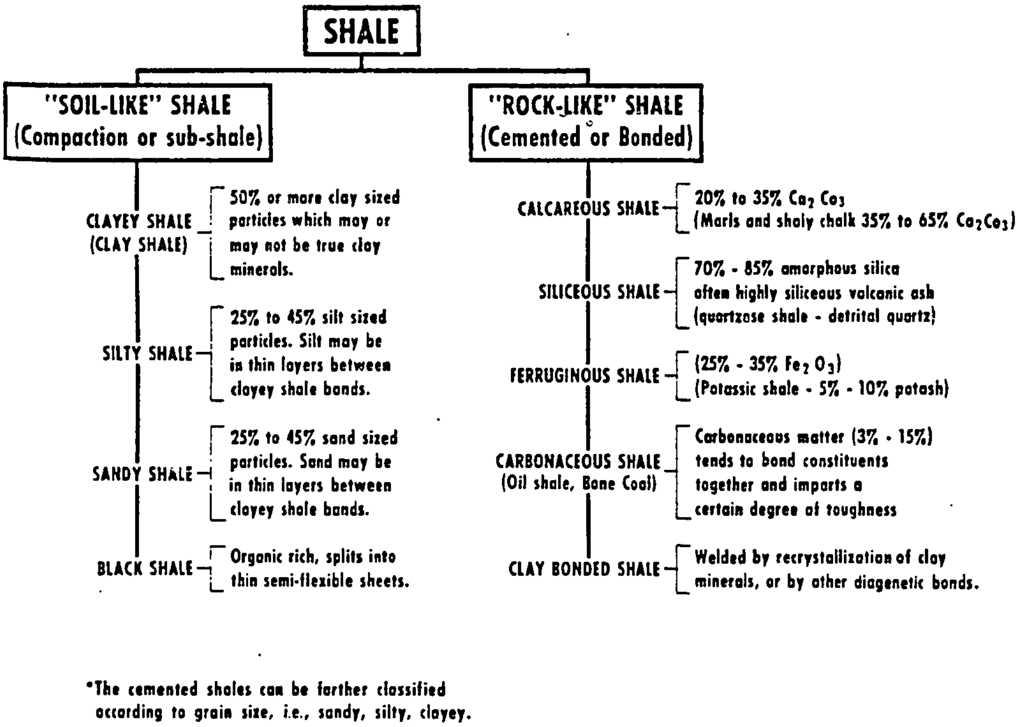

Figure 2. A geological classification of shale (after Mead, 1936)

Morgenstern and Eigenbrod (1974) tested a large number of stiff-clays, clay-shales, and mudstones, including several classical materials from different parts of the world. The disintegration of the samples upon alternate drying and wetting is generally called slaking. During a slaking test, the samples will absorb water and eventually reach water contents equal to their liquid limits (Morgenstern and Eigenbrod, 1974). Based on the liquidity index change (ΔL), the durability of clayshales can be quantitatively determined (Morgenstern and Eigenbrod, 1974). Correlations also exist between the maximum water content, activity, and montmorillonite content (Eigenbrod, 1972). Other slaking tests to determine clayshale durability are also reported (Chapman et al., 1976).

In order to evaluate the stability of overconsolidated clays and clayshales, several factors such as shear strength parameters, regional geomorphology, classification, clay content, and Atterberg limits must be acquired so that the knowledge from case studies can be used. Most of the case studies of stiff overconsolidated clay and clayshale slopes derive from slope failures in London clay, Oxford clay, and Lias clay of England (Skempton, 1964, 1970, 1977, 1985, James, 1970, 1971, Chandler, 1972, 1974, Chandler and Skempton, 1974); Upper Cretaceous shales of Canada and United States (Morgenstern and Eigenbrod, 1974, Dupree et al, 1979, Thompson and Hayley, 1975), Pennsylvanian shales of United States (Tourtelot, 1962, D'Appolonia et al., 1967, Mesri and Gibala, 1972, Linnan, 1986); and other clayshales distributed around the world (Bjerrum, 1967, Nakano, 1967, Yoshimaka, 1967, Esu and Calabresi 1969, Banks et al., 1975, Lutton et al., 1979). In spite of the variability in these shales, the engineering behavior and the shear strength parameters are similar. The residual strength of Pennsylvanian clayshales includes a cohesion near zero and friction angles ranging from 13^{0} to 17^{0}, and landslides occurring in this formation in southern Iowa, probably can be analyzed on the basis of these strength parameters.

Classification of Sliding Movements

The classification of landslides presented in Special Report 29 (Varnes, 1958) has been well received by the profession, but some deficiencies have become apparent. Varnes (1978) therefore reviewed and revised the classification system. His system considers the type of movement as a primary factor, with the material secondary. One obvious change is the use of the term slope movements rather than landslides (Varnes, 1978). Another extension is to include extremely slow distributed movements of both rock and soil; these movements are designated in many classifications as creep (Varnes, 1978). Although the revised classification is widely accepted, the type of both movement and materials may vary from place to place or from time to time, and nearly continuous gradation may exist in both. Therefore, a rigid classification is neither practical nor desirable (Sharpe, 1938, Skempton and Hutchinson, 1969, Varnes, 1978). Nevertheless, a good classification system will make it easier for both engineers and geologists to identify and know how to approach a problem.

Types of movement are divided into five main groups: falls, topples, slides, spreads, and flows (Varnes, 1978). A sixth group, complex slope movements, includes combinations of two or more of the other five types (Varnes, 1978). Materials are divided into two classes: rock and engineering soil. The latter is further divided into debris and earth (Skempton and Hutchinson, 1969, Varnes, 1978). For stability analysis of stiff overconsolidated clays, classes are as followings:

(A) Falls: Clay falls typically are failures in the steep slopes of artificial excavations or eroding river banks (Skempton and Hutchinson, 1969). These falls are usually rather insignificant, nevertheless as a consequence of the removal of lateral support, bulging occurs at the slope foot, and tension cracks open behind its crest. Progressive failure may occur as a result of stress increases in the root area. Water will infiltrate into tension cracks and the softening process will lower soil strength (Skempton and Hutchinson, 1969, Varnes, 1978).

(B) Topples: Topples have been recognized relatively recently as a distinctive type of movement (Varnes, 1978). Movement is due to force that causes an overturning moment about a pivot point below the center of gravity of the unit.

(C) Slides: The process of sliding is defined by Varnes (1978) as "shear strain and displacement along one or several surfaces which are visible or may reasonable be inferred." The presence of a displacement discontinuity is not implied in this definition and the character and thickness of the sliding zone is unspecified (Hungr, 1981). For the purpose of stability analysis it is usually assumed sliding as the movement of a relatively undistorted portion of the slope mass on slip surfaces. Slides involved in the shear failure can be subdivided into rotational and translational slides (Skempton and Hutchinson, 1969, Chowdhury, 1978, Varnes, 1978). The ability to identify these two slides is very important for engineers in the analysis of stability and the design of control methods. Rotational slides occur characteristically in slopes of fairly uniformly clay or shales (Skempton and Hutchinson, 1969). The commonest examples of rotational slides are little deformed slumps, which are slides along a surface of rupture that is curved concavely upward. Slumps and slumps combined with other types of movement make up a high proportion of landslide problems (Varnes, 1978). Such slides are relatively deep-seated with depth:initial downslope length (D/L) ratios between 0.15 and 0.33 and generally are developed on the slopes of excavation and on actively eroding cliffs.

Landslides of similar D/L ratio may behave quite differently during and after failure (Skempton and Hutchinson, 1969). Experience shows that the depth ratio tends to be greater in soils of low consistency (Zaruba and Mencl, 1969), although this may be much affected by nonhomogeneity. The thickness of the sliding zone and the rapidity of movement are highly variable (Hungr, 1981). Slides in fissured overconsolidated clays tend to be noncircular, possibly due to the increased rate of softening in the back scarp (Skempton and Hutchinson, 1969), although they occasionally may retain a rotational character (Hungr, 1981).

Translational slides generally result from pre-determined surface of a heterogeneity located at shallow depth beneath the slope (Skempton and Hutchinson, 1969, Varnes, 1978). The failure surface tends to be relatively planar and run roughly parallel to the slope of of the ground (Skempton and Hutchinson, 1969). The movement of translational slides is commonly controlled by surfaces of weakness, such as faults, joints, bedding planes, and variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus (Varnes, 1978).

Compound slides are formed of a combination of curved and planar elements, and the slide movements have a part-rotational, part-translational character (Skempton and Hutchinson, 1969). The slide masses are correspondingly broken due to several distortion and shearing action accompanying the sliding movements.

(D) Spreads: The dominant mode of movement in spreads is a distributed lateral extension accommodated by shear or tensile fractures (Varnes, 1978). Two types of spreads may be distinguished, the first one without a well-defined controlling basal shear surface or zone of plastic flow (predominantly in bedrock), the second type as on extension or gradation of rock or soil resulting from liquefaction, plastic flow, or subjacent material (Varnes, 1978).

(E) Flows: Slope movement of flow can be classified as bedrock and soil flow (Varnes, 1978). Flow movements in bedrock include deformations that can occur among many large or small fractures or even microfractures, without concentration of displacement along a through-going fracture (Varnes, 1978). The movements can be extremely slow and may more or less steady in time, and may result in folding, bending, bulging, or other manifestations (Varnes, 1978). Much of these have been classified as creep. However, creep is particularly troublesome because it has been used long and widely with different meanings, in both the material sciences such as metallurgy and in the earth sciences such as geomorphology (Varnes, 1978). Flow in soil include movement in debris and earth, which can be more accurately recognized as flows because the relative distributed (Varnes, 1978). Debris flows commonly result from unusually heavy precipitation or from thaw of snow or frozen soil, which are favored by the presence of soil on steep mountain slopes (Skempton and Hutchinson, 1969, Varnes, 1978).

(F) Complexes: Complex movement is a combination of one or more of the five principal types of movement decribed above, either within various parts of the moving mass or at different stages in development of the movements. Included are rock-fall-debris flow, slump and topple, rock slide, rock fall, cambering and valley bulging, and slump-earth flow (Varnes, 1978).

Laboratory and *In Situ* Shear Characteristics of Stiff Overconsolidated Clays and Clayshales

Due to the characteristics of stiff overconsolidated clays and clayshales, some factors that concern stability analysis are pore pressure, effective stress, peak and residual strength, and effects of fissures and anisotropy.

(A) Pore pressure: The design of slopes in terms of long time stability requires a knowledge of the pore pressures within them. Loading or unloading of soils results in pore water pressure changes that are of a transient nature. The rate of change depends on the total stress if no drainage occurs. If drainage occurs, pore water pressure is function of coefficient of consolidation, coefficient of bound, time period, and boundary conditions (Walbancke, 1975).

Two direction of pore water are possible relative to atmospheric pressure, positive and negative. The construction of an earth dam or embankment on a soft clay results in positive excess pore pressures in the foundation as the soil compresses. At the end of construction, pore water pressures are high and consequently the shear strength and factor of safety are low. The pore water pressure will dissipate as time passes, and the strength and factor of safety will increase (Bishop and Bjerrum, 1960). On the other hand, pore water pressure will be negative for overconsolidated clays and clayshales due to stress relief in the excavation or erosion of natural slope. Negative pore water pressure contribute to high shear strength and factor of safety at the end of construction. Again a pore pressure equilibrium is gradually attained as time passes, and the strength and factor of safety therefore decrease (Bishop and Bjerrum, 1960).

The quantification of excess pore water pressure derives from a concept of pore water pressure parameter A and B first proposed by Skempton (1954). He proposed $\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$

where B = empirical coefficient related to the soil compressibility and degree of saturation: B = 1 when soil is saturated, and B = 0 when soil is dry;

$\Delta\sigma_3$ = change in confining or minor principal stress

$\Delta\sigma_1$ = change in major principal stress

Parameter A depends on the type of soil and varies with stress level and stress history. It is significantly influenced by: (1) the level to which the soil has previously been strained, (2) the initial stress system in the soil, (3) the stress history of the soil, and (4) the type of total stress path to which the soil is subjected to the type of stress change, e.g., load or unload (Lambe and Whitman, 1969).

For overconsolidated clays and clayshales, A varies from 0 to -1/2. Skempton's equation is very sensitive to the initial and to the applied stress, and because initial stresses may not be known it can be used only with difficulty in prediction of the behavior of overconsolidated clays and clayshales system. The advantages are simplistic in concept and computation as well as its wide use (Vaid and Campanella, 1974).

(B) Effective stress: Terzaghi (1943) stated that the effective stress σ' in a given direction within an element of saturated soil is

$\sigma' = \sigma - u_w$

where u = the total stress acting in that direction

μ_w = the pore water pressure in the element

The equation is exact within the limits of the most refined experimental methods (Skempton and Hutchinson, 1969). If the soil is partially saturated there will be a pressure u_a in the air voids somewhat greater than the pressure u_w in the water phase (Skempton and Hutchinson, 1969). The effective stress is then given by the expression (Bishop, 1960)

$\sigma' = \sigma - [u_w + (1 - \chi)(u_a - u_w)]$

where χ = a coefficient to be determined experimentally

when the soil is fully saturated $\chi = 1$ and $\sigma' = \sigma - u_w$

when the voids contain only air, $\chi = 0$ and $\sigma' = \sigma - u_a$

For soils with a high degree of saturation, e.g., more than 90%, the term $(1 - \chi)(u_a - u_w)$ can be neglected without significant error (Lambe and Whitman, 1969).

As stated in (A), for prediction of long term stability effective stress analysis will be more appropriate than total stress.

(C) Peak and residual strength: Most heavily overconsolidated clays show stress-strain relations indicative of strain softening. As a sample is strained in a drained direct shear test, the shear resistance increases, ultimately reaching a maximum value identified as the peak strength. The failure envelope defined by peak strength can be represented by the Mohr-Coulomb theory that is expressed as following:

$s = c_p + (\sigma - u) * \tan \phi$

where s = drained shear strength

c_p = cohesion

σ = total normal stress

u = pore water pressure

ϕ = angle of internal friction

Softening is the mechanism which was first suggested to explain a time-dependent strength decrease (Terzaghi, 1936). Due to stress release by excavation or erosion, the fissures open, causing softening of the clay. The softened material eventually deforms, a stress redistribution occurs and finally the clay becomes a normally consolidated or "fully softened clay" (Skempton, 1948). The destruction of the original soil structure results in a complete loss of the cohesion. This softening process is not dependent on large deformations within the slope before failure but is dependent on the presence of fissures and joints within the soil (Eigenbrod, 1972). Skempton (1970) further interpreted this stage as the reduction from peak to fully softened strength which is attributed to an increase in water content along the shear zone. With numerous discontinuous shears, the strength at this point (fully softened) is appropriate to analysis of a first-time slide. For an old or reactivated slide, large deformation and a continuous shear surface are needed to reach the residual strength, which can occur as the result of geological processes including tectonic folding, valley rebound, glacial shove, periglacial phenomena, and non-uniform swelling (Morgenstern, 1977). The interpretation of Skempton's recommendations concerning selection of strength parameters for analyzing long term slope stability was summarized in Table 2 by Lambe (1985). In this table, the first-time slide is one in which the slip surface located entirely in previously unsheared materials, and in reactivated slide it is in previously sheared material.

Typical stress strain curves are sketched in Figure 3, the shape of curves varying in different kinds of soil. For soft silty clays little difference is shown between peak and residual strengths (Skempton, 1970). With higher clay contents the difference tends to increase, even in the normally-consolidated condition, attributed to a decrease in strength from reorientation of clay particles along the slip surfaces (Skempton and Hutchinson, 1969). At fully softened strength, or close to it, there is as yet no principal shear surface but instead a complex of minor shears such as the Riedel, thrust, and displacement shears (Skempton and Petley, 1967) which have not been linked into a continuous surface. The reduction from fully softened to residual strength is attributed to reorientation of platy clay minerals parallel to the direction of shearing (Skempton, 1970).

Table 2. Recommended strengths for analyzing cuts and natural slopes (after Lambe, 1985)

Soil	Deformation	Strength c'	Strength ϕ'
Intact clay	No previous large deformations (first-time slide)	c'_p	ϕ'_p
Overconsolidated fissured clay not highly expansive or not highly organic content	No previous large deformations (first-time slide)	c'_s	ϕ'_s
Overconsolidated fissured clay highly expansive or highly organic content	No previous large deformations (first-time slide)	0	ϕ'_r
Overconsolidated clay	Previous large deformations	0	ϕ'_r

c'_p: cohesion at peak strength state

ϕ'_p: friction angle at peak strength state

c'_s: cohesion at fully softened state

ϕ'_s: friction angle at fully softened state

ϕ'_r: friction angle at residual state

Skempton (1985) summarized the effects of colloidal content on residual strength as follows: (1) clay content less than 25%, residual friction angles are greater than 20^0, a strength reduction from the fully softened strength to the residual strength does not occur; (2) clay contents between 25% and 50%, the residual strength depends on clay content as well as mineralogy; and (3) clay contents greater than 50%, residual strength is controlled almost entirely by mineralogy, a further increase in the percent clay having little effect.

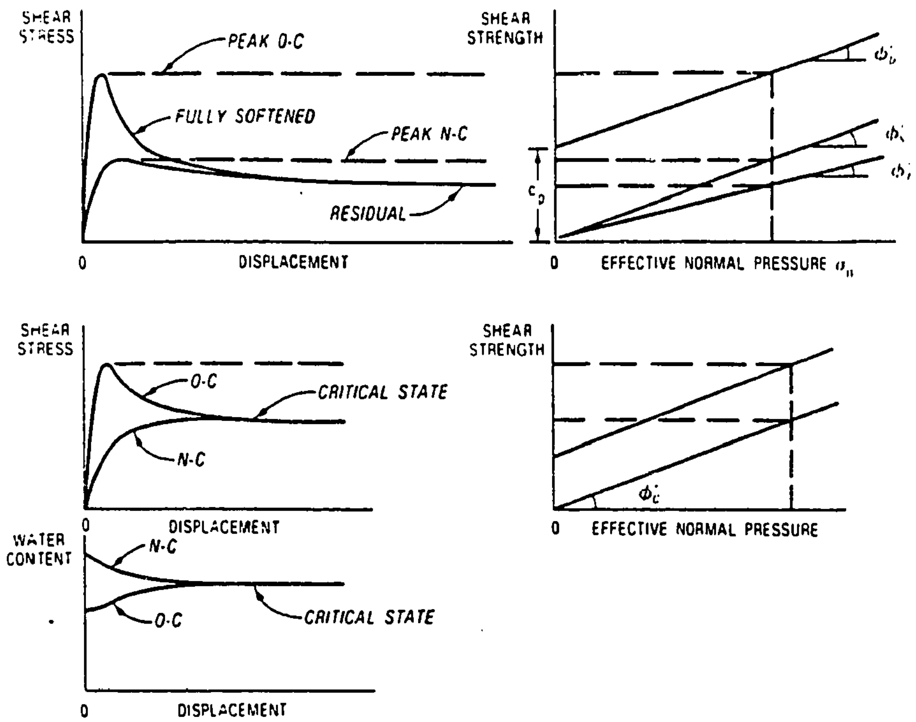

Figure 3. Shear characteristics of stiff clays (after Skempton, 1970)

The residual friction angle can also be influenced by the following factors:

(1) Mineralogy: The angles of residual shearing resistance of platy minerals such as montmorillonite, illite, or kaolinite orient in the direction of shear without interlocking and hence show low residual

strength. However, needle-shaped or sharp mineral particles such as crushed quartz or feldspar tend to interlock and have a higher residual friction angle. The following are in order of decreasing residual strength: massive minerals, micaceous minerals, kaolin and montmorillonite (Kenney, 1967, Hungr, 1981, Skempton, 1985).

(2) Grain size: A decrease in grain size causes an increase of residual friction angle (Olson, 1962, Kenney, 1967).

(3) Type of cation: Residual friction angle is increased by cations of higher valency, $Ca^{+2} > K^{+} = Na^{+}$ (Kenney, 1967).

(4) Ion concentration in the pore fluid: Residual friction angle is increased for increases of salt concentration (Kenney, 1967).

(5) Rate of shearing: The rate of displacement or strain influences residual strength of clays. However, the relationship has not yet been satisfactorily explored (Hungr, 1981, Skempton, 1985). It is found that drained friction angle increases very gradually with increasing rate, but Kenney (1967) found less than 1% increase for a range of minerals subjected to shearing rates increased by five orders of magnitude. Petley (1966) obtained a 5% increase in drained residual shear strength for a rate increase of three orders of magnitude. The results of the test of two clays over a range of speeds from about 100 times slower to 100 times faster than the usual (slow) laboratory test rate gave on an average, a change in strength less than 2.5% per log cycle order of magnitude (Skempton, 1985). This suggests that variations in strength within the usual range of slow laboratory tests (0.002-0.01 mm/min) are negligible. In the field, from observation on

landslides, if the strength at a typical laboratory rate of 0.005 mm/min is taken as standard, the variations over this entire range lie between -3% and +5% (Skempton, 1985).

Skempton (1985) also showed that for clays the increase in strength becomes pronounced at rates exceeding 100 mm/min. This may be associated with disturbance of the originally ordered structure, and turbulent shear produced in contrast to sliding shear where particles are orientated parallel to the plane of displacement (Skempton, 1985). As a result, negative pore pressures are generated and, as displacement continues, those are dissipated within the sample, thus leading to a decrease in strength.

(D) Effects of fissures and anisotropy: Heavily overconsolidated clays and clayshales are commonly jointed and fissured. Terzaghi (1936) first established the practical significance of this observation and the influence on engineering behavior. The presence of fissures results in important size effects when conducting undrained strength tests, and is critical for short-term stability of slopes (Morgenstern, 1977). It is found that large specimens and slope failures often display only 20%-30% of the strength found in small samples and by *in situ* vane tests (Morgenstern, 1977). A similar result also was found with other heavily overconsolidated clays and clayshales (Marsland, 1967, McGown and Radwan, 1975), i.e., strength decreasing with increasing sample size. They suggest that the diameter of test specimens should be twice the fissure spacing to overcome size effects

(Marsland, 1967).

Some anisotropy must be expected in clays as a consequence of their mode of formation, e.g., bedding and one dimensional consolidation in sedimentary deposits, and the presence of discontinuities which may effect a pronounced preferred orientation (Skempton and Hutchinson, 1969).

The undrained strength of many overconsolidated clays and clayshales is anisotropic because of pore water pressure changes associated with various stress paths and rotation of principal stresses (Morgenstern, 1977). McGown et al. (1974) draw attention to the preferred orientation, and lowest values of undrained strength occurred when the test was in the direction of the preferred orientation, in which 50%-60% of the maximum was observed.

PART I: THEORETICAL ANALYSIS OF FAILURE MECHANISMS

Review of Previous Works

It is an important and challenging responsibility for engineers and geologists to identify where landslides are likely to occur. Landslides may be caused by man-related behavior as well as nonhuman related factors, the latter often referred to as "acts of God." In most cases a number of contributing factors exist simultaneously. Attempting to decide the specific factor producing failure is not only difficult it is incorrect. Often the latest factor is only a trigger that sets in motion on earth mass that already was on the verge of failure (Sowers and Sowers, 1970).

All slides involve the failure of earth materials under shear stress. The initiation of the process can be reviewed by (1) the factors that contribute to increased shear stress and (2) the factors that contribute to low or reduced shear strength (Schuster, 1978). Factors that contribute to increased shear stress, such as removal of lateral support; surcharge; earthquakes; and vibrations from blasting, machinery, traffic, thunder; regional tilting; removal of underlying support; volcanic processes; and lateral pressure caused by water, freezing of water, swelling of clay, and mobilization of residual stress.

Factors that contribute to low or reduced shear strength can be divided into two groups: (1) Factors stemming from the initial state or inherent characteristics of the material which are part of the geological setting that may be favorable to landslides. These factors may exist for a long period of time without failure, and include the material composition, texture, and gross structure and slope geometry, (2) Changes that tend to lower the shear strength of the material, such as changes due to weathering and other physicochemical reactions, changes in intergranular forces due to water content and pressure in pores and fractures, and changes in structure due to stress relief (Varnes, 1978).

All of the factors listed above can cause sliding slope movements. However, seldom can a landslide be attributed to a single definite cause. Furthermore, landslides may take place under the influence of geologic, topographic, or climatic factors that are common to large areas (Schuster, 1978). Thus, the causes must be understood if other similar slides are to be avoided or controlled.

For theoretical analyses of failure mechanism, Chowdhury (1980) classified landslides as: (1) landslides due to exceptional causes such as earthquake and liquefaction; (2) ordinary landslides which occur during or soon after ordinary natural phenomena, e.g., rainfall, or man-made changes, e.g., excavation of natural slopes or construction of fills, or other environmental changes; and (3) landslides due to no apparent cause or where it is difficult to identify the immediate disturbing agent or trigger mechanism. In this thesis, I will emphasize factors (2) and (3).

It have always been troublesome that many failures in overconsolidated clays and clayshales occur long after the slopes have been cut. Terzaghi (1936) first interpreted this apparent decrease in stability with time as "softening". He suggested that in stiff fissured overconsolidated clays and clayshales, the lateral stress release which results from excavating a cut could cause some opening of fissures. This softening starts from the face of the open fissures under zero effective stresses and leads to a reduction in average strength. The end product of such a softening process must be a clay reduced essentially to its normally consolidated condition (Skempton, 1948).

Prior to 1950s, the studies were generally limited to a $\phi = 0$ failure analysis (Skempton, 1948). The use of a $\phi = 0$ concept for stability problems in normally consolidated and lightly overconsolidated clay is widely accepted (Chowdhury, 1978). However, if the overconsolidation ratio is higher than about 4 to 8, the soil tends to dilate during shear with a consequent decrease of pore water pressure such that the undrained strength exceeds the drained strength (Chowdhury, 1978). High negative pore water pressures then tend to draw water into the soil with consequent swelling and reduction of strength. Therefore, the undrained strength cannot be relied upon and its use in stability analysis will lead to results on the unsafe side (Terzaghi and Peck, 1967).

A rational approach to stability analysis was made possible with the development of the effective stress method by Bishop (1952, 1955), which provided a more dependable prediction of field behavior. Henkel and Skempton (1954) suggested $c' = 0$ and $\phi'_{residual}$ to apply the laboratory tests for long-term design of slopes in London clay. However, for the long-term stability of slopes, the effective stress analysis will not reflect the critical situation at the end of construction (Eigenbrod, 1972). Unloading due to excavation results in the development of negative pore water pressure, and with time these negative pore water pressures will dissipate until they are in equilibrium with the steady seepage flow pattern appropriate to the new slope profile. This process causes a decrease of the average principal effective stresses that can lead to slope failure (Eigenbrod, 1972).

From a number of cases of slides occurring in highly overconsolidated clays in California, it has been proposed that a relatively small movement may suffice to reduce the strength from that of the laboratory "ultimate" value following brittle failure, and that previous slides or even small movements have permanently damaged the shear resistance of the stiff clay (Gould, 1960). Based on the laboratory triaxial shear test, the cohesion (c) will be mobilized first and friction angle (ϕ) will be mobilized later with larger field strain (Schmertmann and Osterberg, 1960). In the 4th Rankin Lecture, Skempton (1964) proposed the residual strength is the strength on natural slip surfaces after large displacement, and suggested that some mechanism of progressive failure might have caused the low strength values.

Another possible mechanism for progressive failure is that there is a link between recoverable strain energy and the potential for progressive failure, and that weathering increases this potential by destruction of diagenetic bonds in overconsolidated clays and clayshales (Bjerrum, 1967). The condition for this mechanism depends on changes in the followings: (1) The ratio of lateral internal stress to peak shear strength, (2) the ratio of lateral internal strain from recoverable strain energy to peak strain, and (3) the ratio of peak strength to residual strength.

Bishop (1967) proposed a mechanism based on the fact that a zone of plastic equilibrium is formed in a slope due to local overstress long before general failure takes place, then the shearing resistance of progressive extension of failure along the potential slip surface drops from peak to residual state within this zone. Peck (1967) discussed a set of Conlon's tests and proposed a mechanism with respect to progressive failure on overconsolidated clays, in which nonuniform stress-strain conditions resulting in nonuniform mobilization of the shearing resistance and average strength could be mobilized between the peak strength and the residual strength. Turnbull and Hvorslev (1967) discussed the problem of progressive failure, referring to nonhomogeneous stress distribution and local overstressing as reported by Bjerrum and Bishop.

Duncan and Dunlop (1969) modelled the effects of initial lateral stresses on the stresses within a slope. Using a plane strain formulation of the finite element method, they analyzed a homogeneous, linear, elastic, isotropic material and proposed that the stress conditions in a slope after excavation are strongly influenced by the initial horizontal stress (K_0). It was concluded that for high K_0 value large shear stresses might develop at some points within a slope even though the factor of safety was greater than one, and the existence of high horizontal stresses in heavily overconsolidated clays and clayshales increases the probability of progressive failure (Duncan and Dunlop, 1969). Through oedometer tests, it has been shown that K_0 unloading causes shear failure of the specimen, and shearing strains associated with further unloading reduced the shearing resistance of the material along the shear surface (Yudbhir, 1969). A theoretical model to study the effect of horizontal stress release on progressive failure had been proposed (Christian and Whitman, 1969). They considered a single layer bound to a rigid base with elastic, plastic, and strain softening behavior for the bonding between the layer and the base. It was found that the smaller the factor of safety with respect to the residual strength, the greater the length of the failure surface. Even for slopes with a high factor of safety with respect to the residual strength, due to the release of initial stress by erosion or cutting, there is a possibility of propagation of a failure surface (Christian and Whitman, 1969).

By the studies of the long-term creep characteristics of overconsolidated London clay and normally consolidated Pisa clay under drained conditions, Bishop and Lovenbury (1969) found that long-term loading does not necessarily lead to substantial strength reductions. This suggests that there may be no path to the residual strength by passing through the peak strength (Eigenbrod, 1972). This concept is important when later dealing with the creep mechanism. James (1970) analyzed over 50 case histories, most of them are overconsolidated clays and clayshales, which include London clay, Oxford clay, Lias clay, Cretaceous clay, and several clayshales. First-time, slides failed with $c' = 0$ and $\phi' = \phi'_{peak}$, while reactivated, slides failed at reduced ϕ'. The results of the investigations of repeated slides showed that a reduction in friction resistance happens only after very large movements of the order of several feet. Also unless deformations are localized along the interface between two different layers, the strains would be too small for strength to approach the residual (James, 1970).

Skempton (1970) discussed first-time slides in overconsolidated clays and described the post peak changes in strength comprising two successive stages: (1) dilatancy and the opening of fissures leading to increases in water content and culminating in a drop in strength to the fully softened value, at which stage there is a softened shear zone with numerous discontinuous shears, (2) development of principal shears of appreciable length, some of which eventually link together and form a continuous shear such that the residual strength is reached along the entire slip surface.

A nonuniform mobilization of strength from peak to residual along the slip surface might occur for a first-time slide (Bishop, 1971). However, from the results obtained in ring shear apparatus, he concluded that residual strength in clay can only be reached after very large strain.

A new approach based on fracture mechanics concepts considers a planar slip surface to be a crack (Palmer and Rice, 1973). This concept was applied to a slip surface by starting from a step or cut in a long slope. Based on the energy balance, they proposed a theoretical model to calculate the shear band length, which appears particularly pertinent to overconsolidated clays and clayshales.

Vaughan and Walbancke (1973) observed that the pore water pressures may be significantly below equilibrium values for relatively long time periods after cutting of the stiff, fissured overconsolidated clay slopes. Chandler and Skempton (1974) suggested that many long term failures were attributed to long term pore water pressure equilibration effects. They also observed that c' = 0 for analysis of long term slides in these clays is conservative, whereas the use of peak strength measured in the laboratory is unsafe. Eigenbrod (1975) presented a finite difference method for analysis of pore water pressure equilibration based on two dimensional consolidation theory. He observed that for many slopes that the time for pore water pressure equilibration is of the same magnitude as the times to failure. Based on the field evidence in the brown London clay proposed that the main reason for the delayed failure in cutting slope is due to a very slow rate of pore water pressure equilibration which is pertinent to first-time slides.

Chowdhury (1976) investigated the influence of *in situ* stress on the stability of natural slopes by using the limit equilibrium approach. He proposed that the factor of safety depends on the *in situ* stress and varies in a significant way with the inclination of the failure surface. Based on the concepts of the residual strength (Skempton, 1964) and creep (Haefeli, 1965), Nelson and Thompson (1977) derived a theoretical equation to describe the relationship between creep, peak strength, post peak behavior, and progressive failure.

Tavenas and Leroueil (1977, 1981) proposed the concepts of limit and critical state to represent cut and natural slopes, in which the rate of pore water pressure dissipation and the creep behavior may be assumed identical. They also suggested no fundamental difference in the behavior and failure of man-made and natural slopes (Tavenas and Leroueil, 1981). Leonards (1980) postulated that the time to failure is a direct function of the slope height and the slope inclination for London clay slopes. Osaimi and Clough (1979) had attempted to determine the variations of total and effective stresses during the excavation as well as with time and showed the high concentration of shear stresses near the toe of the slope as well as the influence of the initial stress. They also evidenced the important rotations of the principal stress axes which develop during the excavation.

Ter-Stepanian (1980) proposed that the creep in slopes should not be treated as a continuous process, nor does it proceed uniformly, because of seasonal changes caused by fluctuations or periodical accelerations due to drawdown of ground water. Hungr (1981) reviewed available evidence and concluded that continuous creep is sustained at steady rates without change in applied or resisting faces exists only in the surficial layers of soil. At depth, only a decaying creep was measured. A sustained deep-seated creep in soils occurred only on failure generated shear zones or in material that was approaching failure (Hungr, 1981). Savage and Chleborad (1982) used a viscoplastic model to measure velocity profile in creeping landslides, which fits several field landslide cases. Morgenstern (1985) proposed that a more likely explanation of large landslides resides in the geometrical complexity which includes thickness, slope of slip surface, lateral restraint due to channelization, and a number of other factors that vary from place to place in all but the smallest and simplest landslides.

For failure mechanisms, three strength reducing mechanisms will be discussed; (1) delayed failure, (2) progressive failure, and (3) creep.

Delayed Failure

Introduction

Slope cutting or erosion cause unloading of the ground. If the soil permeability is low relative to the rate of excavation or erosion, the expansion of overconsolidated clays and clayshales under partially or undrained conditions leads to pore water pressure reductions (Bishop and Henkel, 1953). The pore water pressures due to unloading are negative with respect to the final equilibrium conditions as shown in Figure 1, these negative excess pore water pressures tend to equalize until steady seepage conditions are reached (Eigenbrod, 1975). The average effective stress in the slope will decrease as the negative excess pore water pressure dissipated. In the long term this reduction of the average effective stress may lead the slope to failure (Eigenbrod, 1975).

The delayed failures of slope cut in overconsolidated clays have been investigated for many years (de Lory, 1957, Henkel, 1957, Skempton, 1948, 1964, 1970, 1977, 1985, James, 1970, 1971, Vaughan and Walbancke, 1973, Eigenbrod, 1972, 1975, Tavenas and Leroueil, 1981, Rulon and Freeze, 1985). Delayed failures may include all processes that contribute to a reduction of shear strength with time (Morgenstern, 1977). The most common factors are pore water pressure equilibration and strain softening, which will lead to a reduction in shear strength from peak to the fully softened strength, $c = 0$, $\phi' = \phi_{peak}$ (Morgenstern, 1977). The time of delayed failure varies from years to decades (Skempton, 1977).

<u>Development of research program and questions studied</u>

The initial stage in Terzaghi's stress relief, cracking and ground softening process is mainly due to a destruction of the original clay structure by swelling, Skempton's softening process is caused by dilatancy during straining and particle orientation along minor shear (Eigenbrod, 1972). Skempton's softening process therefore is not dependent on the presence of fissures or joints as Terzaghi's, even though both softening processes will become similar eventually (Eigenbrod, 1972).

At the end of the softening processes, the shear strength of the soil will drop from the peak to fully softened strength as long as no continuous shear plane is formed. A residual strength is that which develops only after very large displacements along a shear plane (James, 1970, Skempton, 1970).

The rate of softening is influenced by the type of clay and the climatic conditions. Quigley et al. (1971) observed that swelling clays of accelerated soil softening and caused failure of several slopes only 4 to 8 years after construction. Climatic conditions affect the process of weathering, including physical disintegration and chemical decomposition. This also can cause complete destruction of the clay structure and loss of strength. The softening by physical disintegration such as freezing-thawing is generally restricted to relatively shallow depth below the ground surface, and it usually occurs in temperate and cold climates (Eigenbrod, 1972). However, the softening by chemical decomposition can reach much larger depths, more than 20 feet in temperate climates (Eigenbrod, 1972).

Thus, if strain softening can cause the shear strength drops from peak to a fully softened strength, strength may continue to drop from a fully softened to residual state if very large displacements develop or occur in a reactivated slide. However, for analysis of delayed failure of a first-time slide the fully softened shear strength is the more appropriate.

Pore water pressure equilibration has been investigated extensively (Bishop and Henkel, 1953, Peterson, 1954, Lutton and Banks, 1970, Vaughan and Walbancke, 1973, Eigenbrod, 1975, Walbancke, 1975, Skempton, 1977, Tavenas and Leroueil, 1981, Koppula and Morgenstern, 1984, Rulon and Freeze, 1985). Due to improvements in piezometer installations, negative excess pore water pressure can be observed in the field. It is possible to analyze the mechanisms of pore water pressure changes due to excavation of a slope with the presently available techniques and to compare the analytical results with field observations.

Kankare (1969) investigated the slope at Kimola Canal, Finland. The slope is in highly to slightly overconsolidated clays. During the excavation of the canal, the piezometers recorded a rapid drop of pore water pressures, and the lowest values were observed when the excavation was finished. Equilibration of pore water pressures was reached approximately 6 months later.

Lutton and Banks (1970) reviewed previous records in studies of the slopes along the Panama Canal, then geological, field, and laboratory investigations were undertaken and the stability of the slopes was analyzed. Stiff fissured clays of the Cucaracha formation in that region contains an abundance of fractured and highly slickensided montmorillonite-rich clay making it unstable. It was found that deep piezometers below the depth of active sliding indicated pore water pressure below the canal water level, suggesting that negative pore water pressure may still exist 60 years after canal excavation (Lutton and Banks, 1970). Many cuts in London clay failed about 20 to 60 years after they had been excavated (James, 1970, Skempton, 1970). Based on this observation, Vaughan and Walbancke (1973) suggested that failure of cuts in London clay may be delayed primarily by the rate of pore water pressure equilibration. They reported that in other overconsolidated clays, the delayed failure of cut slopes may also be controlled by pore water pressure equilibration, and this should be considered before other mechanisms such as decrease in drained strength with time are postulated. But they are not sure that rapid equilibration may occur due to fissures and discontinuities which may increase permeability and shorten the time for the equilibration of pore water pressure.

In a numerical analysis based on the Koppula's two-dimensional consolidation program (Koppula, 1970) which assumes that consolidation and swelling follow the same theory, Eigenbrod (1975) calculated the pore water pressure changes due to excavation of a slope, and the subsequent dissipation of negative excess pore pressures. The analytical results of these changes due to unloading of a slope agree well with pore water pressure measurements. He further suggested that pore water pressure after excavation can be predicted analytically in homogeneous materials, and that the time for full dissipation is of the same order of magnitude as the time between excavation and failure (Eigenbrod, 1975).

Based on the measurement of pore water pressure in London clay cuttings, Walbancke (1975) proposed that pore water pressure equilibration after excavation of a slope is on the same time scale as delayed failure and is probably a primary cause. She further suggested that the rate of pore water pressure equilibration is a function of the permeability gradient within a clay layer and of the boundary pressures.

Skempton (1977) had summarized researches on first-time slides in cuttings in the brown London clay. According to field piezometer observation, he indicated that the main reason for the delayed failure is a very slow rate of pore water pressure equilibration, despite the fissures of the clay. It was also observed that when the average pore water pressure ratio $\overline{T_u}$ reaches 0.25 to 0.35, the slide will occur. The average pore water pressure ratio $\overline{T_u}$ which is defined (Bishop, 1960) as average value of T_u along the slip surface, and $T_u = T_w h/Tz$

where T_w = unit weight of water

T = unit weight of clay

h = piezometric height

z = the sliding depth

With ongoing natural slope erosion, pore water pressures probably will never attain equilibrium. Peterson (1954) made this observation in river valleys excavated by natural erosion processes in Bearpaw shale. Koppula (1970) analyzed this process and showed that unloading by river erosion that cut into thick impervious clay shales should cause negative pore water pressure still to exist. A theoretical model of "The consolidation of soil in two-dimensions and with moving boundaries" was used in this case (Koppula, 1970). Hutchinson (1969) reported that pore water pressures are below sea level for the coastal slopes in England, in which the low water pressures in the slope have to be referred to the unloading during the initial slide. Tavenas and Leroueil (1981) proposed that no fundamental difference of negative pore water pressure equilibration in the behavior and failure of man-made and natural slopes.

Based on Terzaghi's classical consolidation theory, a mathematical model to describe the behavior of a fully saturated sediment subjected to uniformly eroding load was proposed (Koppula, 1983, Koppula and Morgenstern, 1984). This theoretical model can analyze simultaneous generation and dissipation of negative pore pressures. Even the free water is made available at the top, substantial negative pore pressures are likely to persist in the residual mass, even close to the eroded suface (Koppula, 1983, Koppula and Morgenstern, 1984). Rulon and Freeze (1985) used a finite element model to simulate two-dimensional, saturated-unsaturated, steady state flow through layered slopes, in which the pore water pressure distribution and the distribution of multiple seepage faces are strongly dependent on the position of the impeding layers and their hydraulic properties such as the magnitude of the hydraulic conductivity contrast between adjacent geologic units. In same cases, negative pore water pressure in the unsaturated wedges still exists. This phenomenon has also been verified by laboratory test (Rulon and Freeze, 1985).

The softening process of delayed failure was first proposed by Terzaghi (1936). Skempton (1948, 1970) assumed that the drained strength of clay decrease with time, in which the effect of pore pressure has not been considered. Eigenbrod (1972) postulated that this softening process and weathering is the first stage of progressive failure. However, traditional definition of progressive failure is that very large displacement are generally necessary to develop residual strength along a continuous surface (Bjerrum, 1967, Skempton, 1964, 1970, and James, 1971). For the first-time slides investigated by Skempton (1970), only minor deformation is needed to drop the strength from peak to fully softened. Therefore, we can define the softening process is independent or only a beginning step of progressive failure. The relationship between softening process of delayed failure with strain softening of progressive failure has always been somewhat obscure (Morgenstern, 1977). Terzaghi (1936) and Skempton (1970) have defined the mechanism, but experimental data are not abundant (Morgenstern, 1977). The result of softening is that the soil strength should drop from peak to fully softened.

For negative pore water pressure equilibration, there have several theoretical models to simulate the field situation due to slope excavation or natural erosion (Eigenbrod, 1972, 1975, Vaughan and Walbancke, 1973, Walbancke, 1975, Osaimi and Clough, 1979, Tavenas and Leroueil, 1981, Koppula, 1983, Chandler, 1984a, 1984b, Koppula and Morgenstern, 1984).

No experimental data exist to discuss the relationship between softing process and negative pore water pressure equilibration. Chandler (1984b) postulated that the procedure between these two should be as followings:

After excavation, negative pore water pressure will occur in the slope and the void ratio of the clay will be low. When negative pore water pressure equilibration is completed within the slope and that shear stress is low, there will be an increase in the void ratio of the clay, a process that eventually will take the soil to the "fully softened" state. Thus significant softening can't occur until negative pore pressure dissipation within the slope is completed. The process of negative pore water pressure equilibration may be long compared with a simple softening process, so from long-term stability point, the negative pore pressure equilibration will be a dominant process, especially for first-time slide. However, some exceptional cases still exist such as weathered Upper Lias clay, in which the variability resulting from weathering and periglacial brecciation is too great for a systematic pattern of negative pore water pressure equilibration to be apparent (Chandler, 1984b). Also, the softening process may be the controlling mechanism for a shallow-depth slide.

Rate of pore water pressure equilibration

A slope may fail either as a consequence of the development of positive excess pore water pressures or due to the dissipation of negative pore water pressure (Chowdhury, 1978). The former is related to embankment construction while the later is related to slope cutting, slope erosion, and other man-made behavior. Excess pore water pressures have a direct influence on the short-term and long-term stability. Especially at the end of the construction when soil is undrained, the negative pore water pressure is important for slope cutting and natural slope stability. The variation of pore water pressure responsible for changes in the stresses, which will affect their stability.

Positive pore water pressure occurs below the water table. For point A located at depth h below the water table, the positive pore water pressure is equal to $u = hT_w$

where T_w is the unit weight of the water. The height h is the piezometric head at the point of observation, and can be positive or negative. If the point of observation is located below the water table, the pore water pressure is positive. If the point of observation is above the water table, the equilibrium static pore water pressure is negative.

The value of effective stress will be larger than the total stress. This kind of negative pore water pressure will temporarily help to stabilize the soil masses, as apparent cohesion. The reason for using "apparent" is because it disappears on saturation (Aitchinson and Donald, 1956). Vegetation can maintain a permanently desiccated slope such that negative pore water pressure may be permanent (Blight, 1963).

Other mechanisms of negative pore water pressure also exist for overconsolidated clays and clayshales at fully saturated condition, such as: (1) shearing stage of Borehole Shear Test (Holm, 1985), (2) multiple seepage faces on layered slopes (Rulon and Freeze, 1985), (3) slope cutting and valley formation due to fluvial erosion in the geologic time scale (Koppula and Morgenstern, 1984). The reason for the occurrence of negative pore water pressure in shearing of Borehole Shear Test is believed to be dilatancy (Nelson and Siu, 1971). The hypothesis regarding the cause of dilatancy are (1) particle reorientation, (2) liberation of stored internal energy, (3) changes in interparticle electrical forces, (4) physical interaction between hydrated ions, and (5) interlocking between particle (Holm, 1985). The time precess for the occurrence of negative pore water pressure in shearing of Borehole

Shear Test can be measured in seconds to minutes. For slope cutting, the process often can be measured in years to decades, while for multiple seepage faces on layered slopes, slope cutting, valley formation, and dredging due to fluvial erosion, time is on a geological scale (Koppula and Morgenstern, 1984, Rulon and Freeze, 1985). Although the last glacial ice retreated over 10,000 years during the Pleistocene period, the weight of the ice cap caused the soil to become overconsolidated and elastically rebound, when the ice retreated. However, the elastically rebound is slow and is not complete even to the present day. The erosion of natural slopes and the downcutting of the river valleys are usually sufficient slow and the unloading so small compared to glacial unloading that the dissipation of these negative pore water pressures occurs at the same rate as they are produced (Tavenas and Leroueil, 1981); but that part of the negative pore water pressure produced by retreat of glacial ice may still be locked inside the slope such that the slope is stable.

Theoretically pore water pressure distribution can be calculated by means of stress analysis and by numerical methods. Wilson (1963) calculated the total stress changes due to excavation of a slope by a finite element analysis using a computer program by considering the governing *in situ* soil parameters, such as K_0, A, C_s. K_0 is the earth pressure coefficient at rest that is required to obtain the initial stress condition and the magnitude of the load being removed. A is Skempton's pore water pressure parameter, and C_s is the coefficient of the swelling of the material. It appeared justified to consider a homogeneous, isotropic, elastic material in a semi-infinite half space (Eigenbrod, 1972, 1975).

Duncan and Dunlop (1969) reported that in order to minimize the influence of the boundaries, several factors should be considered: (1) the positions of the lateral boundaries are sufficiently far removed from the slope so that they have a negligible effect on the stresses and displacements in the regions of the slope, (2) the nodal points adjoining these artificial boundaries are constrained to move vertically only, (3) the position of the rigid base influence the stress and displacements in the slope region, and (4) the nodal points adjoining the rigid base are constrained from either horizontal or vertical movement. In their original program, the stress changes are calculated by unloading in one single step, however, this is not the real situation. Clough and Duncan (1970) analyzed unloading by single-step and multi-step and observed that the results of the two analyses are nearly identical except for the elements immediately adjacent to the final surface of excavation. The effective stresses around an excavation may depend on the water pressures (Chowdhury, 1978).

A two-dimensional finite element solutions is shown in Figure 4 by Duncan and Dunlop (1969) and Dunlop and Duncan (1970). It is shown that the distribution of pore water pressure is greatly influenced by the K_0 value. Vaughan and Walbancke (1973) proposed that the change in pore pressure due to excavation can be estimated by

$\Delta u = T\Delta h$------------------(1)

where Δh is the depth of clay removed from above the point.

Equation (1) is derived from Skempton's (1954) equation

$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$-------------------(2)

and finite element analysis had been proposed and shown (Duncan and Dunlop, 1969, Dunlop and Duncan, 1970) that for the central part of a slope, $\Delta\sigma_1 - \Delta\sigma_3$ due to excavation is small and

$\Delta\sigma_3 = T\Delta h$------(3)

From this, the equation (1) was derived.

Based on $\Delta u = T\Delta h$, the one-dimensional solution proposed by Vaughan and Walbancke (1973) is shown in Figure 5. This is similar with the two-dimensional finite element solutions under the central portion of the slope (Duncan and Dunlop, 1969, Dunlop and Duncan, 1970). The results calculated from $\Delta u = T\Delta h$ (Walbancke, 1975) is reasonably closed to the field observation (Kwan, 1971, Kankare, 1969) under the central portion of the slope. However, the equation $\Delta u = T\Delta h$ is modelled based only on vertical unloading will overestimate the pore water pressure changes at the base of the excavation and does not hold for very steep slope cuts. Furthermore, it is not valid at the crest of a cutting, in which pore water pressure is reduced mainly on horizontal unloading (Walbancke, 1975). Kwan (1971) proposed $\Delta u = 0.75\ T\Delta Z$ under the base which is based on the field observation of the Welland cut. The equation $\Delta u = T\Delta h$ is valid for short-term undrained situation. For long-term drainage occurs, pore water pressure is a function of the coefficient of swelling, time period, and boundary conditions. The coefficient of swelling is correlated to the rate of swelling which is accompanied by a reduction in strength and failure that therefore may be delayed by the rate at which swelling can occur. In early stages, the equilibration rate of excavated slopes in clay is comparable with the rates calculated from laboratory values of C_s measured on large samples (Walbancke, 1975).

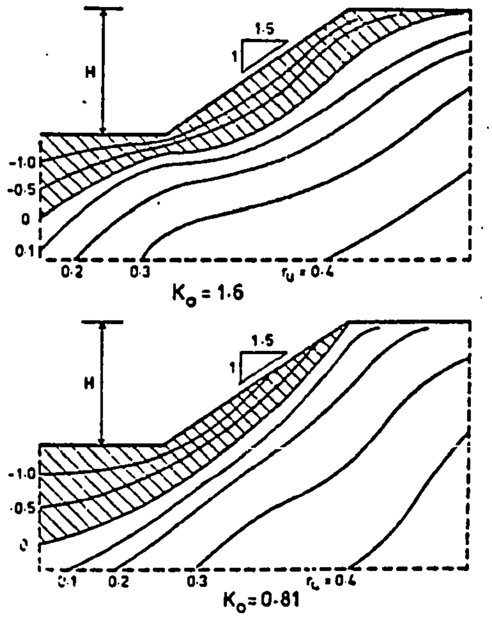

Figure 4. End of construction pore water pressure in a 1 on 1.5 cutting slope with various Ko value (after Duncan and Dunlop, 1969)

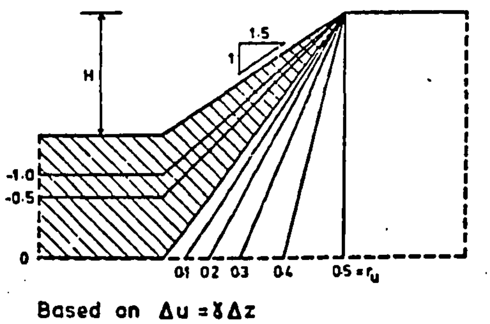

Figure 5. End of construction pore pressure in a 1 on 1.5 cutting slope (after Vaughan and Walbancke, 1973)

Eigenbrod (1972, 1975) calculated the time for equalization of excess pore water pressure Δu using a two-dimensional consolidation program which assumed that consolidation and swelling follow the same theory (Koppula, 1970) as mentioned previously. The equation governing the two-dimensional dissipation of excess pore water pressure under plane strain conditions which is written in a dimensionless form

$$\partial^2 u_e/\partial x^2 + \partial^2 u_e/\partial y^2 = \partial u_e/\partial \tau$$

where u_e = excess pore water pressure at a point (X, Y)

$\tau = C_s * t/H^2$ = time factor, C_s = coefficient of swelling

H = height of the slope

t = time since the beginning of dissipation

$X = x/H$, and $Y = y/H$

The equation was solved by Koppula (1970) and the computer program set up (Eigenbrod, 1975). The boundary condition is assumed as that no drainage is specified along the base of the slope and vertical boundary is considered far away from the excavated face (Duncan and Dunlop, 1969, Koppula, 1970). Eigenbrod (1975) found that the pore water pressure distribution after the first time steps indicated that variations in initial pore water pressures due to different assumptions of K_0 are equalized at very early stages of dissipation, and are little reflected during the later stages of pore pressure equilibration. However, there are no data from field observations to support this point. The influences of slope height on time for dissipation t_s as well as the influence of the coefficient of swelling C_s were evaluated $0.33H^2$ and can be expressed as $t_s = 0.33H^2/C_s$

where H = the height of the slope

C_s = coeffieient of swelling

0.33 = time factor for full pore water pressure equilibration

The time for full equilibration of pore water pressures is greatly influenced by C_s values, but is slightly influenced by slope height (Eigenbrod, 1975). Although this theoretical model of pore water pressure equilibration is fitted for sufficiently homogeneous soil, more field observations and the *in situ* coefficient of swelling in natural slopes are needed.

Based on field piezometer observation, Skempton (1977) reported that the pore water pressures had reached a state of equilibrium after 125 years. In striking contrast the pore pressures of the west side, only about one-half of the equilibrium values was reached after 19 years. No essential difference between the two sides other than of excavation time existed. It is reported that the long term value of $\overline{r}_u$ could be taken between 0.25 and 0.35 (Figure 6), and 0.3 could be used for back-analysis in the absence of reliable piezometric data at any given site (Skempton. 1977). However, this observation had been selected to exclude cut of unusually shallow depth; in these it would be expected that equilibration would be achieved on a shorter time scale. Slips in the zone of seasonal variation also have been excluded. The value of average pore water pressure may be influenced by the size of the landslide and the weathering condition as shown in Figure 7. The Lias clay is more weathered and the size of the landslide is smaller

(Chandler, 1984a). The significantly high pore water pressures (higher than Skempton's report) were observed in some of the London clay landslides at Herne Bay (Chandler, 1984a). Meanwhile, based on the back calculation of Linnan's (1986) landslide case of Pennsylvanian shale in Des Moines, the average T_u value is 0.37 which is not far off compared with Skempton's observation. Skempton (1977) also suggested that the fissures of the clay had little affect on *in situ* permeability of the clay slope after excavation when compared with values measured in the laboratory on small undisturbed samples.

The behavior of equilibration of negative pore water pressure was also used to predict swelling process (Chandler, 1984b). Stress paths can be used to illustrate the effects of swelling on strength and delayed failure for heavily overconsolidated clays as shown in Figure 8. With rapid (undrained) excavation to different depths of cut, the elements follow paths A to E, D, etc. After this excavation, soil swells with corresponding paths E to E', D to D', etc, the length of this portion of the stress path being related to the time period involved. If the excavation is shallow (path A-E-E'), swelling may be completed without failure occurring. If the excavation is deeper, failure may occur as long term pore water pressures are attained (A-D-D'), while path A-C-C' is intermediate term before swelling is complete. If excavation is continued, then a short term failure path A-B is followed with no swelling (Chandler, 1984b).

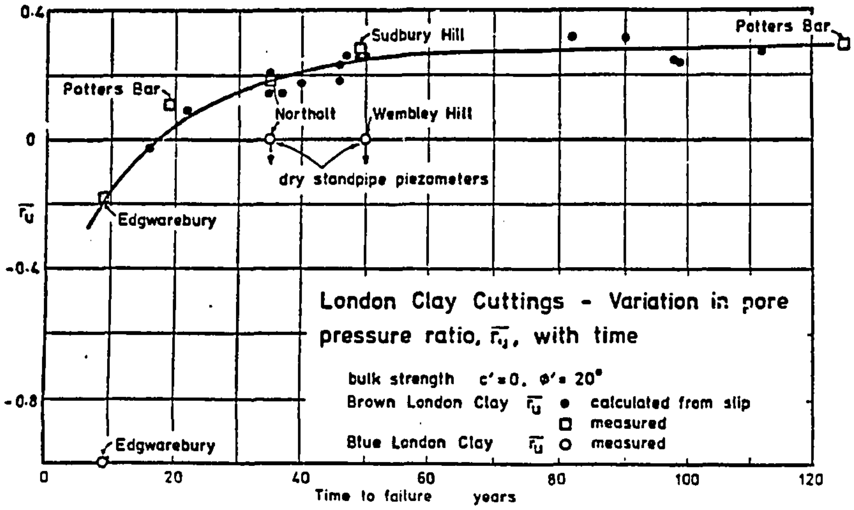

Figure 6. Variation in average pore water pressure parameter $\bar{r}_u$ with time of London clay cutting (after Skempton, 1977)

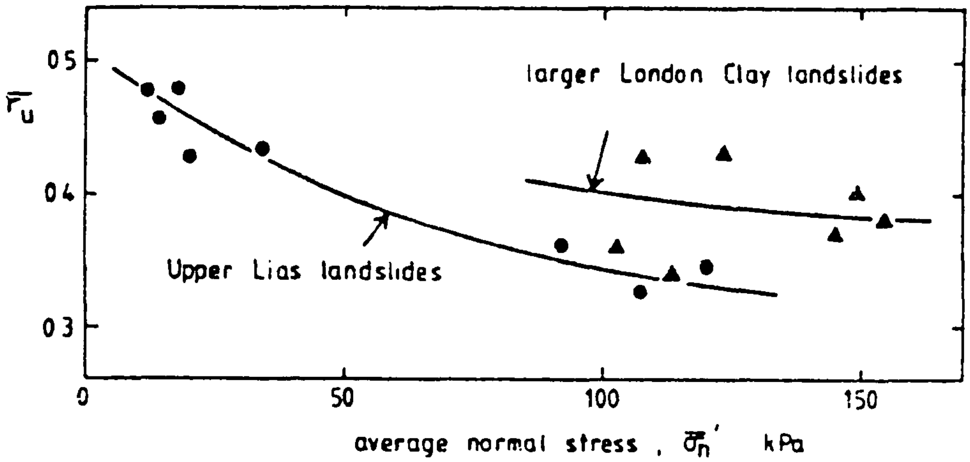

Figure 7. Relationship between average pore water pressure parameter $\bar{r}_u$ with average normal stress (after Chandler, 1984a)

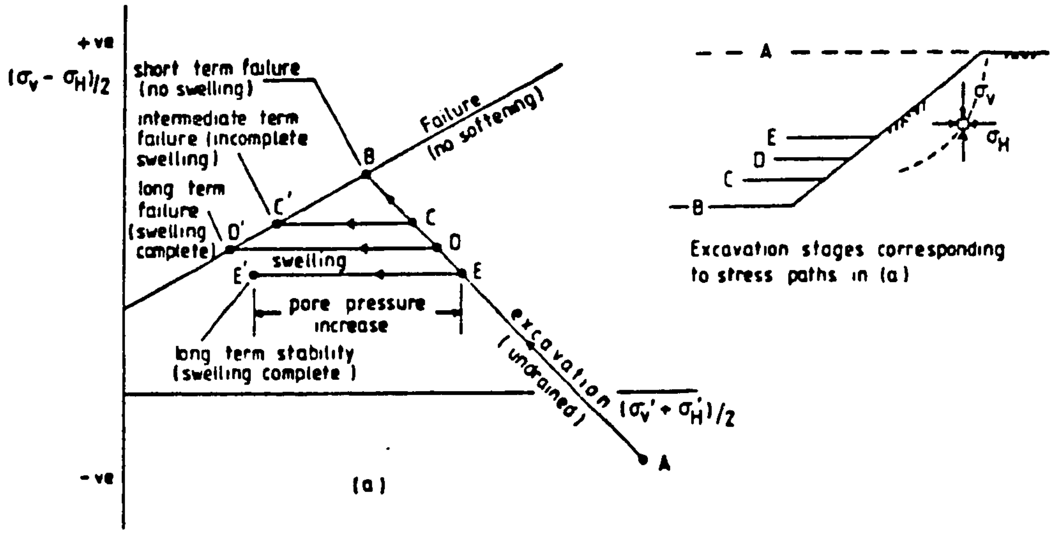

Figure 8. Stress paths for a typical soil element on the potential shear surface of a first-time slide (after Chandler, 1984b)

Tavenas and Leroueil (1977, 1981) proposed that the concepts of limit and critical state are best suited to represent the behavior of a wide variety of natural clays, which can be explained in terms of a rheological time dependent limit state of the clay wherein the strength of the clay ultimately decays to the critical state. Time dependent limit states exist shown as y in Figure 9. If the slope was excavated rapidly in undrained condition up to failure, the effective stress path would be such as Of_u. The rate of displacement of the limit state curve from (y_1) to (y_f), describes the creep characteristics of the clay. Paths such as U_1D_1 or U_2D_2 vary with the negative pore water pressure dissipation process. Failure of the element occurs when its stress path intersects the limit state corresponding to the strain rate of the element. The rate of pore water pressure dissipation and the creep behavior may be assumed identical, in which it will be influenced by the drainage boundary conditions of the clay slope, the coefficient of swelling, and the fluctuations of the water table (Tavenas and Leroueil, 1981).

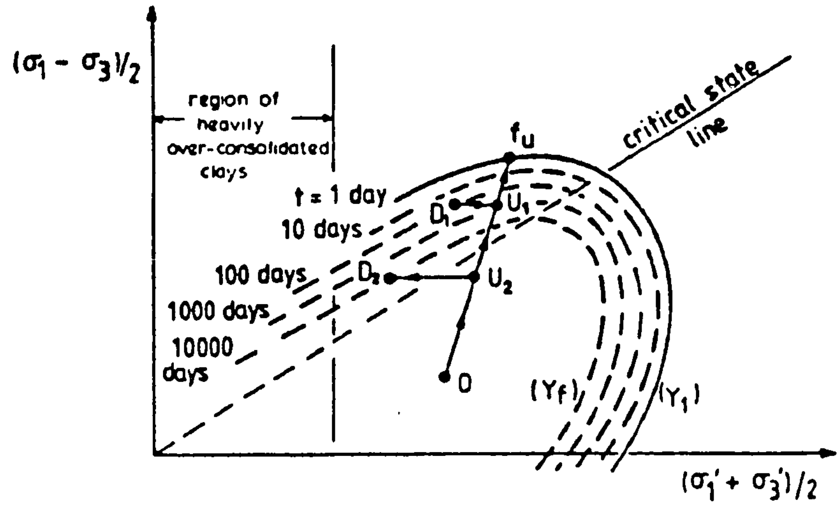

Figure 9. Time dependent limit state (after Tavenas and Leroueil, 1981)

Based on Terzaghi's classical consolidation theory, Koppula and Morgenstern (1984) derived an expression for negative pore water pressure as a function of depth and time. For a semi-infinite, homogeneous, fully saturated soil mass of finite thickness being eroded/excavated from the top at an arbitrary rate, this phenomenon may be viewed as opposite to the case of sedimentation in which additional soil layers are added at the top (Koppula and Morgenstern, 1984). Gibson (1958) presented the following equation for sedimentation

$$C * (\partial^2 u/\partial z^2) = \partial u/\partial t - (d/dt)(\Delta\sigma)$$

where C = coefficient of consolidation of the soil

u = excess pore water pressure at time t

$\Delta\sigma = rh$

T = unit weight of the soil

h = thickness of soil deposited in time t

z = height meaured from ground surface

Koppula and Morgenstern (1984) proposed that for the case of soil that is eroded/excavated from the top, the term $\Delta\sigma$ may be changed to $-\Delta\sigma$ of Gibson's equation, while C is substituted by C_s (coefficient of swelling of the soil).

A solution for the equation based on changing boundary and initial conditions was derived, in which it is assumed that the top of the eroded soil mass is free to drain and that at that point the negative pore water pressure is always zero (Koppula and Morgenstern, 1984). The negative pore water pressure u (z, t) normalized with respect to the weight of soil removed at any time is a function of (1) the depth of soil normalized in terms of z/h, (2) the location of the bottom impervious boundary expressed as H/h, depth factor, and (3) the ratio of the rate of soil removal to the swelling characteristics of the soil, called the erosion-swelling ratio, $R^2 = m^2t/C_s$. Based on these, figures (Figures 10, 11) for different soil removal rate were derived including oermeable or impermeable bases, depth factor, and normalized depth. From theses figures, the negative pore pressure can be predicted. It is seen that dissipation of negative pore water pressure is slow for fast rates of soil removal or for slowly swelling soils or both. On the other hand, the dissipation of negative pore water pressure is faster

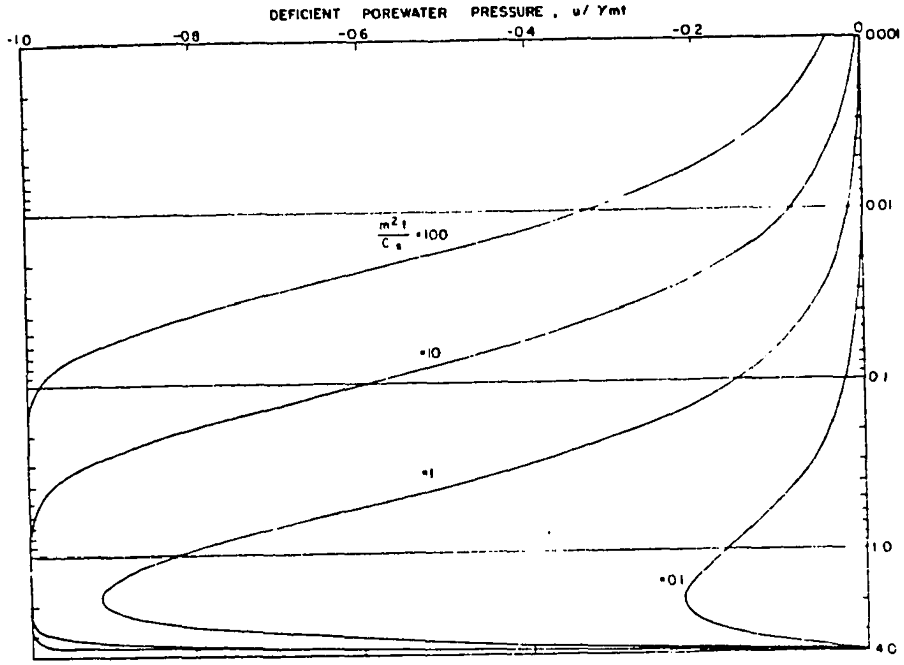

Figure 10. Relationship between negative pore water pressure with depth with pervious base erosion/swelling conditions (after Koppula and Morgenstern, 1984)

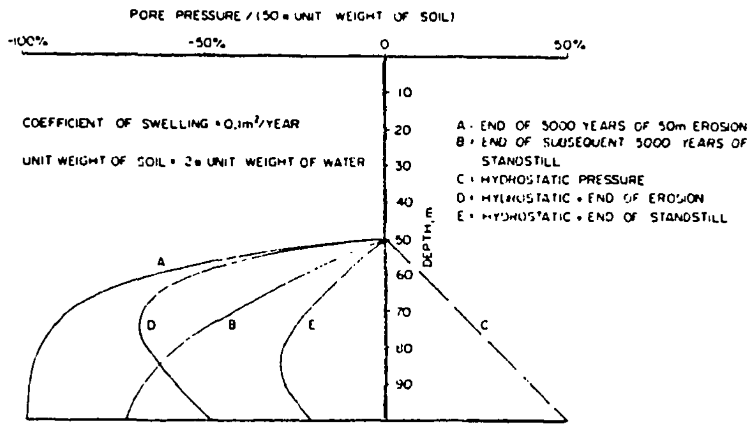

Figure 11. Relationship between negative pore water pressure with depth for valley formation (after Koppula and Morgenstern, 1984)

for slower rates of soil removal or fast swelling soils (Koppula and Morgenstern, 1984). In both cases, it is shown that appreciable negative pore water pressure occurs and there is a substantial potential for the soil to swell even after considerable time has elapsed after excavation.

For the prediction of the pore water pressure in cuts and slopes at equilibrium, the characteristics of different theories will be discussed; (1) finite element analysis (Wilson, 1963, Duncan and Dunlop, 1969, Dunlop and Duncan, 1970), and numerical analysis (Vaughan and Walbancke, 1973, Walbancke, 1975), which assume Skempton's equation for unloading (1954) is valid for limited parts of the slope, and assumes a short-term undrained situation, (2) concepts analysis (Tavenas and Leroueil, 1981, Chandler, 1984b), based on time dependent limit state and the process of negative pore water pressure equilibration respectively. These concepts are qualitative and no quantitative approaches are available at the present time, (3) observation method (Skempton, 1977), used for back-analysis, and a good indicator of slope stability in relation to the pore pressure, and (4) a numerical analysis method (Koppula and Morgenstern, 1984) based on sedimentation or consolidation analysis. By utilizing several data bases such as the rate of soil removal, the swelling potential of the soil mass, the ratio of the thickness of soil layer removed to its original thickness, and the nature of the bottom boundary, negative pore water pressures can be predicted.

Progressive Failure

Introduction

Bishop (1971) had suggested that it may not be possible to distinguish between progressive failure and softening on the basis of back-analysis alone. Both mechanisms describe that strength will decrease with time. However, the softening process is characterized by the "fully softened state of strength" (Skempton, 1970) with $c' = 0$ and $\phi' = \phi'_{peak}$ as the governing parameters, in which the uniform mobilization of a fully softened strength is reached along the failure surface (Skempton, 1970). Progressive failure is characterized by residual strength with $c' = 0$ and $\phi' = \phi'_{residual}$ as the governing parameters (Skempton, 1964, Bjerrum, 1967), in which large displacement must be mobilized in order to drop the shear strength from the peak to the residual value (Skempton, 1970, James, 1971). Morgenstern (1977) defined this process as "the non-uniform mobilization of shear strength along a potential slip surface". It can developed as: In the vicinity of the toe of the slope or where excessive deformations have occurred, localized points are overstressed and exceed the clay peak strength, whereupon the strength of the clay is reduced due to the strain softening characteristics of the clay. This action places additional stress at adjacent points, causing the peak strength of these points to be passed, so that conditions are created for the zone of failure to increase. Thus, failure may progress within a soil mass from one end of a slip to the other, the net result being that the average shear strength available at failure is less than the peak strength. The possible behavior of progressive failure has been recognized for some time (Taylor, 1948, Skempton, 1964, Peck, 1967, Turnbull and Hvorslev, 1967, Bishop, 1971). It can be applied to problems of bearing capacity and earth pressure as well as slope stability (Morgenstern, 1977). The definition of "progressive failure" is not the same as "progressive slide", in that progressive failure spreads in the up-slope direction which is opposite to its motion, and is called a "retrogressive slide" while the progressive slide spreads in a down-slope direction which is the same as its motion (Kjellman, 1955, Chowdhury, 1978).

Development of research program and questions studied

The strength reducing mechanism has been discussed by Terzaghi (1936) and Skempton (1948) as mentioned previously. Also, Cassel (1948) and Binger (1948) compared laboratory strengths and field strength in several England clay and Panama Canal clay, they found that the failed strength of clay in the field occurred much lower than ever measured by laboratory tests.

Prior to mid-1950s, slope analyses were confined to a total stress approach using cohesive strengths alone. Several researchers (Henkel, 1955, de Lory, 1957) suggested that for a long term design of slopes the effective stress method should be used which assumed the strength decay is confined to the effective cohesion parameter c', and the parameter ϕ' was assumed constant. In the early 1960s more case histories were reported in overconsolidated clays and clayshales which could not be explained with the conventional effective stress approach. That relatively small movement may reduce the strength while the angle of shearing resistance (ϕ') can be decreased permanently due to strain was suggested from field observations (Gould, 1960) as well as laboratory tests (Borowicka, 1963). Schmertmann and Osterberg (1960) observed from triaxial shear tests that, strength due to friction between particles is more stable than cohesion. The phenomenon of the residual strength was formerly postulated and defined by Skempton (1964).

Bjerrum (1967) suggested that the mechanism for the progressive failure is the result of the release of recoverable strain energy on weathering in overconsolidated clays. The weathering under his definition includes physical disintegration, chemical changes, and decomposition of the mineral. During physical disintegration, the structure of the clay is disturbed by a breakdown of the bonds, and the locked-in recoverable strain energy will be liberated causing the clay to expand, resulting in water content increase and shear strength decrease. The total amount of expansion depends on the amount of the strain energy in the clay. If the bonds are weak, most of the strain energy is dissipated during unloading, and the effect of disintegration will be small. If the bonds are strong, only a small portion of the strain energy is lost during unloading, and the expansion will be large. Also, the amount of strain energy stored in a clay under load is dependent on the type of clay minerals, the greater the content of active clay minerals, the greater the recoverable strain energy, which may lead to nonuniform swelling accompanied by local nonuniform strains.

Progressively, these local strains may be large enough to produce local shear failure and formation of cracks and fissures (Bjerrum, 1967).

Based on the concept that the shaded area bound by the loading and unloading branches of the curve represents strain energy (Sealye and Smith, 1952), Brooker (1968) ran a series of large scale consolidation test on several different kinds of overconsolidated clays and clayshales. His quantitative data support Bjerrum's (1967) strain energy hypothesis. Brooker (1968) then further proposed that there have relationships between strain energy, overconsolidation ratio, and coefficient of lateral stress at rest (K_0). Based mainly on local overstressing that due to local overstress, Bishop (1967) proposed that a zone of plastic equilibrium is formed in a slope before general failure take place. Also, a change in loading conditions or pore water pressures will lead to a progressive extension of the failure zone along the potential slip surface, then the shearing resistance within the zone will drop from the peak to the residual strength. It had been suggested that in a slope the stress distribution is nonhomogeneous and the strength mobilized during failure must be in a nonhomogeneous way (Conlon, 1966). According to Conlon's test results, Peck (1967) proposed that failure may also initiate from the crest of a slope which did not consist with the widely held belief about failure always initiating and progressing from the toe. Turnbull and Hvorslev (1967) discussed the progressive failure referring to nonhomogeneous stress distribution and local overstressing as described by Bjerrum (1967) and Bishop (1967).

Based on finite element analysis, Duncan and Dunlop (1969) studied the effects of initial lateral stresses around a slope, and found that the stress conditions in a slope after excavation were strongly influenced by K_0 value, they proposed that the existence of high horizontal stresses in heavily overconsolidated clays and clayshales increase the probability of progressive failure. Yudbhir (1969) showed that K_0-unloading causes shear failure of the specimen in oedometer tests, and concluded that the release of horizontal stresses in overconsolidated clays to be a dominant factor. Yudbhir further suggested the K_0 effect a dominant factor in progressive failure. Using a theoretical model, Christian and Whitman (1969) studied the effect of horizontal stress release on pregressive failure. Even for slopes with a high factor of safety with respect to the residual strength, due to the release of initial stress by erosion or cutting, there is a possibility of propagation of a failure surface.

Two stages of strength reduction can be distinguished, a fully softened condition and the residual condition. At fully softened strength, only a complex of minor shears such as the Riedel thrust and displacement shears exist. In order to drop the fully softened to residual strength. Particle reorientation will have occurred along theses minor shears to form a smooth continuous surface (Skempton, 1970). The fully softened state of strength is the same strength at the "critical state" in terms of critical state soil mechanics (Roscoe et al., 1958, Schofield and Wroth, 1968).

James (1970, 1971) suggested that very large movements (in the order of several feet) are prerequisite for strength reductions in a slope from peak to residual. He further pointed out that unless the lateral movements are localized to one thin seam, lateral movement due to high lateral stresses would be insufficient to drop the angle of shearing resistance of the clay to the residual condition. Therefore, in homogeneous clay slopes such as London clay, progressive failure is unlikely, except at the brown/blue clay interface.

Bishop (1971) proposed a brittleness index I_B in relation to progressive failure of overconsolidated clays, the index being defined as: $I_B = (S_p - S_r)/ S_p$

where S_p = peak strength of soil

S_r = residual strength of soil

The higher the I_B value, the higher the possibility of progressive failure.

Palmer and Rice (1973) developed a shear band concept on the basis of fracture mechanics considering a planar slip surface in a given material to be a crack. They considered the case of a slip surface starting from a step or cut in a long slope, in which the growth of slip surface is a progressive phenomenon. Based on the energy balance, they proposed a theoretical model to calculate the shear band length no matter whether the shear stress is between the peak and residual strengths or below the residual strengths. The factor of safety is defined as a ratio of the energy release during unit advance of the shear band and the energy driving the shear band (Palmer and Rice, 1973). Rice and Simons (1976) compared basic cases and found the results favorably close with Palmer and Rice's (1973). Rice and Simons (1976) had extended their analysis to consider some of the time-dependent aspects with pore water pressure redistribution. Chowdhury (1977, 1978) also proposed a shear band model, and for flat slopes the results agree closely with the Palmer and Rice's (1973) energy approach.

Theoretical models of progressive failure

Different concepts and definitions of progressive failure are in use. Although different progressive failure mechanisms had been studied extensively (Terzaghi, 1936, Skempton, 1948, 1964, 1970, Bjerrum, 1967, Bishop, 1967, 1971, Peck, 1967, Christian and Whitman, 1969, Yudbhir, 1969, Eigenbrod, 1972, Palmer and Rice, 1973, Morgenstern, 1977, Chowdhury, 1977, 1978), various opinions still exist for the conditions such as (1) which mechanism is most suitable, (2) what kind of geological condition is suitable for progressive failure, (3) where the crack initiates, and (4) what is the influence of initial stress and strain energy. In order to consolidate and solve these problems, the followings will be discussed: (A) mechanism, (B) geological condition, (C) crack initiation, and (D) effects of initial conditions.

(A) Mechanism: During the progressive failure process, large displacements are needed in order to drop the strength from peak to a fully softened and residual condition, such displacements definitely being larger than those needed to open fissures that represent the starting condition for the Terzaghi mechanism. James (1970) showed a relationship between field strain and reduction in ϕ' (Figure 12). For example, in a 30 ft highway cut in London clay, a slip movement of around 7 ft (field strain ≈ 0.25) would be needed to lower ϕ' by 6^0, from the peak to the residual values ($\phi'_{peak} = 20^0$, $\phi'_{residual} = 14^0$ for London clay). Based on the back-analysis of field failures strength parameters for either first-time slide or reactivated slides can be calculated out (Skempton, 1970, James, 1970). While Skempton (1970) postulated the uniform mobilization of a fully softened strength along the failure surface, many others (Bishop, 1967, 1971, Peck, 1967, Turnbull and Hvorslev, 1967, James, 1971, Morgenstern, 1977, Chowdhury, 1978) proposed that progressive failure is due to the nonuniform mobilization of shear strength along a potential slip surface, which may occur due to local overstress, large deformation, or changes in loading conditions or pore pressures. This failure process can't be interpreted by softening or negative pore water pressure equilibration.

Several theoretical progressive failure model have been proposed by different researchers (Christian and Whitman, 1969, Palmer and Rice, 1973, Rice and Simons, 1976, Chowdhury, 1977, 1978). Christian and Whitman (1969) developed a one-dimensional mathematical model of a single layer bonded to a rigid base, and the material is assumed to be elastic, plastic, and strain softening as Figure 13. Based on simplified brittle nature of shear stress-displacement curve and equilibrium consideration of an infinitesimal element, they integrate the differential equation and suggest that first yield occurs (with no relative displacement until a critical shear stress is reached) when $P/S_p > \{E/K_h\}^{1/2}$

The extent of the failure surface is:

$L/h = \{(-P/S_p) + (E/K_h)^{1/2}\}(S_{p'}/S_{r'})$

The factor of safety against first yield:

$F * S = (2 \sin \beta/K_0)(E/K_h)^{1/2}(FS_p - 1)$

where P = initial stress

S_p = peak strength

S_r = residual strength

L = location

δ = displacement

K = slope of the peak line

E = modulus of the soil

h = thickness of the soil layer

$S_{p'}$ = reduced peak strength obtained by deducting gravitational shear component ($rh \sin \beta$) from S_p

$S_{r'}$ = reduced residual strength obtained by deducting gravitational shear component ($rh \sin \beta$) from S_r

β = angle of inclination

K_0 = the coefficient of lateral stress at rest

FS_0 = the factor of safety for peak strength

This model uses parameters other than those considered in the conventional stability analysis of slopes, and indicates that factors such as initial stress may have significant effect on whether or not a

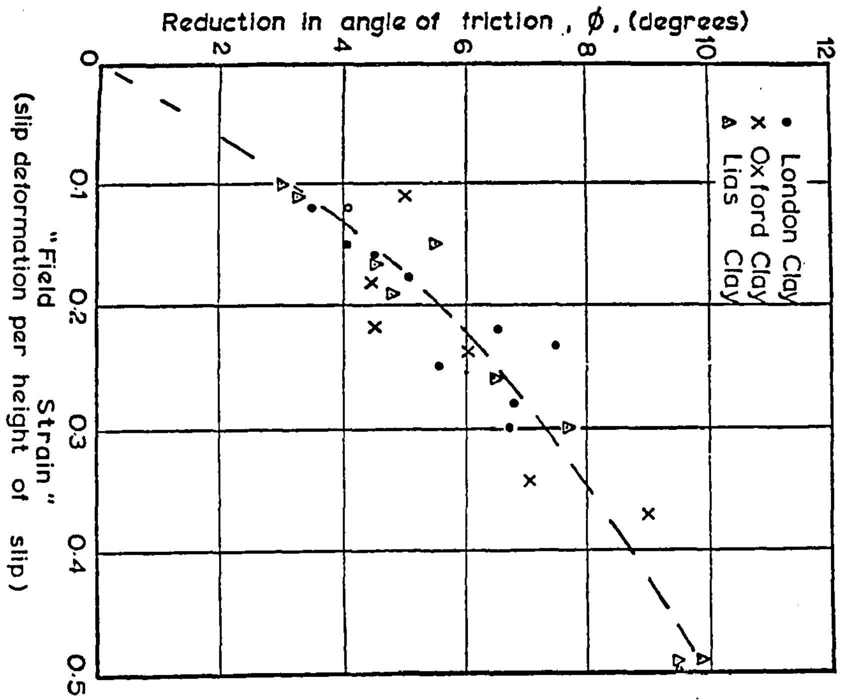

Figure 12. Relationship between movement of slip and loss in strength (after James, 1970)

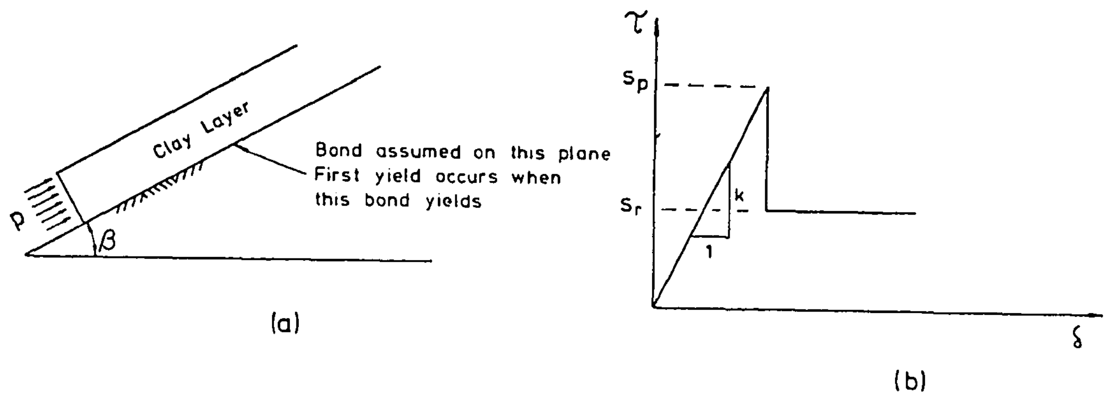

Figure 13. Theoretical progressive failure model: (a) Simple progressive failure model in brittle strain softening soil, (b) Shear stress-displacement curve as assumed for the model (after Christian and Whitman, 1969)

failure will start. Also, this analysis is based on a simplified stress-strain curve which shows an abrupt drop in strength from peak to residual and no displacement between peak and residual strength. However, this behavior is not realistic for real soils which need large displacements in order to drop the strength to residual state (James, 1970, Skempton, 1970). Furthermore, the analysis implies failure takes place along the interface between the soil layer and rigid base and this is not valid for rotational type slides. On the other hand, Skempton (1970) proposed that when the strength drops to the residual state, particle re-orientation will have occurred and a smoothly continuous principal shear surface will have developed, so at this stage the stress-strain relationship in the failure zone may be close to that proposed by Christian and Whitman (1969). Therefore, the theoretical model proposed by Christian and Whitman (1969) is still a good approach but with some restrictions.

The model of Palmer and Rice (1973) assumes that only two shear strengths are mobilized along a potential failure, a residual value along the shear band and a peak value outside the shear band. However, this model can not represent all field conditions (Chowdhury, 1978). Chowdhury (1977) proposed a theoretical model of nonuniform shear stress on a band, and thus the shear stress along the band can be arbitrarily chosen between the peak and residual values. When (1) the material is brittle, (2) the relative displacement in the post peak range is zero, and (3) gravitational shear stress along a shear band is less than residual shear strength (as in very flat slopes), Chowdhury's (1977) model will be close to Christian and Whitman's (1969) model. Neither Palmer and Rice (1973) nor Chowdhury (1977) considered the effect of initial stress. Burland et al. (1977) studied a ground movement caused by a deep excavation in Oxford clay of England. Based on the field data from horizontal extensometers, and from inclinometers, they observed that the growth pattern of a slip surface fit those shear band concepts.

(B) Geological condition: Progressive failure in geological materials have been reported by many researchers, including granular materials (Taylor, 1948, Roscoe et al., 1958, Rowe, 1962), quick clays (Crawford, 1968, Conlon, 1966, Bjerrum et al., 1969), and rocks (Terzaghi, 1962, Haefeli, 1965). Haefeli (1965) reported progressive failure in snow slabs. Also, Cavounidis and Sotiropoulos (1980) reported progressive failure in Marl. For overconsolidated clays and clayshales, the strain softening behavior had been discussed extensively (Terzaghi, 1936, Skempton, 1948, 1964, 1970, Bjerrum, 1967, Bishop, 1967, 1971, Bishop et al., 1971, James, 1970, 1971, Lo, 1970, Eigenbrod, 1972, Lo and Lee, 1973, Morgenstern, 1977, Chandler, 1984a).

James (1971) pointed out that considerable displacement is needed in order to drop the strength from peak to residual state. It has been suggested that unless deformations are localized along one thin layer or at the interface between two somewhat different layers such as brown and blue London clay strains would be insufficient to approach the residual state (James, 1971, Eigenbrod, 1972, Morgenstern, 1977). In other words, progressive failure is unlikely in homogeneous clay slopes. Typically, London clay falls into the homogeneous category, except at the brown and blue interface (James, 1971). Through back-analysis, James (1970, 1971) and Skempton (1970, 1977) proposed that the failures in London clay the first-time slides, in which only fully softened strength is mobilized.

(C) Crack initiation: It often has been assumed that failure in natural and cut slopes begins at the toe or the bottom of a potential surface, based on stress concentration likely being highest at the toe (Chowdhury, 1978). However, Bishop (1967) postulated that under drained conditions progressive failure initiates from both ends of the rupture surface inwards. Peck (1967) discussed Conlon's test results (1966) with respect to progressive failure on overconsolidated clays, and suggested that strength is to be mobilized in a nonhomogeneous way since the stress distribution is nonhomogeneous. Under constant loading the ratio of maximum shear stress to normal stress has a high value at the ends and low value in the middle of a slope. When long-term drained conditions are approached, the shear strength increases with effective normal stress. In the interior of the slope, there is a greater depth of material above points on the slip surface, so the ratio of shear strength to normal stress is higher at both ends, the likelihood of failure starting in the middle is remote under such conditions (Chowdhury, 1978). At low stress levels peak shear strength will be reached after smaller displacements than at higher stresses. A tension crack at the crest of a slope was suggested as a consequence of the initiation of shear failure (Barton, 1972). Hoek and Bray (1977) also viewed that cracks are an evidence of the initiation of progressive failure. Therefore, failure is most likely to progress from one or both ends of a potential slip surface. However, based on field observations with inclinometers, de Beer (1967, 1969) suggested that rupture may progress predominantly from the toe towards the top of the slope. James (1970) ran a series of analyses compare the stability of slopes in London clay, Oxford clay, and Lias clay. In these analyses, the factor of safety between two different circumstances without a tension crack or with a tension crack were compared. The tension crack depth ranged from 5 ft to 18 ft, about 25% to 30% the depth of the slip. James found that in most cases no significant difference exists with or without the tension crack. The stress distribution following the excavation had been analyzed (Duncan and Dunlop, 1969, Clough and Duncan, 1970, Lo and Lee, 1973) and showed that the stress concentrated in the toe area, and this overstressed zone will propagate towards the interior and upslope as time passes. Romani et al. (1972) used a variational calculus approach to analyze the effect of a crack on the stability of the slope; they found that the factor of safety against slope instability varies considerably with the degree of development of cracks. Full crack development yields minimum values with progression from toe to crest, while partial development of the crack is safer and gives a direction from crest to toe. Based on field observations of Leda clays, Mitchell and Eden (1972) reported that creep rates are maximum at the toe of the slope. All of these phenomena suggest that initiation of cracks for overconsolidated clays and clayshales starts from the toe area. However, for normally consolidated clays, the cracks initiate from the crest area (Duncan and Dunlop, 1969).

The rate of crack propagation may be related to the strain softening behavior, or the rate of strength loss with time. Little information exists regarding the rate of decrease of drained strength with time in the field. Skempton and Hutchinson (1969) suggested that the rate of strength loss is 3.5% per log cycle of time for remolded Weald clay. For undisturbed London clay, Bishop and Lovenbury (1969) observed the value was of the order of 4.8% per log cycle in long-term drained creep tests. Based on the finite element analysis, Lo and Lee (1973) proposed that a rate of decrease of drained strength of 6% per log cycle of time will be consistent with field failure records.
For shear bands, Palmer and Rice (1973) proposed that the rate of propagation should be controlled by (1) dilationally induced suctions, and (2) bulk diffusion. Rice (1973) considered some typical laboratory creep data and estimated it would require a 10% to 15% increase in average shear stress to increase the velocity of propagation from an order of 3 ft per year to an order of 3 ft per day. None of the above proposals consider time-dependent aspects of pore water pressure distribution. Rice and Simons (1976) have extended Palmer and Rice's model, in which they considered time effects and the stabilization of shear faults by coupled deformation-diffusion effects, so that speed,

slipping length, and permeability could be related to creep movements. Although the results of Rice and Simons are interesting, so far only qualitative data have been presented.

(D) Effects of initial conditions: Initial stress conditions in overconsolidated clays and clayshales may contribute to the slope stability problem (Duncan and Dunlop, 1969, Dunlop and Duncan, 1970, Henkel, 1970, Lo and Lee, 1973, Chowdhury, 1976, 1977, 1978, Schmertmann, 1985). It has been reported that the horizontal stresses in heavily overconsolidated clays and clayshales may exceed the overburden pressure by 50%, or even more in some cases (Peterson, 1954). Duncan and Dunlop (1969), Clough and Duncan (1970), and Lo and Lee (1973) used finite element analysis to study the effects of initial stress following excavation, the stress distribution being influenced by the initial stress as shown in Figure 4. The higher the initial stress, the large the area of the overstressed zone (Lo and Lee, 1973). The ratio of lateral to axial effective stress (the coeffieient of earth pressure at rest, K_0) increases with increasing degree of overconsolidation under conditions of no lateral strain in laboratory tests (Kjellman 1936, Bishop and Henkel, 1953, and Brooker and Ireland, 1965). Brooker and Ireland (1965) defined relationships between the coefficient of earth pressure at rest and the overconsolidation ratio and plasticity index as shown in Figure 14. Bjerrum (1967) hypothesized that in highly plastic clays, diagenetic bonds may form which inhibit the development of high lateral pressures during unloading, and result in considerable strain energy being stored in these clays after unloading. Stored strain energy increased with increasing plasticity index as shown in Table 3. This supports Bjerrum's strain energy hypothesis quantitatively (Brooker, 1968). The greater the content of recoverable strain energy, the greater the danger of progressive failure. The release of strain energy is related to the diagenetic bonds, the critical will be occurred if a clayshale with strong diagenetic bonds is subjected to the various agents of weathering which results in a energetic swelling and initiating of progressive failure (Bjerrum, 1967). It can be shown that the higher the recoverable strain energy, the higher the tendency for swelling upon weathering. It can be concluded that high initial lateral stresses could result in large shear stresses at the base of an excavated slope as well as increased possibility of progressive failure (Bjerrum, 1967, Duncan and Dunlop, 1969, Dunlop and Duncan, 1970, Lo and Lee, 1973).

<u>Relationship between delayed failure and progressive failure</u>

There are no well-documented case histories of first-time slides in heavily overconsolidated clays and clayshales to indicate that progressive failure plays a dominant role in stability (Morgenstern, 1977). To distinguish between delayed failure and progressive failure therefore is important. First, delayed failure may be mainly due to negative pore water pressure equilibration, and softening may have an important role in weathered clay at shallow depth. The process of failure may be like this: At the end of the negative pore water pressure equilibration, the softening process follows and a first-time slide

Table 3. Characteristics of several overconsolidated clays and clayshales (after Brooker, 1968)

	LL %	PL %	PI	<0.002mm %	Activity	Mineralogy Mont. %	Mineralogy Illite %	Strain energy (in.-lb/in.^{3})
Chicago	28	18	10	36	0.29	5	40	85
Goose Lake Flour	32	16	16	31	0.50	-	15	85
Weald clay	41	21	20	39	0.53	10	15	84
London clay	64	26	38	64	0.60	15	35	100
Bearpaw shale	101	23	78	59	1.53	60	-	130

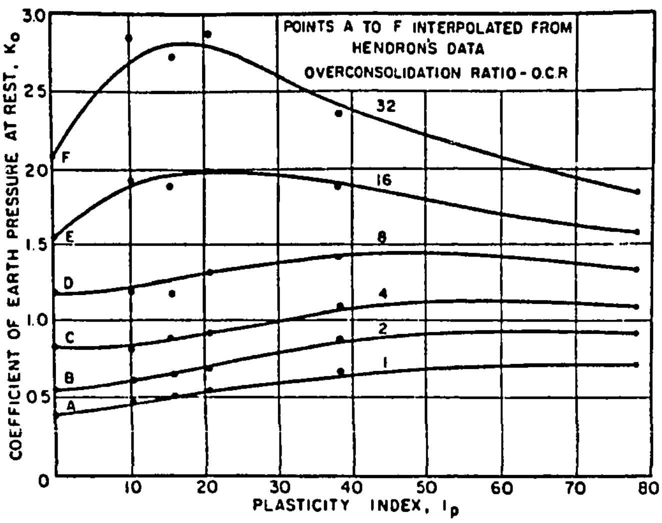

Figure 14. Variation of K_0 values with plasticity index for several values of overconsolidation ratio (after Brooker and Ireland, 1965)

occurs. However, if softening continues with large displacements, prior to sliding, a progressive failure occurs. On the basis of back-analysis alone, it may not be possible to distinguish between progressive failure and softening (Bishop, 1971). Table 4 lists some comparisons between these two failures. For a first-time slide, fully softened strength is mobilized along the whole failure surface in which time-dependent weakening of clay soils is only in cohesion (Skempton, 1970). For a progressive failure there is nonuniform mobilization of shear strength along a potential slip surface, in which a frictional resistance is reduced to the residual state. A short-term or immediate failure of an excavated slope could be considered either a simultaneous one or progressive one depending on the mechanism involved (Chowdhury, 1978). Delayed failure may result primarily from pore water pressure equilibration after a long period of time (Eigenbrod, 1972, 1975, Vaughan and Walbancke, 1973, Walbancke, 1975, Morgenstern, 1977, Skempton, 1977), and progressive failure from local overstressing with large deformations or changes in the loading conditions (Skempton, 1964, Bjerrum, 1967, Bishop, 1967, 1971, Eigenbrod, 1972, Morgenstern, 1977).

Creep

Introduction

Creep is a widespread phenomenon, its action occurring at different scales ranging from creep of atom-size flow units in a deformed crystal, to secondary consolidation movements of soil grains, to creep of continent size tectonic plates (Ter-Stepanian, 1980). Creep in slope

Table 4. Relationship between long-term delayed and progressive failure (adapted from Skempton, 1964, 1970, James, 1970, 1971, Morgenstern, 1977)

Slide Type	Slide Mechanism	Geological Condition	Strength Parameters	Displacements
First-time slide	Delayed failure	Mostly homogeneous soil	$c'=0$, $\phi'=\phi_{peak}$ (fully softened state)	Small
Reactivated slide	Progressive failure	Mostly nonhomogeneous or layered soil	$c'=0$, $\phi'=\phi_{residual}$ (residual state)	Large (in the order of feet)

can be defined as the very slow downward and outward movement of a mass of earth slopes, involving soil, rock, ice, or a combination materials, without the formation of a continuous rupture surface, which usually precedes in landslides (Emery, 1979, Ter-Stepanian, 1980). All slopes are subject to creep, in many cases so small as to be virtually unmeasureable, up to reported measured rates of mass rock creep ranging from 1.78 cm per year to 20 cm per day (Muller, 1964).

Terzaghi (1953) distinguished creep movements as seasonal, or mantle creep, and continuous or mass creep. Seasonal creep proceeds in the upper layers only resulting from a number of seasonal processes including expansion and contraction due to temperature changes, wetting and drying, frost cycles, animal burrowing and treading, and plant root activity (Terzaghi, 1953, Hungr, 1981). For surficial movement, the vertical distribution of the rate of movement is cumulative upwards, with a maximum at the surface and weakening with depth (Kirkby, 1967). The seasonal movement is considerably more important in periglacial regions where strong freeze-thaw cycles and inhibition of drainage by permafrost often occur (Hungr, 1981). Slow seasonal flowage of the active layer above frozen ground is termed solifluction or gelifluction.

Continuous creep is produced below the depth of seasonal variations, and is the result of sustained gravitational shear stresses unaided by other agents (Terzaghi, 1953, Hungr, 1981). Usually continuous creep occurs on a large scale. Six different types of geological setting of continuous deep-seated creep have been described: (1) valleyward squeezing of soft layers overlain by rigid caps, (2) distortion and buckling of rigid inclined layers on soft bases, (3) localized distortion in uniform material, (4) incremental movements on rough-surface inclined discontinuities, (5) deep-seated bending, folding, and plastic flow of rocks on slopes, and (6) bulging, spreading, and fracturing of steep slopes (Radbruch-Hall, 1979). Varnes (1978) defines two corresponding classes, "bedrock flow" and "soil creep", in which the classification did not acknowledge the presence of deep-seated creep in soils (Hungr, 1981). Creep may proceed continuously under normal gravitational stresses, or occur in increments in response to environmental factors such as seasonal high levels of the ground water table (Ter-Stepanian, 1980). In drained laboratory tests on clays, continuing long-term creep is known to take place at stresses that are only a fraction of their peak strength. There exists a critical value of stress, below which creep may take place but creep failure will not occur. At stresses greater than the critical stress, creep failure will eventually occur (Haefeli, 1965, Singh and Mitchell, 1968, 1969).

Development of research program and questions studied

Sharpe (1938) defined creep as the slow downslope movement of superficial soil or rock debris, at a rate that usually is imperceptible. Later, he defined creep as the ground movement in top layers produced by thermal expansion and contraction, swelling and shrinking, freezing and thawing, and other seasonal processes (Sharpe and Dosch, 1942). Since Sharpe's definition, the concept of creep as an independent type of mass movement on slopes has gained general recognition.

Terzaghi (1950) was the first to show a connection between creep and landslides as shown in Figure 15, a diagram illustrating the relationship between creep, sliding, and safety factor with time. A similar movement proceeding at an imperceptible rate is called creep. At point a, a change in the equilibrium of the slope takes place, a slide producing agent begins to act followed by gradual accleration. At point b, the acceleration reaches a maximum and rapid movement occurs which is called failure. At point c, a new steady post-failure stage begins. A simple landslide may behave according to this scheme (Terzaghi, 1950, Ter-Stepanian, 1980, Hungr, 1981). However, complex landslides may act neither uniformly nor steadily, and the horizontal

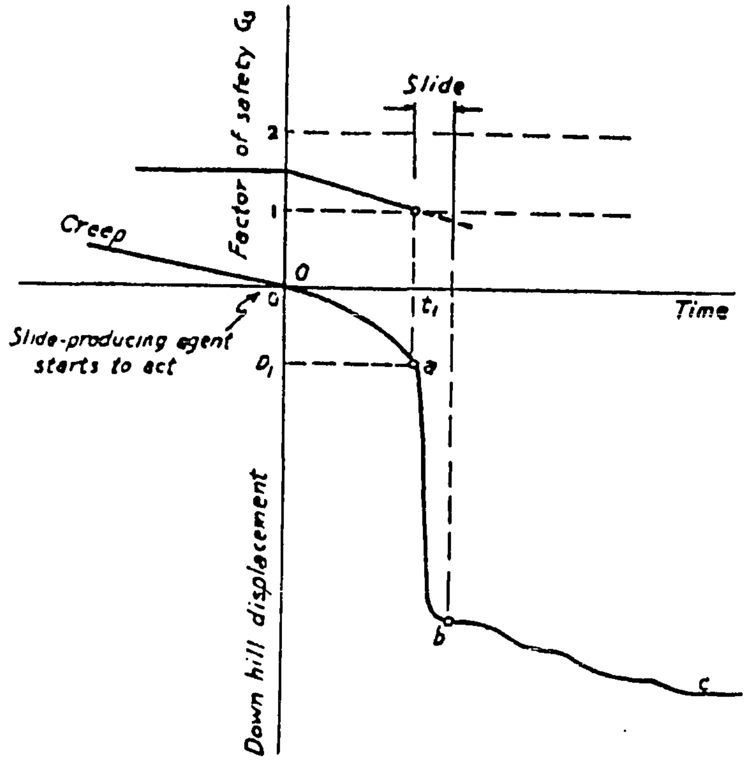

Figure 15. Diagram illustrating the ground movements which precede a landslide and the changes of factor of safety (after Terzaghi, 1950)

displacements prior to the catastrophic Vajont rock slide appear as a complex sequence of several S-shaped curves, as shown in Figure 16 (Muller, 1964). Terzaghi (1953) distinguished between the seasonal creep and continuous creep. He defined creep as an imperceptibly slow movement continuously downward and outward, in which the creep movement is essentially viscous under shear stresses sufficient to produce permanent deformation but too small to produce rapid shear failure as in a landslide. Creep rate is dependent on shear stress intensity and rheological properties of soils, and is subjected to changes caused by landslide-producing agents (Goldstein and Ter-Stepanian, 1957).

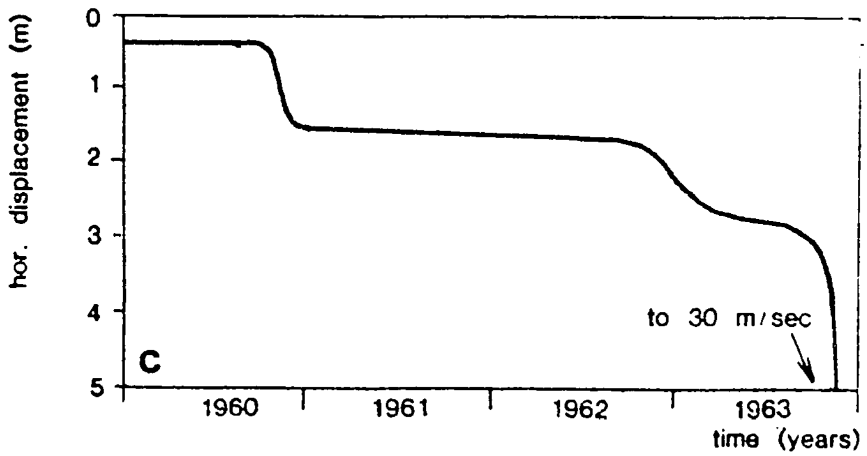

Figure 16. Variation of horizontal displacement with time of Vajont slide (after Muller, 1964)

Terzaghi (1962) distinguished between surficial rock falls and deep seated rock slides, and emphasized that the gradual development of local joint systems is responsible for slides representing combined effects of an increase in shearing stresses and a slow creep deformation of the rock acted upon by these stresses. Types of deep creep of slopes in rock masses had been described as: (1) translational creep which occurs on long slopes containing a network of structural discontinuities that comprise one or more of the following types: faults, bedding, joints, fissures, and dipping, (2) rotational creep which occurs on short slopes with homogeneous rocks without bedding, lamination, or steeply dipping beds of rocks, and (3) general creep in the case of complicated geological structures (Ter-Stepanian, 1966). Hutchinson (1968) defined creep as any very slow permanent deformation of a slope regardless of the mechanism causing it, and distinguished types of creep as: (1) shallow, predominantly seasonal creep, (2) deep seated continuous creep which occurs in all soils and rocks subjected to shear stresses exceeding a critical value, and (3) progressive creep which occurs when the creep movement is approaching failure. Zaruba and Mencl (1969) defined creep as slow, long-term deformations of slopes which usually occur within a thick zone consisting of a system of partial sliding planes and possessing the character of a viscous movement. Also, creep has been defined as the geological long-term movements of nonincreasing velocity without well-defined sliding surfaces, classified as rock creep, talus creep, and soil creep (Nemcok et al., 1972).

Varnes (1978) classified creep as: (1) bedrock flow, and (2) soil creep; his creep definition has a meaning similar to that used in mechanics of materials, with deformation under constant stress. Some of the creep deformation may be recoverable over a period of time upon release of the stress, but generally most of it is not. He further pointed out that creep movements can occur in many kinds of topples, slides, spreads, and flows. Also, the term creep does not need to be restricted to slow, spatially continuous deformation. Radbruch-Hall (1979) used term "mass rock creep" to describe of deep-seated large gravitational creep of rock masses on slopes.

Above are the existing definitions of creep on slopes. For the application of a "creep" concept to long-term stability of overconsolidated clays and clayshales, the knowledge of the potential time-dependent behavior is essential. Due to difficulties in representing the continuum and in relating constitutive relationships determined from idealized laboratory tests and conditions to the actual site conditions, the analysis of time-dependent slope behavior is very complicated (Campanella and Vaid, 1974, Emery, 1979). Most research on creep behavior and the development of constitutive relationships for clays and clayshales are via laboratory creep tests, as shown in Figure 17a (Singh, 1966).

Generally, creep behavior of soils is studied under a constant deviator stress (σ_1-σ_3). For curve I, a very small shearing stress as characterized by D, is applied, and the soil is deformed instantaneously upon application of the load, followed by a state of increasing deformation at a decreasing rate and finally reaching a state of equilibrium. For curve II, a larger load is applied, the range of loads for which this behavior is observed being typical of many engineering problems. Curve II is characterized by four stages, as shown in Figure 17b. The boundaries of curve II are often quite arbitrary and a cohesive soil may not exhibit all of the stages represented (Singh and Mitchell, 1968, Campanella and Vaid, 1974, Emery, 1979). It can be interpreted as: (1) an instantaneous deflection after load application (stage I); followed another stage with a period of increasing deformation at a decreasing rate which has been called "primary" or "transient" creep (stage II); then followed a region of continuing deformation at an almost constant rate, this region has been called "secondary" creep (stage III); this nearly constant creep rate period eventually leads into stage IV, characterized by an increasing rate and subsquent failure. It appears that true steady state creep does not exist (Singh, 1966, Emery, 1979).

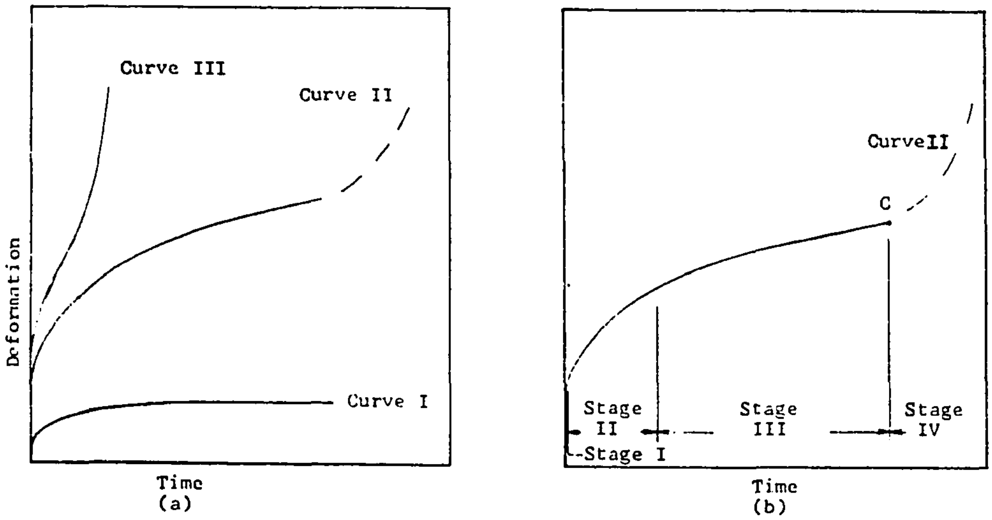

Figure 17. Typical creep behavior for cohesive soils observed in laboratory tests: (a) Typical creep curve, (b) Different stages of creep behavior (after Singh, 1966)

In order to study the creep of geological and other materials, three general approaches have been adopted: the physico-chemical approach, the empirical approach, and the rheological approach (Hirst and Mitchell, 1968, Emery, 1979). In the physico-chemical or micromechanistic approach, creep behavior is related to processes on the molecular scale, based upon experimental evidence that creep involves thermally activated processes, and absolute reaction rate theory or rate process theory has been used to study the creep of many materials. Rate process theory has been used to develop a strain rate equation for the creep of cohesive soil directly from considerations of micromechanistic behavior (Mitchell et al., 1968), which assume that the majority of creep experiments shows a behavior consistent with a thermally activated rate process. In soils, there are several studies on this subject (Christensen and Wu, 1964, Mitchell, 1964, Mitchell et al., 1968, Noble and Demirel, 1969, Andersland and Douglas, 1970, Erol et al., 1977). Typically, the process is that creep rates depend on the temperature by exponential factors $-\Delta H/KT$ and β, where ΔH is the activation enthalpy or bond energy, K is the Boltzman constant, T is absolute temperature, and β is a the stress factor. This strain rate equation may not be adequately developed for use in analysis procedures and the various parameters are difficult to determine with conventional laboratory equipment, but it provides valuable insight into the bonding mechanisms that contribute to shear resistance and creep movements (Hirst and Mitchell, 1968, Emery, 1979). Nelson and Thompson (1974) also point out that a physico-chemical approach may insure physical reasonableness of the creep laws utilized.

In the empirical approach, various parameters such as strain and strain rate are measured experimentally as a function of time, stress and temperature under controlled conditions (Emery, 1979). Singh and Mitchell (1968, 1969) have developed a very generalized stress-strain-time function for cohesive soils which is based on the study of creep curves for many cohesive soils over a range of sustained deviatoric stresses: $\varepsilon = Ae^{\alpha D}(t_1/t)^m$

where ε = the strain rate (axial or shear) at time t

D = deviatoric or shear stress

t_1 = unit time

t = time

A, α, and m = material constants

This equation is applicable irrespective of whether the clays are undisturbed or remolded, wet or dry, normally consolidated or overconsolidated, or tested drained or undrained. A minimum of two creep tests is needed to establish the values of A, α, and m for a soil. The parameter A indicates the order of magnitude of the creep rate for the particular cohesive soil, and reflects the structure, composition and stress history. The parameter α indicates the stress level effect on creep rate and may reflects the number of bonds per unit area resisting the creep movement (Mitchell et al., 1968). The parameter m provides a measure of the creep potential: $m < 1$ for soil with strain softening behavior; $m = 1$ for soil with same strength before and after failure and $m > 1$ for soil with strain hardening behavior. The value of m is not unique for a given cohesive soil and depends on the consolidation history (Singh and Mitchell, 1969). It is critical to have A, α, and m parameters developed for the appropriate soil conditions and stress history anticipated in the field (Campanella and Vaid, 1974).

For studying creep theoretically, the rheological model approach is perhaps the best known method (Singh and Mitchell, 1968, Emery, 1979). To represent loading of an actual material, an idealized model is made up of linear or nonlinear springs, dashpots and sliders. Most of the models have linear spring elements with an elastic modulus; and dashpot elements may be linear with a coefficient of viscosity, or nonlinear with the viscous flow obeying a hyperbolic law based on rate process theory or an exponential law based on empirical studies (Emery, 1979). It is now generally agreed that the creep of most soils is nonlinear so linear rheological models and superposition represent an ideal condition only (Hirst and Mitchell, 1968). However, Emery (1979) pointed out that linear rheological models still can provide useful approximations for the deformation and stress behavior to be examined qualitatively, and that this information is particularly valuable when laboratory test results are not available, and the material properties must be assumed or developed from field measurements. Among several rheological models, the Bingham flow model is appropriately suited to describe the relationship between shear stress and the rate of shear (Ter-Stepanian, 1963, Haefeli, 1965). A graphic representation of Bingham flow is shown in Figure 18. It can be observed that there is no flow until a given stress reach τ_0; after that there is a curve portion which leads to a linear portion similar to those of ideal plastic flow. A point (τ_B) called a yield stress will be intersected at shearing stress ordinate when the linear portion of the curve is extrapolated downward. Unlike the behavior of snow or ice, the creep of the soil begins only after a yield stress is exceeded (Haefeli, 1965). Two kinds of creep are classified with relation to this effect. If the shear stress is less than the yield stress, creep will produce a densification and consolidation of the material resulting in an increase of the strength of the soil. However, if the shear stress is larger than yield stress, creep will produce a gradual concentration of stresses and decrease the strength of the soil until it fails (Haefeli, 1965). Almost all of the creep relationships presented by the various investigators indicate that the creep rate is highly dependent upon the stress level in the soil or rock. The slope of the linear portion of the curve is usually denominated $1/\eta$, where η is interpreted as the viscosity of the soil according to rheological theory. This viscosity coefficient is important to identify the flow behavior of the different materials. It also influences the magnitude of the creeping velocity (Savage and Chleborad, 1982). Under the Bingham flow model, total deformation and the rate of deformation are increased with water content (Haefeli, 1953, Millan, 1969).

Nelson and Thompson (1977) have presented a theory of creep failure in overconsolidated clays, based on the interaction between the phenomena of creep, strain softening and time-dependent failures in such clays. They assumed that some critical strain exists at which point all of the internal bonds in the soil will have failed, and if a clay structure continues to creep when stressed below its residual strength, its peak strength continues to decrease with time. Based on Haefeli's creep concept (1965), they also derived an equation to determine the time until failure of a slope, and proposed that creep failure appears to extend from the crest of a slope of overconsolidated clay, which is opposite to the widely believed concept that failure starts from the toe area.

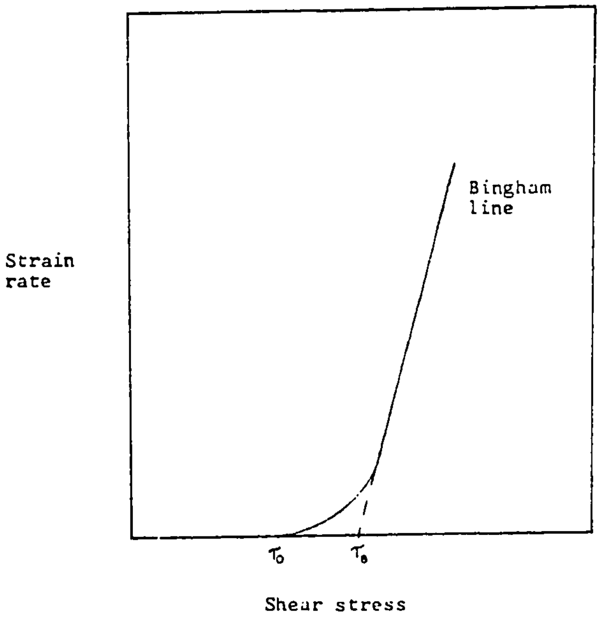

Figure 18. Bingham body behavior (after Haefeli, 1965)

Theoretical creep models and slope failure predictions

Seasonal creep is weaken with depth and the rate of movement is cumulative upwards, and vertical distribution is exponential with a maximum at the surface which may be used to explain the distribution of velocity with depth (Kirkby, 1967).

For continuous, deep-seated creep, numerous attempts have been made to fit with theoretical models. Although there are different approaches such as fundamental (micromechanistic), empirical, finite element, and rheological model approachs, most investigators have adopted some modification of the Bingham model as a rheological approach and used it to interpret the constitutive relationship (Singh and Mitchell, 1968, Emery, 1979, and Hungr, 1981). This is to consider that the soil will not have deformation while stressed to a shear stress level below yield stress, and then flow under high stress as a fluid of a constant viscosity. Several researchers assumed an infinite slope as the kinematic model to develope theoretical creep models (Ter-Stepanian, 1963, Yen, 1969).

These researchers proposed models of slow, steady creeping flow in landslides by considering the flow under gravity of a Bingham or viscoplastic substance based on the two dimensional Coulomb failure criterion. Ter-Stepanian (1963), and Yen (1969) analyzed and compared with actual field observations. Ter-Stepanian (1963) proposed a quantitative approach of deep creep movements in simple natural slopes by considering a zone of creep and its rates, in which yield stress τ_0 is considered as the threshold stress. He distinguished types of depth creep as planar and rotational depth creep. For planar depth creep which takes place in nonhomogeneous soil, the creep rate is a maximum at the contact with the rigid stratum. The rate of the planar creep of slopes does not remain constant, but changes with variations in intensity of processes which lead to landslides. Seasonal fluctuations of pore water presure and the gradual increase of slope inclination due to tectonic uplift may cause alternation of the rate of creep. For rotational depth creep, which takes place in homogeneous cohesive soils, he proposed that the creep zone is located in the middle of the slope, and the creep rate is a maximum in the middle of the slope. The rate of the rotational depth creep is maximum at the potential surface of sliding, and is influenced as the same factors as for planar depth creep. Ter-Stepanian's rotational depth model is opposite to the general belief that the initiation of the crack starts from the toe area. He applied his equations to a Caucasus coastal landslide, and pointed out that the profile of the downslope velocity is a parabola.

Yen (1969) analyzed the same case as Ter-Stepanian did, the Caucasus coastal landslide, using his theory and a soil viscosity value back calculated done by Ter-Stepanian. Although both studies are based on the same constitutive relationship, the results are mutually inconsistent. Yen's (1969) model is based on the residual shear strength and assumes that soil flows viscously with no volume change during slope creep, and the slope is an infinite layer of homogeneous soil. For the creep zone, he assumed that the location of the maximum velocity is inversely proportional to the unit weight of the soil and the cosine function of the slope, and that the depth of the maximum velocity will be increased as the normal surcharge load along the slope increased, and decreased as the shearing surcharge load increased. For the slope without surcharge and at residual condition, the location of the maximum velocity is at the surface of the slope. Theses concepts can not explain the initiation of crack and the failure plane which usually is located under the slope, while not in the slope surface.

Bishop and Lovenbury (1969) concluded from long duration triaxial creep tests that time effects alone are not sufficient to reduce the strength of the London clay from peak to residual values. James (1970) and Lefebvre (1981) suggested that there is no path to the residual by passing the peak strength for overconsolidated clays. By contrast, the peak strength for soft clays is reduced significantly with time, especially for tests with very low reconsolidation pressures (Lefebvre, 1981). Some "by-pass" of the peak strength may occur if there is movement along discontinuities within the slope, and under these circumstance c' may be considerably reduced (Skempton and Petley, 1967). However, if samples are sufficiently large to contain representative discontinuities, then the peak strength of the soil will not be affected. Meanwhile, based on Haefeli's concept (1965), Nelson and Thompson (1977) assumed that creep rate is constant with time, which can not reflect a real slope creep situation. They also proposed cracks initiating from the crest area based on finite element analysis, which is the opposite of the belief that cracks initiate from the toe area. The reason for this may be due to that pore water pressures and climatic effects are neglected in their analysis. Based on field observations in the Ottawa area, Mitchell and Eden (1972) proposed that creep rates are maximum at the toe of the slope, but the several models do not come out with these results (Ter-Stepanian, 1963, Yen, 1969, Nelson and Thompson, 1977).

Tavenas and Leroueil's concept (1981) of limit and critical state are best suited to represent the behavior of a wide variety of natural clays. In this concept, volumetric and shear creep strains in an overconsolidated clay develop at a rate which increases as the applied stress gets closer to the limit state surface, and the accumulation of creep strains with time results in an apparent displacement of the limit state surface and the reduction of the peak strength envelop with time. They further postulated that creep behavior and the rate of pore pressure dissipation may be assumed identical, and there is no fundamental difference between the failure of natural slopes and the delayed failure of cut slopes in fissured clays, as in intact clays. Although the model seems to fit field conditions, so far no quantitative approach is available. Strain energy as a yield and creep criterion as proposed by Tavenas et al. (1979) may not be suitable for practical application.

A three-dimensional Coulomb failure criterion for creep rate profiles of deep landslides was considered by Savage and Chleborad (1982). Their concept utilizes factors such as the elastic shear modulus, invariant of stress tensor, linear and nonlinear viscosity, cohesion and friction angle, and stress-strain relationship to develop constitutive equations. For a landslide model, the material, slope geometry and effects of pore water pressure are also considered. They proposed that for planar deep creep, the creep rate distribution is a parabola with a maximum in the yielding plane, not at the contact layer with a rigid plane, which is different from Ter-Stepanian's model (1963). This deep creep model is compared with the case histories described by Ter-Stepanian (1963), Yen (1969), and to Chleborad's field results (1980). However, the Bingham viscosity is the most difficult parameter to measure independently in order to apply the model for depth creep. Also influencing the magnitude of the velocity is the change of the water table and the transfer of horizontal load (Hungr, 1981, Savage and Chleborad, 1982).

Since Terzaghi (1950) pointed out that creep is the precedent of the landslide, many researches have been considered the possibility of predicting failure from prefailure creep movements. Besides the creep model proposed by Ter-Stepanian (1963), Yen (1969), and Savage and Chleborad (1982), others have tried to predict failure from pre-failure creep movements (Saito, 1965, 1969, 1980). Saito (1965) first proposed a method for predicting the time of occurrence of slope failure by means of steady state strain rate, and then later he extended his method of prediction to include the tertiary creep range. This method follows a characteristic pattern: (1) Slow or zero strain rates exist in a stable slope; (2) approaching failure is preceded by a sudden acceleration to a higher constant strain rate at some points; and (3) a new sudden acceleration brings about the failure. Saito correlated the accelerated pre-failure strain rate measured immediately after (2) and called the time period between (2) and (3) as "creep rupture life". However, sudden acceleration to a new strain rate generally does not appear in pre-failure records. Instead, the displacements and strains usually tend to increase gradually. The model may only apply to regional cases (Hungr, 1981).

Some difficulties still exist in the predictions such as: (1) the scale of acceleration required to cause failure, and (2) the ability to isolate fluctuations of displacement caused by external agents but not leading to failure (Hungr, 1981). The scale of pre-failure velocities varies a lot. Mitchell and Eden (1972) observed that toe displacement rates in Leda clay slopes exceeding 4 cm/month led to failure, while other creep rates were also observed (Skempton and Hutchinson, 1969). The actual pattern of creep movement may be complicated due to environmental factors, as in the record of horizontal displacements prior to the catastrophic Vajont rock slide that appears as a sequence of three S-shaped curves (Muller, 1964). The first two periods of acceleration were due to raises of reservoir level, and can not be classified as "failure" in comparison with the final displacement of 400 m. However, these acceleration may be superimposed and then accelerated the slope to failure.

Relationships between creep, progressive failure, and delayed failure

Since surficial creep is limited to a shallow layer, only deep creep will be discussed with regard to relationships of progressive failure and delayed failure. Terzaghi (1950) noted creep as the precedent of landslide; the creep rate is slow at creep stage and a sudden acceleration occurs at the failure stage. However, there are arguments about the mechanism behind this (Nelson and Thompson, 1977, Hungr, 1981, Morgenstern, 1985). The relationship between deep creep and progressive failure had been discussed (Haefeli, 1965, Ter-

Stepanian, 1975, Nelson and Thompson, 1974, 1977, Morgenstern, 1977). Both relate to time-dependent behavior of soils. Creep is one of the most widespread phenomena which describes the deviatoric and volumetric strain rate exhibited by soils under constant stress, mainly from gravity.

Progressive failure is the nonuniform mobilization of shear strength along a potential slip surface (Bishop, 1971, Morgenstern, 1977). It occurs first in the vicinity of the toe of the slope or where excessive deformations have occurred. These overstressed points will exceed a clay peak strength and then place additional stress at adjacent points, the chain reaction of these processes dropping strength down to the residual strength.

For creep, deformation is a result of sustained gravitational shear stresses and environmental factors such as changes of water table or tectonic movement, as well as excavation or cutting. There are several postulates in regard to creep rate: (1) creep rates decrease continuously with time, but vary exponentially with stress level (Goldstein et al., 1965, Bishop, 1967, Singh and Mitchell, 1968, Lohnes et al., 1972), (2) creep rates become essentially constant after a given time, but vary nonlinearly with stress (Haefeli, 1965, Nelson and Thompson, 1977), (3) creep rates depend on the rate of dissipation of the negative pore pressures after the end of excavation (Tavenas and Leroueil, 1981), and (4) deep creep on slopes should not be treated as a continuous process, nor does it proceed uniformly, being subject to seasonal changes caused by fluctuations or periodical acclerations due to drawdown of water table. The conclusions of (1) and (2) above are all based on the laboratory tests, and do not include pore water pressure, while (3) and (4) above are better suited to interpret the creep rates of landslide practically. For progressive failure, most of the researchers concerned the changes in shear strength due to decrease of the parameters c or ϕ, or the change of pore water pressure. However, shear band model did consider the rate of propagation (Rice and Simons, 1976).

For creep zone, it is postulated that creep deformation is a continuous gradation between the stationary and the moving material, and is unassociated with the presence of a slip surface, but this still remains problematical (Skempton and Hutchinson, 1969, Hungr, 1981, Morgenstern, 1985). Hungr (1981) reviewed available evidence and concluded that at depth, only a decaying creep has been measured in response to specific equilibrium changes, and steady movements occur only in failure-generated zones or in material that is approaching failure. He further pointed out that some rheological models which utilize back-analysis of sustained time-dependent slow slope movement are not productive, the reason being that the rheological model does not consider the horizontal load transfer between adjacent sections of the moving mass. Morgenstern (1985) reported that no successful forecasts appear to have been made based on laboratory creep test data or on back-calculated values, with the exception of one model (Savage and Chleborad, 1982) which utilized a three dimensional Coulomb failure criterion and fit in field results (Chleborad, 1980), and some case histories (Ter-Stepanian 1963, Yen, 1969). In natural, evidences exists for creep phenomena, that may coexist with other failure mechanisms such as progressive failure. The creep cases used by Ter-Stepanian (1980), at the bridges over the Little Smoky river in Canada and the Landquart river in Switzerland, are due to river erosion at the toe area, and may be interpreted as progressive failures. Also, there is a single continuous shear zone in the Little Smoky river case (Thompson and Hayley, 1975), which is not consistent with some creep models.

For progressive failure, Skempton (1964) proposed that fissures in the slope act as stress concentrators. Strain softening by dilatancy and the opening of fissures will drop the strength to a fully-softened stage with discontinuous shears. If there is further large deformation, at this stage particle re-orientation will have occurred, and a continuous principal shear or principal slip surface will form as the strength falls to residual. A width of principal slip surfaces ranging from 2 cm to 20 cm has been observed (Morgenstern and Tchalenko, 1967, Skempton and Petley, 1967). In most of landslide cases, failure without a principal slip surface is not a general phenomenon.

For the initiation of cracking, there are three postulations about the start of the creep deformation, (1) deformation starts from the zone beneath the middle of the slope (Ter-Stepanian, 1980), (2) deformation starts from the crest area (Nelson and Thompson, 1977), and (3) a crack theoretically initiates from the toe area after excavation of overconsolidated clays (Bjerrum, 1967, Duncan and Dunlop, 1969, James, 1970, Bishop, 1971, Lo and Lee, 1973). Meanwhile, based on field inclinometer observations, cracks may progress predominantly from toe towards the crest of the slope (de Beer, 1967, 1969, Mitchell and Eden, 1972). Field observations from Ter-Stepanian (1984) also showed that creep crack initiated from the toe of the slope due to road cutting.

Strength parameters close to the residual strength are used in rheological models by creep researchers (Yen, 1969, Lohnes et al., 1972, Nelson and Thompson, 1974, 1977). These strength parameters will not be able to explain the first-time slide through back-analysis. First-time slides may occur due to negative pore pressure equilibration or softening, wherein the rate of strength lose is not constant. However, a creep model based on laboratory tests cannot simulate these field situations, a restriction that limits the rheological effect as used in a creep model.

As mentioned previously, a creep model can not explain the first-time slide through back-analysis. Nelson and Thompson (1974, 1977) used some failure cases of London clay to justify their strain-softening creep model, and found that failure stress is higher than residual strength. However, as Skempton (1977) pointed out, these failure cases are all first-time slides. From a geological stand point, first-time slides likely occurs mostly in homogeneous clay; while progressive failure likely occurs in a reactivated slide, mostly a nonhomogeneous soil, or along a geological discontinuity in the slope.

Creep is a common phenomenon, for surficial creep can occur in places resulting from seasonal processes. Deep creep can occur in some slopes as precedent for a landslide, and the creep rate may be so slow as to be imperceptible. When environmental factors change, such as a rise of water table, natural erosion, river downcutting, or human activity such as loading on the slope or excavation, creep may accelerate to a landslide. In this way creep and progressive failure will have same meanings with regard to their processes: strength lose with time, and a rate subject to the environmental factors.

The relationships among delayed failure, progressive failure, and creep thus can be focussed on as a difference in driving force, deformation, deformation rate, sliding zone, initiation of crack, geological soil conditions, and strength parameters. Table 5 lists factors related to the different failure mechanisms.

Table 5. Relationships between delayed failure, progressive failure, and creep

	Delayed Failure	Progressive Failure	Creep
Driving Force	Negative pore water pressure equilibration or softening	Various environmental factors	Gravity
Deformation	Small	Large	Small to Large
Deformation Rate	Various	Various	Decreasing, constant, or variable with time
Sliding Zone	Yes	Yes	No
Initiation of Crack	Toe	Toe	Crest or central part of the slope
Slide Type	First-time slide	Reactivated slide	Various
Geological Soil Condition	Mostly homogeneous	Mostly nonhomogeneous or layered	Various
Strength Parameters	Fully softened strength	Residual strength	Residual strength

PART II: EFFECT OF LATERAL STRESS ON SLOPE STABILITY ANALYSIS

At Rest Lateral Stress (K_0) with Stability of Stiff Overconsolidated Clays and Clayshales

Introduction

A knowledge of *in situ* or initial stresses has been of interest to geotechnical engineers and engineering geologists for a long time, it has been recognized qualitatively for many decades that *in situ* stresses are important for analysis and design of strutted excavations, tunnels, underground openings, foundation bearing capacity and settlement, slope, retaining wall, pile, liquefaction potential, and etc. (Chowdhury, 1978, Schmertmann, 1985). The availability of a powerful and versatile numerical technique such as the finite element analysis for performing studies of stress deformation has led to a better understanding of the importance of initial ground stresses qualitatively and quantitatively.

The vertical and horizontal effective stresses at any depth z beneath a level ground surface are:

$\sigma_{v'} = \Upsilon z - u_0$

$\sigma_{h'} = K_0 * \sigma_{v'}$

where Υ = the average density of the overlying material

u_0 = the pore water pressure at the point considered

K_0 = the coefficient of lateral stress at rest

For normally consolidated soils K_0 lies between the limits 0.3 and 0.8 (Terzaghi, 1925, Bishop, 1958), an approximate indication of the value of K_0 for such materials may be obtained from the expression given by Jaky (1944):

$K_0 = 1 - \sin \phi$

in which ϕ is the effective angle of shearing resistance. Based on a detailed experimental study, Brooker and Ireland (1965) proposed that for normally consolidated clay:

$K_0 = 0.95 - \sin \phi$

These two equations (Jaky, 1944, Brooker and Ireland, 1965) are almost identical for practical purpose. Many values of the coefficient of lateral stress at rest are done in laboratory test which is valuable in understanding the development of lateral stresses during loading, unloading and reloading of a specimen of soil under conditions of no lateral strain. These measurements can't simulate other varied natural factors which influence *in situ* stresses such as soil structure, cementation between particles, weathering and secondary time effects associated with loading and unloading (Wroth, 1975).

Kjellman (1936) conducted laboratory tests on sand and concluded that the value of K_0 increased with increasing overconsolidation ratio. Brooker and Ireland (1965) found that the value of K_0 was governed by the stress history and the drained angle of shearing resistance. For remolded clays with high values of overconsolidation ratio, they measured values of K_0 as high as 3. Mayne and Kulhawy (1982) reviewed laboratory data from over 170 different soils and proposed simple empirical methods for predicting K_0 value for normally consolidated and overconsolidated soils. Their findings confirmed Brooker and Irelands' proposition that the value of K_0 was influenced by the stress history and the drained angle of shearing resistance.

Due to the difficulties in laboratory tests to simulate the field test, the actual field measurement of K_0 is becoming more attractive with a variety of recent field techniques which include hydraulic fracturing, Menard Pressuremeter, Self-Boring Pressuremeter, Gloetzl total stress cells, Marchetti Dilatometer, and K_0 Stepped Blade (Bjerrum et al., 1972, Wroth and Hughes, 1973, Tavenas et al., 1975, Massarsch and Broms, 1976, Handy et al., 1982). All of these afford direct methods for measuring *in situ* soil stresses. A major disadvantage of these methods is that some disturbance will occur that will induce error. The K_0 Stepped Blade therefore was devised to minimize the disturbance effect by extrapolating, stresses to their pre-insertion *in situ* condition (Schmertmann, 1985).

For a cohesionless material the coefficient of lateral stress at rest, K_0, is bounded on the low side by the coefficient of active Rankine state of stress, K_a, and on the high side by the coefficient of passive Rankine state of stress, K_p (Lambe and Whitman, 1969). The active ratio, K_a, is less than one, the passive ratio, K_p, is greater than one. The K_0 of a soil may have any value between K_a and K_p, depending on its stress history. For overconsolidated soils, it was found that values of K_0 may be greater than one (Kjellman, 1936, Skempton, 1961). It is now well known that the value of K_0 for overconsolidated clays may approach the passive failure at locations where the overconsolidation ratio is sufficiently high (Chowdhury, 1978).

At rest lateral stress related to slope stability

Smith and Redlinger (1953) described how a 3 inch wide cut in the Fort Union shale closed in about 24 hours. Palladino and Peck (1972) investigated slope failures of overconsolidated clays at Seattle. Due to high initial stress within the soil mass, construction methods utilized must minimize disturbance of the soil mass and provide the confinement against release of lateral stress with corresponding lateral deformation.

Muller (1977) stated that the sliding mass was lying on a rock body under high initial stress, and this mass was jerked off like an arrow from a bow after Vajoint slide. In recent years, experiences gained at soil design section, Iowa Department of Transportation also shown that it is very effective to remedy slope failure in overconsolidated clays and clayshales using trench drains. If a trench is left open overnight, due to stress relief the slope will creep down and fail.

The experiences stated above show the influence of initial stress in the stability of overconsolidated clays and clayshales. The origin of such initial stress may be of tectonic origin or may result from a history of deposition, erosion of overburden, slope formation, the diagenetic swelling of minerals in the rock and probably a variety of other causes (Chowdhury, 1978, Schmertmann, 1985). There is wide evidence that high *in situ* lateral stress still exist in overconsolidated clays and clayshales based either on geological investigations or on field measurements (Dodd and Anderson, 1972).

Clay deposits may become unloaded as overburden is removed by erosion, or by removal of glacial ice (Bjerrum, 1967). Due to such unloading, considerable strain energy will be stored in the clays, a stored strain energy that subsequently will be released if the bonds are destroyed as a result of weathering. This phenomenon is consistent with Brooker and Ireland's results (1965) as shown in Figure 14, which indicate that the value of K_0 decreases with increasing value of plasticity index for the most heavily overconsolidated highly plastic clays (Duncan and Dunlop, 1969). Brooker (1968) has shown that the amount of strain energy stored after rebound for the clays tested increased with increasing plasticity index. The weathering release of this stored bending energy creates high K_0 conditions if lateral movement is restrained. If deformation occurs due to excavation, then there may be a full or partial release of the high K_0. For highway cutting, the response of unloading is rapid, the stored strain energy of overconsolidated clays and clayshales tends to swell in large amount. Under this circumstance, the slope will be less stable than if it was formed slowly in nature.

A high K_0 condition will cause progressive movements which can result in reaching residual strength conditions and possibly progressive failures (Bjerrum, 1967). Based on finite element method studies of cut slopes, doubling K_0 value will produce a doubling of the maximum shear stress, with lateral stresses highest in the toe area (Duncan and Dunlop, 1969). Dunlop and Duncan (1970) showed that a high K_0 condition causes the progressive failure for a cut to begin at the toe and progress upwards, while with a low K_0 condition the failure starts at the crest and progress downwards to the toe. High K_0 condition could cause the potential failure surface to move much deeper into the slope than with a low K_0 condition as shown in Figure 19 (Brown and King, 1966, Lo and Lee, 1973). The shear stress level λ is defined as the ratio of shear stress to the peak strength or residual strength. Where λ equals 1 is a region where the strength has decreased to the residual state. By using finite element method to model strain softening behavior of the clays and the nonuniform stress-strain distribution in frictionless soil, Lo and Lee (1973) found that increasing K_0 value from 1 to 2 decreased the factor of safety from 1.45 to 1.16. The important effect of K_0 on the factor of safety is evident.

The K_0 value inside a slope is not homogeneous. Haimson (1973) reported anisotropic K_x/K_y ratios of 1.5 to 3.2. Dalton and Hawkins (1982) reported measuring an anisotropic K_0 ratio of 1.8 using Self-Boring Pressuremeter testing. The reasons for anisotropic K_0 ratio are: (1) the movement direction of the glaciers, (2) geometry of cutting or erosion, (3) seepage, (4) directional roller compaction (Schmertmann, 1985).

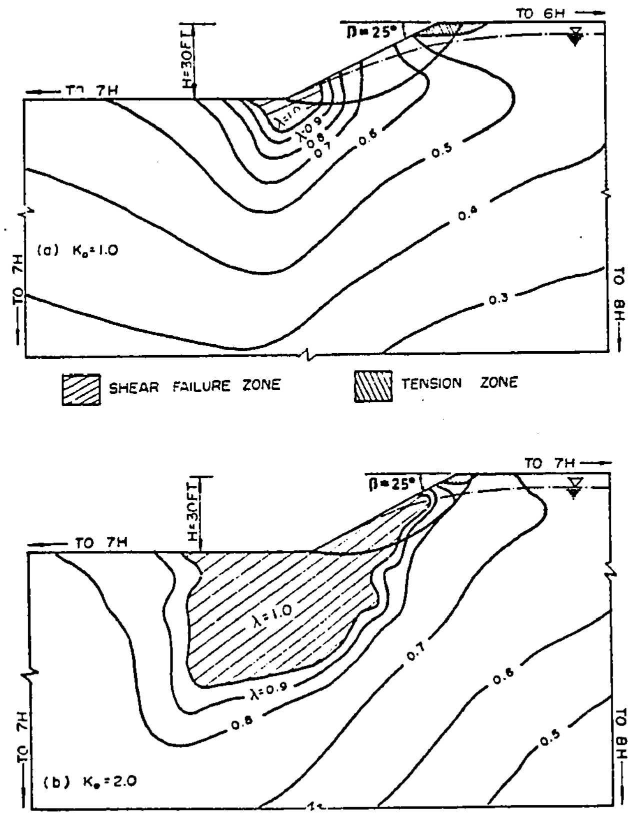

Figure 19. Stress distribution with various K_0 value (after Lo and Lee, 1973)

Stability Analysis

Introduction

In the stability analysis of slopes, there are two basic lines of approach which are popular to engineers and researchers. The first is the limit equilibrium approach, and the second is the stress-strain analysis. The stress-strain analysis is done by finite element technique, such that given the material properties and the cross section of the slope, the deformation and factor of safety can be computed. However, due to unknown characteristic of the soil, it is hard to get accurate input data so the results from these stress-strain analysis are sometimes questionable. They nevertheless are useful for a better understanding of the qualitative soil behavior of the slope under loading.

Limit equilibrium analysis assumes that at the moment of incipient failure, soil elements of a surface in the slope reached the Mohr-Coulomb failure strength simultaneously, and the free body contained within the slip surface and the free ground surface is in static equilibrium (Sarma, 1979). Although limit equilibrium cannot describe the deformation, it is able to produce comparable results as regards to the safety of the structure. The assumed failure surface can be of various shapes: planar, circular, or log spiral, and the factor of safety usually is incorrectly assumed to be the same at all the points along the failure surface (Chen 1975). The factor of safety is defined either as (1) a ratio of moments due to resisting to disturbing forces or as (2) a ratio of available to mobilized unit shear strength at any point along the slip surface.

Methods and conditions analyzed

In the methods, moment equilibrium and/or force equilibrium are satisfied. Some of the methods in the limit equilibrium approach are: Sliding Block, Ordinary Method of Slices (Fellenius Method), Friction Circle Method, Janbu's Method, Simplified Bishop's Method, Morgenstrern and Price's Method, and Spencer's Method. Certain assumptions are made in different methods, such as interslice forces are ignored in Sliding Block Method, Ordinary Method of Slice, and Friction Circle Method. The

Simplified Bishop's Method satisfies vertical equilibrium for each slice and overall moment equilibrium, but does not satisfy horizontal equilibrium or moment equlibrium for each slice. Janbu's, Morgenstern and Price's, and Spencer's Method satisfy overall, and interslice moment equilibrium, vertical and horizontal force equilibrium. Duncan and Wright (1980) summarized the characteristics of different equilibrium methods as Table 6. When it comes to the choice of the method, Fredlund and Krahn (1977) reported that the Spencer's and the Morgenstern and Prices's Methods are at least six times as costly to run as Simplified Bishop's or Janbu's Methods. Right now commercial available programs mostly are programmed with Simplified Bishop's or Janbu's Methods. Due to its simplicity in hand calculation, the Ordinary Method of Slices still is frequently used but under some circumstances, if used for effective stress analyses of slopes with high pore pressures, may give values of factor of safety 50% smaller than the correct value (Duncan and Wright, 1980).

All the methods discussed above are based on limit equilibrium analysis which assumes that at the moment of incipient failure, soil elements of sliding surface reaches the Mohr-Coulomb failure strength simultaneously, which is appropriate for an ideally plastic soil that exhibits no volume change. This may not be true for overconsolidated clays and clayshales which are brittle materials, and their K_0 values along the slope are not constant. After excavation or weathering, K_0 value is high in toe area and decreases upwards. It is believed that

Table 6. Characteristics of equilibrium methods (after Duncan and Wright, 1980)

Procedure	Equilibrium Conditions Satisfied				Shape of Slip Surface	Practical for	
	Overall Moment	Ind. Slice Moment	Vert. Force	Horiz. Force		Hand Calc.	Computer Calc.
Ordinary Method of Slices	Yes	No	No	No	Circular	Yes	Yes
Bishop's Modified Method	Yes	No	Yes	No	Circular	Yes	Yes
Janbu's Generalized Procedure of Slices	Yes	Yes	Yes	Yes	Any	Yes	Yes
Morgenstern and Price's Method	Yes	Yes	Yes	Yes	Any	No	Yes
Spencer's Method	Yes	Yes	Yes	Yes	Any	No	Yes
Force Equilibrium	No	No	Yes	Yes	Any	Yes	Yes
Log Spiral	Yes	--	Yes	Yes	Log Spiral	Yes	Yes

the crack initiates upwards from toe. Once this kind of local failure takes place, the strength of the soil will drop, which initiates a redistribution of interslice forces and lead to some further local failure (Law and Lumb, 1978). The methods of limit equilibrium analysis have failed to take this fact into their development.

Numerous methods of slices have been proposed based on differing assumptions regarding the interslice forces related to the direction, magnitude, or point of application (Fredlund and Krahn, 1977, Wilson and Fredlund, 1983, Fan, 1983, Fredlund, 1984, Fan and Wilson, 1986). The magnitude of the interslice forces on each slice was set as zero (Fellenius, 1936, Janbu, 1954, Bishop, 1955), or the interslice resultant forces was set to a constant angle (Spencer, 1967). The U.S. Army Corps of Engineers assumed that the direction of the interslice force was either (1) parallel to the ground surface, or (2) equal to the average slope from the beginning to the end of the slip surface. Morgenstern and Price (1965) allowed any arbitrarily defined function to be used to define the direction of the resultant interslice forces. However, several researchers postulated that these interslice forces only have an insignificant variations on the computed factor of safety (Bishop, 1955, Morgenstern and Price, 1965, Fredlund, 1984, Fan and Wilson, 1986).

A general and empirical interslice force function was developed through a detailed study on interslice force functions computed from finite element analysis (Wilson and Fredlund, 1983, Fan, 1983). It was proposed that the interslice force function for simple homogeneous slopes with circular slip surfaces are bell-shaped, with the maximum occurs at mid-slope. The interslice force at the crest and the toe area are the same. From their results, the K_0 value is high at the crest and the toe, and lowest in the mid-slope. This can also exemplify that cracks of potential shear may initiate from either the toe or crest. For the case of a cohesive soil, all methods satisfying moment equilibrium (or moment and force equilibrium) give the same results. For the case of a semi-infinite slope, same factor of safety is suggested when the soil is cohesionless. However, the factor of safety for methods satisfying force equilibrium depend on the interslice force function (Fan and Wilson, 1986).

Methods of predicting failure

The history of control of many big, old and persistent landslides is measured by decades, and lots of money is spent annually to repair them. It seems that "the landslide devil" laughs at people and their vain efforts to manage him (Ter-Stepanian, 1984). Can we predict the landslide and remedy it before it totally fail? So far it is problematical to actually forecast the slope movement. The usual methods for landslide forecasting are based on the following possibilities:

(A) Extrapolation from on-site displacement measurements. Landslides involve movement and the magnitude, rate and distribution of this movement are generally the most important measurements required. Most of the prediction methods are based on the creep model which assumes that landslides are slow, steady creeping flow under gravity of a Bingham or viscoplastic substance on an infinitely long slope (Ter-Stepanian, 1963, Yen, 1969, Saito, 1965, 1969, 1980, Savage and Chleborad, 1982). The different mechanisms are discussed in Part I. The main difficulties in such predictions are firstly to recognize the scale of accleration required to cause failure, and secondly to be able to isolate fluctuations of displacement caused by external agents but not leading to failure (Hungr, 1981). The various kinds of measurement instruments are available such as inclinometer, tiltmeter, extensometer, and etc. The location of the sliding plane is shown by measured deformation-depth relationships, which is important for engineers when considering the remedial methods.

(B) Landslide forecasting from pore water pressure measurements. An increase in pore water pressure and corresponding loss of shear strength is a recognized major factor in landslides. The role of pore water pressure in the equilibrium of slopes has long been recognized, it can elucidate the mechanism of short and long-term stability as well as establish relationships between the distribution of pore water pressure versus depth and the slip surface as shown in Figure 20 (Skempton, 1964, Lefebvre, 1981, Hutchinson, 1982). Structured clay of Champlain clay of eastern Canada behaves like highly overconsolidated London clay in that both materials are highly strain softening, and their strength decreases rapidly after failure (Lefebvre, 1981). However, when the structure of the clay collapse under shear a tendency to swell is observed in London clay, while eastern Canadian clays tend to compress. There are also many examples of measurements estabilishing the relationship between variations in pore water pressure and factor of safety as shown in Table 7 (Skempton, 1977, Law, 1980, Lefebvre, 1981). It has been shown that the average pore water pressure parameter is influenced by the clay structure and consolidation state. When the consolidated stage is high and sensitivity is low, the value of $\overline{T}_u$ at failure will be less.

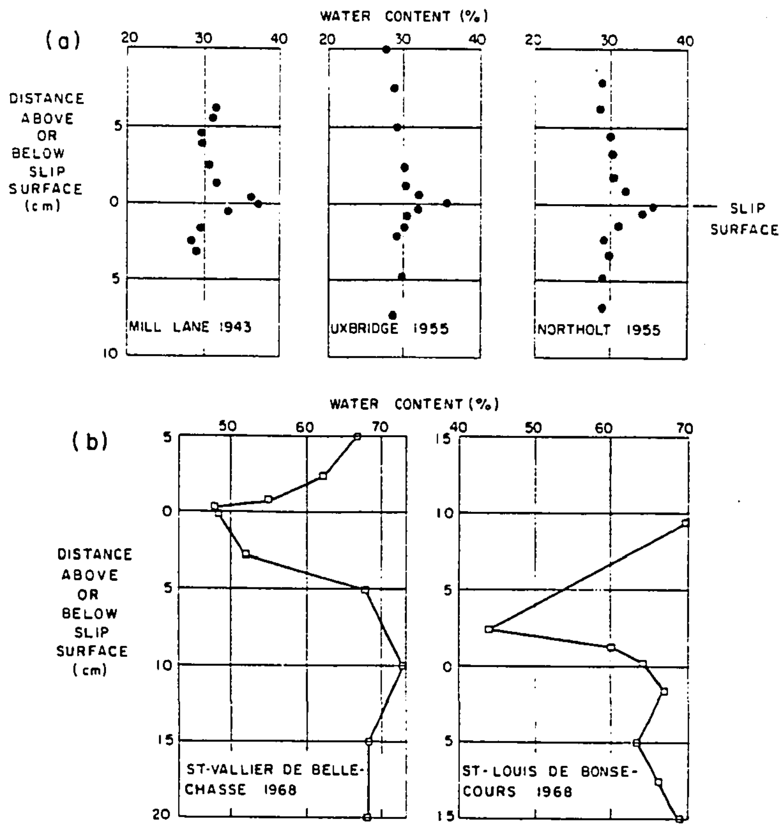

Figure 20. Variation of water content with slip surface: (a) in London clay, and (b) in Champlain clay (after Skempton, 1964, Lefebvre, 1981)

Table 7. Relationship between pore water pressure and factor of safety at failure for London clay and Champlain clay (adapted from Skempton, 1977, Law, 1980, Lefebvre, 1981)

Clay Tpye	Consolidation State	Water Content	Average Pore Water Pressure Parameter, $\overline{r}_u$
London clay	Insensitive, stiff, highly overconsolidated	≈Plastic Limit	0.25-0.35
Champlain clay	Sensitive, soft, lightly overconsolidated	≈Liquid Limit	0.44-0.49

The pore water pressure field as a whole is characterized by the average value of parameter $\overline{r}_u$. However, the value of pore water pressure will be influenced due to geological restrictions such as negative pore pressures still existing in the unsaturated wedges of multiple seepage faces, even with the water table is at the ground surface. The pore water pressure parameter is mostly used in back-analysis to calculate factors of safety on which the stability evaluation will be based. It is a useful parameter to understand the landslide mechanism, especially for first-time slide.

In order to measure the pore water pressure, many types of piezometers have been developed, which include twin-tube hydraulic piezometers, Casagrande piezometer, pneumatic piezometer, electric piezometer, etc. For measurement of pore water pressure in clays or clayshales, in which permeabilities are low or suctions may be present because of unloading, high-air-entry, low-flow piezometers should be used (Vaughan and Walbancke, 1973).

(C) Landslide forecasting from rainfall data. It is possible to predict relationships between records of rainfall, leading to higher pore pressure and the landslides. These relationships are hard to establish because complex phenomena are involved such as: (1) the analysis of the water balance which include the amount of water received at the site among evapotranspiration, runoff, and infiltration, (2) the relationships between the rainfall spectrum and variations in pore pressure (Pilot, 1984). Although some researches have been done in finding a relationship between rainfall and the triggering of landslides, it appears that pattern has some regional limitation. So far no comprehensive relationship exists, especially for overconsolidated clays and clayshales.

(D) Landslide probability maps. These are prepared based on the occurrences of landslides shown on airphotos and ground maps. Compared airphotos with time, the phenomena change of land forms may indicate the susceptibility of landslide. Ground maps can be used with a grid overlay, and the map units are arbitrarily grouped in several classes. The relative susceptibility to landslide is expressed from very low (form I) to very high (form VI). Engineers therefore can evaluate the area of landslide deposits on the average failure record of ground maps (Nilsen and Brabb, 1977).

Field Tests

Introduction

Field tests were conducted in order to ascertain slope stability using the Borehole Shear Test and K_0 Stepped Blade Test. Tests were performed at three different sites designated as Indianola, Osceola, and Guthrie.

The Borehole Shear Test is a device for determining *in situ* cohesion and internal friction of soil and rock, under essentially drained conditions. A diagram of the test apparatus is shown in Figure 21. The test entails performing a series of direct shear-type tests on the inside of a borehole (Handy and Fox, 1967, Spangler and Handy, 1982). This is accomplished by applying a normal force to the sides of the borehole through two opposing and serrated steel plates. After allowing sufficient time for consolidation to occur under the applied normal force, the shear phase of the test is initiated by pulling the shear plates verticaly upward to induce shear. As in conventional direct shear testing, a series of tests with increasing applied normal stress (σ_n) is performed, while the corresponding shear stress (τ) is measured. A difference from the laboratory procedure is that the test is repeated at the same spot, as a stage test (Schmertmann, 1975). Results are plotted to define the Mohr-Coulomb failure envelope (Lutenegger and Hallberg, 1981).

During testing the shear head is lowered to the desired test depth and the plates are expanded against the hole by CO_2 to a known normal stress. The pressure regulator is used to maintain a constant normal stress. Testing procedures recommended by Lutenegger and Hallberg (1981) and currently in preparation as an ASTM standard method are followed.

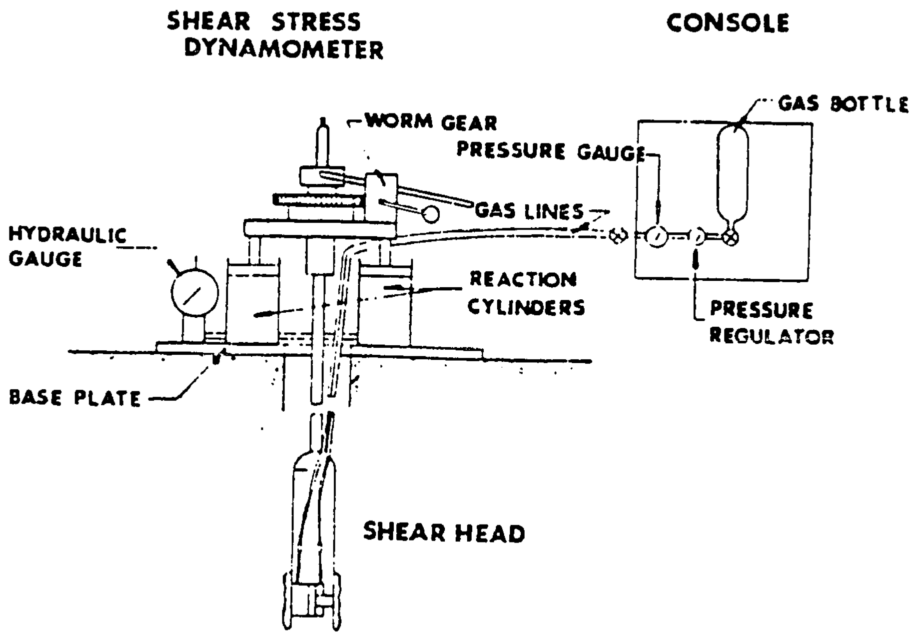

Figure 21. Borehole Shear Test components

In Indianola and Osceola sites, five Borehole Shear Tests were performed. The tests were conducted in holes drilled with a hand auger which cut a hole approximately 3 inches in diameter.

The K_0 Stepped Blade Test was first presented by Handy et al. (1982) as a technique for determining the *in situ* lateral stress in soils. By forcing a stepwise flat-plate penetrometer into the soil and measuring the stress acting on the face of each of the steps, the *in situ* stress state is obtained by extrapolating the data to a zero blade thickness.

Based on a series of controlled laboratory tests using compacted soil, Handy et al. (1982) suggested that the initial stress condition could be described by a simple expression as:

$$p_0 = a * p_1 * e^{-bt}$$

where p_0 = *in situ* stress

p_1 = pressure on a blade of thickness t

a = coefficient (assumed to equal to 1.0)

b = coefficient

Thus, a plot of blade thickness versus logarithm of measured pressure would be linear with slope b and intercept log p_0.

Two K_0 Stepped Blade Tests were conducted at both Indianola and Osceola sites, one test located within the slide area and the other outside of the slide area. The tests were conducted by using a rig of the Iowa Department of Transportation. In the last test the blade was bent after being pushed into what most likely was buried boulders. At a third site the Guthrie slide, only qualitative data were obtained. In all tests the blade was facing the slide zone, so the lateral stress acting on the slide could be measured. The materials tested were highway foreslope compacted fills.

Indianola

(A) Introduction

The test site designated as Indianola is a foreslope at 2:1 slope, compacted silty clay fill. The location of the site is approximately 3-4 miles south of Indianola on Iowa highway 69, Warren county. The road was first constructed in 1945. A pond is located on the east side of the roadway, and a 48 inch underground culvert crosses the roadway at an angle of 30^0 with centerline. The slide first occurred approximately 15 years ago. When the culvert was believed to have been partially blocked, such that water in the pond leaked and saturated the soil, weakening it and making slope unstable (Kirmit Dirks, Iowa Department of Transportation, personal communication). The maintenance forces of the Iowa D.O.T. then cleaned out the culvert and reshaped the slope. As time passed, the culvert was blocked again, and the slide reactivated, and free water seeped out near the toe of the slope. Surface ponded water could be observed in several places. A sequence of scarps about 2-3 ft high and tension cracks were observed in the slope, and the toe slope slid away.

The first K_0 Stepped Blade Test was conducted in the slide zone at Sta. 384+65, Lt. 15 ft, which was underlain by 31 ft of firm gray brown silty clay fill, that was underlain by firm gray brown sandy silty clay glacial till to 37 ft, which was the natural ground soil. Underneath this was gray brown sandstone. The water table was observed at 31.6 ft 24 hours after drilling. A cross section is shown in Figure 22. Second K_0 Stepped Blade Test was conducted at Sta. 385+22, Lt. 15

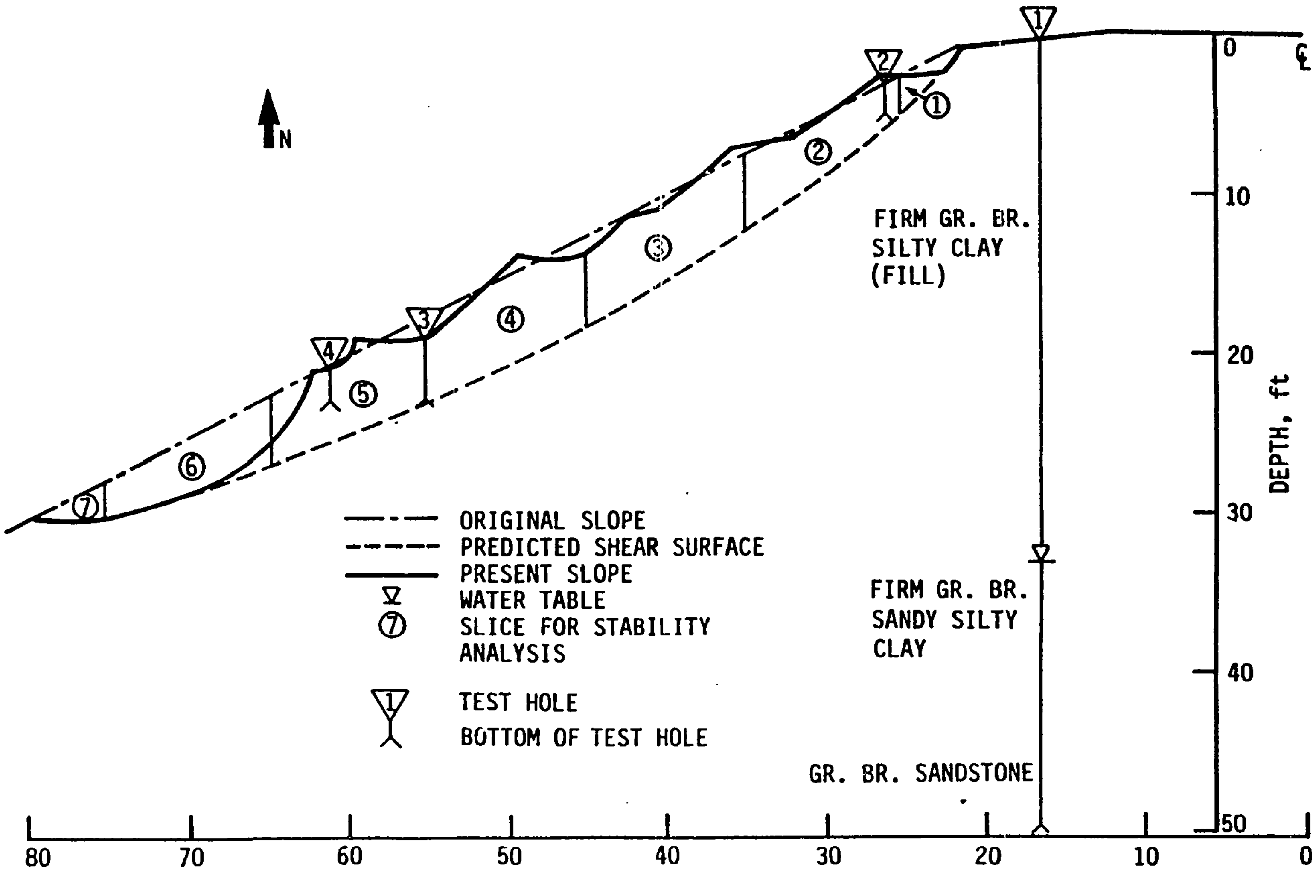

Figure 22. Cross section of Indianola slide

ft which by visual observation was considered outside the sliding area. The fill is firm gray brown silty clay to 25 ft, underlain by firm gray brown sandy silty clay, with occasional boulders to 31 ft as the natural glacial till, followed by layer of gray brown clayey sand to 42 ft, then sandstone. The water table was observed at 22.3 ft 24 hours after drilling. After hitting buried boulder, the blade was bent at 20 ft depth and testing was terminated. The fill has a total wet density of 125 pcf at a moisture content of 26.6%. The liquid limit is 34, and the plastic limit is 18.

Borehole Shear Tests were conducted in the slide zone, the first about 60 ft down from the crest in a hand augered hole. A second hole was drilled about 10 ft farther down slope from the first hole, but was too muddy to test. A third hole was abandoned after hitting rubble.

(B) Results

The results of Borehole Shear Tests are shown in Table 8. The factor of safety is 1.03 calculated by the Ordinary Method of Slices using a cohesion of 0.58 psi and friction angle of 22.9^0. Since no excessive pore water pressure existed in this case, it is appropriate to use the Ordinary Method of Slices for stability analysis (Duncan and Wright, 1980). Based on stability analysis and field observations, it is believed that the sliding surface is located at 4 ft depth at Boring 3. The shear strength parameters meet the previous discussion in part I that cohesion will be lost after a landslide. However, if cohesion is zero, the calculated factor of safety will drop to 0.69 which is too

Table 8. Borehole Shear Test results at Indianola slide

Boring	Depth (ft)	Soil Type	c(psi)	ϕ	r^2
3	3.0	Silty Clay Fill	0.57	24.5^o	0.98
3	4.0	Silty Clay Fill	0.58	22.9^o	0.98

low. These measured Borehole Shear Test strengths therefore are residual strengths.

The results of the K_0 Stepped Blade Tests are shown in Figure 23. Horizontal stresses are corrected to K_0 values by dividing by the overburden stresses at each depth, as shown in Figure 24. K_0 values are calculated on an effective stress basis. Only one test (35 ft) is affected by pore water pressure. Several high K_0 values that are consistent at different depths may be related to the methods of compaction. At shallow depth (5 ft), the K_0 value is high due to compaction coupled with a small overburden weight. At zero depth, the overconsolidated K_0 value theoretically should reach infinity.

By comparing the sliding and nonsliding profiles, a higher K_0 exists in the nonsliding area at 5 ft depth, below which the two profiles show similar trends. It can be interpreted that the initial stress conditions at two sites were very close, as the data are repeatable. The trend of the K_0 profiles and the magnitude of the K_0 values detected in both sliding and nonsliding areas are different from those of elastic theory (Clough and Woodward, 1967, Perloff et al., 1967, Poulos and Davis, 1974), as the maximum K_0 value from elastic

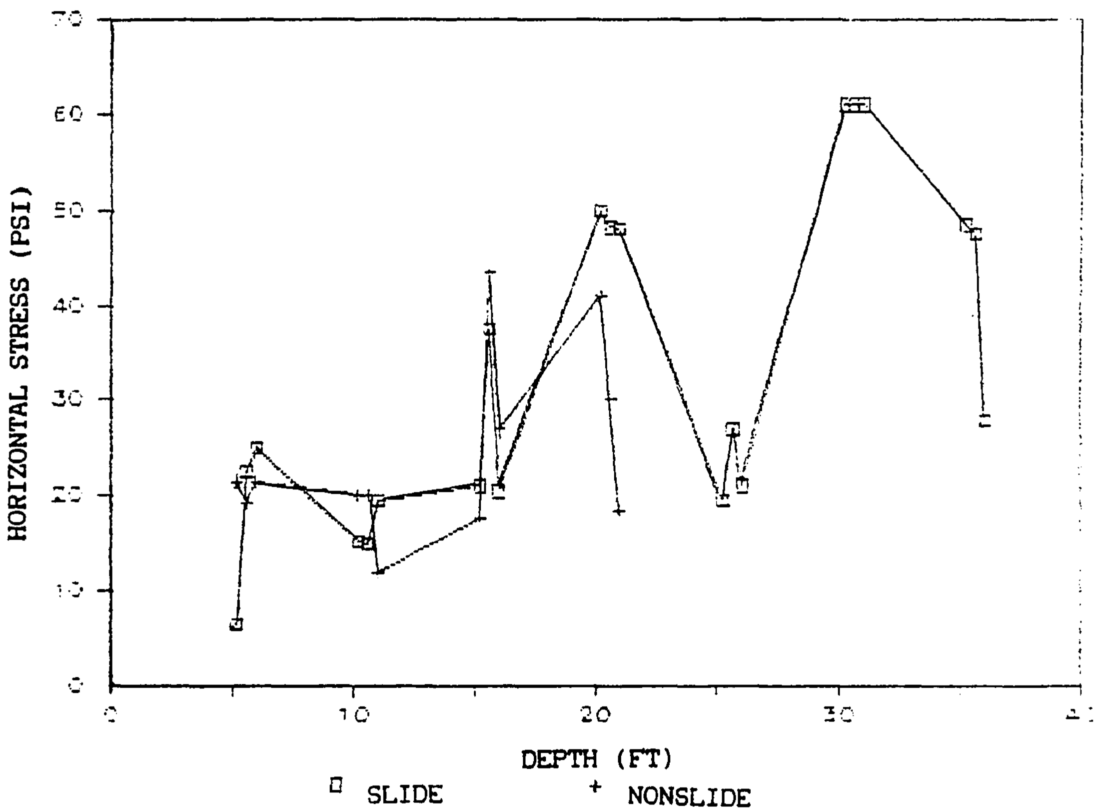

Figure 23. Results of K_0 Stepped Blade Test at Indianola slide

theory is less than 1 and occurs in the middle of the slope. These discrepancies can be attributed to the characteristics of the soil, the methods of compaction, and the effect of stress relief. This slide is a shallow slump restricted to the side slope area, and does not extend deep into the central part of the slope yet. This can also be confirmed by back-analysis of Borehole Shear Test results and field drilling observations, in which no specific weak zone or free water were observed. However, slope failures may go deeper with time. The

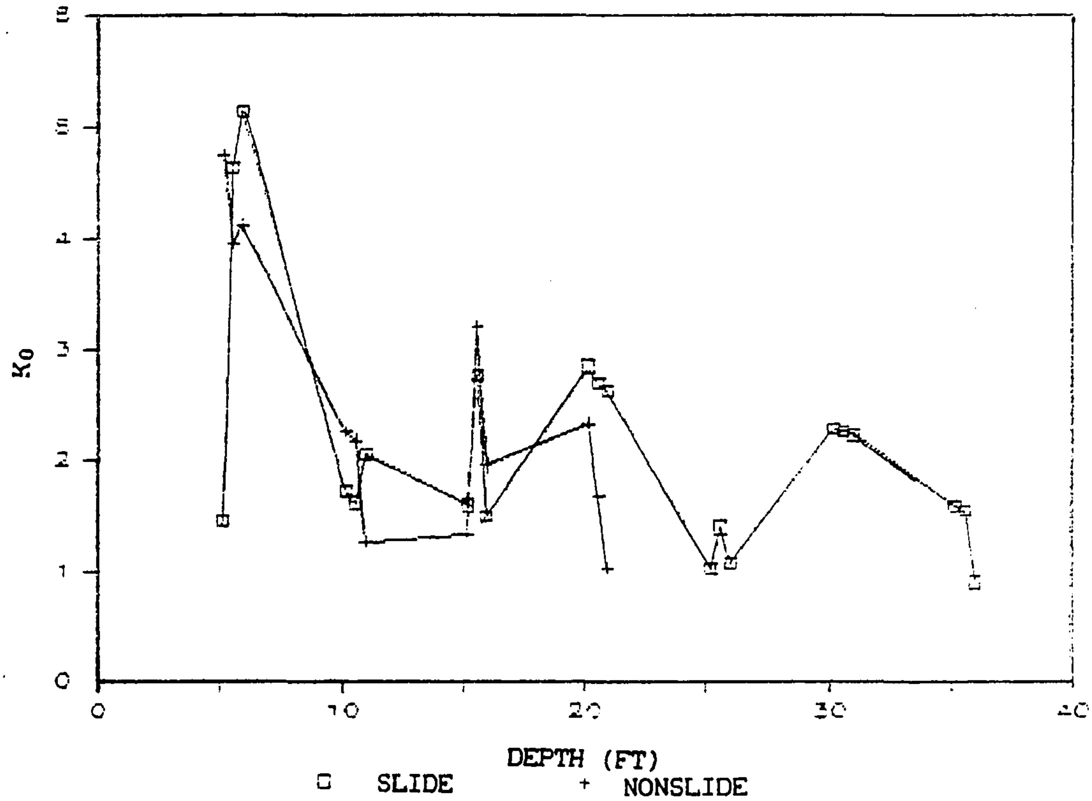

Figure 24. Ko profile at Indianola slide

concrete culvert provided some passive resistance which was helpful to keep scarps from forming.

The compacted silty clay fill is an overconsolidated clay. The first slide occurred approximately 26 years after construction, and the reactivated slide occurred about 15 years later. The reason for reactivation was the plugged culvert. In an artificially compacted soil as opposed to a cut and stress-relieved overconsolidated soil no negative pore water pressures should exist, negating this as a delay mechanism. Based on field observations, this slide initiated from the toe area and progressed upslope which suggests that it is a reactivated progressive failure. The failure mechanism for both the initial and the reactivated slides probably involved water, excess pore water pressures, and strain softening. The recommended way to fix this slide is to re-open the culvert and reshape the slope, or install a toe drain in the west side of the slope and an interseptor trench in the east side to intercept the infiltrating pond water (Dirks, Iowa Department of Transportation, personal communication).

Osceola

(A)Introduction

The test site designated as Osceola is a foreslope 2:1 slope, in compacted firm gray brown glacial clay fill. The location of the site is approximately 4-5 miles south of Osceola on Iowa highway 69, Clark county. The guardrail at SW quadrant of the bridge has moved downslope so that the end is approximately 5 ft below the roadway at approximately Sta. 795. The toe of the slope through this area has moved beyond the right-of-way fence, a distance of about 8-10 feet. From the guardrail upslope, several places of active sliding were observed up to the original backslope. From field inspection, despite the excessive settle down of the guardrail, no obvious scarps or tension cracks were observed.

The road was first constructed in 1931 at 1.5:1.0 slope. In 1960, the bridge guardrail was added at the side slope, and the slope was flattened to a 2:1 ratio (Dirks, Iowa Department of Transportation, personal communcation). A dry pond is located at west side of the slope.

The upper soil layer is 7 ft of stiff gray brown silty clay, underlain by fine gray brown sand with occasional clay layers to 14 ft, then followed by layer of firm gray brown glacial clay. The top layer of silty clay has a total wet density of 120 pcf at a moisture content of 23%, the liquid limit is 33, and the plastic limit is 17. Twenty-four hours after drilling, the water table was observed at 6.2 ft. A buried gas line is located in the west side of the slope, just outside the toe of the slope.

The first K_0 Stepped Blade Test was conducted in the slide area at Sta. 794+57, Lt. 16 ft. This zone has a layer of firm gray brown glacial clay fill to 21 ft, underlain by a layer of firm gray brown sandy silty clay as the natural glacial till. A water table was observed at 20.6 ft 24 hours after drilling. The cross-section is shown in Figure 25.

A second K_0 Stepped Blade Test was conducted at Sta. 793+37, Lt. 20 ft, where was considered outside of the slide area on the basis of visual observation. The fill has a layer of firm gray brown glacial clay fill to 6 ft, underlain by a layer of firm gray brown glacial clay. No water table was observed at a depth of 14.9 ft 24 hours after drilling. The fill has a total wet density of 125 pcf at a moisture content of 21.0%, the liquid limit is 40, and the plastic limit is 19. The natural glacial clay has a total wet density of 130 pcf at a moisture content of 23%. The liquid limit is 24, and the plastic limit

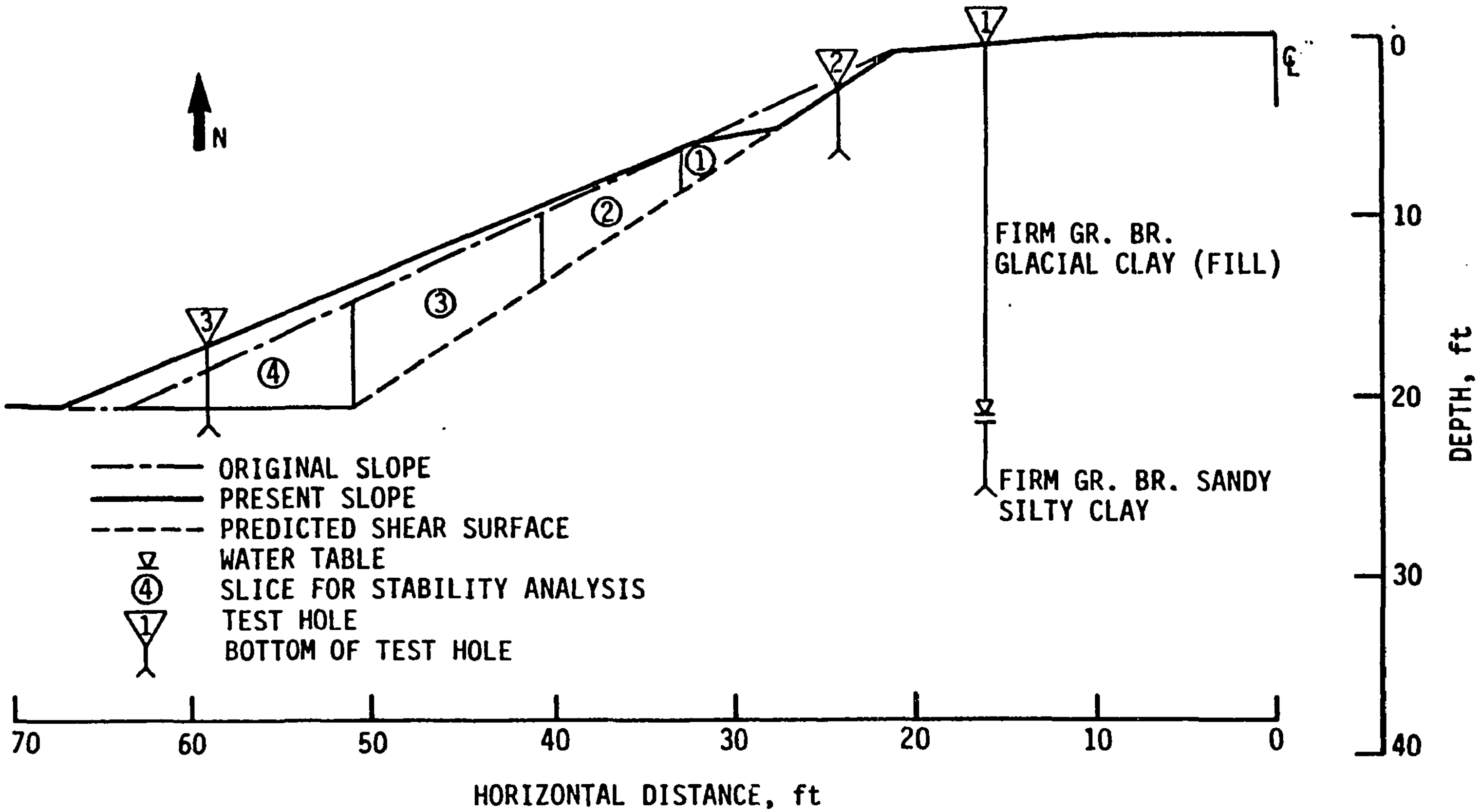

Figure 25. Cross section of Osceola slide

is 15.

Borehole Shear Tests were performed in the crest and toe areas, in hand-drilled holes. Three tests were conducted, two in the crest area and the third in the toe area.

(B) Results

The results of Borehole Shear Test are shown in Table 9. This slide is a block type slide and the predicted shear surface is along the older buried slope (1.5:1.0) based on the field observation. The factor of safety is 3.2 using Ordinary Method of Slices and Borehole Shear Test strength. However, water infiltrating along the shear surface and/or the discontinuity at the shear surface may have decreased the cohesion. When the cohesion drops to 0.43 psi and friction angle is same, the factor of safety is 1.0. Because the tests did not engage the shear surface, the measured Borehole Shear Test strength is a peak strength, while the strength mobilized along the predicted shear surface is fully softened strength. This is consistent with the case studies of Handy (1986).

Table 9. Borehole Shear Test results at Osceola slide

Boring	Depth (ft)	Soil Type	c(psi)	ϕ	r^2
2	2.0	Glacial Clay Fill	5.05	16.0^0	0.64
2	3.5	Glacial Clay Fill	2.38	19.0^0	0.99
3	2.0	Firm Sandy Silty Clay	2.76	13.5^0	0.98

The results of K_0 Stepped Blade Tests are shown in Figure 26. Horizontal stresses are corrected to K_0 values by dividing by overburden weights at each depth, shown in Figure 27. K_0 values are calculated based on effective stress, and thus are not affected by pore water pressures. The results showed that at shallow depth (2 ft), the K_0 value is the highest (7.5-8.4). This may be due to shallow soil overburden weight which will result in high K_0 value. At unsliding profile, the K_0 reaches a peak at a depth of 7.6 ft which coincidentally is at the natural ground surface. In the sliding K_0 profile, a much smaller peak K_0 at 7.6 ft may be due to the original pavement surface

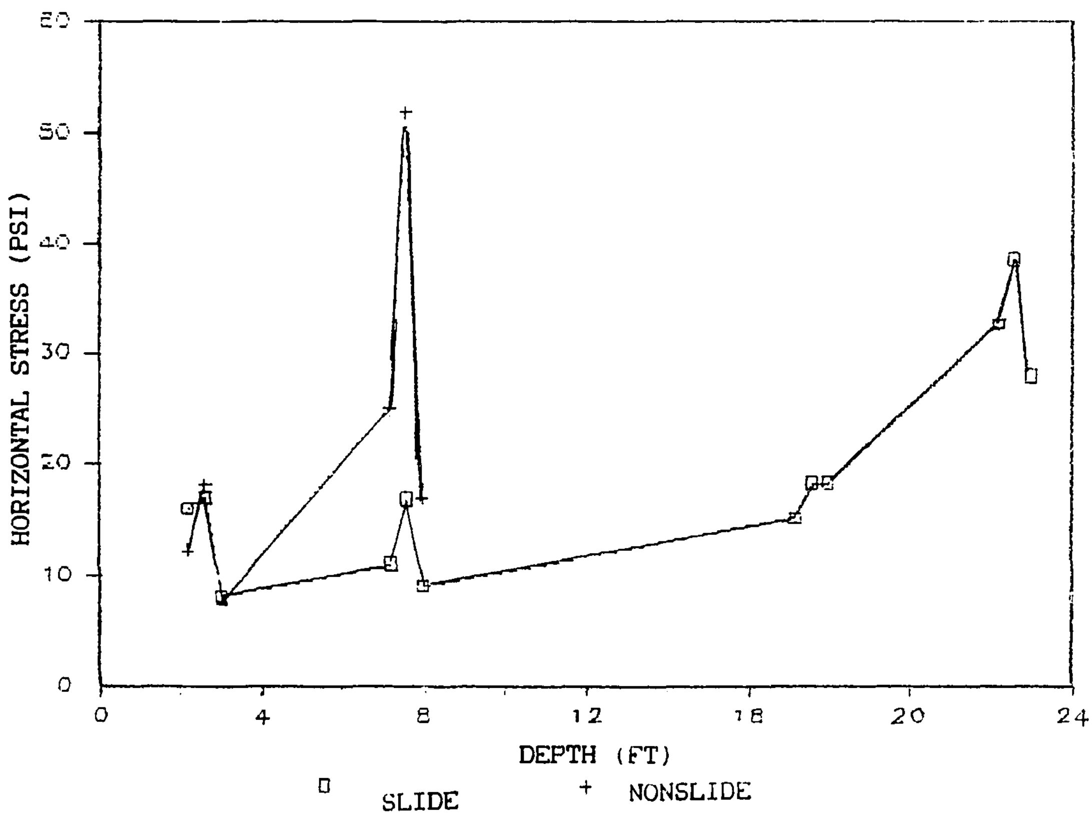

Figure 26. Results of K_0 Stepped Blade Test at Osceola slide

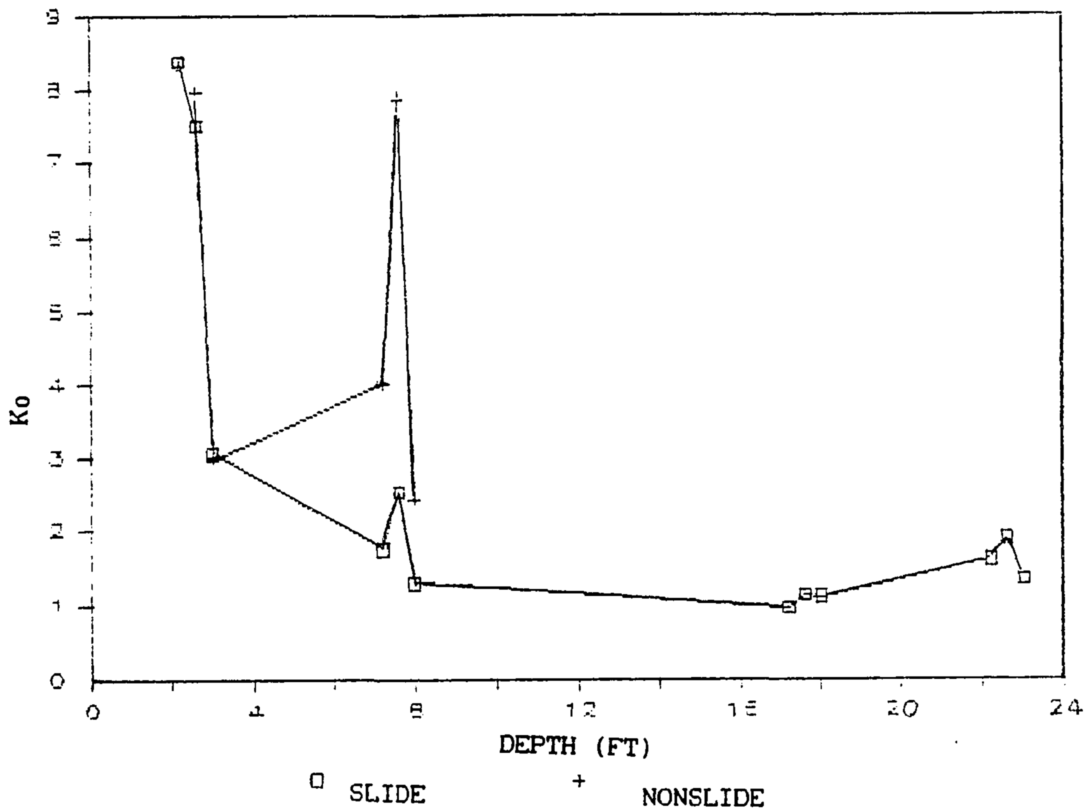

Figure 27. K_0 profile at Osceola slide

which was widened and backfilled in 1960 (Dirks, Iowa Department of Transportation, personal communication). The overall K_0 values of the unsliding profile are higher than the sliding profile above the natural glacial till surface (0-7.6 ft), indicating a substantial release of lateral stress in the slide area. Again, the trend of K_0 profiles and the magnitude of K_0 values are different from prediction of elastic theory. The settlement of the guardrail, back-analysis of Borehole Shear Test results, and K_0 values indicate that the sliding mass is moving along the old buried slope surface (1.5:1.0). This is a block slide, and different materials were observed by hand augering in the central part and side slope of the slide area. The guardrail settlement can be caused by inadequate compaction of the side slope or by downward movement of a vertical wedge. The ability of the K_0 Blade Test to distinguish between the natural glacial till and fill can be important to estimate a sliding surface without resorting to Slope Indicator methods.

The tested soil is compacted glacial clay fill, is an overconsolidated clays. This slide occurred approximately 26 years after the new side slope was put on. No negative pore water pressure effects should have existed. Most probably as surface water infiltrated along the interface between the new and the old fill, it weakened the soil interface. This is particularly likely if vegetative mottes was left in place. Thus, water softening may be the main factor for this delayed failure.

The recommended way to fix this slide is to remove the guardrail and part of the side slope soil, flatten the foreslope to a 3:1 slope, and adequately recompacte the slope (Dirks, Iowa Department of Transportation, personal communication).

Guthrie

(A) Introduction

The test site designated as Guthrie is a foreslope at 2.5:1.0 slope, compacted firm gray brown glacial clay. The location is on the west side of Iowa highway 25 approximately 0.5 mile north of the junction with Iowa highway 384 in Guthrie county. The slide type is a slump. This site has distinctive crack pattern, including a well-developed slide, developing tension cracks, and an unslid zone. A K_0 Stepped Blade Test was attempted and then terminated due to leaking diaphragms. One cell Electric K_0 Blade then was used that did not allow extrapolation to zero blade thickness but is nevertheless useful to show changes in lateral stress. Three tests were performed; test a is adjacent to the slide area, test b is in the developing tension crack zone, and test c is in the unslid zone.

(B) Results

The results of the tests are shown in Table 10. Due to the restriction to one blade cell, no lateral stress at zero thickness can be measured. Two minute readings are given at 3 to 4 ft depth at each location.

At zone a, the lateral stress is relieved by sliding, and the reading is lowest (19.4 psi). The development of tension cracks at zone b is consistent with the partial relief of lateral stress (26.8 spi) compared to the unfailed slope zone c, where the lateral stress is the highest (55.9 psi). In Table 10, the lateral stress relief at different stages of sliding can be clearly seen. The K_0 Blade therefore should prove valuable to detect lateral stress relief in compacted soils prior to sliding.

Table 10. Comparison of lateral stress at different locations of slope (stress sensor tranverse to slope direction)

Location	Slope Situation	Lateral Stress (psi)
On crest, 11 ft from slide scarp	Adjacent to the slide	19.4
On slope, 31 ft from slope edge	Tension crack developing	26.8
On slope, 28.5 ft from slope edge	Unfailed slope	55.9

Discussion

From the preceding case histories it appears that slope failure can be analyzed and perhaps predicted from measured K_0 profiles and back-analyses of Borehole Shear Test results. The type of slide also can be decided.

Engineers concerned with the design and analysis of slopes and embankments have long been interested in determining their stresses and deformations. The analysis of stress in earth embankment is an exceeding complex problem. Most researchers assume that the three-dimensional system can be represented as a two-dimensional or plane strain problem, and assume that the soil is linearly elastic so that the problem can be reduced to a standard plane strain elasticity analysis. Several charts are available for different slope configurations and foundation flexibility, the predicted horizontal stresses along vertical sections varying with depth, with a maximum K_0 value that is less than 1 and is located in the middle of the slope (Clough and Woodward, 1967, Perloff et al, 1967., Poulos and Davis, 1974). Neither the trend of the K_0 profile nor the magnitude of the K_0 values analyzed from elastic theory or by finite element method is similar to the field test results, mainly because of built-in stresses from compaction and the effect of stress relief. The behavior of stiff overconsolidated clays and clayshales is strain softening, not elastic or plastic (Chowdhury, 1978).

Numerous analysis methods have been proposed based on differing assumptions regarding interslice forces including direction, magnitude, point of application, and their effects on the factor of safety. The direction of the interslice forces on each slice may be set at a constant angle that can (1) parallel the ground surface, (2) equal the average slope from the beginning to the end of the slip surface, or (3) be at an arbitrarily defined direction. The magnitude of the interslice forces on each slice can be zero, as in the Ordinary Method, or an arbitrary assumed values. However, the interslice forces have only an insignificant effect on the computed factor of safety (Bishop, 1955, Morgenstern and Price, 1965, Fredlund, 1984, Fan and Wilson, 1986). For the case of a cohesive soil, all methods satisfying moment equilibrium (or moment and force equilibrium) give essentially the same results (Fan and Wilson, 1986). This means that the lateral stress of the individual slice presumably does not strongly influence the results by different limit equilibrium methods. However, none of these methods recognize the role of a high initial lateral stress wherein K_0 is larger than 1.0, nor do they recognize the relevance of strain relief or of progressive failure. This would appear to be a salient area for future research, both analytical and in the field, as with the K_0 Blade.

Although the individual interslice forces did not have a significant effect on the computed factor of safety, from the K_0 profile and Borehole Shear Test results, it is possible to tell the type and the extended area of slide which can not be seen by visual observation. The long retention of lateral stresses in compacted soils indicates that the best way to predict a slide by K_0 Stepped Blade Test is through periodic testing at the crest or toe of a potential slide area, to detect changes in K_0 with time. Because of accessibility the best place to test highway foreslopes is at the crest area, while for natural slopes the crest and toe may possibly be tested.

GENERAL DISCUSSION

Slides may occur in almost every conceivable manner, slowly or suddenly, with or without any apparent provocation. Various studies have shown that most damaging landslides are human-related, and most of the landslide losses can be prevented by thorough preconstruction investigation, analysis, design, and careful construction procedures. Because of abnormal behavior compared to other soil and rock materials, the slope stability of overconsolidated clays and clayshales is of particular interest to researchers.

Case histories of slope failures in stiff overconsolidated clays and clayshales indicate that factors related to stability analysis include pore pressure, effective stress, peak and residual strengths, and effects of fissures and anisotropy. Different failure models are related to the strength reducing mechanisms causing delayed failure, progressive failure, and creep.

For delayed failure, the most common factors are negative pore water pressure equilibration and strain softening, which will lead to a reduction in shear strength from peak to the fully softened strength. The procedure of negative pore water pressure equilibration may be long compared with the softening process. Thus, for a long-term stability, negative pore water pressure equilibration will be a dominant process, especially for a first-time slide. The erosion of natural slopes and the downcutting of the river valleys is usually slow, and because negative pore water pressure was locked inside the slope, the slope is temporarily stable. Negative pore water pressure can be predicted in cuts from the rate of soil removal, the swelling potential of the soil mass, the ratio of the thickness of soil layer removed to its original thickness, and the nature of the bottom boundary.

It may not be easy to distinguish between progressive failure and delayed failure on the basis of back-analysis alone. For delayed failure, the slide type is a first-time slide occurring mostly in homogeneous soil with small displacements, and the strength parameter is characterized by the fully softened state of strength. For progressive failure, the slide type is a reactivated slide occurring mostly in nonhomogeneous or layered soils with large displacements, and the strength is characterized by the residual strength.

Theoretically, progressive failure is due to the nonuniform mobilization of shear strength along a potential slip, which maybe occurred due to local overstress, large deformation, and changes in loading conditions. This failure process can not be interpreted by softening or negative pore water pressure equilibration. An elastic yield model is a good approach but with some restriction, and the shear band models seem to fit some field observations of the growth pattern of slip surface. However, the calculated factor of safety by shear band model is in the upper-bound and is not conservative.

Where cracks will initiate in a slope is an argumentative question. Based on field observations, finite element analysis, and stability analysis, it is believed that the cracks initiate from the toe for overconsolidated clays and clayshales. The rate of crack propagation may be related to the strain softening behavior. The area of overstress also is influenced by the initial stress. A high initial lateral stress can result in large shear stress at the base of an excavated slope, and an increased possibility of progressive failure. This initial lateral stress is related to the stored strain energy and plasticity index.

Creep is a widespread natural phenomenon, and can be defined as the very slow downward and outward movement of earth slopes, without the formation of a continuous rupture surface which usually precedes majority of landslides. Seasonal creep is resulted from several seasonal processes limited at surficial layer, while deep creep is still ambiguous. Creep does not proceed continuously or uniformly, the rate being affected by changes such as the rate of dissipation of the negative pore water pressure, and seasonal changes caused by fluctuations of perched water table. Under these circumstances, the mechanism of creep may coexist with progressive failure.

To interpret landslide by creep model, the rheological model approach is the best known method. The Bingham flow model is appropriately suited. Up to the present, very few creep models fit field conditions theoretically or quantitatively.

In natural landslides, creep, progressive failure, and delayed failure all may coexist at different stages. Their relationships can be focused on as a difference in driving force, deformation, deformation rate, sliding zone, initiation of crack, slide type, geological soil types, and strength parameters.

The Borehole Shear Test and the K_0 Stepped Blade Test were used in several landslide sites including both failed and nearby unfailed slopes. The ability to predict slope failure can be exemplified by the measured K_0 profiles, back-analysis of Borehole Shear Test results, and visual observation. The type and the depth of rotational slide and block slide can also be determined. The lateral stresses in an active slide, a potential slide, and a nonsliding part of slope were related to stress relief. The application of elastic theory or finite element analysis to predict lateral stresses in a compacted or overconsolidated soil is inappropriate since they take into account only the response of the soil to its own weight and externally applied forces, and not the inherited locked-in stresses.

Similarly, although the lateral stress of the individual slices does not have a significant effect on the computed factor of safety in conventional limit equilibrium methods, these methods do not take into account inherited lateral stresses or predict progressive failures. From the K_0 profile and back-analysis of Borehole Shear Test results, it is possible to tell what type of the slide and whether the slide area extends to other zone which can not be detected by visual observation. The best way to predict the slide by the K_0 Stepped Blade may be through periodic testing to determine time-related changes in K_0.

CONCLUSIONS

1. *In situ* lateral stress plays an important role in slope stability of overconsolidated clays and clayshales.

2. Based on the K_0 profile and back-analysis of Borehole Shear Test results, it is possible to distinguish the type of slide and whether the slide area extends to other zones which can not be detected by visual observation.

3. Based on the K_0 profile, it is possible to distinguish natural soil layers that can be inferred to support the sliding zone.

4. The trend of the K_0 profile and the magnitude of the K_0 value analyzed from field tests are not in agreement with elastic theory that does not recognize inherited stresses.

5. The lateral stresses in a slide, a potential slide, and a nonsliding part of a slope show the results of lateral stress relief at different stages of sliding.

The conclusions 6-10 are mainly based on literature reviews. They are also consistent with the results and observations of field tests performed in this study.

6. In natural landslides, delayed failure, progressive failure, and creep may coexist at different stages.

7. The differences between delayed failure, progressive failure, and creep focus on driving force, deformation, deformation rate, sliding zone, initiation of cracks, slide types, geological soil types, and strength parameters.

8. For overconsolidated clays and clayshales, because of stress relief

the initiation of cracks start from the toe of the slope.

9. For different clayshales worldwide, the engineering behavior and shear strength parameters are very similar. Therefore, the experience from case histories can be applied and compared.

10. The creep model that is not associated with the presence of a slip surface is not a landslide phenomenon, and the residual strength can not interpret a first-time slide through back-analysis.

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APPENDIX A

Borehole Shear Test Results:

INDIANOLA:					
Boring 3 & Depth 3 ft					
Normal stress (psi)	4.8	9.7	15.3	20.6	
Shear stress (psi)	2.1	4.5	7.0	8.1	
Boring 3 & Depth 4 ft					
Normal stress (psi)	4.3	10.0	15.0	20.6	
Shear stress (psi)	2.1	4.0	6.4	7.8	
OSCEOLA:					
Boring 2 & Depth 2 ft					
Normal stress (psi)	4.8	9.8	15.3	20.6	25.8
Shear stress (psi)	4.2	6.7	7.2	8.3	8.6
Boring 2 & Depth 3.5 ft					
Normal stress (psi)	4.3	9.8	15.3	20.0	23.2
Shear stress (psi)	3.7	5.9	8.1	9.7	10.2
Boring 3 & Depth 2 ft					
Normal stress (psi)	5.0	10.3	15.3	22.7	27.9
Shear stress (psi)	4.2	5.2	6.4	7.8	10.0

APPENDIX B

Data reduction and interpretation of K_0 Stepped Blade Test usually is done with semi-logarithmic graph paper, with blade thickness plotted on the arithmetic scale and the measured stresses on the logarithmic scale. A best-fit line is drawn through the points of a given data set.

Pressure readings taken at the same depth are considered and plotted as a set. Thus, four data points (3.0, 4.5, 6.0, and 7.5 mm thickness) are obtained at the first advance depth, three data points (3.0, 4.5, and 6.0 mm thickness) at the second advance depth, two data points (3.0, and 4.5 mm thickness) at the third advance depth, and finally a single data point (3.0 mm thickness) at the fourth depth. The single data point usually is discarded. Only those points that show increasing pressure with increasing thickness are used to extrapolate to zero thickness. Several reasons for non-linearity are proposed, and are discussed by Mings (1987) and by Retz (1987).

For example from the table in the next page, corresponding values at 5 ft depth are:

3.0 mm	12.5 psi	4.5 mm	17.0 psi
6.0 mm	12.5 psi*	7.5 mm	34.5 psi

* Not consistent increasing pressure with increasing thickness; data omitted.

An exponential regression, $r^2 = 1.00$, $a = 6.24$, $b = 0.23\ mm^{-1}$ where a is the extrapolated *in situ* lateral stress in psi and b is the slope.

Ko Stepped Blade Test Results:

Location: Indianola, Iowa

Condition: Slide Zone

DEPTH (ft)	PRESSURE (psi)									
	Push 1	Push 2		Push 3			Push 4			
(cell)	1	2	1	3	2	1	4	3	2	1
(thickness, mm)	3.0	4.5	3.0	6.0	4.5	3.0	7.5	6.0	4.5	3.0
5	12.5	17.0	38.0	12.5	49.0	27.0	34.5	33.0	28.0	4.0
10	29.5	37.0	38.0	13.0	57.0	55.5	33.0	52.0	77.5	4.0
15	21.5	21.0	42.0	7.0	50.5	39.0	36.5	45.5	53.0	60.0
20	52.5	51.0	47.5	36.0	46.0	49.0	44.0	43.0	56.0	44.0
25	26.0	41.5	29.0	26.5	44.0	31.5	45.0	33.0	54.5	70.0
30	64.0	64.0	62.0	66.0	62.0	60.5	75.0	60.0	66.0	48.0
35	54.0	64.0	46.0	61.0	62.0	47.5	73.0	56.0	63.0	51.0

Location: Indianola, Iowa

Condition: Nonsliding Zone (By Visual Observation)

DEPTH (ft)	PRESSURE (psi)									
	Push 1	Push 2		Push 3			Push 4			
(cell)	1	2	1	3	2	1	4	3	2	1
(thick-ness, mm)	3.0	4.5	3.0	6.0	4.5	3.0	7.5	6.0	4.5	3.0
5	21.0	32.0	23.0	24.0	37.0	39.0	42.0	29.0	45.0	40.0
10	43.5	49.0	22.5	52.0	36.0	25.5	68.0	24.5	38.0	26.0
15	11.0	24.0	49.0	18.0	55.0	42.0	35.0	55.0	55.0	32.0
20	45.5	48.0	3.0	30.5	15.0	42.0	41.0	15.0	65.0	39.0
25	Blade Bended at First Push									

Location: Osceola, Iowa

Condition: Slide Zone

DEPTH (ft)	PRESSURE (psi)									
	Push 1	Push 2		Push 3			Push 4			
(cell)	1	2	1	3	2	1	4	3	2	1
(thickness, mm)	3.0	4.5	3.0	6.0	4.5	3.0	7.5	6.0	4.5	3.0
2	20.5	17.0	22.0	13.0	18.0	11.0	17.0	18.0	13.0	25.0
7	16.0	14.0	23.0	15.0	20.0	13.0	16.5	21.5	15.0	19.0
12	43.0	33.0	27.0	Hitted something hard, pulled out and drilled through.						
17	18.5	16.5	16.5	15.0	22.0	21.5	15.0	23.5	23.5	24.5
22	37.5	35.5	41.0	40.5	44.0	39.0	27.5	44.0	46.5	39.0

Location: Osceola, Iowa

Condition: Nonsliding Zone

DEPTH (ft)	PRESSURE (psi)									
	Push 1	Push 2		Push 3			Push 4			
(cell)	1	2	1	3	2	1	4	3	2	1
(thickness, mm)	3.0	4.5	3.0	6.0	4.5	3.0	7.5	6.0	4.5	3.0
2	29.5	15.0	25.0	12.0	21.0	12.0	5.0	21.5	15.0	38.0
7	40.5	39.0	56.0	32.0	56.0	32.0	34.0	49.5	45.0	76.0