STABILITY OF EMBANKMENTS IN THE YENAGOA AREA OF THE NIGER DELTA

BY

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ABSTRACT: This paper presents case records of embankment failures, while the computed safety factors were higher than unity. A modified method of stability analysis is therefore adopted in a programme of research into the problem of embankment stability in the Yenagoa area of the Niger Delta. Good results are obtained with the modified method. A brief outline is also given of some construction expedients for countering problems of settlement and stability.

1.0 INTRODUCTION

Although traditionally road builders have always tried to keep the line of the highway on well-drained stable ground, many constraints now restrict the freedom of choice of Engineers. As a result, highway Engineers are often faced with the necessity of having to site lengths of major new road on poor ground where deposits of soft fine-grained or organic soils exist to considerable depths.

Difficult subsoil conditions are frequently encountered near major rivers or estuaries where recent deposits of alluvial soils are common. However, soft clays and silt deposits may be encountered in many areas of the Niger Delta Nigeria, particularly in the freshwater swamp (fig. 1) During the past two decades, about one quarter of the major motorway schemes in the Niger Delta involved the construction of significant lengths of road over compressible soils. Serious problems of settlement and instability have had to be overcome, necessitating considerable expenditure in some cases. More roads are still expected to be built in the Yenagoa area of the fresh water swamp, being the capital of the newly created Bayelsa State, Nigeria.

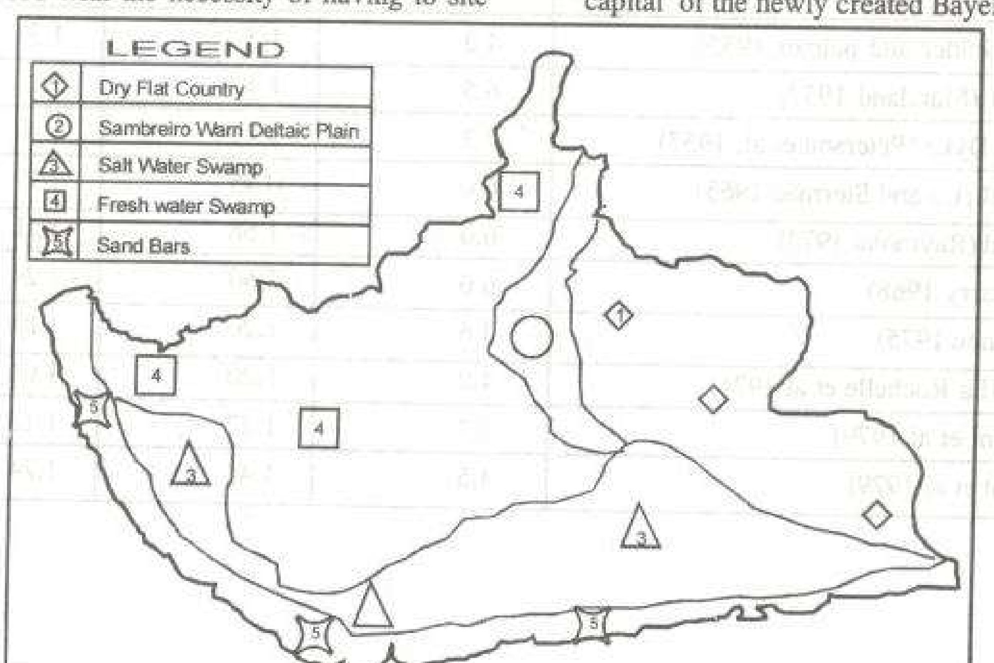

Fig. 1: Geomorphological Zones of the Niger Delta, Nigeria

During the design stage of a major road scheme where compressible soils are likely to be encountered in the subsoil the engineer needs to have reliable information on the likely magnitude and rate of settlement of the embankments and on the possible development of instability in the subsoil arising from stresses produced by the weight of the embankments. Armed with this information the engineer can take steps to minimize the effects of the problems. Unreliable information can lead to the adoption of unnecessary and costly expedients or to large claims from contractors who have to cope with unexpected difficulties during the construction work.

Data on nation-wide or on Niger Delta geotechnical investigation works in respect of soil mapping for engineering purposes are rather scanty. The closest available data for Rivers and Bayelsa States in this regard are the geological survey works of the country and particularly the work of Madedor et al (1987) who examined the engineering properties of the main subgrades of Rivers and Bayelsa States and presented the results in the form of a simple engineering geological map for use in preliminary engineering planning for route location and pavement design of highways. Data on the problems posed by road construction on compressible soils are rather scanty. Improved procedures for the prediction of settlement, and the control of instability is needed. A considerable amount of factual information need to be obtained from field studies, including the value of some practical techniques for accelerating or minimizing settlement and of controlling the rate of construction to avoid the occurrence of instability. This paper discusses the problems generally in the light of case records of embankment failures and field work in the Yenagoa area of the Niger Delta.

2.0 Embankment Slope Stability

Table 1: presents case records in which the calculated safety factors at failure, by both total and effective stress methods have been found to be considerably higher than unity. This suggests that the computed safety factors are affected by some factors.

Table 1: Case records of embankment failures

Embankment and references	Failure Height (m)	Calculated safety factor at failure	
		Total stress method	Effective stress method
Scrapegate (Golder and palmer 1955)	4.3	1.3	1.3
Thames Bank (Marsland 1957)	6.5	1.4	1.2
Seven sisters Dyke (Peterson et al, 1957)	7.3	1.3	1.4
New Liskeard (Lo and Stermac 1965)	6.0	0.95	1.2
New Liskeard (Raymond 1973)	6.0	1.58	1.21
Scottsdale (Parry 1968)	6.0	1.60	1.24
Bangkok (Jambu 1975)	3.6	1.20	1.10
Saint-Alban (La Rochelle et al 1974)	4.0	1.20	1.04
Lanester (Pilot et al 1979)	3.7	1.27	1.13
Cubzac (Pilot et al 1979)	4.5	1.44	1.24

TABLE 2
DETAILS OF SLOPE STABILITY ANALYSIS (COMPARATIVE ANALYSIS FOR 90° SLOPE ANGLE ONLY)

BOREHOLE NO.	Subsoil Profile	BULK UNIT WEIGHT γ (kN/m^2)	UNDRAINED COHESION Cu (kN/m^2)	FRICTION ANGLE $\emptyset u$ (DEGREES)	Critical slope Height Hc (Taylor's Chart)	Critical slope Height Hc (Limit equilibrium /Upper bound) $= \frac{4Cu}{\gamma}$	Critical slope Height Hc (Lower bound) $= \frac{2Cu}{\gamma}$	Critical slope Height Hc (Otoko 1997 method) $= \frac{3Cu}{\gamma}$
	Firm to stiff greyish brown organic fibrous silty peaty clay	13.5	20	6				
1.	Loose brown fine sand	17.7	-	-	5.8	6.0	30	4.5
	Firm brownish grey, mottled brown, silty peaty clay with mica	14.6	22	4				
	Medium dense brownish grey fine sand	20.5	-	-				
	Firm greyish brown, silty peaty clay	13.7	21	5				
	Firm to stiff dark grey silty clay with mica	14.0	23	3	6.0	6.2	3.1	4.7
2	Soft to firm dark grey silty peaty clay	13.2	20	5				
	Medium dense grey fine to medium sand	20.3	-	-				
	Firm grey, mottled brown silty peaty clay	13.9	23	4				
	Firm brownish grey, mottled brown, silty peaty clay	14.1	22	4	5.9	6.1	3.1	4.6
3.	Soft to firm greyish brown silty peaty clay	14.4	21	5				
	Soft to firm dark grey silty clay with mica	13.8	20	6				
	Firm brownish grey, mottled brown, silty peaty clay	13.4	23	4				
	Soft to firm greyish brown silty peaty clay with mica	14.3	20	5	6.0	6.2	3.1	4.7
4.	Soft to firm dark grey silty peaty clay with mica	13.5	21	5				
	Medium dense grey fine to medium sand	20.1	-	-				
	Firm brownish grey mottled brown silty peaty clay	13.8	21	4				
	Firm grayish brown silty peaty clay with mica	14.1	23	4	5.5	5.7	2.9	4.3
5	Soft to firm brown clay with mica	16.7	20	6				
	Medium sense brown fine sand with mica	21.5	-	-				
	Firm brownish grey, mottled brown, silty peaty clay	14.0	22	4				
	Firm greyish brown silty peaty clay with mica	14.5	23	3	5.5	5.8	2.9	4.4
6	Soft to firm brown silty clay with mica	16.9	21	5				
	Medium dense brown fine sand with mica	20.8	-	-				

The many factors that control slope stability have been described in reports by Parry (1972), Bjerrum (1973) and Otoko (1987), some of which are:

(a) **Increase of Shear Strength with Depth**
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 Kenney (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/s_p, or perhaps more accurately by DC_u/Ds_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 overconsolidation ratio s_{Pc}¢/ s_{Po}¢. Brown (1969) and Mesri (1975) proposed that normalization in some clays is more meaningful if expressed as C_u/s_{Pc}¢ rather than C_u/s_{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 be 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 $Æ_u=O$ analysis of embankments should be reduced by an empirical correction factor m to bring them closer to the strengths 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).

(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 localised 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, Landva 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 1997). 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; Jakobson 1946).

In 1972 and 1973 Bjerrum proposed new general correction factors for the undrained shear strength.

C_u, that depends on the plasticity index (I_p) 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 Æ$_u$ = 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. All the same, stability calculations presented in tables 2 & 3 have been done using (Otoko 1997) method, which makes use of a conventional total stress analysis, corrected to take into consideration the preconsolidation pressure and the undrained shear strength profiles. This method is related to both total and effective stress concepts. Bjerrum's correction for total stress analysis and Otoko's (1987) correction for effective stress analysis appear as particular cases only statistically applicable, of the new method.

3.0 Analysis of Laboratory Test Results.

Six boreholes were sunk at Yenagoa by percussion drilling not less than 10m. disturbed and undisturbed samples were carried out in accordance with BS 1377 (1990.)

The programme of research into the problem of embankment stability in the fresh water swamp of the Niger Delta, is in two phases namely: (1) The problem of predicting the magnitude and rate of settlement of embankments constructed on compressible subsoils and (2) the problem of predicting the possible development of instability in the subsoil arising from the stresses produced by the weight of the embankments. It is the latter that is contained in this paper. Comparative slope stability analysis for 90° slope were first carried out. (Table 2). The table shows that the Otoko (1997), given by 3Cu/g (see Otoko, 1997), gave better results than the limit equilibrium method, Taylor's chart, and upper and lower bound limit analysis. Therefore, the Otoko (1997) method was used to compute critical slope heights for various slope angles and depth factors shown in Table 3, and which is expected would be used for preliminary planning work and provide control data for subsequent detailed fieldwork in the study area.

Significant flooding is not expected in Yenagoa (Bayelsa State capital). However, in the event of flooding, the external water force is taken into account in the calculations, by replacing the bulk unit weight g, by the submerged unit weight g¢, within the soil mass below the level of the external water.

4.0 Construction expedient for countering problems of settlements and stability.

Frequently even a realistic assessment of the settlement of an embankment may indicate

Table 3: Average critical heights (Hc) of embankments for various slope angles and depth factors

Slope angle (degrees) / Depth Factor	75	15	22.5	30	45	53	60	75	90
	Hc (m)	Hc (m)	Hc (m)	Hc (m)	Hc (m)	Hc (m)	Hc (m)	Hc (m)	Hc (m)
1	45.5	15.0	12.3	11.0	8.7	7.8	7.1	5.9	4.0
2	11.0	7.9	7.1	6.9	6.6	6.4	-	-	-
3	8.4	7.0	6.7	6.6	6.5	6.4	-	-	-
4	7.6	6.8	6.7	6.6	6.5	6.4	-	-	-

that the duration of the consolidation process will extend beyond the construction period and that significant movements of the completed road structure are likely to occur. Provided that these movements are relatively uniform over the length of the road its riding quality will not be adversely affected and preventative action will be unnecessary. However, if significant differential settlement are expected after completion of the road pavement, say in excess of 25mm over a 10m length, an alternative construction technique will usually be required to alleviate the problem. The methods which are most commonly adopted are: (a) excavation of the compressible material. (b) reduction in the settlement by using lightweight fill (c) acceleration of the settlement by use of surcharge, vertical or horizontal drains.

Frequently a combination of two or more of these processes may provide the most economic solution for a given situation. For example, the use of light weight fill for the main embankment combined with heavier fill to act as surcharge could lead to a significant reduction in the quantity required of this latter material.

5.0 Conclusions

Road embankments constructed over recent deposits of alluvial soils are likely to be unstable and induce significant settlements even with comparatively low embankments.

From a number of case records in which the calculated safety factors at failure, have been found to be higher than unity, it is clear that stability analyses by both total and effective stress methods are affected by some factors.

In the light of such factors, new method of stability analysis proposed by Otoko (1997) was used in the stability analyses presented in this paper. The new method gave better results than the limit equilibrium method, Taylor's chart, upper and lower bound limit analysis and thus results obtained, could be used for preliminary planning work and provide control data for subsequent detailed fieldwork in the study area. A brief outline is also given of some construction expedients for countering problems of settlement and stability.

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