Virginia Polytechnic Institute
and State University

The Charles E. Via, Jr.
Department of
Civil and Environmental Engineering

CENTER FOR
GEOTECHNICAL PRACTICE AND RESEARCH

AN ENGINEERING MANUAL FOR SLOPE STABILITY STUDIES

by

J. M. Duncan

A. L. Buchignani

and Marius De Wet

Report of a study performed by the Virginia Tech Center for Geotechnical Practice and Research

March, 1987

Center for
Geotechnical Practice and Research
200 Patton Hall
Blacksburg, VA 24061

Virginia Tech

VIRGINIA TECH

DEPARTMENT OF CIVIL ENGINEERING

AN ENGINEERING MANUAL FOR SLOPE STABILITY STUDIES

by

J.M. Duncan[1]

A. L. Buchignani[2],

and Marius De Wet[3]

March 1987

[1]W. Thomas Rice Professor of Civil Engineering, Virginia Tech, Blacksburg, Virginia

[2]Consulting Engineer, Mill Valley, California.

[3]Instructor, Department of Civil Engineering, University of Stellenbosch, South Africa

TABLE OF CONTENTS

INTRODUCTION

The purpose of this manual is to provide a simple, practical guide for slope stability studies. It is concerned with (1) the characteristics and critical aspects of various types of slope stability problems, (2) geologic studies and site investigation procedures, (3) methods of designing slopes, including field observations and experience, slope stability charts, and detailed analyses, (4) factors of safety, and (5) methods of stabilizing slopes and slides.

The emphasis of this manual is on simple, routine procedures. It does not include advanced analysis procedures, nor does it deal with specialized problems such as design of dams or the stability of slopes during earthquakes. References are given to the sources of the material contained in the manual, and to more advanced procedures where appropriate, to provide avenues for studies going beyond the scope of this manual.

The first edition of this manual was written under contract with the U.S. Army Corps of Engineers Waterways Experiment Station in 1975. This edition incorporates changes to reflect improvements in the state-of-practice since 1975, and to provide clearer explanations of some of the analysis procedures.

CHARACTERISTICS AND CRITICAL ASPECTS OF VARIOUS TYPES OF SLOPE STABILITY PROBLEMS

Cohesionless Fills Built on Firm Soil or Rock. The stability of fill slopes built of cohesionless gravels, sands, and silts depends on (a) the angle of internal friction of the fill material, Φ' , (b) the slope angle, and (c) the pore pressures. The critical failure mechanism is usually surface ravelling or shallow sliding, which can be analyzed using the simple infinite slope analysis.

Values of Φ' for stability analyses can be determined by drained triaxial or direct shear tests, or by correlations with grain size distribution, relative density, and particle shape. Pore pressure due to seepage through the fill reduces the stability of the slopes, but static water pressure, with the same water level inside and outside the slopes, has no effect on stability.

Slopes in fine sands, silty sands, and silts are susceptible to erosion by surface runoff; benches, paved ditches, and planting on slopes can be used to reduce runoff velocities and retard erosion. Saturated slopes in cohesionless materials are susceptible to liquefaction and flow slides during earthquakes, and dry slopes are subject to settlement and ravelling; relative densities of 75% or larger are required to insure seismic stability under most conditions.

Cohesive Fills Built on Firm Soil or Rock. The stability of fill slopes built of cohesive soils such as clays, clayey sands, and clayey gravels depends on (a) the strength of the fill, as characterized by the parameters c and ϕ or c' and Φ', (b) the unit weight of the fill, (c) the height of the fill, (d) the slope angle, and (e) the pore pressures. The critical failure mechanism is usually sliding on a deep surface tangent to the top of the firm foundation.

For fills built of cohesive soils which drain slowly, it may be necessary to analyze the stability for a number of pore pressure conditions:

(1) <u>Short-term or end-of-construction conditions</u>. This can be analyzed using total stress methods, with strengths determined in unconsolidated-undrained (U-U or Q) triaxial compression tests on specimens compacted to the same density and water content as in the field. The tests should be conducted using the same range of stresses as will occur in the field.

Internal pore pressures are not considered explicitly in such analyses; the effects of pore pressures in the undrained tests are reflected in the values of the strength parameters c and Φ. The pore pressures in compacted cohesive soils loaded under undrained conditions depend primarily on the density, the water content, and the applied total stresses. If laboratory specimens are compacted to the field density and water content, and loaded under undrained conditions, the pore pressures induced in the specimen will be the same as the short-term pore pressures in the field at locations where the total stresses are the same. The use of total stress strength parameters therefore accounts properly for pore pressure effects in short-term, undrained conditions.

External water pressures have a stabilizing effect on slopes, and should be taken into account in both total stress and effective stress analyses of all types of slopes.

(2) <u>Long-term conditions</u>. This condition can be analyzed using effective stress methods, with strength parameters determined in drained (D or S) triaxial or direct shear tests, or consolidated-undrained (C-U or $\overline{R}$) tests with pore pressure measurements, on specimens compacted to field density and water content, and tested in the range of stresses that will occur in the field. The measured strengths are related to the effective stresses by means of the strength parameters c' and Φ'.

Pore pressures are governed by the seepage conditions, and can be determined using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analyses.

(3) Rapid drawdown condition, or other conditions where the slope is consolidated under one loading condition, and is then subjected to a rapid change in loading, with insufficient time for drainage. This condition can be analyzed using total stress methods, with strengths measured in consolidated-undrained (C-U or R) triaxial compression tests on specimens compacted to field density and water content. The undrained strengths are related to the consolidation pressures as shown in Fig. 1, without using values of c and Φ'.

Stability analyses are performed by determining, for each point through which a trial failure surface passes, the effective stress before drawdown or change in loading. This effective stress is the consolidation pressure, which determines the undrained strength at the point. The undrained strength is determined from the strength diagram. When the undrained strength has been determined for each point along the trial failure surface, the stability is analyzed using total stress methods. See Lowe and Karafiath (1960) for further explanation of the procedure.

Pore pressures are not considered explicitly in such analyses. Pore pressure effects are accounted for by the relationship between undrained strength and consolidation pressure.

Fills Built on Soft Subsoils. The stability of fill slopes built on soft subsoils depends on (a) the strength of the fill, as characterized by the parameters c and Φ or c and Φ', (b) the unit weight of the fill, (c) the height of the fill, (d) the slope angle, (e) the strength of the foundation, as characterized by the parameters c or c' and Φ or Φ', and (f) the pore pressures. The critical failure mechanism is usually sliding on a deep surface tangent to the top of a firm layer within the foundation. A large part of the failure surface usually lies within the foundation, especially in cases where the soft subsoils extend to great depths, and the stability of the embankment depends to a large extent on the strength of

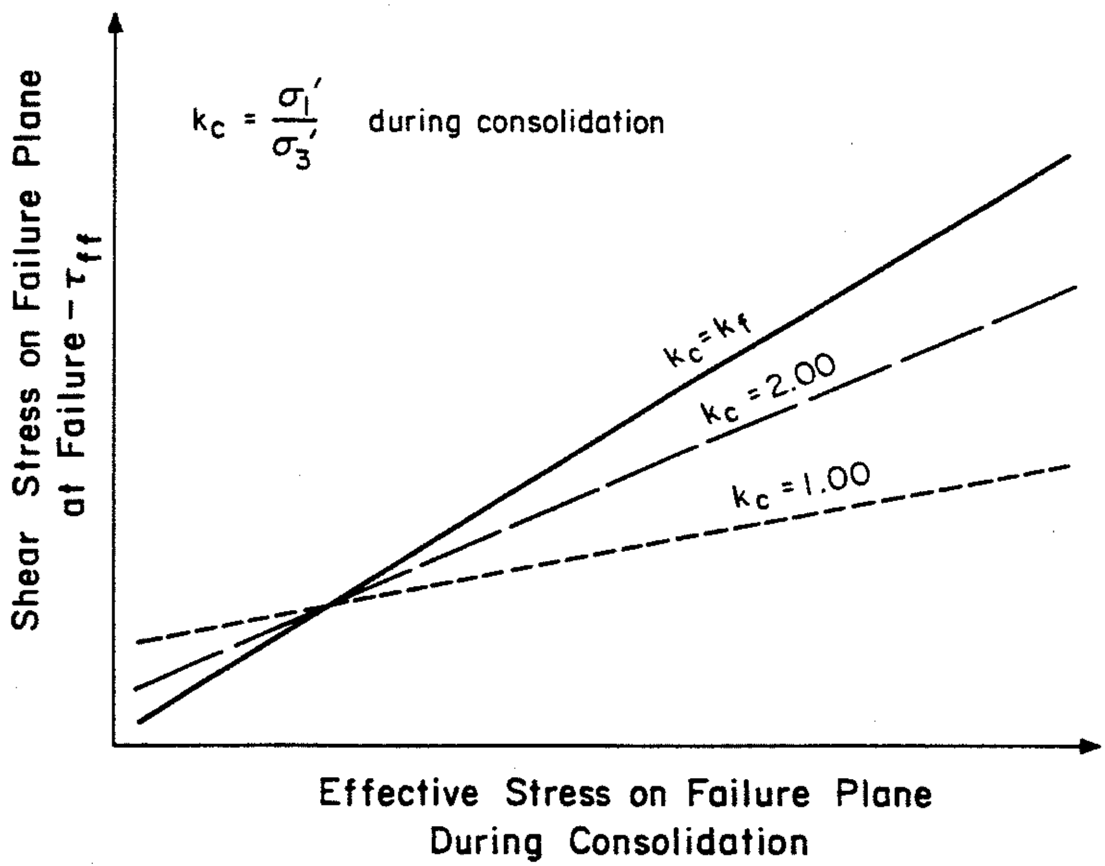


Fig. 1 VARIATION OF UNDRAINED STRENGTH WITH CONSOLIDATION PRESSURE (AFTER LOWE AND KARAFIATH, 1960)

the foundation soils. Surface ravelling may also occur in cohesionless fills.

The short-term stability of embankments on soft subsoils is usually more critical than the long-term stability, because the foundation soil consolidates under the weight of the embankment and gains strength over a period of time. It may, however, be necessary to analyze the stability for a number of pore pressure conditions:

(1) Short-term or end-of-construction condition. If the fill is free-draining sand or gravel, the strength of the fill should be treated in terms of effective stresses. Values of ϕ' for use in the analyses should be determined from drained triaxial or direct shear tests, or by correlations with grain size distribution, relative density, and particle shape. Pore pressures in a free-draining fill are governed by the seepage conditions, and can be determined using flow nets or other types of seepage analysis.

If the fill is built of cohesive soil which drains slowly, the strength of the fill for short-term analyses should be treated in terms of total stresses. The strength of the fill material can be determined by performing unconsolidated-undrained (U-U or Q) triaxial compression tests on specimens compacted to the same density and water content as in the field. The test pressures should encompass the range of pressures that will occur in the field.

Soft clay foundations ordinarily drain so slowly that there is little or no dissipation of excess pore pressures during construction. For such conditions, the strength of the clay should be treated in terms of total stresses, and its strength determined using unconsolidated-undrained (U-U or Q) triaxial compression tests on undisturbed specimens, conducted using pressures in the range of pressures in the field.

Under undrained conditions, the strengths of saturated clays can be expressed as

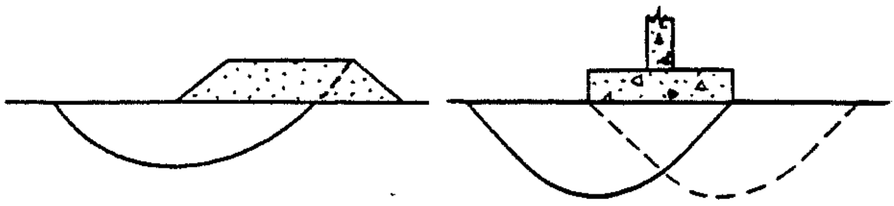
$$S_u = c_u$$

$$\Phi_u = 0$$

in which S_u = undrained shear strength, which is independent of total normal stress, c_u = undrained cohesion intercept of the Mohr-Coulomb failure envelope, and Φ_u = undrained friction angle. The strengths of clays which conform to this $\Phi_u = 0$ failure criterion may be measured using unconsolidated-undrained triaxial tests, unconfined compression tests, or vane shear tests. Strength values measured using field vane shear tests should be corrected for the effects of anisotropy and strain rate using Bjerrum's correction factor, μ , which is shown in Fig. 2.

Embankments on soft foundations may fail progressively because of differences in the stress-strain characteristics of the embankment and the foundation. The strengths of both the embankment and the foundation should be reduced to allow for progressive failure effects, using the reduction factors R_E and R_F shown in Fig. 3. The use of strength parameters reduced by these factors will ensure that neither the embankment nor the foundation are stressed so highly that progressive failure can begin. Even when the strength reduction factors R_E and R_F are used, a factor of safety greater than unity should be used to account for possible inaccuracies in measuring shear strengths.

Internal pore pressures are not considered explicitly in total stress analyses, but the effects of the pore pressures in the undrained tests are reflected in the values of c and Φ . If the laboratory specimens are representative of the soils in the field, the pore pressures in the laboratory specimens will be the same as the pore pressures in the field at locations where the total stresses are the same, and the use of total stress strength parameters from undrained tests therefore accounts properly for pore pressure effects in short-



$$(s_u)_{field} = (s_u)_{vane} \cdot \mu$$

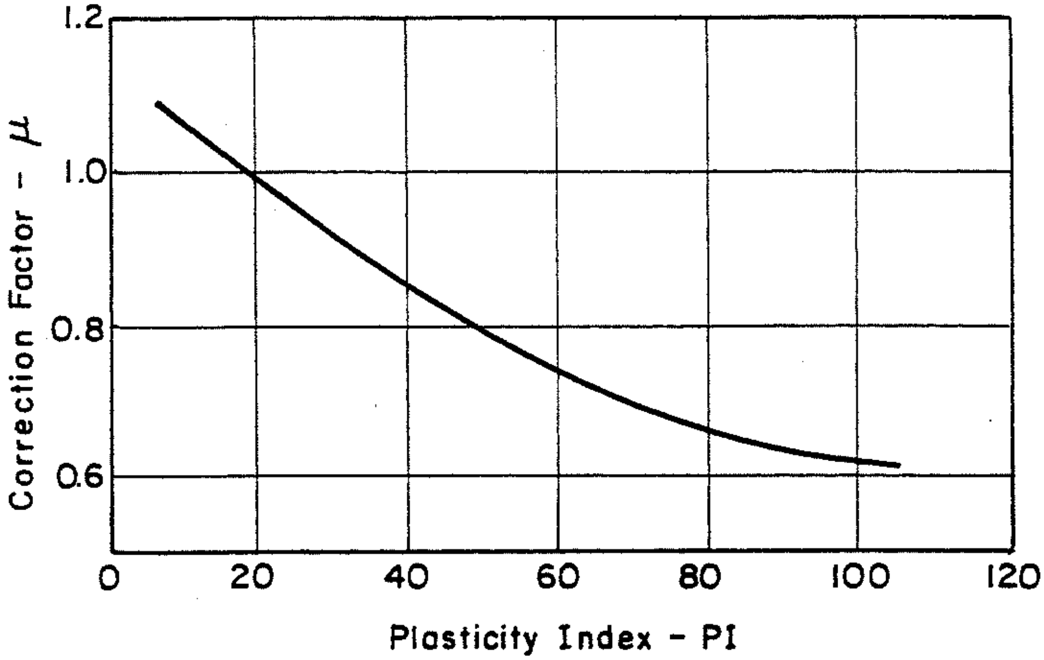


Fig. 2 CORRECTION FACTOR FOR VANE STRENGTH.
(after Bjerrum, 1973)

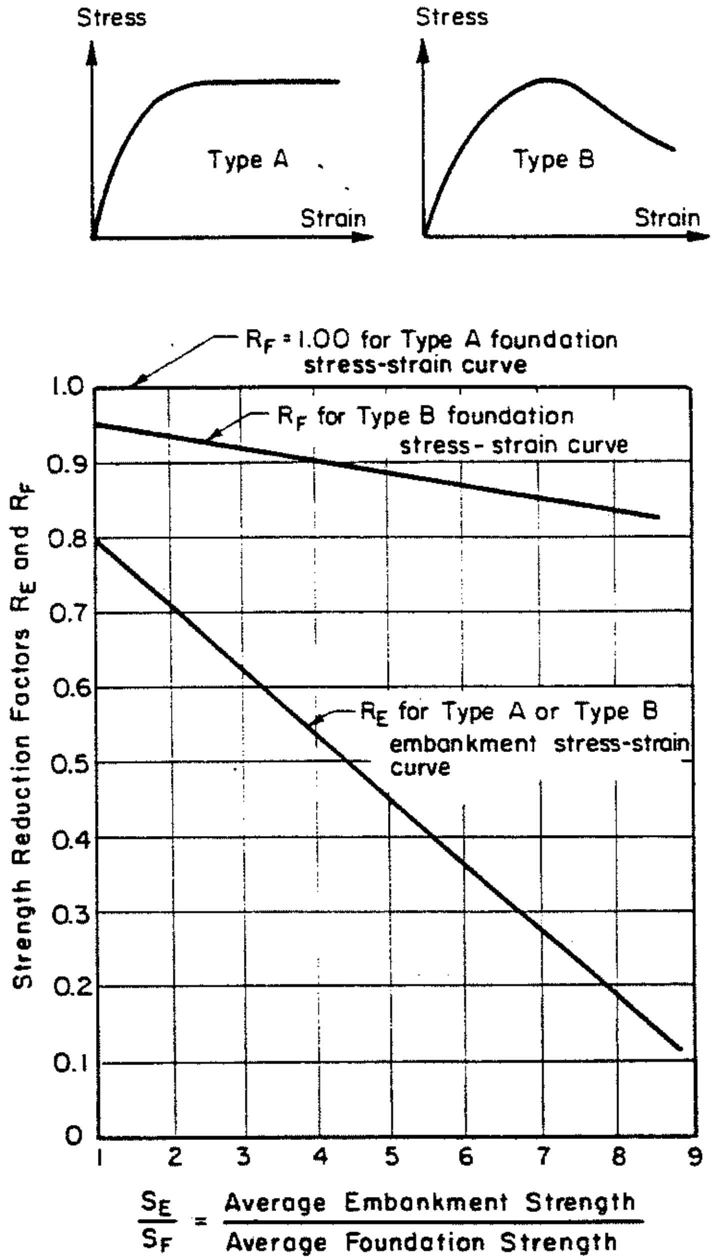


Fig. 3 CORRECTION FACTORS R_E AND R_F TO ACCOUNT FOR PROGRESSIVE FAILURE IN EMBANKMENTS ON SOFT CLAY FOUNDATIONS.
(after Chirapuntu and Duncan, 1975)

term, undrained conditions.

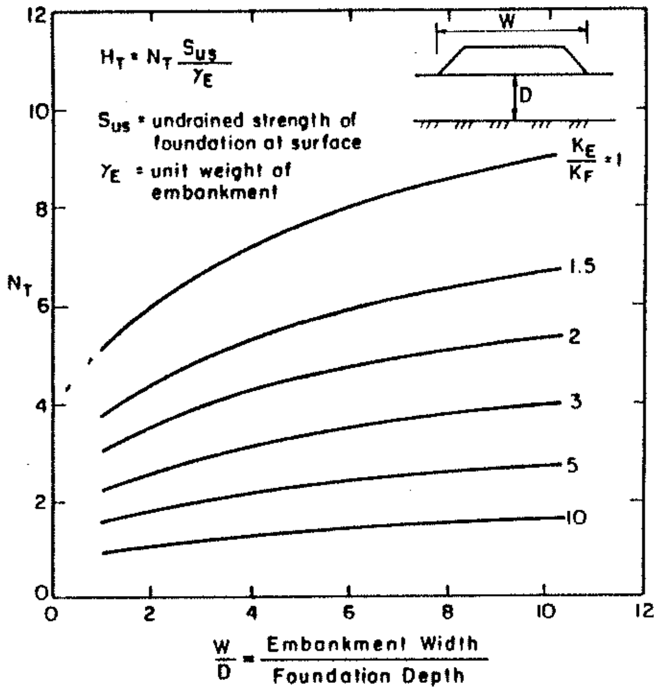
External water pressures should be taken into account in the stability analyses, whether they are performed in terms of total or effective stresses.

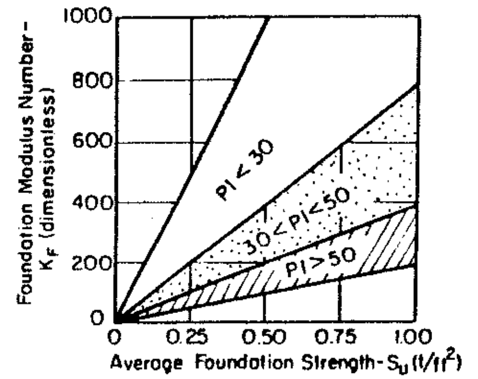
If an embankment constructed of cohesive fill is built higher than some critical height, H_T, there will be a tendency for tension to develop in the embankment, and the embankment will crack. The approximate value of H_T can be determined using Fig. 4. Embankments which are built higher than this critical height should be analyzed assuming that the fill is cracked to a depth of

$$H_c = \frac{4c}{\gamma} \tan (45 + \Phi/2)$$

in which H_c = crack height, c = cohesion intercept of embankment fill, Φ = friction angle of embankment, and γ = unit weight of embankment. If H_c exceeds the embankment height, the crack should be assumed to extend through the full height of the fill, but not into the foundation. In performing stability analyses of cracked embankments, it should be assumed that cracks may exist in any part of the embankment. Each trial failure surface analyzed should be assumed to intersect a crack in the embankment, and zero shear resistance should be assigned to the vertical portion of the failure surface which coincides with the crack.

(2) Long-term conditions. This condition can be analyzed using effective stress methods, with strength parameters for both the fill and foundation determined in drained (D or S) triaxial or direct shear tests or consolidated-undrained (C-U or $\overline{R}$) tests with pore pressure measurements. The test specimens of the foundation soil should be undisturbed, and the test specimens of the fill material should be compacted to the field density and water content and tested in the range of pressures that will occur in the field.





Typical values of K_E for compacted fills

Unified Class.	Compaction Water Content		
	Optimum - 3%	Optimum	Optimum + 3%
GC	300 - 1200	200-500	75 - 300
SP	400 - 1000	400-1000	400-1000
SM	300 - 750	300 - 750	300 - 750
SC	250 - 1000	150 - 600	50 - 250
ML	250 - 1000	150 - 600	50 - 250
CL	250 - 1000	100 - 400	30 - 200
CH	100 - 400	50 - 200	20 - 100

Values shown apply to fill materials compacted to dry densities from 90% to 95% of the Std. AASHO maximum. In general, the value of K_E increases with increasing dry density at a given water content.

Fig 4 CHART FOR ESTIMATING H_T = HEIGHT OF EMBANKMENT WHEN CRACKING WILL BEGIN.
(after Chirapuntu and Duncan, 1975)

Pore pressures are governed by the seepage conditions, and can be determined using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analyses.

(3) Rapid drawdown condition, or other conditions where the slope is consolidated under one loading condition, and is then subjected to a rapid change in loading, with insufficient time for drainage. This condition can be analyzed using total stress methods, with strengths for both the embankment and the foundation measured in consolidated-undrained (C-U or R) tests. Test interpretation and analysis procedures for these conditions have been described by Lowe and Karafiath (1960) and Ladd and Foote (1974).

Table 1 summarizes the important aspects of the stability of slopes in fills, and the applicable analysis procedures.

Excavation Slopes. The stability of excavation slopes depends on (a) the strength of the soil in which the slope is excavated, as characterized by the strength parameters c and Φ or c' and Φ', (b) the unit weight of the soil, (c) the height of the slope, (d) the slope angle, and (e) the pore pressures. The critical failure mechanism is usually a deep surface in homogeneous cohesive soils, and surface sloughing or shallow sliding in homogeneous cohesionless soils. In nonhomogeneous slopes the critical shear surface may be either shallow or deep, depending on the strength characteristics and distributions of the soils within the slope.

The long-term stability of excavation slopes in cohesive soils is usually more critical than short-term stability, because the soil around the excavation swells under the reduced stresses and becomes weaker over a period of time. It may, however, be necessary to analyze the stability of excavated slopes for a number of pore pressure conditions:

(1) Short-term or end-of-construction conditions. If the slope is excavated wholly or partly in free-draining sands or gravels, in which there are no excess pore pressures at the end of construction,

Table 1. Important Aspects of the Stability of Compacted Fills

	Type of fill and foundation		
	Cohesionless fill on firm foundation	Cohesive fill on firm foundation	Any type of fill on weak foundation
Factors that control stability	Φ' of fill Slope angle Pore pressures External water	Strength of fill γ of fill Slope angle Slope height Pore pressures External water	Strength of foundation Depth of weak foundation layer Strength of fill γ of fill Height of fill Slope angle Pore pressures External water
Failure mechanism	Surface ravelling	Sliding tangent to top of foundation	Deep sliding extending into foundation
Special problems	Surface erosion Liquefaction during earth-quakes	Surface erosion Weathering and weakening of Compacted shales	Embankment cracking Progressive failure Surface erosion
Critical stages for stability	Long-term, or Earthquake	End-of-Construction, Longterm, or Rapid drawdown	End of construction, Longterm, or Rapid drawdown
Analysis procedures	Effective stress, or Dynamic	Total stress, Effective stress or, Combination	Total stress, Effective stress, or Combination

their strengths should be treated in terms of effective stresses. Values of Φ' for these soils should be determined from drained triaxial or direct shear tests performed using an appropriate range of stresses, or by correlations with grain size distribution, relative density, and particle shape. Pore pressures in these free-draining soils will be governed by the seepage conditions, and can be determined using flow nets or other types of seepage analysis.

If the slope is excavated wholly or partly in cohesive soils which drain slowly, their strengths for short-term analyses should be treated in terms of total stresses. The strengths of these soils can be determined by performing unconsolidated-undrained (U-U or Q) triaxial compression tests on undisturbed specimens, and can be expressed in terms of the total stress shear strength parameters Φ_u and c_u.

Under undrained conditions, the strengths of saturated clays can be expressed as

$$S_u = c_u$$

$$\Phi_u = 0$$

in which S_u = undrained shear strength, which is independent of total normal stress, c_u = undrained cohesion intercept of the Mohr-Coulomb failure envelope, and Φ_u = undrained friction angle. The strengths of clays which conform to this $\Phi_u = 0$ failure criterion may be measured using unconsolidated-undrained triaxial tests, unconfined compression tests, or vane shear tests. Strength values measured using field vane shear tests should be corrected for the effects of anisotropy and strain rate using Bjerrum's correction factor, μ, which is shown in Fig. 2.

Internal pore pressures are not considered explicitly in such analyses; the effects of the pore pressures in the undrained tests are reflected in the values of c and ϕ. The pore pressures induced in undisturbed laboratory specimens will be the same as the pore pressures in the field at locations where the total stresses are the same, and the use of total stress strength parameters from undrained tests therefore accounts properly for pore pressure effects in short-term, undrained conditions.

External water pressures should be taken into account in the stability analyses, whether they are performed in terms of total or effective stresses.

(2) Long-term conditions. This condition can be analyzed using effective stress methods, with strength parameters determined from drained (D or S) triaxial or direct shear tests, or consolidated-undrained (C-U or $\overline{R}$) tests with pore pressure measurements, performed on undisturbed test specimens at comparable stress levels.

Pore pressures are governed by the seepage conditions, and can be determined using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analyses.

(3) Rapid drawdown condition, or other conditions where the slope is consolidated under one loading condition, and is then subjected to a rapid change in loading, with insufficient time for drainage. This condition can be analyzed using total stress methods, with strengths determined in consolidated-undrained (C-U or R) tests on undisturbed specimens. Ladd and Foote (1974) have developed detailed procedures for evaluating the undrained strength of natural soils using C-U or R tests.

Table 2 summarizes the important aspects of the stability of excavated slopes, and the applicable design procedures.

Natural Slopes. If a natural slope is modified by grading operations, it may be necessary to analyze the stability for a number of pore pressure conditions. Fills on natural slopes may be treated in accordance with the

Table 2. Important Aspects of the Stability of Excavation Slopes

	Soil Type	
	Cohesionless	Cohesive
Factors that control stabilitry	ϕ' of soil Slope angle Pore pressures External water	Strength of soil γ of soil Slope angle Pore pressures External water
Failure mechanism	Surface ravelling	Deep sliding, possibly extending below toe of slope
Special problems	Surface erosion Liquefaction during earthquake	Strength loss in stiff-fissured clays Surface erosion
Critical stages for stability	Long-term, or earthquake	End-of-construction, Long-term, or Rapid drawdown
Analysis procedures	Effective stress, or Dynamic	Total stress, Effective stress, or Combination

procedures described previously for fill slopes, and excavations in natural slopes may be analyzed following the procedures described for excavation slopes.

If a natural slope has existed in the same condition for many years, and has come to equilibrium with the prevailing groundwater seepage conditions, it should be analyzed using effective stress procedures. Strengths can be determined using drained (D or S) triaxial or direct shear tests, or consolidated-undrained (C-U or $\overline{R}$) tests with pore pressure measurements. Pore pressures can be determined by field measurements or by using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analyses.

<u>Slopes in Soils Presenting Special Problems</u>. There are a number of different soil types which present special or unusual problems with respect to the stability of natural or excavated slopes. These include:

(1) <u>Stiff-fissured clays and shales</u>. The shearing resistance of many stiff-fissured clays and shales may be reduced considerably if they are subjected to shearing displacements which are much larger than the shear displacement corresponding to peak strength. There is evidence for some of these soils that slope failures may occur progressively, and that over a long period of time the shearing resistance may be reduced to the residual value--the minimum value which is reached only at extremely large shear displacements. In some cases, however, slopes in these types of soils stand for many tens or hundreds of years at angles which are steeper than would be consistent with mobilization of only residual shear strength. Local experience and local practice is the best guide to appropriate design procedure for such soils.

(2) <u>Loess</u>. Because loess deposits contain networks of interconnected channels formed by plant roots which have decayed, these deposits have high permeability in the vertical direction. Unless vertical infiltration is prevented, water percolating downward through the soil can destroy the weakly cemented bonds between particles, causing

rapid erosion and slope failure. Slopes in loess are therefore frequently more stable when cut vertical to prevent infiltration. Benches at intervals can be used to reduce the effective slope angle. Horizontal surfaces on benches and at the top and bottom of the slope must be paved or planted to prevent infiltration. Local experience and practice are the best guide to procedures for spacing benches and for protecting such slopes against infiltration and erosion.

(3) Residual soils. Depending on rock type and climate, residual soils in many areas may present special problems with respect to slope stability and erosion. Such soils may contain pronounced structural features characteristic of the parent rock or the weathering process, and their characteristics may vary quite significantly over short distances. Under such conditions, determination of design shear strength parameters from laboratory tests may be difficult. It may be possible in such conditions to determine more representative shear strength parameters by back-analyzing slope failures, or to use empirical design procedures based on local experience, without analysis.

(4) Highly sensitive clays. Some marine clays exhibit dramatic loss of strength when disturbed and can actually flow like syrup when completely remoulded. Because of the effects of disturbance during sampling, it may be difficult to measure representative strengths for such soils in laboratory tests. Local experience is the best guide to the reliability of laboratory shear strength values for such clays.

PROCEDURES FOR INVESTIGATION AND DESIGN OF SLOPES

Depending on the type of slope, and the amount of time and effort which can appropriately be devoted to site investigation and analysis, a number of different procedures may be used for investigation and design of slopes. Three frequently used procedures, which represent increasing levels of complexity and cost, are the following:

1. Use of field observations and experience alone, with no test borings, laboratory tests, or slope stability calculations.

2. Use of slope stability calculations by means of charts, in combination with field observations and a minimum number of test borings and laboratory tests.

3. Use of detailed slope stability calculations, in combination with a thorough program of site investigation and laboratory tests.

The use of field observations, slope stability charts, and detailed analysis procedures are discussed in the following sections.

<u>Field Observations</u>. Slopes are often designed based on observation and examination of existing slopes in the same area and the same types of soil. The use of field experience alone implies that no soil strength data from laboratory tests or back analysis is available. This procedure is appropriate when the costs of drilling test borings and making laboratory tests are high compared to the cost of repairing a slope failure.

The first steps in designing a slope based on field observation and experience is to review the available geologic maps and to make a geological reconnaissance of the area. In the geological reconnaissance, particular attention is devoted to evidence of seepage, the characteristics of the existing topography, and the condition of nearby cut or fill slopes. Seepage emerging from the ground indicates unfavorable conditions for cut or fill slopes, and the existence of a hummocky ground surface or old slide scarps is strong evidence of past instability. The type and condition of vegetation and ground cover should also be noted. Sparse vegetation or steeply leaning trees are often indications of slope stability problems.

A field slope chart, like the one shown in Fig. 5, is a useful tool for slope design based on experience. This type of chart summarizes field data for a particular geologic formation in a form which makes it useful for design of slopes in the same formation. To prepare such a chart, existing landslides and stable slopes are surveyed. Each failed or stable slope is plotted as a point on the chart, which shows slope angle (or cotangent of slope angle) versus height of slope. Each point is identified as stable or unstable. When the number of failures is sufficient, as was the case for the area which Fig. 5 applies, it is only necessary to include data for unstable slopes. When the number of failures is small, however, it is useful to include data for stable slopes, which would be represented by a different symbol. The data for the highest and steepest stable slopes will help to define the limits within which slopes can be expected to remain stable. By comparing these data with the height and slope angle for a planned slope, a prediction of the stability of the planned slope can be made based entirely on experience. For example, the data shown in Fig. 5 indicate that within the area it covers, almost all slope failures were in slopes steeper than 20°.

Stability Chart Solutions. Slope analysis by use of slope stability charts has commonly been used for the preliminary stage of analysis. However, charts are now available which make it possible to perform quite accurate analyses for many conditions. Charts which include the effects of surcharge, tension cracks, submergence, seepage, and increasing strength with depth allow a wide range of variables to be considered in the design of a slope by this method.

Using slope stability charts, the factor of safety for a slope can be calculated within an accuracy of 15% in most cases. Thus, when the available data on site conditions and soil strengths are not extensive, calculations using slope stability charts provide sufficient accuracy for design. Slope stability charts are also very useful for preliminary design

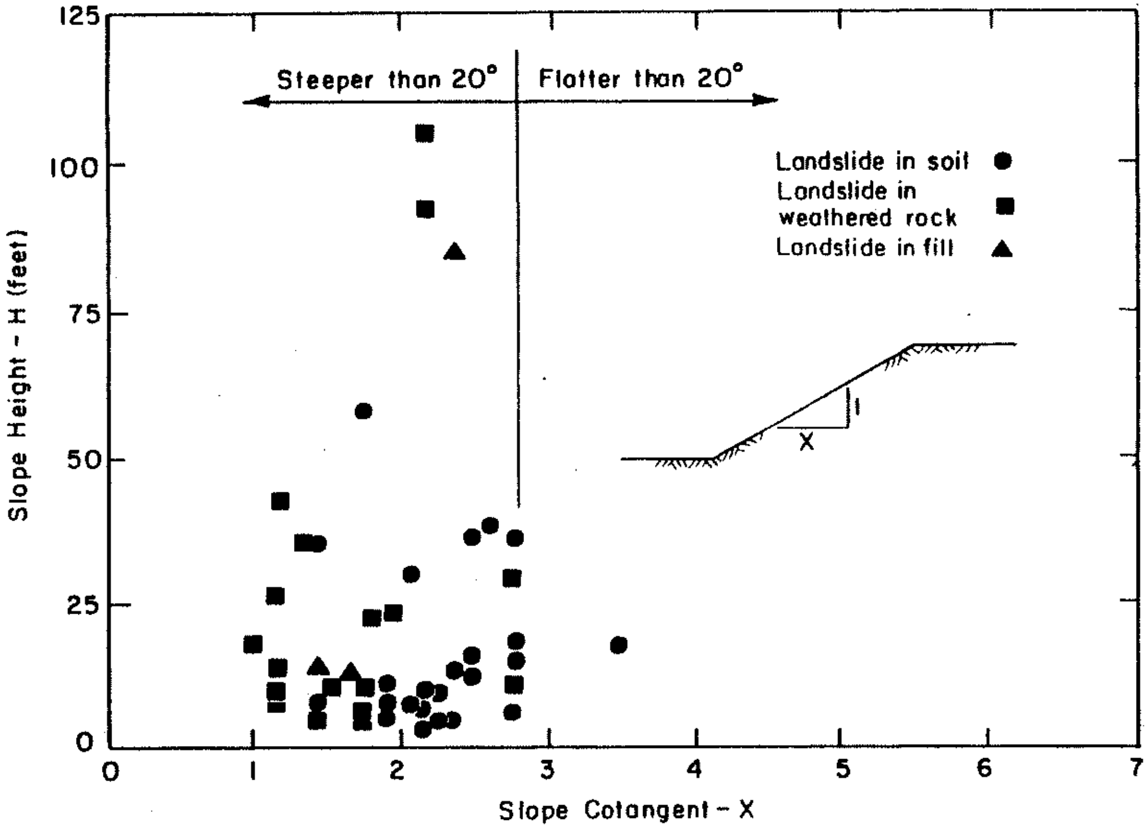


Fig. 5 EXAMPLE OF METHOD OF ESTIMATING SLOPE STABILITY USING FIELD DATA.

calculations, to compare alternatives which can be examined more thoroughly subsequently using detailed analysis procedures. Chart solutions also provide a rapid means of checking the results of detailed analyses.

A further use for slope stability charts is to back-calculate strength values for failed slopes to aid in planning remedial measures. This can be done by assuming a factor of safety of unity for the conditions at failure and solving for the unknown shear strength. Since soil strength usually involves both cohesion and friction, there is no unique value of cohesion (c) and angle of internal friction (Φ) which will give a factor of safety equal to unity; therefore, several pairs of values should be calculated and judgment used to select the most reasonable values. If the material in the slide zone is clay, and the slide occurred under undrained conditions, a unique solution for shear strength can be obtained by assuming $\Phi = 0$ and back calculating a value of cohesion.

Detailed Analysis. A detailed investigation of slope stability includes a geological study; field observations, test borings, laboratory testing, and detailed slope stability calculations. The analysis can be performed using a computer or detailed hand calculations as described in subsequent sections. Slope stability charts may be used for preliminary studies or to check the final analysis.

Field instrumentation studies can be used to monitor the performance of existing slopes, and may be very useful in conjunction with detailed investigations. Data from slope indicators can be helpful for determining potential or existing failure zones. Instrumentation can also be used to monitor the movements of a slope after construction as a check on the design. This is especially useful for sites with very complex soil conditions.

GEOLOGIC STUDIES AND SITE INVESTIGATION PROCEDURES

Detailed geologic studies and site investigations are used to obtain the information required for analysis of new slopes and for planning remedial work on landslides. The first step in a field exploration program is to make a geological reconnaissance, including field mapping of the area. Field notes should be recorded on a large-scale topographic map. The reconnaissance should make note of the uniformity of the topography, seepage, existing landslide scarps, plumbness of trees, and the condition of nearby slopes. Note should also be made of the accessibility of the site to drilling equipment. The locations of the test borings or test pits should be planned and staked in the field during this phase of the investigation.

A sufficient number of test borings should be planned so that detailed surface and subsurface data are obtained throughout the planned slope area. Sometimes, because of rugged topography, equipment access may be difficult and track-mounted equipment may be needed. Large diameter borings are preferable to smaller ones, especially when drilling in a known slide area, because it is often possible to locate the slide plane by examining the cuttings from a large diameter hole. Shear or slickensided zones in the cuttings provide evidence of past movements. Undisturbed samples should be taken at selected depths in the borings, and when a change in soil type is encountered. In the case of an existing slide, samples should be obtained within the failure zone, if possible. In some cases it may be necessary to have a geologist enter a bore hole and examine the sides of the hole for fault zones, evidence of movement or seepage. In this case, as a safety precaution, the hole should be cased. The depths of the borings should extend well below the toe of the slope and, if possible, should extend to a firm soil layer or bedrock.

To obtain water table information, borings should be fitted with perforated casings and backfilled with gravel so that long-term measurements of the fluctuations of the ground water can be made. Piezometers can also be installed at selected locations so that measurements of pore water pressure can be obtained. Sometimes piezometers can be installed in test borings after sampling has been completed.

Based on the reconnaissance and test boring data, surface and subsurface profiles should be drawn showing the soil conditions and water levels. The unit weights, classification test data, and strength data obtained from laboratory tests should be shown on the profiles.

SLOPE STABILITY CHARTS

The stability of slopes can be analyzed quickly using the stability charts shown in Figs. 6 through 11. Although the charts assume simple slopes and uniform soil conditions, they can be used to obtain reasonably accurate answers for most complex problems if irregular slopes are approximated by simple slopes, and carefully determined average values of unit weight, cohesion, and friction angle are used.

Charts for Slopes in Soils with Uniform Strength throughout the Depth of the Soil Layer, and $\Phi = 0$. The stability chart for slopes in soils with uniform shear strength throughout the depth of the layer, and with $\Phi = 0$, is shown in Fig. 6. Charts giving correction factors for surcharge loading at the top of the slope, submergence, and tension cracks are given in Figs. 7a and 7b.

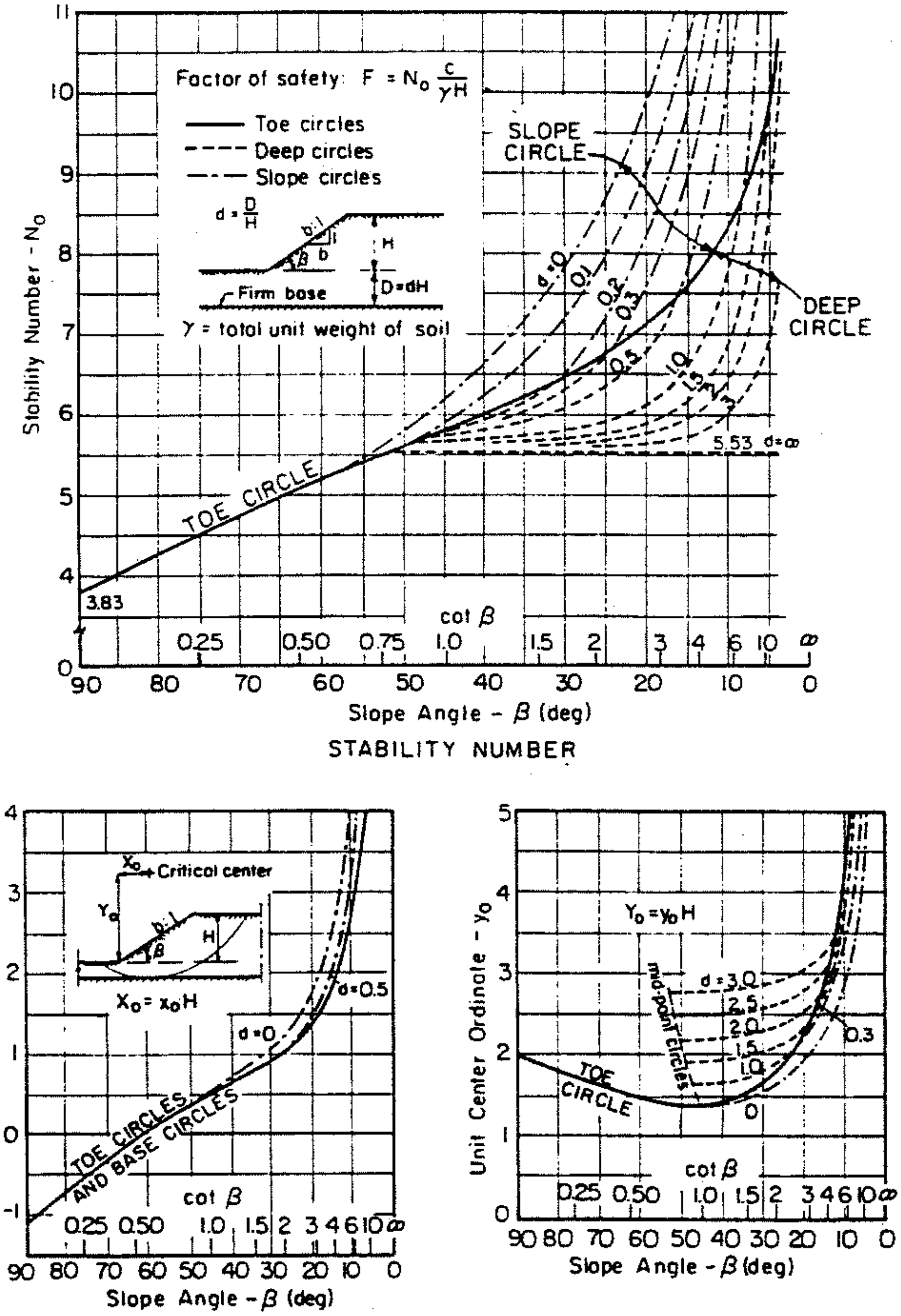
Steps for use of charts:

1. Using judgment, decide which cases should be investigated. For uniform soil conditions, the critical circle passes through the toe of the slope if the slope is steeper than about 1 (H) on 1 (V). For flatter slopes, the critical circle usually extends as deep as possible and is tangent to some deep firm layer.

   If the conditions are not uniform, or there is water outside the slope, the following criteria can be used to determine which possibilities should be examined:

   - If there is water outside the slope, a circle passing above the water may be critical;

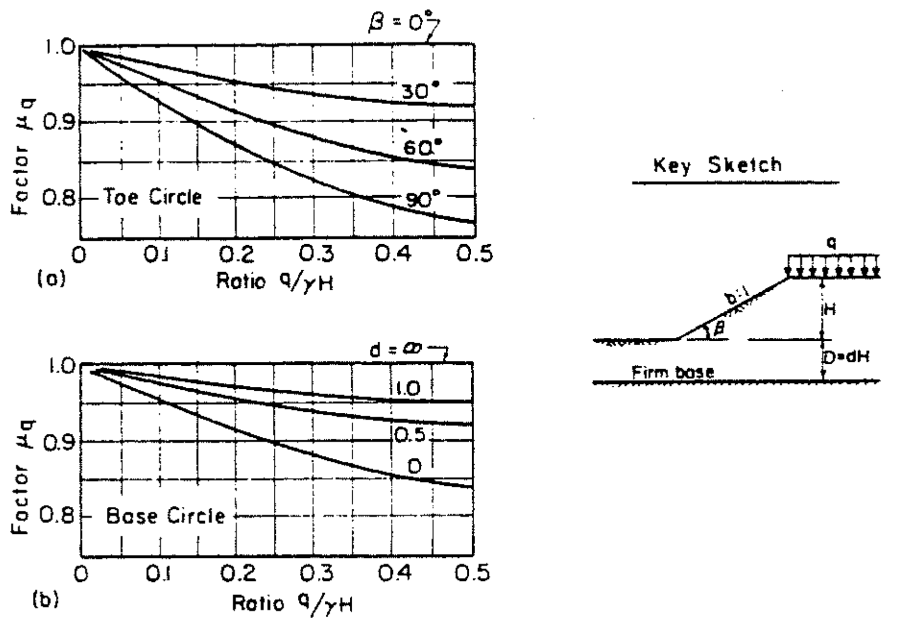
   - If a soil layer is weaker than the one above it, the critical circle may be tangent to the base of the lower (weaker) layer. This applies to layers above as well as below the toe;



CENTER COORDINATES FOR CRITICAL CIRCLE

Fig 6. SLOPE STABILITY CHARTS FOR $\phi = 0$ SOILS (after Janbu, 1968)

REDUCTION FACTORS FOR SURCHARGE



REDUCTION FACTORS FOR SUBMERGENCE (μ_w) AND SEEPAGE (μ'_w)

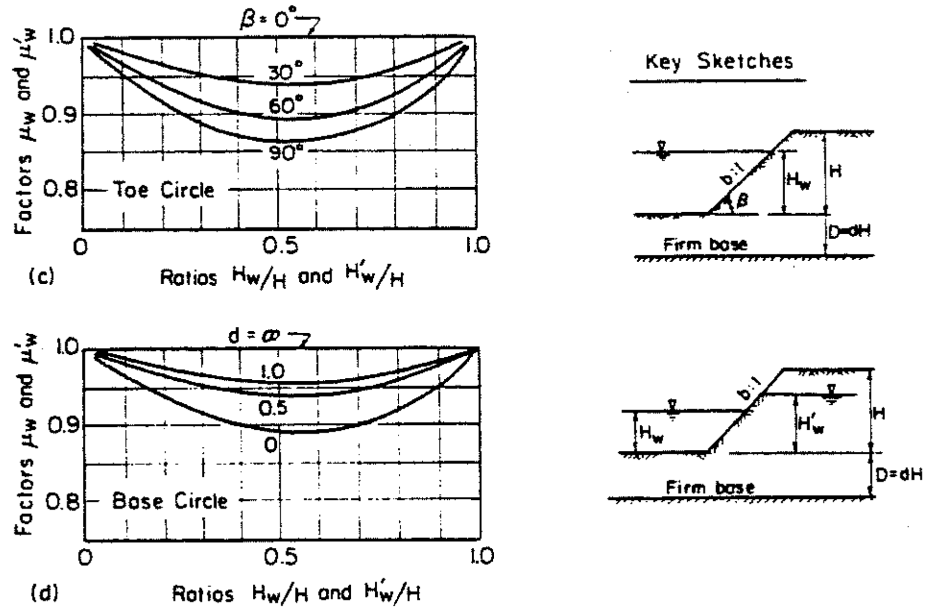
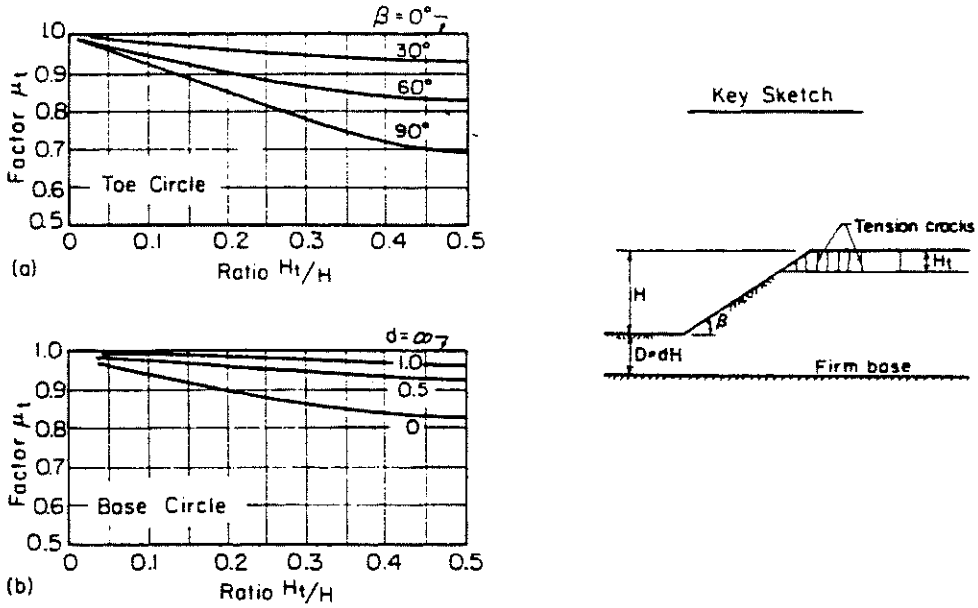


Fig. 7(a) REDUCTION FACTORS FOR SLOPE STABILITY CHARTS FOR $\phi = 0$ AND $\phi > 0$ SOILS. (after Janbu, 1968)

REDUCTION FACTOR FOR TENSION CRACK
No Hydrostatic Pressure in Crack



REDUCTION FACTOR FOR TENSION CRACK
Full Hydrostatic Pressure in Crack

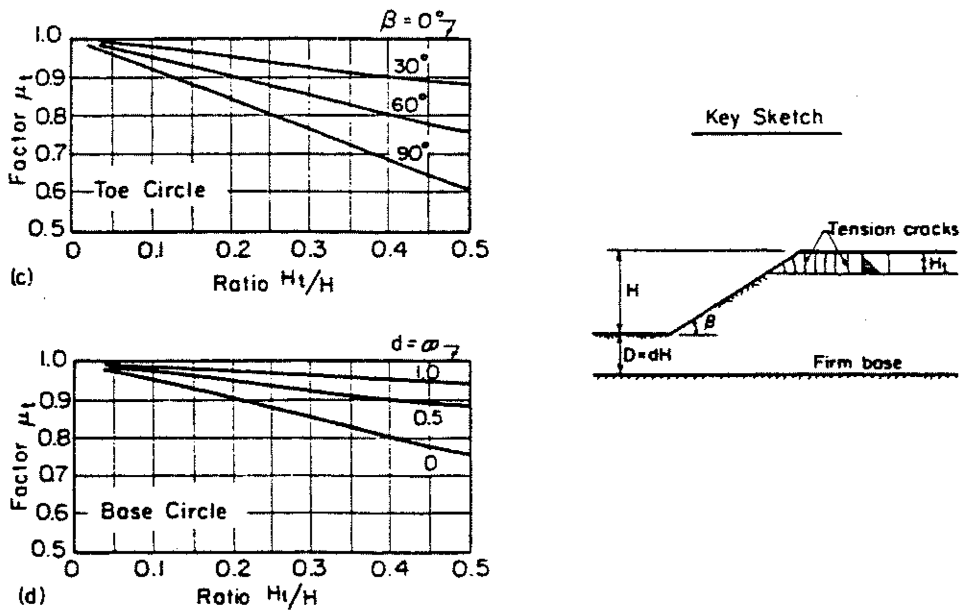
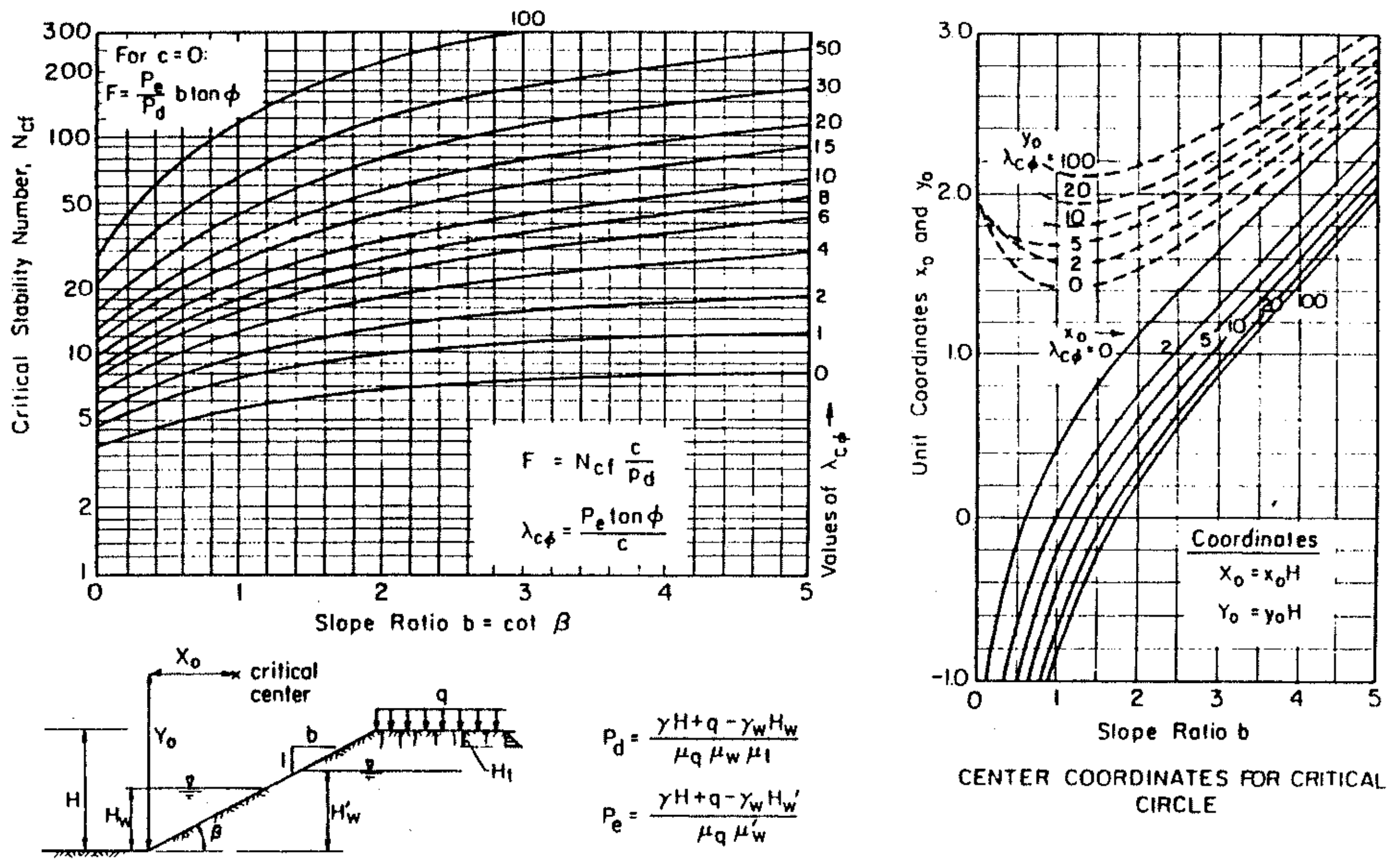


Fig. 7(b) REDUCTION FACTORS FOR SLOPE STABILITY CHARTS FOR $\phi = 0$ AND $\phi > 0$ SOILS. (after Janbu, 1968)



$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

$$P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu_w'}$$

(In formula for P_e take $q = 0$, $\mu_q = 1$ for unconsolidated condition)

Fig.8 SLOPE STABILITY CHARTS FOR $\phi > 0$ SOILS. (after Janbu, 1968)

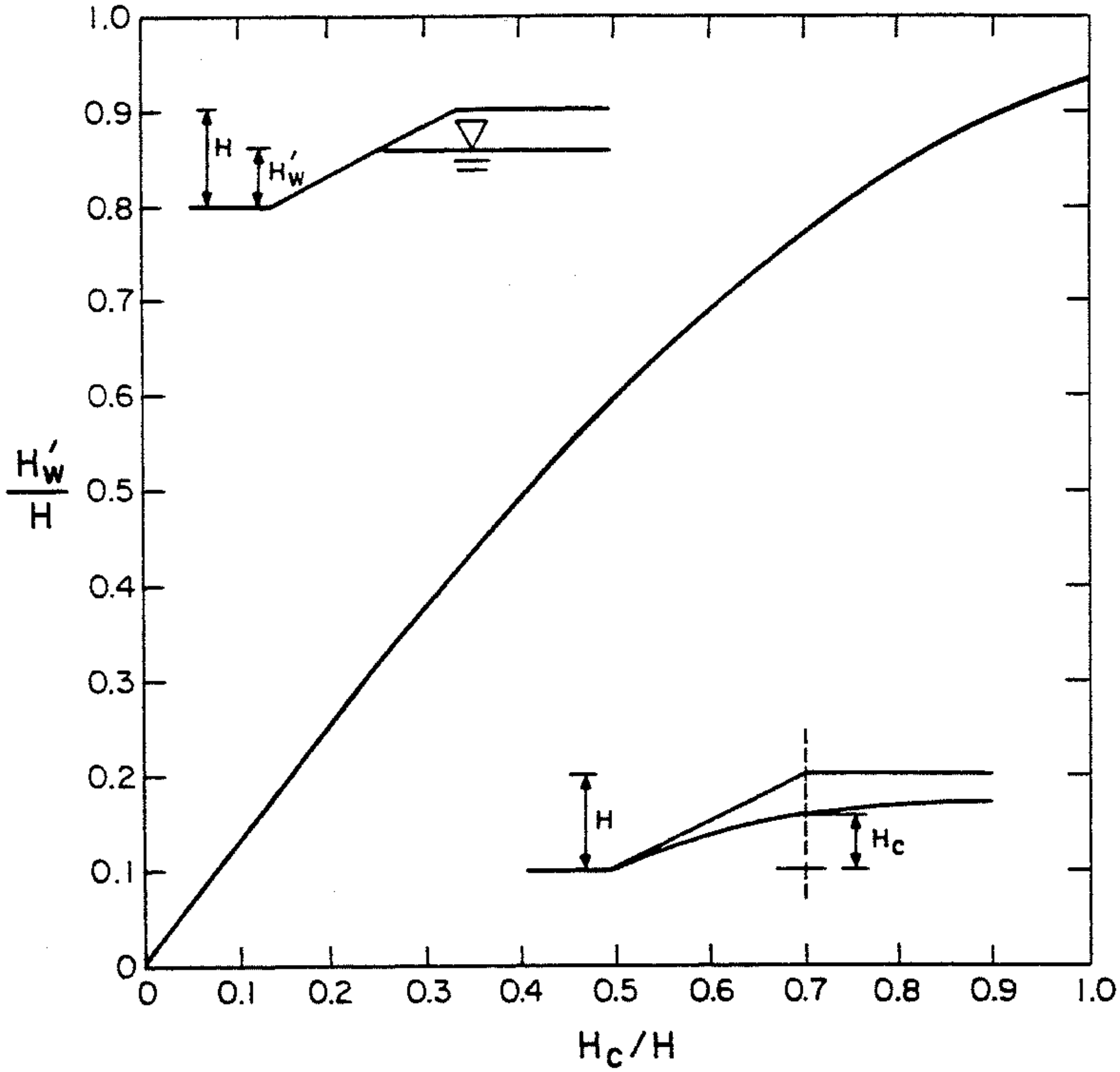
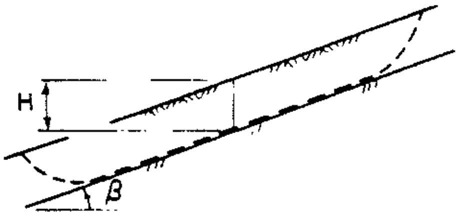
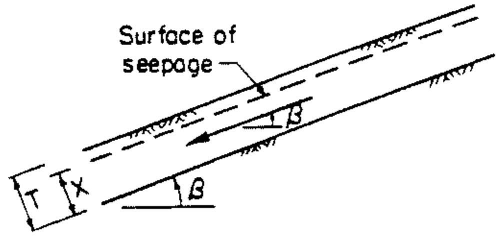


Fig. 9 CHART FOR DETERMINING STEADY SEEPAGE CONDITIONS





γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept } Effective Stress

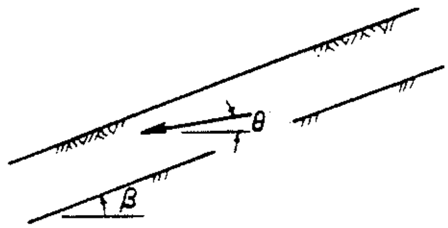
ϕ' = friction angle } Effective Stress

r_u = pore pressure ratio = $\frac{u}{\gamma H}$

u = pore pressure at depth H

Seepage parallel to slope

$$r_u = \frac{X}{T}\,\frac{\gamma_w}{\gamma}\cos^2\beta$$



Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\,\frac{1}{1+\tan\beta\tan\theta}$$

Steps:

① Determine r_u from measured pore pressures or formulas at right

② Determine A and B from charts below

③ Calculate $F = A\,\frac{\tan\phi'}{\tan\beta} + B\,\frac{c'}{\gamma H}$

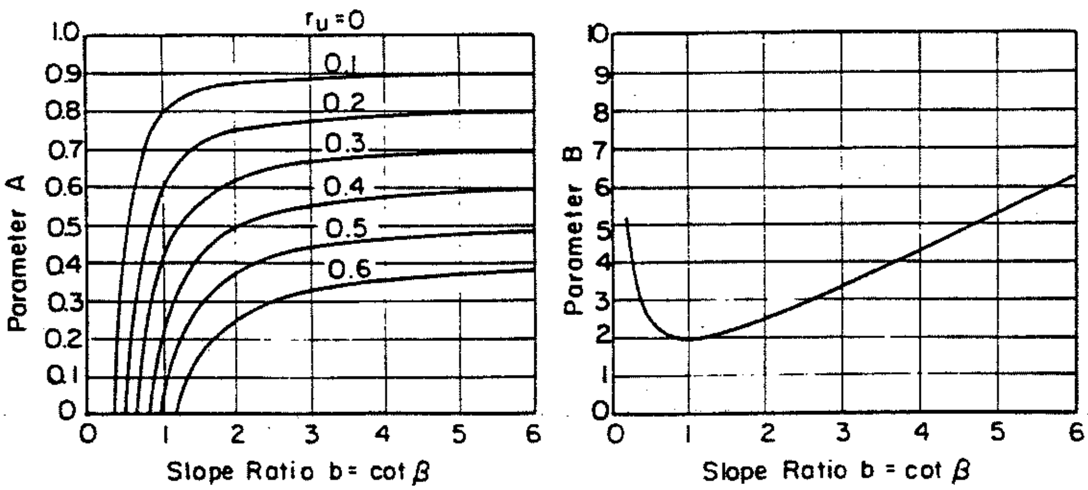
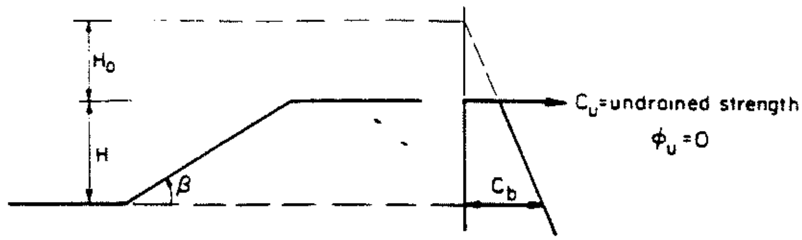


Fig.10 STABILITY CHARTS FOR INFINITE SLOPES.



Steps:

① Extrapolate strength profile upward to determine value of H_o, where strength profile intersects zero

② Calculate $M = H_o/H$

③ Determine stability number N from chart below

④ Determine C_b = strength at bottom of slope

⑤ Calculate $F = N \frac{C_b}{\gamma(H + H_o)}$

Use $\gamma = \gamma_{buoyant}$ for submerged slope

Use $\gamma = \gamma_m$ for no water outside slope

Use average γ for partly submerged slope

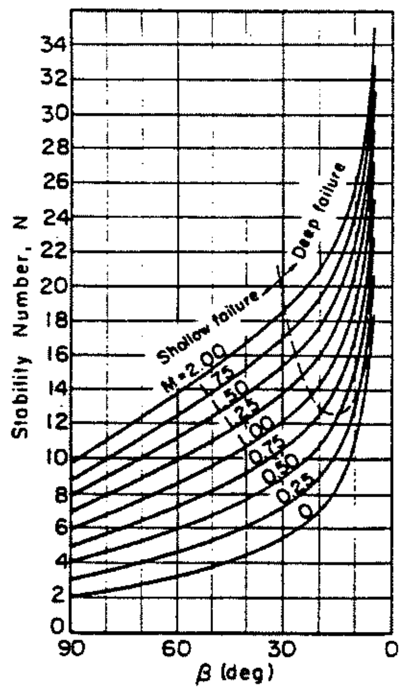


Fig. 11 SLOPE STABILITY CHARTS FOR $\phi = 0$, AND STRENGTH INCREASING WITH DEPTH. (after Hunter and Schuster, 1968)

- If a soil layer is stronger than the one above it, the critical circle may be tangent to the base of either layer, and both possibilities should be examined. This applies to layers above as well as below the toe of the slope.

Once the cases to be investigated have been decided upon, the remaining steps should be performed for each one.

2. This step may be omitted for toe circles. Calculate the depth factor, d, using the formula

$$d = \frac{D}{H}$$

in which

D = depth from the toe of the slope to the lowest point on the slip circle (L; length)
H = slope height above the toe of the slope (L)

If the circle is entirely above the toe, i.e. $D < 0$, its point of interaction with the slope should be taken as an adjusted "toe", and all dimensions like D, H and H_w adjusted accordingly in the calculations.

3. Find the center of the critical circle using the charts in the bottom half of Figure 6 and draw this circle to scale on a cross section of the slope.

4. Determine the average value of the strength, c, for the circle. This is done by calculating the weighted average of the strengths along the failure arc, using the number of degrees intersected by each soil layer as the weighting factor. An example is shown in Figure 12.

5. Calculate P_d using the formula below

$$Pd = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

in which

γ = average unit weight of soil (F/L^3)
H = slope height above toe (L)
q = surcharge (F/L^2
γ_w = unit weight of water (F/L^3)
H_w = height of external water level above toe (L)
μ_q = surcharge correction factor (Figure 7a, top)
μ_w = submergence correction factor (Figure 7a, bottom)
μ_t = tension crack correction factor (Figure 7b, top)

If there is no surcharge, $\mu_q = 1$; if there is no external water above toe, $\mu_w = 1$; if there are no tension cracks, $\mu_t = 1$.

6. Using the chart at the top of Figure 6, determine the value of the stability number, N_o, which depends on the slope angle, β , and the value of d.

7. Calculate the factor of safety, F, using the formula

$$F = \frac{N_o c}{P_d}$$

in which

N_o = stability number
c = average shear strength (F/L^2)

See Fig. 12 for an example. Note that the charts were used to calculate factors of safety for circles tangent to two different depths. For the case shown in Fig. 12, the deeper circle is more critical.

<u>Charts for Slopes in Uniform Soils with $\Phi > 0$</u>. The stability chart for slopes in soils with $\Phi > 0$ is shown in Figure 8. Correction factors for surcharge loading at the top of the slope, submergence, seepage and tension cracks are given in Figures 7a and 7b. The stability chart in Figure 8 can be used for analyses in terms of effective stresses. The chart may also be used for total stress analysis of unsaturated slopes, if $\Phi_u > 0$.

Steps for use of charts:

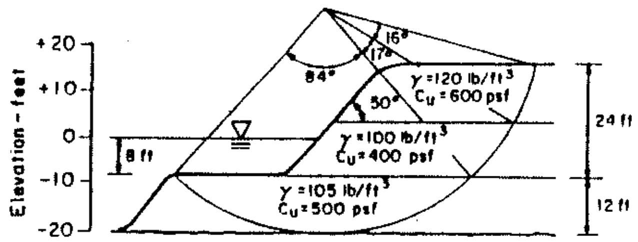
1. Using judgment, estimate the location of the critical circle. For most conditions of simple slopes in uniform soils with $\Phi > 0$, the critical circle passes through the toe of the slope. The stability numbers given in Figure 8 have been developed by analyzing toe circles. In cases where c = 0, the critical mechanism is shallow sliding. Figure 10 can be used in this case. In cases where there is water outside the slope, the critical circle may pass above the water.

   If conditions are not homogeneous, a circle passing above or below the toe may be more critical than the toe circle. The following criteria can be used to determine which possibilities should be examined:

   - If a soil layer is weaker than the layer above it, the critical circle may be tangent to the base of the weaker layer. This applies to layers above as well as below the toe.

   - If a soil layer is stronger than the layer above it, the critical circle may be tangent to the base of either of the two layers, and both possibilities should be examined. This applies to layers above as well as below the toe.

   The charts in Figure 8 may be used for non-uniform conditions provided the values of c and ϕ used in the calculation represent the correct



Investigate a deep circle tangent to elevation - 20

$d = \frac{12}{24} = 0.5 \qquad H_w/H = \frac{8}{24} = 0.33$

From Fig. 6, $x_o = 0.35 \quad y_o = 1.5$ @ $\beta = 50°$

$X_o = (H)(x_o) = (24)(0.35) = 8.4$ ft
$Y_o = (H)(y_o) = (24)(1.5) = 36$ ft

Plot critical circle on slope above

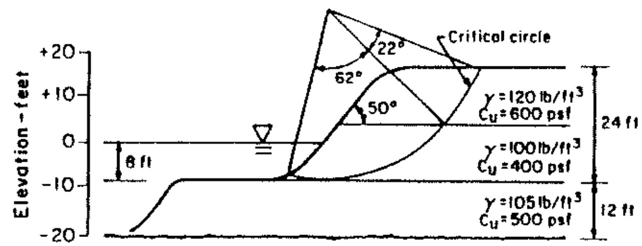
$$C_{ave} = \frac{(16)(600) + (17)(400) + (84)(500)}{16 + 17 + \not{18}\ 84} = 499 \text{ psf}$$

From Fig. 7a., $\mu_w = 0.95$ for $d = 0.5$ and $H_w/H = 0.33$

$$P_d = \frac{(108)(24) - (62.4)(8)}{(1)(0.95)(1)} = 2203$$

From Fig. 6, $N_o = 5.6$ for $d = 0.5$ and $\beta = 50°$

$$F = \frac{(5.6)(499)}{2203} = 1.27$$



Investigate a circle tangent to elevation - 8 ft

$d = 0 \qquad H_w/H = \frac{8}{24} = 0.33$

From Fig. 6, $x_o = 0.35$, $y_o = 1.4$ critical circle intersects near toe of slope

$X_o = (24)(0.35) = 8.4$ ft $\qquad Y_o = (24)(1.4) = 33.6$ ft

Plot circle on slope above

$$C_{ave} = \frac{(22)(600) + (62)(400)}{22 + 62} = 452 \text{ psf}$$

From Fig. 7a., $\mu_w = 0.93$ for $\beta = 50°$ and $H_w/H = 0.33$

$$P_d = \frac{(110)(24) - (62.4)(8)}{(1)(0.93)(1)} = \frac{(2640 - 499}{0.93} = 2302$$

From Fig. 6, $N_o = 5.8$ for $d = 0$ and $\beta = 50°$

$$F = \frac{(5.8)(452)}{2302} = 1.14$$ (more critical than circle tangent to elevation - 20 ft).

Fig. 12 EXAMPLES OF USE OF CHARTS FOR SLOPES IN SOILS WITH $\phi = 0$.

average values for the circle considered.

Once the types of circles to be investigated have been selected, the following steps should be performed, for each case studied.

2. Calculate P_d using the formula

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

in which

γ = average unit weight of soil (F/L^3)
H = slope height above slope (L)
q = surcharge (F/L^2)
γ_w = unit weight of water (F/L^3)
H_w = height of external water above toe (L)
μ_q = surcharge correction factor (Fig. 7a, top)
μ_w = submergence correction factor (Fig. 7a, bottom)
μ_t = tension crack correction factor (Fig. 7b)

If there is no surcharge, $\mu_q = 1$; if there is no submergence, $\mu_w = 1$; and if there are no tension cracks, $\mu_t = 1$.

If the circle being studied passes above the natural toe of the slope, the point where the circle intersects the slope face should be taken as the "toe" of the slope for the calculation of H and H_w.

3. Calculate P_e using the formula

$$P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu'_w}$$

in which

H_w' = height of water within slope above toe (L)

μ_w = seepage correction factor (Fig. 7a, bottom)

and the other factors are as defined previously.

H'_w is the average level of the piezometric surface within the slope. For steady seepage conditions this is related to the position of the phreatic surface beneath the crest of the slope as shown in Fig. 9.

If the circle being studied passes above the natural toe of the slope, H'_w must be measured relative to the adjusted "toe" as defined in step 2.

If there is no seepage, $\mu'_w = 1$; and if there is no surcharge, $\mu_q = 1$.

In an effective stress analysis of a slope in soil of low permeability, if the surcharge is applied so quickly that there is not sufficient time for the soil to consolidate under the surcharge, take $q = 0$ and $\mu_q = 1$ in the formula for P_e.

In a total stress analysis, internal pore water pressure is not considered, so $H'_w = 0$ and $\mu'_w = 1$ in the formula for P_e.

4. Calculate the dimensionless parameter $\lambda_{c\Phi}$ using the formula

$$\lambda_{c\Phi} = \frac{P_e \tan \Phi}{c}$$

in which

$\tan \Phi$ = average value of $\tan \Phi$

c = average value of c (F/L^2)

For $c = 0$, $\lambda_{c\Phi}$ is infinite. In this case the charts for infinite slopes are appropriate.

Steps 4 and 5 are iterative steps. On the first iteration, average values of tan Φ and c have to be estimated by inspection of the layers through which the circle under investigation will pass.

5. Using the chart on the right side of Figure 8, determine the center coordinates of the circle under investigation. The coordinates X_o and Y_o are measured relative to the adjusted "toe" of the slope, if applicable.

   Plot the critical circle on a scaled cross section of the slope and calculate the weighted average values of tan Φ and c along the failure arc, using the number of degrees intersected along the arc by each soil layer as a weighting factor.

   Return to step 4 with these average values of the shear strength parameters and repeat this iterative process until the value of $\lambda_{c\Phi}$ becomes constant. See Fig. 13 for two examples.

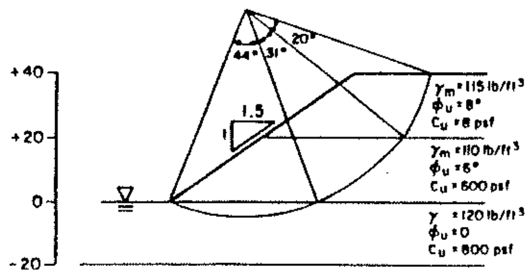
6. Using the chart on the left side of Figure 8, determine the value of the stability number N_{cf}, which depends on the slope angle, β , and the value of $\lambda_{c\Phi}$.

7. Calculate the factor of safety, F, using the formula

$$F = N_{cf} \frac{c}{P_d} \qquad (\text{for } c > 0)$$

Examples of the use of the $\Phi > 0$ charts for both total and effective stress analyses are shown in Fig. 13.

If $c = 0$, the value of $\lambda_{c\Phi}$ is infinite and the factor of safety is



Investigate a toe circle in terms of total stress

$$P_d = \frac{(112.5)(40)}{(1)(1)(1)} = 4500 \text{ psf}$$

$$P_e = \frac{(112.5)(40)}{(1)(1)(1)} = 4500 \text{ psf}$$

Estimate C_{ave} = 700 psf, $\tan\phi$ = 0.122

$$\lambda_{c\phi} = \frac{(4500)(0.122)}{700} = 0.8$$

From Fig 8, x_o = 0.6 and y_o = (1.5)(40) = 60 ft
X_o = (0.6)(40) = 24 ft ; Y_o = (1.5)(40) = 60 ft

$$C_{ave} = \frac{(20)(800) + (31)(600) + (44)(800)}{20 + 31 + 44} = 735 \text{ psf}$$

$$\tan\phi_{ave} = \frac{(20)(\tan 8°) + (31)(\tan 6°) + (44)(\tan 0)}{20 + 31 + 44} = 0.064$$

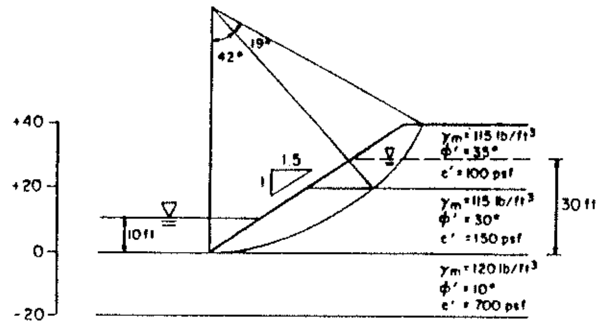
$$\lambda_{c\phi} = \frac{(4500)(0.064)}{735} = 0.4$$

From Fig 8, x_o = 0.65, y_o = 1.45 for b = 1.5 and $\lambda_{c\phi}$ = 0.4
X_o = (0.65)(40) = 26 ft Y_o = (1.45)(40) = 58 ft

This circle is close enough to the previous iteration, so keep $\lambda_{c\phi} \approx 0.4$ and C_{ave} = 735 psf

From Fig 8, N_{cf} = 6 for b = 1.5 and $\lambda_{c\phi}$ = 0.4

$$F = \frac{(6)(735)}{4500} = 1.0$$



Investigate a toe circle in terms of effective stress

$$P_d = \frac{(115)(40) - (62.4)(10)}{(1)(0.96)(1)} = 4141 \text{ psf}$$

$$P_e = \frac{(115)(40) - (62.4)(30)}{(1)(0.95)(1)} = 2870 \text{ psf}$$

Estimate C_{ave} = 120 psf and $\tan\phi_{ave}$ = 0.64

$$\lambda_{c\phi} = \frac{(2870)(0.64)}{120} = 15.3$$

From Fig 8, x_o = 0.0 and y_o = 1.9 for b = 1.5 and $\lambda_{c\phi}$ = 15.3
X_o = 0 ft, Y_o = (1.9)(40) = 76 ft

$$C_{ave} = \frac{(19)(100) + (42)(150)}{19 + 42} = 134 \text{ psf}$$

$$\tan\phi_{ave} = \frac{(19)(\tan 35°) + (42)(\tan 30°)}{19 + 42} = 0.62$$

$$\lambda_{c\phi} = \frac{(2870)(0.62)}{134} = 13.3$$

From Fig 8, x_o = 0.02 and y_o = 1.85 for b = 1.5 and $\lambda_{c\phi}$ = 13.3
X_o = (0.02)(40) = 0.8 ft and Y_o = (1.85)(40) = 74 ft

This circle is close enough to the previous iteration, so keep $\lambda_{c\phi}$ = 13.3 and C_{ave} = 134 psf.

From Fig 9, N_{cf} = 35 for b = 1.5 and $\lambda_{c\phi}$ = 13.3

$$F = \frac{(35)(134)}{4141} = 1.13$$

Fig 13 EXAMPLES OF USE OF CHARTS FOR SOILS WITH $\phi > 0$

calculated using the formula

$$F = \frac{P_e}{P_d} \; b \tan\phi \quad (\text{for } c = 0)$$

in which

b = slope ratio = cot β, and the other factors are as defined previously. Fig. 10 can also be used for analysis of infinite slopes.

<u>Slope Stability Charts for Infinite Slopes</u>. Two types of conditions can be analyzed accurately using the charts shown in Fig. 10, which are based on infinite slope analyses. These conditions are:

1. Slopes in cohesionless materials, where the critical failure mechanism is shallow sliding or surface ravelling.

2. Slopes in residual soils, where a relatively thin layer of soil overlies firmer soil or rock, and the critical failure mechanism is sliding along a plane parallel to the slope, at the top of the firm layer.

Steps for use of the charts for effective stress analyses:

1. Determine the pore pressure ratio, r_u, which is defined by the formula

$$r_u = \frac{u}{\gamma H}$$

in which u = pore pressure (F/L^2)

γ = total unit weight of soil (F/L^3)

H = depth corresponding to pore pressure, u (L)

For an existing slope, the pore pressure can be determined from field measurements, using piezometers installed at the depth of sliding.

For seepage parallel to the slope, which is a condition frequently used for design, the value of r_u can be calculated using the following formula:

$$r_u = \frac{X}{T}\,\frac{\gamma_w}{\gamma}\,\cos^2\beta \qquad \left(\text{for seepage parallel to slope}\right)$$

in which X = distance from the depth of sliding to the surface of seepage, <u>measured normal to the surface of the slope</u> (L)

T = distance from the depth of sliding to the surface of the slope, <u>measured normal to the surface of the slope</u> (L)

γ_w = unit weight of water (F/L^3)

γ = total unit weight of soil (F/L^3)

β = slope angle.

For seepage emerging from the slope, which is more critical than seepage parallel to the slope, the value or r_u can be calculated using the following formula

$$r_u = \frac{\gamma_w}{\gamma}\,\frac{1}{1 + \tan\beta\,\tan\Theta} \qquad \left(\text{for seepage emerging from slope}\right)$$

in which Θ angle of seepage measured from the horizontal direction, and the other factors are as defined previously.

Submerged slopes with no excess pore pressures can be analyzed using $\gamma = \gamma_b$ (buoyant unit weight) and $r_u = 0$.

2. Determine the values of the dimensionless parameters A and B from the charts at the bottom of Fig. 10.

3. Calculate the factor of safety, F, using the formula

$$F = A\,\frac{\tan\Phi'}{\tan\beta} + B\,\frac{c'}{\gamma H}$$

in which Φ' = angle of internal friction in terms of effective stress

c' = cohesion intercept in terms of effective stress (F/L^2)

β = slope angle

H = depth of sliding mass <u>measured vertically</u> (L)

and the other factors are as defined previously.

Steps for use of charts for total stress analyses:

1. Determine the value of B from the chart in the lower right corner of Fig. 10.

2. Calculate the factor of safety, F, using the formula

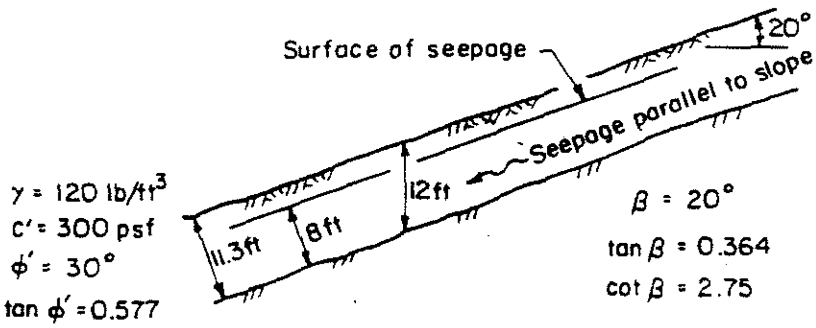
$$F = \frac{\tan\Phi}{\tan\beta} + B\,\frac{c}{\gamma H}$$

in which Φ = angle of internal friction in terms of total stress

c = cohesion intercept in terms of total stress (F/L^2)

and the other factors are as defined previously.

An example of the use of the infinite slope charts is given in Fig. 14.



For seepage parallel to slope, with X = 8 ft., T = 11.3 ft.

$$r_u = \frac{8}{11.3}\,\frac{62.4}{120}\,(0.94)^2 = 0.325$$

} Formula from Fig. 10

From Fig. 10, $A = 0.62$ for $r_u = 0.325$ and $\cot\beta = 2.75$

$B = 3.1$ for $\cot\beta = 2.75$

$$F = 0.62\,\frac{0.577}{0.364} + 3.1\,\frac{300}{(120)(12)} = 0.98 + 0.65 = 1.63$$

For horizontal seepage emerging from slope, $\theta = 0$

$$r_u = \frac{62.4}{120}\,\frac{1}{1 + (0.364)(0)} = 0.52$$

} Formula from Fig. 10

From Fig. 10, $A = 0.41$ for $r_u = 0.52$ and $\cot\beta = 2.75$

$B = 3.1$ for $\cot\beta = 2.75$

$$F = 0.41\,\frac{0.577}{0.364} + 3.1\,\frac{300}{(120)(12)} = 0.65 + 0.65 = 1.30$$

Fig. 14 EXAMPLE OF USE OF INFINITE SLOPE CHARTS.

Charts for Slopes in Soils with Strength Increasing with Depth, and $\phi = 0$. The chart for slopes in soils with strength increasing with depth, and $\phi = 0$, is shown in Fig. 11.

Steps for use of chart:

1. Select the linear variation of strength with depth which best fits the measured strength data. Extrapolate this linear variation upward to determine H_o, the height at which the strength profile intersects zero, as shown in Fig. 11.

2. Calculate $M = H_o/H$, where H = slope height.

3. Determine the dimensionless stability number, N, from the chart in the lower right corner of Fig. 11.

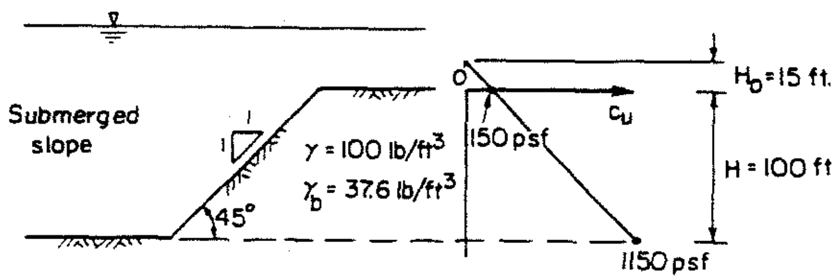
4. Determine the value of strength, c_b , at the elevation of the bottom of the slope.

5. Calculate the factor of safety, F, using the formula

$$F = N \frac{c_b}{\gamma \ (H + H_o)}$$

in which γ = total unit weight of soil for slopes above water,
γ = buoyant unit weight for submerged slopes, and
γ = weighted average unit weight for partly submerged slopes.

An example of the use of this chart is given in Fig. 15.



$$M = \frac{15}{100} = 0.15$$

From Fig. 11, N = 5.1 for M = 0.15 and $\beta = 45°$

C_b = 1150 psf

$$F = (5.1)\frac{1150}{(37.6)(115)} = 1.36$$

Fig. 15 EXAMPLE OF USE OF CHART FOR STRENGTH INCREASING WITH DEPTH, AND $\phi = 0$.

DETAILED ANALYSES OF SLOPE STABILITY

When the site conditions and strength values have been investigated thoroughly and defined accurately, it is appropriate to perform detailed analyses of the stability. Three methods of detailed analysis are described in subsequent sections. These are:

1. The Method of Moments for $\phi = 0$. This is a very simple but theoretically accurate method for analysis of circular slip surfaces in $\phi = 0$ soils.

2. The Ordinary Method of Slices. This is a simple and conservative procedure for analysis of circular slip surfaces in soils with $\phi > 0$. It can also be used for slopes in $\phi = 0$ soils, and it gives accurate results for this case. For flat slopes with high pore pressures, the factors of safety calculated by this method may be much smaller than values of F calculated by more accurate methods, and it should not be used for such problems.

3. The Wedge Method. This is a simple and conservative procedure for analysis of noncircular surfaces in soils with $\phi = 0$ or $\phi > 0$.

Method of Moments for $\phi = 0$ Soils. For short-term stability problems in saturated soils, the undrained soil strengths can be expressed as

$$S_u = \text{constant} \quad (\phi_u = 0)$$

with $\phi_u = 0$, the undrained strength is not dependent on normal stress, and a very simple method of analysis (the $\phi = 0$ method) can be employed to calculate the factor of safety.

The factor of safety with respect to sliding on a particular circular arc is defined by the equation

$$F = \frac{RM}{OM}$$

in which RM resisting moment due to mobilizing the shear strengths of all of the soils through which the arc passes, and OM overturning moment due to the weight of the soil mass bounded by the circular arc.

The factor of safety defined by this equation can be shown to be exactly the same as the ratio between the shear strength of the soil divided by the shear stress required for equilibrium of the slope. Thus the factor of safety calculated by this method may be considered as the factor by which all of the soil strength values would have to be divided to bring the slope into a state of barely stable equilibrium. The factor of safety should always be at least as large as the margin of uncertainty regarding soil strengths.

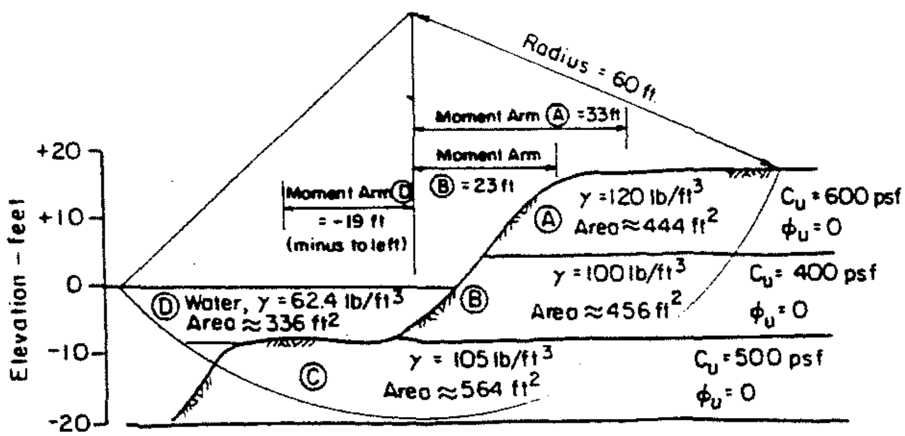
The factor of safety of a slope is calculated using the following procedure:

(1) Select a trial circular slip surface (an example is shown in Fig. 16).

(2) Divide the mass bounded by the circular arc into a number of sections, following soil boundaries. If there is water outside the slope, it should be represented by one or more sections just as if it was a soil with weight but no strength.

(3) For each section calculate the area, the weight, the moment arm, and the moment. The areas may be estimated using a planimeter, or by approximating the sections by rectangles and triangles. The moment arms are measured horizontally from the circle center to the centroids of the areas. Note that for a left-facing slope, as in Fig. 16, moment arms are positive to the right and negative to the left. The algebraic sum of the moments of the sections is the overturning moment (OM) of the mass bounded by the circular arc.

(4) For each segment of arc, determine the arc length, the shear strength, the resisting force (product of arc length multiplied by shear



Section	Area (ft²)	γ (lb/ft³)	Weight (lb/ft)	Moment Arm (ft)	Moment (ft-lb/ft)
Ⓐ	444	120	53,280	+ 33	$+1.76\times10^6$
Ⓑ	456	100	45,600	+ 23	$+1.05\times10^6$
Ⓒ	564	105	59,220	0	0.0
Ⓓ	336	62.4	20,970	− 19	-0.40×10^6
			Total Overturning Moment	=	$+2.41\times10^6$

Section	Ave. Length (ft)	Cu (psf)	Force (lb/ft)	Moment Arm = Radius (ft)	Moment (ft-lb/ft)
Ⓐ	14	600	8,400	60	0.50×10^6
Ⓑ	16.5	400	6,600	60	0.40×10^6
Ⓒ	69	500	34,500	60	2.07×10^6
Ⓓ	18	0	0	60	0.00
			Total Resisting Moment	=	2.97×10^6

$$\text{Factor of Safety, } F = \frac{\text{Resisting Moment}}{\text{Overturning Moment}} \quad \frac{2.97\times10^6}{2.41\times10^6} = 1.23$$

Fig. 16 METHOD OF MOMENTS FOR $\phi = 0$.

strength) and the moment (product of resisting force multiplied by circle radius). The sum of the individual moments is the resisting moment (RM) of the soil through which the arc passes.

(5) Calculate the factor of safety for the selected circle, F = RM/OM.

(6) Repeat steps (1) through (5) for a number of circles tangent to the same elevation as the first, until the most critical circle (the one with the lowest value of F) tangent to this elevation has been located.

(7) Repeat for other tangent elevations until the overall critical circle has been located.

<u>Ordinary Method of Slices or Fellinius Method for Soils with $\Phi = 0$ or $\Phi > 0$</u>. The Ordinary Method of Slices can be used to calculate the factor of safety for a circular slip surface in soils whose strengths are governed by any of the following equations:

$$s = c \quad (\Phi = 0)$$

$$s = \sigma \tan\Phi$$

$$\text{or} \quad s = c + \sigma \tan\Phi$$

in which s = shear strength, σ = normal stress on the failure plane, c = cohesion intercept, and Φ = friction angle. To be able to determine the strengths of soils with $\Phi > 0$, the normal stress on the failure plane must be known. Therefore, to analyze the stability of slopes in such soils, it is necessary to determine the normal stress on the shear surface analyzed.

For analysis by the Ordinary Method of Slices, the mass above a trial circular slip surface is divided into a number of vertical slices as shown in Fig. 17. The basic assumption in the method is that the resultant of the side forces on any slice acts parallel to the base of the slice and therefore does not influence the normal stress on the base of the slice. This assumption is conservative, and the method results in factors of safety which are lower than values calculated by more accurate methods. For most cases the error due to this assumption is no more than 10%. For very high pore pressures and flat slopes, however, the error may be 50% or even more. For high pore pressures and flat slopes, a more accurate method such as Bishop's Modified Method (Bishop, 1955) should be used.

The factor of safety by the Ordinary Method of Slices may be expressed as

$$F = \frac{\Sigma(W\cos\alpha - u\ell)\tan\Phi + \Sigma c\ell}{\Sigma W\sin\alpha}$$

in which F = factor of safety, c = cohesion, Φ = friction angle, W = slice weight, α = inclination of base of slice, u = pore pressure on base of slice, and ℓ = length of base of slice.

The factor of safety defined by this equation can be shown to be exactly the same as the ratio between the shear strength of the soil and the shear stress required for equilibrium of the slope. Thus the factor of safety calculated by this method may be considered as the factor by which all of the values of c and tan Φ would have to be divided to bring the slope into a state of barely stable equilibrium. The factor of safety should always be at least as large as the margin of uncertainty regarding soil strengths.

The factor of safety of a slope is calculated using the following procedure:

(1) Select a trial slip surface (an example is shown in Fig. 17).

(2) Divide the mass bounded by the circular arc into a number of vertical slices. The slices should be chosen so that the base of any slice lies wholly within a single soil layer. For hand calculations, 8 to 12 slices are sufficient; for computer analysis 30 or more slices are used. If there is water outside the slope, it should be represented by one or more slices, just as if it was a soil with weight but not strength.

Layer	γ (lb/ft^3)	c (lb/ft^2)	ϕ (degrees)
(A)	110	60	35
(B)	105	100	30
(C)	110	750	5

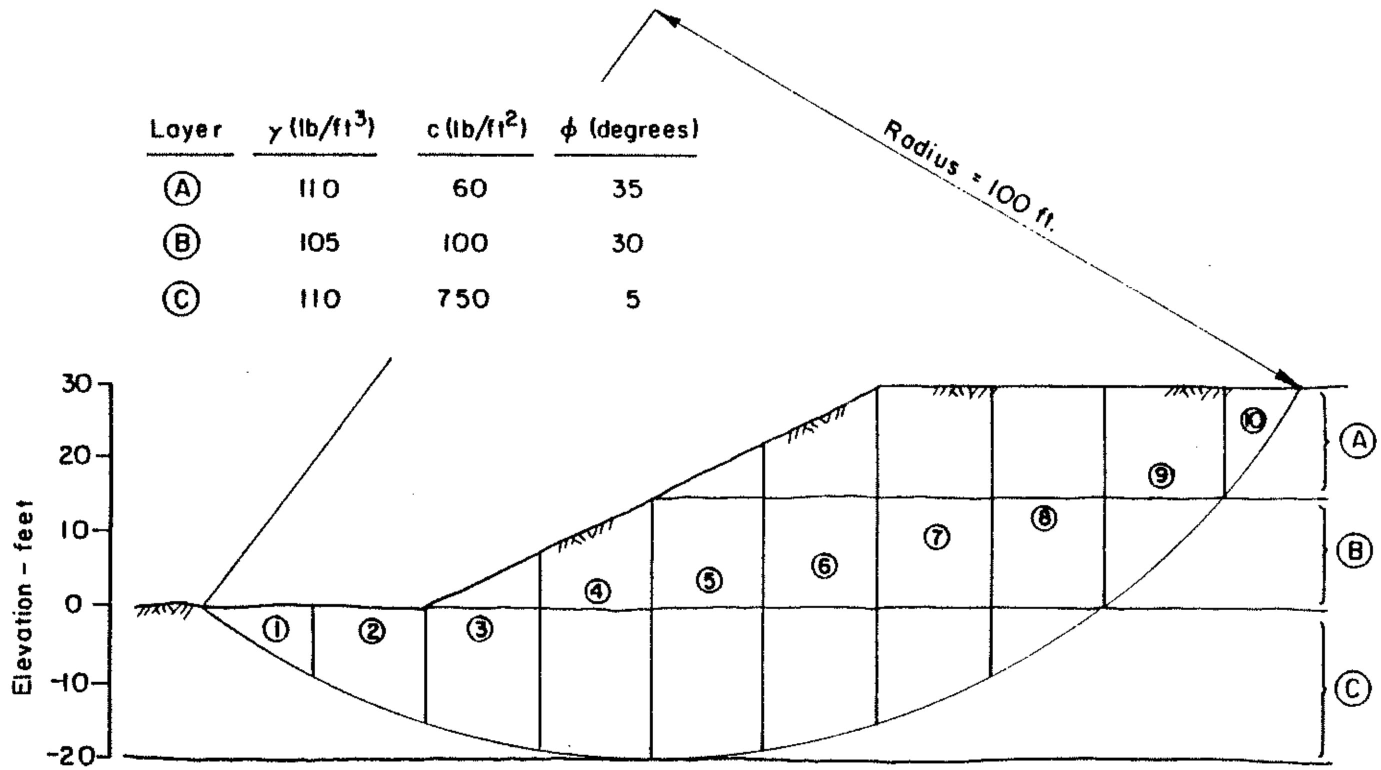


Fig. 17 EXAMPLE PROBLEM FOR ORDINARY METHOD OF SLICES.

(3) Calculate the weight of each vertical slice. When a slice crosses more than one layer having different unit weights, the weights within each layer are summed to determine the total weight of the slice. This may be done conveniently using the tabular computation form in Fig. 18. An example is shown in Fig. 19.

(4) For each slice, determine the length of the base (ℓ), the angle of inclination of the base (α), the cohesion of the soil at the base (c), the friction angle of the soil at the base (ϕ), and the pore pressure at the base (u). (If the analysis is being done with total stresses, use u = 0). Enter these values, along with the weight of each slice, in the tabular computation form shown in Fig. 20.

(5) Calculate the factor of safety following the procedure indicated on the computation form. An example is shown in Fig. 21.

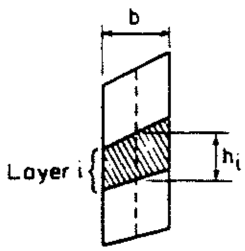
(6) Repeat steps (1) through (5) for a number of circles tangent to the same elevation as the first, until the most critical circle (the one with the lowest value of F) tangent to this elevation has been located.

(7) Repeat for other tangent elevations until the overall critical circle has been located.

<u>Wedge Method for Soils with $\phi = 0$ or $\phi > 0$</u>. The Wedge Method can be used to calculate the factor of safety for a noncircular slip surface in soils whose strengths are governed by any of the following equations:

$$s = c \quad (\phi = 0)$$
$$s = \sigma \tan\phi$$
$$s = c + \sigma \tan\phi$$

in which s = shear strength, σ = normal stress on the failure plane, c = cohesion intercept, and ϕ = friction angle. To be able to determine the strengths of soils with $\phi > 0$, the normal stress on the failure plane must



γ_i = unit weight of layer i

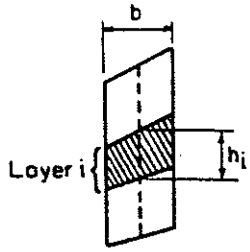
h_i = height of layer at center of slice

W_i = partial weight = $b h_i \gamma_i$

ΣW_i = total weight of slice

Slice No.	b	h_i	γ_i	W_i	ΣW_i

Fig. 18 TABULAR FORM FOR COMPUTING WEIGHTS OF SLICES.



γ_i = unit weight of layer i

h_i = height of layer at center of slice

W_i = partial weight = $b h_i \gamma_i$

ΣW_i = total weight of slice

Slice No.	ft b	ft h_i	lb/ft³ γ_i	lb/ft W_i	lb/ft ΣW_i
1	15	5	110	8200	8,200
2	15	13	110	21400	21,400
3	15	4	105	6300	
		17.5	110	28700	35,200
4	15	11.5	105	18100	
		19.5	110	32200	50,300
5	15	4	110	6600	
		15	105	23600	
		19.5	110	32200	62,400
6	15	11.5	110	19000	
		15	105	23600	
		17.5	110	28900	71,500
7	15	15	110	24800	
		15	105	23600	
		15	110	21400	69,800
8	15	15	110	24800	
		15	105	23600	
		5	110	8200	56,600
9	16	15	110	26400	
		7.5	105	12600	39,000
10	11	7.5	110	9100	9,100

Fig. 19 EXAMPLE OF USE OF TABULAR FORM FOR COMPUTING WEIGHTS OF SLICES.

Slice No.	W	ℓ	α	c	ϕ	u	$u\ell$	$W\cos\alpha$	$W\cos\alpha - u\ell$	$(W\cos\alpha - u\ell)\tan\phi$	$c\ell$	$W\sin\alpha$
									Σ			

c = cohesion intercept
ϕ = friction angle
u = pore pressure

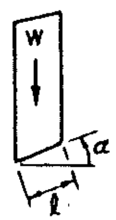
} at base of slice

$$F = \frac{\Sigma (W\cos\alpha - u\ell)\tan\phi + \Sigma c\ell}{\Sigma W\sin\alpha}$$

F = ―――――――― =

Fig. 20 TABULAR FORM FOR CALCULATING FACTOR OF SAFETY BY ORDINARY METHOD OF SLICES.

Slice No.	W kip/ft	ℓ ft	α degrees	c kip/ft²	ϕ degrees	u zero for total stress analysis	$u\ell$	$W\cos\alpha$ kip/ft	$W\cos\alpha - u\ell$	$(W\cos\alpha - u\ell)\tan\phi$	$c\ell$	$W\sin\alpha$
1	8.2	17.5	-32	0.75	5			6.9		0.61	13.12	-4.35
2	21.4	16.2	-22	0.75	5			19.8		1.73	12.15	-8.02
3	35.2	15.5	-13	0.75	5			34.3		3.00	11.62	-7.92
4	50.3	15.1	-4	0.75	5			50.2		4.39	11.32	-3.51
5	61.4	15.1	4	0.75	5			61.2		5.44	11.32	4.35
6	71.5	15.5	13	0.75	5			69.7		6.10	11.62	16.08
7	69.8	16.2	22	0.75	5			64.7		5.66	12.15	26.15
8	56.6	17.5	32	0.75	5			48.0		4.20	13.12	30.00
9	39.0	22.0	43	0.10	30			28.5		16.45	2.20	26.60
10	9.1	18.5	55	0.06	35			5.2		3.64	1.11	7.45
									Σ	51.2	99.7	86.8



c = cohesion intercept, ϕ = friction angle, u = pore pressure — at base of slice

$$F = \frac{\Sigma (W\cos\alpha - u\ell)\tan\phi + \Sigma c\ell}{\Sigma W\sin\alpha}$$

$$F = \frac{150.1}{86.8} = 1.74$$

Fig. 21 EXAMPLE OF USE OF TABULAR FORM FOR CALCULATING FACTOR OF SAFETY BY ORDINARY METHOD OF SLICES.

be known. Therefore, to analyze the stability of slopes in such soils, it is necessary to determine the normal stress on the shear surface analyzed.

For analysis by the Wedge Method the mass above the trial slip surface is divided by vertical lines into a number of wedges or slices as shown in Fig. 22. This method satisfies both horizontal and vertical force equilibrium. The basic assumption in the Wedge Method as described in this manual is that the side forces between slices are horizontal. This assumption is conservative, and the method gives factors of safety which are lower than the values calculated by more accurate methods. For most cases the error due to this assumption is no more than 15%. Greater accuracy can be achieved using methods which satisfy all conditions of equilibrium, such as Janbu's Generalized Procedure of Slices (Janbu, 1973), Spencer's Method (Wright, 1969) or Morgenstern and Price's Method (Morgerstern and Price, 1965).

The Wedge Method is most appropriate for conditions where the failure surface is not likely to be circular. For example, the embankment shown in Fig. 22 rests on a thin layer of weak clay, and it is likely that a considerable portion of the critical failure surface will lie within this layer. For this type of problem the wedge mechanism may be more critical than a circular surface.

The factor of safety calculated by the Wedge Method is defined as the ratio between the shear strength and the shear stress required for equilibrium. The factor of safety is the factor by which the strength parameters (c and tan ϕ) for each soil would have to be divided to bring the slope into a state of barely stable equilibrium. The factor of safety should always be at least as large as the margin of uncertainty regarding soil strengths.

The Wedge Method factor of safety is calculated by trial and error. A value for F is assumed, and then checked to determine if the assumed value satisfies equilibrium. The analysis can be performed either graphically or numerically. The first three steps are the same whether the graphical or the numerical method is used.

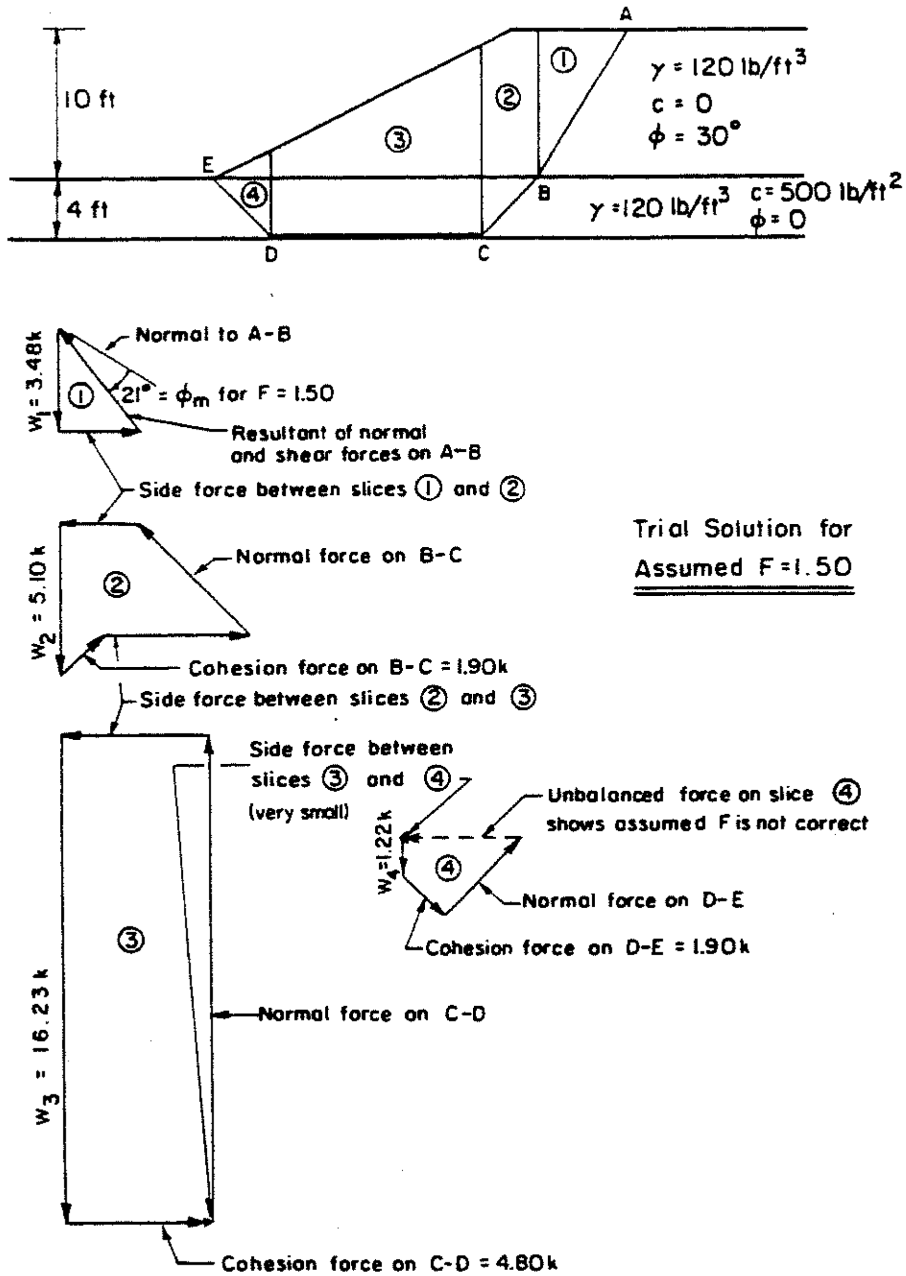


Fig. 22 EXAMPLE OF GRAPHICAL PROCEDURE FOR WEDGE METHOD

(1) Select a trial slip surface (an example is shown in Fig. 22).

(2) Divide the mass above the slip surface into wedges (or slices). The wedges should be chosen so that the base of any wedge lies wholly within a single soil layer. Three to five wedges are usually sufficient. If there is water outside the slope, it should be represented by a wedge, just as if it was soil with weight but no strength.

(3) Calculate the weight of each wedge. If the top as well as the bottom of each wedge is a straight line, the weights can be calculated using the tabular computation form described previously for the Ordinary Method of Slices. If the top boundary of a wedge is a broken line, as for wedge 2 in Fig. 22, the weight of the wedge can be calculated by dividing it into two parts, as shown in Fig. 23.

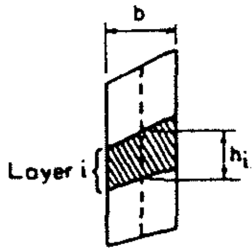
To solve for the factor of safety graphically, follow steps (4) through (9) below.

(4) Assume a value for the factor of safety, and calculate trial values of mobilized cohesion and mobilized friction angles for each soil using the following formulas:

$$c_m = \frac{c}{F} \tag{1}$$

and $$\tan\phi_m = \frac{\tan\phi}{F} \tag{2}$$

in which F = assumed value for the factor of safety, c = cohesion, c_m = mobilized cohesion, ϕ = friction angle, and ϕ_m = mobilized friction angle.



γ_i = unit weight of layer i

h_i = height of layer at center of slice

W_i = partial weight = $b h_i \gamma_i$

ΣW_i = total weight of slice

Slice No.	ft b	ft h_i	kip/ft³ γ_i	kip/ft W_i	kip/ft ΣW_i
1	5.8	5.0	0.120	3.48	3.48
2 (right)	1.9	10.0	0.120	2.28	
		1.0	0.120	0.23	
2 (left)	1.8	9.0	0.120	1.94	
		3.0	0.120	0.65	5.10
3	14.4	5.4	0.120	7.33	
		4.0	0.120	6.90	16.23
4	3.7	0.9	0.120	0.40	
		1.85	0.120	0.82	1.22

Fig. 23 EXAMPLE OF USE OF TABULAR FORM FOR COMPUTING WEIGHTS OF WEDGES.

(5) Construct the force polygon for wedge 1. An example is shown in Fig. 22. First draw the weight vector vertically, to scale. Next, draw the mobilized cohesion vector, which is equal to the mobilized cohesion multiplied by the length of the base of the slice, and acts parallel to the base of the slice. The tail of this vector connects to the head of the weight vector. (In the example the cohesion is zero on the first slice.) Then, if the analysis is done in terms of effective stress, draw the pore pressure vector, which is equal to the pore pressure on the base of the slice multiplied by the length of the base, and acts perpendicular to the base. The tail of this vector connects to the head of the cohesion vector. If the analysis is done in terms of total stresses, as the example in Fig. 22, the pore pressure is taken as zero, and there is no pore pressure force in any of the force polygons. Next, lay off the direction of the resultant of the normal and frictional forces on the base of the slice. This resultant acts at an angle of Φ_m from the normal direction, and the head of this vector connects to the tail of the weight vector. The remaining force, which closes the polygon, is the side force exerted on wedge 1 by wedge 2. This vector is assumed to act horizontally, as indicated previously. The position of the intersection of the resultant of the normal and frictional forces with the side force determines the lengths of these two vectors, which are unknown until the intersection point is determined.

(6) Construct the force polygon for wedge 2. First draw the weight vector vertically, to scale. Then draw the side force exerted on wedge 2 by wedge 1. Note that this is equal but opposite to the force exerted on wedge 1 by wedge 2, and that the head of this vector connects to the tail of the weight vector. Next, draw the mobilized cohesion vector, which is equal to the mobilized cohesion multiplied by the length of the base of the slice, and acts parallel to the base of the slice, with its tail connected to the head of the weight vector. Then, if the analysis is done in terms of effective stresses, lay off the pore pressure force, from the head of the cohesion force, acting perpendicular to the base of the slice. Next,

lay off the direction of the resultant of the normal and frictional forces on the base of the slice. This resultant acts at an angle of Φ_m from the normal direction, and the head of this vector connects to the tail of the vector, which represents the side force exerted on wedge 2 by wedge 1. (In the example, $\Phi = 0$ for the second slice, and there is therefore no frictional force. In this case the vector consists of only the normal force and acts normal to the base of the slice.) The remaining force, which closes the polygon, is the side force exerted on wedge 2 by wedge 3. This vector is assumed to act horizontally. The position of the intersection of the resultant of the normal and frictional forces with the side force determines the lengths of these two vectors, which are unknown until the intersection point is determined.

(7) Construct the force polygons for the remaining wedges in sequence, using the same procedures as for wedges 1 and 2. If the assumed factor of safety is correct, the force polygon for the last wedge will close, with no unbalanced force. However, if the assumed factor of safety is not correct, an additional force will be required to close the polygon. If the force required to close the polygon would have to act in the direction which would make the slope more stable, the assumed factor of safety is too high. If the required force would have to act in the direction which would make the slope less stable, the assumed factor of safety is too low. This is true for the trial solution with F = 1.50 in Fig. 22.

(8) Assume a new factor of safety and repeat steps (4) through (7). This has been done in Fig. 24. Try additional factors of safety until the unbalanced force on the last slice is negligibly small compared to the magnitudes of the other forces. Then the assumed value of F is the correct one for the assumed failure mechanism. Usually no more than two trials are needed to determine F. By plotting the assumed factor of safety against the magnitude of the unbalanced force for the first two trials, a third trial value of F can usually be estimated which will be very close to the correct value, as shown in Fig. 25. If the value of F determined by this procedure differs

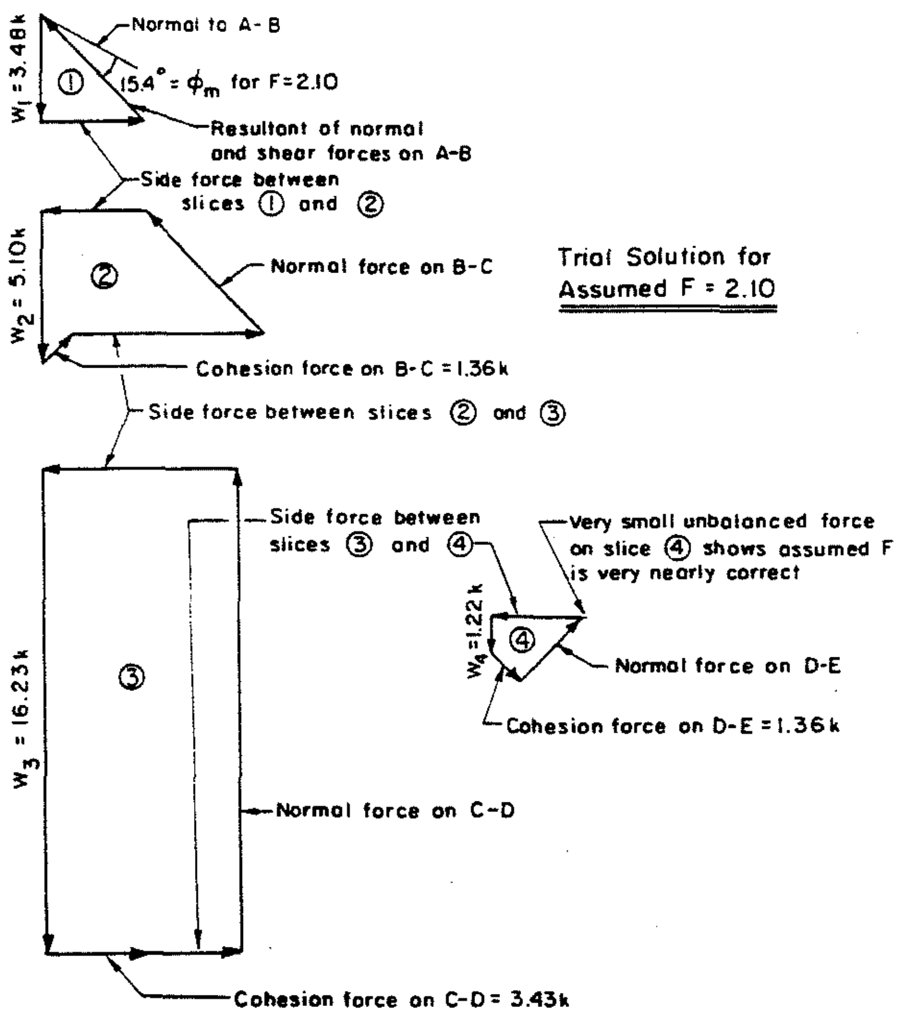


Fig. 24 EXAMPLE OF GRAPHICAL PROCEDURE FOR WEDGE METHOD.
(continued from Fig. 22)

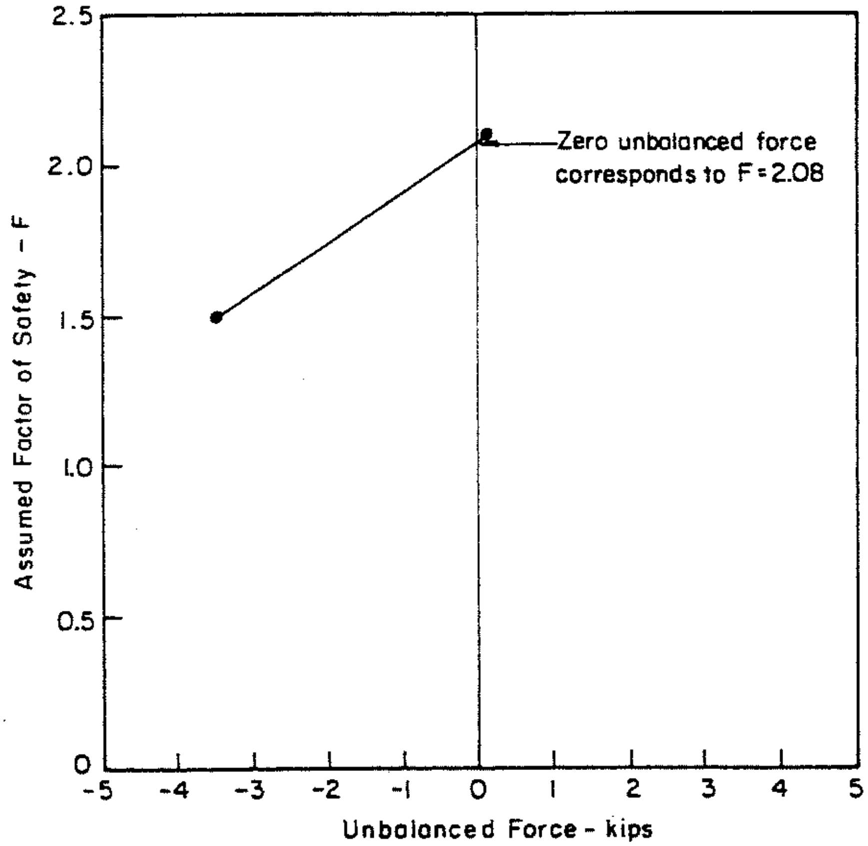


Fig. 25 DETERMINING FACTOR OF SAFETY BY WEDGE METHOD.

greatly from both of the first two trial values, a third trial may be necessary.

(9) Select a new failure mechanism and repeat steps (1) through (8). Try several different failure mechanisms in order to find the one with the lowest factor of safety.

To solve for the Wedge Method factor of safety numerically, use the tabular computation form shown in Fig. 26. Steps (1) through (3), as described previously, are the same for the numerical analysis as for the graphical analysis. Steps (4) through (9) proceed as described below. An example is shown in Fig. 27.

(4) For each wedge, determine the inclination of the base (α), the length of the base (ℓ), the cohesion of the soil at the base (c), the friction angle of the soil at the base (Φ), and the pore pressure at the base (u). (If the analysis is being done with total stresses, use u = 0.) Enter these values, along with the weight of each slide, in the tabular computation form shown in Fig. 26.

(5) Calculate the quantities $c\ell/\cos\alpha$, $W\tan\Phi$, and $u\ell\tan\Phi/\cos\alpha$ for each wedge, and enter these in the table.

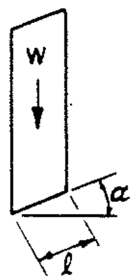
(6) Assume a trial value for the factor of safety, and calculate the value of ΔE for each wedge as indicated in the table. ΔE is the difference between the side forces on the left and right sides of each slice, and is given by the equation:

$$\Delta E = \frac{FW\tan\alpha - \dfrac{c\ell}{\cos\alpha} - W\tan\Phi + \dfrac{u\ell\tan\Phi}{\cos\alpha}}{F + \tan\Phi\,\tan\alpha}$$

$$\Delta E = \frac{N_4 - N_1 - N_2 + N_3}{N_5}$$

Slice No.	W	α	ℓ	C	ϕ	u	N_1: $\frac{C\ell}{\cos\alpha}$	N_2: $W \tan\phi$	N_3: $\frac{u\ell \tan\phi}{\cos\alpha}$

Trial	F	Slice No.	N_4: $FW\tan\alpha$	N_5: $F + \tan\phi \tan\alpha$	ΔE	$\Sigma\Delta E$



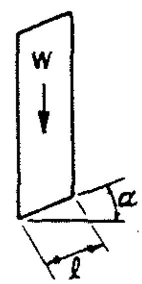
C = cohesion intercept
ϕ = friction angle
u = pore pressure at base of slice

Fig. 26 TABULAR FORM FOR CALCULATING FACTOR OF SAFETY BY WEDGE METHOD.

$$\Delta E = \frac{N_4 - N_1 - N_2 + N_3}{N_5}$$

Slice No.	W (kips)	α	ℓ (ft)	c	ϕ	u	N_1: $\frac{c\ell}{\cos\alpha}$	N_2: $W \tan\phi$	N_3: $\frac{u\ell \tan\phi}{\cos\alpha}$
1	3.48	60°	11.4	0	30°	0	0	2.01	0
2	5.10	45°	5.7	0.50	0	0	4.03	0	0
3	16.20	0	14.4	0.50	0	0	7.20	0	0
4	1.20	-45°	5.7	0.50	0	0	4.03	0	0

Trial	F	Slice No.	N_4: $FW\tan\alpha$	N_5: $F + \tan\phi \tan\alpha$	ΔE	$\Sigma\Delta E$
1	1.50	1	9.05	2.50	2.81	
		2	7.65	1.50	2.41	
		3	0.00	1.50	-4.80	
		4	-1.80	1.50	-3.89	-3.46
2	2.10	1	12.66	3.10	3.44	
		2	10.71	2.10	3.18	
		3	0.00	2.10	-3.43	
		4	-2.52	2.10	-3.12	+0.07
3	2.08	1	12.54	3.08	3.42	
		2	10.61	2.08	3.16	
		3	0.00	2.08	-3.46	
		4	-2.50	2.08	-3.14	-0.02



C = cohesion intercept
ϕ = friction angle
u = pore pressure at base of slice

Fig. 27 EXAMPLE OF USE OF TABULAR FORM FOR CALCULATING FACTOR OF SAFETY BY WEDGE METHOD.

(7) Calculate the sum of the terms ΔE for all slices. If the assumed factor of safety is correct, this sum will be zero. If its value is less than zero, the assumed value of F is too low. If it is greater than zero the assumed value of F is too high.

(8) Assume a new value of F and repeat steps (6) and (7). Try additional values of F until the sum of the ΔE's is negligibly small. Then the assumed value of F is the correct one for the assumed failure mechanism. Usually no more than two trials are needed to determine F. By plotting the assumed factor of safety against the value of $\Sigma\ \Delta E$ for the first two trials, a value of F can usually be estimated which will be very close to the correct value, as shown in Fig. 25. If the value of F determined by this procedure differs greatly from both of the first two values, a third trial may be necessary.

(9) Select a new failure mechanism and repeat steps (1) through (8). Try several failure mechanisms in order to find the one with the lowest factor of safety.

MINIMUM FACTOR OF SAFETY

Locating the Critical Circle. When detailed analyses of slope stability are performed using the Method of Moments for $\Phi = 0$, or using the Ordinary Method of Slices, a number of circles must be examined to locate the most critical circle, with the lowest factor of safety. This can be done conveniently using the following procedure:

(1) Calculate the factors of safety for a number of circles having some common feature. For example:

   (a) all circles tangent to the same elevation, or
   (b) all circles pass through the toe of the slope.

(2) Plot the factors of safety at the locations of the circle centers, and draw contours of F. An example is shown in Fig. 28. If the contours enclose the minimum value of F, the critical circle with the selected common feature can be located readily. If the contours do not enclose the minimum, more circles should be analyzed.

(3) Calculate the factors of safety for additional circles having a second common feature, such as a different tangent elevation, and draw contours of F for these circles. An example is shown in Fig. 29.

(4) Repeat this process until the overall critical circle has been located. A good procedure for many problems is to locate the critical circle passing through the toe of the slope first, and then to examine higher and lower tangent elevations to see if they are more critical.

For complex slopes there may be more than one minimum enclosed by the contours for circles tangent to the same elevation or passing through the toe of the slope. An example is shown in Fig. 30. For these conditions

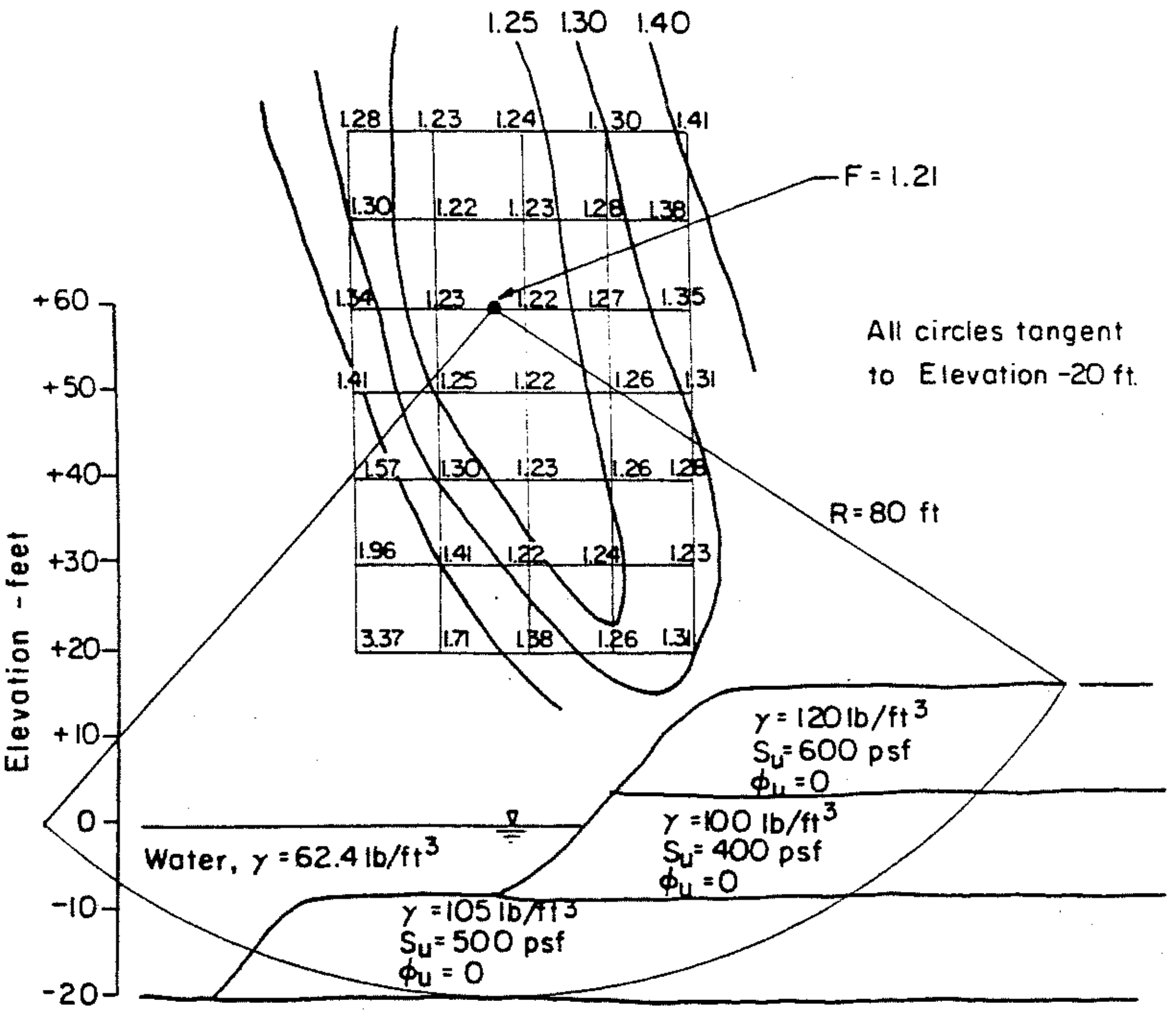


Fig. 28 CONTOURS OF F FOR CIRCLES TANGENT TO ELEVATION -20 FT.

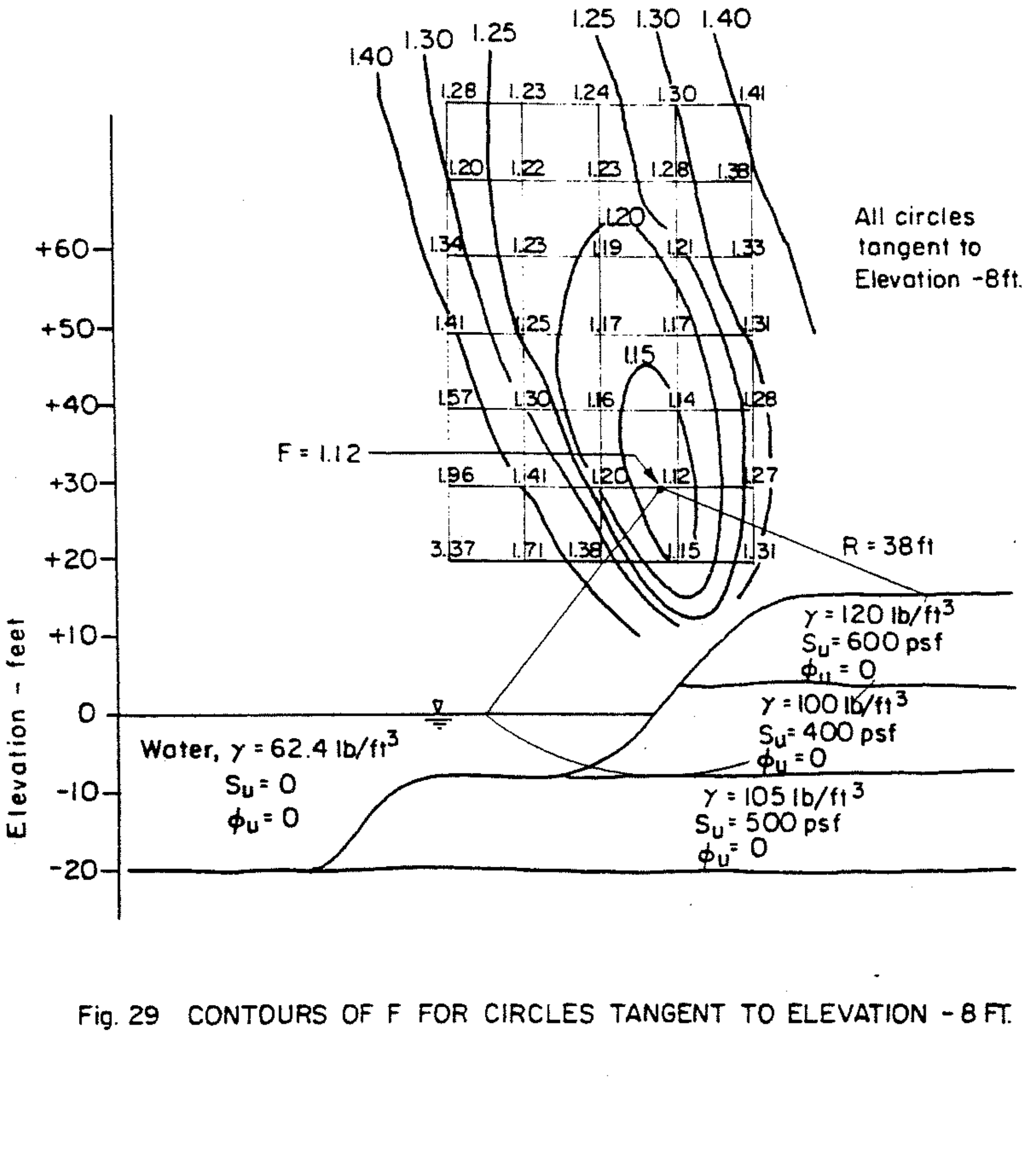


Fig. 29 CONTOURS OF F FOR CIRCLES TANGENT TO ELEVATION - 8 FT.

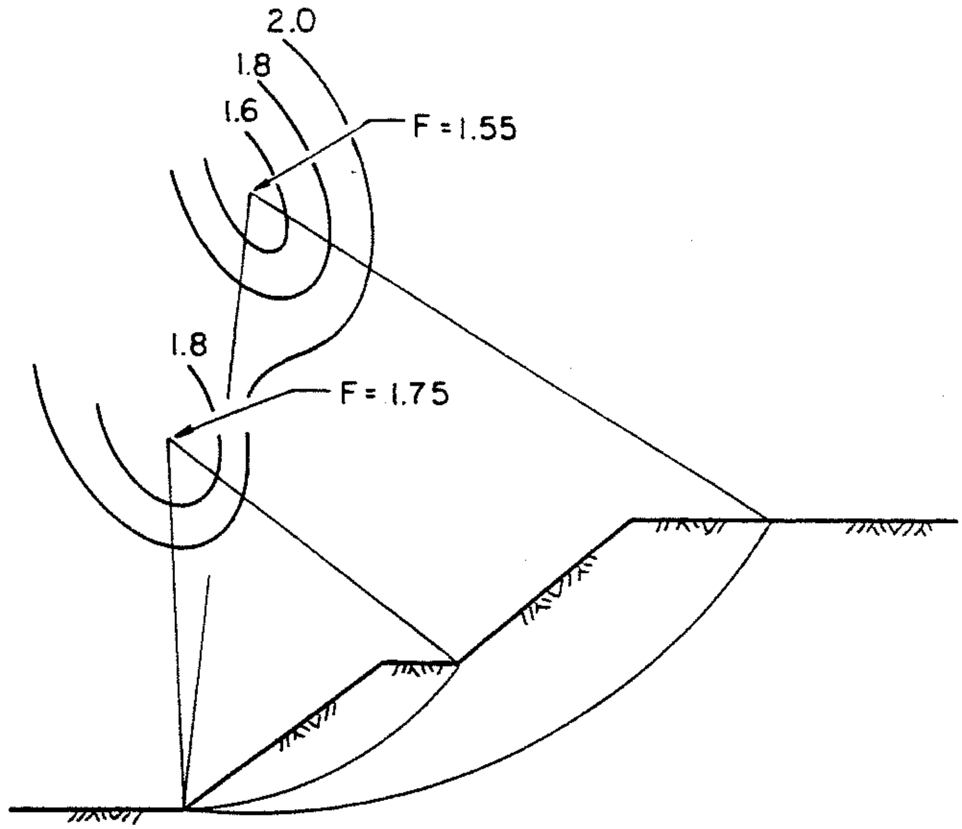


Fig. 30 SLOPE WITH COMPLEX FACTOR OF SAFETY CONTOURS.

widely spaced circle centers should be studied to begin, to be sure that the lowest minimum is located.

Locating the Critical Wedge Mechanism. When detailed analyses are performed using the Wedge Method, enough trial surfaces must be analyzed to determine the minimum factor of safety. The first wedge can be selected as shown in Fig. 31. Then the positions and shapes of the wedges can be varied to find the most critical wedge mechanism.

Sources of Inaccuracy in Calculated Factors of Safety. Under most conditions, the uncertainties due to approximations and assumptions in the method of analysis are smaller than the uncertainties due to inaccuracies in measuring the shear strength. Approximations in the analysis usually amount to 15% or less, but the margin for error in evaluating shear strength may be considerably greater.

Minimum Recommended Values of Safety Factor. The minimum allowable value of F for a slope depends on:

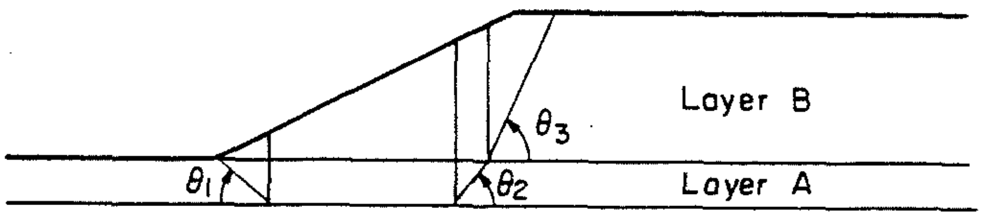
(1) The degree of uncertainty in the shear strength measurements, slope geometry, and other conditions.

(2) The costs of flattening or lowering the slope to make it more stable

(3) The costs and consequences of a slope failure.

(4) Whether the slope is temporary or permanent.

The values of factor of safety in Table 3 are given for guidance.



$$\left.\begin{aligned} \theta_1 &\approx 45 - \frac{\phi_{mA}}{2} \\ \theta_2 &\approx 45 + \frac{\phi_{mA}}{2} \\ \theta_3 &\approx 45 + \frac{\phi_{mB}}{2} \end{aligned}\right\}$$

ϕ_{mA} = mobilized friction angle in layer A

ϕ_{mB} = mobilized friction angle in layer B

Fig. 31 TRIAL WEDGE MECHANISM OF FAILURE.

TABLE 3 .-- RECOMMENDED MINIMUM VALUES OF STATIC FACTOR OF SAFETY

Costs and Consequences of Slope Failure	Uncertainty of Strength Measurements	
	Small [1]	Large [2]
Cost of repair comparable to cost of construction. No danger to human life or other property if slope fails.	1.25	1.5
Cost of repair much greater than cost of construction, or danger to human life or other valuable property if slope fails.	1.5	2.0 or greater

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete and logical picture of the strength characteristics.

[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when the available strength data do not provide a consistent, complete, or logical picture of the strength characteristics.

STABILIZATION OF SLOPES AND LANDSLIDES

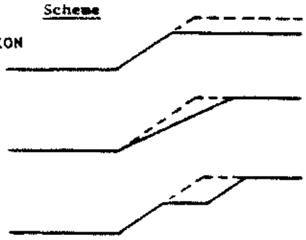
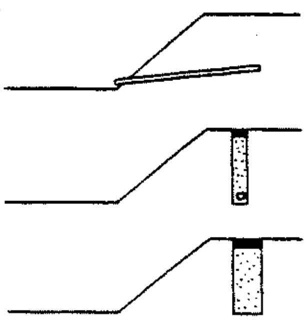
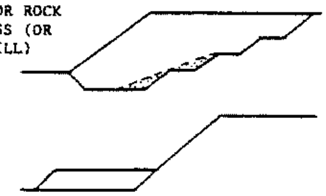
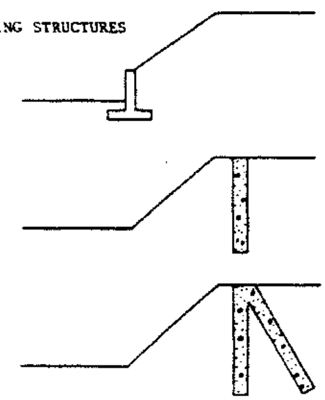
For purposes of planning effective procedures for stabilizing landslides, it is important to understand the cause or causes of the instability. The most common causes are: oversteepening of the slope by cutting or filling, excess pore water pressure caused by high ground-water levels, strong seepage, or blockage of drainage paths, undercutting due to erosion from surface water, and loss of strength with time due to creep and weathering.

A thorough geological study and a detailed subsurface exploration are the first steps to determine the cause and to plan corrective action for a slope failure. The location of the shear zone within a landslide can sometimes be determined by drilling test borings or by monitoring slope movements using slope indicators which extend beneath the failure zone.

If a slide is being stabilized by flattening the slope, or by use of a buttress or retaining structure, the shear strength of the soil at the time of failure should be determined by calculating what value of strength would correspond to a factor of safety equal to one at failure. This value of strength can be used to evaluate the factor of safety of the slope after stabilization, or to estimate the design loads for retaining structures. Values of shear strength back-calculated from a failure may not agree with values of strength measured in laboratory tests because of difficulties in obtaining representative samples and difficulties in duplicating field conditions in laboratory tests. Usually, the shear strength back-calculated from the field condition is a more reliable value for use in designing remedial measures.

A number of methods of stabilizing slopes and landslides are summarized in Table 4. Often one or more of these schemes may be used together. Schemes 1 through V are listed approximately in order of increasing cost. Excavation (Scheme I) and drainage (Scheme II) are often the most effective schemes for small to medium-sized landslides Buttresses or berm fills (Scheme III), can be used for fairly large slides provided an area is available for equipment access and for temporary stockpiling of excavated

TABLE 2.-- METHODS OF STABILIZING SLOPES AND LANDSLIDES (after Turnbull and Hvorslev, 1967)

Scheme	Applicable Methods	Comments
I. EXCAVATION 	1. Reduce slope height by excavation at top of slope. 2. Flatten the slope angle. 3. Excavate a bench in upper part of slope. 4. Excavate the entire slide mass.	Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated in this method.
II. DRAINAGE 	1. Small diameter, horizontal drains (hydraugers). 2. Continuous deep subdrain trench. Generally 5 to 15 feet deep. 3. Drilled vertical wells--generally 18- to 36-inch diameter. 4. Improve surface drainage along top of slope with open ditch or paved gutter. Install deep-rooted, erosion-resistant plants on slope face.	1. Most effective if can tap natural aquifer. Drains are usually free-flowing. 2. Trench bottom should be sloped to drain and be tapped with an outlet pipe. Perforated pipe should be placed on trench bottom. Top of trench should be capped with impervious material. 3. Can be pumped or tapped with a gravity outlet. Several wells in a row, joined at bottom can form a drainage gallery. Top of each well should be capped with impervious material. 4. Good practice for most slopes. Direct the discharge away from slide mass.
III. EARTH OR ROCK BUTTRESS (OR BERM FILL) 	1. Excavate slide mass and replace with compacted earth or rock buttress fill. Toe of buttress must be keyed into firm soil or rock below slide plane. Drain blanket with gravity flow outlet is provided in back slope of buttress fill. 2. Compacted earth or rock berm placed at and beyond the toe. Drainage may be provided behind berm.	1. Access for construction equipment and temporary stockpile area required. Excavated soil can usually be used in fill. Underpinning of existing structures may be required. Might have to be done in short sections if stability during construction is critical. 2. Sufficient width and thickness of berm required so failure will not occur below or through berm.
IV. RETAINING STRUCTURES 	1. Retaining wall - crib or cantilever type. 2. Drilled, cast-in-place vertical piles, bottomed well below bottom of slide plane. Generally 18 to 36 inches in diameter and 4- to 8-foot spacing. 3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles bottomed well below slide plane. Generally 12 to 30 inches in diameter and at 4- to 8-foot spacing. 4. Earth anchors and rock bolts.	1. Usually expensive. Cantilever walls might have to be tied back. 2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter (6 feet ±) piles have been used for deep slides. 3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam. 4. Can be used for high slopes, and in very limited areas. Conservative design should be used, especially for permanent support.
V. SPECIAL TECHNIQUES	1. Grouting 2. Chemical injection 3. Electroosmosis (in fine-grained soils). 4. Freezing 5. Heating	1. and 2. Used successfully in a number of cases. Used at other times with little success. At the present, theory is not completely understood. 3. Generally expensive. 4. and 5. Special methods which must be specifically evaluated at each site. Can be expensive. All of these techniques should be carefully evaluated in advance to determine the probable cost and effectiveness.

soils. Retaining structures (Scheme IV) are generally not used for large landslides because of high cost. The methods shown in Scheme V, special techniques, are generally used under unusual conditions which make them more effective or economical than the other procedures.

REFERENCES

Bishop, A.W. (1955) "The Use of the Slip Circle in the Stability Analysis of Slopes," Geotechnique, Vol. V, No. 1, pp. 7-17.

Bishop, A. W. and Bjerrum, L. (1969) "The Relevance of the Triaxial Test to the Solution of Stability Problems," Proceedings of the ASCE Research Conference on the Shear Strength of Cohesive Soils, Boulder.

Bjerrum, L. (1973) "Problems of Soil Mechanics and Construction on Soft Clays," General Report, 8th Int. Conf. on Soil Mech. and Found. Eng., Moscow, Proc. Vol. 3, pp. 11-159 (also Norwegian Geotech. Inst. Pub. 100).

Chirapuntu, S. and Duncan, J. M. (1975) "The Role of Fill Strength in the Stability of Embankments on Soft Clay Foundations," Geotechnical Engineering Research Report, Department of Civil Engineering, University of California, Berkeley.

Hunter, J. H. and Schuster, R. L. (1968) "Stability of Simple Cuttings in Normally Consolidated Clays," Geotechnique, Vol. 13, No. 3, September, 1968, pp. 372-378.

Janbu, N. (1968) "Slope Stability Computations," Soil Mechanics and Foundation Engineering Report, The Technical University of Norway, Trondheim.

Janbu, N. (1973) "Slope Stability Computations," published in Embankment-Dam Engineering, the Casagrande Volume, John Wiley and Sons, pp. 47-86.

Kenney, T.C. (1963) "Stability of Cuts in Soft Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM5, pp. 17-37.

Ladd, C.C. and Foote, R. (1974) "A New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, pp. 763-786.

Lowe, J. and Karafiath, L. (1960) "Stability of Earth Dams Upon Drawdown," Proceedings of the First Pan American Conference on Soil Mechanics and Foundation Engineering, Mexico City, Vol. 2, pp. 537-552.

Morgenstern, N.R. and Price, V.E. (1965) "The Analysis of the Stability of General Slip Surfaces," Geotechnique, Vol. 15, No. 1, pp. 79-93.

Turnbull, W.J. and Hvorslev, M.J. (1967) "Special Problems in Slope Stability," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM4, July, 1967, pp. 499-528.

Wright, S.G. (1969) "A Study of Slope Stability and the Undrained Shear Strength of Clay Shales," Ph.D. Thesis, Department of Civil Engineering, University of California, Berkeley.