Three-Dimensional Stability Analyses of Four Embankment Failures

Analyses Tridimensionnelles de la Rupture de Quatre Remblais

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SYNOPSIS This paper presents four case histories of embankments rapidly loaded to failure on saturated clay foundations. Two-and three-dimensional total stress ($c = c_u$ and $\phi = 0$) circular arc stability analyses are performed utilizing two strength models: 1) the (uncorrected) field vane strength, and 2) the Stress History and Normalized Soil Engineering Properties strength after accounting for strain compatibility along the shear surface and strength anisotropy. End effects are shown to increase the conventional two-dimensional factor of safety by as much as 30%, and hence, if neglected, can lead to significant overestimation of backfigured strengths.

INTRODUCTION

Embankments constructed on soft to stiff saturated clay foundations are commonly designed for the end of construction condition using the $\phi = 0$ and $c = c_u$ circular arc method of analysis (Bishop and Bjerrum, 1960). In these analyses, the major uncertainty in evaluating stability is the undrained shear strength of the soil. Based on 16 well-documented embankment failures on cohesive foundations, Bjerrum (1972) introduced the field vane, FV, correction factor, μ, in order to estimate the in situ c_u from field vane measurements. Ladd et al. (1977) present results from additional embankment failures after 1972 and conclude that the data are distributed fairly evenly about Bjerrum's curve and generally fall within ± 20% of his recommended values. Ladd and Foott (1974) propose a design procedure known as the Stress History and Normalized Soil Engineering Properties, SHANSEP, procedure. SHANSEP attempts to minimize the influence of sample disturbance and explicitly considers the effects of strength anisotropy in evaluating the stability of clay foundations.

The uncorrected field vane and the SHANSEP strength models are used herein to analyze the stability of four embankment failures taking into consideration:

(1) End Effects. Circular arc stability analyses assume that failures have infinite length along the embankment, whereas actual slope failures take place along a finite length and hence involve end effects. Baligh and Azzouz (1975) developed a systematic three-dimensional (3-D) method to include end effects on the stability of slopes. The method considers shear surfaces of revolution consisting of cylinders and ellipsoids and/or cones instead of the infinite cylinders assumed in two-dimensional (2-D) circular arc analyses.

(2) Strength Anisotropy and Strain Compatibility The undrained stress-strain-strength behavior of a given saturated clay sample depends on the mode of shearing, e.g., compression, extension and direct simple shear stress systems. Furthermore, the peak strength is generally reached at different strain levels. A comprehensive stability analysis must therefore account for strength anisotropy and strain incompatibility of different soil elements along the shear surface. Davis and Christian (1971) propose an elliptical two-dimensional model describing strength anisotropy of clays. The model is extended to three dimensions (Azzouz and Baligh, 1978) and used herein to represent the undrained shear strength, defined as the shear stress on the failure plane at failure, i.e., $c_u = \tau_{ff}$, after adjustments for strain compatibility according to the simplified approach proposed by Ladd (1975).

CASE STUDIES

Four case histories of embankments on saturated clay foundations are analyzed using the computer program STAB3D (Azzouz and Baligh, 1978) based on the two strength models described previously. Figure 1 shows the I-95 embankment constructed over a lean, medium to stiff deposit of Boston Blue Clay, BBC. One of the many surprising features of the I-95 case study was the length of failure (in plan) which extended over a distance of 1030 ft, while the loaded length was 300 ft (M.I.T., 1975). An estimate of the end effects was thus necessary in order to backfigure the shearing strength of BBC. Figure 1 also shows the location of the predicted shear surface in relation to the locations of the ruptured inclinometers and the extent of failure (in plan). As mentioned previously, stability analyses performed according to the SHANSEP strength model take into consideration the

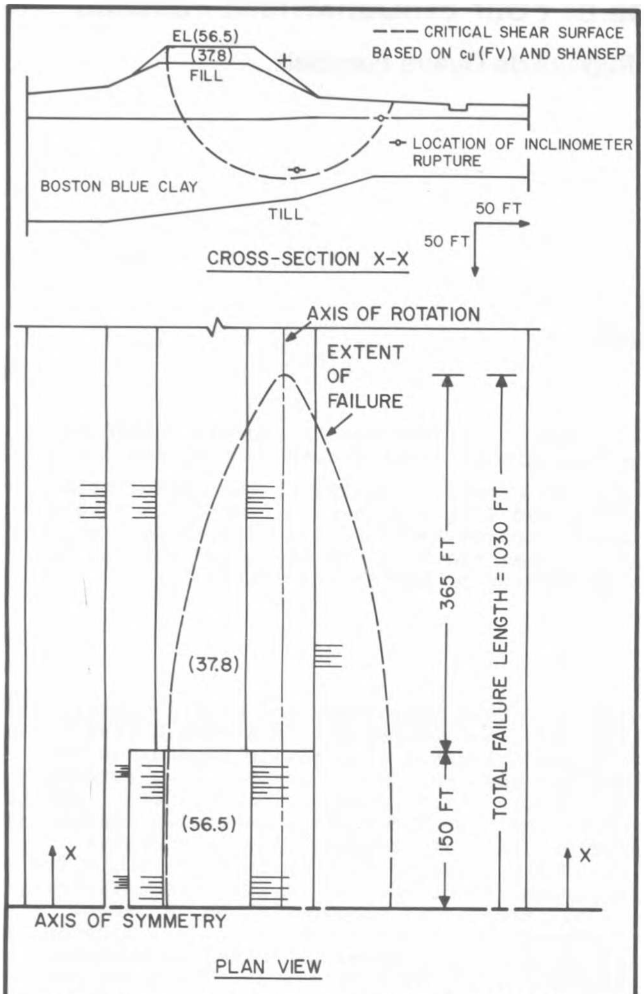

FIGURE I. I-95 EMBANKMENT FAILURE, MASSACHUSETTS

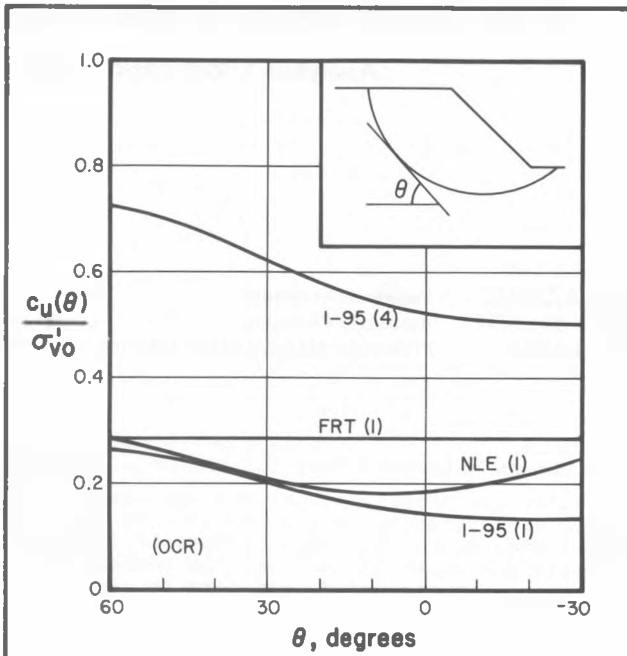

FIGURE 2. STRENGTH VARIATION ALONG SHEAR SURFACE

strength anisotropy along the shear surface. Figure 2 shows the variation of the undrained shear strength, normalized with respect to the effective overburden stress, σ'_{vo}, with the failure surface inclination, θ , for representative OCR values for three of the four clay types considered in this study. In order to obtain this distribution, the angle between the failure plane and the major principal plane was assumed equal to $45° + \phi'/2 = 60°$. Hence when $\theta = 60°$, $c_u(\theta)/\sigma'_{vo}$ corresponds to $c_u(V)/\sigma'_{vo}$ obtained from compression tests and when $\theta = -30°$, $c_u(\theta)/\sigma'_{vo}$ corresponds to $c_u(H)/\sigma'_{vo}$ obtained from extension tests. Figures 3,4 and 5 show cross sections of the predicted shear surfaces at the remaining three case studies. Figure 3 corresponds to the Experimental Test Section, ETS, case study in New Hampshire, where a granular embankment was purposely constructed to fail a lean, medium to soft very sensitive Portsmouth Clay. The case study is described in detail in Ladd (1972). The SHANSEP strength for the Portsmouth Clay was computed based on the maximum value of the maximum past pressure, σ'_{vm}, given by Ladd (1972) since it closely represents the in situ σ'_{vm} (Ladd, 1972). The variation of c_u with θ for the ETS case was very similar to that of BBC. Figure 4 presents the New Liskeard Embankment, NLE, failure in Canada, where the medium to soft New Liskeard Varved Clay, NLVC, deposit failed during construction of a granular embankment (Lacasse et al., 1977). The stability analyses described herein for the NLE based on the SHANSEP strength model were computed according to the minimum σ'_{vm} profile given by Lacasse et al. (1977). Unfortunately, no data are available on the strength anisotropy for the NLVC (only c_u measured by the direct simple shear test, $c_u(DSS)$). Hence, the c_u-θ variation in Fig. 2 was determined from test data on the noncemented Connecticut Valley Varved Clay reported by Ladd (1975). This was achieved by assuming that the ratios $c_u(V)/c_u(DSS)$ and $c_u(H)/c_u(DSS)$ are the same for both types of clays. Figure 5 illustrates the test embankment at the Fore River Test Section, FRT, constructed over a tidal mudflat deposit of soft, slightly organic Maine Clay (Ladd et al., 1969). The SHANSEP strength used in the analysis was assumed constant and equal to $c_u(DSS)$.

Finally, Table 1 presents a summary of the case studies considered herein. It should be noted that the end effects, expressed as the ratio of the three-dimensional to two-dimensional factors of safety, $(F/F°)$, were all computed for the observed failure length. The critical shear surfaces predicted according to the $c_u(FV)$ and SHANSEP models are practically the same for all cases.

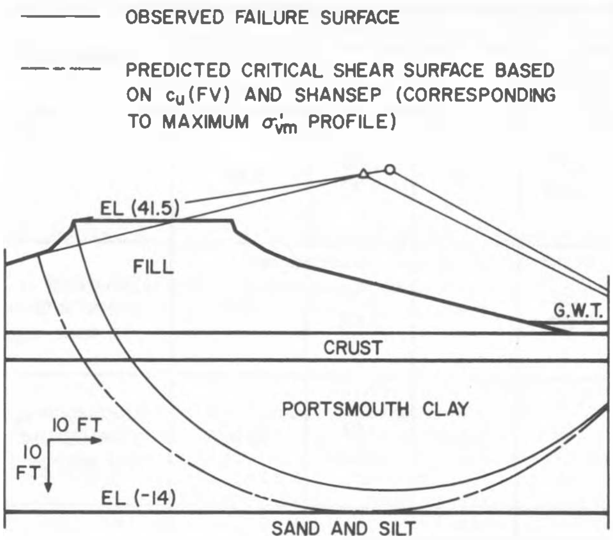

FIGURE 3. FAILURE OF AN EXPERIMENTAL TEST SECTION, NEW HAMPSHIRE

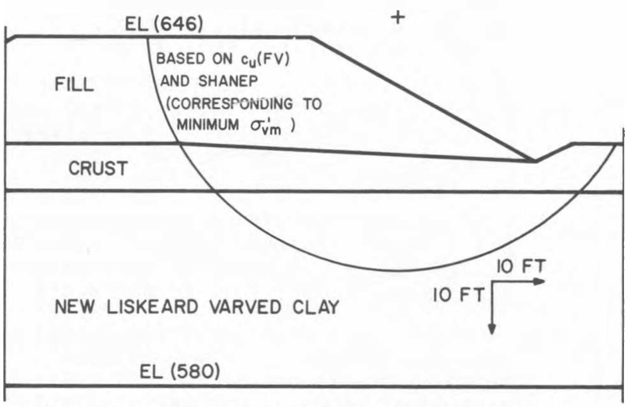

FIGURE 4. NEW LISKEARD EMBANKMENT FAILURE, ONTARIO

RESULTS

1. The end effects, F/F°, increase the plane strain factor of safety, F°, by a maximum of 30% (I-95 case) and a minimum of 7% (FRT and NLE cases). Therefore, when three-dimensional effects are neglected in the analysis of test fill failures, the back-calculated in situ shear strength will be overestimated, and hence will lead to unsafe designs of long embankments. In particular, this applies to FV correction factors, μ , proposed by Bjerrum, (1972).

2. Three-dimensional analyses based on the SHANSEP strength model, which accounts for strength anisotropy and strain compatibility along the shear surface of revolution, yielded

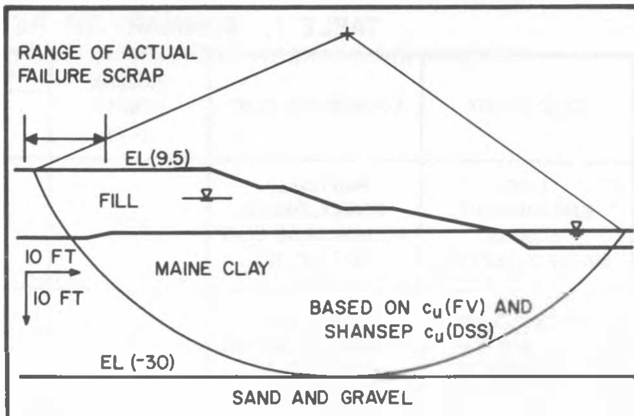

FIGURE 5. FAILURE OF FORE RIVER TEST SECTION, MAINE

satisfactory results, i.e., F close to unity at failure, except for the FRT case.

3. Three-dimensional analyses based on the uncorrected field vane strength, c_u(FV) provide reasonable predictions (within 15%, say) in two cases (I-95 and ETS), but overestimate the safety in the other two cases.

4. Strength anisotropy can be accounted for in plane strain circular arc stability analyses by using an equivalent strength, c_u(equ), given by:

$$c_u(\text{equ}) = \alpha[c_u(V) + c_u(H)]$$

For nonlayered soils, e.g., Boston Blue Clay and Portsmouth Clay, α was found equal to 0.45, whereas for the New Liskeard Varved Clay, α = 0.40. However, it should be noted that for varved clays, the strength is at or near its minimum for horizontal failures. Hence, the use of [c_u(V) + c_u(H)] to account for strength anisotropy in noncircular (wedge type) shear surfaces, where a large portion of the failure surface is horizontal, may be unsafe.

5. End effects, F/F°, were not significantly affected by the strength model used, i.e., FV vs. SHANSEP. This means that for routine investigations, the FV test can be used to evaluate the likely magnitude of 3-D effects.

REFERENCES:

NOTE: ASCE = American Society of Civil Engineer

CGJ = Canadian Geotechnical Journal

ICSMFE = International Conf. on Soil Mechanics and Foundation Engineering

JGED = Journal of the Geotechnical Engineering Division

JSMFD = Journal of the Soil Mechanics and Foundation Division

TABLE I. SUMMARY OF RESULTS FROM CASE STUDIES

CASE STUDY	FOUNDATION CLAY	FAILURE LENGTH (FT)	BASED ON c_u(FV)		BASED ON SHANSEP		FV CORRECTION FACTOR	REMARKS
			F°	F (F/F°)	F°	F (F/F°)	$\mu = 1/F(FV)$	
1. I-95 EMBANKMENT, SAUGUS MASSACHUSETTS	PARTIALLY CONSOLIDATED BOSTON BLUE CLAY (P.I.=21%)	1030	0.89	1.17 (1.30)	0.82	1.06 (1.30)	0.86	
2. EXPERIMENTAL TEST SECTION, PORTSMOUTH NEW HAMPSHIRE	SOFT, VERY SENSITIVE MARINE ILLITIC CLAY (P.I.=15%)	300	0.91	1.00 (1.10)	0.94	1.03 (1.10)	1.00	SHANSEP c_u CORRESPONDS TO MAX. σ'_{vm}
3. NEW LISKEARD EMBANKMENT, NEW LISKEARD ONTARIO	SOFT TO MEDIUM STIFF VARVED CLAY (P.I.=10-47%)	325	1.16	1.24 (1.07)	0.94	1.03 (1.10)	0.81	SHANSEP c_u CORRESPONDS TO MIN. σ'_{vm}
4. FORE RIVER TEST SECTION, PORTLAND MAINE	SOFT, SILTY SLIGHTLY ORGANIC CLAY (P.I.=34%)	240	1.92	2.05 (1.07)	0.80	0.86 (1.07)	0.49	

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