as steel-strip or plain longitudinal-bar reinforcements in gravelly sand soil.

3. Bar-mesh reinforcement embedded in dense silty class soil exhibited greater pull resistance than bar-mesh reinforcement embedded in less dense gravelly sand soil.

4. An increase in mesh opening will substantially reduce the pullout resistance of the bar-mesh reinforcement.

5. The skin friction angle between a galvanized steel strip and soil for granular material is only slightly smaller (6 to 13 percent) than the internal friction angle of the soil. For practical design purposes, the skin friction angle between the galvanized steel strip and soil material can be assumed to be 10 percent smaller than the internal friction angle of the soil.

6. Cohesive soil of low plasticity can be used in reinforced earth providing that bar-mesh reinforcement is used.

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Some Uncertainties of Slope Stability Analyses

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Some practical limitations of total stress and effective stress analyses are discussed. For clays having a liquidity index of 0.36 or greater, ϕ-equal-zero analyses based on laboratory undrained shear strengths give factors of safety close to the actual factor of safety. However, ϕ-equal-zero analyses based on field vane sheer strengths may yield factors of safety that may be too high. The difference between field vane and calculated shear strengths increases as the plasticity index increases. For clays having a liquidity index less than 0.36, ϕ-equal-zero analyses that use laboratory undrained shear strengths give factors of safety that are too high; however, the strength parameters can be corrected by the empirical relation presented here. An empirical relation for correcting field vane shear strength is also presented. A method is proposed for predicting the probable success of ϕ-equal-zero analysis. Data suggest that overconsolidated clays and clay shales or clays having a liquidity index less than 0.36 pose a slope design dilemma for engineers. An effective stress analysis based on peak triaxial shear strength parameters generally yields factors of safety that are too high; residual shear strength parameters frequently yield factors of safety that are too low. The theoretical strength of an overconsolidated clay that has undergone a softening process is approximated by using the effective stress parameters that might be obtained from triaxial tests performed on remolded, normally consolidated clay. It is suggested the soil be remolded to a moisture content equal to the plastic limit plus the product of 0.36 and the plasticity index.

Two limiting conditions (2) must be considered when designing a cutting in a clay or an embankment on a clay foundation to ensure against a first-time failure (no preexisting shear plane). The first condition is the short-term or end-of-construction case in which the water content of the clay does not change. In this case, excess pore pressures are controlled by the magnitude of the stresses acting in the clay or tending toward instability; therefore, significant dissipation of pore pressure does not occur. However, it is difficult to predict the excess pore pressures. Consequently, the short-term design is made by using the ϕ-equal-zero analysis and the undrained shear strength obtained from unconsolidated-undrained (UU) triaxial tests, unconfined compression (U) tests, field vane shear (FV) tests, or a combination of these tests.

The second condition is the long-term, steady seepage case. In this case, pore pressure do not depend on the magnitude of total stresses but are controlled by the flow pattern of underground water or the groundwater level. Excess dissipation of pore pressure occurs and the clay exists in a drained state. Long-term design is performed in terms of effective stress and the drained shear strength parameters (ϕ' and c') that are conventionally obtained from consolidated isotropically, drained (CID) triaxial tests; consolidated isotropically, undrained (CIU) triaxial tests with pore pressure measurements; consolidated-drained, direct shear (CDS) slow tests; or a combination of these tests.

For a cutting in a clay, the long-term stability is considered critical because pore pressures are initially low or negative and gradually increase toward steady seepage pore pressures. The increase of pore pressures causes a decrease in the shear strength of the clay because there is a reduction in effective stresses. For an embankment on a clay foundation, the short-term stability is considered critical because pressures steadily increase to maximum values during construction and gradually decrease thereafter toward the initial pore pressures thus increasing shear strengths with time.

LIMITATIONS OF TOTAL STRESS ANALYSES

If the stress history and moisture state of the clays in the foundation or slope are not regarded, then application of the ϕ-equal-zero analysis for designing embankments founded on clay foundations or a slope cut in a clay may lead to erroneous conclusions concerning the safety factor (11). These conclusions may be erroneous because the undrained shear strengths obtained from laboratory or field tests may be higher than the actual (back-computed) shear strengths existing at failure.

Long-Term Stability of Cut and Natural Slopes

Bishop and Bjerrum (2) summarized the results of a number of failures in natural slopes and cuts and showed that application of the ϕ-equal-zero analysis for slopes where pore pressure and water content equilibrium have been attained is unreliable. In these cases, the ϕ-equal-zero analysis gave safety factors ranging from 0.6 for sensitive soils to 20 for heavily overconsolidated soils. Two reasons for the differences between the in situ shear strength and the shear strength obtained from the undrained test are differences between field and laboratory pore pressures and migration of water to the failure zone of a slide in overconsolidated clays (9, 18). Lo and others (13) have also shown that for stiff-fissured clays the effect of sample size is an important factor in stability analyses. The shear strength of large samples is less than that of small samples.

Examination of case records for long-term failures in cuts and natural slopes revealed that high safety factors are associated with low to negative values of the liquidity index while low safety factors are associated with high values of the liquidity index. In data cited by Bishop and Bjerrum (2) there were four cases in which the safety factor was near one; the liquidity indexes ranged from 0.20 to 1.09. In the other cases, the liquidity indexes ranged from about 0.19 to -0.36 while the safety factors ranged from 1.9 to 20.

Short-Term Stability of Loads on Soft Foundations

A number of case records was assembled by Bjerrum (3, 5) to show that the procedures normally used to determine the short-term stability of embankments, footings, and load tests on soft clay foundations are unsatisfactory. In those cases, use of ϕ-equal-zero analysis and undrained shear strengths from field vane shear tests overestimated the safety factor for soils having liquid limits and plasticity indexes in excess of approximately 80 and 30 percent respectively. Also, the difference between field vane [$(S_u)_{vane}$] and corrected shear strengths [$(S_u)_{corrected}$] increases as the plasticity index (PI) and the liquid limit of the clay increase. By assuming that there is a linear relation between safety factor and plasticity index, the corrected shear strength may be expressed as follows:

$$(S_u)_{corrected} = (S_u)_{vane}/[(0.84 + 0.0082\ PI) \pm 0.12] \tag{1}$$

Figure 1 shows a comparison of data representing end-of-construction failures of footings, fills, and excavations on saturated clay foundations assembled by both Bishop and Bjerrum (2) and Bjerrum (3, 5). Liquidity indexes by the former ranged from about 0.25 to 1.44. The undrained strengths of the soils in these analyses were obtained primarily from unconsolidated-undrained tests. Bjerrum's data showed that the difference between vane and back-computed shear strengths increases as the plasticity index of the clay increases whereas Bishop and Bjerrum's data, in marked contrast, showed that the back-computed shear strength and laboratory shear strength were almost equal.

Short-Term Stability of Embankments on Overconsolidated Clays and Clay Shales

A number of short-term failures of embankments on overconsolidated soils occurred even though the ϕ-equal-zero analysis indicated the embankment slopes should have been stable. Some examples include case histories by Beene (1), Wright (20), Peterson and others (16), and Hopkins and Allen (10). Safety factors from ϕ-equal-zero analyses ranged from 1.23 to 4.0 for these cases; all had liquidity indexes less than 0.36.

Short-Term Stability of a Cut or Excavated Slope in Overconsolidated Clays and Clay Shales

Because the short-term safety factory is usually at maximum during or near the end of construction, the ϕ-equal-zero analysis is often used to determine the short-term stability of a cut or excavated slope. However, stability of cuts in overconsolidated clays and clay shales may not always conform to this concept. For instance, Skempton and Hutchinson (19) described two slides in a stiff overconsolidated London clay. Based on a ϕ-equal-zero analysis and undrained shear strengths, the short-term safety factors were about 1.8.

Proposed Method of Predicting Success in a ϕ-Equal-Zero Analysis

Peck and Lowe (15) presented a portion of Bishop and Bjerrum's data (long-term failures in cuts and natural slopes) that showed that the computed safety factor of failed slopes, obtained from a ϕ-equal-zero analysis and undrained strengths, was apparently a function of the liquidity index. Peck and Lowe suggested the possibility of using that empirical relation to determine correction factors for laboratory undrained strength parameters.

By plotting additional portions of Bishop and Bjerrum's data (2) and Bjerrum's data (5) (safety factor as a function of liquidity index), a distinctive division can be observed. All data in Figure 2 represent failures obtained by using the ϕ-equal-zero analysis and undrained shear strengths from UU, U, or FV tests. For failures in soils with a liquidity index equal to or greater than approximately 0.36, the safety factors estimated by using ϕ-equal-zero analysis and UU or U strengths should have an accuracy with ±15 percent (Figure 2), and design safety factors as low as 1.3 may be justified in many routine designs. For obtaining undrained strength from in situ vane shear tests, the vane strength should be corrected.

For failures in soils with a liquidity index less than about 0.36, use of ϕ-equal-zero analysis and UU or U strengths gives safety factors that are too high; in situ shear strengths are overestimated by laboratory tests. For soils having liquidity indexes less than 0.36, the safety factor appears to be a function of the liquidity index (LI) as follows:

$$F = (3.98)(0.0192)^{LI} \quad (2)$$

The safety factor can be expressed as

$$F = S_u/S_s, \quad (3)$$

where S_u is the laboratory undrained shear strength; therefore, the corrected laboratory or softened shear strength may be expressed in terms of the standard error as follows:

$$S_s \approx (0.252)S_u(0.0192)^{-LI}\,(10^{\pm 0.24}) \quad (4)$$

The error in the corrected shear strength may be as large as 70 percent.

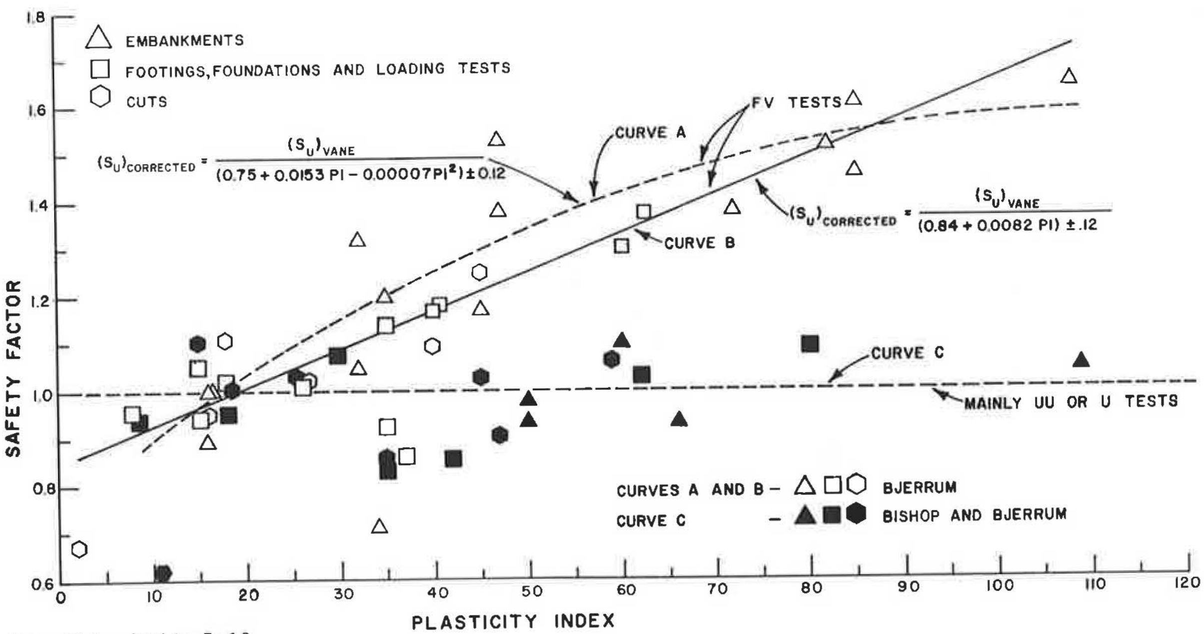
LIMITATIONS OF EFFECTIVE STRESS ANALYSES

Uncertainties in the application of the effective stress approach to the design of earth slopes arise in the selection of shear strength parameters (ϕ' and c') and the evaluation of pore pressures. Although the effective stress method has been successfully applied to normally consolidated and lightly overconsolidated clays and silty clays having an intact structure (free of fissures or joints), the method is not successfully applied to the design of slopes composed of overconsolidated clays and clay shales. Although much research (4, 17, 18) has been directed toward understanding the characteristics of those soils, overconsolidated soils still pose a slope design dilemma for engineers.

Shear Strength

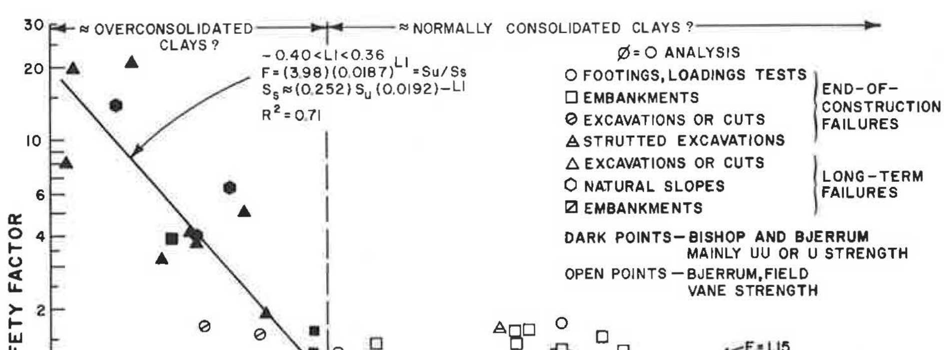
Typical stress-strain curves for normally consolidated

Figure 1. Factor of safety as a function of plasticity index.



Notes: All slopes failed, i.e., F = 1.0.
Data are from Tables 1, 4, 5, and 7 in Bjerrum (3, 5) and from Table 2 in Bishop and Bjerrum (2).
All curves are by Hopkins and others (11).

Figure 2. Factor of safety as a function of liquidity index.



Note: Data are from Tables 2, 3, and 5 in Bishop and Bjerrum (2); Peterson and others (16); and Hopkins and others (11).

and overconsolidated clays, similarly tested under drained conditions, show that both clays reach a peak strength. When the overconsolidated soil is strained beyond peak strength, shear resistance decreases until higher strains are attained in which case the strength value decreases to a (nearly) constant value. This lower limit of resistance is the residual or ultimate strength (17, 18, 19). After the peak strength has been attained, the shear resistance of the normally consolidated clay may decrease only slightly. After higher strains are attained, the shear resistance of the overconsolidated and normally consolidated clays coincide. In heavily overconsolidated plastic clays, there is a large difference in the peak and residual strengths. In silty clays and soils of low plasticity, this difference is very small. With an increase in clay content, this difference increases even in normally consolidated clays, although not as much as in overconsolidated clays.

The softened shear strength of an overconsolidated clay (as obtained from Equation 4) may be defined as the intersection of a horizontal line projected from the peak strength of the normally consolidated clay with the stress-strain curve of the overconsolidated clay (17). The softened strength is intermediate to the peak and residual strengths and probably occurs at much lower strains (representing a condition in which a number of small, independent shear planes exist) than the residual strength (representing a condition in which the shear planes have joined to form a well-defined failure plane).

The critical state of a normally consolidated clay can be defined (17) as the state (in a drained condition) in which any further increment in shear distortion will not result in any change in water content. The water content at the critical state is equal to that ultimately attained in an overconsolidated clay that has expanded during shear.

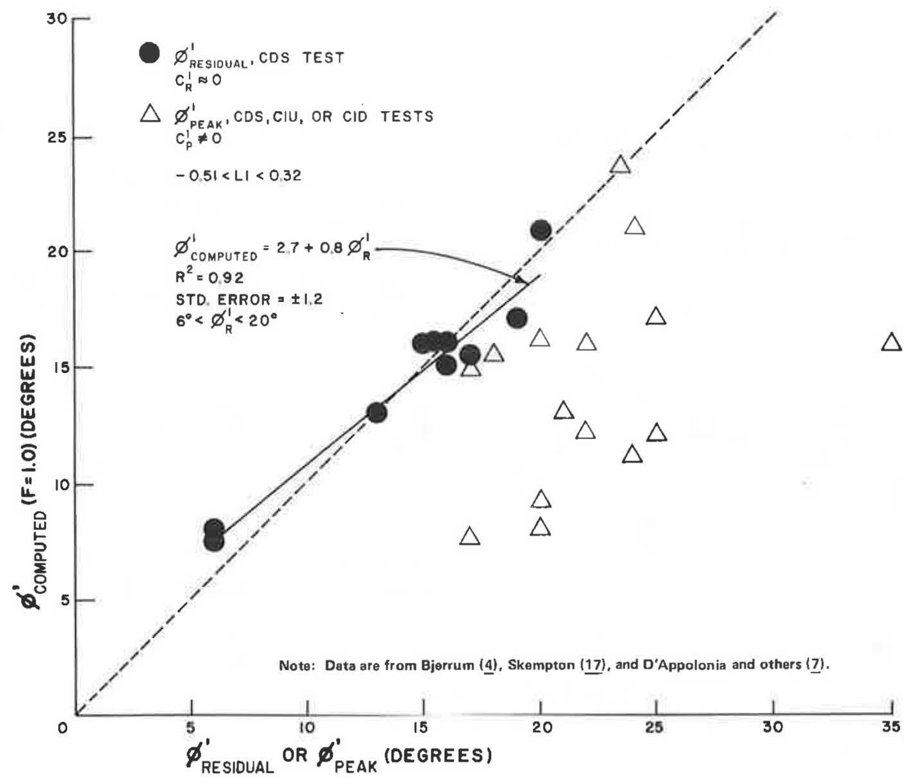


Figure 3. Back-computed shear strength parameter as a function of peak shear strength parameter from triaxial tests and residual shear strength parameter from consolidated-drained, direct shear tests.

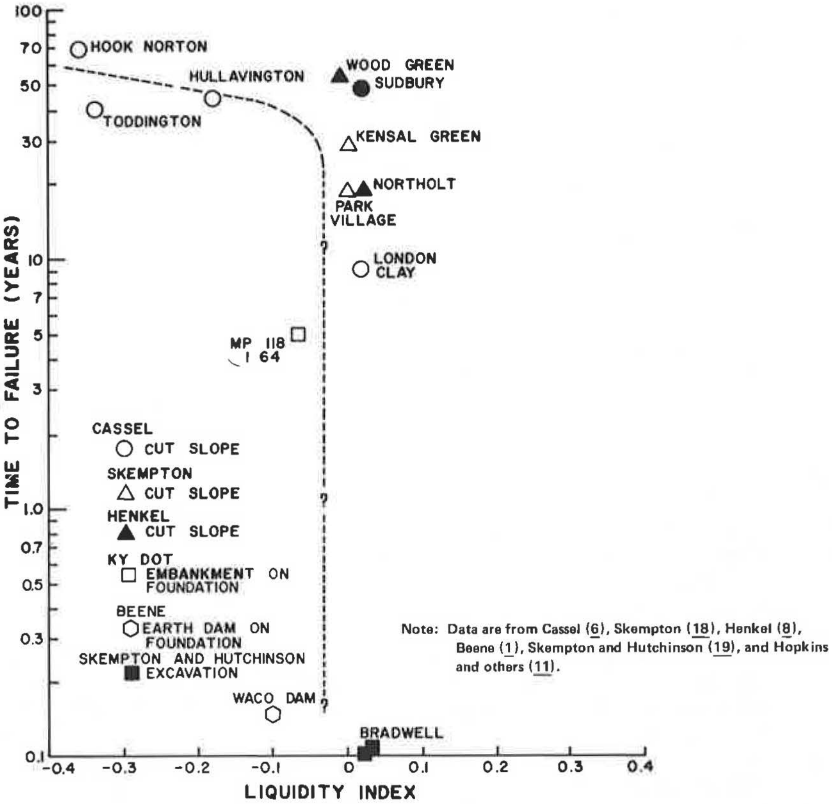
Table 1. Case histories based on effective stress analysis.

Location	Type of Slope	Stress Analysis Factor					Factor of Safety			Reference
		w	PL	LL	PI	LI	Peak	c = 0	Residual	
Bluegrass Pky, MP 21	Fill[a]	20	24	34	10	-0.40	1.46	–	–	11
West Ky Pky, MP 96	Fill[a]	15	19	31	12	-0.33	1.94	1.12	–	11
Bluegrass Pky, MP 43	Fill	17	34	16	18	-0.08	≫1.00	1.10	–	11
Selset	NS	12	13	26	13	-0.08	1.03	<1.00	0.69	2
I-64, MP 118	Fill[a]	23	25	53	28	-0.07	1.28	0.97	0.77	10
Weirton, W. Va.	Cut	26	25	51	26	0.04	1.51	–	≈1.00	7
Northolt	Cut	30	28	79	51	0.04	1.63	0.77	0.54	19
Jackfield	NS	21	22	44	22	0.05	2.06	>1.00	1.11	18
Kensel Green	RW	33	30	83	53	0.06	1.60	–	0.60	18
Sudburry Hill	Cut	31	28	82	54	0.06	2.27	1.05	0.74	19
I-64, MP 44	Fill[a]	21	18	40	22	0.14	1.01	0.72	–	11
Amuay	NS	30	24	64	41	0.15	2.27	<1.00	0.50	12
US 119	Fill[a]	21	19	24	7	0.25	1.13	1.13	–	11
Seven Sisters, S-6	Fill	45	26	85	59	0.32	1.65	–	–	16
Lodalen	NS	31	18	36	18	0.72	1.07	–	0.73	19
Drammen	NS	35	18	25	17	1.00	1.01	–	–	19

Note: NS = natural slope and RW = retaining wall.

[a] Fills located on sloping foundations.

Figure 4. Time to failure of overconsolidated clays and shales as a function of liquidity index.



Peak and Residual Shear Strengths

Bjerrum (4) assembled shear strength data on a number of first-time failures of natural and cut slopes in overconsolidated clays and clay shales that showed that the average shear stress along the failure surface was much lower than the shear strength measured from laboratory triaxial tests. In each case, the peak shear strength parameters (ϕ_p' and c_p') were higher than the back-computed parameters (ϕ_c' and c_c'), which are assumed equal to zero, and, therefore, the safety factors were too high. The liquidity indexes of these clays ranged from -0.51 to 0.25. Discrepancies between the field (back-computed) and laboratory strengths are shown in Figure 3. The back-computed effective stress angle of shearing resistance is plotted as a function of the peak effective stress parameter obtained from triaxial tests. Even though cohesion was not considered, the data plot below the line of equality. If residual shear strengths are used, there is better agreement between thre computed shear strengths and those determined by direct shear tests.

Use of the residual shear strength parameters (ϕ_r' and c_r') in effective stress analyses does not necessarily yield safety factors that are in agreement with the actual safety factor at failure, although the error in the safety factor based on residual strength is generally smaller than the error in the safety factor based on peak strength. Table 1 gives a number of well-documented embankment, cut slope, and natural slope failures based on the effective stress analysis summarized and arranged according to increasing values on the liquidity indexes. Except for the case by D'Appolonia and others (7), all cases are first-time failures. Those case records clearly show that the effective stress analysis based on residual strength generally gives safety factors that are less than one. All of those cases, except for the last two failures given in the table, involve soils that have liquidity indexes less than 0.36. Also, the effective stress analysis based on peak strength yields safety factors that are too high and may be as much as 100 percent in error. Additionally, the use of c_p' equal to zero and ϕ_p' does not always yield the correct safety factor.

Evaluation of Pore Pressures

If the excess and initial pore pressures are known when designing a cutting in a clay or an embankment on a clay foundation, the stability of these earth structures may be determined during or at any time after construction piezometers are installed to obtain the necessary pore pressure data, the effective stress analysis is limited pore pressures are difficult, and the results obtained from such methods are highly questionable [Moh and others (14]. Additionally, determination of stability of the cut or embankment at any time requires that dissipation of excess pore pressures must be estimated, and these estimations are generally based on the results of consolidation tests that may be inaccurate. Unless percent in error. Additionally, the use of c_p' equal to zero and ϕ_p' does not always yield the correct safety factor. to analyzing the long-term stability of cuts and embankments. For this condition, the excess pore pressures are assumed equal to zero. In the case of a cutting in clay, the pore pressures are obtained from a prediction of the steady seepage pore pressures. In the case of an embankment, the pore pressures are usually obtained from groundwater level observations in boreholes. If large fluctuations in groundwater levels may exist, then pore pressure data may be inaccurate. If the embankment is located on a sloping foundation and damming of the groundwater may occur, then prediction of the steady seepage pore pressures is difficult, especially where large fluctuations of the groundwater level may occur.

If a valid comparison between field and laboratory shear strengths in terms of effective stress is to be made, then accurate values of pore pressures existing at the time of failure must be known. The back-computed shear strength parameters (ϕ_o' and c_o') are particularly sensitive to the magnitude of the pore pressures used in the computation. Inaccurate pore pressures may produce an error of several degrees in the computed parameter (ϕ_o'). An accurate determination of the pore pressures in a landslide at failure poses certain difficulties. Even when piezometers are installed, measurements obtained may not correspond to the pore pressures existing at the time of failure, particularly when the failure is preceded by a heavy rainfall and field personnel may not be present at the time of failure. For delayed failures in which several years may be required for the pore pressures to reach the steady-state values, use of measured pore pressures obtained before pore pressure equalization has occurred will lead to computed parameters (ϕ_o' and c_o') that cannot validly be compared to laboratory shear strength parameters.

Slope Design Dilemma

Observations (4) suggest the rate of development of a continuous sliding surface in a clay slope before failure varies from one type of clay to another. In the stiffer clays, the rate may be very slow; delay of the failure may be in years. The data shown in Figure 4 suggest that, for clay soils having liquidity indexes less than approximately -0.1 to -0.2 (very stiff clays), the failure delay may be several years. In slopes where the liquidity indexes are higher, the delay in failure may be very short.

Because the critical-state shear strength of overconsolidated clays cannot readily be determined, a practical approximation to the critical state might be obtained from triaxial tests performed on normally consolidated samples remolded at a water content (w_o) as follows:

$$w_c = (0.36)PI + PL \quad (5)$$

where PL is the plastic limit and the constant 0.36 is the liquidity index at the break point shown in Figure 2.

SUMMARY AND CONCLUSIONS

1. If the stress history and moisture state of the clay are not regarded, then application of the ϕ-equal-zero analysis for designing an embankment on a clay foundation or a slope cut in a clay may lead to erroneous conclusions concerning the stability of the slope. For clays having a liquidity index equal to or greater than approximately 0.36, ϕ-equal-zero analysis based on laboratory undrained strengths will yield fairly reliable safety factors, provided the liquid limit and plasticity index of the clay are equal to or below values of about 80 and 30 percent respectively. For clays having a liquidity index below a value of about 0.36, ϕ-equal-zero analysis will probably yield safety factors that are too high. The reliability of the high safety factors may depend on the liquidity index of the clay. For clays having a liquidity index less than about -0.1, the time to failure may vary from a few days or months to several years. If high safety factors are obtained from a ϕ-equal-zero analysis, then Figure 2 should be reviewed to evaluate the probable success of the slope design. The stability of the slope might be checked by using the corrected undrained shear strength given by the empirical relation in Equation 4.

2. The use of uncorrected vane shear strength to determine the stability of an embankment on a soft foundation, cut slopes, footings, and loading tests may yield unreliable results. The vane shear strength should be corrected by the empirical relation in Equation 1.

3. The liquidity index appears to be a general indicator of the stress history of a clay. Clays having a liquidity index less than about 0.36 might be considered to be overconsolidated while clays having a liquidity index greater than 0.36 might be considered normally consolidated.

4. The use of residual shear strength may be too conservative and expensive in many slope design problems involving overconsolidated clay, especially in cases in which temporary cuts are made. However, the use of peak shear strength in such soils may be unreliable and unsafe. The intermediate shear strength obtained from triaxial tests performed on normally consolidated clays remolded to a water content given by Equation 5 might provide a practical value for use in designing slopes against first-time failures.

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State-of-the-Art Report on Field Instrumentation for Pavement Experiments

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The various instruments available for taking in situ measurements of stress, strain, deflection, temperature, pore pressure, soil suction, and axle load in pavement experiments are described. Discussions of the desirable objectives of pavement experiments and comments about instrumentation concerning the unavailability of general purpose equipment and the need to design instruments for specific applications are presented. Earth pressure cells are discussed and information is given on the theory of their in situ performance, design considerations, and installation procedures. The need for correct calibration is emphasized. Pressure cells that have been used in various projects are described as illustrative of the kinds of instruments that can be used. A detailed design procedure for the simplest type of cell and a discussion concerning the use of charts to assist with the calculations are provided. Strain measuring devices for use in soils, granular materials, and asphalt materials are described and comments are made on the relative merits of each. Even though there is a lack of available information about other instrumentation, the current state of the art is described for each case presented.

Developments in analytically based design procedures for flexible pavements have reached the stage in which information about theoretical analysis and material properties is available. Furthermore, design methods that use the information dealing with the traffic-associated failure mechanisms of cracking and rutting have been reported in some detail in the literature. However, in practice the situation is that design agencies still use empirical methods based on the findings from limited, full-scale test sections and often incorporate an overall California bearing ratio (CBR) thickness requirement.

The gap between current practice and current research knowledge available can best be bridged by more agencies carrying out well-instrumented, full-scale experiments that are properly planned to monitor for correct parameters. In short, recent research developments must be verified in practice on a scale larger than that previously used so that the economic worth of these developments can be assessed by highway engineers.

The success of full-scale or pilot-scale experiments in assisting with the development of improved design procedures depends to a large extent on the accuracy of the measurements made on the structure. This success can only be achieved by use of adequate instrumentation that is installed correctly. This report explains the principles of the various instruments that can be used in pavement experiments and describes many of those that are successful in practice. The emphasis is on stress-measuring devices because these have been researched in the past. Instrumentation for evaluating in situ stress, strain, deflection, temperature, pore pressure, soil suction, and axle load is discussed. Because the amount and kind of instrumentation depends on the objectives of the experiment and the money available, this report can be used to assist engineers in planning future experiments.

Instrumented pavement experiments, particularly on public highways, should not be undertaken lightly. These experiments can be expensive both in terms of instrumentation and labor as well as in interruptions to the normal processes of construction. It is better to use fewer instruments that are well understood and reliable to provide good but limited data, than to use a vast array of ironmongery whose behavior is something of a mystery. Currently, field instrumentation is definitely not a matter of buying commercial equipment that can be easily installed in the road and expected to produce quick and reliable answers. While there is some equipment commercially available for measuring some parameters, this equipment should only be used with a full understanding of its operating principles because it is rare that field instruments have universal applicability. These instruments generally need to be designed for a purpose.

Many of the difficulties and costs of full-scale experiments on public highways can be avoided by using pilot-scale experiments or full-scale trials on special test roads. Many more projects involving these more carefully controlled experiments seem desirable because, if new design concepts do not work under such