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"STABILITY OF EARTH SLOPES FOR ROAD IN EMBANKMENTS AND CUTTINGS"

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C O N T E N T S.

SYNOPSIS

This dissertation deals with the stability of earth slopes, as applicable to highway embankments and cuttings. Different types of stability problems are met with, some of which are complex. There are two distinctly different types of approach for the problem of stability analysis- one, by assuming a definite failure surface and analysing the forces that act, and the other by finding the stresses from point to point within the soil mass and then analysing the stability of the soil slope. A review of various methods of stability analysis is made and the assumptions and limitations of each are discussed. There are various definitions of the factor of safety, and that with respect to strength of the soil seems to be most logical. Investigations of the stability problem have also been carried out by model study. In the case of a slide, the factors causing the failure of slope should be investigated and suitable remedial measures adopted.

List of Figures.

General Notations and Abbreviations.

B-	Resultant Actuating Force.
b-	Width of Element.
C-	Resultant Cohesion.
c-	Cohesion per Unit Area .
c_d-	Cohesion Developed per Unit Area.
c_e-	Effective Cohesion per Unit Area.
C.G.	Centre of Gravity.
d-	Distance of the Centre of the circle to the lime of action of C.G. of the Sliding Mass.
F_c-	Factor of Safety with respect to Cohesion.
$F\phi$	Factor of Safety with respect to Friction.
Fs,F	Factor of Safety with respect to Strength.
F.S.	Factor of Safety.
H.	Height of the Slope.
Hc	Critical Height of the Slope.
i	Angle of the Slope with the Horizontal.
La	Length of the Failure Arc.
Lc	Length of Chord of Failure Arc.
N	Normal component.
O.	Centre of the Failure Arc.
O_1	Centre of Trial Arc.
R.	Radius of the Circular Arc.
T	Tangential Component.
U	Pore Water Pressure.
W	Weight of the Sliding Mass.
w	Unit Weight of Soil.

W_t- Unit Weight of Soil Water System.

W_w- Unit weight of water.

w.r.t- With respect to.

ϕ- Angle of Internal Friction.

ϕd- Developed Angle of Internal Friction.

ϕ e- Effective Angle of Internal Friction.

T- Shearing Stress.

Note- Other notations are explained under each Art. where ever they occur first.

1. INTRODUCTION.

Engineers have to deal with many practical problems in which earth slopes are involved. Common examples include the slopes in embankment and cuttings along highways and rail roads, banks of canals and slopes of earth dams. Stability of earth slopes concerns chiefly to Highway and Railway Engineers who are confronted with two problems- (i) Study of mechanism of large scale natural slips in hill faces (ii) the design and construction of smaller artificial slopes in cutting and embankments, for safety and economy. Similar problems are to be faced with the Irrigation Engineer in the design and construction of earth dams and canal slopes.

Highway Engineers have to face acute problems when they have to deal with land slides and subsidences which are of common occurence in the construction of hill roads. In engineering practice, stability computations serve as a basis either for the redesign of slopes after failure, or for the initial design with specified safety requirements in advance of construction. Slopes that are at the verge of failure may be made more stable by adopting suitable corrective measures.

A large amount of money is spent every year to eliminate and control land slides, but usually anticipated results are not achieved to commensurate with the expenditure incurred. In a country where mountenous regions are vast, rainfall intensity and concentration are high, seismological disturbances are frequent, geological structure is young and unstable, the problem of land slides and subsidence are very complex and as such, thorough assessment are necessary to decide upon any preventive or control measures that have to be adopted.

In Assam, land slides occur frequently and the problems are very common in the hilly districts of Khasi and Jaintia Hills, Mizo Hills, Garo Hills, North Cachar and Mikir Hills and severe in the case of Naga Hills (83). This may be due to more frequent earthquakes in Assam; but the trouble is much less in other parts of India.

Preventive measures of land slides may be more economical than corrective measures (42). The greater the construction cost on a new work, the more justified are additional expenditure for prevention of slides. If embankments are properly designed and constructed slope failures should be infrequent; but foundation failures are separate problems.

Development of Stability Analysis- (94,75,12).

The most exhaustive analysis of land slide phenomenon was carried out by the Swedish Geotechnic commission whose investigations were prompted by the failure of a quay wall in Geoteberg in 1916. Subsequently the commission made a complete study of slides in road and railway cuts

Note: Figures within bracket refer to bibliography number.

and found that the shape of sliding surface showed a decided curvature that could be approximated by an arc of a circle. The circular are analysis was first introduced to Swedish Engineers through two papers by K.E.Petterson and S.Hultin in 1916. The method was further developed by W.Fellenius, H.Krey and others. Later on modifications were introduced by several authors who tried either to simplify the time consuming trial and error procedure or to increase the accuracy in calculations. The failure surface is approximated to the arc of a logarithmic spiral also and both graphical and analytical solutions have been suggested. A few simple methods were also put forward assuming plane failure.

A different procedure of analysing stability of slopes is by finding the stresses at various points within the soil mass by the use of theories of elasticity and plasticity, and the shearing strength at the corresponding points. Photo-elastic methods are also being used to find the stresses within the slope. Electronic computors have been invented by use of which lot of time could be saved in analysing the stability by the circular arc analysis.

2. DIFFERENT TYPES OF FAILURES.

Type I- Falls. (42,43)

There are two types of falls-rock falls and soil falls, in which the moving mass travels mostly through the air by free fall, leaping, bounding, or rolling with little or no interaction between one moving unit and another. Movements are very rapid and may or may not be preceded by minor movements.

Type II Slides- (42,43,46,83).

In true slides, the movement results from shear failure along one or several surfaces, which are either visible or reasonably be inferred. There are two sub-groups -A)- those in ~~xxx in~~ which the moving mass is not greatly deformed and B) those which are greatly deformed or consists of many small units.

A-Group-

(i) Slumps- Commonest type of this group. The movement in slumps takes place only along internal slip surfaces. Usually rotational shear failure takes place resembling a cylindrical surface. Rotational shear failure may be either toe failure, base failure or slope failure.

(ii) Block glides- The mass progresses out or down, as a unit along more or less planar surface, without the rotary movement and backward tilling characteristic of a slump.

B- Group.

i) Rock slides and Debris slides- Loose rock slides are common variety of group B slide consisting of many units. Various kinds of slides involving natural soil, unconsolidated sedimentary material, and rock detritus are included as debris slides. These slides are often limited by the contact between loose material underlying firm bed rock. With increase in water content or with increasing velocity the flowing movement of debris avalanches.

ii) Failures by lateral spreading- In most places the failures take place along zones of high porewater pressure in homogeneous clay or along partings of sand, or silt in clay. The movement in these types of slides is usually complex, involving translation, breaking up of material, some slumping, and some liquefaction and flow.

If the slide grows in the direction of its own motion it is termed progressive; one that grows in the opposite direction (backward) is called retrogressive.

Type III- Flows- (42,68).

In flows, the movement within the displaced mass is such that the form taken by the moving material resembles those of viscous fluids. Slip surfaces within the moving mass are usually not visible or are short-lived, and the boundary between moving and stationary material may be sharp or it may be a zone of plastic flow.

Dry flows follow the fore-going characteristics, but are nearly or quite dry.

Wet flows require water in various proportions. Debris slide and debris flow have different water contents. Debris flow denotes material that contain relatively high percentage of coarse fragments whereas the term mud flow is reserved for material with at least 50% sand, silt and clay-size particles. An earth flow is a flow of slow to very rapid velocity involving mostly plastic or fine grained non-plastic material. Liquid sand or silt flows occur mostly along banks of non-cohesive clean sand or silt.

2.4- Type IV- Complex Land Slides. (42,95).

Often on land slide shows several types of movement within its various parts or at different times in its development, thus being complex.

3. STABILITY ANALYSIS.

In engineering practice stability computations serve as a basis either for the redesign of slopes after failure, or else for choosing slope angles in accordance with specified requirements in advance of construction. Slopes can be analysed for stability and reasonable results can be obtained if the geological cross-section is known and if the shearing strength can be determined with satisfactory accuracy. Many factors introduce complications with stability analyses. Most embankments contain hetrogeneous soils, often of several types. In many slopes there are a number of questions that can not be answered regarding the variables that affect the shearing strengths of soils. The boundary conditions which define the flow net and thus determine the neutral pressures are some times known only roughly. These complifcations usually necessitates to adopt simplified, average soil characteristics which are as good as a representation of the actual variable characteristics as can be obtained; also a simlified cross section may have to be used in some cases. Generally Coulomb's Law is adopted to determine the shear strength characteristics. Also it is usually assumed that the ground water conditions and pressures as represented by a given flow net, are known.

Infinite Slopes of Dry Cohesionless Sand.(95).

The term infinite slope is used to designate a constant slope of unlimited extent which has constant conditions and constant soil properties at any given distance below the surface of slope. If the slope consists of numerous strata, then the boundaries should be parallel to the surface of the slope.

In the case of dry cohesionless sand of angle of internal friction ϕ, the slope will be stable for an infinite height, if the angle of the slope i is less than ϕ. Factor of safety = $\frac{\tan \phi}{\tan i}$.

When seepage is present throughout in an infinite slope of cohesionless sand, the steepest stable slope angle is given by $i = \tan^{-1} \frac{W_t - W_w \tan\phi}{W_t}$ (95). Where Wt and Ww are the unit weights of soil water system and water respectively.

Stability of Cohesive slopes. (95).

When the angle of slope i is less than ϕ, it will be stable; where $i > \phi$ the slope is stable only upto a particular height which has to be determined. When there is no seepage, Taylor's stability number $N = \frac{cd}{wH} = \cos^2 i \times (\tan i - \tan \phi_d)$ and when there is seepage parallel to the ground surface throughout, $N = \cos^2 i \left[\tan i - \frac{w_t - w_w}{w_t} \tan\phi_d\right]$ where ϕ_d and Cd are the developed friction angle and unit cohesion.

Generally finite slopes are met with, the soil having both cohesion and friction. In order to analyse stability of a slope the soil properties have to be determined by laboratory and field investigations, including the ground water conditions.

4. VARIOUS METHODS OF STABILITY ANALYSIS.

The existing methods of stability analysis of earth slopes may be mainly sub-divided into two classes.(27).

I- Methods in which failure is assumed to take place along a more or less regular surface and the shearing stress along the surface and the strength available on the same surface are computed. The process is repeated to find the most dangerous sliding surface. The sliding surface is either assumed to be along a straight line, arc of a circle or along an arc of a logarithmic spiral, thereby introducing an approximation. Stresses at individual points are not to be checked. The approach is deficient mostly because of the unwarranted assumptions that must be made as to the distribution of shearing stress and shearing resistance along the assumed failure arc.

II- Methods by which stability is checked by computing stresses at individual points of the earth mass (62). These methods take into account the physical properties of the materials, but involve intricate mathematical computations and are therefore abstruse and difficult of application. The theories of elasticity and plasticity have been used for the computation of stresses.

In most cases of analysis, a number of simplifying assumptions are to be made. In some cases the use of a simplified cross section that is nearly representative of the actual cross section, is necessary to be made. It is often necessary to adopt simplified average soil characteristics, and also make use of Coulomb's Empirical Law to find the shearing strength. In all cases, the mass that is analysed is of unit dimension in the direction normal to the section.

4.00. Methods which Assume Failure Surface.

Now the various methods coming under the head'I', which assume that failure of slope occures along a definite regular sliding surface shall be discussed one by one. Under this, there are methods which assume that rupture will occur on a plane, along an arc of a circle (i.e. cylindrical in shape), and along an arc of a logarithmic spiral.

4.01. Methods Assuming Plane Failure Surface.

4.02. CULMANN'S METHOD- (94,95).

This method developed by K.Culmann is based on the assumption that failure occurs on a plane, through the toe of the slope. Figure 1 represents the type of section to which this analysis applies.

First any trial line AD (Ref.Fig.1) making an angle θ with the horizontal is taken and the unit cohesion required for equilibrium is calculated. Then the critical angle θc of the plane of failure is found out corresponding to the maximum value of cohesion developed. The critical height of the slope is given by $H_c = \dfrac{4c \sin i . \cos \phi}{w\left[1-\cos(i-\phi)\right]}$

and the stability number $N = \dfrac{c}{F_c w H} = \dfrac{1-\cos(i-\phi)}{4 \sin i . \cos\phi}$

where c, and ø are the actual values of unit cohesion and friction for the soil and w the unit weight.

Discussions.

According to this method, the stability is not affected by the value of 'δ' the angle of surcharge. However, this is an incorrect concept, which is inherent in the questionable assumption of plane failure (95,27). Since the assumption of plane failure represents a limited choice of failure surfaces, the results must be on the unsafe side. From Table No.1, Art:4.5, page it is clear that the values obtained by this method do not agree with others.

This method is mainly of interest because it serves as a test of the validity of the assumption of plane failure which is used in some methods of stability analysis. The plane assumption leads to generally acceptable accuracies if the slope is vertical or nearly so, especially when the angle of internal friction is great. But it is far from correct in flat slopes.

4.03. WEDGE METHOD (19).

In this method the soil mass under analysis is divided into an active wedge, a passive block and a passive wedge. The most critical combination of active wedges, passive blocks, and passive wedges is found by trial, and the safety factor is expressed as the ratio of available shear strength to that required for stability.

For any position of the vertical face of an active or passive wedge, there is one most critical position of the slide plane which will give the greatest thrust or least passive resistance. (Ref. Fig.2). The critical wedges are found by trial and examples of such trials are the active wedges GNF, GNE, and GND, and passive wedges HMJ, JMK and JML. The forces considered to act on the active wedges are- (i) the total Weight W of the soil and wats in the wedge (ii) the hydro-static forces U_{HL}, U_{HR} and U_V which are, the horizontal component or uplift on the inclined face, respectively. These three forces are computed from the pressure diagrams, which are plotted for the pressures given by the hydrostatic equipotential lines (ii) the cohesion C acting along the slide plane equal to the effective length of the plane, L times the unit cohesion C for the material (iv) the effective force, $\bar{F}$ acting at an angle ϕ_d with the normal to the slide plane where ϕ_d is the developed friction (v) The direction of the active wedge driving force PA is assumed horizontal and the magnitude is determined by a graphical solution. This is accomplished by first assuming a trial safety factor and determining Cd and the direction of $\bar{F}$ for a given wedge. A force polygon is constructed using the weight of wedge W, the hydrostatic forces U, the developed cohesion cd and the direction of the force $\bar{F}$. The inter section of the force $\bar{F}$ with the horizontal line PA gives the active pressure for that wedge. Since the maximum active pressure is desired it is necessary to analyse several wedges. By superimposing the graphical solutions for the various trial wedges drawing a curve tangent to the $\bar{F}$ vectors, the maximum value of PA is found.

The resisting force of the passive wedge Ppw is found in a similar manner. The sliding resistance of the passive block P_{PB} is the sum of frictional resistance $\bar{F}_{HB}$, developed cohesion Cd along the base and the difference between the hydrostatic forces U_{HR} and U_{HL} at the ends of the block. The frictional resistance F_{HB} is the product of the effective weight of the blocks $(W - U_v)$ and the developed tan ϕ d of the material at base.

The resistance of the passive wedge P_{pw} plus the resisting force of the block P_{PB} comprises the total passive resistance P_P. If the active driving force P_A equals the total resistance P_P the trial safety factor selected is correct. But if P_A and P_P are not equal, then another complete trial must be made and the process repeated until balance is obtained.

Discussions.

The complete analysis by this method will require the investigation of both active and passive wedges with vertical forces at several positions along the embankment slope. The wedge method is particularly adaptable to the determination of the stability of embankment on foundations containing weak strata of considerable extent or zones of low shear strength is widely used in analysing the slopes of earth dams with puddle core.

4.04. METHOD OF SUPERIMPOSED LOAD. (2)

In this method the hypothetical strength of the slope line itself is analysed by superimposing a load on the slope and allowing the load to slide down the slope.

An earth fill ABE_1E_2 of height H is considered, with unit cohesion C, and angle of internal friction ϕ, supported by a retaining wall with a smooth surface BA. (Ref. Fig.3-a). If the support BA is suddenly rotated in an anticlockwise about A, wedges of earth ABC_2, ABC, ... will disintegrate from the mass of the fill so long as the shear resistance along AC_2, AC_1 ..... is less than the disintegrating force along the corresponding line, and till a line AC is met where the shear resistance along AC just balances the shear force caused by the wedge ABC. Let this failure line AC, approximated to a straight line instead of a curve, make an angle i with the horizontal. Then for equilibrium, factor of safety = $1 = \frac{CL + W \cos i . \tan\phi}{W \sin i}$

where, L is the length of failure line AC and W the weight of the wedge ABC.

The factor of safety is given by, (2) $F = \frac{2C}{wH \tan i} + \frac{\tan\phi}{\tan i}$. This is the mathematical formula recommended for design of earth slopes by B. Behra, who has made the following recommendations.

For saturated condition it is necessary to take the particular value of C or 50% the value of C at optimum moisture current. The unit weight w is taken as saturated weight and the value of ϕ obtained from slow shear test for saturated sample at equivalent density is to be taken.

For sudden draw down conditions, cohesional fraction as for saturated condition and frictional fraction as for saturated condition and frictional fraction as 2/3 that of slow shear value of ϕ

For non-homogeneous sections or partly saturated or moist conditions, average value of cohesion obtained from proportional base lengths should be taken and average value of ϕ obtained from proportional vertical weights above base lengths should be taken.

Discussions.

The method is subject to all the criticisms and drawbacks of the plane failure assumptions. The author of the method himself is of the opinion that the failure arc more resembles an arc of a circle or a logarithmic spiral; still in order to simplify the calculations he has approximated the failure arc to a straight line.

Though the formula and the application of the same is very simple for design and investigations of slope, this can be considered as a very rough method and hence be used only for trial design and finally should be checked by more accurate methods of analysis. This method may give better results for steep slopes with high value of ϕ.

4.10 METHODS ASSUMING CIRCULAR ARC FAILURES SURFACES.

The most exhaustive analysis of land slide phenomenon was carried out by the Swedish Goeotechnic Commission whose investigations were prompted by the failure of a quay wall in Goeteberg in 1915 (75,95). Subsequently the Commission made a complete study of slides in road and railway cuts and found that the shape of sliding curve in Cohesive soils showed a decided curvature that could be approximated by an arc of a circle. The expression of the fundamental principles of the Swedish circular arc method are credited to K.E.Petterson and S. Hultin. In 1927, W.Fellenius and H.Krey perfected the method for practical application. Now several authors have contributed their own procedures assuming circular failure arc.

An infinite number of arcs may be drawn for any given slope, the one to be used being that which is least stable. Fellenius prepared a table to locate approximately the centre of the critical circle (Fig.9). There are other methods also to locate the critical circle (95,96).
If the positions of atleast two points are known before and after a slide, the arc representing sliding surface may be determined (42), as shown in Fig.5, where A and A' are positions of two known points before slide and, c & C' are their corresponding positions after slide. Perpendicular bisectors of AC and A'C' are drawn to meet at centre O.

A great number of authors have tried to improve and simplify the trial and error process of circular arc analysis;

each author claims that his procedure saves time and labour, or that it is more accurate.

<u>General discussions of methods assuming circular failure arc.</u>

The assumption that the failure surface resembles the arc of a circle is probable in homogeneous, cohesive soil, and that two is only an approximation. In several cases the shape of failure surface is found to be more curved and vertical towards the top, and flatter at the bottom resembling more to a logarithmic spiral or arc of an ellipse.

The circular arc analysis is statically indeterminate unless some simplifying assumption is made.

When stratification exists, in an embankment or cut, there is a tendency for rupture arc to deviate from the approximate circular arc, following the strata. This action results because the shearing strength parallel to the strata is smaller than that across the strata (95). In stratified soil compound curves may be tried after selecting the simple critical curve. In doing so it may be noted that radius of curvature is greater with higher angles of friction (19). The possibility of translation in combination with rotation should not be overlooked, especially where the dip of the strata favours such action.

An approximate method that may be used in investigations dealing with stratified soils consists first, of laboratory tests to obtain shearing strengths on planes parallel to the strata, then the stratification is ignored, the shearing strengths that have been obtained are assumed to be valid for all surfaces through the embankment and the usual circular arc analysis is used(95). Any two trial failure surfaces that are close together have particularly the same safety factors; thus the small differences in position between the circular arcs and actual failure surfaces will not have an appreciable effect on the results(Ref.Fig.6-a and b). The approximate method explained above is slightly conservative if there is a moderate amount of difference between the shearing strengths across and parallel to the strata, and it may be quite conservative if the difference is relatively large.

If the sub-soil contains one or more thin exceptionally weak strata, the surface of sliding is likely to consist of three or more sections that do not merge smoothly into one another (Ref. Fig.7). In the stability computations such a surface can not be replaced by a continuous curve without the introduction of an error on the unsafe side (97).

When a particular slip surface is considered, it is assumed that the upper soil mass slides upon the lower mass along the circular surface, both masses being regarded as rigid in themselves. Also it is assumed that when failure occurs, the shearing stress at each point of the slip surface reaches the shearing strength.(57). These two assumptions are questionable for all types of assumed regular slip surface, including the circular arc analysis.

But even though there are more approximations introduced in the circular arc analysis, the methods have been widely used for analysing the stability of embankments, cuts and also earth dams consisting of more than one type of soil, and found to be satisfactory, in most of the cases. When compared to the probable errors in the soil sampling, estimation of values of cohesion, angle of internal friction and pore water pressure the methods assuming circular arc failure surface may be satisfactory for ordinary purposes since the real sliding surface could be more or less replaced by a circular arc with sufficient experience in the field.

4.11. SWEDISH METHOD. (12.95).

The most widely used method of analysing stopes is the Swedish method based on circular failure surfaces.

(a) Application to cohesive soils. The angle of internal friction ϕ is assumed to be zero and hence the shear strength is taken as that due to cohesion alone acting along the failure arc.(Refer Fig: 4-a). The factor of safety is given by

$$F = \frac{\text{Restoring moment}}{\text{Disturbing Moment}} = \frac{Cd.R^2\theta}{Wd}$$

where R is the radius of the arc, θ, the central angle of failure arc, W the weight of the sliding mass and d the distance of its centre of gravity from the centre of circle. The mass ABCD is assumed to rotate about the centre $Q1$. (12).

xxxx (b) Applicatbn for Frictional Soils- Method of Slices (12,77,95)

Most of the soil have both friction and cohesion and hence the method of slices is adopted. The method is based on the statical analysis of the mass above any trial failure arc with this mass considered to be made up of vertical slices. The vertical pressure on any area of slip-surface is assumed to be equal to the weight of the column of earth directly above. The resultant thrust of earth between the vertical sides of the slices is ignored. Weight of each slice is resolved into normal and tangential components to the arc, N and T. The factor of safety is given by

$$\frac{\text{Restoring moment}}{\text{Disturbing moment}} = \frac{Cd\ R\theta + \Sigma N \tan \phi\ d}{\Sigma T}$$

several trial arcs are analysed and the one with least factor of safety is found out. If the pore water pressure U is known from the flow net or field measurements at the sliding surface,

$$F.S. = \frac{\Sigma (N - U) \tan \phi d + Cd.L}{\Sigma T.}$$

The stability may also be analysed graphically.(95)

Discussions.

The Swedish method is applicable to homogeneous, isotropic, finite slopes (95) and assumes a circular failure surface which is an approximation. But the method is simple and most commonly used, and in several cases, the results agree closeby with those found in practice. In addition to the criticisms about the approximation of the failure surface to be a circular arc, the simplifying assumptions in this method about the magnitude of the thrusts between the slices generally leads to a conservative estimate of the factor of safety. In order to make the problem statically determinate, it is assumed that the forces on opposite sides of each slice is equal and opposite. This is equivalent to the assumption that the pressure on any area of slip surface is equal to the weight of the column of earth directly above it, which is not correct.(5).

Ref: Fig: 4-c. It is assumed that $(x_n - x_{n+1}) = 0$ and $(E_n - E_{n+1}) = 0$, which is not true except where angle of internal friction is constant along the slip surface and the anlge between the element PQ and horizontal, α, is also a constant. The error involved by these simplifying assumptions is likely to be of particular importance where deep slip-circles are involved and particularly marked for higher values of excess pore pressure (5). The magnitude of the error caused by these simplifying assumptions are discussed in detail under Art. 4.13 and also illustrated in figures 11,12, and 13. This method of slices is advantageous because it is flexible and it can be applied even when the slope to be analysed consists of different materials with different specific gravities, and also when external loads are present.

From the investigations of several failures in clay slopes, it is seen that there is a progressive decrease in the cohesion intercept with time, after construction and $\phi = 0$ analysis give results, in some cases, on the unsafe side. In the above cases, analysis assuming $c = 0$ gave too conservative results. For normally consolidated clays, $\phi = 0$ analysis is satisfactorily used.

For post-glacial clays no condition of anisotropy need be introduced (4). But it is always advisable to take the pore water pressure into consideration and analyse a slope based on effective stress.

Errors are introduced not only by the assumptions of the method of analysis, but also by the sampling, and testing procedure and so in many cases an approximate method of stability analysis may be considered adequate. But where considerable care is exercised at each stage, and in particular where accurate field measurements of pore pressures have been made, the simplified slices methods of analysis is not sufficiently accurate. When the actual pore pressure is not known from accurate field measurements, the use of simplified slices method may reasonably be justified, because pore pressure is a factor which is most difficult to assess from laboratory data alone.

4.12 MODIFIED METHOD OF SLICES (66,100,31,53,81).

In this method, each slice is reduced to a thin vertical strip of width dx, so that its weight is proportional to the height of the slip and then resolved to normal and tangential components n and t, and are plotted in a horizontal scale.(Ref. Fig.8). The values of ΣN and ΣT are found by multiplying the area found by planimeter by the unit weight w of the soil, and the factor of safety is found as in Art. 4.11. When pore water pressure exists, the value of   also should be plotted as shown in fig. 8-e and included in the calculation of factor of safety. The method may be extended for more complicated cases such as irregular slopes, (100) and for composite section as in Fig. 8-f.

Discussions.

In the Swedish method of slices the area and centre of gravity of each slice is to be found out separately which consumes more time, whereas this can be avoided in this method, by taking thin strips ~~and~~ so that the weight of the strip is proportional to the height. Moreover labour and time can be saved by the use of planimeter to find the normal ,& tangential forces and the pore water pressure. The method is easily applicable for complicated cases with irregular and composite sections. Otherwise the method is subject to the same criticisms as the Swedish Slices Method, (Ref. Art. 4.11) and also in general, for the circular arc analysis, Art : 4.10)

This method is widely used for the design and analysis of cuts and embankments and also earth dams slopes. The present place of this method in design and analysis of slopes is due to the fact that what it lacks in theory is made up by experience.

4.13. BISHOP'S METHOD (5).

A.W.Bishop (5) suggested a more rigorous method of stability analysis for eliminating the error introduced in the method of slices in ignoring the resultant thrust between the vertical sides of the slices. A typical slice is considered (Fig.10) and it is assumed that no external

__Discussions.__

Errors may be introduced xxx into the estimate of the stability by both sampling and testing procedures, and in many cases the approximate method of stability analysis may be considered adequate. Where, however, considerable care is exercised at each stage, and, in particular, where field measurements of pore pressure are being used, the simplified slices method is not sufficiently accurate and hence for more accurate results the Bishop's' method may be used. In a number of cases the uniformity of the soil conditions or the importance of the problem will justify a more accurate analysis, particularly if this is coupled with field measurements of pore pressure which is a factor most difficult to assess from laboratory data alone.

The likely error in the simplified method is of particular importance where deep slip circles are involved and the central angle of the arc increases. The error is particularly marked for higher values of excess pore water pressure, as seen from figures 11,12 and 13, in which the factor of safety found by the conventional method of slices is F_1 and that found by Bishop's method is F_2.

From practical point of view it may be noted that, although there are a number of distributions of $(x_n - x_{n+1})$ which satisfy eqn:(iii), the corresponding variations in the value of F.S. are found to be insignificant (less than 1% in a typical example) and so if the analysis is carried out as far as the $(x_n - x_{n+1}) = 0$ stage, this may be used for routine works also. It may be noted that since factor of safety. The method may be extended for more complicated cases such as irregular slopes, (100) and for composite section as in Fig. 8-f.

Discussions.

In the Swedish method of slices the area and centre of gravity of each slice is to be found out separately which consumes more time, whereas this can be avoided in this method, by taking thin strips ~~and~~ so that the weight of the strip is proportional to the height. Moreover labour and time can be saved by the use of planimeter to find the normal ,& tangential forces and the pore water pressure. The method is easily applicable for complicated cases with irregular and composite sections. Otherwise the method is subject to the same criticisms as the Swedish Slices Method, (Ref. Art. 4.11) and also in general, for the circular are analysis, Art : 4.10)

This method is widely used for the design and analysis of cuts and embankments and also earth dams slopes. The present place of this method in design and analysis of slopes is due to the fact that what it lacks in theory is made up by experience.

4.13. BISHOP'S METHOD (5).

A.W.Bishop (5) suggested a more rigorous method of stability analysis for eliminating the error introduced in the method of slices in ignoring the resultant thrust between the vertical sides of the slices. A typical slice is considered (Fig.10) and it is assumed that no external force act on the surface of the slope and that equilibrium exists between the weight of the soil above ABCD and the resultant of total forces on ABCD. The factor of safety works out to, F.S. $= \frac{R}{\sum wx} \cdot \sum \Big\{ c'l . \tan \phi' \times$ $(W \cos \alpha - ul) + \tan \phi' \left[(x_n - x_{n+1}) \times \cos \alpha - (E_n - E_{n+1}) \sin\alpha \right] \Big\}$

where, C' and ϕ' denote unit cohesion and friction angle in terms of effective stress, u, the pore pressure, W, the weight of the slice considered, l the length BC and α, the angle made by BC with horizontal. In the above equation, terms containing x_n and E_n do not disappear except where ϕ' and α are constant along the slip surface.

In practice, in order to simplify the above equation, the factor of safety is found out by first assuming that $(x_n - x_{n+1}) = 0$ in the following equation.

$$F.S. = \frac{1}{\sum W \sin \alpha} \cdot \sum \left\{ \left[c'b + \tan\phi' . W(1 - \bar{B}) + (x_n - x_{n+1}) \right] \times \left[\frac{\sec \alpha}{1 + \frac{\tan \phi' . \tan \alpha}{F.S.}} \right] \right\}$$

$$= \frac{1}{\sum W \sin \alpha} \cdot \sum (m) \quad \ldots\ldots\ldots\ldots (ii)$$

where $\bar{B}$ is a soil parameter given by $u = \bar{B} \frac{W}{b}$, and b is the breadth of the element. Then the values of $(x_n - x_{n+1})$ are introduced to satisfy the equation,

$$\sum \left[\frac{m}{F} \sec \alpha - (w - x_n - x_{n+1}) \tan\alpha \right] = 0 \quad \ldots.(iii)$$

These values then can be finally adjusted either graphically or analytically, until equilibrium conditions are fully satisfied for each slice.

Discussions.

Errors may be introduced ~~xxx~~ into the estimate of the stability by both sampling and testing procedures, and in many cases the approximate method of stability analysis may be considered adequate. Where, however, considerable care is exercised at each stage, and, in particular, where field measurements of pore pressure are being used, the simplified slices method is not sufficiently accurate and hence for more accurate results the Bishop's' method may be used. In a number of cases the uniformity of the soil conditions or the importance of the problem will justify a more accurate analysis, particularly if this is coupled with field measurements of pore pressure which is a factor most difficult to assess from laboratory data alone.

The likely error in the simplified method is of particular importance where deep slip circles are involved and the central angle of the arc increases. The error is particularly marked for higher values of excess pore water pressure, as seen from figures 11,12 and 13, in which the factor of safety found by the conventional method of slices is F_1 and that found by Bishop's method is F_2.

From practical point of view it may be noted that, although there are a number of distributions of $(x_n - x_{n+1})$ which satisfy eqn:(iii), the corresponding variations in the value of F-S. are found to be insignificant (less than 1% in a typical example) and so if the analysis is carried out as far as the $(x_n - x_{n+1}) \neq 0$ stage, this may be used for routine works also. It may be noted that since the error in the simplified method varies with the central angle of the arc, it will lead to a different location of the critical circle than given by this method, and hence it will be necessary to examine a number of circles.

No doubt, the method suggested by Bishop is more accurate one, among other simplified methods assuming circular failure arc. But the method is much more time consuming and laborious that it can not be adopted for common use. Moreover circular arc analysis itself in general is subject to various criticisms and hence it does not appear to be justifiable to go into such accurate calculations. Still, it is worthwhile to study the amount of error introduced by the simplifying assumptions.

4.131. GRAPHICAL APPLICATION OF BISHOP'S METHOD. (19)

In brief, this method consists of locating by trial the most dangerous slide curve, dividing the sliding section into slices, determining all forces on each slice, and then adjusting both components of the shearing strength of all the soils involved by the same trial safety factor, until closure of force polygon is obtained.

Trial curves of circular shapes are selected so that the greater portion of the curve will lie in the weaker soils or in zones in which hydrostatic pressures are most unfavourable. These curves may be circular arcs or combinations of arcs of different radii, connected by tangents, depending upon the uniformity of the section. Since both the slopes and position of the curves are determined by trial, a number of solutions must be made before the most probable critical curve may be approximated. One such trial curve AB is shown in Fig.No. 17-a.

The sliding section above curve AB is divided into a number of slices, the width of which are selected roughly in proportion to the radius of curvature of AB and to facilitate computation. As an approximate guide, the slice width should usually be chosen such that the chord subtended by that portion of curved failure surface is essentially the same length as the arc. The forces which are presumed to act on each slice are shown in fig.17-b. The hydrostatic forces U_1, U_2 and U_3 acting on the vertical faces and base of the slice are computed from the equi-pressure lines. The intersurface soil forces E are unknown in magnitude and direction. However, some latitude may be exercised in arbitrarily fixing the direction of these forces without greatly influencing the computed safety factor. Although the lines of action vary considerably at the different interfaces, a satisfactory simplifying assumption is that the E forces act approximately parallel to or make somewhat greater angle with the vertical than the embankment slope. The resisting force of cohesion, Cd, assumed to act parallel to chord GH, is equal to chord GH times the developed unit cohesion Cd. Force $\overline{F}$ acting at angle Ø d with the normal to GH is the resultant of the effective normal force at the base and the developed frictional force. The design values of C and tan Ø for all soils through which curve AB passes are divided by a trial safety factor to obtain Cd and tan Ød. The known forces W, W_1 U_z and U_3 for each slice are resolved into force R as in Fig.17-C. The trial force polygon is then drawn (Fig.17-d) using for each slice the known magnitudes and directions for forces R and Cd, the assumed directions of the E forces, and the directions of the $\bar{F}$ forces as determined by the angle Ø d, If this polygon fails to close another trial safety factor is chosen and the procedure repeated until closure is obtained and that will be the factor of safety for the assumed failure surface AB. Thus several failure surfaces should be analysed and the one with least factor of safety found out.

Discussions.

The method of determining the safety factor is merely one means of expressing the two conditions for equilibrium, that summation of vertical and horizontal forces must each be equal to zero. Applications of the requirement that the summation of moments of external forces acting on the sliding mass be zero, is not applicable since in this modified method there is no single centre of rotation.

This method is particularly useful in the analysis of composite embankment and foundation sections. While more laborious in application than other simplified methods of circular arc analysis like the Ø circle method, slices method and Taylors stability charts, this modified method is more accurate and may be recommended in all cases in which the embankment or its foundation is composed of zones of different soils having widely varying shear strengths.

4.14. THE "Ø - CIRCLE" METHOD.

The Ø-circle method was first proposed by Prof. G. Gilboy and A casegrande, its initial use being in the development of a completely graphical solution. A mathematical solution also was suggested by D.W. Taylor (94). The method is based on the assumption that the resultant reaction of the circular failure line touches the friction circle of radius R sin Ød.

(a) Graphical Solution. (12,63,95).

The actuating forces are weight of sliding mass W, the resultant boundary neutral force U, and the resisting forces are resultant cohesion and boundary intergranular forces(Fig. 14-a). The vector U, and vector B, the resultant of W and U, may also be found graphically (95). The resultant cohesion force C, acts at a distance $R.\frac{La}{Lc}$, parallel to the chord AB of failure arc. Though the intergranular force at each element of arc is tangent to the friction circle, all these forces do not pass through a single point and hence the resultant of all intergranular forces P, does not actually touch the friction circle. So a coefficient K is introduced so that P touches the modified friction circle of radius K.R.sin Ød, where K is statically indeterminte and depends upon the distribution of intergranular stresses along the arc, and also on the central angle AOB. The value of K is found for an assumed pattern of stress distribution, usually of sinusoidal form (Fig.15-a). Force triangle is drawn with knwon magnitude and direction of B, and the directions of P and C as in Fig. 14-c. The factor of safety with respect to strength may also be found as shown in Fig.15-b and C (95).

Several trial failure arcs are analysed to locate the critical circle having least value of F.S. with respect to strength. The method is also applicable for clay slopes (91) as given in Fig.18.

(b) Mathematical solution of Ø-circle Method (94).

For the case of failure arc passing through the toe of the slope, the value of stability number $\frac{C}{F_c wH} = \frac{\frac{1}{2}\,\mathrm{cosec}^2 x \times (y\,\mathrm{cosec}^2 y - \cot y) + \cot x - \cot i}{2\cot x \cdot \cot v + 2}$ when the rupture surface passes below the toe of the slope at a distance nH from the toe end, $n = \frac{1}{2}(\cot x - \cot y - \cot i + \sin\phi\ \mathrm{cosec}\ x.\ \mathrm{cosec}\ y)$.

and $$\frac{C}{F_c wH} = \frac{\frac{1}{2}\,\mathrm{Cosec}^2 x\,(y\,\mathrm{cosec}^2 y - \cot y) + \cot x - \cot i - 2n}{2\cot x.\ \cot v + 2}$$

These equations are derived mathematically in Ref.(No.94).

Different values of x and y are chosen (Fig.16) and the corresponding value of $\frac{C}{F_c wH}$ is computed in each case and the maximum value of $\frac{c}{F_c wH}$ corresponds to the critical failure arc. The value of K for sinusoidal variation of force P is given by $K = \frac{1 - (2y/\pi)^2}{\cos y} - 1$.

Discussions.

The ϕ-circle method is one which assumes a circular failure surface and hence is subject to all criticisms of circular arc analysis in Art: 4.10. The method also assumes ~~in~~ soil to be homogeneous. To make the problem statically determinate it is assumed that the resultant intergranular force tangents the friction circle of radius $R \cdot \sin \phi d$, which is not correct. The correction factor K which is introduced in the modified method, is also statically indeterminate unless the distribution of intergranular stresses along the sliding arc, is assumed, since it is unknown. Hence the distribution of the stress is assumed to be uniform, or as having zero values at ends of arc and sinusoidal variation in between, which need not necessarily be correct because in some typical examples, the actual distribution of stress is not symmetrical. But otherwise, it bears quite close resemblance to the sinusoidal form of stress variation and hence the values of K obtained by this assumptions may give sufficiently correct results.

The modified ϕ-circle method of analysis agrees closely with the values obtained by the Swedish Method of slices. But the drawback of the friction circle method is that it is not as flexible as the method of slices so as to accomodate variations of the slope, different values of density and soil strength factors.

4.15. TAYLOR'S STABILITY NUMBER AND CHARTS(12,63,94,95).

For geometrically similar slopes, i.e. slopes with the same inclination, but of different heights, the critical surface of rupture is similar in proportions, provided the materials have the same angle of internal friction and it is seen that $\frac{Cd.La}{W}$ is a constant for such similar slopes, where La is the length of the arc of rupture, (assuming circular arc analysis). The length of the chord L of the rupture arc will be proportional to the height of slope H and the rupture area proportional to H^2. Then,

$$\frac{Cd.La}{w} = const = \frac{Cd.H}{H^2 w} = \frac{Cd}{wH}$$, where w is

the ~~xx~~ unit weight of the soil. Factor of safety w.r.t. cohesion $Fc = \frac{c}{cd}$ where c is the actual unit cohesion and cd is the developed or mobilised unit cohesion.

Then $\frac{cd}{wH} = \frac{c}{Fc\ wH}$ = a dimensionless number, which Taylor called, the stability number N.

Taylor analysed a number of slopes using circular arc analysis and prepared a set of curves called stability curves connecting stability number and the angle of the slope for various values of Ø. Ref. Fig.29. These stability curves may be used for design of slopes and also checking the stability of a particular slope. If the values of c, w, H and Ø are known for a soil then the angle of slope i, could be directly obtained for design purposes, from the Taylor's stability chart connecting $\frac{c}{F_c wH}$ and i for the desired factor of safety. Also the factor of safety Fc could be found out for analysing a particular slope.

Discussions.

The stability charts are prepared based on circular arc analysis and all short-comings of circular arc analysis will be present in these design charts also. But one can say that for approximate design of slopes and analysis, this is one of the most useful bit of work which saves time and practically eliminates the long and tedious calculations. The introduction of stability number also has shown way for further development of dimensionless analysis for slopes. This is a great contribution to designers who would like to use the ready made charts and save lot of time and labour. But the greatest limitation of the method is that it is strictly applicable for homogeneous soil only, and the factor of safety with respect to cohesion Fc only can be obtained. No allowance is made for the pore water pressure which is also a greatest draw back.

So this method may be recommended for the designers to estimate the angle of slope only approximately, and then the whole analysis will have to be carried out by one of the more accurate methods, to find the correct factor of safety with respect to strength for the existing conditions allowing for pore water pressure etc.

4.151. TERZAGHI'S STABILITY FACTOR AND CHARTS (96,97).

In the circular arc analysis of $\phi = 0$, case, let i be the slope angle, 2θ the central angle of failure arc and α the inclination of chord of failure arc with horizontal. Let W be the weight of the sliding mass and d the distance from the centre of failure arc to its line of action; R the radius of the circle, La the length of failure arc and cd the unit cohesion developed. Then for equilibrium taking moments about the centre O,

$$\therefore\ cd = W.\ \frac{d}{R.Ld} = wH.\ \frac{1}{f(\alpha, i, \theta)}$$

where $f(\alpha, i, \theta)$ is a function of the angles α, i and θ, and w is the unit weight of soil.

If c is the available cohesion, and Hc is the critical height of the slope $c = \frac{w\,Hc}{f(\alpha, i, \theta)} = \frac{wHc}{Ns}$

or $Ns = \frac{wHc}{c}$, where Ns is a dimensionless number, called the stability factor, by K. Terzaghi.

In case where $\phi > 0$, then $c = \frac{wHc}{f(\alpha, i, \theta, \phi)} = \frac{w\,Hc}{Ns}$

and $Ns = \frac{wHc}{c}$; here the stability factor depends on α, i, θ and ϕ.

Terzaghi has also analysed several slopes and prepared charts connecting the stability factor Ns and the angle of slope for various values of ϕ, which also as Taylor's charts, may be used as design charts. Please refer page No. 159, (97).

Discussions.

It is seen that for any soil, when the factor of safety Fc, w.r.t. cohesion is unity, Taylor's stability number $N = \frac{c}{wHc}$ and Terzaghi's stability factor $Ns = \frac{wHc}{c}$ which is just the reciprocal of the former one. But Terzaghi's stability factor Ns is always greater than one where as the stability number N is always less than one.

The stability factor N_s and the charts have the same limitations as discussed under Taylor's stability Number and charts.

4.16. SHEAR RESISTANCE ENVELOPE METHOD (101, 14).

The method assumes circular failure arc; the distributions of the normal pressure and shearing resistance are unknown. Let S & N be the magnitude of resultant shearing resistance and normal pressure, found out by incorporation of suitable assumptions.

$$\text{Let} \quad \bar{S} = \frac{S}{Lc.wH}$$

$$\text{and} \quad \bar{N} = \frac{N}{Lc\ wH}$$

where $\bar{S}$ and $\bar{N}$ are dimensionless numbers. For a particular slope, several set of values of $\bar{S}$ and $\bar{N}$ may be obtained corresponding to each trial arc and all these may be plotted on a $\bar{S}$ - $\bar{N}$ diagram as shown in fig.30, and the envelope obtained is called shear resistance envelope, for the particular slope, representing a general relation between normal pressure and shear strength but is independent of soil characteristics, this being a plot between dimensionless quantities.

Assuming the shear strength to follow Coulomb's Law, the strength line is plotted on the $\bar{S}$ - $\bar{N}$ diagram by having an ordinate $\frac{c}{wH}$ and a line PQ at an angle ϕ to the $\bar{N}$ axis, as shown in fig.30.

$$\text{F.S.} = \frac{\text{Shear strength available}}{\text{Shear strength required for equilibrium}} = \frac{Y_1}{Y_2}$$

The smallest ratio of $\frac{y_1}{y_2}$ gives the F.S.

On the basis of above procedure several plane slopes from 15° to 90° were analysed by P.C.Varghese and the results tabulated (101). The method may be extended for irregular slopes, when pore water pressure exists, for sub-merged slopes and sudden draw down cases.

Discussions.

The average shear resistance may be found out only by means of simplifying methods. P.C.Varghese has assumed uniform shear resistance along the circular failure arc, and found that this arc gives approximately correct results. The average shearing resistance may also be found by the modified method of slices which assumes that the stresses on the vertical boundaries of slices may be neglected, and by drawing the curves for normal and tangential forces as given by modified method of slices , Art:4.12. But any one of the simplifying assumption has to be made for the assumed circular failure surface, when the actual distribution is not known.

In this method using shear resistance envelope, the calculations are reduced where the site conditions are such that investigation of stability is to be checked for varying values of c and Ø. But when only one set of shear constants are to be used for checking stability, this method is in no way advantageous than the usual methods.

The method is also subject to criticisms for the assumption of circular arc failure surface and homogeneous soil mass.

The values of $\bar{S}$ and $\bar{N}$ used in the shear envelope by dividing the total shear force and normal force by LwH may be said to be empirical since it has no rational significance. But to arrive at diamensionless relation slopes for simplifying the problem such terms are to be introduced.

When the value of Ø is assumed and the factor of safety of slope is found with respect to cohesion by trial and error, if the value of Ø is to be altered, then the whole process of trial and error will have to be repeated. Under such circumstances if the method using shear resistance envelope is adopted, the trial and error process may be carried out only once, and this is the advantage of the methods.

4.17. STABILITY ANALYSIS WITH DIMENSIONLESS PARAMETERS.(57)

The potential sliding surface is assumed to be cylindrical and a two dimensional analysis is made.

(a) Application for purely Cohesive soils, Ø = 0.

Shear strength is assumed to be constant along the entire surface and in case soil islayered strength is constant within each layer.

For simple slopes F.S. = $\frac{Ns.c}{wH}$ where Ns is the stability factor equal to $\frac{wHc}{c}$.

F.S. is also given by $\frac{\text{Resisting moment}}{\text{overturning moment}}$ and the least value of it should be found out.

F.S. = $\frac{\int_0^\theta S_z R^2 d\alpha}{a L a - G \cdot LG}$ where Sz is the shearing strength at point z, Q is the resultant of all vertical forces, G, the resultant of all horizontal forces including hydrostatic pressure in cracks, L_a and L_b are moment arms of Q and G about O.

When there is a surcharge of constant intensity of on its horizontal surface, the theoretical investigation of stability of slopes yields,

$$F = \mu_q \cdot NS. \frac{c}{wH+q}$$ where is a dimensionless reduction factor.

For partial submergence and draw down conditions,

$$Fs = \mu_w Ns. \frac{c}{wH - WwHw}$$, where μ_w is a dimensionless reduction factor whose magnitude depends on the ratio $\frac{Hw}{H}$ and the slope characteristics β and d.

When tension crack exists, $Fs = No. \frac{c}{WH}$

For combination of surcharge, submergence and tension cracks,

$$F = \frac{N's.C}{wH + q - WwHw}$$

where $N's = \mu_q \cdot \mu_w \cdot \mu_t \cdot Ns = \mu_d \cdot Ns.$

(b) <u>Stability Analysis of soil Slopes when $\emptyset > 0$</u>

In this also the potential sliding surface is assumed to be cylindrical, and shear strength is supposed to be fully mobilised at every point along the sliding surface, except in zones containing tension cracks. It is assumed that the soil above the sliding. Surface is made up of vertical slices and the combined lateral forces on the two sides of each slice are equal. Shear strength is assumed to follow Coulomb's Law.

Factor of safety is given by $F = \frac{N_c \cdot C}{WH}$, where

N_c is a dimensionless number, depending upon the angle of slope and parameter $\lambda_{c\phi} = \frac{Pe \tan\phi}{c}$

When surcharge, tension crack, submergence and steady seepage exist, $F = \frac{N_{cf} \cdot C}{pd}$

Where $Pd = \frac{W_{sat} \cdot H + q - W_w \cdot H_w}{U_d}$

where $U_d = U_w \cdot U_q \cdot U_t$, found as before.

For derivations of formulae and details please refer Appendix of Ref.No:57.

<u>Discussions.</u> (14,57)

The method is subject to all the draw backs of Swedish circular arc method, because the same simplifying assumptions are being used. But the trial and error procedure is reduced by the introduction of dimensionless parameters and simplifying graphs from which values of parameters can be directly obtained. For simple slopes the method is similar to the resistance envelope method.

By the introduction of the appropriate set of dimensionless parameters, the mathematical calculations necessary for the determination of critical stability condition can be carried out once and for all independent of the individual values of shear strength characteristics.

In analysing the xx influence of surcharges of constant intensity, water filled tension cracks of known depth, partial submergence and various drawdown conditions, on the factor of safety, the stability number for single slopes can be utilised. The critical values of the dimensionless factors included in the working formulas can be obtained by interpolation from numerical solutions presented in the forms of graphs, and thus the time needed for a complete analysis is considerably reduced as compared to the trial and error procedure.

From the studies the following conclusions are arrived at.

During a gradual lowering of the waterlevel, from complete submergence to complete drawdown, the factor of safety decreases very much quicker at the beginning than at the end of the draw down, chiefly because the stabilising hydrostatic pressures components decrease in proportion to the square of the depth of water. It is, therefore, observed that the %loss in factor of safety during the first half of the draw down is from four to six times as large as the %loss for last half of the draw down.

Regardless of the type of soil it is found that the % reduction in safety caused by a surcharge of constant intensity decreases with decreasing slope angle.

In analysing stability of slopes it is helpful to include the conditions of slow and sudden draw down because analyses of these conditions lead to the maximum and minimum possible value of an safety factor.

4.18 IVANON'S METHOD.

Cylindrical sliding surface is assumed. Arbitrary system of forces is assumed to act, as shown in Fig. 19. Total shearing stress tending to move the section, T, is given by $\int_A^C t\,ds$, where ds is the differential of the arc of the slip surface. Total frictional force $F = f \int_A^C n\,ds$ where $f = \tan \phi$, is assumed to be constant. Factor of safety against sliding $F = \frac{f \int n\,ds + CS}{T\,ds}$ where C is the unit cohesion and S the total length of arc. Factor of safety in terms of coefficient of friction, $F\phi_1 = \frac{f \int n\,ds}{\int \tau\,ds - cs}$

and factor of safety in terms of cohesion $Fc_1 = \frac{cS}{\int T\,ds - f \int n\,ds}$

Let R be the resultant of all forces acting and Σx and Σy be its resolved components along ox and oy, (xo, yo) being the co-ordinates of the points of intersection of resultant with the sliding arc of radius R.

$$\int t\,ds = \frac{xo\,\Sigma y - yo\,\Sigma x}{R} \quad \text{and} \quad \int n\,ds = \frac{xo\,\Sigma x + yo\,\Sigma y}{R}$$

$$\therefore \quad F = \frac{f\,[xo\,\Sigma x + yo\,\Sigma y]}{xo\,\Sigma y - yo\,\Sigma x + CRS}$$

and
$$F_{c1} = \frac{C.S.R.}{xo\,[\Sigma y - f\Sigma x] - yo\,[\Sigma x + f\Sigma y]}$$

$$F_1 = \frac{xo\,\Sigma x + yo\,\Sigma y}{xo\,\Sigma y - yo\,\Sigma x} + \frac{CRS}{xo\,\Sigma y - yo\,\Sigma x}$$

i.e. $F_1 = F\phi_1 + Fc_1$.

When the minimum value of F_1 is required to be found out for all circles passing through A (Fig. 19), any point c is first to be chosen representing where the sliding surface intersects the outer surface of the slope and let be the angle which AC makes with horizontal. The minimum value of F_1 is first found for all circles passing through A and C, the centres of these circles will be on the perpendicular bisector of AC. Curves $F\phi_1$, Fc_1 and F_1 are plotted (Fig.20-a) and F_1 min. is found out. Then a different value of λ and the procedure repeated. A graph is plotted connecting λ vs. F_1 min. is obtained, for the slope. Again if necessary various locations of point A, other than the toe of the slope may be tried and the whole calculations repeated to see if a lesser value of factor of safety could be obtained.

Discussions.

Circular arc failure surface is assumed first. The value of total normal stress $\int n ds = N$, is taken as the geometrical sum of normal stresses along the sliding arc. But this assumption is not quite true since $\int n ds$ represents an algebraic sum of stresses. The assumption leads to an error of about 2% in a typical example. More over it is not yet clear which of the two interpretations approaches the actual case more closely.

In contrast to the methods advocated by Hultin, Krey, Fellenius and Terzaghi, who solve the problem by grapho-analytical integration method, and who thereby obtain the total normal strain acting on the sliding surfaces in the form of a scalar, this method has expressed the force in the form of a vector.

This method is limited to dry slopes of arbitrary form subjected to the action of an arbitrary system of forces. A.I. Avanov, the author of this method claims that the labour is reduced by this method of analysis, which is doubtful.

4.19. MEYER'S ANALYTICAL METHOD. (68)

In this method, the usual circular arc failure surface is assumed. The solution given is based on the further assumption that the rupture arc passes through the toe of the slope and edge of the crown, as shown in fig.21. Factor of safety is given by,

$$F = \frac{Ns}{f_1} + f_2 \cdot \frac{\tan \phi}{\tan i}$$

where Ns, the stability factor is $\frac{c}{wH}$,

and $\frac{1}{f_1} = 6y \operatorname{cosec}^2 y.$, and

$$f_2 = 1 + \sec^2 i \left[-1 - 3/2 \cot y \, (y \operatorname{cosec}^2 y - \cot y) \right]$$

The arc giving lowest factor of safety is given by the expression,

$$\frac{\tan \phi}{Ns. \sin i.\cos i} = \frac{8\, y.\cot y - 4}{3y \operatorname{cosec}^2 y - 3\cot y - 2y}$$

where, 2 y is the central angle of the failure arc.

The expression $\frac{\tan \phi}{Ns. \sin i \cos i}$ is plotted against y, and for any set of values of c, ϕ, W, H and i, the value of y for minimum F can thus be determined without trial and error.

When the arc does not pass through the edge of the crown and toe of the slope, the value of F.S. for trial arc is given by

$$F = \frac{d}{d'} \left[\frac{wNs}{W' f_1} + f_2 \frac{\tan \phi}{\tan i} \right]$$

where w' is the total weight including superimposed load if any divided by the area of the segment and the change in the load distribution on the arc is neglected; d' is the horizontal distance from the new centre of gravity to the centre of circle.

When there is seepage water, F.S. is given by

$$F = \frac{c'}{wH f_1} + f_2 \frac{\tan \phi}{\tan i} - \frac{6u \cdot \tan \phi \cdot \sin \phi}{wH^2 \sin y} .$$

where 2y' is the central angle of the portion of trial arc below the known seepage water surface and c' the weighted mean cohesion.

<u>Discussions</u>

The trial and error work of the Swedish methods is avoided by this method but this is applicable to only the failure circles which pass through the edge of the crown and the toe of the slope, and usually this is not the case. When the failure arc is different from this pre-assumed path, the value of Ns no longer is independent of y. H, remaining constant, w' will have a different value ~~of~~ for each value of y; where a portion of the mass becomes saturated due to seepage or other conditions of nonhomogeneity of the soil exist, the cohesion c may vary with different values of y. So this method of mathematical analysis is of limited application and should be also restricted to purely homogeneous soil. In zoned banks, where values of c, w and ϕ may vary from one zone to another, trials can not be altogether be avoided.

In few cases, a large number of circles have factor of safety within 0.6 % of the theoretical minimum and for flat slopes of homogeneous bank the critical circle is difficult to be located even by drawing contours of equal F.S., since the locus of minimum some times elongates and reduces to a line. So in such cases a few trials are to be made to find out the absolute minimum F.S.

In some cases the slide circle determined by the above analytical method may cut even a hard stratum or rock foundation when it is present just below the embankment and in such cases more trials will have to be made by individual judgement according to the site condition.

This method is applicable only for a limited cases and has much more limitations than the usual methods using circular arc analysis, even though the trial and error process is avoided. When the failure arc does not pass through the edge of the crown and toe of the slope, trial and error process can not be avoided.

4A.3. JAKY'S METHOD OF STABILITY ANALYSIS.

This method was first proposed by Franz Kotter in 1903 in Bericht der Berliner Akad and then J.Jaky developed it in 1936 (49) A slip surface is assumed and an element is considered on it to be in equilibrium and acted on by forces as shown in Fig.22-a and b.

Taking cylindrical coordinates, the equations of equilibrium are $\leq F_R = 0$, and $\leq F_\theta = 0$.

The basic equation for the element considered at failure surface is given by, $\frac{\partial \tau}{\partial \theta} - 2T \tan \phi + WR \sin \phi . \sin (\theta - \phi) = 0$. This is to be integrated for the whole failure surface, for which it should be assumed that the soil is homogeneous and isotropic. The equation to be integrated may be written as $\partial \sigma - 2c d\theta - 2 \sigma R \tan \phi . d\theta + WR \cdot \cos \phi . \sin (\theta - \phi) \, d\theta = 0$. This equation can be integrated for one of the following four conditions only.

i) $\phi = 0$.
ii) $W = 0$
iii) R = Constant
iv) $R = \infty$

Case (i) is applicable to purely cohesive soils only. Case (ii) is when gravitational forces are negligible which is not found ordinarily. Case (iii) when R = constant is a circular arc and case (iv) is a plane surface.

So Jaky's method finally reduces to a circular arc failure surface, if it should be possible to be used for ordinary soil slopes. If the equation be integrated between points 1 and 2 for R = constant, case (iii), the value of $\frac{cd}{wH}$ is given by,

$$\frac{F_1\phi\left[\sin\theta_2 - \sin\theta_1 \cdot e^{2\tan\phi(\theta_2-\theta_1)}\right] + F_2\phi\left[\cos\phi_2 - \cos\theta_1 \cdot e^{2\tan\phi(\theta_2-\theta_1)}\right]}{\left[(1-\sin\phi) - (1+\sin\phi)\, e^{2\tan\phi(\theta_2-\theta_1)}\right] 2\sin(i-\beta) \cdot \sin\frac{\theta_1+\theta_2}{2}}$$

where $F_1\phi = \frac{3\sin^2\phi}{1+4\tan^2\phi}$ and $F_2\phi = \frac{1}{2}\,\frac{\sin 2\phi(1-2\tan^2\phi)}{(1+4\tan^2\phi)}$

Theabove value of stability no = $\frac{cd}{wH}$ was calculated for various values of i and β.

Discussions.

When the element on failure surface is considered, the assumptions are that it is in static equilibrium and that there is continuity of stress. But in order to integrate for the whole failure surface further assumptions are also to be introduced that the soil is homogeneous and isotropic, as in most of the other methods. But it is possible to integrate the equation only for one of the four limited conditions, of which, w = 0 and R = ∞ are not practicable. The case when ϕ = 0 also has several limitations as discussed under Art: 4.11. Atleast the condition R = constant, could only be developed, which degenerates to circular failure arc, but this circular arc does not coincide with that found by other methods assuming circular failure arc, and hence the values of stability number found by this method do not agree closely with more accepted methods, as seen from Table No.1, Art. 4.5 comparing different methods. Moreover the method is very tedius and time consuming, that it is not in common use now, and it has no specific advantage over the usual swedish and friction circle method.

4.3 RESAL-FRONTARD METHOD.

This method, developed by J. Resal and M.Frontard, assumes the soil mass to act as a slope of infinite extent, and by use of conjugate stress relationships, results in an equation for the rupture surface. By this method, the depth upto which tension exists, is given by, $y_o = \frac{2c}{\omega} \times \tan(45 + \frac{\phi}{2})$.

The value of stability number is given by,

$$\frac{c}{F_c \omega H} = \frac{\sin(i-\phi)}{2\sin^2 i \cdot \cos\phi} \cdot \left[\frac{1}{\frac{\cos\phi}{\sin i(1-\sin\phi)} + \frac{\cos^{-1}\left\{\frac{\sin^2 i - \sin\phi}{\sin i(1-\sin\phi)}\right\}}{\sqrt{\sin(i-\phi)\sin(i+\phi)}}} \right]$$

Discussions.

This method has been criticised because the results indicate that the mass above the rupture surface is not in static equilibrium when just at the point of failure with all shearing strength being utilised. However, the results are on the side of safety, and it is interesting to note that on one very important point they agree with actual conditions by indicating that the upper part of the mass is in tension. This method assumes that no cohesion may be depended upon within the depth to which tension occurs.

4.4. LOGARITHMIC SPIRAL METHOD.(29,94,95).

From several observations of failure of slopes, it was suggested that the failure surface may be approximated to the arc of a logarithmic spiral. The use of log. spiral was recommended by Rendulic, to avoid the assumptions made by circular arc methods of analysis to make the problem statically determinate. The important property of a log. spiral expressed in polar co-ordinates by the equation $r = r_1 e^{\theta \tan \phi}$, is that all the radius vectors cut the curve at an angle of obliquity ϕ. In the above equation, r is the variable radius vector and r_1 is that at the begin point B of arc BA (Fig.23), θ, the variable angle between r and r_1 and e is the base of Naperian logarithms.

(i) <u>Analytical Solution suggested by D.W.Taylor(94).</u>

It is assumed that the resultant of the forces across the rupture surface is directed towards the pole O. Let Z be the central angle BOA, t the chord angle with horizontal, $m = e^{z \tan \phi}$, $g = \dfrac{1}{\sin t \sqrt{1+m^2-2m\cos Z}} = \dfrac{r_1}{H}$

$$j = t + \sin^{-1}\left[\frac{\sin Z}{\sqrt{1+m^2-2m\cos Z}}\right] \quad \text{and}$$

$q = \pi - z - j$. When the rupture surface passes through the toe of the slope, the value of stability No. is given by,

$$\frac{c}{F_c w H} = \frac{\tan\phi}{3g^2(m^2-1)}\left\{\frac{2g^3\left[(m^3\sin j - \sin q) - 3\tan\phi\,(m^3\cos j + \cos q)\right]}{g\,\tan^2\phi + 1}\right.$$
$$\left. + g^3\sin^3 q\,(\cot^2 j - \cot^2 q) + 3mg\cos j\,(\cot i - \cot j) - \cot^2 i + \cot^2 j\right\}$$

Similar expression is found for failure surface, passing below toe also. Stability numbers for different values of z and ~~xx~~ t are to be calculated and the critical failure are obtained, by trial, The F.S. with respect to strength may be calculated as discussed under Art. 4.14.

(ii) Analytical method suggested by OK. Frohlich(29).

In this method, the forces acting are found out and moments equated for equilibrium. But the actual centre of rotation of the mass is not known, which is assumed to be O_μ, of radius r_μ for the average value of $\mu = 0.5$.

The F.S. is calculated from the equation

$$F.S. = 1 + \frac{\frac{c}{R} \cdot \frac{r_2^2 - r_1^2}{2\tan\phi} - a_0}{a_0 + r_1 e^{\mu a \cdot \tan\phi} \cdot \tan\phi \cdot \sin(\mu a + \beta)}$$

Refer figures 24 to 27. R is the resultant of all driving forces, a_0 is the distance of R from O, a_1 is the distance of R from O_μ and β is the angle between r_1 and the normal to the direction of R.

(iii) Graphical method suggested by O.K. Frohlich. (29).

The polygon of forces is drawn as shown in fig.27, with forces R, C and Q acting on the ~~xxx~~ sliding mass. In order to make it in limiting equilibrium the force R is moved to a point M (Fig.27-a) by a distance e, and acting parallel to R, represented by Re. Then F.S. = $1 + \frac{e}{a}$

The point O_μ is plotted graphically as given below The point is chosen on arc AB such that $O\mu$ divides angle AOB symmetrically. Draw a perpendicular at O to $O\mu$. Erect a perpendicular to the tanget to curve AB at and let there two perpendicular lines meet at O_μ, which may be considered as the centre of rotation.

Discussions. (29,94,95). In some cases the failure surfaces resembled an arc of a logarithmic spiral, since the failure arc was more curved and steep at the top and then flattered towards the bottom. Moreover some authors introduced logarithmic spiral failure are to replace circular arc since the latter method of analysis is statically indeterminate unless some simplifying assumption regarding the distribution of normal stress along the circular arc, is introduced. In a logarithmic spiral the normal at every point on it makes a constant angle Ø with the radius vector from the asymtotic point to the point considered, the angle Ø, being the characteristic for the form of spiral. Because of this condition the analysis is statically determinate without an assumption relative to the pressure distribution.

Investigations by Taylor's Mathematical solution of the log. spiral method, has shown that the stability numbers closely agree with the Ø-circle method, (Please refer Table No.1, Art 4.5 comparison of various methods), and in addition the failure surfaces according to these two methods almost coincide. It was found that a computation by this method required about twice as long as one by the Ø-circle method.

The main disadvantage of the log. spiral method is that the centre of rotation is unknown and hence the problem is kinematically indeterminate (29). The procedure is suitable for giving a lower and upper limit of the factor of safety with respect to sliding, and an approximate average value of m it.

From the figures given under Frohlich's methods (Figs 24,25,26 and 27) it is seen that the surface seen that the surface element at B can only rotate about its centre of curvature O_1, the element at A about O_2 and element at about O_μ. The points O_1, O_μ, O_2 are located at the evolute of the ~~gig~~ given log.spiral which is also a log-spiral with characteristic Ø. The point O_μ which is considered to be the average point of centre of rotation for $\mu = 0.5$, is an approximation.

The graphical method suggested by OK.Frohlick is comparatively simpler than the analytical method. More correct graphical analysis may be possible by dividing the arc into several elements and taking the corresponding centre of rotation for each. But the process may be laborious since various such trial arcs are to be analysed to find the critical one.

From the analytical solution by Frohlick, it can be seen that for cohesionless soil, the limiting equilibrium is reached, when R passes through the centre O of the log. spiral. Also when $\phi = 0$, the log spiral degenerates into a circle of radius r_o, and the F-s is given by $F = \frac{a\, r_o^2 c}{R\, a_o}$, which is same as computed by $\phi = 0$ analysis.

TABLE NO. 1

4.5. TABLE COMPARING STABILITY NUMBERS BY VARIOUS METHODS.

Angle of slope i°	Ød°	Swedish Method of Slices.	Stability Numbers N = Cd/WH Ø-Circle Method	Log: spiral method	Shear Resistance Envelope Method.	Jaky's Method ✓	Culmann's Method	Resal Frontand Method. ✓
1	2	3	4	5	6	7	8	9
90°	0	-0.261	-0.261	-0.261	0.261	—	0.250	0.500
	5	-0.239	-0.239	-0.239	0.240	0.229	0.229	0.458
	15	-0.199	-0.199	—	0.200	0.192	0.192	0.384
	25	-0.165	-0.166	-0.165	0.168	0.159	0.159	0.319
75°	0	-0.219	-0.219	-0.219	0.218	—	0.192	0.396
	5	-0.196	-0.195	—	0.195	0.172	0.171	0.353
	15	-0.154	-0.152	—	0.155	0.135	0.134	0.280
	25	-0.118	-0.117	—	0.122	0.102	0.102	0.215
60°	0	-0.191	-0.191	-0.191	0.191	—	0.144	0.328
	5	-0.165	-0.162	-0.162	0.161	0.135	0.124	0.284
	15	-0.120	-0.116	-0.116	0.117	0.098	0.088	0.208
	25	-0.082	-0.079	-0.078	0.080	0.063	0.058	0.143
45°	0	-0.170	-0.170	-0.170	0.171	—	0.104	0.280
	5	-0.141	-0.136	—	0.136	0.109	0.083	0.231
	15	-0.085	-0.083	—	0.087	0.064	0.049	0.148
	25	-0.048	-0.044	—	0.046	0.028	0.023	0.081

1	2	3	4	5	6	7	8	9
30°	0	-0.156	-0.156	0.156	0.156	—	0.067	0.244
	5	-0.114	-0.110	—	0.110	0.082	0.047	0.183
	15	-0.048	-0.046	—	0.047	0.027	0.018	0.085
	25	-0.012	-0.009	0.008	0.007	0.00	0.002	0.017
15°	0	-0.145	-0.145	0.145	0.145	—	0.033	0.217
	5	-0.072	-0.068	0.068	0.073	0.035	0.015	0.118
	10	—	-0.023	—	0.025	0.00	0.004	0.043

Colns. 3, 4, 5 and 8 from (95)

" 6 from (101)

" 7 and 9 from (94)

It may be noted that the values of stability numbers given under Swedish, ϕ- circle, Log. spiral, and shear resistance envelope Methods agree very closely for all values of i and ϕ. From experience also these methods are found to give reasonably accurate results. But the stability numbers obtained by Jaky's and Culmann's methods (columns and 8) are found to be lesser than the corresponding values of the above mentioned four methods, indicating that these two methods give a greater value of factor of safety for any given slope and hence the results may be on the unsafe side. Jaky's and Culmann's methods are in closer agreement with the other methods when the slope is steep and the value of ϕ is greater ,and they deviate widely for flatter slopes. For a 90° - slope with $\phi = 15^{\circ}$, the error in Jaky's and Culmann's methods are in the order of 3.5 % each,where as for a 30°- slope with $\phi = 5^{\circ}$, the error in Jakey's method is about 26% and that in Culmann's method is about 55%. In the case of Resal Frontard method, (column -9) the values of stability numbers are very high and hence the values of factor of safety will be too conservative; the error is very great in steeper slopes.

4.6. Stability Analysis Using Pavlavsky's Theory for Phreatic Line (55,18)

The analysis of stability of slopes developed by N.N.Pavlosky is based on the consideration of forces exerted by percolating water; the theory is that the hydraulic pressures which occur in the downstream region below the line of saturation of the earth structure have a definite effect on the stability of slopes and should be taken into considerations. The stability of slopes where the line of saturation intersects the slope may be studied by an analysis of the forces which act on the particle of soil at the outlet point of saturation line, viz., the weight of the particle and the outward force of seepage flow in a tangential direction to the line of saturation at the outlet point (Fig.28). The F.S. may be found from the expression.

$$w'' (\tan \phi - \tan i) \cos i = F \tan i.$$

where $w'' = \frac{w'}{W_w}$ and w' is submerged unit weight and W_w the unit weight of water.

The stability of an earth slope may be improved by placing a blanket of selected granular material at the top of the slope and in this case the equation is given by,

$$(w'' + as') (\tan \phi - \tan i) \cos i = F \tan i,$$ thereby increasing the stability by $1 + \frac{as'}{w''}$, where, a is the thickness of the granular material and s' its bulk

specific gravity. Simplifying graphs have also been prepared by K.P.Karpoff, to find the stability directly.(55).

Discussions (38,65).

The method is applicable to check stability of an embankment when it retains water on one side and also in cuts and natural slopes where the face of the slope intercepts the natural position of the ground water surface. The Pavlovsky's theory is applicable only for homogeneous, isotropic soil. In most practical cases the formulas are found to be of only limited value, and the position of phreatic line will have to be estimated by the individual judgement from the available information.

The theory that involves Dupuit's assumption as that presented by Karpoff (55) can be regarded as, merely an approximate method for the determination of phreatic line, and can not be applied either for anisotropic soils or for the determination of head distribution in the saturated zone below phreatic line. This method cannot in general be used for analysing stability of earth slopes as the other methods.

4.70. METHODS USING THEORY OF ELASTICITY AND PLASTICITY.

In the usual methods a regular slip surface is assumed first and the forces are analysed to find the factor of safety. But in assuming a mathematical slip surface itself there is a lot of deviation from reality, for, rupture in soil will be irregular, passing through the heavily stressed regions (57). Moreover for analysing the stability of slopes further simplifying assumptions are to be introduced (3).

On the contrary methods using theory of elasticity and plasticity have been developed in which the stability of a slope is analysed by point to point stress analysis, and not by assuming any failure surface first. Much work have been done by J.H.A. Brahtz by this method towards the stability analysis. But the analysis by this method are extremely long and tedius so that an engineer not familiar with higher mathematics, if he attempts to analyse by the original statement of the method may find himself lost in the maze of formulas and completely divorced from physical concepts (100).

4.71. GLOVER-CORNWELL METHOD (33,100).

In this method the theories of elasticity and plasticity are used to find out the stresses in the particles and for this it is assumed first that a small element of volume is in equilibrium under the stress resultants and forces that act upon it, as shown in fig. 31. For every point to be in equilibrium $|T| \leq c + f\sigma$ and let $R = c + f\sigma - |T|$. T is the shear stress, σ the normal stress, ~~mmdxxxxxfxR~~ ~~the normal stress~~ and f equals tan ϕ. If Ro is the minimum value of R, when Ro = 0 there are two planes along which sliping is incipient and at such point the material is said to be in plastic state. When Ro > 0, it is assumed to be in elastic state obeying Hooke's Law.

For elastic region the equations are,

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial u}{\partial x} + hg = 0$$

and $$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial u}{\partial y} + w_o + w_w \cdot e = 0,$$

when c = 0, $$f \, \frac{\sigma_x + \sigma_y}{2} \geqq \frac{\sqrt{1+f^2}}{2} \sqrt{(\sigma_x - \sigma_y)^2 + 4\tau_{xy}^2}.$$

where hg is the earth quake acceleration, wo is the dry unit weight and e the void ratio. (For other notations please refer Fig.31). For a definite region the linear solutions may be obtained for the values of σ_x, σ_y and τ_{xy}, and the arbitrary constants introduced are to be evaluated from boundary conditions. The plasticity equation is given by $\sigma_x + \sigma_y = B\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau_{xy}^2}$ where $B = \frac{\sqrt{1+f^2}}{f}$

The directions of two families of slip line are given by $$\tan \psi_1 = -\frac{2f\tau_{xy} + \sigma_y - \sigma_x}{2\tau_{xy} - 2f\sigma_y}$$

and $$\tan \psi_2 = -\frac{2f\tau_{xy} - 2f\sigma_y}{2\tau_{xy} + 2f\sigma_y},$$ where ψ

is the angle between the y axis and slip line, positive clockwise. The existance of plastic zone means that the material in the zone has acquired all the load it can carry; any further load must be accomodated by encroachment on the reserve in elastic zones by a re-adjustmant of the boundary between the elastic and plastic zones. By this method factor of safety can not be evaluated;

an idea of reserve strength available, if any, may be obtained. For application of the method to an actual case, please refer (100).

Discussions.

The following assumptions are made (i) There must be equilibrium every where (ii) The pressure system must agree with the known conditions at the boundaries.(iii) The condition for shear failure is given by a relation of Coulomb's type (iv) In the elastic zones where the stress system does not permit shearing in any direction, the stress system must satisfy the conditions of compatability in the elastic sense (v) In the plastic zones the stress system is such that slippage between grains is ~~eminent~~. imminent.

The solution given by this method does not permit the evaluation of the effects of cohesion and neglecting cohesion altogether may lead to great error in estimating factor of safety in some types of soils.

In this method, there is no evolutation of a factor of safety, but only a determination of "reserve strength" in the elastic regions. Since the plastic regions used in this method are those in which failure is likely to occur, the method in question approaches the methods of class A, more closely than that of Brahtz, given later.

Some error results from the application of Hooke's Law to the elastic regions because, the stress-strain relationship for the soil is not quite linear; the curve of unloading does not completely coincide with the curve of loading, and there is some uncertainty about the legitimacy of applying the compatibility equation to soil.

The tenants of the Glover-Cornwell School of thought believe the grain structure to be less resistant to volume strain than the pore water and who postulate a readjustment of pore pressures occuring simultaneously with the removal of the reservoir load and that the pore pressure readjustment in this case is generally favourable to the stability; but this is in sharp contrast with the thought of Brahtzs and others.

4.72. BRAHTZ'S METHOD.(11,100).

In this method the physical properties of the material, pore pressure and plastic action of the earth, are taken into account. The equilibrium of every point in the earth bank may be determined by examining the stability of individual points; by the use of theory of elasticity the stresses are determined and by Coulomb's Emperical Law, the strength at corresponding points.

Various cases of draw down, full and empty reservoir cases with pore pressure, are considered separately. The following assumptions are made- (i) All stress components are zero in all boundaries of the embankment (ii) Equilibrium is maintained for all the points in the structure and (iii) the ratio between the total horizontal and vertical stresses is very nearly equal to K, the experimental compaction factor, which resembles Rankines Ratio $\frac{1 \mp \sin \phi}{1 \pm \sin \phi}$ and taken here as $\frac{50}{W_t}$ where wt, is the unit weight of saturated earth material.

The basic state of stress is found, neglecting the horizontal stresses and then a set of correction factors are introduced to the basic system. Also the embankment is treated as a triangular section with deduction on the effect of the smaller triangle above the crown. (Fig.32).

For details and derivation of stress functions please refer No.(11).

For a required F.S., the section of the bank may be made economical by decrdasing the critical pore pressure. The necessary decrease is given by

$$\Delta p = \frac{\frac{1}{\pi}(\sigma_1' - \sigma_2')(\text{Desired F.S}) - C}{\tan \phi} - \frac{\sigma_1' + \sigma_2' + 2P}{2} \quad \text{....(i)}$$

where P is the pore water pressure. It is also necessary to analyse the stability of foundation. Factor of safety is given by $F = \frac{Ta}{T}$, ---------- (ii)

where Ta is the available contact shear and T is the existing contact shear. Ta is found experimentally, governed by the relation $Ta = C + \sigma \tan \phi$, and even if this value is changed the shear stress will nct change and hence the calculation of the stress need not be repeated. The mean factor of safety against slippage in contact area becomes,

$$F = \frac{\pi}{2} \cdot \frac{2C + (\sigma_1' + \sigma_2' - 2P)\tan \phi}{(\sigma_1' - \sigma_2')} \quad \text{.......(iii)}$$

Brahtz's method of stability analysis has been reduced to a workable basis by U.S.Bureau of Reclamation (100) so that it will involve only interpolation from charts and graphs, and simple arithmetic.

Discussions.

From the equation (iii) above, it is seen that the F.S. is a function of both sum and difference of contact stress. The numerical value of factor of safety will entirely depend on the reliability of the physical constants C, and tan ϕ, and must be fixed by experience for each region of the structure in each individual case. The factor $\pi/2$ of equation (iii) may be left open as a correlating parameter to be adjusted in connection with the analysis, the method of experimentation, and the factor of safety finally adopted. The factor of safety obtained in this type of analysis are point functions, where as in the 'circle analysis' there are mean values obtained over circular arc. Therefore the minimum factor of safety as defined in this analysis will always be less than that found for the same structure by circular arc analysis.

In an earth structure, Hooke's Law does not hold good. The stress-strain relation-ship in earth is so decided by plastic that the stress computed in an earth structure assuming Hooke's law in elasticity and by use of which one unique mathematical solution may be chosen from the infinite number of possible stress distributions all of which are in a state of static equilibrium. Lacking a definite law of stress-strain behaviour we must look into the behaviour of the material under stress and study the laws of failure.

It is evident that the unique solution for the distribution of stresses in an earth dam is dependent upon numerous factors like, the physical properties of the earth structure, its geometrical shape, the forces acting i.e. ~~xxx~~ weight of the material, boundary forces and effect of percolation pressure, the method of construction and time and the loading history of the earth structure.

It is assumed that the mean properties of every cubic volume of material remains constant within certain specified region, where as actually the particles may vary in shape, size, specific weight, roughness, elastic properties, voids etc.

J.H.A. Brahtz was of the opinion that the amount of air to be trapped in the pores was sufficient to make the air water combination less resistant to volume strain than the grain structure. This has an important bearing on the question of stability following a rapid draw down since it leads to the conclusion that the pressures developed in the dam by a high reservoir level remain after the draw down until relieved by a process of diffusion. This is in sharp contrast to the tenets of the Glover-Cornwell school of thought, who believe the grain structure to be less resistant to volume strain than the pore water and who therefore postulate a readjustment of pore pressures occuring simultaneously with the removal of the reservoir load. The pore pressure re-adjustment in this case is generally favourable to stability. The merits of these two views will eventually have to be settled on the basis of tests.(100)

In the Glover-Cornwell method, cohesion has not been taken into consideration and there is no evaluation of factor of safety where as in Brahtzs method, cohesion has been taken into account and factor of safety can also be evaluated.

4.73 METHOD USING DIFFERENCE EQUATION. (20)

Shearing and normal stress for a trapezoidal embankment are computed by the substitution of finite difference equations for the differential equations of elasticity and by using the method of successive approximations or "Method of Relaxation" for solving the difference equations. The equations of the boundary planes of the elastic body are given in Fig. 33. The embankment and foundation are assumed to be symmetrical about the plane $x = 0$; the boundary $y = d$ is assumed to be rigid and no slipping occurs along that bounday. Poisson's ratio is taken as 0.5.

After the equations for stress functions have been written the method of relaxation is applied. A square mesh net formed by horizontal and vertical lines with the boundaries of embankment and foundation are drawn, the mesh length is taken as a constant. The computed or estimated value of ϕ is writeen at all net points and if these initial values do not satisfy the difference equations, then the method of relaxation consists in changing the originally estimated values of ϕ, point by point until all difference equations are satisfied or nearly so, at every point on the net where they apply. Decreasing the mesh size, increases accuracy. Contours of average normal stress divided by WL and contours of maximum shearing stress divided by WL are drawn and the two superimposed. A Mohr's diagram of state of stress at point of greatest shear stress and rupture line are drawn and a continuous stress envelope is drawn. From these another diagram can be drawn, as shown in fig.34, giving the cohesion and internal friction required in the embankment and foundation to prevent over stress at any point. For details please refer no.(20).

Discussions.

The solution presented in this method is applicable to 45 degree slopes only with added stringent conditions, that the foundation soil and the superimposed fill possess identical elastic properties and that the problem solved has identical dimensional similarity to the solution given. These limitations point immediately to the full scope of work encompassed in providing a set of charts. In addition to varying slope angles, variation in basic dimensions for each slope angle is also required. This does not resolve the additional problem of indicating variation in the solution due to values of Poisson's ratio other than 0.5.

The values obtained in fig.34 showing the cohesion and internal friction required to prevent overstress at any point are much greater than the values required to prevent slide according to the results obtained from the use of circular arc analysis. In fig. 35, the values obtained by sliding circle method for factor of safety of 1 and 2 are superimposed as fig. 35., to indicate the difference between the two methods (Discussions- (2)). In view of this, one may question the value of elastic solution. But the overstressed points in the embankment cross section become plastic and still resist additional stress to adjoining material, which was neglected in this method. If the plastic zone is generally confined, this transfer may be effected without danger of progressive failure. The strong point of the elastic solution lies in its ability to indicate the location of the plastic zone.

A limitation results from the assumption of identical elastic properties of the fill and the foundation material, which condition rarely occurs in practice.

4.74. ANALYSIS USING THEORY OF PLASTICITY (21).

An approximate mathematical theory has been developed for plastic failure of a soil slope due to its own mass, which would permit the determination of the possible position and form of the sliding curve and of approximate stress conditions leading to failure. According to the theory of plasticity the stress in the plastic region is independent of the plastic deformation.

The body forces of external boundary loading conditions and equilibrium conditions are shown in Fig. 36 a and b.

The angle β that the maximum shearing stress makes with the radial line through a given point (r_1, θ) is given by,

$$\tan(2\beta) = \frac{\sigma_r - \sigma_\theta}{\tau_{r\theta}} = \cot \psi \quad \ldots\ldots\ldots(i)$$

where ψ is an auxiliary function and may be determined from $2\theta + \alpha = \pm \frac{2c}{\sqrt{c^2-1}} \tan^{-1}\left[\frac{\sqrt{c+1}}{\sqrt{c-1}} \tan\frac{\psi}{2}\right] \mp \psi$.

The value of the parameter ψ for determining shear stress T in a wedge in plastic material is plotted in a graph connecting ($\theta_0 - \theta$) Vs. values of coefficient C.

When the directions of maximum shearing stress are found by eqn.(i), for a large number of points in — the plastic region, the slip lines can be drawn. EK - Khoo Tan analysed slopes and slip lines were drawn by this method; it was found that these slip lines were in the form of arc of circles and also resembling that obtained by model tests. But the stresses and strength relationship obtained by this method was different from those obtained from Swedish method and also by photo-elastic analysis.

Discussions.

The solution of the problem is approximate because it is necessary to assume the entire wedge to be in a plastic state, where as in nature only the region above the rupture surface is in a plastic state. According to the theory of plasticity the stress in the plastic region is independent of the plastic deformation .(21,52).

In the analysis, considerable concessions particularly in the way of approximations are made to avoid complexities for analytical solution. The stress and strengthrelationship obtained by this method show no resemblance with those obtained by this method show no resemblance with those obtained by the Swedish method and photo-elastic analysis.

In nature, no rigid boundaries exist which would influence some what the stress condition within the plastic zone. In the laboratory, this problem is complicated by the fact that body forces are acting, that the boundary conditions can not be satisfied readily, and that the solution of non-linear differontial oquations is required. Therefore only a certain class of solutions are investigated, in the method to develop one that would represent the physical phenomenon reasonably well. The theoretical investigation is concerned with determinining the plasticity equations, which would define approximately the stress conditions for the plastic state and would permit drawing the potential slip planes in the slope.

4.75 ANALYSIS BY PHOTO ELASTIC METHODS.

A rational method of designing or analysing an embankment slope would be to determine the maximum intensity of shearing stress within the embankment and to divide it by the shear strength of the soil to determine the actual factor of safety against overstress. An approach of this problem has been developed by applying the photo-elastic principles of stress analysis. Photo-elastic studies are made in gelatin models of slopes to obtain a clear picture of the distribution, and relative order of magnitudo, of the shearing stresses set up within a slope. When a transparent model of gelatin is stressed in its plane, and a linearly polarised light is passed through the strossed body by means of Nicol prism, interference phenomenon and coloured fringes called isochromatics are produced which can be observed on a screen by the use of a second Nicol prism. On the screen these isochromatic lines will appear in repetition, each colour corresponding to a certain principal stress difference : ($\sigma_1 - \sigma_2$)=constant = $2T_{max}$, in which σ is the unit principal stress and T is the shear stress (21).

This method assumes that the full strength of the material is developed at only one point in the section where as the usual Swedish methods assume that the full strength of the material is developed along the most dangerous circular arc.

5. SPECIAL CONDITIONS OF STABILITY OF SLOPES.

(i) Effects of Water (61,95)

Various effects of flowing or sooping water are generally recognised as very important in stability problems. Flowing water, due to heavy rains or a stream may cause erorsion in the form of undercutting at the toe of a slope, thus decreasing the stability. Increased water content also reduces the shearing strength of soil. When a slope retains water the following four cases may be taken into consideration for analysing the stability.

(a) The Submerged case (94,95,97).

The submerged weight is the only actuating force, in this case. The remainder of the true total weight (equal to V x Ww) is ~~fur~~ first supported by the by the boundary neutral or buoyancy forces. So the slope must be more stable in this case. This case is common for highway embankments.

(b) Instantaneous, complete drawdown case(23,28,51,94,95, 100)

In this case,one of the forces due to the weight of water on the slope is removed suddenly, which causes more severe loading to the actuating force. Thus the actuating force is due to the total weight of soil and water within the mass. Since the suddenly added force ($V.W_w$) can introduce no intergranular pressure and can develop no friction, the neutral force acts across the arc,passing through the centre of the arc. This is the worst condition possible, causing least stability.

But in practice a sudden, complete drawdown is not practicable in the case of highway embankment slopes. But there can be comparatively slow drawdown and not a complete one. But even this, for soil with low permeability affects the stability considerably and hence this case will have to be considered depending upon the type of soil and the drawdown conditions.

(c) Steady seepage case (17,18,35,84,107)

After a drawdown, whether it is sudden or relatively slow, seepage occurs. After sufficient time is allowed, consolidation takes place under the new system of forces and a steady seepage condition may be reached; but partial consolidation is an intermediate stage between case (b) and (c). The resultant seepage force will have to be determined from a flow net of field measurements. However, the case of steady seepage is in general slightly more stable than draw down case.

Heavy and prolonged rain fall, maintaining free water surface at ground level gives seepage throughout the slope and is the most unfavourable of steady seepage cases. Since such conditions are possible to occur in many places, this case should be carefully considered. In highway cuts, the position of ground water level should be invariably be determined before analysing the slope.

(d) Case with zero resultant boundary neutral force (95, 99, 109)

Cases in which the resultant neutral forces across the incident failure surfaces are equal to zero or approximately equal to zero, are encountered only occasionally in embankments and cuts of clay. If the pore water pressure is so small in bore holes, this case may be assumed. However there are instances in which this case (d) can reasonably be assumed, and frequently occuring examples of cases falling between cases (b) and (d) may be solved approximately by interpolations between results based on these two special cases.

One of the other effects of water is by filling in tension cracks, and increasing the instability; this is more when the water freezes in side the crack in cold regions.

(ii) Earthquake effects: (7,22,24,70,95)

Earthquakes may cause failure of earth slopes which under ordinary conditions would be stable. Relatively little is known of the forces introduced by the earth quakes. The greatest danger at the time of an earthquake is expected to happen if the embankment resonates with the vibration of the earthquake. When the height of the earth structure is in the order of 200 to 300 ft or more, the danger of resonance is great. But in the case of highway embankments the fear of resonance is little because maximum of heights of these embankments hardly exceed 50 to 60 ft. The soil properties, C and Ø, are likely to have different values under earthquake conditions than they have under normal conditions. A relatively small earthquake acting for a very short period may not cause appreciable change in pore pressure.

An empirical procedure that has been used in regions that are subject to earthquakes consists in assuming that quake imposes a horizontal acceleration of sinusoidal variation with an amplitude that is equal to some given percentage of gravity. Usually it is assumed that a horizontal force of 0.1W acts and the resultant of this force, together with the weight and the pore water pressure are to be found, for stability analysis. Also an assumption is made that cohesion will increase 1.5 to 2.0 times the original value for cohesive soils, which if neglected will give conservative value of factor of safety.

According to information furnished by L. Don Leet, Professor of Geology at Harvard University land slides do not occur at distances greater than 50' miles from the epicentre of the largest earthquakes known to have occurred. In India only in Assam earthquake hazards are frequent.

5.3 Effect of tension cracks (12,94,95,97,105)

Tension cracks are possible in cohesive soils, but the depth of the zone of cracking can seldom be accurately known. The presence of tension cracks reduces the effective length of rupture are along which shearing resistance act, and hence suitable allowance should be made. For infinite cohesive slopes, the maximum depth of tension crack is calculated to be $\frac{2c}{w}\tan(45+\frac{\phi}{2})$. But this expression is not always reliable for ordinary slopes. Moreover cracking is of progressive nature, aided by water and ice pressure and so it is not exactly limited to a particular depth.

When a value for the depth of tension zone has been estimated, a satisfactory allowance for its effect may be made in the stability analysis.

6. Discussion of Different Definitions of Factor of Safety (27, 51, 95)

Indirect method (27)

Upto some years ago, the way, generally used, to find the factor of safety against sliding was an indirect one. For any given value of C are tried to find the required value of ϕ req., which was necessary to establish equilibrium for an arbitrarily chosen slip circle. By trial and error the radius of this circle and the location of its centre was varied until $\phi_{req.}$ was minimum. The result of this computation was equilibrium of the sliding mass for C, $\phi_{req.}$ for the critical circle; the F.S. according to this procedure was unity. There are two rules for the indirect way of computing factor of safety.

Fellinius' Rule. (expressed Schematically)

Characteristics of the shear strength of a slope.	Factor of Safety (F.S.)
c_1, ϕ_1	1
c, ϕ	η

Condition:-

$$c = \eta c_1$$

$$\tan \phi = \eta \tan \phi_1$$

Ohde's Rule.

$$F.S. = \frac{c}{c_{req}\,,\ \phi = 0} + \frac{\tan \phi}{\tan \phi_{req}\,,\ c = 0.}$$

Review of definitions in current use. (51)

Factor of Safety with respect to Cohesion, F_c.

Let C_d be the maximum unit cohesion required for equilibrium for a known value of ϕ, and c be the available unit cohesion. Then $F_c = \frac{c}{cd}$

Factor of Safety with respect to friction, $F\phi$.

If ϕ_d is the maximum value of angle of internal friction required for equilibrium for a known value of c, and ϕ the available value, then $F\phi = \frac{\phi}{\phi_d}$

Factof of Safety with respect to height, F_H.

If Hc is the maximum height or critical height at which it is possible for a slope to be stable, for known values of c and Ø, and H is the actual height of the slope, then,

$$F_H = \frac{H_c}{H}$$

It is seen that for simple slopes, $F_c = F_H$

Factor of Safety with respect to Shear Strength Fs.

If S_d is the average shear stress required for equilibrium along a sliding surface and Sa is the average available shear strength along the same surface, then

$$F_s = \frac{S_a}{S_d}$$

It is seen that for $\phi = 0$, $F_s = F_c$

and for $C = 0$, $F_s = F_\phi$

Factor of safety Fs may be found from the shear resistance envelope (Ref.Art:4.16 and Fig.30) which is given by the least value of the ration $\frac{y_1}{y_2}$

$$= \frac{\text{Shear strength available}}{\text{Shear strength reqd. for equilibrium}}$$, which was ~~[struck out]~~ first suggested by A Casagrande.

For those procedures which are based on cylindrical sliding surfaces (circular arc analysis) the following additional definitions have been proposed.

$$F.S. = \frac{\text{Sum of all stabilising moments.}}{\text{Sum of all driving moments.}}$$

= Minimum, which was first suggested by D.M. Burmister.

$$F_s = \frac{\text{Total available resisting Moment}}{\text{Total over-turning moment}}$$

$$= \text{Minimum.}$$

These different definitions, are likely to deviate in numerical value, except for slopes in equilibrium when $F_c = F\phi = F_H = F_s = 1$

For the purpose of examining the numerical differences, a comparative study is made below.

Let the shear strength characteristics of a soil sample be given by $\phi = 15^o$ and c = 600 ps.f. and an embankment is to be designed. Let the working values be taken as $\phi_d = 12^o$ and $c_d = 400$ psf.

For the above example, $F_c = \frac{c}{cd}$

$$= \frac{600}{400} = 1.5$$

$$F\phi = \frac{\tan\phi}{\tan\phi} = \frac{\tan 15^o}{\tan 12^o}$$

$$= 1.26$$

Now let it be assumed that the average value of direct intergranular pressure on the critical surface be 2300 psf. Assuming Coulomb's Law for shear strength, average shearing strength on the surface, $= 600 + 2300 \times \tan 15^o$

$$= 1216 \text{ psf.}$$

Average value of shear stress on the surface is

$$400 + 2300 \times \tan 12^o$$

$$= 889 \text{ psf.}$$

Factor of safety w.r.t. shearing strength

$$Fs = \frac{1216}{889}$$

$$= 1.37$$

Out of the unlimited number of possible combinations of Fc and FØ values that could apply in this case, few are given below-

Fc	1.00	1.26	1.37	1.50	2.20
F Ø	2.13	1.50	1.37	1.26	1.00

When FØ = 1, F_c = 2.2 and this is called the F.S. w.r.t. cohesion when the full friction is mobilised. For the case FØ is arbitrarily taken as unity, Fc becomes equal to F_H .

But when Fc = FØ , this will also be equal to Fs. If we draw a graph connecting Fc and FØ drawn for various values of Fc, the corresponding values of FØ , (Refer Fig. 15-C) and then draw a 45° line from the origin to meet the curve, this point of intersection will give the value

Fc = FØ = Fs.

In view of the observations made above, it is evident that in order to avoid misleading interpretations, it is necessary to rationalise the use of the different definitions. The most plausible definition of F.S. seems to be the factor of safety w.r.t. strength

$$Fs = F = \frac{\text{Available shearing strength}}{\text{Actual shearing stress.}}$$

at the sliding surface.

Prof. K.Terzaghi has suggested using three categories of definitions , as follows.

(i) The F.S. w.r.t. shear strength, should be used in connection with the design of embankments, earth dams, and dikes and construction of slopes where the type of soil and the slope angle, can be altered, and for analysing the stability of natural slopes.

(ii) The factor of safety w.r.t. height may be used for analysing stability for excavations in very soft clay.

(iii) Burmisters definition of the ratio of stabilising moments to driving moments (minimum) may be used when investigating the effect of couter weights applied for stabilising purposes.

7. SOIL TESTING.

Sampling. (15,19,69,95,97) No attempt at accurate analysis is justified except on the basis of reliable data from tests made on undisturbed samples taken from sufficient depth to be representative of conditions along potential slip surfaces.

Before starting office work, preliminary field, laboratory investigations are necessary. This should include measurement of the depth and thickness of each stratum intersected by the cut, and determination of friction angle, unit cohesion, unit weight of soil, and ground water data including the depth and slope of the water table, the depth of flow, and the direction angle of ground water flow.

Shearing Strength (10,13,14,19,73,82,95,97)

The shearing strength of a soil is usually assumed to follow Coulomb's empirical law,

$s = C + N \tan \phi$. It must be noted that the values of c and ϕ are not necessarily constants for a given soil.

The linear relation expressed by Coulomb's law must be looked upon as an approximation. However, the approximated straight line generally do not introduce any serious error.

The values of cohesion and friction angle may be found from the following tests.

Triaxial Compression Test. (38,19).

The test may be started with both lateral and axial pressures equal to the weight of the natural over-burden at the point from which the sample was secured. Then, the lateral pressure is reduced until failure occurs. This test procedure increases the reliability of test data by similating conditions in the proto-type, where also failure results from the removal of lateral support, and not from ~~the~~ an increase of the vertical load.

Friction and cohesion can be found out graphically using Mohr's diagram, from the triaxial test data.

Direct Shear Tests. (38 , 19)

Direct shear tests are satisfactory if representative undisturbed samples can be obtained. Because of the relatively small dimensions of the usual shear base, and because the plane of failure is fixed by the apparatus, the

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presence of pebbles or stones in the sample are more troublesome than in compression tests. As a rule soil containing particles larger than ¼ in. should not be tested in direct shear box of conventional size. Rapid shear tests introduce a viscous resistance to the deformation which may not be equally present under field conditions, and on the other hand, may under value the shear strength of the specimen if it has been allowed to absorb water in sampling or in storage.

Except where the natural soil in the field has not full consolidated under the weight of its own over burden, slow shear tests are preferable supplimentary data taken during test such as stress- strain relationships, and water content or volumetric changes during test are all essential to an understanding of the shearing properties of the soil. The neglect may introduce some hazard; therefore, in cases of exceedingly important cuts the advice of a soil specialist is to be sought.

The plane of failure in a direct shear test is predetermined by the apparatus. This is acceptable with homogeneous fill; in hetrogeneous natural soil it may yield erroneous high strengths as compared to compression tests, where failure can occur along some natural plane of weakness.

The direct shear test is satisfactory with fill, where the grain size of the specimen is subject to control, it is less satisfactory than the compression tests for undisturbed samples of till, gravel and other soils containing large particles of unknown grading.

Unconfined Compression Test (104, 38, 19).

This may be used in case of normally consolidated clays when undisturbed specimens are available, for the "$\phi = 0$ analysis" of Swedish method. The shearing strength is half the compression strength since the Mohr's circle passes through the origin.

Vane shear test (34,82) is also used in some cases to determine the shear strength of soil, especially when representative undisturbed samples can not be got easily from deep borings.Iskymeter (59) is also some times used.

Specific gravity, water content, and natural density of undisturbed samples can be measured using ordinary laboratory methods.

Coefficients of permeability of soil strata to be encountered in the cut may be found by laboratory tests upon undisturbed samples, especially where the soil is fairly homogeneous. But an approximate value may be obtained directly from the field.

Ground Water (38, 61,99, 109, 19)

The location of the water table is a more potent factor in the stability of slopes than any ordinary variation in the physical properties of the soil itself, simply because it is so indefinite. The approximate position of the ground water surface as it is expected to exist after completion of the excavation is to be determined.

For preliminary field investigation a minimum of three test-holes is required, and it is convenient to locate one of these at the toe of the proposed cut. All three holes must reach a source of water supply, however small, and the one nearest the toe of the slope s-should extent to the full depth of the proposed cut.

8. INVESTIGATIONS BY MODEL STUDY. (21,30,71, 80)

If investigations can be made by model studys, the mechanics of failure of slope can be studied and there can be a better understanding of the conditions at failure of a slope. But there are so many difficulties in investigating stability of soil slopes by models. According to the usual concept of model analysis there should first be a geometric similarity between model and prototype, and the forces controlling the phenomenon should have a constant ratio. These requirements can not be satisfied readily, because if the particle size of the soil is reduced in accordance with the scale ratio, the properties of the soil are altered materially. Gravitational and molecular forces can not be altered. Capillary forces are difficult to control in the laboratory in the small scale tests. This means that every model test is really full scale. However, in a small scale model, if a slide phenomenon can be produced similar to the slides observed in nature, then it can be stated that the performance of the model is similar to nature.

A good deal of work on investigation of soil slopes by models was done by EK-Khoo Tan and some of his investigations are given below.(21).

Model tests of soil slopes were made by him to study the mechanics of failure in case where the slide phenomenon is controlled by gravitational influences only. The apparatus used for model investigation consist of a galvanised iron box with tight joints and open end, as shown in Fig.37. The box, placed on a firm support is provided with a device for tilting it to any desired angle, the angle of slope of soil being measured by a clinometer. A course to fine sand was used throughout the investigation.

(i) The Angle of Repose Phenomenon for Dry Cohesionless Material.

A series of tests were done with sand deposited in loose as well as dense states. In each case the slope was formed to its natural angle of repose. After placing the sand, the apparatus was tilted slowly until the first visible movement of the sand was observed. The angle of repose of the dry sand used in these tests was found to vary between two limiting values- a max: of 36°, just before a slide and a minimum of 33°, just after the slide.

Failure was observed to begin at the top of the slope and was confined entirely to the surface layers of the slope From tests it was noticed that the angle of repose practically remained the same for both dense as well as loose sands.

(ii) Phenomenon of slides in slopes of Cohesive soils.

In order to produce a slide in the small scale model, it is necessary to develop special technique which, because cohesion is the important factor, involve in creating an equivalent cohesion in a cohesionless material that could be controlled so as to satisfy model requirements and produce a slide failure in the laboratory. However, the influence of the mass of soil involved in the failure can not be learned from a small scale model of this type; but it does afford a good insight into the mechanics of failure.

Equivalent cohesion in the model soil slope was obtained by covering the cohesionless sand slope by a rubber membrane. This was stretched loosely over the slope but was tightly clamped to the sides of the apparatus, as shown in fig. 37., so that the air in the soil mass could be evacuated. The soil was thus subjected to a uniform external pressure which could be maintained constant and be easily controlled or varied at will. This uniform external pressure produced an essentially uniform intergranular stress throughout the soil mass. The most important advantage was that the cohesion could be made sufficiently small so that a failure by slide could actually take place.

The model slopes were formed with sand placed in a very loose state and also with medium dense state. The sand was then transferred to a soil possessing equivalent cohesion. The applied uniform system of external forces was determined by measuring the vacuum within the soil mass by means of water manometers, placed at intervals along the slope. The fact that they gave equal reading shows that the pressure was practically uniform over the slope. The apparatus was then tilted until failure occured. The angle of slope was measured and the form of failure observed, after removing the rubber membrane. A new slope was built for each test.

Graphs were drawn to note the relative between the equivalent cohesion and the resulting angle of slope at failure for loose and dense states. It was noted that,(a) the angle of slope at failure increases linerly with the equivalent cohesion, except for very small value where the restraining influence of the rubber membrane itself is appreciable compared with the air pressure. (b) The density of the sand has a very marked effect on the angle of slope at failure, because of the important increase in the angle of friction with density.

(iii) <u>Model Investigation of the deformations within the soil slopes, and the location and the form of the rupture surface.</u>

A procedure was devised to obtain a record of the deformation within soil slopes, and the location and forms of rupture surface. A piece of glass which was cut to the shape of the sand slopes, and ruled with vertical lines of lamp black and oil, about 1/16" wide was placed within the soil mass (Fig.38). After the glass has been placed against the side of the apparatus, the sand was carefully deposited against it and the slope was formed as before.

When any deformation or sliding occured within the soil mass, the movement of sand was registered on the glass, by the displacement of vertical lamp black lines. These movements ~~wa~~ were observed after the sand had been carefully removed and the glass plate was withdrawn. ~~From these tests influences were drawn~~ . From these tests inferences were drawn that (a) as cohesion increases, a larger mass of soil is involved in the failure and the principal failure curve receedes deeper from the surface of the slopes.

slopes. (b) In all cases, the zone of disturbance extends a considerable depth below the final sliding curve (e) An appreciable volume change was observed in the soil slope after failure. A subsidence occured in the case of loose sand and an expansion in the case of dense sand.

(iv) Investigation of failure of model vertical bankk.

In these tests, the initial equivalent cohesion was made much greater than the required for the stability of the bank of the given height. The cohesion was then decreased slowly until the bank failed.

Graphs were drawn to show the relation between the critical height of a vertical bank and the magnitude of equivalent cohesion. The results of the investigation show that the critical height at which the laterally unsupported embankment will stand vertically is a linear function of the cohesion.

(v) Photo-elastic Studies of Gelatin Models.

Photo-elastic studies were made on gelatin models of slopes to obtain a clear picture of the distribution and the relative order of magnitude of the shearing stresses set up within a slope. When a transparent model of gelatin is stressed in its plane, and if linearly polarised light is sent through the stressed body by means of nicol prism, interference phenomena and coloured fringes called isochromatics are produced which can be observed on a screen by means of a second Nicol prism. On the screen these isochromatic lines will appear in repetition, each colour corresponding to a certain principal stress difference.

$(\sigma_1 - \sigma_2)$ = constant = 2 T max., in which σ is the unit principal stress, and T the shear stress. The isochromatics gave a clear picture of the distribution and the order of magnitude of the maximum shearing stress throughout the slope in the gelatin model.

The values of maximum shearing stresses, as determined on the gelatin model, were transferred to the corresponding values in the sand model by multiplying the calibration constant for the maximum shearing stress values obtained on the gelatin model by a scale ratio.:

$\frac{W_s}{W_g} \times \frac{L_p}{L_m} = W_r \cdot L_r$, in which, W_r is the scale ratio of the unit weight of soil (Ws) to the unit weight of gelatin (W_g) and L_r is the linear scale ratio of the sand prototype (L_p) to the gelatin model (L_m).

2.

9. DESIGN PROCEDURE OF HIGHWAY CUTS (38)

In cohesionless sand the stability of the cut bank is independent of the depth of cut and of the unit weight of the earth, where as it is not so in cohesive soils which are often met with.

Highway cuts through cohesive soil ordinarily will encounter the water table at moderate depths. First, field and laboratory investigations are necessary, including measurement of the depth and thickness of each stratum intersected by the cut, and determination of friction angle, unit cohesion, c, and unit weight w of the soil, and also the depth and slopes of the water table, the depth of flow, d, and the direction angle of ground-water flow . Coefficient of permeability of the soil strata to be encountered in cut may be found by laboratory tests or from field.

Trial Design-

When a slope cuts through several different soil strata, or when it intercepts ground water flow, the various factors upon which stability depends can not be related in a simple equation. A trial slope angle must be selected either from experience, or by simplifying the actual conditions, and analysed, preferably by graphic methods. The Taylor's Stability curves (Ref.Fig.29) can be used advantageously for this purpose, if average values of ϕ, c, and w are selected. In less general use are the various empirical formulas.

Upon a cross section of the cut, drawn to some convenient scale, the water table should be located as it will appear after construction. This water table, called the "phreatic line", represents the locus of zero hydrostatic pressure. Strictly speaking, it does not mark the extent of saturation, as voids in the soil above the phreatic line may be filled with capillary water.

The location of the phreatic line, depends upon the extent and permeability of the aquifer, upon the location and nature of the water supply, and upon the drainage to be installed. In natural soil formation only fragmentary knowledge of many of these items may be possible, and only a crude approximation of the phreatic line can be expected. As a guide to judgement the following two equations establish approximate limits within which most practical cases will fall.

Case (1), homogeneous soil on a horizontal impermeable base; water supply obtained entirely from percolation within the drainage area-

$$y^2 - h_o^2 = \frac{Px(2R - x)}{31(10)^6 K},$$

where y = elevation of phreatic line above base in feet.

x = horizontal distance normal to direction of drains, ft.

ho = elevation of water surface in drain above base, ft.

p = Max: rate of percolation in inches/months.

R = Width of watershed, assumed to equal $5280\sqrt{A}$, ft.

A = Area of watershed above cut in sq.miles.

K = Coefficient of permeability, ft/sec.

Case(ii), Artisan flow from remote source

$y = ho + S_1x$

where $S_1 = S \cos\delta$.

and δ = Angle between direction of flow and a normal to the drain.

S = True slope of water table and will be unchanged by the excavation if the source is sufficiently remote.

In most practical cases, the above formulas are found to be of only limited value, and the position of phreatic line must be estimated by individual judgement of the available information.

For homogenous material the location of trial slip surface may be done by the circular arc method of analysis, and the factor of safety calculated by the modified method of slices as given under Art. 4.12.

When strata of different materials are encountered above the selected element of slip surface, it becomes desirable x to replace the actual depth of cover by an equivalent column of uniform density. As this procedure must be repeated a number of times, the graphical method given below may be found convenient.

Let w_1 represent the greater, and w_2 the lesser soil density, and let h be the ground water pressure head.

Ref. Fig. 39. Draw a b" parallel to AB" and ob" parallel to OB". Project ob" on oa; then ob is the equivalent column of density w_1, and pb, its normal component.

The effective earth pressure on the elementary area is computed by substracting the hydro-static pressure of tho ground water from the normal stress previously obtained. Considering the uncertainty of the true location of the phreatic line it is sufficiently accurate to take the height of the phreatic line above the ship surface as equal to the pressure head at the ship surface. This value should be reduced to an equivalent earth column by the above method. Draw qw" parallel to AW" , and pw" parallel to OW". Project pw" on pq. Then pw is the ground water pressure head, expressed per unit horizontal area (in the modified method of slices which is recommended, Ref. Art. 4.12 and fig.8), and since the water pressure acts per unit area of the slip-surface, the latter pressure must be divided by cos β before subtraction, β being the angle made by the normal to the xx slip-surface with the vertical. This is done graphically by subtracting its projection on the radial but from the original normal earth pressure, as in fig.39, where pw_1 is the equivalent head corresponding to the horizontal distance , dx 62.5 hdl = 62.5 h sec β dx). The remainder is the effective normal earth pressure, n, and this length is transferred to a normal pressure diagram as in fig.8, where it is plotted against horizontal distance from the toe of the slope. The hydrostatic pressure has no tangential component, and so the tangential component of the earth pressure is transferred without reduction in value to a similar diagram. The total normal pressure is multiplied by the tangent of the angle of friction to obtain the total frictional resistance. If the slip surface cuts across strata having different Ø' s, the projection of its intersection with the strata boundaries will divide the N-x diagram into various sub areas, each of which must be multiplied by its own coefficient of friction. With a high water table, negative values of N are possible, signifying tension across slip surface, and a open crack may occur. The length of arc, L, should include only that portion of the slip-surface for which N has a positive value, when used in the term c x L to compute the total cohesive resistance.

The factor of safety against sliding along the assumed trial slip surface is taken as the ratio of the total resisting forces to the total driving forces i.e., $F = \frac{N \tan \emptyset + cL}{T}$

The process is repeated to find the most critical circle corresponding to the least factor of safety, for this particular slope.

It is suggested that for highway purposes the slope should be designed to provide a factor of safety between 1.6 and 2.0.

Limitations of the above method.

The limitations given under discussion of circular arc analysis Art. 4.10 apply in this case also.

This method is concerned solely with the resistance of the e of the earth mass as a whole. Progressive failure may occur if an underlying plastic lay is squeezed out, before the stability of the integral slope itself is threatened. This must be prevented either by limiting the pressure at the toe to the value of compressive strength of local stratum or by providing local support at the toe by bulkhead, piling or rock fill. If the thickness of the underlying clay layer, 2a, is relatively small, plastic flow may occur if its shearing strength $c < (w.a.\tan i)$, where w is the unit weight of the over lying soil.

A perched water table will introduce an additional seepage forces, and evidence of seepage in cut face after construction should lead to further analysis to insure that the factor of safety is still ample, or to determine the need for additional drainage.

10. DESIGN OF HIGHWAY EMBANKMENT SLOPE (38,95,12).

The selection of suitable soil for embankment is to be made first. For any thing other than highly organic soils (which should be avoided) and clean sands, control of water content at which the soil is placed is highly important. The optimum moisture content can be found from laboratory tests at which the embankment is to be compacted. But generally the slope is designed for the condition of saturation regardless of the water content at placement. Consequently the shearing resistance of soils for embankment should be obtained from samples that have been compacted at optimum moisture to the density expected in construction, and then allowed to swell or consolidate under a specified normal load (in contact with water) before being sheared. The direct shear test will generally be more convenient than compression tests for this purpose, wherever the maximum particle size permits its use.

If the fill material is reasonably homogeneous, and if seepage forces are eliminated by proper drainage of the base of the fill, the slope angle may be directly be obtained from Taylor's stability charts. The factor of safety F, is introduced into the analysis by using the values of C and Ø equal to their test values, divided by F. For w, it is best to use the weight of the saturated soil at maximum density. Rather than to investigate the opportunity for tension at crest, it is more convenient to use only 80% of the otherwise allowable value of c, afterwards applying the safety factor to both c and i.

If the fill is not drained at the base, if the soil is not free draining, and if a state of capillary saturation is expected eventually, a prolonged period of rain fall may cause the stress in the pore water to change from tension to pressure. A flooded ground surface is not in itself sufficient to produce this development; in addition, there must be time enough and water enough to permit the elastic expansion of the soil mass, for the change in stress can not be separated from the change in strain. Usually, however, available data will be sufficient for a theoretical treatment of the situation, and where base drainage is lacking, it becomes necessary to design for the possibility of saturated soil with a temporary water table coincident with the surface of the fill.

When the embankment may have to retain water on one side or if it may be partially or completely submerged, the cases of saturated and draw down also should be considered in the design. The pore water pressure may be estimated from the flow nets for the critical condition. When there is an impervious layer, the pore pressure may be assumed to be $1\frac{1}{4}$ the depth of impervious material over the slip circle (100). But in the field accurate measurements of pore water pressures can be made for analysis after construction.

Taylor's stability charts permits analysis for saturated soil also by using a modified value of $\phi = \phi \frac{(Wt - Ww)}{Wt}$ where Wt is the unit weight of saturated soil at maximum dry density and Ww is the unit weight of water. A few trial computations would emphasize the economy of providing base drainage. The nature of the safety factor is Taylor's method is some what less conservative than in the graphical analysis of cuts as in Art.9.

It is suggested that the factor of safety be selected in the range of 1.5 to 1.7 for highway embankment slopes. The presence of fewer unknown elements in the embankment problem, more than off sets the more conservative definition used in the problem of cut slopes.

<u>Stability of Embankment foundations (38)</u>

The stability of the embankment and foundation should be jointly considered when the circular are method of analysis is employed. It might be considered separately however, if the theories of elasticity and plasticity are used.

If the embankment is placed upon a relatively weak, non-plastic base, the bearing capacity of the foundation may be computed by the modified Prandtl formula,

$$q = (c \cdot \cot\phi + w_\gamma \cdot B \tan\alpha)(e^{\pi \tan\phi} \cdot \tan^2\alpha - 1)$$

where w_γ = Unit weight of the supporting soil.

$2B$ = Crest weight plus horizontal projection of one slope.

$\alpha = (45 + \frac{\emptyset}{2})$, $\emptyset$ being the angle of friction of supporting soil.

c = Unit cohesion of the supporting soil

e = Base of natural lograthms.

If the embankment is placed on a thin, plastic base (thickness of plastic stratum, 2a, less than one fourth the width of the fill at its base, 2B) the bearing capacity of the foundation is approximately, $q = \frac{cB}{a}$.

For thicker plastic base, the bearing capacity becomes $q = c$.

In each of these cases, the factof of safety maybe found by dividing the computed bearing capacity, q, by the by the average vertical pressure at the base of the fill. Factor of safety of 1.5 is suggested minimum value.

All these cases of computing the stability of the embankment foundation are required only when the subsoil is relatively weaker than the fill; otherwise no separate analysis of foundation stability is necessary.

11. Recent Developments Saving Time in Trial and Error Procedure of Stability Analysis of Slopes. (69)

In many practical cases of stability analysis the trial and error process could not be avoided. For analysing a slope by the modified Swedish method using a planimeter, an experienced designer can check one circle in about two hours and nearly two to three man hours of continous work should be allowed as a minimum for obtaining the critical safety factor, for one set of conditions.

By the use of electronic computers, one can find solutions to almost all variations of the problem of slope stability without need for the designer to resort to laborious and lengthy computations. Programmes for stability analysis of embankment by the slip circle method have been developed by different organisations for different types of electronic computers, certain types of which is applicable to various conditions like, homogeneous embankments, zoned embankments, uniform slopes, variable or benched slopes, embankments on rock foundations and soil foundations, sudden drawdown condition at upstream slope, steady seepage condition at downstream slope and, combined circular and horizontal failure plane along rock foundation.

In G-15 type of computor, the factors of safety can be typed out for a graphic representation at the location of the centre of the slide cirlce, if desired. It can k also check one circle at a time typing out such significant data as the sliding and resisting forces, radius of circle, coordinates of the centre and width of unit strip assumed in the analysis. With this particular the computer and programme, the actual computation time per slide circle is less than one minute for homogeneous section and about two minutes for zoned section.

In zoned embankments, often the designer is not so much interested in studying the change in factor of safety due to change in thickness and slopes of various zones- but only interested in finding minimum factor of safety for a particular type because the process is too long and time consuming. The capability of the electronic computer in investigating a very large variety of embankment sections in a very short period is impressive induced. With the computer doing all the monotonous computations for him, the designer has much more time at his disposal to evaluate the results and study their influence on stability and economy of the structure.

12. CAUSES OF SLIDES AND INSTABILITY.

Failure of a slope may occur due to either an increase in loads on the slope (high shear stress) or a decrease in shear resistance of the material.

Factors that contribute to high shear stress (42,43,46,83).

A) Removal of lateral support.

(i) Action of erosion by streams and rivers, glacier ice, waves and long shore or tidal, currents, sub-aerial weathering, wetting and drying, and frost action.

(ii) Creation of steeper new slope by preveous rock fall, slide, subsidence or large scale faulting.

(iii) Human agencies like, cuts, quarries, pits and canals, removal of retaining bodies, drains of lakes and draw down of reservoirs.

B) Surcharge.

(i) Natural agencies like weight of rain hail, snow, accumulation of talus over riding landslide material.

(ii) Human agencies like, construction of fill, stock piles of ore or rock, waste piles, from strip mining. Weight of structures and trains, weight of water from leaking pipe lines, canals etc.

C) Transitary earth Stresses- Earthquakes increases shear stress producing horizontal acceleration and sometimes decreases shear strength also; vibrations from blasting, machinery and traffic also produce transistory earth stresses.

D) Regional Tilting-

Progressive increase in slope angle through regional tilting has been suspected as a contributing cause to some land slides. The slope must obviously be on the point of failure for such a small and slow acting change to be effective.

E) Removal of Underlying Support-

i) Under cutting of banks by rivers and waves, (ii) Sub-aerial weathering ,wetting and drying and frost action.

(iii) Subterranean erosion like removal of soluble material (e.g. carbonatils salt or gypsum), ~~xkx~~ collapse of caverns., washing out of granular material beneath firms material,

(iv) Human agencies such as mining.

I) Loss of Strength in Underlying Material

Lime stone over shale, compact till over clay, and failure by natural spreading.

F) Lateral pressure due to water in cracks and caverns, freezing of water in cracks, swelling due to hydration of clay.

Factors that contribute to Low Shear Strength (43,46,83)

The factors that contribute to low shear strength of rock or soil may be divided into two groups. The first group includes factors deriving from the initial state or inherent characteristics of the material. They are part of geologic setting that may be favourable to land sliding, and they change a little or not at all during the useful life of the structure. They may exist for a long period without failure occuring. The second group includes the changing or variable factors that tend to lower shear strength of material.

A) The initial state.

i) Composition- Inherently weak materials, or those which may become weak upon change in water content, or other changes. E.g. Sedimentary clays and shales, decomposed rocks, rocks composed of volcanic tuff which may weather to clayey material, materials composed dominantly of soft platy minerals such as mica, schist, talc, or serpentine, organic materials.

II) Texture- Loose arrangement of individual particles in sensitive clays; marl, , sands of low density and porous organic matter and roundness of grains.

iii) Gross Structure.

a) Discontinuities such as faults, bedding planes, foliation in schist, cleavage, joints, and brecciated zones, (b) Massive beds over weak or plastic materials (c) Strata inclined toward free face (d) Alteration of permeable beds, such as sand stone, and weak impermeable beds, such as shale or clay.

B) Changes due to weathering and other Physico-chemical reactions.

i) Physical disintegration of granular rocks such as granite or sand stone under action of frost, thermal expansion etc.- Decrease of cohesion.

ii) Hydration of clay minerals. Absorption of water by clay minerals and decrease of cohesion of all clayey soils at high water contents. Swelling and loss of cohesion of montmorillonitic clays.

iii) Base Exchange in clays.

iv) Drying of clays- results in cracks and loss of cohesion and allows water to seein.

v) Drying of shales- Creates cracks on bedding and shear planes.

vi) Removal of cement by solution in sand stone reduces internal friction and deterioration of cementing property.

C) Changes in intergranular forces due to pore water.

i) Buoyancy in saturated state decreases effective intergranular pressure and friction.

ii) Intergranular pressure due to capillary tension in moist soil is destroyed upon saturation.

iii) Seepage pressures of percolating ground water result from viscous drag between liquid and solid grains.

D) Changes in Structure.

i) Fissuring of pre-consolidated xxxx clays due to release of lateral restraint in cut.

ii) Effect of disturbance or remolding or sensitive materials such as loess and dry or saturated loose sand.

Effect of time on deterioration of shear strength(85,88,89)

Strength of a cohesive soil decreases considerably with time scale. From experience of several failures in clay slopes, it is seen that there is a progressive decrease in cohesion intercept with time, after construction and that in the limit this intercept tends to zero, when it becomes normally consolidated. In addition there is a softening effect which appears to continue for years after construction. These conclusions were arrived at, after thorough investigations of failures in clay slopes, especially in London clay. It is found that analysis based on effective stress allowing for ground water pressure gives better results, and that $\phi = 0$ analysis is not reliable in some cases. But analysis based on assumption that $c = 0$ gives very conservative results.

13. REMEDIAL MEASURES.

In general, no standard corrective measures to be adopted in dealing with land slides can be specified as each land slide requires special investigations and different treatment. The following are the possible methods so far advanced by which landslides are eliminated or controlled. After theoretical, physical, and laboratory analysis, one or more of the following measures suitable to each particular problem may be tried.

i) Relocation of the road by completely abandoning the slip area with comparative study of cost of such relocations and protective measures proposed to be resorted to (42,83).

ii) Removal of land slides completely by cutting and dredging up to the bed rock inthe slipping plane. This can be ascertained by taking a series of borings to the maximum depth and then plotting them into the log. (42,83).

iii) Bridging the slip area if it is small, and two stable ends are available (42,83)

iv) Construction of cribbing of either concrete or timber.

v) Cementation of loose materials in the entire area by any other method such as cement grouting, bitumen grouting etc. (42,46,83).

vi) Retaining walls, breast walls either of masonry or concrete or boulder sausages.(83).

vii) Piling with steel, concrete or timber. These piles will produce additional resisting forces. The depth of penetration should not be less than one third of its length below the slip surface. The resistance s to shear developed by the piles at the surface of rupture may be determined by applying the following formula advanced by Hennes. $S = \frac{Ap}{D} \cdot f_v$ where s = shearing resistance of the pile in psf., Ap, the cross sectional area of the pile in sq.inch, f_v the allowable shearing stress for piles in psi., and D, the c/c spacing of piles in ft. divided by number of lines of pile (42, 43, 46, 83).

viii) Net work of surface drainage system in the entire slip area (56,65,105,83,46)

ix) Slope treatment.

x) Sub-surface drainage of French type.

xi) Vertical and horizontal sand drains (103,105)

xii) Chemical treatment. Clayey, water logged and sandy soils may be hardened by spraying a strong solution of calcium or magnesium chloride (83).

xiii) Tunnelling.

xiv) Sealing the joints in open fissures, peripheral cracks, etc.

xv) Providing buttresses (46,83)

(xvi) Steel dowels, grouted into bed on top of which sliding xxx occured and anchored with concrete blocks or light walls near the upper edge of sliding areas.

(xvii) Perforated horizontal drains to release the hydrostatic pressure.

(xviii) Method of soil stablisation by injecting asphaltie emulsions, cement grouting, electro-asmosis or electro chemical treatment.

(xix) Stabilisation by re-seeding with green deep-rooted vegetation or ecological treatments.

(xx) Drainage by tunnelling.

(xxi) Underground walls by filling drill holes with cement grouting.

(xxii) Sluicing. The stream is controlled from top and directed where washing off the detritus material to get the hill side to natural slope is necessary.

(xxiii) Toe Loading.

(xxiv) Benching. (77)

Without going into much detail when facilities are not available for such investigations, the following methods may be tried as control measures to different of land slides(83).

i) Natural landslides in homogeneous soil. piling, proper drainage of surface water .

growth of proper vegetation and underdrainage.

(ii) Natural land slides in hetrogenous soil. surface drainage, under drainage by tiles, well drainage, tunnelling, piling and grouting.

(iii) Slide due to plastic flow:

Homogeneous clay: Piling and retaining walls.

Sandy soil:- Grouting and piling.

Creep: Retaining walls and surface drainage.

(iv) Over hanging strata and collapse of soil mass: Cohesive soil: retaining walls and piles. Pervious soil strata: reverse filling and grouting.

14. CONCLUSIONS.

For a Highway Engineer, the problem of stability of slopes is of two types. First, for Highways passing by the side of natural hill slopes, the stability problem is more complicated and uncertain. In this case geological factors are to be given more importance and a qualitative analysis of stability may be made on this basis. Then one of the simplified methods of stability analysis may be carried out by an experienced Engineer, considering the geological factors also. The other type of stability problem is for artificially made Highway Embankment slopes, and which the soil characteristics are better known and controlled during construction and hence more accurate calculations of stability analysis are possible.

There is no doubt that the stability analysis and engineering investigations are essential to build stable and economical slopes, by allowing the required amount of factor of safety according to site conditions. Before adopting remedial measures for slopes which have failed, or before taking steps for preventive measures, through investigations and analysis of the slope should be made so that the problem could be solved most effectively as well as economically. The choice of the method of analysis to be used for a particular problem will depend upon the site conditions and the importance of the project, and an experienced engineer should be able to judge. In cases where accurate measurements of strength characteristics of the soil is not possible, it is useless to go for more rigorous methods of stability analysis. But when the values of cohesion, angle of friction and the pore water pressure could be found out correctly, approximate and simplifying methods of analysis should not be adopted.

Various methods of analysis have been discussed so far and it is difficult to say that a particular method is the best. Strictly speaking, none of the method is accurate suiting the actual site conditions and several simplifying assumptions are made in one way or other. But some methods could reasonably accomodate the varying conditions at site with some approximations, and could give reasonably accurate results.

Even though there are several drawbacks in the modified Swedish method using planimeter (assuming circular failure arc) this method has been used widely even for designing or analysing earth dams in which the soil is not homogeneous. Under most circumstances the sliding circle method with its adaptability to many effects will suffice as a design criteria.

The accuracy of the circular arc analysis can be improved by determining the distribution of normal pressure along the slip curve by using the theory of elasticity and plasticity, instead of by assuming a vertical pressure at each point proportional to the height of the overlying column of earth. The methods for the determination of accurate and reliable soil characteristics, are yet to be developed.

Recommendations for the use of existing methods of stability analysis

Type of slope	Method recommended for design or analysis of slopes. For ordinary purposes.	For more accurate calculations when the correct values of pore pressure and soil characteristics are known.
1. Single slope homogeneous soil.	i) Method of slices (Art. 4.11) ii) Modified method of slices (Art.4.12)	i) Bishop's Method (Art.4.13) ii) Graphical application of Bishops' Method (Art.4.131)

Type of slope	Methods recommended for design or analysis of slopes.	
	For Ordinary purposes.	For more accurate calculations values of pore pressure and soil characteristics are known.
1.(Contd.)	(ii) Ø -circle Method(Art.4.14) (iv) Taylor's stability charts (Art.4.15)	(iii) Photo elastic methods (Art .4.75)
2. Irregular slopes, stratified soil.	Modified method of slices (Art.4.12)	Graphical application of Bishops' Method (Art.4.131)
3. Slope with composite section like that of an earth dam.	i) Wedge Method (Art.4.03) ii) Method of slices (Art 4.11) iii) Modified Method of slices (Art.4.12)	1) Graphical application of Bishops' method (Art.4.131) ii) Modified methods of slices (Art.4.12)

15. BIBLIOGRAPHY

(1) Baker, R.F. " Analysis of Corrective actions for Highway Land slides"- Proc. A.S.C.E.79,1953.

(2) Behera, B. " A Mathematical Formula for Design Earth Slopes" J.Central Board of Irrigation and Power, 12 (600-610) 1955.

(3) Bennett, P.T. " Notes on Embankment Design"- IV Cong. Large Dams, 1 (223-245) 1951.

(4) Bernt Jakobson " Isotropy of Clays"- Proc. Europ. Conf. Stability of Earth Slopes, L, 1954.

(5) Bishop, A.W. "The Use of the Slip Circle in the Stability Analysis of Slopes"- Proc. Europ. Conf. Stab. Slopes, 1, 1954.

(6) Bishop, A.W. " Some factors involved in the Design of Large Earth Dam in Thames Valley"-Proc. II Int. Conf. S.M.& F.E., 2. 1948.

(7) Binger, W.V. "Excavation Slopes"- Trans.A.S.C.E. 114 (734-754)

(8) Bjerrum, L." Stability of natural slopes in quick Clay- Geo.5 (101-119) 1955.

(9) Bjerrum, L. " Geotechnical Properties of Norwegian Marine Clays" -Geo 4 (49-69) 1959.

(10) Bjerrum, L and Kjaernsli, B. "Analysis of the

Stability of Some Norwegian National Clay Slopes"- Geo 7(1-16) 1957.

(11) Brahtz, J.H.A. "Rational Design of Earth Dams"- II Cong. Large Dams, 4 (543-572)

(12) Capper, P.L. and Cassie, W.F. " The Mechanics of Engineering Soils"

(13) Casagrande, A. " Characteristics of Cohesionless Soils affecting Stability of Slopes and Earth Fills"- J. Boston.Soc.C.E.Jan.1936(Contribution to Soil Mech. 1925-40)

(14) Casagrande, A " Notes on the design of earth dams"- J.Boston Soc. C.E. Oct. 1950 (Contribution to Soil Mech. 1941-1953)

(15) Cassel, F.L. " Slips Fissured Clay"- Proc. II Int. Conf.S.M. and F.E., 2.

(16) Cooling, L.R. and Golder H.Q. " The Analysis of the Failure of an earth dam during Construction"- J. Inst. C.E. 19(38-55).

(17) Daehn, W.W. & Hilf, J.W. " Implications of pore pressure in design and Construction of rolled earth dams" - Trans. IV Cong. Large Dams. 1 (259-182)

(18) Datta, R.R. " The stability of Soil at exit end of Hydraulic Structure due to Seepage "-J.Central Board of Irrigation and Power 13(373-375) 1956.

(19) Deptt. of Army Corps of Engineers- "Soil Mechanics Design- Stability of Slopes and Foundations"-Engg.

Manual, Civil Works Constn., Part C. XIX, Chap.2.

(20) Dingwall, J.C. and Scrivner, F.H. " Application of the Elastic Theory ~~x~~ to Highway Embankments ~~(21)~~ by the use of difference equation" Proc. H.R.B. 33, (474-482) 1954.

(21) EK-Khoo Tan " Stability of Soil Slopes"- Trans. A.S.C.E., 113 (139-180).

(22) Elbert E.Esmol " Seismic Stability of earth dams".

(23) Erling Reinius " Stability of Slopes of Earth Dams" Proc. Europ. Conf.Stab. Slopes, 1, 1954.

(24) Ervin Nonveiller, and Lujo Suklaja " Landslide Zalesina"-Proc. Europ. Conf-stab. Slopes.1, 1954.

(25) Fellenius B. "The Land Slide at Gun top"- Geo.5(120-125)

(26) Fellenius, W. " Calculation of the Stability of Earth Dams -II Cong. Large Dams, 4(445-859)

(27) Frohlich O.K. "General Theory of Stability of Slopes"- Proc. Europ. Conf. Stab. Slopes.1, 1954.

(28) Frohlich, O.K. "On the Danger of Sliding of the u/S Embankment of an earth dam, caused by complete or partial discharge of the reservoir"- IV Cong-Large Dams, 1 (229-242)

(29) Frohlich, O.K. " The factor of safety with respect to Sliding of a Mass of Soil along arc of a log. spiral"- Proc.III Int. Conf.S.M.& F.E.2(230-233)1953.

(30) Fukuoka, M. "Land Slides in Japan"-Proc.III Int. Conf.S.M. and F.E.2. (234-238) 1953.

(31) Galloway, J.H.H. "A modification to the slip circle method of stability analysis of earth dams"- Proc. II Aust. and New Zeal Conf. S.M. and F.E. (53-57) 1956.

(32) Gilboy, G. " Stability of Embankment Foundations -II Cong. Large Dams 4(577-592)

(33) Glover, R.E. and Cornwell, F-E. " Stability of Granular Materials"- Trans. A.S.C.E. 108 (46-82)

(34) Golder, H.Q. and Palmer, D.J. "Investigation of a bank failure at Scraps gate Isle of Sheppey, Kent"- Proc. Europ. Conf. Stab. Slopes, 1, 1954.

(35) Haefeb, R. "Stability of Slopes acted upon by parallel seepage- Proc. II Int. Conf. S.M. & C.E., 1-

(36) Henkel, D.J. "Investigation of two long term failure in London clay Slopes at Woodgreen and Northolt"- Proc. IV Int. Conf. S.M. & E.E., 2 (315-32).

(37) Henkel, J. and Skempton, A.W. " A land slide at Jackfield, Shropshire, in a heavily over consolidated clay- Geo. 5 (131-137) 1955.

(38) Hennes, R.G. "Analysis of Stresses in cuts and Embankments- Proc.H.R.B. 24 (349-357).

(39) Hennes, Hawkins and Mc Coy. " An appraisal of measure for improvement of slope stability"-

University. Engg. Dept. Stn. Reprint No.165, Orig. Bull. 216 (1959) H.R.B.

(40) Honsel,W.S. "Internal Stability of granular Materials- Proc. A.S.T.M., 36, -2 (427 - 468)

(41) H.M.S.O. " Soil Mechanics for Road Engineers"- Dept. S.I.R., R.R.Lab., London.

(42) H.R.B. " Land Slides and Engineering Practice"- Sp. Report.29,H.R.B.Wash.(1958)

(43) H.R.B. "Land slides- A selected Annotated Bibliograph".

(44) H.R.B. "Vertical Sand drains for stabilising Embankments"- Bull. No.115.

(45) Ireland, H.D. " Stability Analysis of the Congress Street Open Cut in Chicago"- Geo. 4(163-168)1954.

(46) I.R.C.Journal " Causes of Landslips and suggested remedies "- I.R.C. XVII (1952-53)

(47) Ivanov, A.I. "Stability Conditions of Earth bodies. & the analysis of Slopes and Foundation of Earth Dams"- II Cong.Large Dam 4 (615-634)

(48) Jakobson, B. "Isotropy of clays"- Geo.5(23-28)1955

(49) Jaky, J. "Stability of Earth Slopes"- Proc.I.Int. Conf.S.M. and F.E., 1.

(50) Jaky, J. "Validity of Coulomb's Law of Stability"- Proc-II Int.Conf.S.M. and F.E., 1.

(51) Janbu, J. "Stability analysis of slopes with dimension

less parameters"-Harv. Soil Mech.Series No.46,1954.

(52) Jumgenson, L. " The application of theories of elasticity and plasticity to foundation problems"- J.Bost Soc. C.E.1934 (Contribution to Soil Mech. 1925-1940.)

(53) Justin, Hinds and Creager " Engineering for Dams"-

(54) Kaposcy, J. "Three dimensional stress distribution and slip surfaces in earth works at rupture"- Proc IV. Int. Conf. S.M. and F.E. 1, (339-342)

(55) Karpoff, K.P. "Pavlooskey's Theory for Phreatic Line and Slope stability"- Proc.A.S.C.E. 80 (Sep.386)1954.

(56) Kellog, F.H. "Investigation of drainage rates affecting stability of earth dams"- Trans.A.S.C.E 113 (1261-1309)1948.

(57) Kiellman,W. "Do slip Surface Exist "-Proc. Europ Conf. Stab. Slopes, 1, 1954.

(58) Kiellman,W. "Mechanics of Large Swedish Land -slides Proc Europ-Conf.Stab. Kopee, 1, 1954.

(59) Kiellman,W. "Investigation of clay slopes in the Gota River Valley"- Proc. Europ.Conf.Stab. Slopes, 1, 1954.

(60) Kiellman, W."Effects of ground wats on stability of natural slopes in Swedish Clay soils"-Proc. Europ.Conf.Stab. Slopes., 1., 1954.

(61) Krynine, D.P. & Wood Ward, R.J. "Experiences with subsurface water"- Proc. H.R.B,32 (512-522)1953

(62) Krynine, D.P. " Discussion on Granular Materials"- - Trans.A.S.C.E., 108, 1943.

(63) Krynine, D.P. " Soil Mechanics".

~~(64) Krynine, D.P. " Soil Mechanics"~~

(64) Marivoet, L. "Control of the stability of a sliding slope in a railway cut, near Wetteren -"Proc.II Int. Conf.S.M. & F.E. 2.

(65) Li C.Y. "A concept of slope design of earth dam embankment "Proc. A.S.C.E. 79 (Sep.327)1953.

(66) May, D.R. " Application of the planimeter to Swedish method of analysing stabilis of earth slopes"- Trans. II Conf. Large Dams, 4, 1936.

(67) May, D.R. "Mays' Modification of slip circle method"- Earth Dam, Ch.8., Treatise on Dams, U.S.B.R.

(68) Meyer, O.H. "Comutation of stability of slopes"-Proc. A.S.C.E. 84, No.: S.M. 4 (J.S.M. and F.E. Dn). Oct. 1958 Pt. I.

(69) Middle Brocks, T.A. " Fort Peck Slide"- Trans.A.S.C.E. 107 (723-764)

(70) Mononobe Nagaho, Takata A. and Matumura, M. Seismic stability of Earth dams" - II Cong. Large Dams, 4, (435-444)

(71) Non-veiller, E. " The stability of slopes of dams

composed of hetrogeneous materials-Proc.III Int. Conf.S.M. and F.E., 2 (268-272) 1953.

(72) Peck, R. ~~xB~~ and Kaun, W.V. " Description of a flow slide in loose sand"- Proc. III Int.Conf.S.M.& F.E., 2.

(73) Peynircioglu, H "Earth movement investigation in a land slide area on the Bosporus"- Proc.IV.Int.Conf. S.M. and F.E. 2 (353-360).

(74) Petterson, R, Iverson, N.N. and Rivard PJ. "Studies of several dam failures on Clay Foundations"-Proc. IV Int.Conf. S.M. and F.E. 2(348-352).

(75) Petterson, K.E. " The early history of circular sliding Surfaces"- Geo.5(275-296) 1955.

(76) Raeds Chelders, H. " Improvement of the method of calculation of the equilibrium along sliding circles"- Proc.II Int. Conf. S.M. and F.E., 1.

(77) Rayaraman, S. "Stability of Slopes-" J.Inst.Engrs. (India) Jan.1956.

(78) Reinius, E. " The stability of slopes of earth dams". - Geo-5 (181-189) 1965.

(79) Reynolds, H.R. and Protopapadakis, P."Practical problems in Soil Mechanics".

(80) Rocha, M. "The possibility of solving soil mechanics ~~k~~ Problems by the use of Models"- Proc. IV. Int.Conf. S.M. and F.E. 2 (183-188).

(81) Root, A.W. " Correction of Land slide and slip outs"- Trans. A.S.C.E. Vol. 120 (281-289)

(82) Seval Don, R.A. " The slide in Loadalen, Oct 6, 1959 Geo. 6 (167-182) 1956.

(83) Sharma, B.C. "Land slide investigations and treatment in Assam "- I.R.C. XXIII- 3, Dec. 1958)

(84) Sing, P. " Slip Circle Method for analysis of Stability of Earth dams". J.Central Board of Irrn. (India) 6 (194- 202) 1949.

(85) Skempton, A.W. " London Clay Part II- Stability of Cuttings and Retaining walls in Clay"- Reprint.

(86) Skempton, A.W. " The Ø = 0 analysis of stability" Proc. II Int. Conf. S.M. and F.E. 1,

(87) Skempton , A.W. and Golder, H.Q. "Practical Examples of Ø = 0 analysis of stability of clays"- Proc. II Int. Conf. S.M. and F.E., 2 (63-70) 1948.

(88) Skempton , A.W. and Delory, F.A.- "Stability of natural slopes in London Clay " -Proc. IV Int. Conf.S.M. and F.E. 2 (379-381) 1957,

(89) Skempton, A.W. " Contribution to discussions"- Proc. Europ .Conf. Stab. Slopes, 3 (113-119) 1954.

(90) Slee, J.P. "Investigation of two unusual slips in Railway Cuttings"- Civil Engg. (Lond) 48, 1953.

(91) Smith, E.C." Soil Mechanics, Factor of Safety of Clay banks"- J.Inst. C.E. 18 (293-296) 1941-42.

(92) Sowers and Sowers " Stability of earth masses"

(93) Spelman, H.J. Handling landslides for Road Engineers Engineering News Record 98, (34-38)

(94) Taylor, D.W. " Stability of earth slopes"- J.Boston Soc. C.E. July, 1937 (Contribution to soil mech. 1925-40)

(95) Taylor, D.W. "Fundamentals of Soil Mechanics".

(96) Terzaghi,K. " Theoretical Soil Mechanics".

(97) Terzaghi, K. and Peck R.B. " Soil Mechanics in Engineering Practice."

(98) Toms A.H. " The present scope and possible future development of Soil Mechanics in British Railway Civil Engg. Constn and Maintenance" Proc. II Int. Conf. S.M. and F.E. 4 (226-236).

(99) Toms, A.H. " Recent Research into Coastal Landslide at Folken Warren, Kent, England"- Proc. III Int. Conf. S.M. and E.E. 2 (288-293) 1953.

(100) U.S.Bureau of Reclamation " Treatise on Dams"- Cha.8.

(101) Varghese, P.C. " A New Approach to the Analysis of slopes"- I.R.C. XIX-1.1955.

(102) Viner Brady, N.E.V. " Folkenstone Warren Landslips." Renudial Measures 1948 - 1948-1954"-Proc. Inst. C.E. 4 (ii) (429-466) 1955.

(103) Ward, W.H., Penman A and Gibson, R.E. " Stability of a bank on a thin peat layer"- Proc. Europ. Conf. Stab. Slopes, 1, 1954.

(104) Ward, W.H. " A slip in Flood Defence Bank constructed on a peat bog. " Proc. II Int. Conf. S.M. and F. Eny, 2, 1948.

(105) Ward, W.H. " A Coastal Land slip " - Proc. II Int.Conf. S.M. and F.E., 2, 1948.

(106) Watson, J.D. " Earth Movement Affecting L.T.E. Railway in deep cutting of Ux bridge "- Proc.Inst. C.E. 5 (ii) (302) -531).

(107) Watt, A.J. "Pore Pressure and Shear Strength tests on earth dam materials" -Proc. II Anstr. New Zeal. Conf.S.M. and F.Eng. (58-65) 1956.

(108) Wilson V. Binger " Analytical Studies of Panama Canal Slides " Proc. II Int. Conf. S.M. and F.E. 2, 1948.

(109) Wood, A.M.M. " Folkestone Warren Landslips" Investigations 1948- 1950" - Proc. Inst. C.E., 4- (ii) (410 - 428)

Additional Bibliography (which are not referred to in the dissertation.

1) Akai " On the Relationship between Stress Distribution and Degree of compaction of Earth Embankments" - Proc. 6th Japan Nat. Cong.Appl. Mech., 1956.

2) Alden, W.C. " Lanel slides and flood at Gross Ventre Wyoming" - Am. Inst. Min. Met. Eng. Trans.,76 (347-361) 1928.

3) Allen, W.H. " Arresting of a Land slide on an Indiana Highway" - Civil Eng. 6 (384-385) 1936.

4) Alter, J.C." Mud Floods in Utah" - Monthly weather Rev.,58 (319-321) 1930.

5) Aramco Culvert Manufacturer's Association " Land slides, Types, Causes and Cures" - Hand Book of Culverts and Drainage Practice (353- 364) 1930.

6) Baker, R.E. " Determining the correcting Action of Highway Land-slide Problems" - Bull.H.R.B. 49(1-27)1952.

7) Barber, E.S. " Electrical Stabilization of Soil" - Roads and Streets, 91-6 (64-65)1948.

8) Barber, E.S. and Ershen, C.E." Graphical Analysis of stability of Slopes" -Pub. Roads, 21 (147-157) 1940.

9. Bayer, E.J." Usual Apethods of Handling Problems in correction with slips, slides and water pockets" - Bull. Am.Rly.Eny.Assn.,28,NoL295 (845-848) 1927.

10.Bendel,L." Protected sliding surface" - Prov. II int. Conf. 8. M & F.E., 3 (53-57) 1948.

11) Biblodeau, P.M. " The Nicolet Landslide" - Proc.10 Canad. Soil Mech. Conf. Tech. Memo. No.46

12) Bishop, A.W. - " Stability of Earth Dams 11 Ph.D. Thesis, London Uny.

13) Burmister, D.M. - " Factor of safety in stab. Analysis" Civil Eng., 16, No.7, 1946.

14) Chugaev " Approximate Determination of Stability of Earth Dams" - II Cong. Large Dams, 4 (593-614) 1936.

15) Deere, D.U. " Seepage and Stability Problems in Deep Cuts in Residual Soils" - Proc. Am. Rly. Eng. Assn.58 (728-745).

16) Farquharbon, F.B. and Hennes, R.G. - " Gelatin Models for Photoelastic analysis of stress in earth masses" - Civil Eng. April 1940) p. 211)

17) Hardy, R.M. " Diagnosis and Treatment of Slide Conditions Affecting Highways" - Roads and Eng. Constn., 93-10 (151-156)

18) Huinga, T.K. " Design of cross section of earth embankments" - Proc. H.R.B. 18-2 (81-89)

19) Jurgenson, " On the Stability of Foundations and Embankments" - Proc. II. Int. Conf.S.M. and F.E. 2 (194-200)

20) Johnson, F. " Calculation of stability of Earth Dams" - II Cong. large dams 4 (463 - 480)

21) Karpoff, P. " Laboratory Investigations of Electrical Drainage and Electro chemical Stabilization of Soils"- Struet Res. Sec. Rept. No. SP-29, U.S.B.R.

22) Ladd, G.E. " Landslides, Subsidences and Rock falls as problems for Rail road Engineer II Proc. Am. Rly.Eng. Assn. 36 (1091-1162) 1935.

23) Ladd, G.E. " Landslide and their relati on a to Highways"- Pub. Roads 8 (21-31) and 9 (153-163)

24) Minikin, R.L. " Stability of Clay Slopes" - Civil Eng. (Lon.) 85 (820-322) 1940.

25)Nevins, T.H.F. " Stability of Earth Slopes" - C.W. Engr. 37 (178-83) 1949.

26)Skempton, A.W. and Bishop, A.W. " The Gain in Stability due to Pore Pressure Dissipation in Soft Clay Foundations" - Trans.V Cong. Large Dams, 1. (613-638) 1955

27)Tan,E.K. " Stability of Soil Slopes" - proc. th. Inst. Soil Engrs. 1 (49-73) 1949.

28)Taylor, D.W. " Notes on Stability of Slopes" Proc. I Int Confg. S.M. and F.E.,3, Paper G-16

29)Taylor, D.W. " Limit Design of Foundation and Embankments" - Proc. -1 + .R.13. 19 (454-459)

30)Terzaghi,K. " Critical height and Factor of Safety of Slopes against Sliding" - Proc.I. Inst.Conf. S.M. & F.E. 1, Paper G.6

31)Terzaghi, K. " Stability of slopes of nationa clay"- Proc. I. Int. Conf. S.M. and F.E., 1, Paper G.7.

32)Ward,W.H. " The Stability of National Slopes" - Geog. Journ.,London, 105, (170-197) 1945.

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10/6/1960

Roorkee
10.6.1960.

(C.E.G. JUSTO)

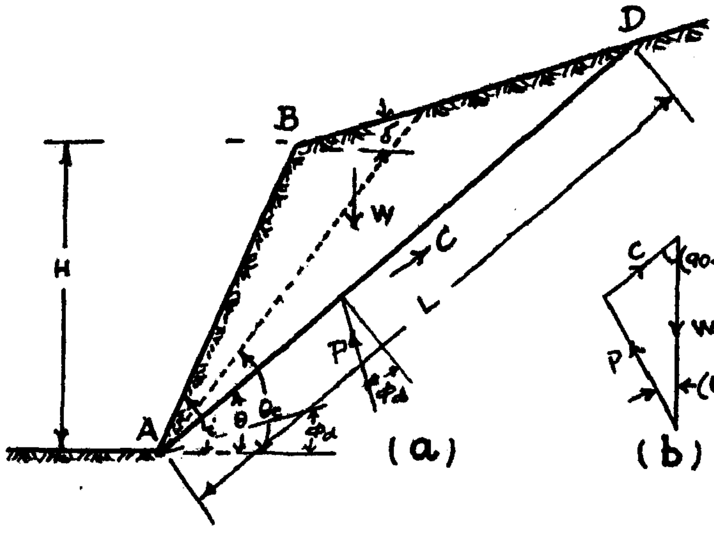


FIG: 1. CULMANN METHOD.

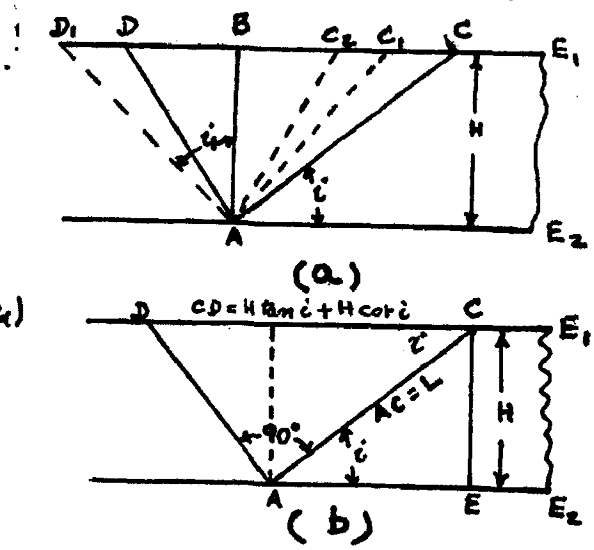


FIG: 3. METHOD OF SUPER-IMPOSED LOAD.

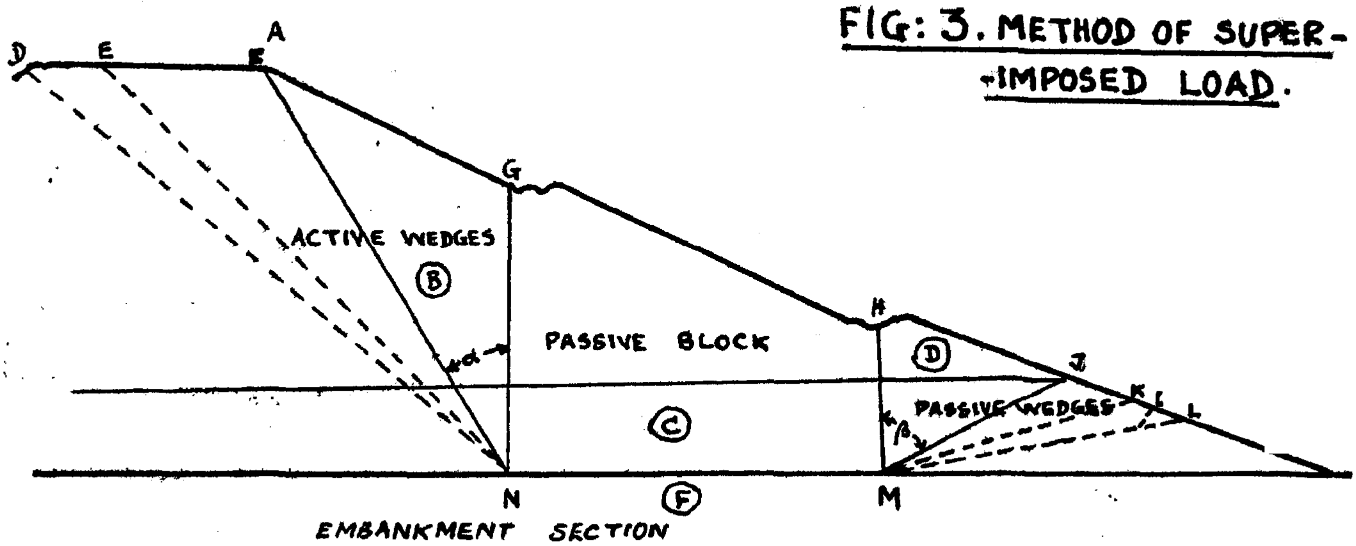
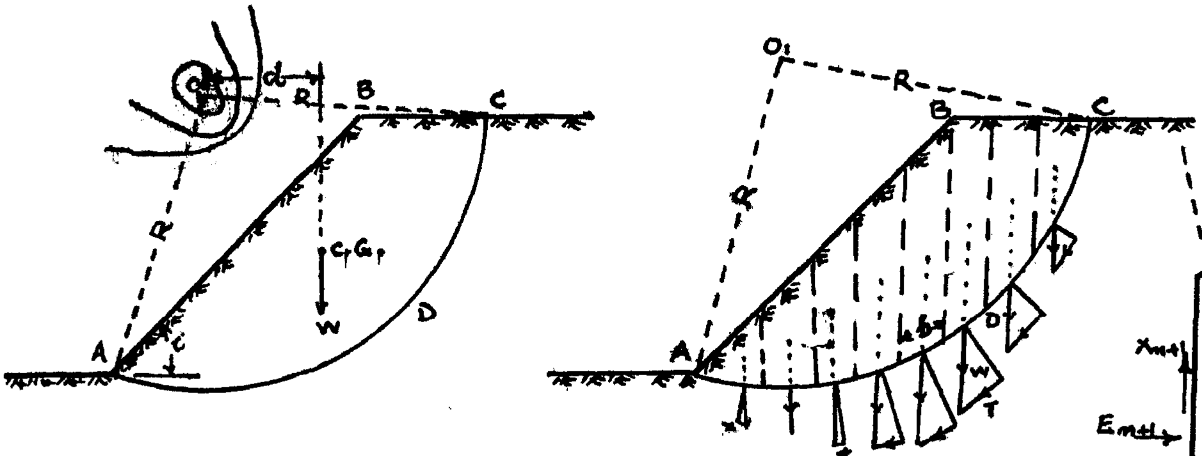
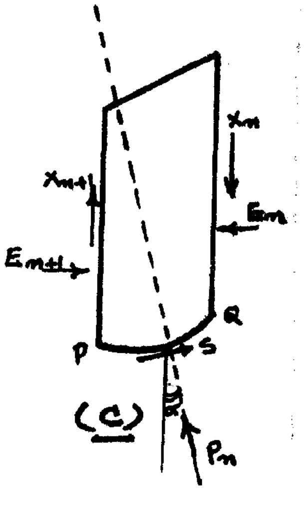


FIG: 2. WEDGE METHOD.



(a) 'φ=0' ANALYSIS

(b) METHOD OF SLICES.



(c)

FIG: 4. SWEDISH METHODS.

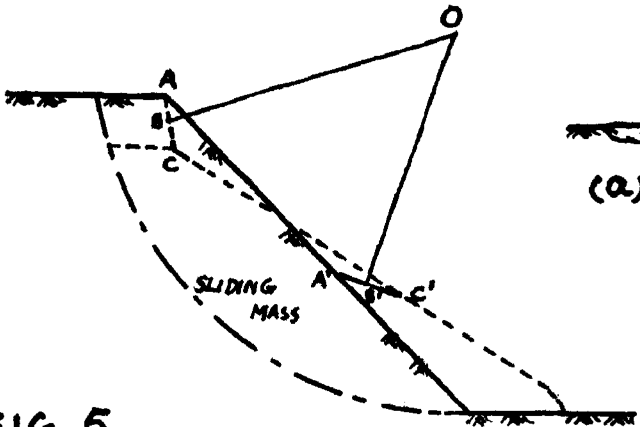
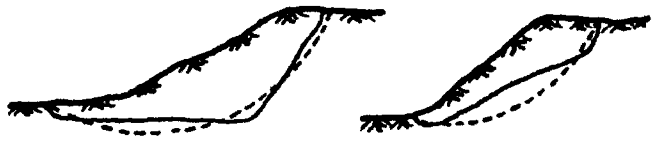


FIG: 5,
METHOD OF LOCATING CENTRE OF ROTATION OF A SLIDE MASS.



(a) HORIZ: STRATA (b) DIPPING STRATA

FIG: 6. SHAPES OF FAILURE SURFACES IN STRATIFIED SOILS.

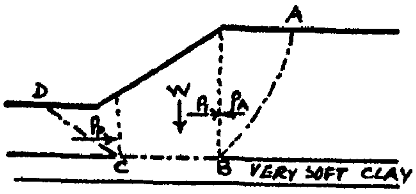
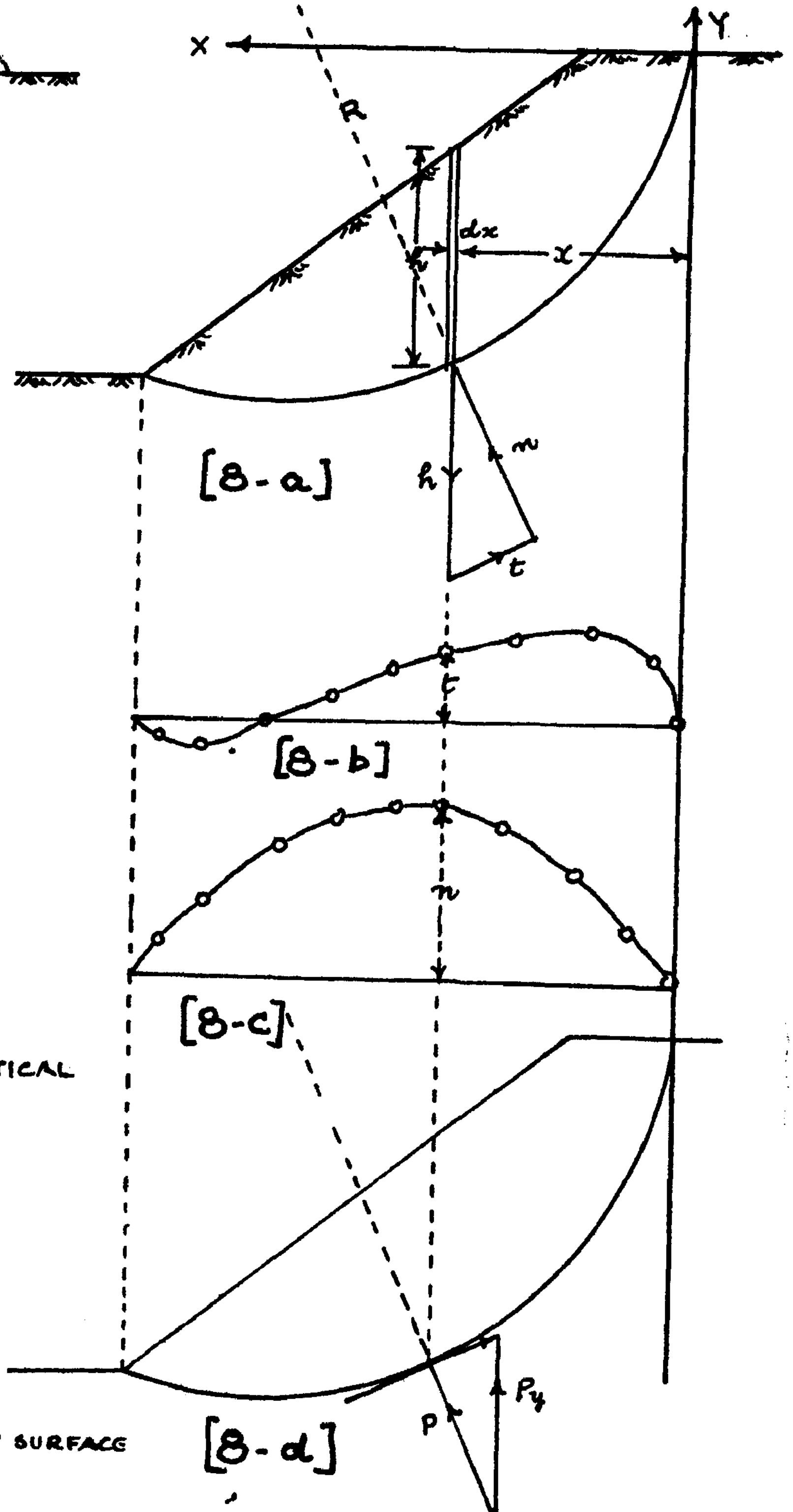
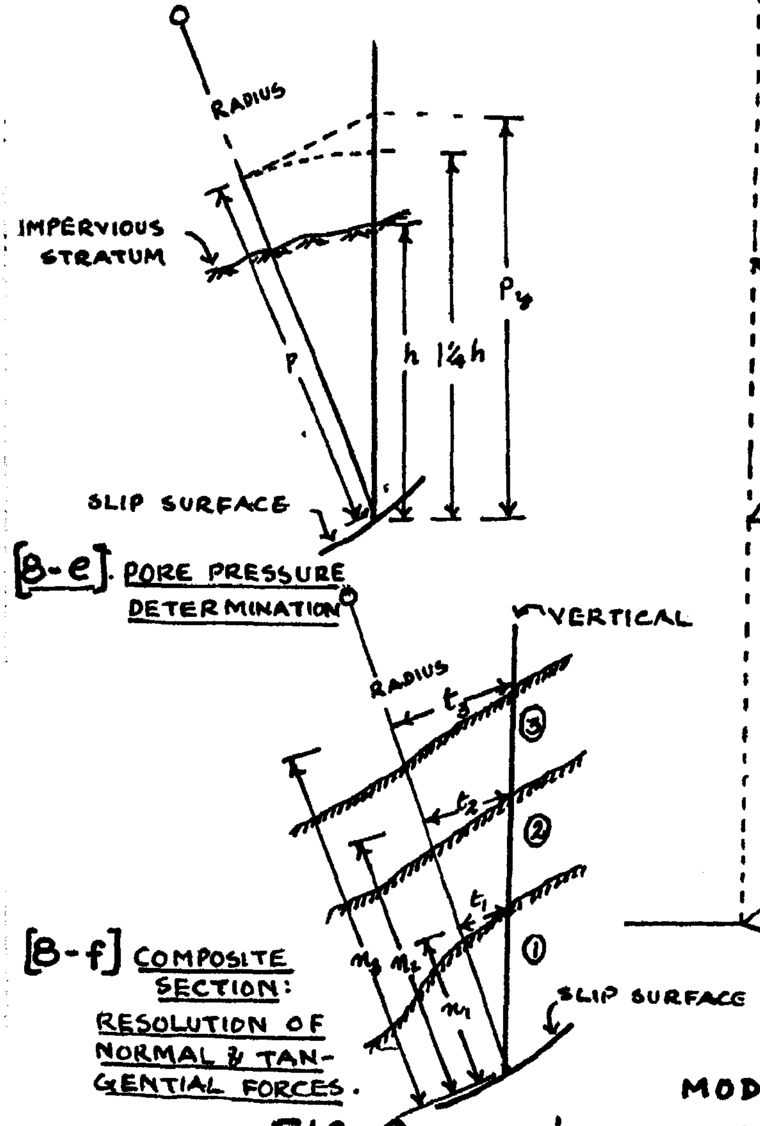


FIG: 7.

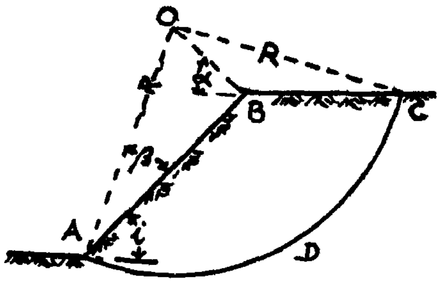




MODIFIED METHOD OF SLICES.

FIG. 8 : MAY'S MODIFICATION OF SLICES METHOD.

SLOPE	ANGLE $i°$ WITH HORIZ:	$\alpha°$	$\beta°$
1 : 0.58	60°	40°	29°
1 : 1.00	45°	37°	28°
1 : 1.50	33° 47'	35°	26°
1 : 2.00	26° 34'	35°	25°
1 : 3.00	18° 26'	35°	25°
1 : 5.00	11° 19'	37°	25°



9. FELLENIUS' CONSTRUCTION FOR LOCATING CRITICAL CIRCLE.

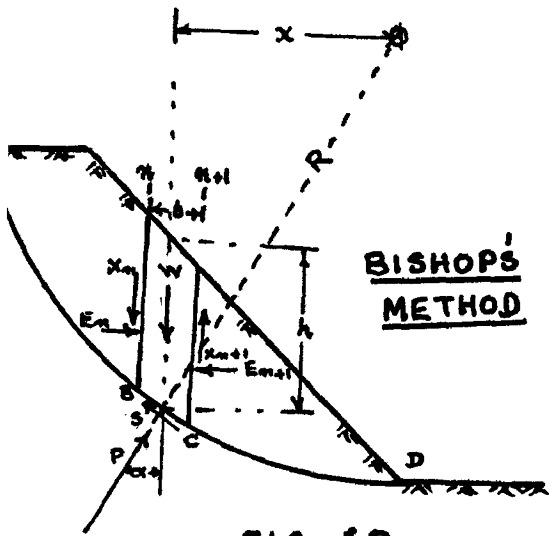


FIG: 10.

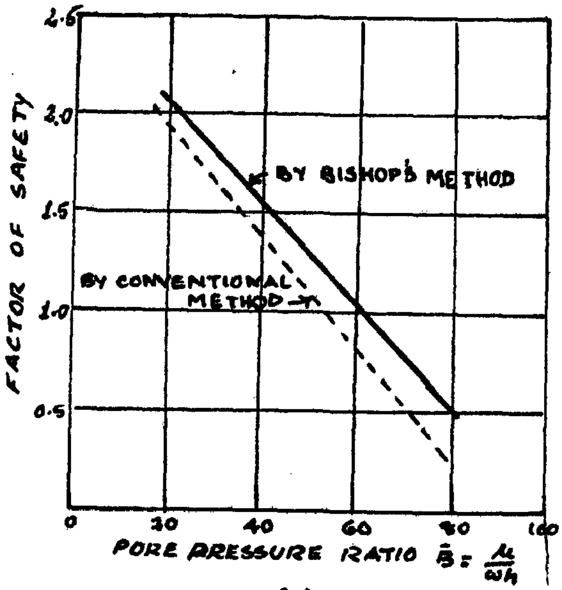


FIG: 11.

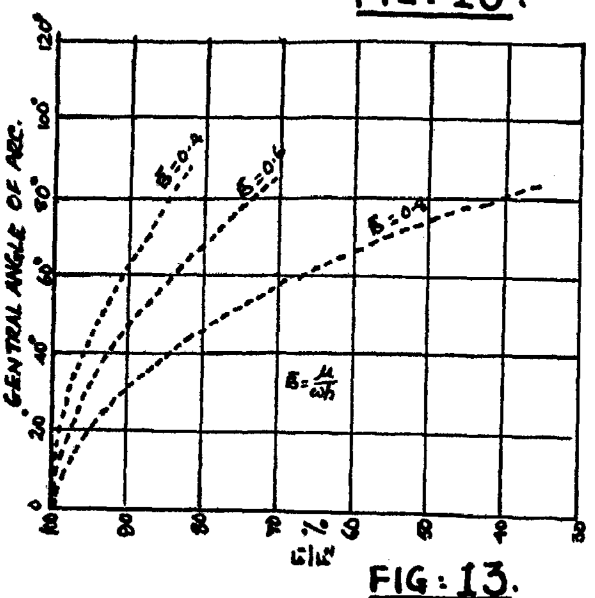


FIG: 13.

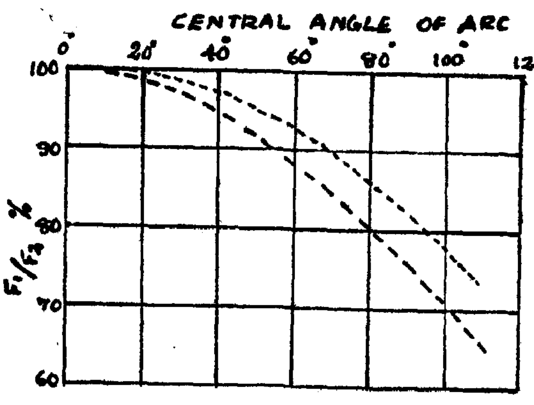


FIG: 12. TYPICAL DRAWDOWN CASES

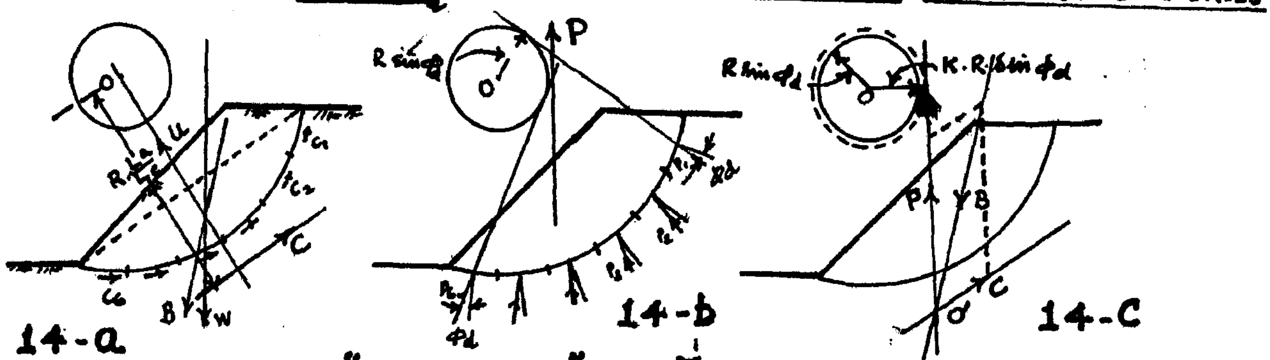


FIG: 14. "ϕ-CIRCLE" METHOD (GRAPHICAL)

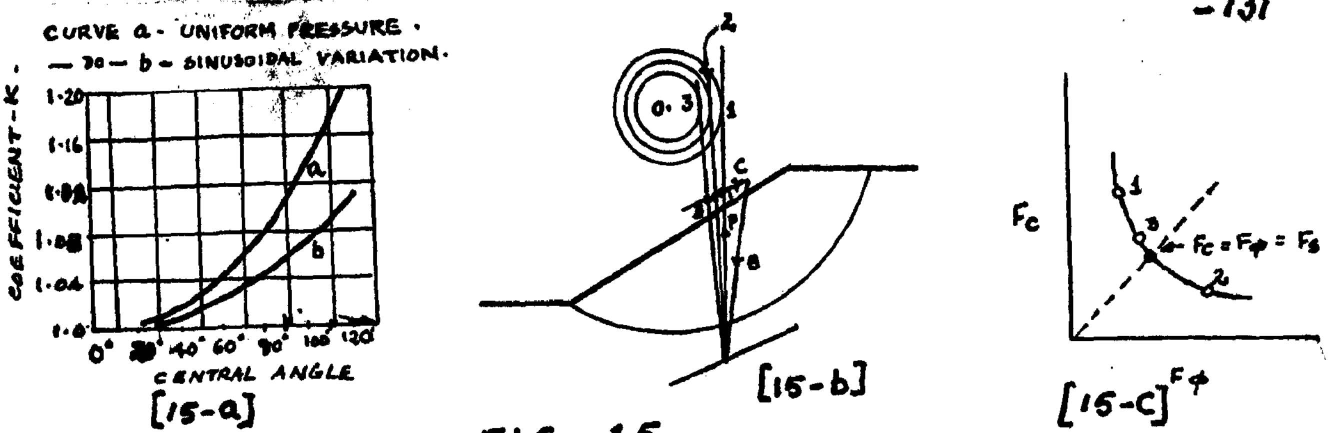


FIG: 15

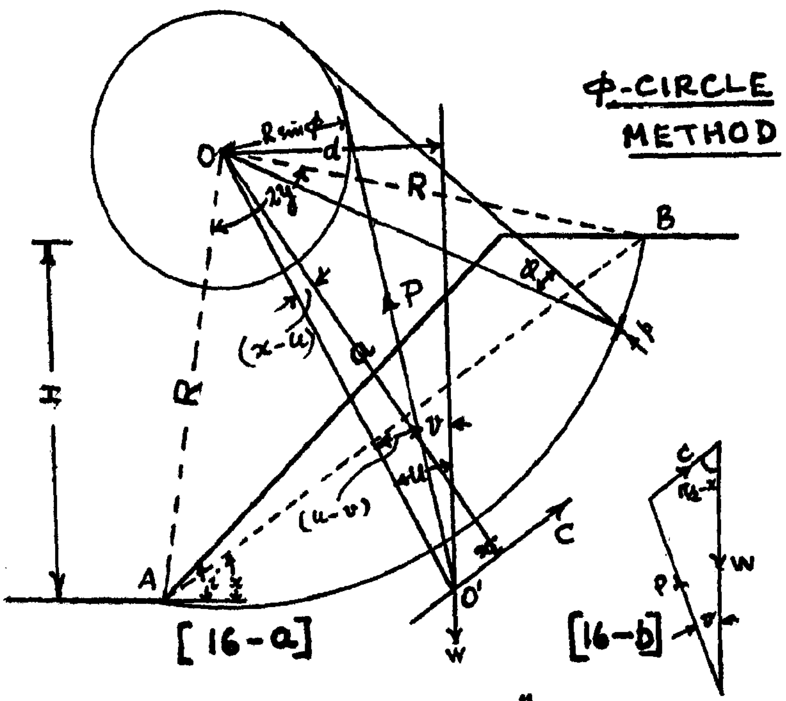


FIG: 16. "φ-CIRCLE" METHOD.

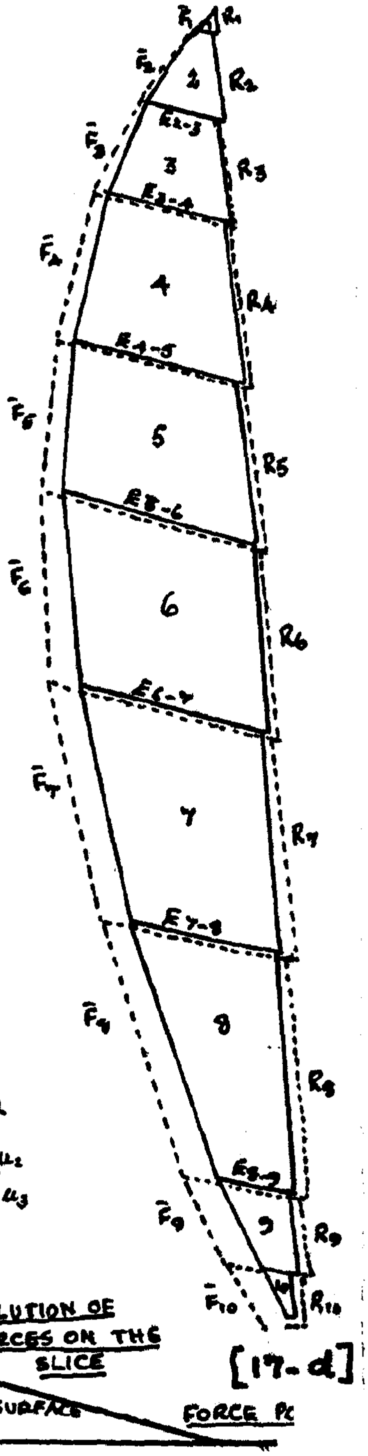
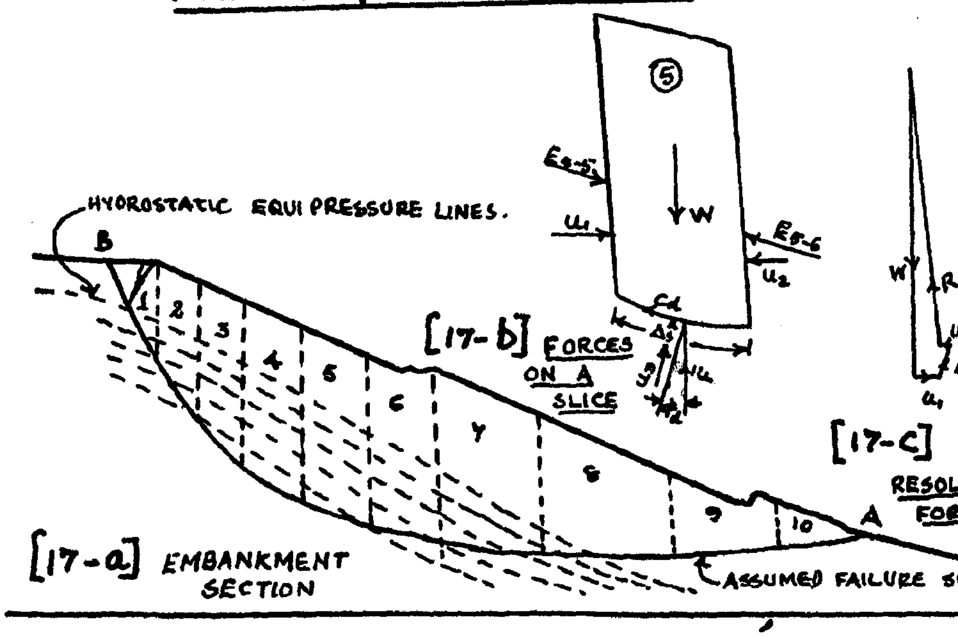



FIG: 17. GRAPHICAL APPLICATION OF MODIFIED SWEDISH METHOD.
[BISHOPS]

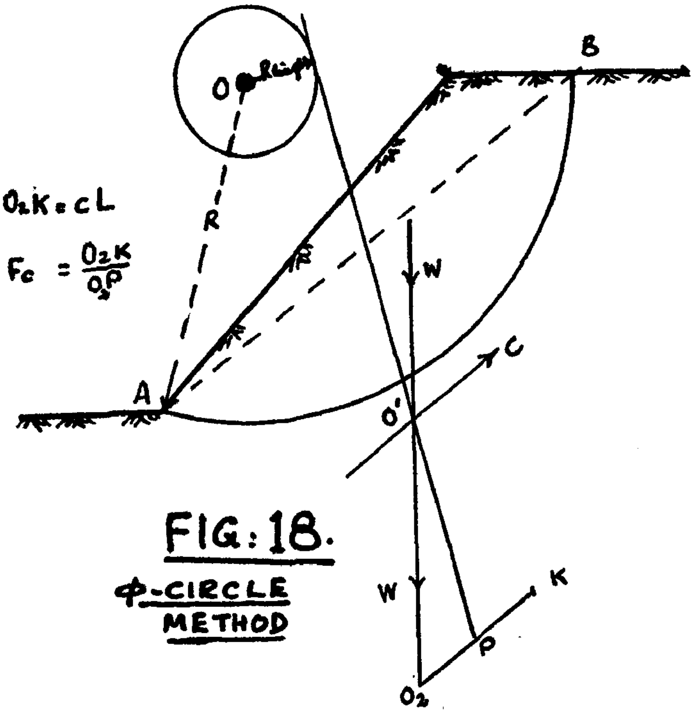


FIG: 18.
φ-CIRCLE METHOD

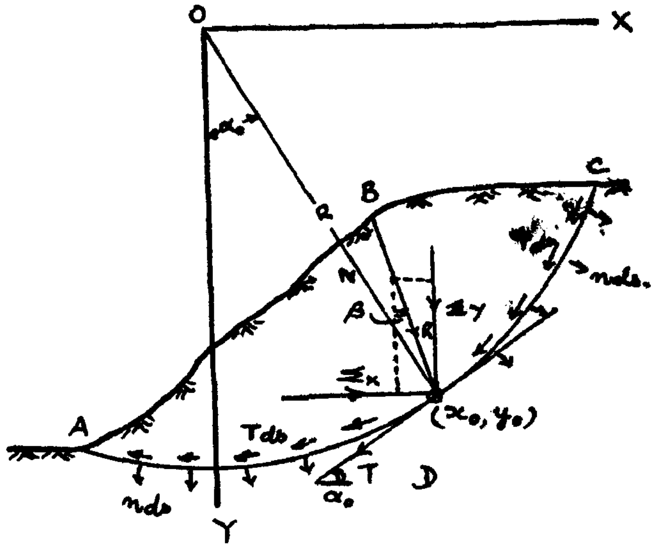


FIG: 19. IVANOV'S METHOD

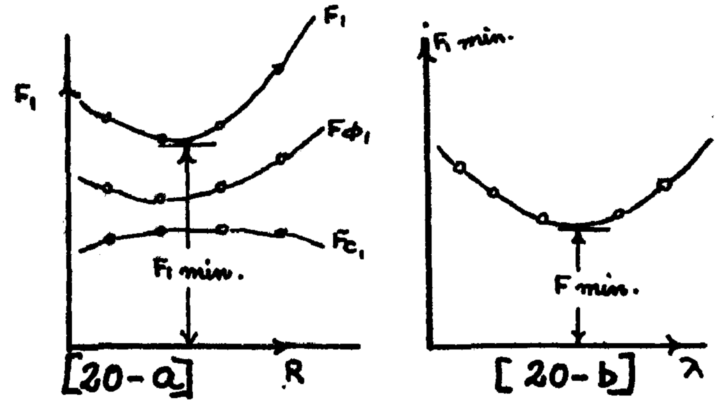


FIG: 20.

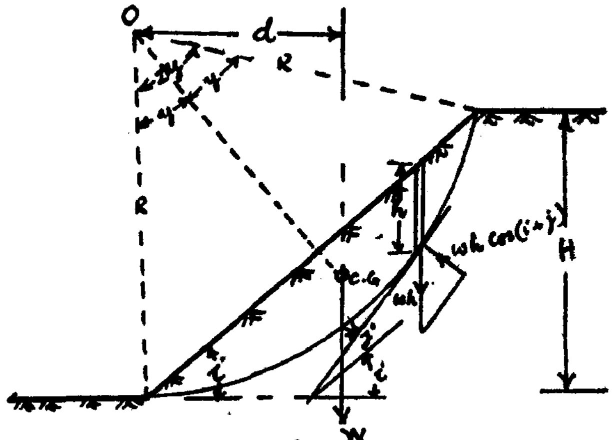


FIG: 21. MEYER'S METHOD.

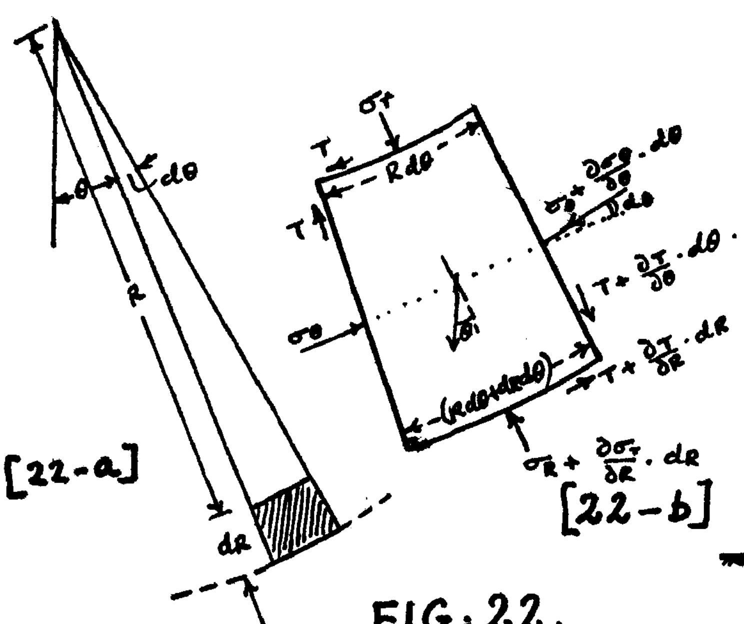
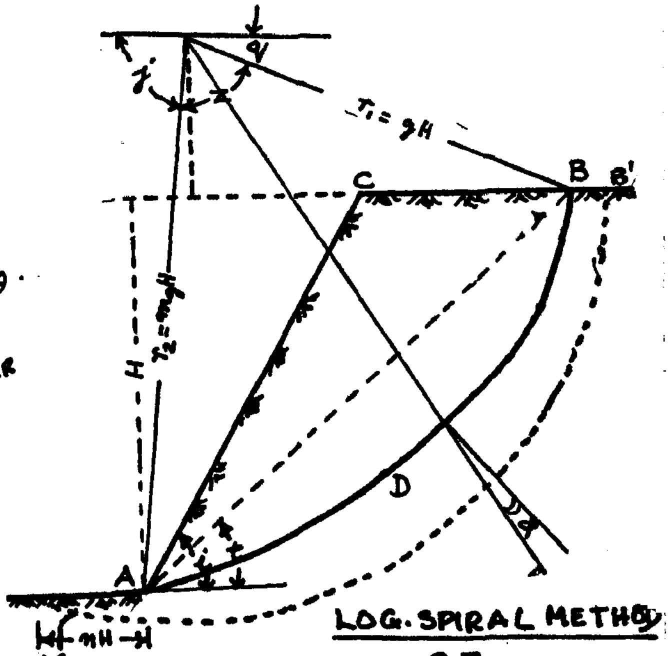


FIG: 22.
JAKY'S METHOD.



LOG. SPIRAL METHOD
FIG: 23.

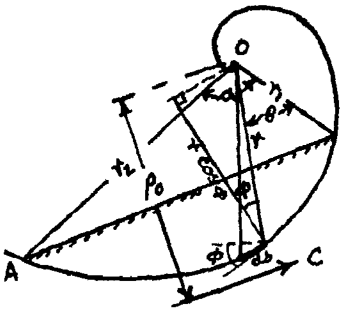


FIG: 24

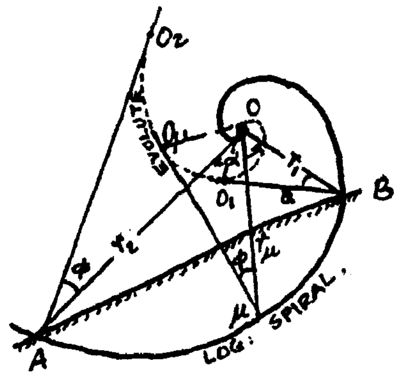


FIG: 25.

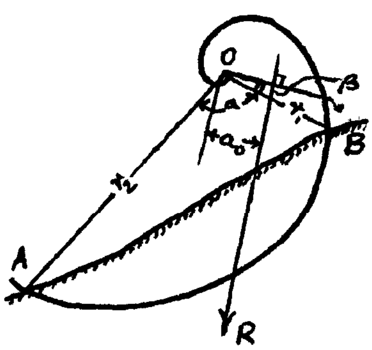


FIG: 26.

LOG: SPIRAL METHODS

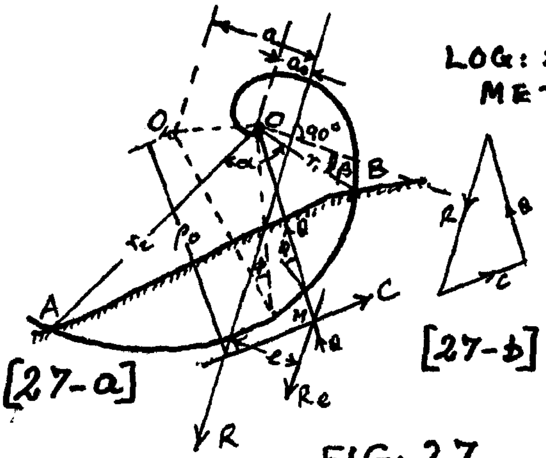


FIG: 27.

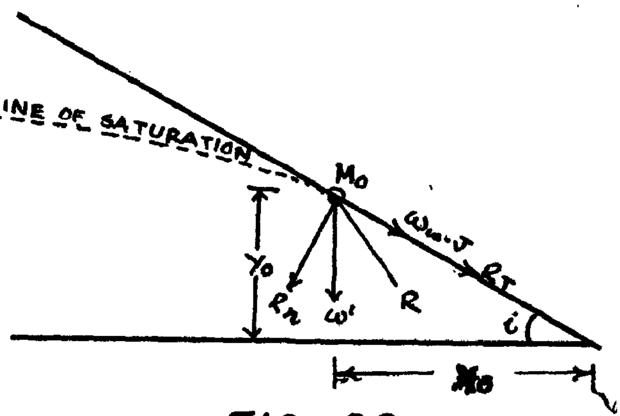
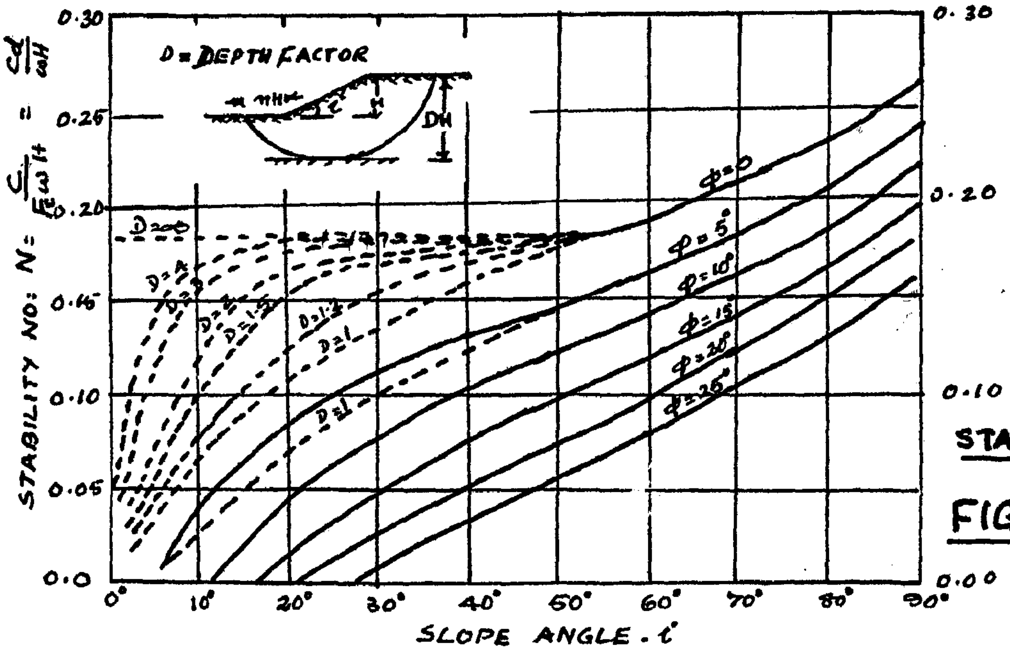


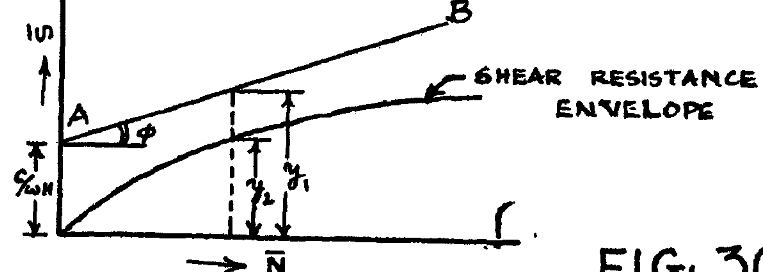
FIG: 28.

PAVLOVSKY'S THEORY.



TAYLOR'S STABILITY CURVES.

FIG: 29.



SHEAR RESISTANCE ENVELOPE METHOD.

FIG: 30.

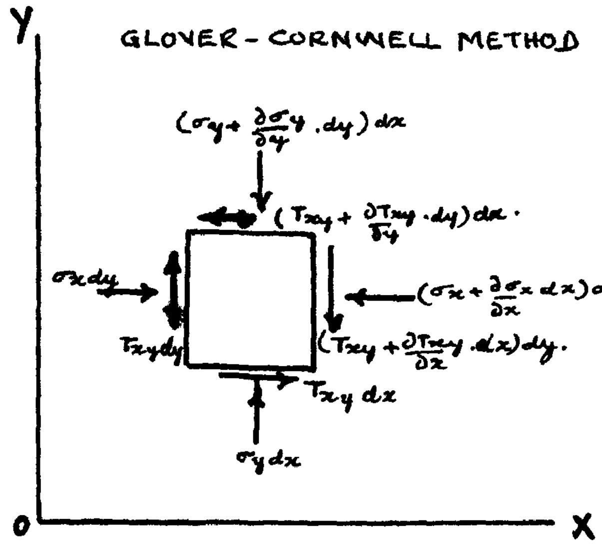


FIG: 31.

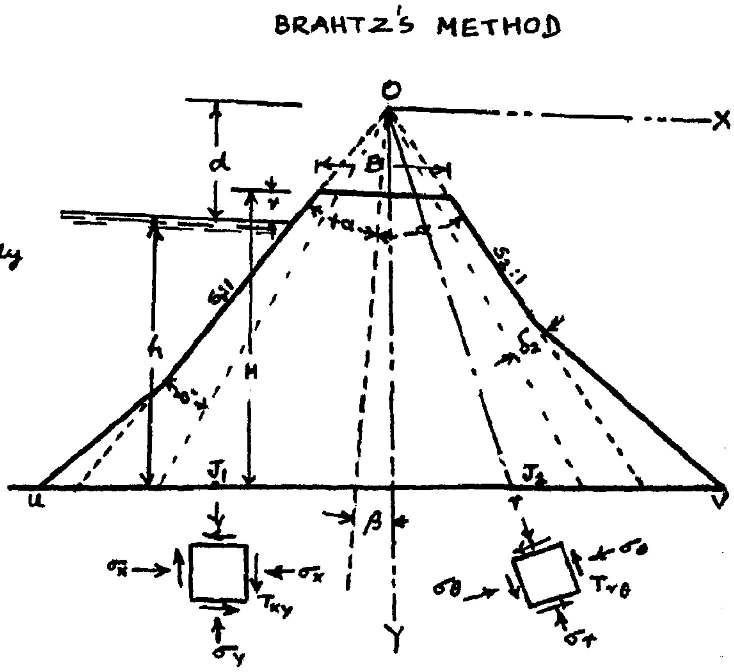


FIG: 32.

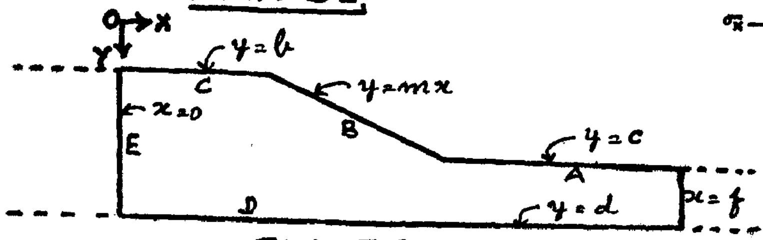


FIG: 33.

DIFFERENCE EQUATION METHOD.

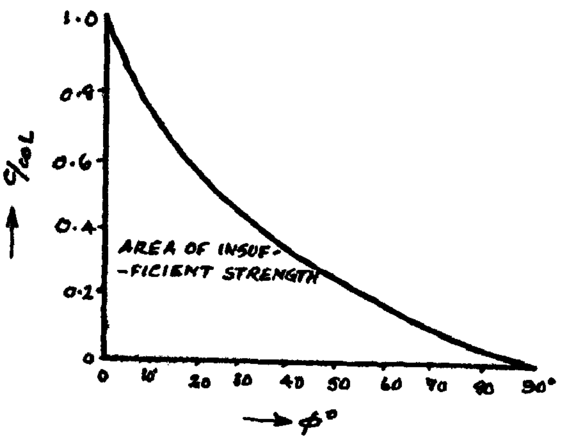


FIG: 34.

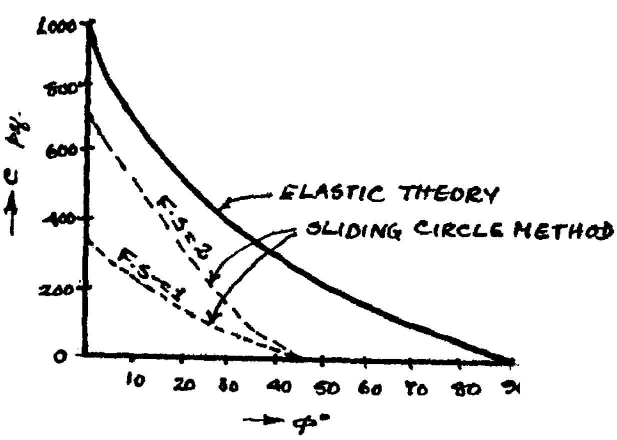
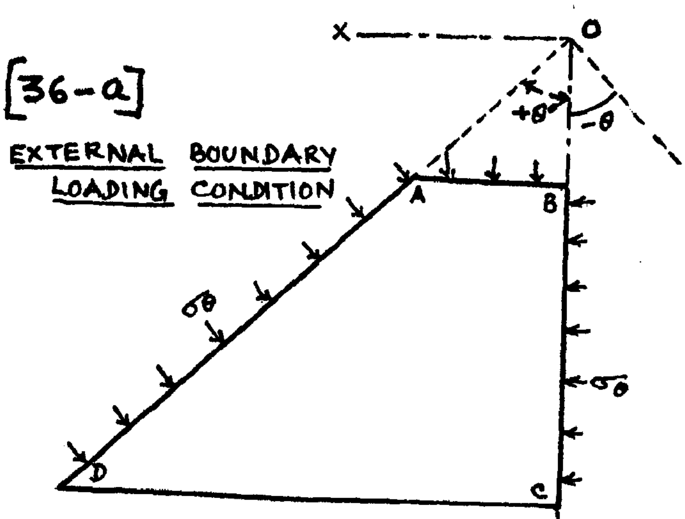


FIG: 35.



ANALYSIS BY THEORY OF PLASTICITY

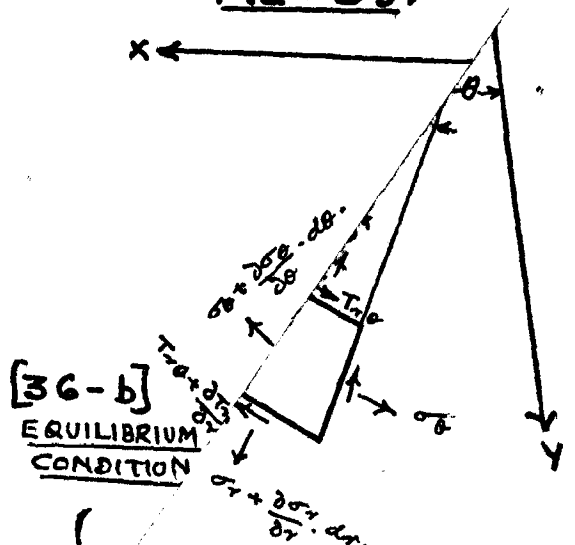


FIG: 3(

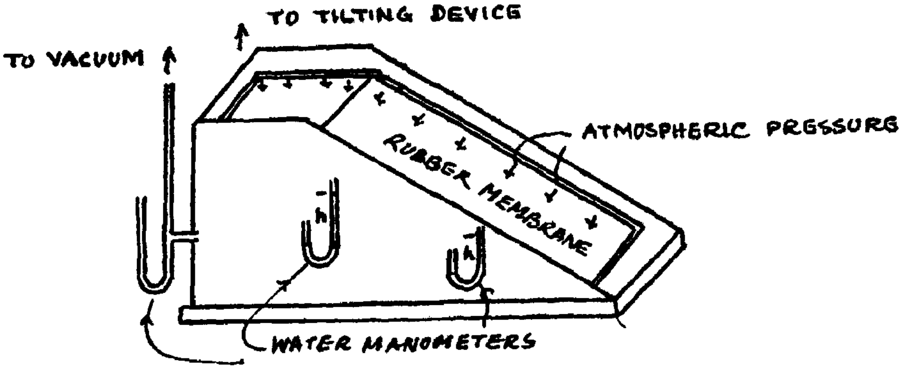


FIG: 37.

INVESTIGATIONS BY MODEL STUDY

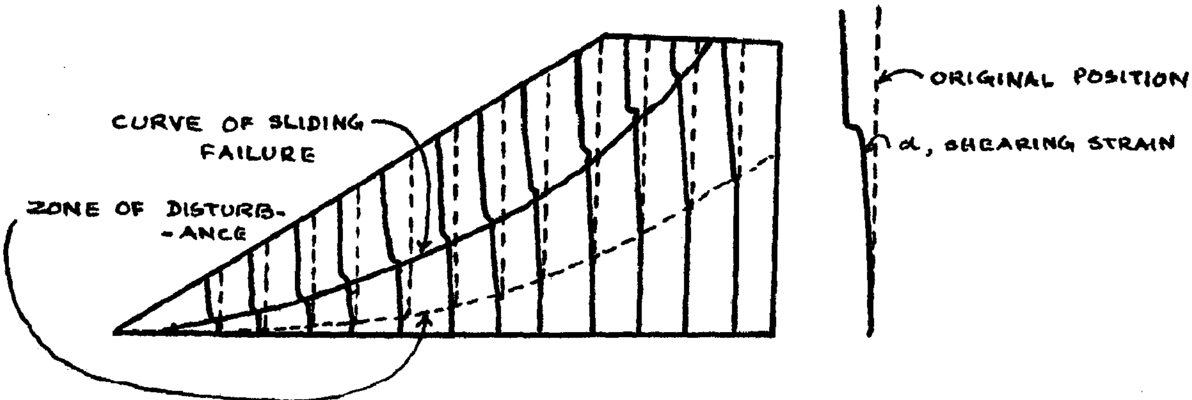
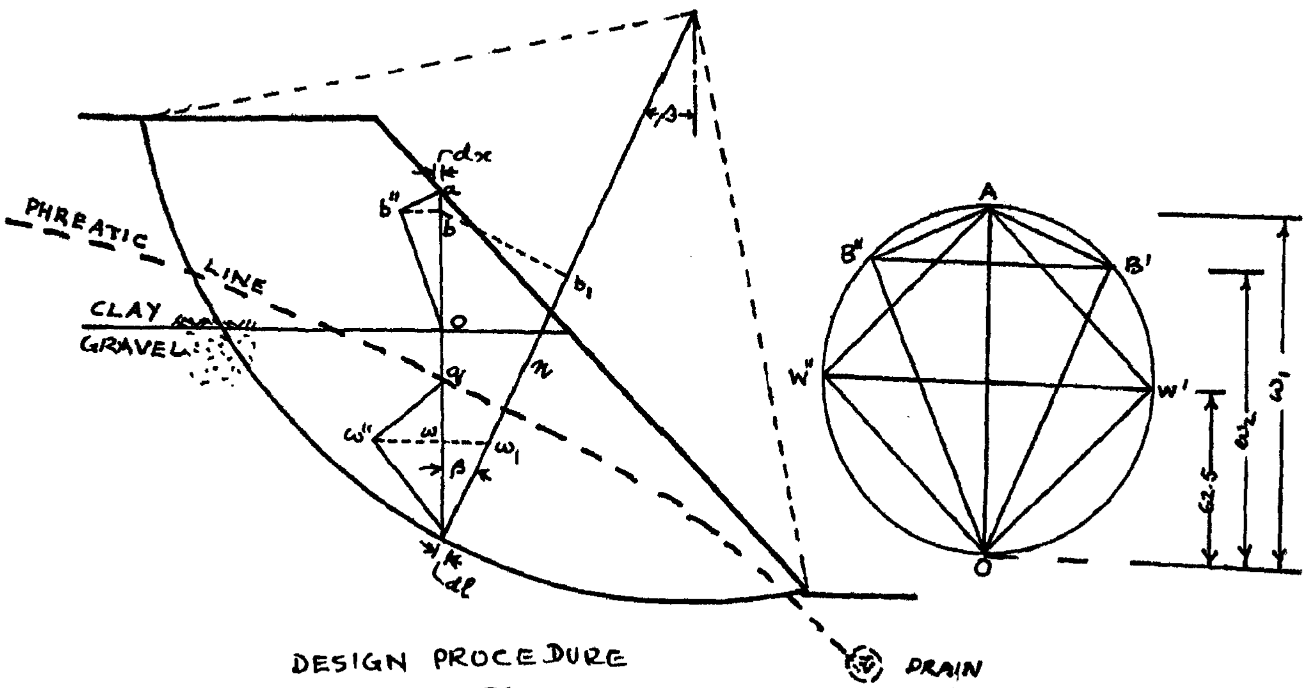


FIG: 38.



DESIGN PROCEDURE
FOR CUTS

FIG: 39