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Research article

Analysis of the stability of embankments on clay foundations

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ABSTRACT

This paper proposes a method of analysis of the stability of embankments on clay foundations which makes use of a conventional total stress analysis, corrected to take into consideration the preconsolidation pressure and the undrained shear strength profile. Good results are obtained for three well documented cases of embankment failures, the data of which allow a direct application of the proposed method. Many other failure cases, for which the published data are insufficient to allow such a direct application, are analyzed on statistical basis using a relationship between $Cu/\sigma_{p'}$ and I_p. Bjerrum's correction for total stress analysis and Otoko's correction for effective stress analysis appear as particular cases, only statistically applicable, of the proposed method. Based on the data generated in the Niger delta, Nigeria, the results obtained from the new method are compared with those obtained from limit equilibrium methods, Taylor's chart, upper and lower bound limit analyses. While Taylor's chart and limit equilibrium methods gave approximately the same results as upper bound limit analysis, the new method gave results between upper and lower bound limit analyses, which is considered to be a great advantage over the other methods.

Keywords: Slope stability; Proposed method; Otoko's correction; Bjerrum's correction; Taylor's chart; Limit equilibrium; Limit Analysis

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1.0 INTRODUCTION

The many factors that control slope stability have been describe in reports by Parry (1972), bjerrum (1973) and otoko (1987). A review of some of the analytical work that has been published on these factors are:

(a) <u>Increase of Shear Strength with Depth</u>

Solutions for cuttings and naturally formed slopes in clays whose strengths increase linearly from zero at the ground surface, were presented by Gibson and Morgenstern (1962) and by Kenny (1963). These were extended by Raymond (1967) and by hunter and Schuster (1968) to include cases where the surface shear strength is discrete.

The strength of soft clays is often described by the ratio C_u/σ_p' or perhaps more accurately by $\Delta C_u/\Delta\sigma_{p'}$ (Brown 1970). 'Normalized soil parameters' were used by Ladd and Foott (1974) in studies of a number of field projects. They emphasized preconsolidation effects, especially the over consolidation ratio $\sigma_{pc'}/\sigma_{po'}$.

(b) Anisotropy

Even if clays appear uniform, the grain structure, the in situ stresses, and the strengths are all usually anisotropic (Lo and Milligan 1967; Bjerrum 1973; Bhaskaran 1974). Anisotropic strengths are often measured from samples trimmed at different orientations to the vertical (Lo and Morin 1972; Loh and Holt 1974). However, the results are often highly variable, even in closely neighbouring sites (Crooks and Graham 1976). Calculations of embankment stability using anisotropy measured in this way suggest, in any case, that the effects are relatively minor (Delory and Salvas 1969; Davies and Christian 1971). With some exceptions, they have not received detailed attention in analytical studies (Law 1978). On the other hand, stress induced anisotropy is often significant (Di-Biagio and Aas 1967; Graham 1969; Mitchel 1970; Tavenas and Leroueil 1977; Crooks and Graham 1976). If the variation of shear strength with direction is expressed functionally (Bishop 1966; davis and Christian 1971) then the shear strength, which can be mobilized at any point on an assumed failure surface can evaluated (Lo 1965; Ranganatham et al 1969; Law 1978).

(c) Strain Rate Effect

The times taken to reach failure in field vane tests are usually quite different from the times-to- failure in full-scale embankments. Bjerrum (1972) suggested that vane strengths used in $Ø_{u=}O$ analysis of embankments should be reduced by an empirical correction factor µ to bring them closer to the strength actually mobilized in the clay. His reduction factor was zero or small for lean clays, but increased significantly with increasing plasticity, torstensson 1973; cassagrande and Wilson 1951 have also reported increased strength with increasing rate of strain. Very sensitive soils also show high strain rate effects Dascal and Tournier 1975; Baecher and Christian 2003; Duncan and stephen 2005.

(d) Sample size Effect

The strength of stiff fissured clay measured in triaxial tests were found to be several times that of the field strength; (Bishop 1966, 1971; Lo 1970; Marsland 1971, 1972). But with large size samples, Marsland (1979) noted marked reduction in average strength; clearly showing the effect of sample size.

(e) Progressive Failure

Progressive failure results from strain softening when a localized zone becomes over-stressed. The use of a post peak failure envelope has been shown necessary for the back analysis of slope failures (Lefebvre and La Rochelle 1974; Law and Lumb 1978).

(f) Sample Disturbance

Effect of sample disturbance is clearly given by Kallstenius 1958, 1963, 1971, Milovic 1971, Schjetne, 1971, Landa 1964 and Berre 1969. Mechanical disturbance resulting from sampling operations seem to be the most obvious disturbance to clay samples.

(g) Plane Strain- vs. – Triaxial

Field failures approximate to plane strain conditions while the samples are tested in triaxial conditions in the laboratory (Otoko 1985; 1987; 1988).

Factors (c) and (e) would lead to an unsafe design neglecting all other influences on the safety factor (Otoko 1977). With the exception of factor (d), the remaining factors would lead to a conservative design. Which of these factors are the most important is not known, although it seems clear the net result is conservative for most clays (Otoko 1997).

It is with all these factors in mind that the measured shear strength had to be reduced for high plastic and organic soils (Skaven – Haugh 1931; Hultin 1937; Caldenius 1938; Jacobson 1946).

In 1972 and 1973 Bjerrum proposed new general correction factors for the undrained shear strength, C_u, that depends on the plasticity index (Iρ) of the clay. But soon, cases were reported where the correction was not always applicable and conservative. Thus, necessitating the adjustment of the correction to local experience for each clay. Other correction factors were suggested by Pilot (1972) and Dascal and Tournier (1975); and other improvements in the $Ø_{u=}o$ method were: a semi-empirical method proposed by Trak et al (1980) based on an interpretation of Bjerrum's data made by Mesri (1975); the USUALS (Undrained strength at large strains): method proposed by La Rochelle et al (1974) and the SHANSEP (stress history and normalized soils engineering parameters) method proposed by Ladd and Foot (1974).

However, as pointed out by Janbu (1975), Schmertmann (1975) and otoko (1987) the effective stress method is the theoretically correct method of slope stability analyses; as such, otoko (1987) proposed new empirical relationships for correction of factors of safety obtained from effective stress stability analyses.

At present, the interpreparation of undrained tests in terms of effective stresses is difficult. Thus the stability of embankments on clay foundation is still usually assessed by total stress methods, mainly because of difficult pore pressure predictions in clayey foundations.

This paper gives a stability method using four simple steps to evaluate the safety factor. After a conventional total stress analysis, the method introduces the consolidation profile, $\sigma_{p'}(z)$, of the clay. Thus it makes use of the $\sigma_{p'}$ term, which is both an effective stress parameter and a deformability (indirect) parameter. Consequently the method makes a double transition: the first one from total to effective stresses; and the second from standard failure methods, considering only the stress level, to other methods (to be developed) considering an excessive deformation level.

2.0 PROPOSED METHOD

The proposed method derived initially from observations of embankment behaviour prior to failure; it is also related to effective stress concepts.

Observations

A clay layer loaded by an embankment has at first: pseudo elastic behaviour: its settlement and horizontal displacements are approximately proportional to the vertical load. When the conventional safety factor (total stress analysis) falls below 1.5- 1.4, marked increases are registered in the pore pressures (D′ Appolonia et al 1971), the settlements (Bourges 1970), and the horizontal displacements (Marche and Chapius (1974).

Relation to effective stress concepts

It is well known that the amount of pore pressure generated in clay depends on the relative rigidity of the pore water and the clay structure. If a marked increase is registered in the pore pressures, this can only result from a change in the rigidity of the clay, which consequently allows greater displacements of solid mass. It is well known that this change of rigidity corresponds to the preconsolidation pressure, $\sigma_{p'}$, which is usually defined from oedometer tests. Furthermore, a paper by Leroueil et al (1978) convincingly established, from many recorded observations, that the change in pore pressure behavior, at a given point in the foundation, is related to $\sigma_{p'}$ at this point.

The proposed method is derived from the preceding observations and comments, with the following reasoning:

a) It is known that two clay layers having the same profile $C_u(z)$ but different geological origins may have different behaviours when identically loaded, and may fail for different heights of embankments having the same geometry. Bjerrum's correction tries to take into account such a difference.

b) It is the author's opinion that two clay layers having the same profile $\sigma_{p'}(z)$ will fail identically when identically loaded (short term condition), even if they have different $C_u(z)$

Consequently it was assumed that the usual formula of stability charts.(equation 1)

$$P_{ult} = \gamma\, H_{failure} = N_{\O o} C_u \text{ ---------------------------------------(1)}$$

should be replaced by a formula like:

$$P_{ult} = \gamma\, H_{failure} = N_{\O o'}\, \sigma_{p'} \text{ ---------------------------------------(2)}$$

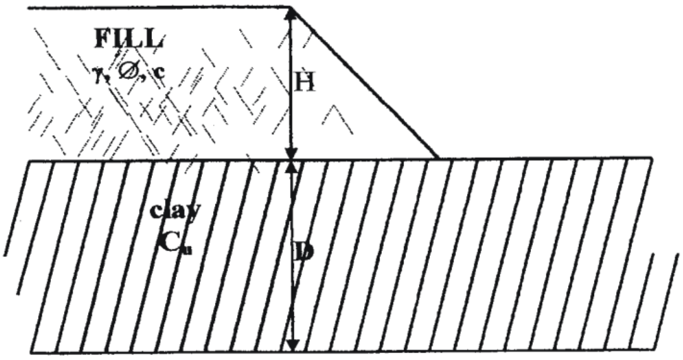

Fig 1: Ordinary cross-section used by stability charts with a constant C_u. Note: $P_{ult} = \gamma\ H_{failure} = N_{Øo}C_u$; $N_{Øo} = f(D/H, \beta, _{Ø}, c)$

Both equations (1) and (2) are quite impractical since C_u and $\sigma_{p'}$ are variable within the clay layer. Equation (1) is relevant to failure circles within a layer having a constant C_u. It does not involve any stability analysis. The stability number $N_{Øo}$ depends on the embankment characteristics and the thickness of the clay layer which is schematized by a mean C_u. Usually the choice of such a mean value is difficult because it depends on the geometry of the problem. Consequently, in current practice, equation (1) is only used as a rule of thumb in order to get a first evaluation of an embankment's stability and thus a first choice of the external slope.

Equation (2) was given an expression similar to equation (1), with a mean $\sigma_{p'}$ that is probably as difficult to evaluate as the mean C_u. However, it is possible to compare the two similar equations in order to get a correction factor defined by:

Correction factor= $N_{Øo}\overline{C_u}/N_{Øo}\overline{\sigma_{p'}}$……………… (3)

which may be applied to the safety factor obtained from a conventional total stress analysis; where the new function may be taken empirically as a constant, i.e. $N_{Øo'}$ = constant = 1.4. Consequently the method may be described by the four following steps.

1) Calculation of conventional safety factor by the usual total stress method.
2) Calculation of $\overline{C_u}/\overline{\sigma_{p'}}$ for the clay layer.
3) Calculation of $N_Ø o$ from a stability chart.
4) Correction of the conventional safety factor (step 1) by the factor given by (3) $N_{Øo}\ C_u/\ (1.4\ \sigma_{p'}$

It must be realized that the mean values, C_u and σ_p, of equations (1) and (2) are never calculated in this method, which makes only a comparison between these two similar equations.

3.0 VALIDITY OF THE PROPOSED METHOD

1. Examples using available consolidation data

In order to illustrate and validate the method, three examples are given below for which consolidation data are available, and consequently the proposed method applies directly; they are:

a) Saint-Alban

La Rochelle et al. (1974) have reported a test embankment failure on Champlain clay. A mean value of 0.310 was considered, with the $N_{Øo}$ value of 5.49 (Ø=44°) obtained from stability charts by pilot and moreau (1973). Consequently the correction factor is equal to 0.31 x 5.49/1.4= 1.22, compared with a correction of 1.0 by Bjerrum's method. Considering the vane test results for the clay and Φ= 44° for the fill, the author found a safety factor of 1.20. Thus the corrected safety factor is 0.99.

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b) Bangkok A and B

Eide and Holmberg (1972) have reported the failure of embankments on a Bangkok clay of high plasticity. The ratio of C_{uv} to the effective vertical stress $\sigma_v ó$ is 0.58, where as $\sigma\acute{\rho}/\sigma vo$ is in the range of 1.5 -1.7. Considering a mean value of 1.6, the C_{uv}/σ_p' ratio is 0.363. The $N_{Øo}$ value is 5.52 for embankment A, and 6.27 for embankment B with berms. The correction factors are 1.43 and 1.63 respectively for embankments A and B, as compared with the single value of 1.55 by Bjerrum's correction. The safety factors computed by the authors are 1.46 and 1.61. The corrected factors are 1.02 and 0.99 respectively

c) Cubac-Ies-Ponts

Blondeau et al (1977) have reported an embankment test failure on clay of high plasticity (Ip=54%). The measured ratio of $C_{uv}/\sigma_{\acute{p}}$ is 0.38 ($C_{uv =}$ 19-20kPa and $\sigma_{\acute{p}}$ = 50-52kPa), with a $N_\varphi 0$ value of 5.42 from stability charts, thus leading to a correction factor of 1.47 compared with 1.29 by the Bjerrum method. The safety factor computed by the authors was 1.48. The corrected FS is 1.01

2. Examples not using consolidation data

Most papers relating an embankment failure have no consolidation data and the proposed method is not directly applicable. For such cases, the only way to proceed is to assume some statistic relationship between $C_u/\sigma_{p'}$ and the plasticity index, Ip, such as the one proposed by Skempton (1948) for normally consolidation clays,

$C_u/\sigma_{\acute{p}}$ = 0.11 + 0.037 Ip -------------------- (4)

Or the one proposed by Osterman (1959) for marine clays (Lame and Whitman 1969, Fig. 29.19),

$C_u/\sigma_{\acute{p}} = 0.14 + 0.030\ I_p$ ---------------------(5)

In stability charts it is found that, for a slope between 2 vert: 3 horiz and I vert: 3 horiz, an embankment with Ø between 30 and 40^0 , and an embankment to clay layer thickness ratio greater than 1.0 (see Fig. 1), the stability number $N_\varphi 0$ ranges between 5.4 and 6.6 with a mean value of 6.0. The resulting correction factors are:

Correction = 0.471 + 0.0159 lp – (6) for equation (4), and

Correction = 0.600 + 0.0129 lp – (7) for equation (5)

These proposed corrections are in surprisingly good agreement with otoko (1987) corrections:

Correction = 0.600.+ 0.0167 Ip

Both proposed corrections (equations (6) and (7) are further plotted against otoko (1987) line

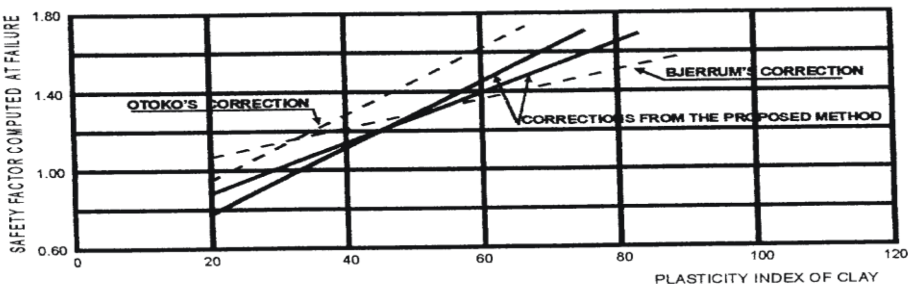

Fig. 2 FACTOR OF SAFETY AT FAILURE VERSUS PLASTICITY INDEX OF CLAY
Bjerrum's correction, Otoko's correction and corrections derived from the proposed method

and Bjerrum (1972) line in fig 2. The two proposed lines are also found to have good agreement with bjerrum's correction line (derived in total stress terms) and Otoko's correction line (derived in effective stress terms).

To further validate the new method, it was used to do comparative stability analysis to obtain the critical height of a vertical cut in deltaic clay of the Niger delta, Nigeria. Data from geomorphological zone 1 was used as typical (see fig. 3 and table 1). The new method gave better results than the other methods. While Taylor's chart

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gaveresults that are approximately equal to the limit equilibrium/upper bound method, the new method gives results between the lower bound and upper bound limit analysis, which approximates to:

$$H_{eretical} = \frac{3Cu}{\gamma_s}$$

As compared to: $H_{eretical}$ (Lower bound) $= \frac{2Cu}{\gamma_s}$

And $H_{eretical}$ (Upper bound) $= \frac{4Cu}{\gamma_s}$

Where Cu = the undrained shear strength Of the clay
γ_s = the unit weight of the clay

This is considered to be a great advantage over the other methods, as the new method is safer than the lower bound limit analysis and more cost effective than the upper bound limit analysis.

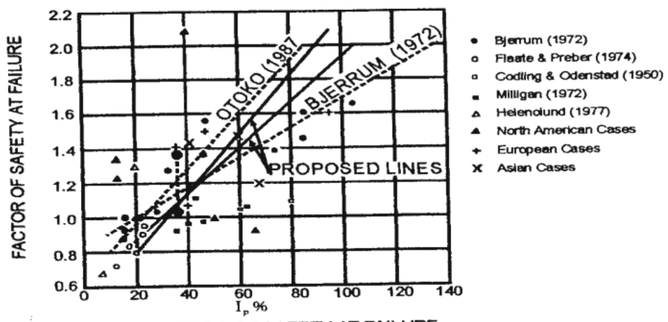

FIG. 4 FACTORS OF SAFETY AT FAILURE FROM ALL PUBLISHED CASE HISTORIES (After Tavenas and Leroueil, 1980)

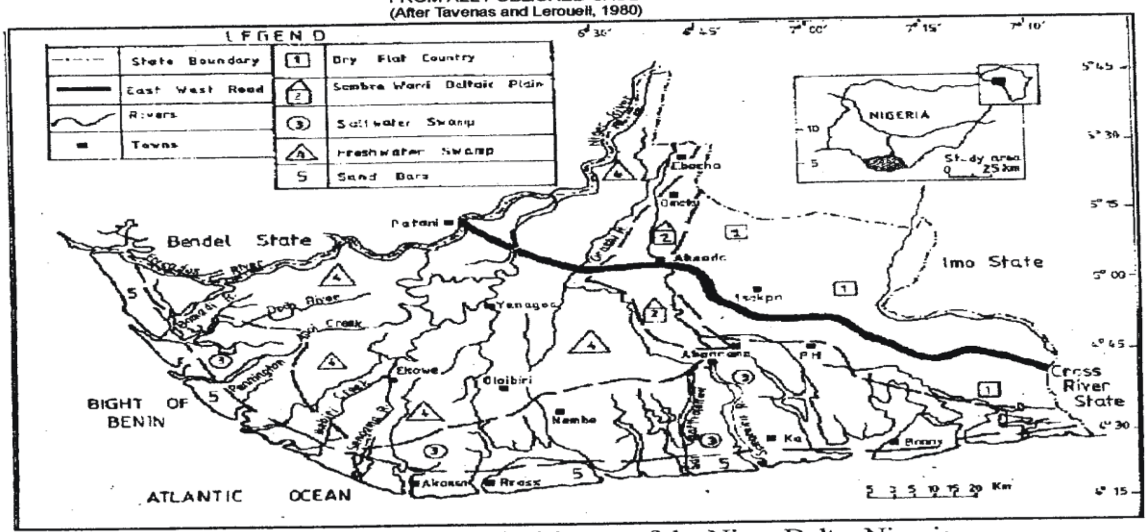

Fig. 3 Geomorphological Zones of the Niger Delta, Nigeria

Finally, to further validate the proposed new method, all available case histories of slopes failures have been compiled in fig. 4. as of 1979 (After Tavenas and Leroueil, 1980). The new method is seen to have good agreement with Bjerrum's (1972) line and Otoko's 1987 line; and the four lines are found to be in the middle of the data points, although the scatter of the individual data is far from negligible.

4.0 DISCUSSION AND CONCLUSION

Then proposed stability method makes use of a conventional total stress analysis corrected to take account of the σ_p' and Cu profiles. Good results are obtained for the three well-documented cases of embankment failures that allow a direct application of the methods. Bjerrum's correction did not give always good results for these three cases. For many other failure cases, the published data are insufficient to allow such a direct application. These cases are statistically analyzed with crude assumptions about the relationship between Cu/ .σ_p' and Ip (fig. 3). Then the Bjerrum's correction and Otoko's correction appear as particular cases of the proposed method and can only be used statistically.

Although the method appears more successful than Bjerrum's method for the three well documented cases, and more successful than other methods in Table 1, it must certainly have some limitations. In its present form, it could not apply to varved, sandy, or organic clays.

TABLE 1: DETAILS OF COMPARATIVE SLOPE STABILITY ANALYSIS

Site	Average Bulk unit weight (kN/m³)	Average plasticity index (%)	Average strength parameters		Critical slope Height Hc (Taylor's chart) (m)	Critical Slope Height Hc (Limit equilibrium /upper bound) (m)	Critical slope Height Hc (Lower bound) (m)	Critical slope Height Hc (New method) (m)
			Cu (kN/m^2)	Øu (deg)				
Dere	18.2	14.4	43	6	10.0	10.5	5.2	7.9
Egbeda	16.5	14.4	44	5	11.1	11.6	5.8	8.7
Ibaa	16.7	16.7	47	5	11.7	12.3	6.1	9.5
Igwuruta	17.1	15.2	50	7	12.6	13.2	6.6	10.0
Korokoro	17.2	13.3	47	4	11.1	11.7	5.9	8.6
Kpite	17.6	15.3	39	7	9.6	10.0	5.0	7.7
Ndashi	18.0	15.1	45	10	11.4	11.9	6.0	9.1
Obio,	16.2	16.5	47	4	11.8	12.4	6.2	9.6
Rumuol-umeni	17.4	15.8	49	5	11.7	12.3	6.1	9.4
Umue-chem	18.0	15.9	40	15	11.1	11.6	5.8	8.9

In spite of a first impression of empiricism, the method agrees well with fill behaviour observations and effective stress concepts, and it appears well suited for the first loading of an embankment. Furthermore, the method appears especially interesting because of two advantages: it is independent of the type of test that established the Cu(z) profile, and it is not related to a single clay parameter as is the correction of Bjerrum.

In conclusion, it must be emphasized that the purpose of this paper is not only to propose a method but also to make geotechnical practitioners realize that conventional stability analyses make use of stresses (local mathematical tools) or elementary forces, whereas a failure is identified by abnormal displacement (mass physical properties) which are not considered in these analyses.

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