EVALUATION OF THE MOBILIZED SHEAR STRENGTH OF SOFT CLAYS UNDER DIFFERENT LOADING AND UNLOADING CONDITIONS

by

T. M. Lam

A thesis
presented to the School of Graduate Studies and Research
at the University of Ottawa
in partial fulfillment of the
requirements for the degree of
Master of Applied Sciences
in
Civil Engineering

ABSTRACT

Failures of geotechnical structures located on soft sensitive clays have been analyzed using shear strength values determined from the expression, $c_u=0.22\sigma'_p$. The case studies include an embankment at Bangpli, Thailand, two silos, one at New Liskeard and the other at Vankleek Hill, in Ontario, and an excavation at Welland, also in Ontario. It is shown that this empirical relationhip between the mobilized shear strength and the preconsolidation pressure is representative of the soil at all locations and that it is applicable under both loading and unloading conditions.

The shear strength value obtained by using this expression seems to be more reliable than the value obtained from a vane test which requires a correction factor and arbitrary crust strength assumptions. Furthermore, the only data required for this approach are the preconsolidation pressure values which are routinely determined in geotechnical investigations involving soft sensitive clays.

RÉSUMÉ

La stabilité de structures géotechniques dans des dépôts d'argiles molles sensibles a été analysée en utilisant les valeurs de résistance au cisaillement déterminée par l'expression $c_u = 0.22\sigma'_p$. L'étude porte sur un remblai à Bangpli en Thaïlande, sur deux silos, un situé à New Liskeard et l'autre à Vankleek Hill, en Ontario et sur une excavation à Welland, également situés en Ontario. Il est démontré que cette relation empirique entre la résistance au cisaillement non drainé et la pression de préconsolidation est valide pour tous les dépôts d'argile étudiés et qu'elle est applicable dans des conditions de chargement et de déchargement.

La valeur de la résistance ~~au~~ cisaillement obtenue en utilisant cette expression semble être plus fiable que celle obtenue au scissomètre et qui nécessite l'utilisation d'un coefficient correcteur et une évaluation arbitraire de la résistance de la croûte argileuse. De plus, les seules données requises pour l'application de cette méthode sont les valeurs de pression de préconsolidation, qui sont d'ailleurs déterminées de façon systématique lors de toute reconnaissance géotechnique dans les dépôts argileux.

ACKNOWLEDGEMENTS

The author is grateful to all those who have contributed directly and indirectly to the completion of this project. Especially:

Dr. B. Trak, the thesis advisor, for his constructive criticism and direction during the entire project;

Mr. D. D. Brown, Mr. H. M. Morgado and Mr. B. Morse for their invaluable help in reviewing this thesis before its final submission;

TABLE OF CONTENTS

ABSTRACT ii

RESUME iii

ACKNOWLEDGEMENTS iv

TABLE OF CONTENTS v

LIST OF FIGURES ix

LIST OF TABLES xiii

NOTATION xiv

Chapter Page

I. INTRODUCTION 1

1.1 Statement of the problem 1

1.2 Objective of the study 2

1.3 Thesis overview 5

II. LITERATURE REVIEW 9

2.1 General 9

2.2 Contribution by Bjerrum (1972,1973) 9

2.1.1 Description 9

2.1.2 Bjerrum's modified method 11

2.1.3 Time effect 13

2.1.4 Effect of anisotropy 14

2.3 Contribution by Pilot (1972) 15

2.4 Contribution by La Rochelle et al. (1974) . . . 19

2.5 Contribution by Ladd and Foott (1974) 21

2.5.1 Description 21

2.5.2 Design procedure 23

2.6 Contribution by Aas (1976(a) and (b)) 25

2.7 Recent contributions 27

2.8 General discussion 28

2.8.1 Bjerrum's method 28

2.8.2 Pilot's method 29

2.8.3 USALS analysis 30

2.8.4 SHANSEP method 31

2.8.5 ADP method 32

2.5 Conclusion 33

III. METHODS OF ANALYSIS 50

3.1 General 50

3.2 $\phi=0°$ analysis 50

3.3 Analyses of slope stability based on the limit equilibrium method 51

3.3.1 Description 51

3.3.2 Bishop's method 53

3.4 Bearing capacity 54

3.4.1 Description 54

3.4.2 Slip surface method 56

3.4.3 Methods based on plastic failure theory . 58

3.5 Stability of vertical cuts 61

3.6 Factor of safety 63

3.7 Discussion 65

IV. CASE HISTORIES 74

4.1 General 74

4.2 Embankment failure 75

4.2.1 Embankment on Bangkok clay 75

4.2.1.1 Geological and geotechnical descriptions 76
4.2.1.2 Undrained shear strength 77
4.2.1.3 Preconsolidation pressure 78
4.2.1.4 Fill 78
4.2.1.5 Stability analyses 79

4.3 Bearing capacity failures 82

4.3.1 New Liskeard (silo) 82
4.3.1.1 Geological and geotechnical descriptions 83
4.3.1.2 Structure 84
4.3.1.3 Failure 84
4.3.1.4 Undrained shear strength profiles 85
4.3.1.5 Stability analyses 86

4.3.2 Vanklek Hill (silo) 89
4.3.2.1 Geological and geotechnical descriptions 89
4.3.2.2 Structure 90
4.3.2.3 Failure 91
4.3.2.4 Undrained shear strength profiles 92
4.3.2.5 Stability analyses 93

4.4 Vertical cut failure 97

4.4.1 Welland canal 97
4.4.1.1 Geological and geotechnical descriptions 97
4.4.1.2 Undrained shear strength and preconsolidation pressure 99
4.4.1.3 Excavation 99
4.4.1.4 Failure 100
4.4.1.5 Stability analyses 101

4.5 Discussion 103

4.5.1 Bangpli embankment 103

4.5.2 Bearing capacity failures 104

4.5.3 Vertical cut at Welland 105

V. CONCLUSIONS 134

5.5 General 134

5.2 Summary of the study 134

5.3 Major conclusions 135

5.4 Recommendations 138

REFERENCES 139

APPENDIX Page

A. THE DETERMINATION OF PRECONSOLIDATION PRESSURE AND ITS RELATION TO THE SENSITIVITY OF SAFETY FACTORS . . 145

A.1 General 145

A.2 Determination of preconsolidation pressure . . 145

A.3 Sensitivity of factor of safety 148

LIST OF FIGURES

Figure		Page
1.1	(a) Computed factors of safety versus plasticity indices of foundation clay (b) Correction factor to be applied to the vane strength of the clay foundation (Bjerrum 1972)	6
1.2	Theoretical factor of safety at failure of footings and loading plates plotted against the plasticity index of the clay (Bjerrum 1973)	7
1.3	Theoretical factor of safety at failure for cuts and unsupported excavations plotted against the plasticity index of the clay (Bjerrum 1973)	8
2.1	Correction factors established for the effect of time and anisotropy (Bjerrum 1973)	38
2.2	Relevance of laboratory shear tests to shear strength in the field (Bjerrum 1972)	39
2.3	Correlation between factor of safety and liquid limit (Pilot 1972)	40
2.4	Correlation between factory of safety and plasticity index (Pilot 1972)	41
2.5	Strength profiles with different crust strength assumptions, St. Alban (La Rochelle et al. 1974)	42
2.6	(a) Triaxial test results for Drammen Clay (Bjerrum and Simons 1960) (b) Normalized plot of triaxial test results, Drammen clay	43
2.7	Basic principles in an ADP analysis (Aas 1976(a))	44

2.8 Correlation between factor of safety, $c_{u(vane)}/\sigma'_{vo}$ and $c_{u(vane)}/c_{u(ADP)}$ (Aas 1976(b)) 45

2.9 Typical ratios for normally consolidated late glacial and postglacial clays (Bjerrum 1973, Mesri 1975) 46

2.10 Similarity of c_u=0.22σ'_p and the undrained strength at large strain (USALS) for different sites of sensitive clay in Quebec (Trak et al. 1980) 47

2.11 Variation of normalized $\overline{CK_oU}$ direct simple shear (DSS) strength parameters with overconsolidation ratios (OCR) for five clays (after Ladd and Foott 1974), and the corresponding ratios of c_u/σ'_p (Trak et al. 1980) 48

2.12 Comparison between Bjerrum's (1972) and Pilot's (1972) vane strength correction factors 49

3.1 Bishop's method (Bishop 1955) 68

3.2 Modes of bearing capacity failure (Sowers 1979) 69

3.3 Method of slip circle analysis 70

3.4 Plastic failure, bearing capacity theory (Terzaghi and Peck 1967) 71

3.5 Critical height method derived from limit equilibrium (Chen and Scawthorn 1970) 72

3.6 Moment equilibrium mechanism (Fellenius 1927) 73

4.1 Soil profile at Bangpli (Brand et al. 1976) 113

4.2 Strength profiles at Bangpli 114

4.3 Slip circle analyses using vane strength profile, Bangpli embankment 115

4.4 Slip circle analyses using vane strength profile with tension crack assumption, Bangpli embankment 116

4.5 Slip circle analyses using c_u=0.22σ'_p, Bangpli embankment 117

4.6 Soil profile at New Liskeard (Eden and Bozozuk 1962) 118

4.7 Strength profiles at New Liskeard 119

4.8 Details of the silo base after failure, New Liskeard (Bozozuk 1972) 120

4.9 Slip circle analysis, New Liskeard silo 121

4.10 Soil profile at Vankleek Hill (Bozozuk 1972) 122

4.11 Attitude of silo foundation after failure, Vankleek Hill silo (Bozozuk 1972) 123

4.12 Strength profiles at Vankleek Hill 124

4.13 Slip circle analysis, Vankleek Hill silo 125

4.14 Soil profile at Welland (Kwan 1971) 126

4.15 Location of tension cracks, Welland vertical cut (Kwan 1971) 127

4.16 Slip circle analysis, Welland vertical cut 128

4.17 Shear strength profiles of Welland clay 129

4.18 $\mu c_u/\sigma'_p$ ratio and OCR profiles at New Liskeard . . 130

4.19 Potential and actual failure surfaces, Welland vertical cut (Kwan 1971) 131

4.20 Generation of cycloidal arc 132

4.21 Comparsion between actual failure surface and surface generated by cycloidal functions 133

A.1 (a) Effect of sample disturbance (b) Effect of load increment ratio (Holtz and Kovacs 1981) 152

A.2 Effect of time on σ'_p in odemeter tests, St. Alban (Tavenas and Leroueil 1977) 153

A.3 Graphical proceedure for determination of preconsolidation pressures (Holtz and Kovacs 1981) 154

A.4 Preconsolidation pressure profiles at New Liskeard 155

A.5 Shear strength profiles determined by $c_u = 0.22\sigma'_p$, New Liskeard 156

LIST OF TABLES

Table		Page
2.1	Comparison between the results of compression and extension tests, direct simple shear tests and in situ vane tests on soft clay (Bjerrum 1972)	36
2.2	Typical values of α_a, α_d, and α_p (Morgenstern et al. 1977)	37
4.1	Computed factors of safety of Bangpli embankment	108
4.2	Computed factors of safety of New Liskeard silo	109
4.3	Computed factors of safety with average shear strength values, Vankleek Hill silo	110
4.4	Computed factors of safety of Vankleek Hill silo	111
4.5	Computed factors of safety of Welland excavation	112
A.1	Computed factors of safety of New Liskeard silo with different $0.22\sigma'_p$ profiles	151

NOTATION

a = intercept of the ordinate (ADP method)

ADP = active, direct shear, and passive analysis (Aas 1976(a) and (b))

b = breadth of foundation (bearing capacity)
= width of slice (Bishop's method)

c' = cohesion intercept

c_u = undrained shear strength

CAU = anisotropically consolidated undrained test

$\overline{CK_oU}$ = K_o-consolidated undrained shear test with measurement of pore pressure

c_{ua}, c_{ud}, c_{up} = undrained shear strength values of active, direct shear, and passive tests (ADP analysis)

D = depth to foundation level

DSS = direct simple shear test

F = factor of safety

FEM = finite element method

H = height of vertical cut or embankment

H_{cr} = critical height of vertical cut

I_p = plasticity index

K_o = coefficient of earth pressure at rest

ℓ = length of the base of a slice

L/A = perimeter/base area ratio (Skempton's bearing capacity)

N_c, N_γ, N_q = bearing capacity factors

NSP = normalized soil parameter (SHANSEP)

OCR = overconsolidation ratio

P = normal reaction force at base of slice

q = bearing pressure

q_u = ultimate bearing capacity

r = radius of slip circle

PSA = plane strain active shear test

PSP = plane strain passive shear test

S = shear stress

TC = triaxial compression test

TE = triaxial extension test

SHANSEP = stress history and normalized soil engineering properties

w_n = natural water content

w_L = liquid limit

w_p = plastic limit

X_n = horizontal interslice force (Bishop's method)

α_a, α_d, α_p = slopes of the shear strength profiles of active, direct shear, and passive tests

μ = Bjerrum's vane strength correction factor

μ_a = Bjerrum's vane strength correction factor for anisotropy

μ_r = Bjerrum's vane strength correction factor for time effect

σ_1 = major principal total stress

σ_3 = minor principal total stress

σ' = effective stress

σ'_c = isotropic effective consolidation pressure (SHANSEP)

σ_n = normal stress

σ'_p = preconsolidation pressure

σ'_{vo} = in situ vertical effective stress

ϕ = angle of internal friction

S = shear strength of soil

γ = bulk unit weight

θ = angle subtended by arc of slip surface
= angle of slip surface measured from horizontal

Chapter I

INTRODUCTION

1.1 STATEMENT OF THE PROBLEM

The stability of embankments on soft soil deposits is a common problem in geotechnical engineering, because soft clays, especially those having high sensitivity, have low shear strength and their behaviour is complex due to such factors as anisotropy, excess pore pressure and rate of loading. These factors arise from different conditions, such as the depositional environment, the stress history, and the variety of geological and geomorphological processes that have acted on the natural materials.

A good part of the earth is covered with these problematic soft soils. They are found in the lowlands of the St. Lawrence River and in the Ottawa river valley in Eastern Canada, along the coast of Norway, Sweden, and Finland, in the deltas of the Nile, Mississippi and Yang-Tze rivers, and in large parts of South-east Asia. These areas are usually highly populated and, therefore, economical design methods are necessary to ensure the integrity of the structures and the safety of the inhabitants.

1.2 OBJECTIVE OF THE STUDY

A method was recently proposed to determine the mobilized shear strength values under embankments in soft clays (Trak et al. 1980). In this method, the strength values are determined by using the expression $c_u = 0.22\sigma'_p$ derived by Mesri (1975). The objective of this study is to examine the applicability of this method to the stability analyses of three types of geotechnical structures: 1) embankments, 2) silos, and 3) excavations.

When the stability analysis of an embankment is undertaken, three different aspects are usually taken into consideration: (1) The numerical models employed to analyze the forces causing and resisting shear failure; (2) the methods employed to determine the mobilized shear strength parameters of the soil in the embankment and of the subsoil; and (3) the practical application of both numerical models and shear strength parameters to predict local and general failures of embankments.

Although the first aspect, i.e. the numerical models of stability analysis, will be briefly discussed in chapter III, this thesis is mainly concerned with aspects (2) and (3) of the stability analysis of embankments, silos and excavations.

The evaluation of mobilized shear strength values of soils and the application of these values in numerical models to predict the performance of the structure under a particular loading condition has been a major area of research in geotechnical engineering. The shear strength values used in a stability analysis can be determined either by laboratory or field tests. The most commonly employed field test to evaluate the shear strength is the in-situ vane test. However, the shear strength measured by the vane test may overestimate the mobilized shear resistance of most clay soils of medium to high plasticity (Bjerrum 1972). Bjerrum showed from case studies of unexpected embankment failures that this overestimation was related to the plasticity index (I_p) and proposed that an empirical "correction factor" ($\mu = f(I_p)$) should be applied to the measured field vane value to obtain a more reliable mobilized shear strength of the soil (Fig. 1.1). The observed overestimation of shear strength based on vane measurements has been confirmed by Pilot (1972) and Dascal et al. (1972) who proposed correction factors similar to μ.

However, a number of researchers have critized the use of "corrected" vane shear strength values because these are based totally on an empirical adjustment which neglects simulation of the actual stress paths and of soil behaviour (Ladd and Foott 1974; Trak 1974; and Morgenstern et al. 1977). Other methods, such as SHANSEP (Ladd and Foott 1974), and ADP methods (Aas 1976(a) and (b)), try to avoid this problem by the use of triaxial and direct shear tests to simulate field loading conditions. Recently, Trak et al.

(1980) successfully used a normalized soil parameter, $c_u/\sigma'_p=0.22$, derived by Mesri (1975) who showed that this mobilized shear strength value under embankments was in fact independent of I_p. Mesri's method combines the laboratory testing (oedometer test) and an empirical approach because the $c_u=0.22\sigma'_p$ expression was obtained by an interpretation of Bjerrum's (1972, 1973) data on actual embankment failures.

In 1973, Bjerrum showed that the vane strength correction based on the plasticity index was also valid for shallow foundation and unsupported cut failures (Fig. 1.2 and 1.3). This suggests that Mesri's expression, $c_u=0.22\sigma'_p$, may also be employed to determine the mobilized shear strength for unsupported cuts and shallow foundations. Values of mobilized shear strength similar to the ones obtained by Mesri's expression were recently used by Aas (1981) in the analysis of landslides in quick, normally consolidated clay in Norway. Trak (1981) suggested that these results may indicate that the $c_u=0.22\sigma'_p$ method could be used in estimating mobilized shear strength values for conditions other than embankment loading. Therefore, it was decided to examine in this thesis the range of applicability of Mesri's expression by analyzing data on the following common types of failure of geotechnical structures:

1. embankment failures,
2. silo failures,
3. cut and excavation failures.

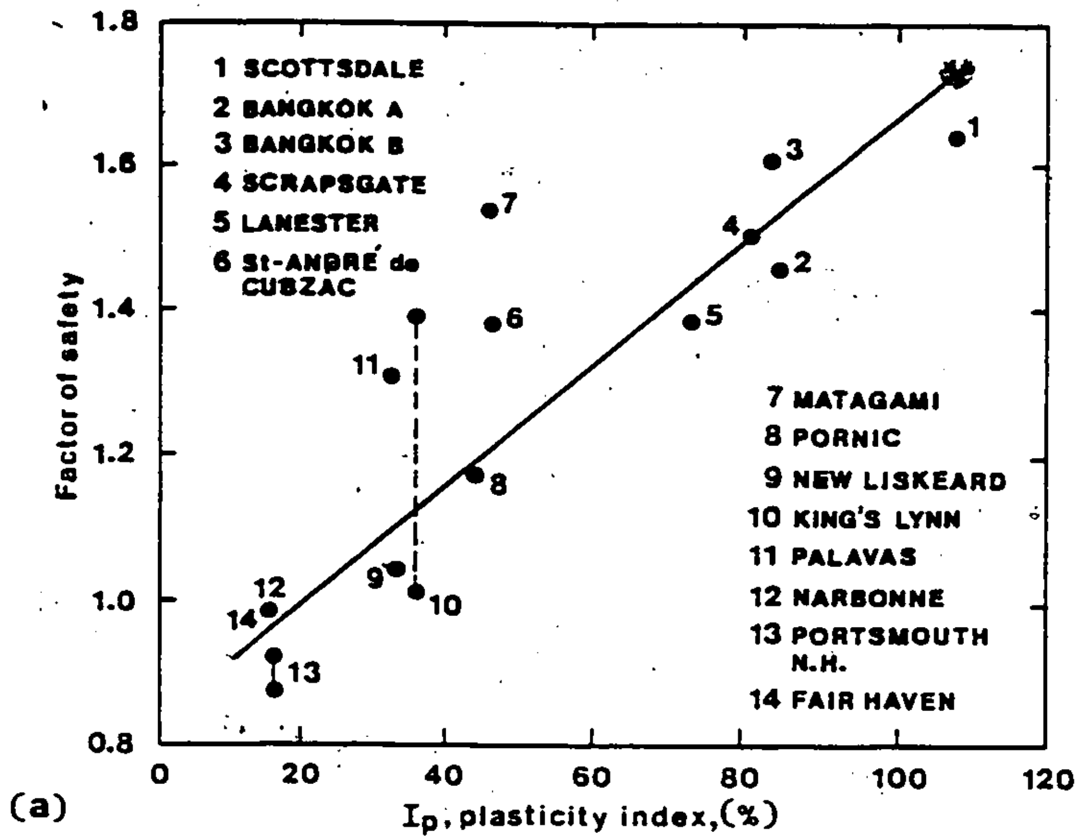
1.3 THESIS OVERVIEW

In Chapter 2, methods which were developed during the 1970's to estimate the shear strength of soils for embankment stability analysis are reviewed. Six methods are examined. Two of these are simple empirical methods and four are semi-empirical methods or laboratory methods.

Chapter 3 presents a detailed description of analytical methods used to evaluate the stability of embankments, silos, vertical cuts and excavations.

Chapter 4 presents four case histories of failures with different types of loading, and includes a description of the geological and geotechnical characteristics of the soils involved, the slope geometries and the stability analyses.

Chapter 5 summarizes the findings of this study, draws together the main conclusions, and gives suggestions for further research.



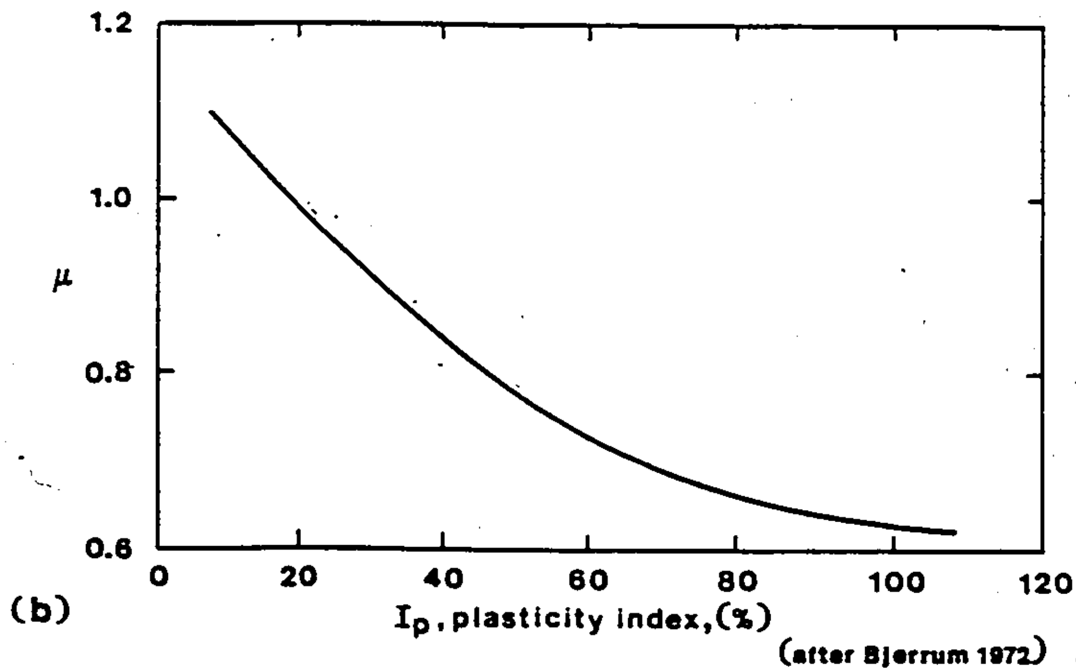


FIG. 1.1 (a) Computed factors of safety versus plasticity indices of foundation clay.

(b) Correction factor to be applied to the vane strength of the clay foundation.

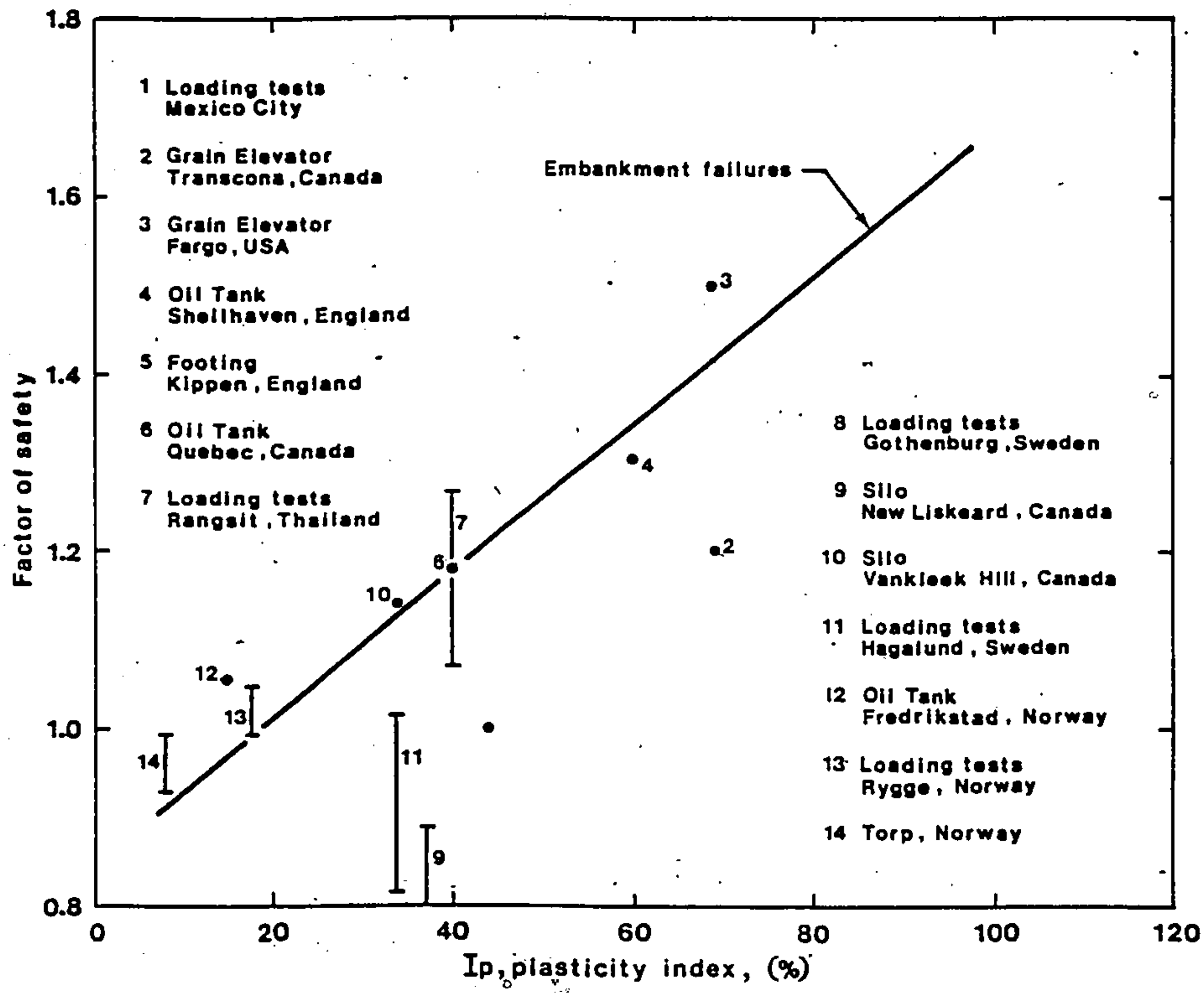


FIG. 1.2 Theoretical factor of safety at failure of footings and loading plates plotted against the plasticity index of the clay (after Bjerrum 1973).

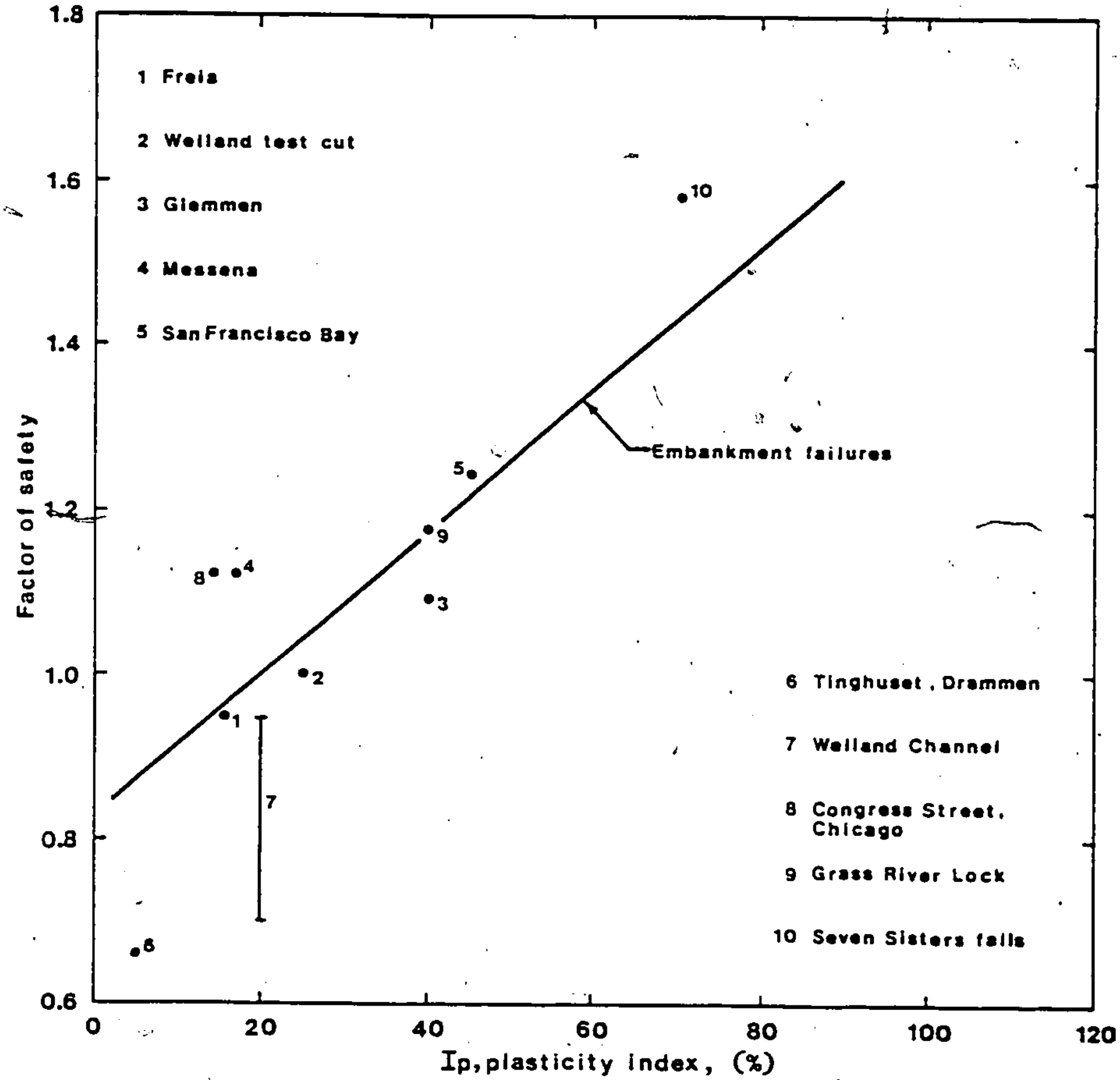


FIG. 1.3 Theoretical factor of safety at failure for cuts and unsupported excavations plotted against the plasticity index of the clay (after Bjerrum 1973),

Chapter II

LITERATURE REVIEW

2.1 GENERAL

Total stress analysis employing c_u, the "undrained shear strength", has been adopted by most engineers as a practical design method to determine the stability of embankments and foundations. Although research continues on total stress analysis, the approach remains problematical. The following is a literature review of developments on this subject within the last decade.

2.2 CONTRIBUTION BY BJERRUM (1972, 1973)

2.2.1 Description

The reliability of stability calculations based on in-situ vane shear strength values ($c_{u(vane)}$) measured in soft cohesive soils has been a matter of concern since the development of the field vane apparatus by Cadling in 1948. In the past, the shear strength values measured by the field vane were believed to be representative of the true shear strength of soils. However, within the last two decades, unexpected embankment failures have occurred on structures designed with relatively high factors of safety. Bjerrum (1972) back-calculated a number of these embankment failures and discovered that the vane strength values used in the designs either underestimated or overestimated the mobilized shear strength of the soils. By plotting the calculated factors of safety with respect to corresponding plasticity indices (Fig. 1.1(a)), he was able to establish, despite a certain scatter, a linear relationship between these two variables. He then proposed a correction coefficient (μ) to be applied to the measured vane strength values as a function of the plasticity index of the soil. As pointed out by Holtz and Wennerstrand (1972), this idea was not new; in fact, correction factors such as this were determined from backcalculated slope failures and used in Sweden for stability calculation.

Bjerrum (1972) interpreted the underestimation and the overestimation of the value of undrained shear strength measured by the in situ vane test as being due to the effect of time, anisotropy, and to the mechanism of progressive failure.

Bjerrum (1972) obtained the correction factor (μ) from a linear regression analysis of the factors of safety as a function of the plasticity index. (Fig. 1.1(a)). The analysis indicated that the difference between the field vane values and the actual mobilized shear strength increases as the plasticity indices of the clays increase. The correction factor, μ, for the field vane shear strength was calculated by reciprocating the factors of safety along the regression line. This correction curve, shown in Fig. 1.1(b), can be expressed approximately by $\mu = 1.2/(1+I_p)$ (Helenelund 1977).

Bjerrum's correction factor can thus be used directly to modify the measured field vane strength to give the following mobilized shear strength of the soil:

$$c_{u(mobilized)} = c_{u(vane)} \times \mu$$

2.2.2 Bjerrum's modified method

In 1973, Bjerrum proposed a modified version of the previously mentioned vane strength correction procedure, based on two main factors that were thought to influence the measured shear strength. These factors, the effect of time (rate) and the effect of anisotropy, are considered separately in this new method.

Bjerrum argues that the discrepancy between the vane shear strength and the mobilized shear strength is caused principally by the reduction of shear strength with time as a consequence of the rate of loading. In addition to this factor, Bjerrum also pointed out that the discrepancy may, in part, be caused by the effects of anisotropy on clay soils resulting from their deposition and stress history. Therefore the previously mentioned equation may be written in the following form:

$$c_{u(mobilized)} = c_{u(vane)} \times \mu_r \times \mu_a$$

where μ_r and μ_a are the reduction factors for the time (rate) effect and the effect of anisotropy, respectively (Fig. 2.1).

The factor μ_a varies along the slip surface depending on the inclination of the slip surface. It will also depend on the plasticity of the soil: the value will be larger the less plastic the clay. Bjerrum (1973) suggested the use of three different tests to simulate the varying shearing conditions along a slip surface (Fig. 2.2). The three tests are triaxial compression, triaxial extension, and direct shear tests, and all samples used in these tests should be reconsolidated under the same stresses as those existing in situ. From such tests, Bjerrum produced the c_u/σ'_{vo} ratios for different sites (Table 2.1) and suggested the use of these ratios to determine the μ_a value.

The μ_r value used in the modified equation can be selected from Fig. 2.1 according to the type of stability problem involved and according to the plasticity index of the soil involved.

The following is a brief review of Bjerrum's (1973) discussion on these two factors that may effect the measured shear strength of clay soils.

2.2.3 Time effect

It is observed that for many materials including soft clays, the slower the material is loaded, the lower the resistance becomes. The first to study this phenomenon was Alexandre Collin, a French engineer who, in 1846, observed a relationship between the material's strength, the applied load, and the loading duration. From his observations, Collin suggested that the shear strength of the material can be separated into two components, the instant and the permanent cohesion. The instant cohesion, as defined by Collin, is the cohesion that disappears after 30 seconds of loading and the permanent cohesion is equal to the shear stress that a sample can sustain without any visible strain (Trak 1980).

Using a series of field vane tests carried out on Scandinavian soft clays at strain rates varying from 0.1° per sec to 1.0° per sec, Cadling and Odenstad (1950) have experimentally shown that by increasing the strain rate, the evaluated undrained shear strength can be increased by 20%. The same phenomenon has been confirmed by Parry (1971), who found that the increase in undrained triaxial strength values obtained at 0.05% per min and 1% per min strain rates is equal to 8% or higher. Many authors have confirmed that the strength mobilized at failure is higher when the time to achieve failure is shorter (Aas 1965).

Bjerrum (1973) considered that this behaviour is associated with the cohesive component of the strength parameters and that the time effect can be explained in terms of the existence of a critical strain. For different loading time, different strains may be reached for the same loading intensity because of the process of creep or secondary compression.

Based on all these observations, Bjerrum (1973) tried to incorporate the time effect by introducing the factor, μ_r, to correct the shear strength values measured by field vane apparatus.

2.2.4 Effect of anisotropy

In general, anisotropy can be divided into two main categories, inherent strength and, stress-induced anisotropies. The inherent strength anisotropy of clay is closely related to its structure which is dependent on the depositional environment of the soil as well as the change in stresses subsequent to deposition. The clays deposited in salt water develop an open card-house structure with the particles randomly oriented. In a fresh water environment, the structure of the deposit is somewhat dispersed and a certain degree of parallelism is developed between the clay particles. In the former case, the shear strength of the clay is more or less isotropic, while in the latter it will exhibit an anisotropic behaviour. It is also known that when clay is subjected to consolidation, its particles align perpendicular to the direction of the imposed load. This tranforms the initially isotropic structure into an anisotropic one.

The stress-induced anisotropy is that which is created by the rotation of the principal stress axes and by the variations in the intermediate principal stress during the loading of soils by structures such as embankments.

In 1973, Bjerrum noted that a vane test can produce only horizontal shear along a vertical surface and argued that this is probably why the shear strength measured by the field vane does not represent the mobilized shear strength at each point along a sliding surface. As mentioned in a previous section, in order to overcome this discrepancy, Bjerrum (1973) proposed a way to include the anisotropy in stability analyses by assuming three different shearing conditions.

2.3 CONTRIBUTION BY PILOT (1972)

At about the same time as Bjerrum (1972) introduced his method to estimate the stability of embankments, Pilot (1972) introduced a similar one based on some unexpected embankment failures which occurred in France (Fig. 2.3). He suggested that the discrepancy between the calculated and the theoretical factors of safety of the embankments at failure might be due to one or more of the following reasons:

1. unsuitability of circular slip surface analysis,
2. poor estimation of field and laboratory undrained shear strength values,
3. effect of progressive failure,
4. and effect of excess pore pressure.

After studying the characteristics of five embankment failures in France, most of which were located in highly plastic clays, Pilot concluded:

1. In homogeneous soils, the failure surfaces are found mostly to be rotational and cylindrical in shape. The shape of these slip surfaces are affected though by the presence of a thick, dessiccated clay crust with high shear strength values. These may then lead to an alteration of the shape of a sliding surface from circular to non-circular.
2. The existence of tension cracks in the embankment fill has a considerable influence on the stability computations. This consideration is especially significant in designs which have low safety factors (F=1.2 to 1.5).

   The formation of tension cracks is caused by the lateral displacement resulting from an increase in horizontal stress during the loading of the subsoil. The degree of lateral displacement depends on the lateral deformability of the foundation subsoil, which decreases as the overconsolidation ratio of the soil increases. In other words, an increase in the restraint of the soil against lateral movement will limit the formation of tension cracks. Therefore, the presence of a thick overconsolidated stratum close to the ground surface would indicate that the fill resistance should be included in the stability calculation.

3. In the case studies, large discrepancies in the factor of safety, up to 0.5, were found between the calculated factors of safety and the factor of safety at failure (F=1.0). There is a tendency for these discrepancies to increase as a function of the plasticity index.

From the calculated factors of safety in Pilot's (1972) case studies and some case studies which were analyzed by Mieussens and Pilot (1971), the following empirical relationship between the factor of safety and the liquid limit (w_L) was established (Fig. 2.3):

$$F = \frac{6w_L}{1000} + 0.7$$

If the above equation were written in terms of the plasticity index (I_p) instead of w_L (Fig. 2.4), it would appear in the following form:

$$F = \frac{7I_p}{1000} + 0.9$$

It can be seen from these relationships that there is a tendency for the overestimation of the safety factor to become significant as the plasticity index or the liquid limit of the clay increases.

According to Pilot (1972), the discrepancy between the shear strength measured from field vane and the mobilized shear strength under the failed embankment, may be attributed to one or more of the following factors:

a) anisotropy of mechanical properties,

b) progressive failure of foundation under embankment loading,

c) inadequate representativeness of the field vane test.

Since special tests were not carried out in the five case studies (Pilot 1972), the first two factors were not examined. The importance of the third factor can be appreciated if one compares the time required to bring an embankment to failure with the time needed to reach failure in a standard vane test at a rotation rate of about 3/10 of a degree per second. This represents a rate that is 5,000 to 10,000 times less than the time required to reach failure in a very rapid embankment construction, and it is 100,000 times less than the time required under normal construction conditions. In an attempt to correlate the time effect on shear strength and the stability of embankments, Pilot (1972) plotted the factors of safety against the time required for the embankment to fail, but no correlation was found, possibly because of the scarcity of the data.

2.4 CONTRIBUTION BY LA ROCHELLE ET AL. (1974)

In 1974, La Rochelle et al. argued that progressive failure may be the most important factor affecting the shear strength of the clay. They proposed a method which would incorporate this effect as well as the effect of time and anisotropy under embankment loading by employing shear strength values measured at large strains in the triaxial apparatus. This approach, called the undrained residual strength method, was used successfully in the analysis of the St. Alban embankment, Quebec. This method was later renamed by Trak et al. (1980) as the Undrained Strength At Large Strains (USALS) analysis. To determine the undrained shear strength at large strains, unconsolidated undrained triaxial tests are conducted. In the St. Alban embankment analysis, the tests were carried out at two different confining pressures: one was equal to half of the original effective overburden, and the other was equivalent to half of the effective overburden after the embankment was constructed. These two confining pressures were used to see whether they had some influence on the measurements. However, no significant difference was detected in the test results.

The residual strength can be reached, according to the experience of La Rochelle et al. (1974), at an axial strain of about 15 per cent, in triaxial tests on Champlain clay samples.

In the analysis of the stability of an embankment, the evaluation of the shear strength of the dessiccated crust has always been a problem. According to La Rochelle et al. (1974), two different crust strength assumptions can be employed when USALS analysis is used. It may be assumed that (1) the USALS of the crust is a constant equal to the minimum value measured in the layer, or (2) the shear strength is extrapolated upward through the crust using the USALS values at the base of the crust (Fig.2.5). Although in the St Alban embankment, La Rochelle et al. (1974) showed that the stability analysis based on the second assumption apparently gave more reasonable values of the factor of safety than those using the first assumption, this kind of crust strength selection still induces some uncertainty in a stability analysis.

2.5 CONTRIBUTION BY LADD AND FOOTT (1974)

2.5.1 Description

In 1974, the SHANSEP method was developed by Ladd and Foott (1974). SHANSEP is an abbreviation for Soil History And Normalized Soil Engineering Properties. The term Soil History refers to the in-situ pressures to which a soil has been subjected in the past. The Normalized Soil Engineering Properties (NSP) refers to cases in which the stress-strain relation can be normalized by effective pressures to give a single relationship such as the one described in Fig. 2.6.

The shear strength behaviour of soils is often best illustrated in terms of normalized soil parameters. The most common ones are the c_u/σ'_{vo}, c_u/σ'_p, and σ'_p/σ'_{vo} ratios, where c_u, σ'_{vo}, and σ'_p are the undrained shear strength, the initial effective overburden pressure and the preconsolidation pressure, respectively. In slightly overconsolidated and overconsolidated clays, the strength is usually normalized with respect to the preconsolidation pressure. Based on such normalized soil parameters, the SHANSEP method, according to Ladd and Foott (1974), can be used to account for a number of effects which influence the mobilized shear strength of soils.

According to Ladd and Foott (1974), the differences between the undrained shear strength values measured in the laboratory and the mobilized shear strength determined by field methods can be attributed to the following factors:

1. sample disturbance,
2. strength anisotropy,
3. strain rate effects.

Sample disturbance is one of the principal cause of loss in shear strength of samples in sensitive clays. During the sampling process, the structure of the clay is disturbed with consequent loss of shear strength. The degree of disturbance in the sample is largely a function of the sampling technique employed. Disturbance is clearly due to the mechanical distortion of the soil during intrusion of the sampler and the in-situ stress relief in the sample after it has been removed from the ground.' This mechanical distortion can be easily minimized, but not totally eliminated, by using better sampling techniques and good equipment such as the large diameter sampler developed by researchers at Laval University (La Rochelle et al. 1981). According to Ladd and Foott (1974), the relief of in-situ stresses, which in turn creates negative pore pressures in the sample, may lead to up to a 20 to 50 per cent reduction in undrained shear strength. Ladd and Foott (1974) claimed that this problem can be eliminated if the SHANSEP method is used to determine the shear strength of the soil, because the reconsolidation process in the SHANSEP method would eliminate the effect of such disturbances. More on this subject will be detailed in the discussion section at the end of this chapter.

2.5.2 Design procedure

The principal steps in the design procedure to evaluate the NSP for the soil as a function of OCR are as follows (Ladd and Foott 1974):

1. Examine and subdivide the soil profile into component deposits on the basis of boring logs, field vane data, visual classifications, etc..
2. Obtain good quality samples and investigate the stress history of the soil profile using total unit weight, pore pressure, and preconsolidation pressure measurements.
3. Check whether the NSP concept is applicable to this type of soil. First, the samples have to be consolidated to approximately 1.5, 2.5, and 4 times the in-situ σ'_p; and, then, the c_u/σ'_c values of the soil have to be measured, where σ'_c is the effective consolidation pressure applied in the laboratory. If the clay exhibits normalized behaviour, it will yield a constant value of c_u/σ'_c, at least at the two higher stresses.
4. Decide which shear strength tests best simulate the in-situ stress condition, and determine the range of OCR values for which data are required.
5. To obtained NSP versus OCR, first reconsolidate the samples back to their virgin compression lines and then reduce the stresses to give the OCR values of

2±0.5, 4±1.0, and 6±2.0. The selected tests are then carried out to obtain the NSP values.

6. Apply these NSP values to the soil profile obtained in steps 1 and 2 to give the distribution of undrained shear strength throughout the foundation subsoil.

To obtain c_u/σ'_p values, various laboratory tests such as plane strain active (PSA), triaxial compression (TC), direct simple shear (DSS), plane strain passive (PSP), and triaxial extension (TE) tests may be employed. All these tests, according to Ladd and Foott (1974), should be performed under K_o-consolidated undrained ($\overline{CK_oU}$) conditions with pore pressure measurements. Strain rates of 0.5 to 1.0 per cent axial strain per hour and 5.0 per cent shear strain per hour are recommended for the triaxial and DSS tests.

The best value of shear strength to represent the average strength along both circular and non-circular slip surfaces may be obtained from the direct simple shear tests. According to the work of researchers at the Massachusetts Insitute of Technology, the obtained values have been close to or slightly lower than the average shear strength values obtained from plane strain compression and extension tests (Ladd and Foott 1974).

2.6 CONTRIBUTION BY AAS (1976(a) AND (b))

The ADP method was introduced by Aas (1976(a) and (b)) to evaluate the undrained shear strength of soft Norwegian clays. ADP is an abbreviation for active, direct shear, and passive analysis. This method is based on Bjerrum's (1973) proposal that the shear strength along a potential slip surface beneath an embankment can be represented by three types of laboratory shear strength tests: the compression, the extension, and the direct shear tests (Fig. 2.2). The ADP method attempts to simulate the failure in anisotropic cohesive soils under embankment loading conditions.

The ADP analysis employs the anisotropically consolidated undrained triaxial (CAU) tests in extension and compression, and the direct shear test. The extension and the compression tests on samples taken from a particular depth give the active shear strength (c_{ua}) and the passive shear strength (c_{up}) of the soil, respectively, and thus represent the active and passive failure states of a soil element. According to studies by Aas (1976(a) and (b)), the undrained shear strength measured in triaxial extension is generally 66 per cent smaller than the shear strength measured in triaxial compression (Aas 1976(a) and (b)). The shear strength (c_{ud}) measured by the direct simple shear test is approximately equal to the mean of the triaxial compression and triaxial extension test measurements.

The shear strength values resulting from the three tests on samples taken at different depths are plotted against the depth of the subsoil (Z) (Fig. 2.7(b)). The following correlations were obtained (Aas 1976(b)):

$$c_{ua} = \alpha_a \; (Z + a)$$

$$c_{ud} = \alpha_d \; (Z + a)$$

$$c_{up} = \alpha_p \; (Z + a)$$

The coefficients α_a, α_d and α_p are the slopes of the linear portion of the shear strength profiles in Fig. 2.7(b), and 'a' is the Z-axis intercept. In practice, 'a' is usually taken as zero; typical values of the other coefficients are shown in Table 2.2.

Aas (1976(b)) also compared the empirical relationship $F = 2.7c_{u(vane)}/\sigma'_{vo} + 0.38$, derived by him from a number of case records, with the relationship between the shear strength ratios $c_{u(vane)}/c_{u(ADP)}$ and $c_{u(vane)}/\sigma'_{vo}$ (Fig. 2.8). It was found that the shear strength at actual failure is better represented by $c_{u(ADP)}$, calculated by averaging the results from the triaxial compression, the triaxial extension and the direct shear tests (Helenelund 1977). This also means that the shear strengths c_{ua}, c_{ud} and c_{up} are assumed to each represent one third of the slip surface (Fig. 2.7(a)). Since the shear strength obtained from a direct shear test is approximately equal to the mean value of the active and passive triaxial tests, the shear strength profile of the subsoil can be obtained simply by averaging the compression and the extension CAU triaxial shear strengths at the corresponding depth. From this shear strength profile, values for a foundation design can be selected.

2.7 RECENT CONTRIBUTIONS

By modifying Bjerrum's (1972) diagrams showing the relationship between the plasticity index and the normalized soil parameters, $c_{u(vane)}/\sigma'_p$ and σ'_p/σ'_{vo} ratios (Fig. 2.9(a) and (b)), Mesri (1975) noted that c_u/σ'_p is a single curve for both aged and young cohesive soils (Fig. 2.9(c)). When Bjerrum's correction factor (μ) is applied to the curve, the function $\mu c_u/\sigma'_p$ is a constant. Hence, the mobilized shear strength seems to be independent of the plasticity index (I_p) of the soil and can be expressed as a constant fraction of σ'_p, i.e. $c_u = 0.22\sigma'_p$ (Fig. 2.9(d)). However, Mesri (1975) did [illegible] go further with his investigation of this interesting finding.

This expression was later used by Trak et al. (1980) in the stability analyses of embankments on soft clays. The method was found to be applicable to the soft, sensitive clays of Eastern Canada and the Bangkok clay in Thailand. When presenting the results of the USALS analysis of sensitive clay in Quebec, Trak et al. (1980) confirmed that the average value of $c_{u(USALS)}/\sigma'_p$ agreed with the average value found by Mesri (1975). A plot of this ratio is shown in Fig. 2.10.

A similar ratio was also found using Ladd and Foott's (1975) SHANSEP results. The ratio of $c_{u\overline{(CK_oUDSS)}}$ at OCR values of one and two were approximately equal to 0.22 (Fig. 2.11, Trak et al. 1980).

On the other hand, the c_u=$0.22\sigma'_p$ expression was found to be not applicable to some cohesive soils, such as very young and nearly normally consolidated organic clays. The low factors of safety found in back-calculations of failures in these soils can be attributed to unrealistic c_u/σ'_p ratios (Trak et al. 1980). These ratios were caused either by an overestimation of the field vane strengths, or an underestimation of the preconsolidation pressures as measured by the oedometer tests. However, the low factors of safety may also simply indicate that the method is not applicable to these young organic clays (Larsson 1980).

2.8 GENERAL DISCUSSION

2.8.1 Bjerrum's method

Since Bjerrum's method of vane strength correction was introduced in 1972, a number of case studies have confirmed that the shear strength values measured by the vane in soft clays are generally overestimated (Pilot 1972; Bozozuk 1972;

Ladd and Foott 1974; and Helenelund 1977). Bjerrum (1973) had pointed out that the effects of time (rate of loading) and anisotropy have a strong influence on shear strength. Dascal et al. (1972) demonstrated in a case study of the Matagami test fill that progressive failure also affects the shear strength of the soil, especially in sensitive soils such as the clays of Eastern Canada.

Numerous other attempts have been made to improve Bjerrum's original method by introducing other correction factors such as corrections for the effect of progressive failure and for anisotropy effects (Bjerrum 1973, Helenelund 1977). Some researchers have also applied the method by correcting the vane strength of individual layers of the same deposit according to their corresponding I_p (Dascal et al. 1972; Lacasse and Ladd 1973). Such attempts are beyond the original scope of Bjerrum's method and undoubtedly will give rise to more variability in the stability calculations.

2.8.2 Pilot's method

In Pilot's (1972) approach, the two relationships given in section 2.3 can be used to estimate the factor of safety of an embankment in soft soil. However, one can also tranform the two equations into a form which makes use of a correction factor as in Bjerrum's method. This can be done simply by taking the reciprocal of the two equations obtained by Pilot which expressed the safety factor as a function of the liquid limit, w_L, and plasticity index, I_p.

$$\mu = \frac{1000}{7I_p + 900}$$

$$\mu = \frac{1000}{6w_L + 700}$$

When the above correction factor is plotted on a graph of μ versus I_p, the curve obtained is very similar to the one which Bjerrum (1972) had derived (Fig. 2.12). This is not suprising because most of the case histories studied by Pilot appear also in Bjerrum's list.

In general, the above vane strength correction (μ) should preferably be obtained from the equation expressed as a function of w_L because the determination of w_L requires only one test and is both faster and subjected to less variability than the values obtained for I_p (Tavenas and Leroueil 1980).

2.8.3 USALS analysis

La Rochelle et al. (1974) have shown the applicability of the USALS method in the analyses of embankments in soft sensitive clay deposits of Eastern Canada. It was suggested by La Rochelle et al. (1974) that the undrained strength at large strains (USALS) could incorporate sample disturbance, progressive failure, time and anisotropy effects, which are the main factors causing the discrepancy between the test results and the true mobilized shear strength. The USALS analysis has been shown by Trak et al. (1980) to yield reliable stability calculations for the Champlain clays and other fluvioglacial and lacustrine clays of Canada.

The USALS method is, however, subject to experimental difficulties. According to Trak et al. (1980) and Tavenas and Leroueil (1980), these difficulties are related to the corrections of the cross-sectional variation of the sample, and the membrane effects in the triaxial testing at large strains.

2.8.4 SHANSEP method

The SHANSEP and ADP methods have apparently been derived from similar lines of thinking. Both methods emphasize the effect of anisotropy in the determination of the shear strength of soft clays. Neither of the methods relies on in-situ vane strength values which are markedly influenced by the anisotropic strength characteristics of soils. Shear strength in these methods is determined by the direct shear, triaxial compression and extension tests on anisotropically consolidated clay samples, to simulate strength variations on the failure surface.

The SHANSEP method is claimed by Ladd and Foott (1974) to account for the effect of sample disturbance and the effect of the rate of loading because of the derivation of the normalized soil parameters by reconsolidation of the clay to overconsolidation ratios greater than one. The method has been criticized by Mesri (1975) on the grounds that the reconsolidation of the samples to values greater than the in-situ preconsolidation pressure prior to shearing will further remold the clay rather than restructure it to overcome sample disturbance as claimed by Ladd and Foott (1974).

Ladd and Foott (1974) indicated that the laboratory testing required in the SHANSEP method could result in the collapse of highly structured 'quick' clays and naturally cemented deposits but would be applicable to other clays. However, Mesri (1975) pointed out that almost all the natural clays have developed structures, so the applicability of SHANSEP method is extremely limited.

2.8.5 ADP method

The basis of the ADP method is to simulate the actual failure modes by means of three different types of laboratory triaxial tests, the compression and the extension tests, and the direct shear test (Fig. 2.2). According to Aas (1976(a) and (b)), this simulation can take into account the effect of the discrepancy between the actual and the measured shear strength values. However, because of the effect of the rotation of principal stresses due to embankment loading, the state of stresses other than the ones at the center line of the structure is not known. This shows that the states of stress in all aforementioned tests, especially in the direct shear test where the direction of principal stresses are unknown, cannot truly represent the conditions in the field.

2.9 CONCLUSION

In this chapter, two categories of shear strength estimation methods were discussed. The first type is the empirical methods which involve the determination of undrained shear strength by means of simple in situ tests, such as the field vane test, without any knowledge of the state of the in-situ effective stress. Analyses of this type include Bjerrum's (1972) and Pilot's (1972) methods. Analyses of the second type are the laboratory methods including the USALS method (La Rochelle et al. 1974), the SHANSEP method (Ladd and Foott 1974), the ADP method (Aas 1976(a) and (b)), and the c_u=$0.22\sigma'_p$ method (Mesri 1975 and Trak et al. 1980). This second type can involve complicated laboratory tests, such as CAU, plane strain active, and plane strain passive tests.

As mentioned in section 2.2.4, because the shear strength is stress path dependent, the values measured by two different tests are rarely the same. In turn, this means that the stress path which the soil under embankment loading follows toward failure may not have the same pattern as the ones in the tests. As a consequence of this discrepancy for most of the previously mentioned methods, empirical correlations have to be made to obtain a good representation of the mobilized shear strength underneath an embankment. The main reason for using empirical methods in determining shear strengths for stability analysis is because of the difficulty in predicting the excess pore pressure and the state of stress during rotation of principal stress axes in the subsoil.

In 1973, Bjerrum showed in two F.S. vs I_p plots that the results of excavation and bearing capacity failures can be well represented by the same regression line as in embankment failures (Fig. 1.2 and Fig. 1.3). Therefore he suggested that his vane strength correction approach can also be used in loading and unloading conditions. Because the $c_u=0.22\sigma'_p$ expression was derived by making use of Bjerrum's correction factor (Fig. 2.9(d)), it appeared that this expression could also be used to estimate the mobilized shear strength under embankment loading conditions and, by extension, could also be applied to bearing capacity and excavation problems (Trak 1981). The application of such an approach to embankments on Canadian sensitive clays, has successfully been confirmed by Trak et al. (1980). In the following chapters, the applicability of this expression to evaluate the mobilized shear strength of soft sensitive clays under excavation and bearing capacity loading conditions will be examined. There is also an investigation on an embankment failure in Bangkok clay using the same expression.

In the next chapter, a description of the methods of analysis used to investigate the case histories will be presented.

TABLE 2.1

COMPARISON BETWEEN THE RESULTS OF COMPRESSION AND EXTENSION TESTS, DIRECT SIMPLE SHEAR TESTS AND IN SITU VANE TESTS ON SOFT CLAYS

TYPE OF SOIL	PHYSICAL PROPERTIES				TRIAXIAL TEST c_u/σ'_{vo}		SIMPLE SHEAR TEST c_u/σ'_{vo}	VANE TEST c_u/σ'_{vo}	
	w	w_L	w_p	I_p	Comp.	Tens.		Observed	Corrected for rate
BANGKOK	140	150	65	85	0.70	0.40	0.41	0.59	0.47
MATAGAMI	90	85	38	47	0.61	0.45	0.39	0.46	0.40
DRAMMEN (PLASTIC)	52	61	32	29	0.40	0.15	0.30	0.36	0.30
VATERLAND	35	42	26	16	0.32	0.09	0.26	0.22	0.20
STUDENTERLUNDEN	31	43	25	18	0.31	0.10	0.19	0.18	0.16
DRAMMEN	30	33	22	11	0.34	0.09	0.22	0.24	0.21

(AFTER BJERRUM 1972)

TABLE 2.2

TYPICAL VALUES OF α_a, α_d, AND α_p (MORGENSTERN ET AL. (1977))

PLASTICITY INDEX, I_p (%)	ACTIVE α_a	DIRECT α_d	PASSIVE α_p
10 - 20	0.30 - 0.40	0.15 - 0.25	0.08 - 0.15
40	0.35 - 0.50	0.25 - 0.35	0.15 - 0.25
40 - 100	0.65 ±	0.40 ±	0.40 ±

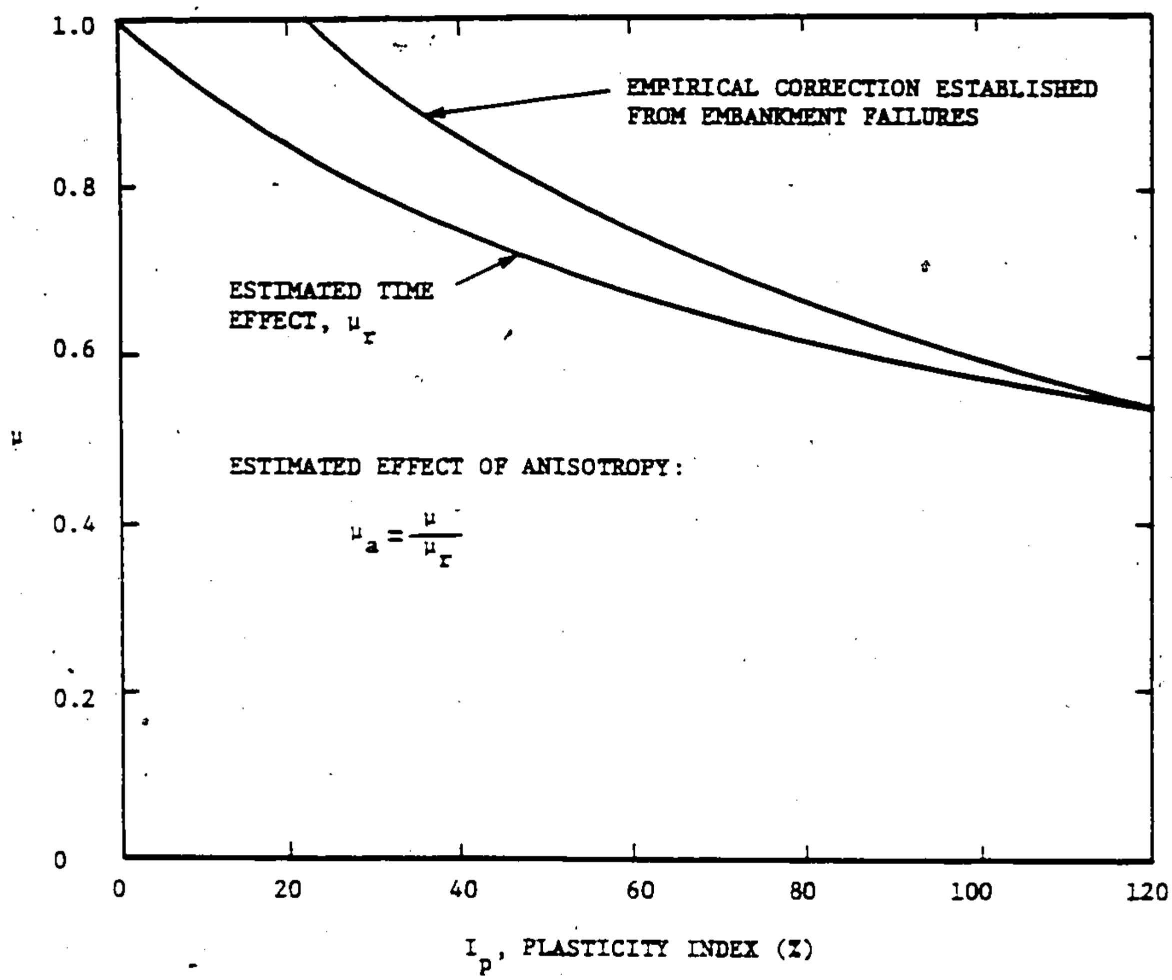


FIG. 2.1 CORRECTION FACTORS ESTABLISHED FOR THE EFFECT OF TIME AND ANISOTROPY.

(after BJERRUM 1973)

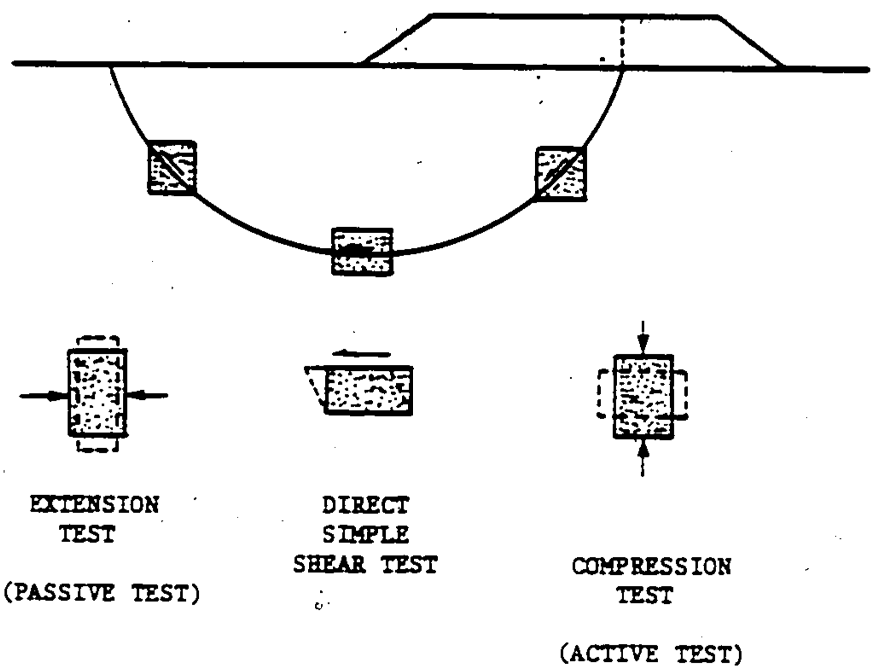


FIG. 2.2 RELEVANCE OF LABORATORY SHEAR TESTS TO SHEAR STRENGTH IN THE FIELD

(after BJERRUM 1973)

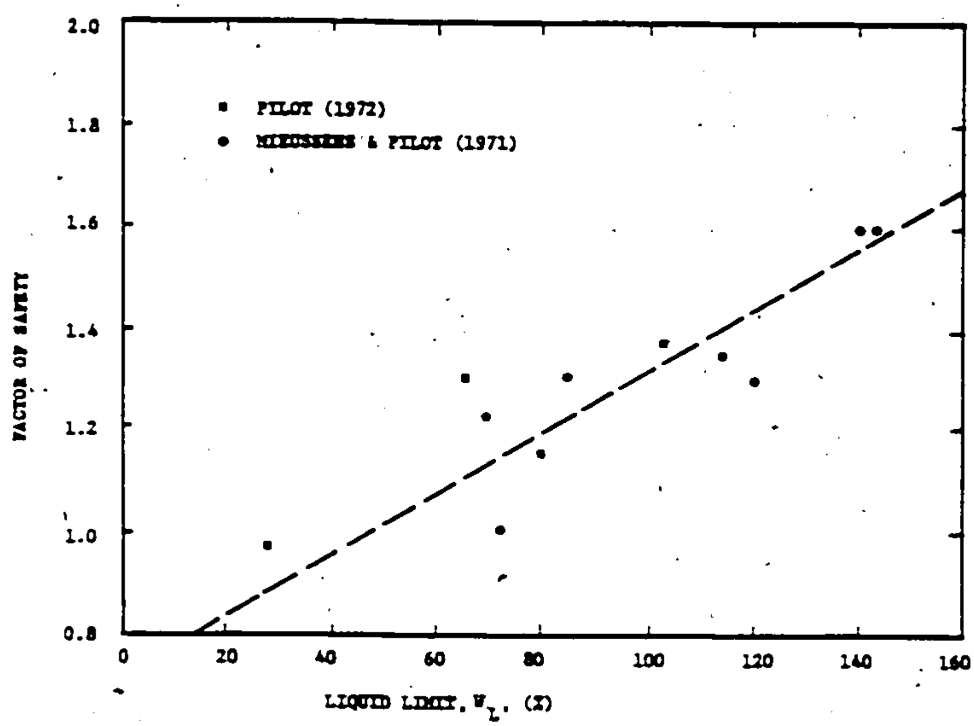


FIG. 2.3 CORRELATION BETWEEN FACTOR OF SAFETY AND LIQUID LIMIT

(after PILOT 1972)

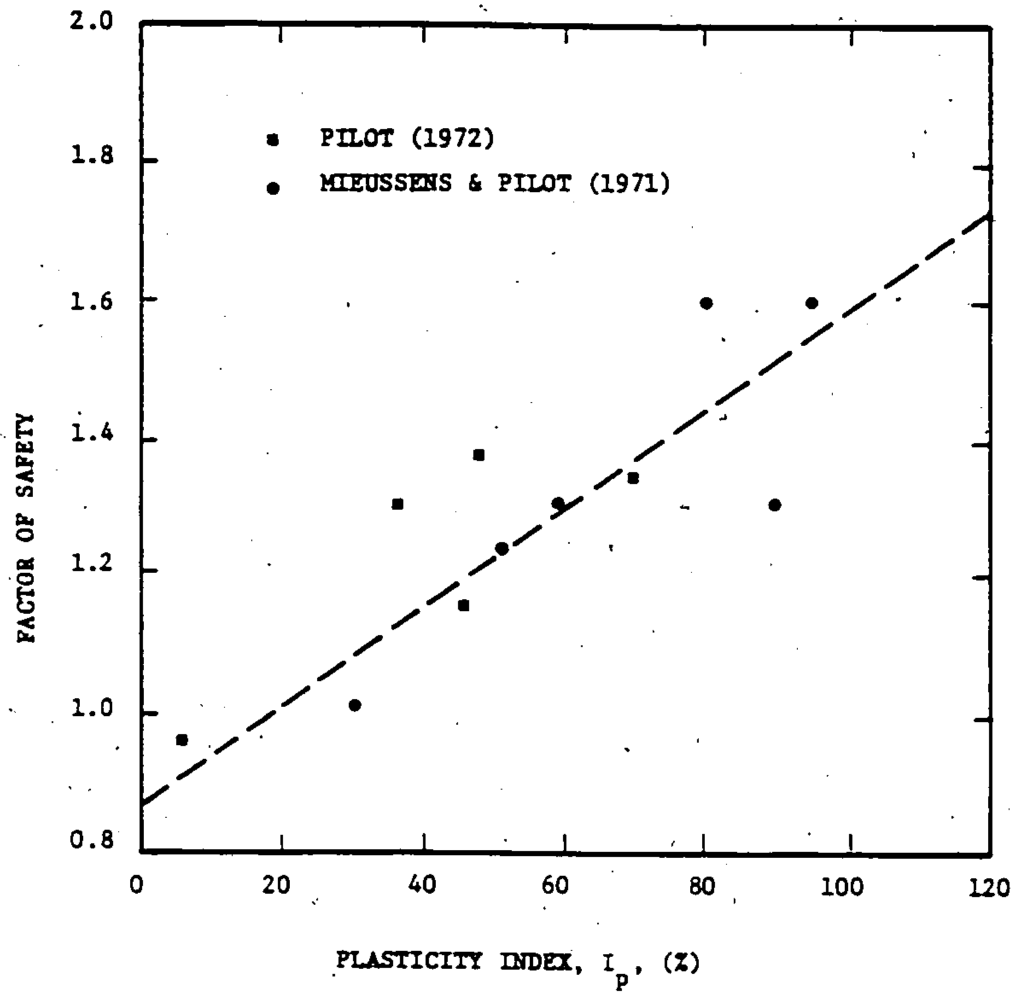


FIG. 2.4 CORRELATION BETWEEN FACTOR OF SAFETY AND PLASTICITY INDEX

(after PILOT 1972)

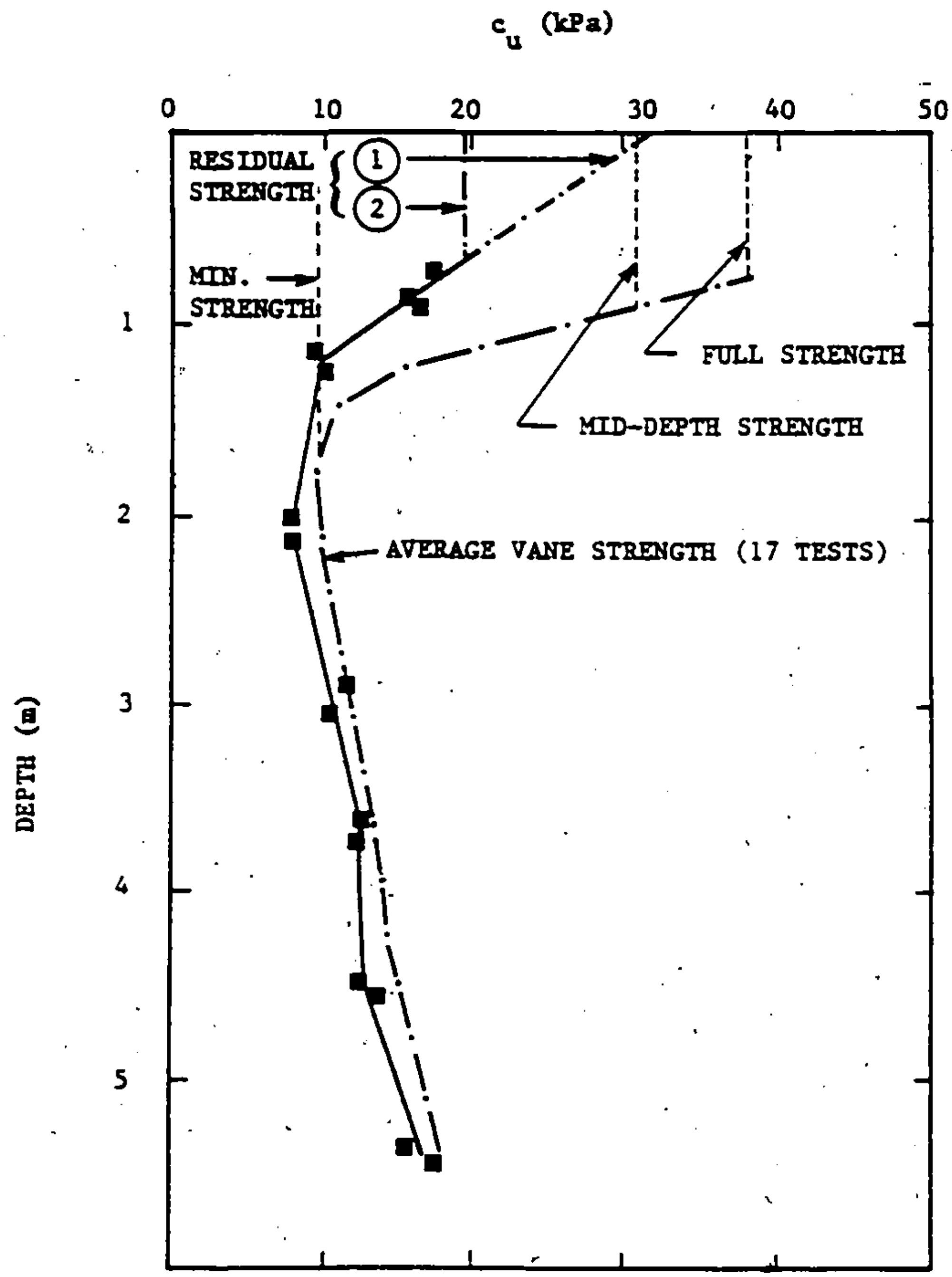


FIG. 2.5 STRENGTH PROFILES WITH DIFFERENT CRUST STRENGTH ASSUMPTIONS, ST. ALBAN

(after LA ROCHELLE et al. 1974)

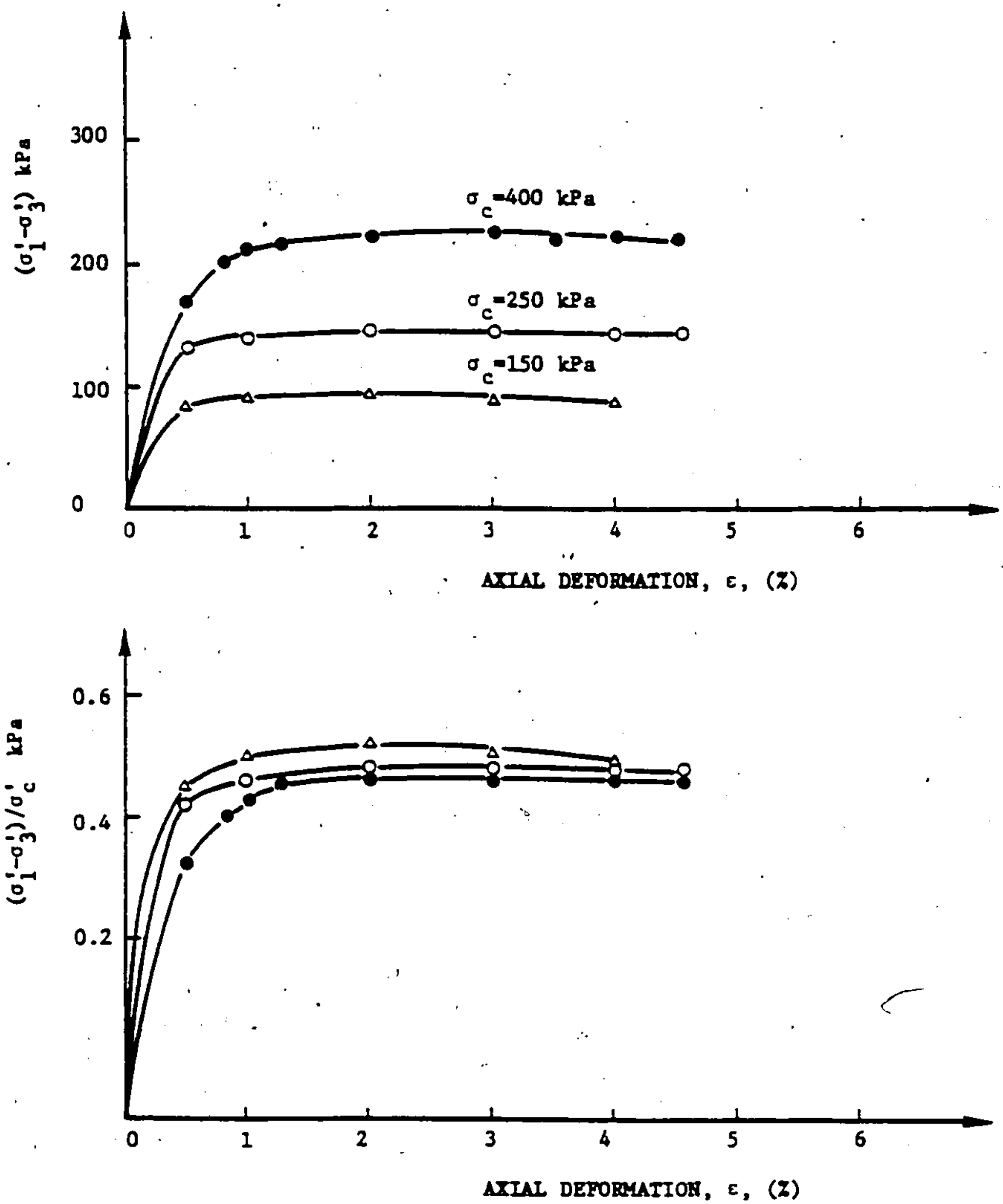


FIG. 2.6 (a) TRIAXIAL TEST RESULTS FOR DRAMMEN CLAY (after BJERRUM & SIMONS 1960)

(b) NORMALIZED PLOT OF TRIAXIAL TEST RESULTS, DRAMMEN CLAY

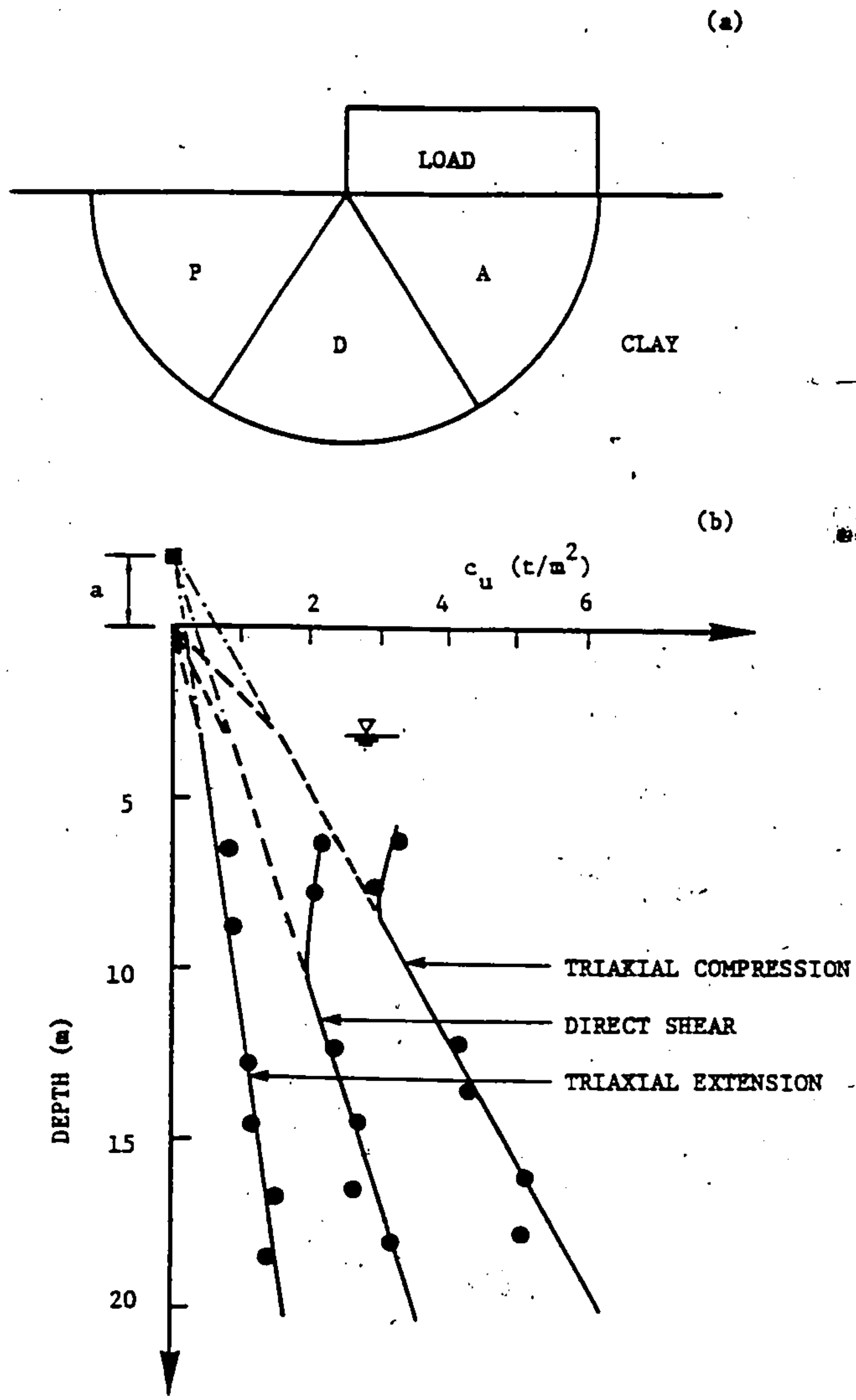


FIG. 2.7 BASIC PRINCIPLES IN AN ADP ANALYSIS

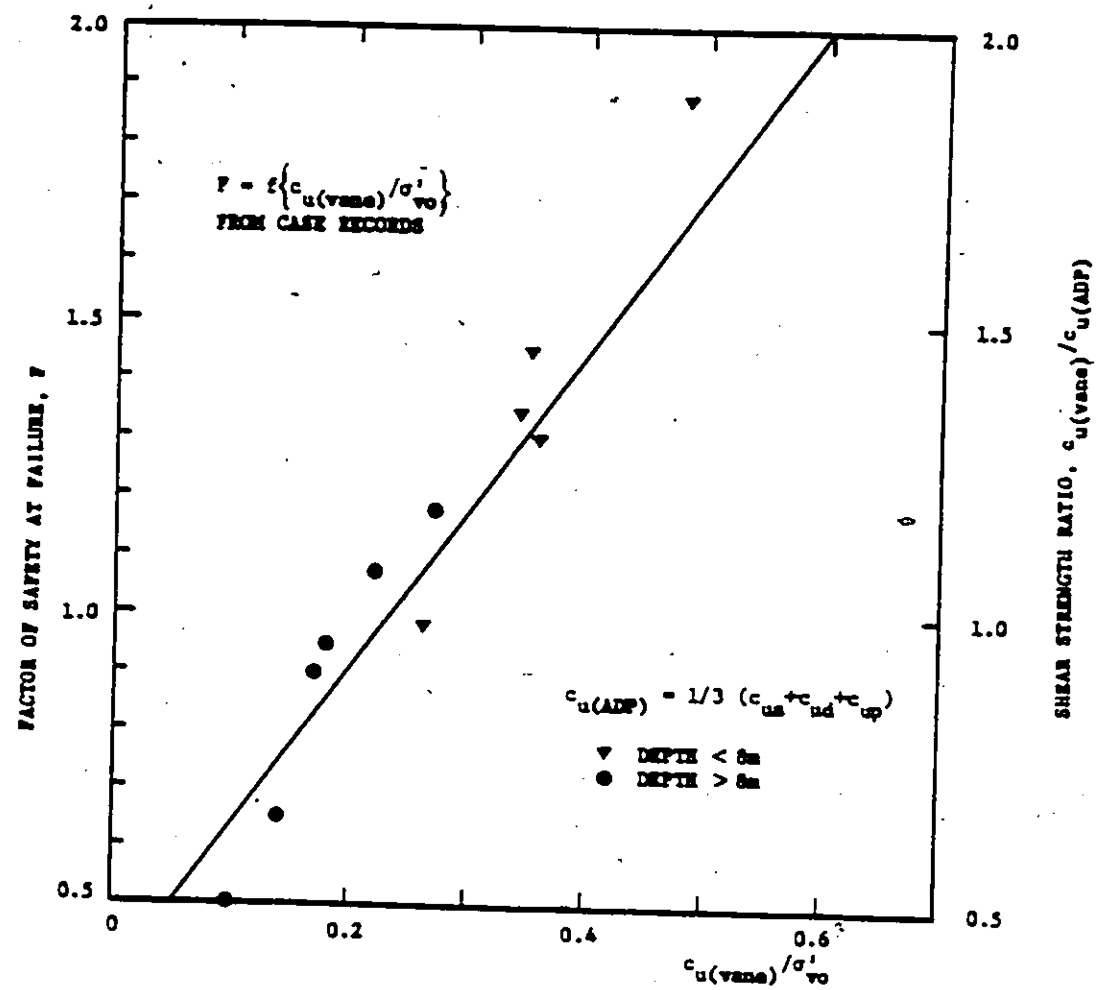


FIG. 2.8 CORRELATION BETWEEN FACTOR OF SAFETY, $c_{u(vane)}/\sigma'_{vo}$ AND $c_{u(vane)}/c_{u(ADP)}$

(after AAS 1976(b))

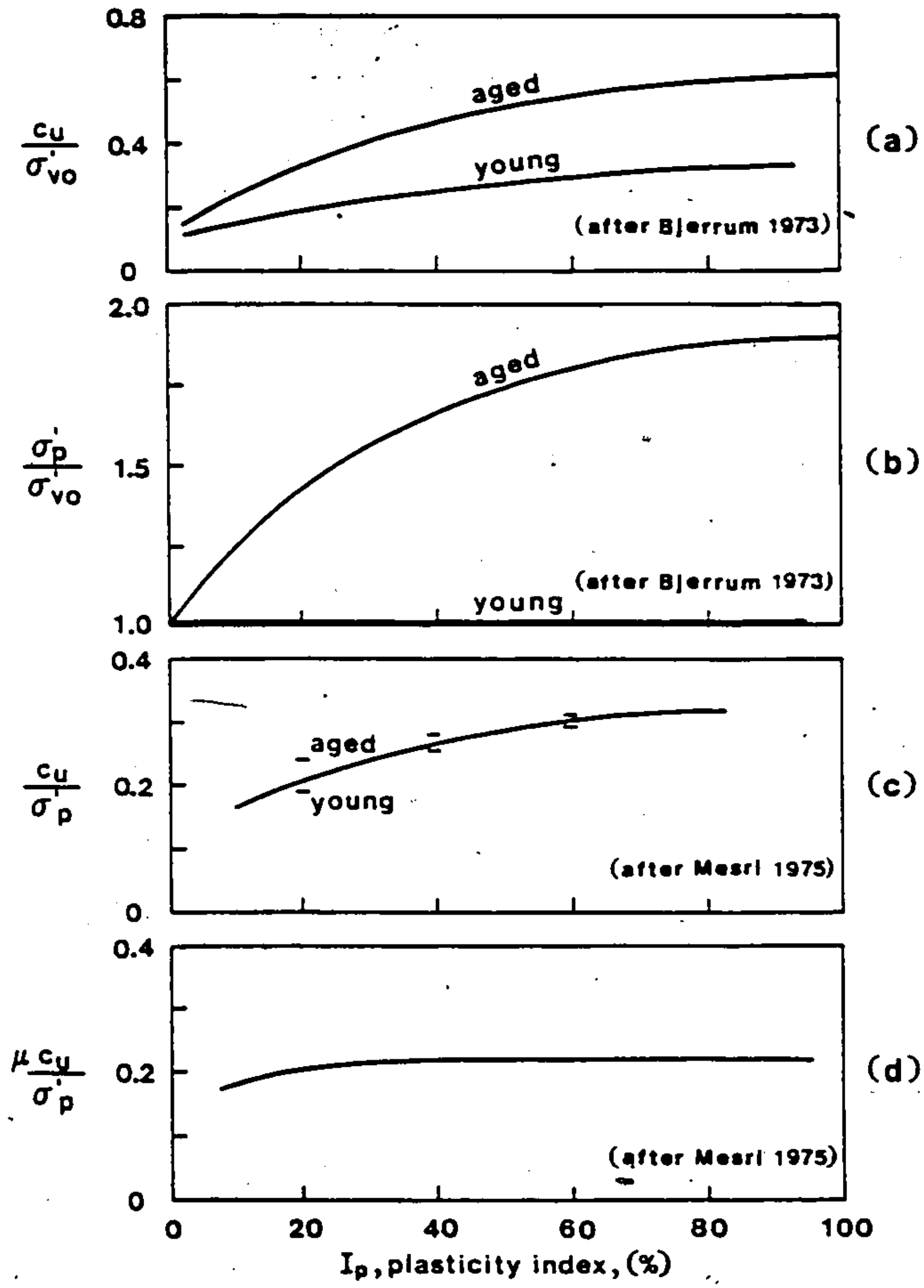


FIG. 2.9 Typical ratios for normally consolidated late glacial and postglacial clays.

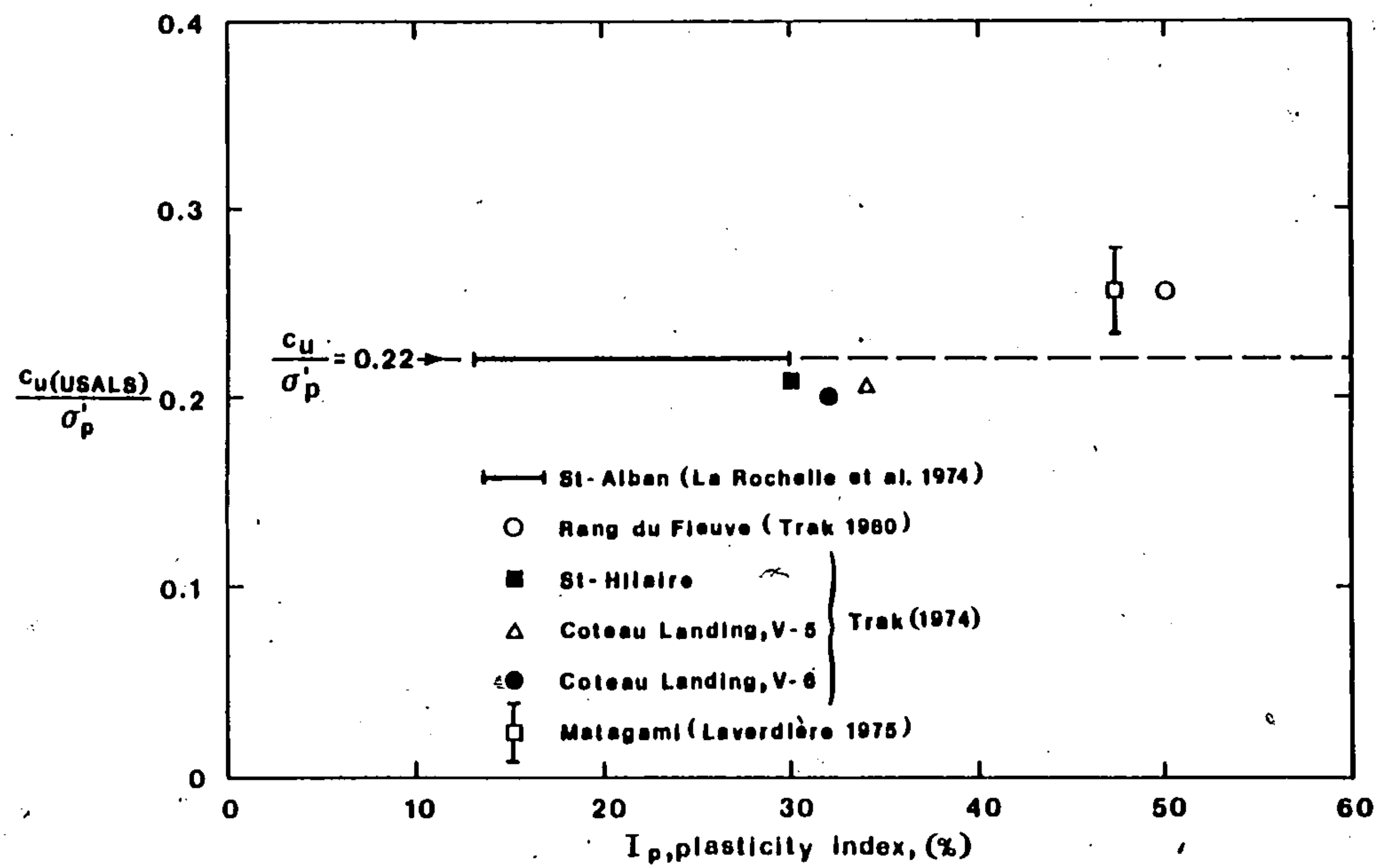


FIG. 2.10 Similarity of $c_u = 0.22\,\sigma'_p$ and the undrained strength at large strain (USALS) for different sites of sensitive clay in Quebec.

(after TRAK et al. 1980)

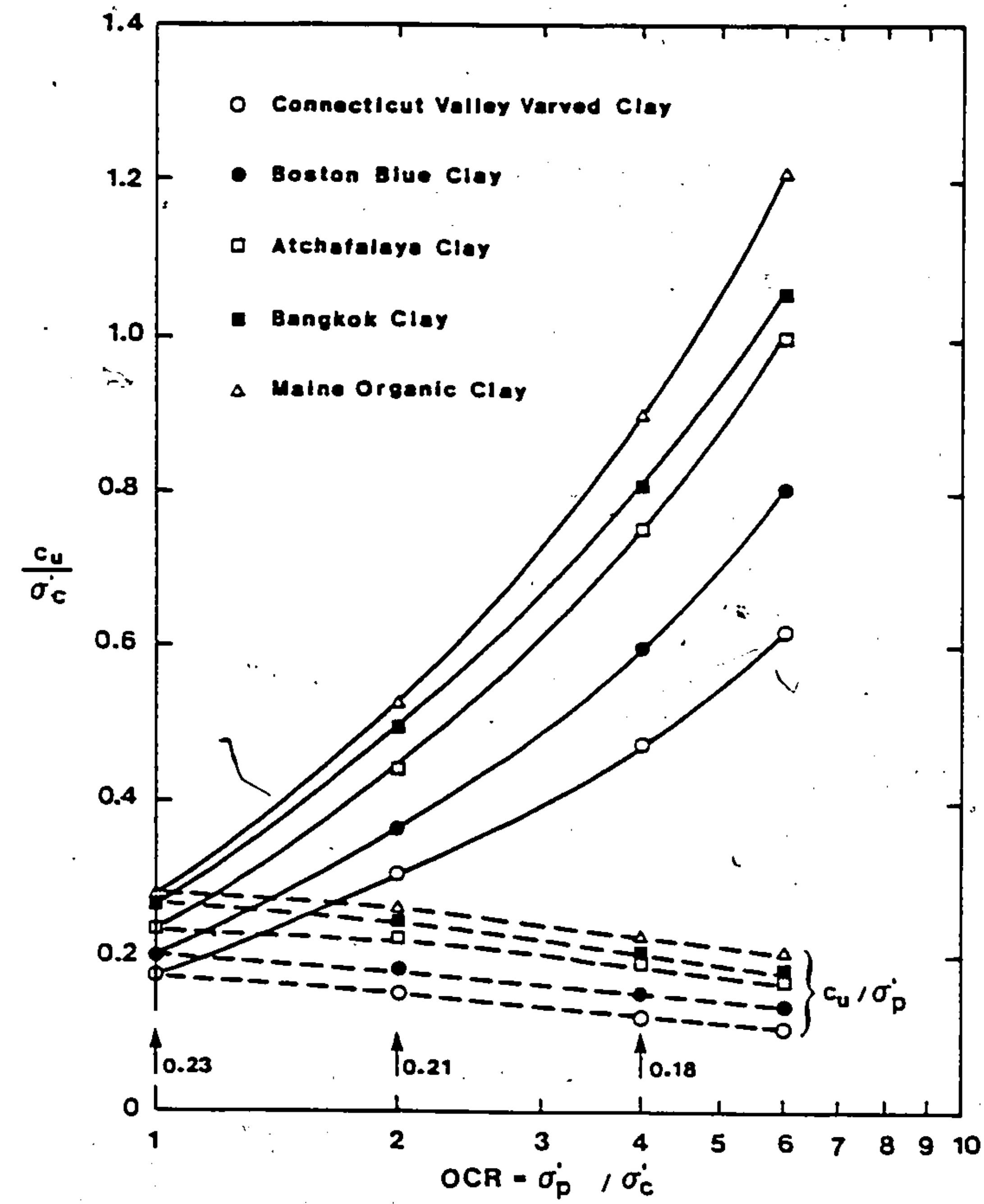


FIG. 2.11 Variation of normalized CK_0U direct simple shear (DSS) strength parameters with overconsolidation ratios (OCR) for five clays (after Ladd and Foott 1974), and the corresponding ratios of c_u / σ'_p

(after TRAK et al. 1980)

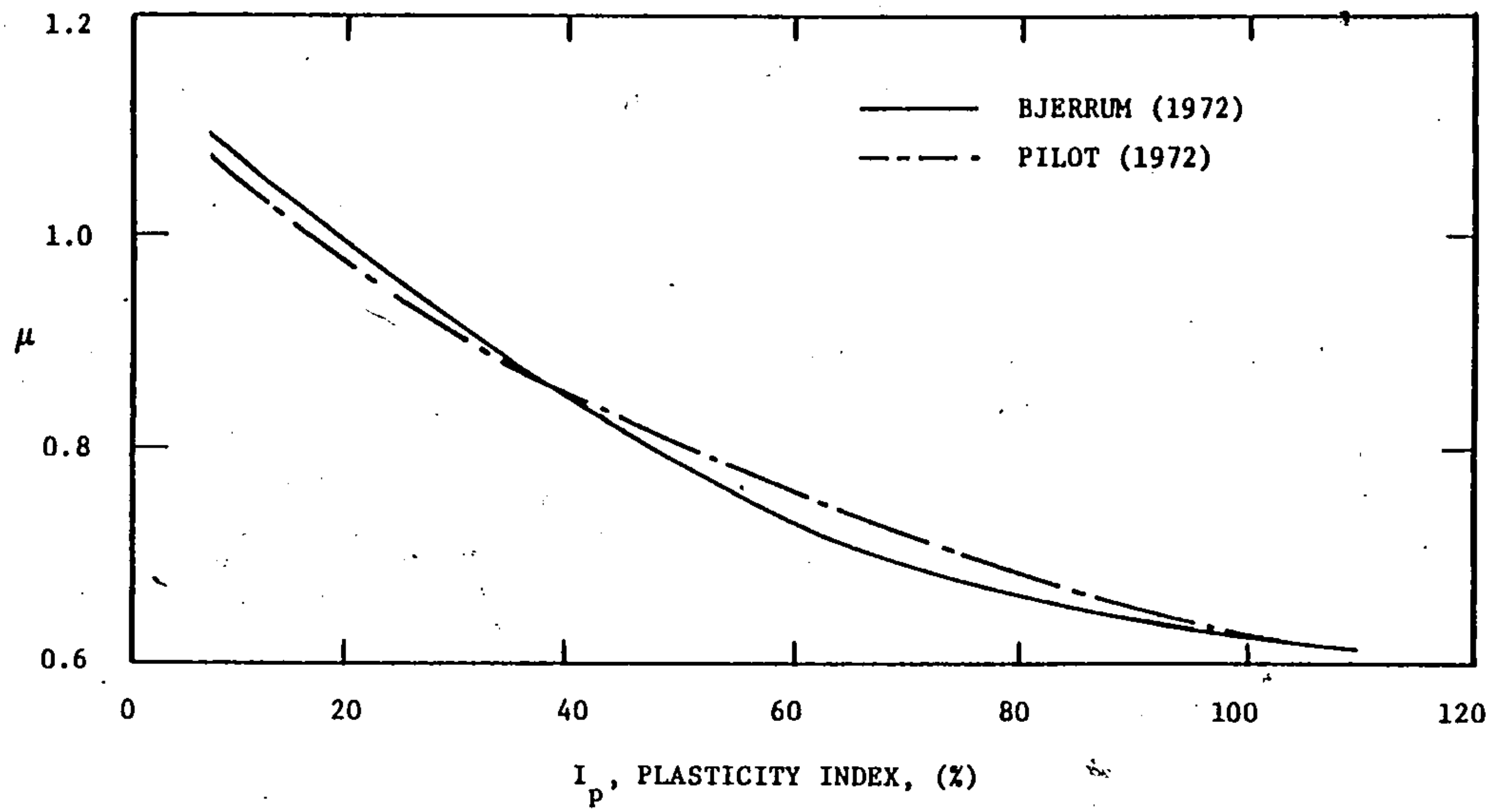


FIG. 2.12 COMPARISON BETWEEN BJERRUM'S (1972) AND PILOT'S (1972) VANE STRENGTH CORRECTION FACTORS

Chapter III

METHODS OF ANALYSIS

3.1 GENERAL

The stability of embankments, vertical cuts and shallow foundations are commonly analyzed by limit equilibrium methods with $\phi=0°$ as an assumption. One advantage of these methods over other stability methods is the simplicity of the design calculations. The methods were used to analyze the case studies presented in Chapter IV, as described in this chapter.

3.2 $\phi=0°$ ANALYSIS

In 1916, the slip of the Stigberg Quay in the harbour of Gothenburg, Sweden, initiated a significant development in the study of slope stability problems. Following this slide, a series of studies were started, and they culminated in the development of the Swedish slip circle method. According to the concepts of the time, clay was treated as a purely frictional material in slope stability problems (Petterson 1955). Hellan, a Norwegian civil engineer with the Trondheim Harbour Authority, suggested in 1917 that, in evaluating the shear strength, the clay should be treated as a purely cohesive material. Fellenius later combined the circular slip surface assumption suggested by Petterson and the Hellan pure cohesion concept to develop a comprehensive method, known today as the Swedish circle method. Later, Olsson applied the $\phi=0°$ analysis to predict the stability of existing and future railway embankments in Sweden (Bjerrum and Flodin 1960). Since then, the $\phi=0°$ analysis has been in steady use in Scandinavia and elsewhere, whenever stability problems concerning soft clay deposits have occurred.

According to the concept of "undrained shear strength", or $\phi=0°$ analysis, a soil sheared in undrained conditions, is assumed to have an angle of internal friction equal to zero. The shear strength under this condition is the same, regardless of the level of stress used for testing. This shear strength may be determined from laboratory tests such as the unconfined compression, the consolidated undrained triaxial and the direct simple shear tests; or by field tests, such as the vane, static penetrometer and pressuremeter tests.

3.3 ANALYSES OF SLOPE STABILITY BASED ON THE LIMIT EQUILIBRIUM METHOD

3.3.1 Description

Various methods of analysis have been suggested for investigating the stability of embankments and cuts. Limit equilibrium methods are the ones most commonly used by geotechnical engineers in these types of stability analysis.

The theories on which most of these methods are based are similar in principle to the wedge theories of earth pressure where a potential surface of rupture is assumed. The material above this slip surface is considered as a whole mass, in a state of equilibrium. The driving and resisting forces along an assumed slip surface are estimated, enabling the formulation of equations concerning force and/or moment equilibrium of the potential sliding mass for the computation of the factor of safety which will be discussed later.

In limit equilibrium analysis, the calculation process is customarily repeated a number of times to locate the most unfavorable slip surface, also called critical slip surface, for which the factor of safety has its minimum value. The shape of the slip surface can be influenced by geological features; therefore, the number of trials may be reduced when there is a well-defined discontinuity forming a potential weakness plane.

In the last few decades, more than a dozen slope stability methods have been developed. They differ in the assumptions used to render the problem determinate (Fredlund and Krahn 1977). The most common methods are Fellenius', Bishop's simplified, Janbu's simplified, and Janbu's rigorous methods. Others, such as Spencer's and Morgenstern-Price methods, are rarely used, although they are the only ones that satisfy both the force and the moment equilibrium equations simultaneously. The latter two methods require considerable computer time, making them more costly. For example, a slope stability problem analyzed by the Morgenstern-Price method requires six times more computer time than when analyzed by the Bishop's simplified method (Fredlund and Krahn 1977).

The following presentation is a review of one of the most commonly used stability methods, Bishop's simplified method. This method is used extensively in the stability analyses of our study.

3.3.2 Bishop's method

Bishop (1955) formulated a method using the concept of slices. This is similar to Fellenius' method which will be discussed later. Bishop took into account the forces between slices in the equations of equilibrium. He demonstrated that by combining a moment equilibrium analysis on the whole sliding mass and an iteration process, one was able to obtain a factor of safety for a slope. Also, he showed that a simplified method can be derived from his original rigorous method and that it gives fairly accurate results.

Bishop's simplified method of analysis is based on the limit moment equilibrium of a chosen circular failure surface of which the sliding portion of the slope is divided into slices. The basic assumptions of the method are:

1. The soil behaves as a Mohr-Coulomb material ($\tau = c' + \sigma' \tan\phi'$),
2. the factor of safety is the same for all slices, and
3. the sum of the vertical shear forces on each side of the slices is negligible: ($X_n + X_{n+1} = 0$).

The equation for the factor of safety under the condition $X_n + X_{n+1} = 0$ (Fig. 3.1) is:

$$F = \frac{\Sigma(c'b + (W - ub)\tan\phi')/m_\alpha}{W\sin\alpha}$$

where $m_\alpha = \cos\alpha\,(1 + \frac{\tan\alpha\,\tan\phi'}{F})$

For the $\phi = 0°$ condition and $c' = c_u$,

$$F = \frac{\Sigma c_u b/\cos\alpha}{W\sin\alpha}$$

$$= \frac{\Sigma c_u \ell}{W\sin\alpha}$$

3.4 BEARING CAPACITY

3.4.1 Description

In many cases, such as shallow foundations, the load supported by the soil is that of a footing. Slip failures similar to those of embankments can occur within the soil mass even if the ground surface is horizontal. If the loading of the footing is sufficiently large, the foundation may fail by sinking into the ground with the soil heaving around the footing (Parcher and Means 1968).

Three different failure modes have been identified in bearing capacity problems. They are: the general shear failure, local shear failure, and punching shear failure. General shear failure is characterized by a continous rupture of the surface throughout the foundation subsoil (Fig. 3.2(a)) as in slope stability failure. Such failures may be sudden and lead to total structural collapse. If a foundation failure does not lead to the sudden collapse of the structure, then it is called a local shear failure (Fig. 3.2(b)). This type of failure is caused by the generation of a local shear zone underneath the footing, and it creates a relatively large amount of settlement prior to the development of plastic equilibrium. When the structural load is very intense or the soil is relatively loose, another form of failure, the punching shear failure, may occur. Punching shear failure means the footing plunges deep into the soil (Fig. 3.2(c)); the resistance is provided by the shear force developed in the soil around the perimeter of the footing and by the bearing resistance underneath the footing which increases as the footing penetration increases (Lambe and Whitman 1973).

A number of attempts have been made to predict the structural load at which a foundation failure occurs. Many theories and approaches have been developed, some of which have shown agreement with the results of experiments and full scale tests. In general, the methods of analysis to predict the ultimate bearing capacity of a foundation can be classified into the following four groups:

1. Methods based on active and passive earth pressures occurring under the footing,
2. slip surface methods of the so-called limit equilibrium methods,
3. analyses based on plastic failure of the subsoil, and
4. direct measurement by loading tests.

In the case histories investigated by the original authors, only the second and third types of analyses were used. Therefore, to enable a comparison of the results, only these two types of analyses were considered in this study.

3.4.2 Slip surface method

The limit equilibrium methods can be adapted, with slight modifications, to the estimation of the bearing capacity of footings.

The best known method is the circular arc method, known as Fellenius' method. This method was originally proposed by Fellenius in 1927 for a strip load applied at ground level, and was subsequently modified for footings founded below ground surface. Failure of the footing is assumed to take place by the heaving up of a mass of soil on one side only.

The following is an illustration of the slip circle method used to predict the bearing capacity of footings. The foundation shown in Fig. 3.3 is assumed to have failed by rotating about a point O, slightly above the ground level. The rotation failure is induced by a uniform pressure (q) and it forms a failure arc with an angle of θ . The foundation is located at a depth ,Z, below ground surface in a soil having a unit weight of γ.

The driving moment about point O is:

$$2qb\ (r\sin\theta - b)$$

The resisting moments about point O are:

1) The moment due to cohesion along the failure arc:

$$r^2\theta c_u$$

2) The moment due to cohesion along the slip surface above the foundation level:

$$Zc_u r\sin\theta$$

3) The moment due to the soil above the foundation level :

$$2Zb\gamma\ (r\sin\theta - b)$$

At equilibrium,

$$2qb\ (r\sin\theta - b) = r^2\theta c_u + Zc_u r\sin\theta + 2Zb\gamma\ (r\sin\theta - b)$$

Then the bearing capacity of the footing is:

$$q_u = \frac{c_u\ (2r^2\theta + 2Z r\sin\theta) + 2Zb\gamma\ (r\sin\theta - b)}{2b\ (r\sin\theta - b)}\ .$$

In bearing capacity analysis using limit equilibrium methods, the three dimensional nature of the real problem is rarely considered. These methods usually employ the assumption of cyclindrical failure surface instead of a nearly spherically shaped one. The degree of error introduced by this two dimensional slip surface assumption is not significance according to Smith (1978), unless the foundation is very deep this will have little effect on the value of the ultimate bearing capacity calculated.

A similar slip surface analysis, but with slices, is also included in this study. The method selected to analyze the rotational failure of footing foundations is Bishop's method which has been described in section 3.3.2.

3.4.3 Methods based on plastic failure theory

In this type of failure, the soil is assumed to be a mass of material having a failure pattern as indicated in Fig. 3.4. The soil wedge ABC which remains intact may be imagined to be pushed down, producing lateral thrusts which overcome the passive resistance of the soil masses AFG and BDF lying to each side of the footing. A state of plastic equilibrium thus exists above the surface GFCDE, the rest of the soil remaining in a state of elastic equilibrium.

Prandtl in 1921 developed an expression for the ultimate bearing capacity of soils under a strip footing located on ground level. The derivation of this expression was based on the theory of plasticity. The curved part of the slip surface was formed from a logarithmic spiral. Under the $\phi=0°$ condition, the curvature of the spiral slip surface becomes circular (Taylor 1948).

In 1943, Terzaghi investigated the problem using somewhat similar assumptions for the mechanism of failure. Allowing for the development of friction between the base of the footing and the adjoining soil, he derived the following expression to estimate the ultimate bearing capacity under a shallow strip footing.

$$q_u = c'N_c + \gamma D N_q + 0.5\gamma B N_\gamma$$

where q_u = ultimate bearing capacity,

c' = cohesion of the soil,

B = width of the footing,

γ = unit weight of the soil,

D = depth to the base of the footing,

N_c, N_γ, N_q = bearing capacity factors.

The bearing capacity factors in this expression depend on the angle of internal friction of the soil. In $\phi=0°$ analysis, the factors become $N_c=5.7$, $N_q=1.0$, $N_\gamma=0$ and $c=c_u$.

The expression will then appear in the following form: $q_u = 5.7c_u + D\gamma$.

Meyerhof (1951) further extended Terzaghi's method to deep and circular foundations: the main differences lie only in the evaluation of the bearing capacity factors. These factors depend on depth, shape and roughness of the footing, as well as the angle of internal friction of the soil.

In the case of a circular foundation in a cohesive soil under undrained conditions, the ultimate bearing capacity (q_u) of the soil, according the Meyerhof (1951), can be defined as follows:

$$q_u = c_u N_c + K_s \gamma D$$

where K_s is the coefficient of earth pressure between the soil and the sides of the foundation.

In a review of bearing capacity theory, Skempton (1951) suggested that the ultimate bearing capacity of a circular foundation on saturated clay under undrained condition is best expressed by the equation:

$$q_u = c_u N_c + \gamma D$$

Skempton's N_c factor is a function of the shape of the footing and the depth/breadth ratio. According to Skempton (1951), the convenient approximation of $N_c = 9$ is often used when $\phi = 0°$ and the footing is located at a depth exceeding two and a half times the width.

Earlier, in 1942, Skempton had proposed a similar expression that takes into account the adhesion between the sides of a foundation and the adjoining soil. The expression is basically the same as the previous one but it has an additional term for the adhesion effect. The ultimate bearing capacity is therefore given by:

$$q_u = c_u N_c + \gamma D + c_a L/A$$

In the $c_a L/A$ term, L/A is the outside perimeter/base area ratio of the foundation and c_a is the adhesion between the soil and the foundation. The remaining terms are the same as in Skempton's (1951) expression.

3.5 STABILITY OF VERTICAL CUTS

The stability of an unsupported vertical cut in totally saturated cohesive material is often analyzed by the critical height method. The critical height (H_{cr}) is the height at which an unsupported vertical cut will collapse due to the weight of the soil mass.

In common practice, the stability of an unsupported vertical cut is evaluated by means of the limit equilibrium method. To simplify the calculations, the surface of the failed cut shown in Fig. 3.5 is assumed to be a plane inclined at an angle θ to the horizontal. The Mohr-Coulomb failure criterion is also assumed to be applicable.

In limit equilibrium analysis, the soil above the slip surface is considered as a free body. The normal stress (σ_n)

and the shear stress (S) are the two unknowns along the failure plane. The critical height of a vertical cut can be expressed as (Chen and Scawthorn 1970):

$$H_{cr} = \frac{4c'}{\gamma} \tan(\frac{\pi}{4} + \frac{\phi}{2})$$

The same expression of the critical height can also be obtained by means of Rankine's theory (Terzaghi and Peck 1967).

When a circular failure surface is assumed, a slight reduction of the critical height would occur. The critical height corresponding to this additional assumption is (Fellenius 1927):

$$H_{cr} = \frac{3.85c'}{\gamma} \tan(\frac{\pi}{4} + \frac{\phi}{2})$$

This critical height is about 5 per cent less than the one obtained according to the plane slip surface assumption.

The above-mentioned expressions used to determine the critical height in vertical cuts are only valid under short term conditions in saturated cohesive soils. When the $\phi=0°$ concept is applied, the critical height becomes:

$$H_{cr} = \frac{4c_u}{\gamma}$$

and for Fellenius' version,

$$H_{cr} = \frac{3.85c_u}{\gamma}$$

A vertical cut of height equal to its H_{cr} will remain stable unless changes in the soil condition occur, such as the accumulation of surface water in open tension cracks, application of surcharge load, redistribution of water and swelling in the soil which subsequently decrease the shear strength, etc. (Chowdhury 1978).

3.6 FACTOR OF SAFETY

In stability analyses, there are many ways in which a factor of safety can be defined. Since a number of different stability methods are involved in this study, it is necessary to explore the different definitions of factor of safety, used in each method.

The factor of safety can be defined in terms of various variables, including height and steepness of slope, pore water pressure, surcharge loading, etc.. The most common basis for defining the factor of safety is to express it as the ratio of the available unit shear stress (S) to the unit shear strength (τ) required to mobilize the soil (Bishop and Bjerrum 1960),

$$F = \frac{\tau}{S}$$

For a second definition of a factor of safety, in the case of an assumed cylindrical surface of sliding, the equation is:

$$F = \frac{\text{RESISTING MOMENT}}{\text{DRIVING MOMENT}}$$

This expression was tranformed by Fellenius (1927), from a shear strength expression used by Geotechnical Commission of the Swedish State Railways (Statens Jarnvagar 1922). He applied this expression to the stability analysis of clay slopes (Fig. 3.6).

In almost all stability methods based on limit equilibrium theories, the factor of safety is assumed to be a constant value along the slip surface although it has been recognized that the stress distribution along a slip surface is always non-uniform. Provided that the normal and the shear stresses are known, a local factor of safety can be defined at each point along the surface as the ratio of shear strength to shear stress. Such factors quite often show a great deal of variation over the length of the slip surface.

In the case of a simple slope, bounded by two horizontal planes and an inclined or vertical plane, Taylor (1948) defined the factor of safety as a ratio between the critical height (H_{cr}) and the actual height (H) of the slope,

$$F = \frac{H_{cr}}{H}$$

Another definition of the factor of safety is used in shallow foundation analyses. This factor of safety is determined as the ratio of the maximum allowable bearing capacity of the soil (q_u') to the applied pressure (q) on the foundation,

$$F = \frac{q_u}{q}$$

In this way, the factor of safety is expressed as a ratio of loads instead of being based on shear strength values or the height of the slope.

3.7 DISCUSSION

This chapter presents a number of commonly used methods for the prediction of the stability of different geotechnical structures. The methods can be divided into two main categories, those based on the theory of limit equilibrium and those based on the plastic failure theory. The limit equilibrium methods used in this analysis are the Bishop's method, the circular arc method, and the critical height method, whereas the plastic failure methods used are the different bearing capacity equations.

All the limit equilibrium methods have two things in common. They treat the soil above the slip surface as a free undeforming body and make assumptions that transform the problem into one which is statically determinate. However, the methods are problematic because the stress-strain relationship of soils is not taken into account in the analyses.

The performance of a slope is often controlled by its allowable deformation. Methods such as the Finite Element Method have been developed to take this into account. However in strain-softening soils, the stress-strain relationship still cannot be included in simple and practical manner in the FEM analysis (Trak 1980).

Another uncertainty involved in a stability analysis is the three-dimensional nature of the problem. In usual procedures, the sliding mass is assumed to be defined by the geometry of the slope and by an assumed slip surface. The influence of the resisting forces acting on the lateral boundaries of the mass is ignored. For cohesive soils, according to Baligh and Azzouz (1975), the end effect can lead to a 20 to 30% increase in factors of safety. This influence depends on the actual dimensions of the slide. When the width of the sliding mass is greater than its length, the error involved in analyzing the problem in two dimensions is small, and the problem can be treated as being in a plane-strain condition.

Both for embankments and excavations, the occurrence and location of tension cracks can hardly be predicted. In excavations, cracks usually result from shrinkage or from tensile stresses associated with a tendency toward a down-slope movement. In the case of embankments, there is also the effect of differential settlement. Cracking nearly always occurs in these geotechnical structures, and the effect of cracking on stability is very important because slip surfaces often start at the bottom of existing cracks. Since there is no reliable way of predicting the extent and depth of possible cracks, any assumptions regarding cracking in the analysis should be conservative (Tschebotarioff 1973).

The actual shape of the rupture surface which will develop is never known unless failure actually occurs. In a stability analysis, the usual assumptions made regarding the shape of slip surfaces are mainly based on the observation of the failures which have previously occurred in a particular region of concern.

In the computation of the factor of safety in stability analysis, one may say that the methods used are at best empirical approaches because the deformation aspect of the soils is not included in the analysis. The factor of safety cannot be known exactly except when the slope has failed, i.e. when it is known to be unity. Therefore the calibration of case histories by back-calculation is necessary to determine the reliability of methods for specific soils and geotechnical problems.

As a result of the limitations of the computational methods discussed this far, sophisticated methods such as FEM have been introduced. Though they may give a better understanding of the behaviour of a soil, at their present stage of development, the results obtained from such methods are not more reliable than those obtained with simple approximations. Therefore, from a practical point of view, there seems to be no additional advantage in employing such complex methods at this time.

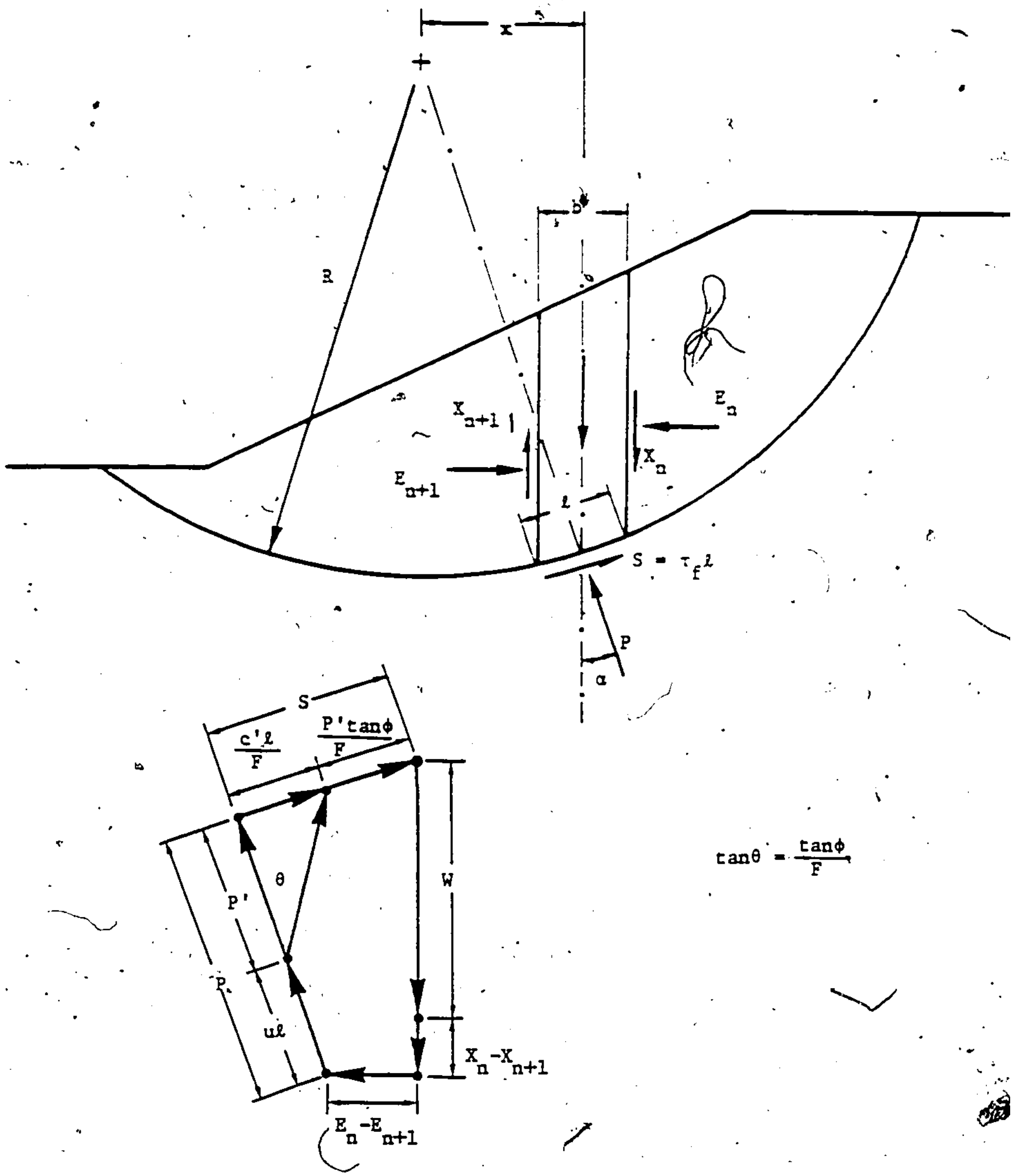
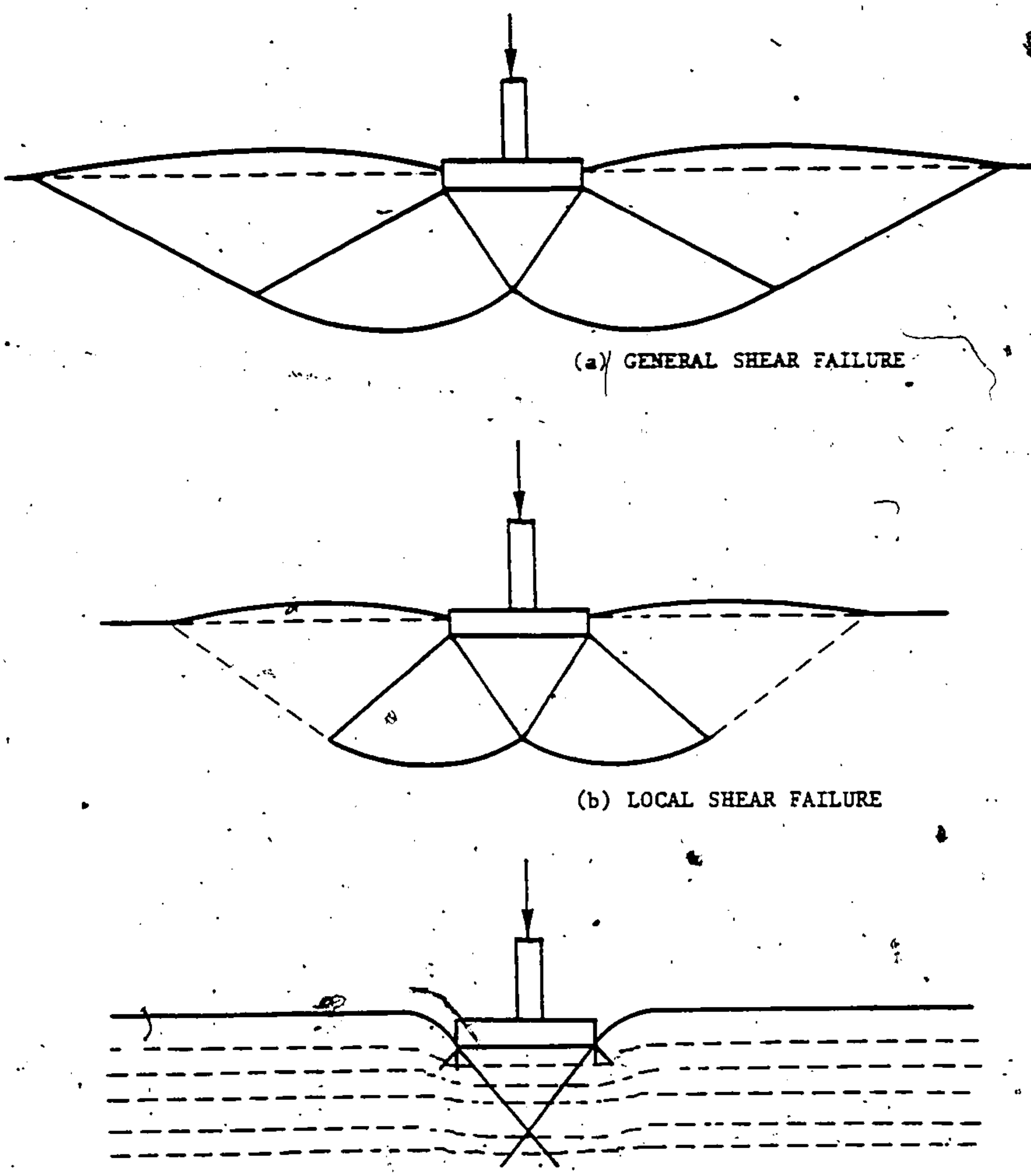


FIG. 3.1 BISHOP'S METHOD

(after BISHOP 1955)



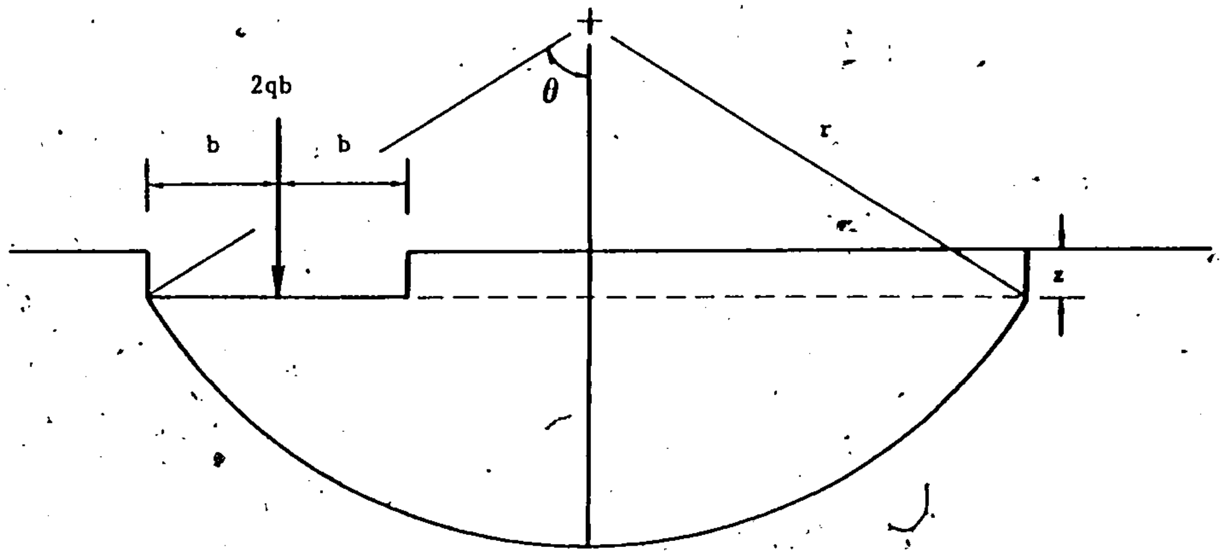
(a) GENERAL SHEAR FAILURE

(b) LOCAL SHEAR FAILURE

(c) PUNCHING SHEAR FAILURE

FIG. 3.2 MODES OF BEARING CAPACITY FAILURE

(after SOWERS 1979)



AT EQUILIBRIUM:

$$2qb\,(r\sin\theta - b) = r^2\,\theta\,c_u + z\,c_u\,r\sin\theta + 2\,zb\gamma(r\sin\theta - b)$$

FIG. 3.3 METHOD OF SLIP CIRCLE ANALYSIS

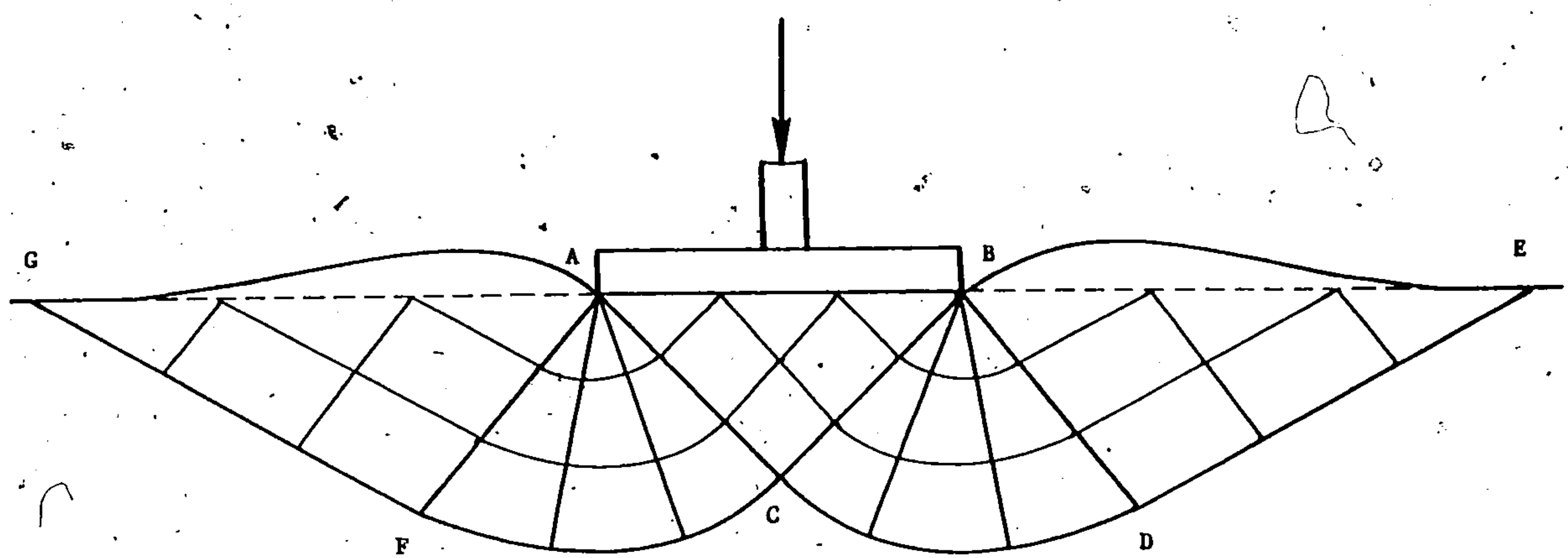


FIG. 3.4 PLASTIC FAILURE, BEARING CAPACITY THEORY

(after TERZAGHI and PECK 1967)

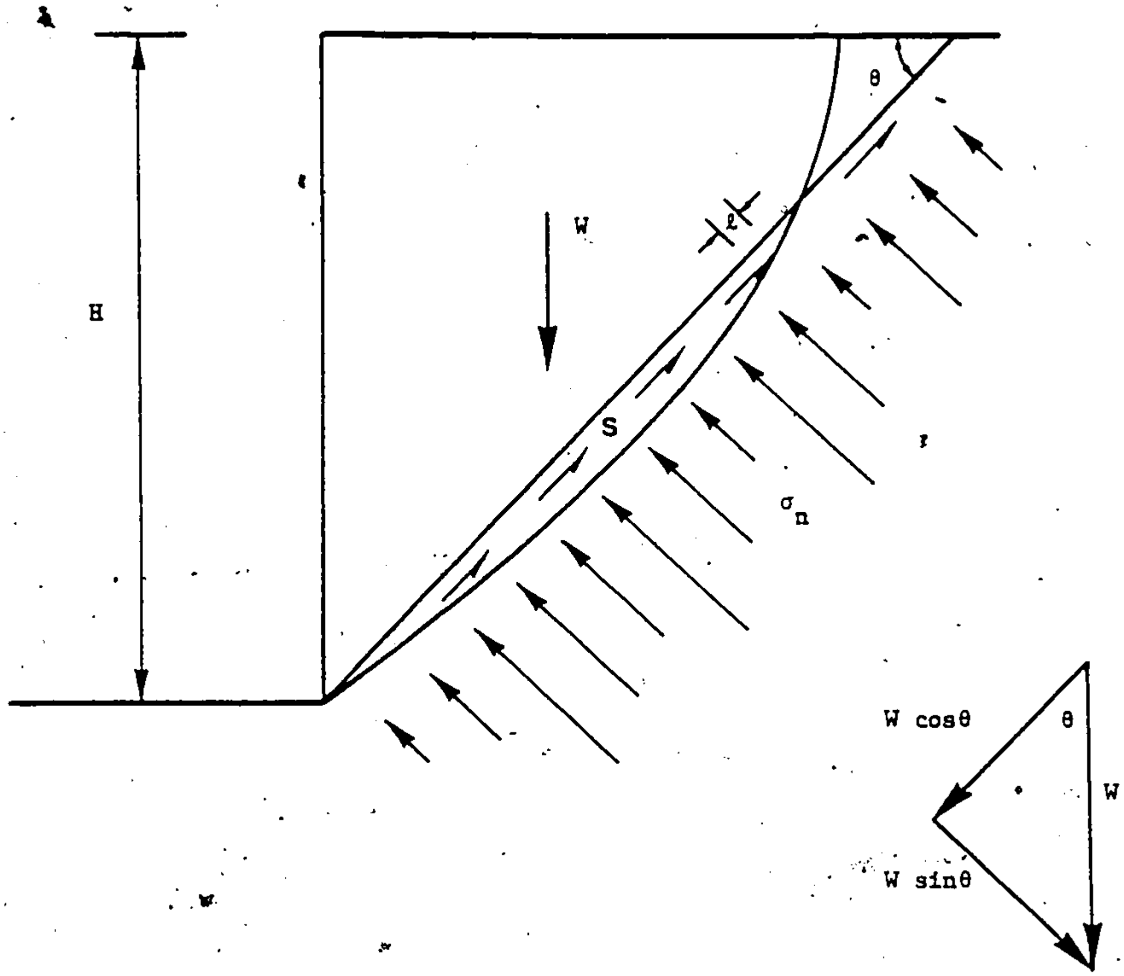
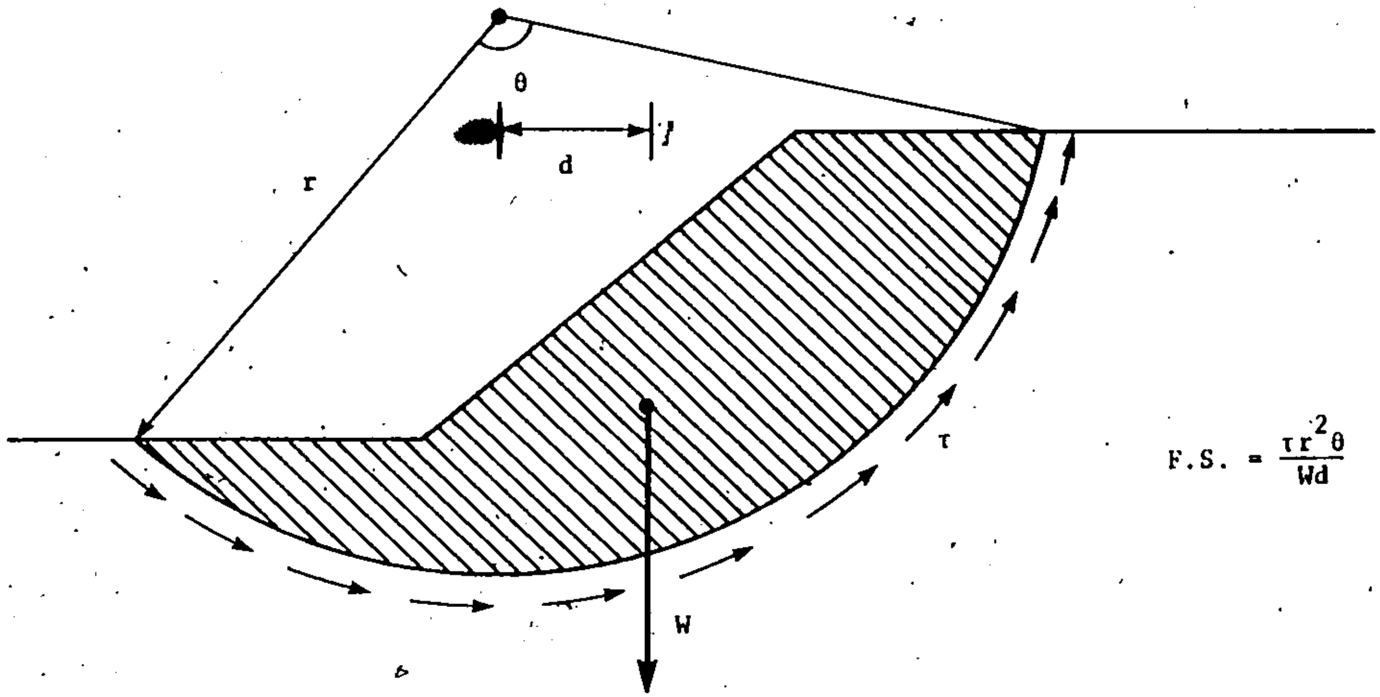


FIG. 3.5 CRITICAL HEIGHT METHOD DERIVED FROM LIMIT EQUILIBRIUM

(after CHEN and SCAWTHORN 1970)



$$F.S. = \frac{\tau r^2 \theta}{Wd}$$

FIG. 3.6 MOMENT EQUILIBRIUM MECHANISM

(after FELLENIUS 1927)

Chapter IV

CASE HISTORIES

4.1 GENERAL

The applicability of Trak et al.'s (1980) mobilized shear strength evaluation technique, discussed in detail in section 2.7, is investigated in this chapter for different types of loading and unloading conditions. Four recently-published case records are discussed: one embankment failure, two silo failures and one excavation failure. For each case, a brief description of the geotechnical properties of the clay deposit is provided, as determined by the original authors. This is followed by a summary of the results of the stability analyses performed by the same authors, and a comparison between these results and the ones obtained in this study. All the analyses were performed in terms of total stresses using the relevant numerical methods described in chapter III.

4.2 EMBANKMENT FAILURE

The $c_u=0.22\sigma'_p$ method was shown by Trak et al. (1980) to give satisfactory results in analyzing the stability of embankments on soft sensitive clay deposits in Eastern Canada. The method was also used for embankment failure on soft clay deposits in other parts of the world, but, with the exception of an embankment on Bangkok clay in Thailand, the results were not as satisfactory. It was therefore decided to verify the conclusions of Trak et al.'s (1980) study by analyzing another embankment failure on a soft Bangkok clay deposit. The case study chosen for this purpose is a test embankment reported by Brand et al. (1976).

4.2.1 Embankment on Bangkok Clay

An extensive research program was conducted by Brand et al. (1976) at a 400 m x 400 m test site at Bangpli, southeast of the city of Bangkok in Thailand. The research program consisted of a large number of Dutch cone and field vane tests carried out beneath a full scale embankment of 20 m width, loaded to failure. The tests were run in order to derive a correlation between the Dutch cone and the field vane test data.

4.2.1.1 Geological and geotechnical description

Bangpli is located near the coastline of the Gulf of Thailand, about 25 Km southeast of Bangkok. Bangpli, like the city of Bangkok, is underlain by a soft clay deposit of marine origin, known as Bangkok clay. This clay extends over the large Chao Phraya Plain and varies in thickness from 10 to 20 m. A typical profile of the subsoil at Bangpli is shown in Fig. 4.1. The subsoil in this region can be divided into the following layers (Moh et al. 1969):

1. weathered zone of dark gray clay which forms an apparently hard crust to a depth of about 4 m;
2. soft, highly compressible, dark gray clay beneath the weathered zone and extending to a depth of about 15 m;
3. stiff, often fissured, light gray and brown clay to a depth of about 20 m;
4. sand and gravel beds with some sandy clay, which occur alternatively to a depth of at least 300 m.

The upper two layers were recognized by Moh et al. (1969) as Bangkok clay which was deposited on the margins of a shallow prograding Tertiary sea. In the early stages of deposition, sedimentation of the clay took place over the alluvial layers of sand and gravel which were deposited before the prograding movements. The sea level rose a second time, and a new clay layer was deposited. Finally the water withdrew completely; the surface of the most recently deposited soft clay was exposed to weathering, and, thus, the desiccated clay crust was formed.

As reported by Brand et al. (1976), the range of natural moisture content of Bangkok clay is between 50 to 150 per cent depending on the depth and the distance from the coastline. The liquidity index of the clay is generally close to 1.0. Leaching of pore fluid has left the clay with a sensitivity varying from 4 to 12.

As shown in the geotechnical profile in Fig. 4.1, the water content and Atterberg limits of the clay at Bangpli indicate a clay stratum with different characteristics between 10 m and 15 m depth. The plasticity and water content of this stratum are different from those of overlying strata because this clay was deposited during a different geological era.

4.2.1.2 Undrained shear strength

Vane tests on Bangkok clay at Bangpli were carried out using a 13 cm x 6.5 cm vane rotated at a rate of 6 degrees per minute (Brand et al. 1976). The undrained shear strength values measured by this vane borer at vertical intervals of approximately 1 m are shown in Fig. 4.1. High shear strength values were recorded in the extensively desiccated clay crust. It was found that the undrained shear strength of the clay decreases with increasing depth, reaching a minimum of 9.8 kN/m² at a depth of 5 m. Below this level, the shear strength increases linearly with depth. The average shear strength of the strata was found to be 24.5 kN/m² (Brand et al. 1976).

4.2.1.3 Preconsolidation pressure

The weathered zone of the clay at Bangpli is easily identified because of its high overconsolidation ratio (OCR). The highest value of OCR in this region, as recorded by Brand et al. (1976), was equal to 3.3. Below the weathered crust and down to a depth of 15 m, the clay is slightly overconsolidated (Fig. 4.1).

All the preconsolidation pressure values in Fig. 4.1 were established from oedometer tests, with a load increment ratio ($\Delta\sigma/\sigma$) of 1.0 and a load duration of 24 hours, on "undisturbed" soil specimens measuring 64 mm in diameter. The specimens were obtained from continous sampling using a 250 mm diameter piston sampler and thin wall sampling tubes at 1.0 m intervals.

4.2.1.4 Fill

The test embankment at Bangpli was a compacted sand fill 3.3 m high, and 9.2 m wide at the top, with a berm 1.2 m high and 10 m wide. The sides of the embankment were sloped at a 2:1 ratio (Fig. 4.2).

In the stability analysis of embankments, one of the important problems is to simulate the fill condition at failure. The assumed mobilized shear strength values of the fill material are usually based on the appearance of vertical tension cracks in the embankment. Brand et al. (1976) observed no such cracks in this case and assumed $c'=0$ and $\phi=37°$ for the mobilized shear resistance.

4.2.1.5 Stability analysis

Analyses by Brand et al. (1976)

The analyses which Brand et al. (1976) performed were based mainly on the Dutch cone measurements. Besides the shear strength obtained from Dutch cone tests, an average field vane strength profile was also used in the stability analyses. All the stability analyses were performed using the Swedish method of slices, also called the Fellenius' method. This method gave a calculated minimum factor of safety of 1.2 for the failed embankment based on field vane results and the authors indicated that the mobilized shear strength associated with the embankment failure was equal to 83 per cent of the field vane strength.

Analyses based on field vane strength values

Because high vane strength values were observed in the desiccated Bangkok clay crust at Bangpli, three assumptions of the crust strength which differ from the average shear strength assumption used by of Brand et al. (1976) have been made and compared in the analyses. The assumptions were: (1) The full strength assumption with an increase of shear strength up to 23.5 kN/m² near the surface; (2) the mid-depth strength assumption with a constant strength of 17.6 kN/m² between 0 and 2 m; and (3) the minimum strength assumption of 12.3 kN/m² throughout the entire weathered stratum (Fig. 4.2).

Since the shape of the actual failure surface was nearly circular (Brand et al. 1976), the ϕ=0° stability analysis was performed using Bishop's simplified method. All the analyses in this case, as well as in other case studies, were carried out using a computer program (Lam and Trak 1982). From the results presented in Table 4.1, it can be seen that the computed factors of safety, including the observed and the critical circles (Fig. 4.3 and 4.4), vary from 0.97 to 1.24 depending on the combination of assumptions. It is also interesting to note that the trend of the factors of safety computed on the basis of different fill strength assumptions, from the fully mobilized fill strength (ϕ_{fill}=37° and c'=0), to the presence of fissures, are almost identical for any given crust strength assumption (Table 4.1).

The results shown in Table 4.1 demonstrate that the shear strength of the subsoil is probably both underestimated and overestimated, when respectively, minimum and maximum crust shear strength assumptions are employed in the stability analyses. The most reliable stability prediction based on back-calculation of this case study was obtained from the analysis which utilized the mid-depth strength assumption. These calculations gave factors of safety very close to unity (Table 4.1).

Stability analyses using $c_u=0.22\sigma'_p$

The stability of the Bangpli embankment was also back calculated using the same geometry of embankment and fill strength assumptions, but the shear strength of the subsoil was taken as equal to $c_u=0.22\sigma'_p$ (Fig. 4.2). The preconsolidation pressure (σ'_p) values used here are the ones given by Brand et al. (1976). In comparing the profile to the vane strength profile, it is of interest to note that the values obtained at each corresponding depth by using the $c_u=0.22\sigma'_p$ method are generally 27 per cent smaller than the uncorrected vane strengths. This percentage is very close to the amount of reduction (30%) used in Bjerrum's correction approach.

The results of the analyses using the $c_u=0.22\sigma'_p$ expression and the positions of the critical slip circles are shown in Table 4.1 and Fig. 4.5, respectively. All the factor of safety calculated are close to unity even in the case of different fill strength assumptions. Also, it can be seen from Table 4.1 that the stability analyses using the $c_u=0.22\sigma'_p$ profile yield values similar to the ones obtained by using the corrected vane strength profile with a mid-depth crust strength assumption. Both methods used for back calculating the stability of the Bangpli embankment after failure gave a factor of safety close to unity. It can be concluded that the $c_u = 0.22\sigma'_p$ method provides satisfactory results in determining the mobilized shear strength of Bangkok clays under embankment loading. This method is also more convenient to use than Bjerrum's method because no arbitrary selection of the crust strength profile is required.

4.3 BEARING CAPACITY FAILURES

In 1973, Bjerrum had shown that the vane strength correction factor (μ) developed for embankments could also be applicable to shallow foundations. Therefore, the expression $c_u = 0.22\sigma'_p$ could also apply to bearing capacity problems as suggested by Trak (1981).

In the following sections, two silo foundation failures are examined. One is in a varved clay deposit at New Liskeard, Ontario; and the other in the slightly overconsolidated marine clay deposit at Vankleek Hill, also in Ontario.

4.3.1 New Liskeard (silo)

A silo located approximately four miles north of New Liskeard in Northern Ontario, Canada, failed in July, 1961. The structure was founded on a soft clay deposit. The failure of this silo was studied by Eden and Bozozuk (1962) and Bozozuk (1977).

4.3.1.1 Geological and geotechnical descriptions

New Liskeard is on the edge of the vast pre-glacial Lake Barlow-Ojibway clay plain in northeastern Ontario, which was deposited during the Pleistocene period. This region is also known as the Little Clay Belt. The subsoil consists mainly of varved silty clays with alternating clay (dark) layers, and clayey silt (light) layers. Such Eastern Canadian varved clay deposits are usually normally consolidated, or slightly overconsolidated (Soderman and Quigley 1965; Lo and Stermac 1965; and Stermac et al. 1967).

As reported by Eden and Bozozuk (1962), two boreholes were made about 4 m from the failed silo outside the failure zone. In one of the boreholes, thin walled samples were taken from depths of 1 to 10 m at about 1 m intervals. Vane tests were made from the 1.2 m level to a depth of 14 m in the other borehole; these tests were carried out using a 10 mm x 55 mm Geonor vane. Additional vane tests were conducted at an undisturbed area approximately 30 m from the foundation.

The soil profile (Fig. 4.6) shows that the subsoil consists of slightly overconsolidated varved clays up to a depth of more than 14 m, with a 1.5 m weathered clay crust. Between the depths of 1.5 m and 6.7 m, the soil was found to be very sensitive.

4.3.1.2 Structure

The silo was built of precast concrete staves, and had a sheet aluminum dome roof. The height was 15 m and the inside diameter 6 m. The silo was constructed on a concrete base ring, with 6.7 m outer and 5.5 m inner diameters. The ring was 1.5 m high and founded at a depth of 1.2 m. Four clay tiles which passed through the concrete ring were installed to drain off the excess silage juices at ground level. The foundation ring was roughly cast, and provided a good adhesion between the soil and the concrete.

According to Eden and Bozozuk (1962), the weight of the structure was estimated to be approximately 55 metric tons. The foundation ring and the soil retained in the ring weighed approximately 52 and 55 metric tons, respectively.

The silo was located about 9 m from the corner of a barn. Before the failure, about 30 cm of topsoil had been removed from one side of the silo in order to pave the barnyard. The paving was to extend from the barn to within 0.9 m of the silo.

4.3.1.3 Failure

The failure occurred on July 24, 1961, one day after the silo was filled to capacity. The loading had started on July 15. On the day of the silo failure, a slight tilt was observed and it became more pronounced toward early afternoon. Finally the silo failed and the structure broke into sections on falling away from the area of the 30 cm topsoil excavation (Fig. 4.8; Eden and Bozozuk 1962).

Prior to failure, it was noted that the drain tiles did not function and that the silage juices had seeped under the foundation ring, bubbling up to the surface near the foundation.

4.3.1.4 Undrained shear strength profiles

As mentioned in section 4.3.1.1, vane tests were made (Eden and Bozozuk 1962) in one of the two boreholes at the site. The shear strength values obtained are shown in the soil profile of Fig. 4.6. The first 1.5 m of the subsoil consists of a weathered clay crust with measured shear strength values of up to 24.5 kPa. Below 1.5 m, to about 6.7 m, the clay is very soft and sensitive with vane strength values gradually increasing with depth. A minimum strength of about 12 kPa is found at a depth of 1.5 m.

A second shear strength profile is derived from the preconsolidation pressure profile by using the expression $c_u = 0.22\sigma'_p$. The preconsolidation pressure profile (Fig. 4.6) indicates that the soil is slightly overconsolidated with OCR values ranging from 1.1 to 1.9. An unusual feature of this profile is the drastic increase in σ'_p values between 4 and 5 m. The shear strength profile computed using these σ'_p values has more or less the same shape as the one measured using the field vane, but smaller in magnitude (Fig. 4.7).

4.3.1.5 Stability analysis

Analyses by Eden and Bozozuk (1962) and Bozozuk (1977)

Bearing capacity analyses were performed by Eden and Bozozuk (1962) using Skempton's (1942 and 1951) and Meyerhof's (1951) bearing capacity equations, and Fellenius' method. The load of the silo at failure was estimated to be 449 metric tons (grass silage: 314 tons, and structures and foundation: 135 tons; Bozozuk 1977). The shear strength value used in the analyses was the arithmetic mean of all the measured shear strength values between the bottom of the footing and a depth below the footing equal to two thirds of its diameter. The average estimated shear strength is 15.6 kPa. The factors of safety calculated using the above methods are shown in Table 4.2: the lowest value corresponding to the bearing capacity equations is 0.97, however a slightly lower factor of safety of 0.94 was obtained by using the Fellenius' method.

Stability analyses based on vane strength

A new stability analysis was performed in this study, based on the vane strength profile (Fig. 4.7) and using Bishop's simplified method. The results are also shown in Table 4.2. It should be noted that the shear strength values were reduced, when applicable, by applying a correction factor of .85 (I_p=36%) as suggested by Bjerrum (1972). The stability analysis results indicate that the use of Bjerrum's vane strength correction factor was necessary to bring the factor of safety closer to unity in the back-calculation.

Limit equilibrium analyses using the Bishop's simplified method gave factors of safety of 1.31 and 1.26, with the maximum and the minimum crust strength assumptions, respectively. In order to make comparisons between different limit equilibrium methods, analyses using the Fellenius' method were also conducted (Fig. 4.9). The factors of safety obtained by this method were much closer to unity than Bishop's simplified method, having magnitudes of 1.07 and 1.03, respectively, for the same crust strength assumptions.

For analyses using bearing capacity equations, Skempton's (1951) equation gave factors of safety of 0.97 and 0.94, with maximum and minimum crust strength assumptions. When Skempton's (1942) equation was used, where soil adhesion is included, it gave factors of 0.99 and 0.95, with the same assumptions. Using Meyerhof's formula, the factors of safety were found to be 1.08 and 1.05 for maximum and minimum crust strengths, respectively. The closest value to unity was obtained by combining Skempton's (1951) equation and the maximum crust strength assumption.

All the factors of safety computed by using Bjerrum's (1972) correction approach (Table 4.2), except the ones determined by the limit equilibrium methods, are very close to unity. This may indicate that it is probably not suitable to use the limit equilibrium method with Bjerrum's correction factor for analyzing bearing capacity failure.

Stability analyses using $c_u=0.22\sigma'_p$

Similar limit equilibrium and bearing capacity analyses, using the shear strength profile determined from the $c_u=0.22\sigma'_p$ expression, were also performed in this study. The results of these analyses, shown in Table 4.2, indicate that the calculated factors of safety have values close to unity, with the exception of Fellenius' method. Comparing these results with the results of the analyses using the vane strength profile, the factors of safety determined by Bishop's method gave values close to the uncorrected ones with maximum and minimum crust strength assumptions. However, when bearing capacity equations were used, they gave values which were closer to the corrected factors of safety than to the uncorrected ones. This phenomenon is attributed to the different assumptions between the limit equilibrium and the bearing capacity analyses and the different ways of defining the factor of safety. But, irrespective of the type of method used, satisfactory results were obtained using the $c_u=0.22\sigma'_p$ method without the need for making any arbitrary crust strength assumptions, except for Fellenius' method.

4.3.2 Vankleek Hill (silo)

A second silo, located at Vankleek Hill suddenly overturned on September 30, 1970. Like the silo described above, it was also founded on a soft clay deposit. But instead of being founded on a varved clay, this silo was founded on a Champlain Sea clay deposit in the St. Lawrence lowlands.

4.3.2.1 Geological and geotechnical description

The site of Vankleek Hill is located on the marine clay deposit which overlies the major part of the Ottawa-St. Lawrence lowlands and is known as the Champlain Sea clay. Although Champlain clays have not been preconsolidated geologically, they have developed a quasi-preconsolidation due to the effects of aging and possibly, thixotropic hardening. The geological origin, formation and extent of Champlain clays have been studied in detail by Crawford (1965) and Soderman and Quigley (1965).

Continuous "undisturbed" 50 mm diameter piston samples were taken by Bozozuk (1972), using a NGI soil sampler about 18 m away from the silo. The sampling extended to a depth of 10 m. In situ shear strength values were measured by means of a 55 mm x 110 mm Geonor vane to a depth of 19.6 m, which was the depth of refusal.

The soil profile shown in Fig. 4.10 indicates an organic topsoil, about 0.3 m thick, overlying a 3 m desiccated red brown silty clay. The layer between the depths of 3.4 m and 10.3 m consists of soft grey silty clay, except for a thin silt layer at a 4.5 m depth. The grey silty clay contains some black mottling commonly found in the Champlain clay deposits of that region. The Atterberg tests indicated that the average plasticity index of the layer is 36 per cent. Except for the silt layer at the 4.5 m level, 85 to 90 per cent of the soil particles fall within the clay size fraction. Sand is completely absent from the soil profile. The average initial void ratio is approximately 1.5 for the desiccated crust and about 2.5 for the soft clay lying below.

Bozozuk (1972) conducted a number of consolidation tests on specimens of 2.5 cm diameter with a loading ratio ($\Delta\sigma/\sigma$) of 1/2. It was determined that the soil is slightly overconsolidated. A detailed preconsolidation pressure profile is shown in Fig. 4.10.

4.3.2.2 Structure

The silo was constructed in May, 1970. The structure was 21 m high and consisted of a 6 m inside diameter reinforced concrete tube with walls 16.5 cm thick. A 10 cm thick concrete loading chute, cast monolithically with the main walls on the outside, extended for the full height of the silo. The structure was constructed on a concrete ring foundation having a 7.2 m outer, and a 5.4 m inner diameter.

No floor was provided. The ring was placed directly in an excavated trench, 1.2 m in depth.

The average unit weight of the concrete was determined by Bozozuk (1972) to be equal to 22.1 KN/m^3. Based on this result, the weight of the structure was estimated to be 166 metric tons, and the ring foundation, 49 metric tons.

The silo was located at the corner of a concrete apron, 21 m wide, 30 m long and 10 cm thick, placed on 20 cm of fill. The apron was approximately 8 m away from the main barn.

4.3.2.3 Failure

On September 30, 1970, the silo suddenly overturned. During the summer before the failure, the silo had been filled with about 15 m of hay silage. The silo was later reloaded with corn silage. It was estimated by Bozozuk (1972) that the weight of silage at the time of failure was about 464 metric tons, because a considerable quantity of silage juice from the corn had escaped through the drainage ports at the base of the silo, causing a reduction of the original weight.

When the silo collapsed, it tipped in the direction of the loading chute onto the paved apron. On the opposite side of the silo, the soil heaved beyond the apron.

The position of the ring foundation following the bearing capacity failure is shown in Fig. 4.11. The final position of the foundation had an inclination of 50 degrees from the horizontal, which is identical to that measured in the New Liskeard failure. One end of the silo heaved approximately 1.8 m, while the opposite end sank 4 m (Bozozuk 1972). The quantity of the soil heaved could not be measured because it had been removed prior to investigation. From the position of the failed silo, Bozozuk (1972) constructed the circular failure surface shown in Fig. 4.11.

4.3.2.4 Undrained shear strength profiles

As mentioned previously, after the silo failed, vane tests were conducted by Bozozuk (1972) at the site. The results of the tests are shown in the soil profile (Fig. 4.10). The shear strength values vary from a high of about 98 kN/m² in the desiccated crust at a depth of 1.5 m, to a minimum of 9.8 kN/m² in the soft grey silty clay at 3.7 m depth. From that point, the strength increases gradually with increasing depth to about 78.5 kN/m² at 18 m.

According to Bozozuk (1972), the clay is extremely sensitive, with a range of sensitivity from 6 to 22 for the desiccated crust and from 20 to over 100 for the rest of the deposit.

The shear strength profile obtained from the expression $c_u=0.22\sigma'_p$, is shown in Fig. 4.12. The preconsolidation pressures were derived from oedometer test results (Bozozuk 1972). This profile has a similar shape to the field vane strength profile, but the shear strength values are smaller.

In the desiccated crust, these values ranged from 30 to 40 per cent of the in situ vane shear strength values. In the silty clay below 4 m depth, the difference between the $c_u=0.22\sigma'_p$ and the in situ vane strength values remains approximately constant, the former being 76 per cent of the latter (Fig. 4.12). The minimum strength of the $c_u=0.22\sigma'_p$ profile was found between 3.4 m and 4.3 m depth, and is approximately equal to 11 kN/m².

In addition to these profiles, another shear strength profile was also constructed by Bozozuk (1972) using the results from anisotropically consolidated undrained triaxial tests. The average shear strength value was found to be 63 per cent of the in situ vane strength.

4.3.2.5 Stability analyses

Analyses carried out by Bozozuk (1972)

The silo failure was analyzed by Bozozuk (1972), using three different profiles of undrained shear strength: (1) the in situ shear strength measured by the field vane, (2) the field vane strength corrected for the inclination of the failure surface by means of laboratory vane tests at various inclinations, and (3) the triaxial (CAU) strength. The average shear strength values of these profiles are 27.1, 23.9, and 17.1 kPa, respectively.

The same bearing capacity formulae as the ones used in the New Liskeard analyses were used to analyze this silo failure. According to Bozozuk (1977), the total weight of the structure including the weight of the silage was estimated at 672 metric tons. The factors of safety computed by Bozozuk (1972), using different formulae, are shown in Table 4.3. High factors of safety, ranging from 1.10 to 1.24, were found when using the average in situ vane strength in the analyses.

Vane strength analysis

Similar analyses were performed in this study. But instead of employing the average shear strength of the subsoil, the shear strength value was taken as the weighted average along the slip surface as determined by Bozozuk (1972). Also in these analyses, three different crust strength assumptions were considered. These assumptions were: (1) the maximum crust strength, (2) the minimum crust strength, and (3) the mid-depth crust strength, as described in section 4.2.1.5. The results are shown in Table 4.4.

In addition to the bearing capacity formulae of Skempton (1942 and 1951), and Meyerhof (1951), the failure of the silo was analyzed by employing two limit equilibrium methods; the ordinary and Bishop's simplified methods. The results of these analyses are also shown in Table 4.4.

The factors of safety determined using the Skempton (1951) bearing capacity equation were smaller in magnitude than the values calculated by means of the other two bearing capacity equations. For the minimum, the mid-depth, and the maximum crust strength assumptions, the Skempton equation gave factors of safety of 0.80, 1.12, and 1.52, respectively. These values, except the one based on the assumption of a minimum crust strength, indicate an overestimation of the factor of safety if the vane shear strength is used in the calculations. However, when the Bjerrum's correction factor of .86 (I_p =36%) is applied in these analyses, the last two factors of safety are reduced to 0.96 and 1.31, respectively.

The factors of safety against bearing capacity failure were also estimated using Meyerhof's equation, but factors much greater than unity were obtained.

One can observe that the best results are obtained from the bearing capacity analyses when the corrected mid-depth crust shear strength assumption is combined with Skempton's equations.

The results obtained using limit equilibrium methods (Fig. 4.13) generally gave higher values when the maximum and mid-depth crust strength assumptions were used (Table 4.4). Even when the Bjerrum's correction factor was applied, the factor of safety associated with the maximum crust strength assumption was reduced by only a small magnitude.

Stability analyses using $c_u=0.22\sigma'_p$

A stability analysis of general failure of the Vankleek Hill silo was also conducted as above, but this time based on shear strength values estimated by the expression $c_u=0.22\sigma'_p$. For a comparison with the results of analyses carried out by Bozozuk (1972), two methods of estimating the shear strength of the subsoil were used. First, the shear strength was expressed as an average shear strength of the subsoil. Secondly, a weighted average of the shear strength along the actual slip surface of the silo was calculated. The results of these analyses are shown together with the ones of previous analyses, in Tables 4.3 and 4.4. All estimates of the factor of safety, except the ones calculated by means of Skempton's (1942) and Meyerhof's (1951) formulae which use the average shear strength of the subsoil and the one calculated using Meyerhof's formula with the average shear strength along the slip surface, have values close to unity. These analyses show that the expression $c_u=0.22\sigma'_p$ is quite adequate to obtain the mobilized shear strength of the soil under a shallow foundation loading.

4.4 VERTICAL CUT FAILURE

Having shown some results suggesting the applicability of his vane strength correction method to bearing capacity failures (Fig.1.2), Bjerrum (1973) also proposed that the same approach could be applied to excavation problems (Fig.1.3). Since the c_u=$0.22\sigma'_p$ method was derived from the same principles as in Bjerrum's method, it should also be applicable to cuts and unsupported excavations (Trak 1981). The following is a case study of the failure of an unsupported vertical cut in soft clay, analyzed by this approach.

4.4.1 Welland Canal

A 13.4 km long cut for the realignment of the southern half of the old Welland canal necessitated the excavation of about 3.2×10^7 cubic meters of earth. To ensure the safety of the excavation, a field test to study the behaviour of deep vertical cuts in soft clay deposits was conducted by Kwan (1971). During the test program, in situ shear strength was recorded and the pore pressure behaviour and creep characteristics were studied.

4.4.1.1 Geological and geotechnical description

Welland is located in Ontario, about 17 km west of Niagara Falls. The canal is 106.7 m wide with a water depth of 9.1 m. It joins Lake Erie to Lake Ontario. The whole region lies on an area called the Haldimand Clay Plain.

A typical subsoil profile of Welland is shown in Fig. 4.14. The overburden is about 27.4 m thick, and is composed of lacustrine clays, clay tills and a non-plastic till. Two different kinds of lacustrine clay, stratified and non-stratified clays, can be found. A detailed description of the subsoil of this region can be found in Kwan (1971):

1. a weathered and desiccated lacustrine silty clay layer with stratified clay bands extends to a depth of about 6 m. Closed fissures exist throughout the whole layer;
2. a reddish-brown clayey silt layer about 7.5 m in thickness containing sand and subangular gravels extends to a depth of 12.5 m;
3. a layer of stratified clay with $w_n = 49\%$, $w_L = 62\%$ is found down to a depth of 15 m;
4. a reddish-brown, occasionally layered lacustrine silty clay with gravel is found between 15 and about 18 m;
5. a 2 m thick stratified clay layer similar to the one mentioned in point 3 is found at 18 m;
6. finally, the last two strata are of about 3 m in thickness and are composed of silt and clay in the top stratum, and a sandy silt layer in the lower one.

All these materials were probably deposited during the Pleistocene age.

As it can be seen from the soil profile in Fig. 4.14, the amount of clay content in the soil increases with depth until, at 16 m, the homogenous lacustrine silty clay layer is reached.

4.4.1.2 Undrained shear strength and preconsolidation pressure

A number of vane tests were carried out by Kwan (1971). The results of these tests are shown in Fig. 4.14, and indicate that the deposit is characterized by an almost constant shear strength value of about 48 kPa, except for the top 6 m where higher values were measured.

As shown in Fig. 4.14, the preconsolidation pressure profile indicates that the Welland clay above 10 m is slightly overconsolidated with an OCR of 1.8. For the clay below the 10 m level, the preconsolidation pressure increases gradually with depth.

4.4.1.3 Excavation

The excavation was carried out in the late winter of 1967. It involved the removal of the top 5.2 m of desiccated layer, followed by instrumenting and excavating the trench. The final stage, when the trench was cut to an average depth of 9.1 m with a length of 15.2 m, took about 4 days. To ensure no 'end effects', two vertical trenches were cut along the side of the instrumented trench (Fig. 4.15).

During the entire excavation period, precipitation was low and daily temperatures averaged -8°C, rarely dropping below -18°C (Kwan 1971).

4.4.1.4 Failure

The failure of the trench occurred on the 22nd of February, four days after the end of excavation. The day before the failure occurred, a sump was excavated on top of the vertical block in an attempt to break the frozen surface. Water was pumped into the sump. It was found that all the water seeped to the side trenches through a hair line crack which had developed across the sump. This first tension crack was located at a distance of 6.4 m from the vertical face of the trench. A post-failure measurement indicated that this crack had a depth of 3.7 m.

Minutes before the collapse of the trench, cracking noises of increasing volume were heard. It was reported that the trench seemed to fail like a "free-falling body". The final rupture surface was located at a distance of about 4.5 m behind the ~~vertical~~ face instead of continuing from the first tension crack. The break line on the top of the vertical block was semi-elliptic in shape, whereas the first tension crack ran parallel to the crest of the vertical slope (Fig. 4.15).

According to the post-failure measurements taken by Kwan (1971), the width of the fissure was 1.5 m, and its depth was estimated to be 6.1 m. A thin layer of organic substances and roots was found in the fissure.

4.4.1.5 Stability analyses

Stability analyses by Kwan (1971)

Kwan (1971) carried out various back-calculations to analyze the stability of the vertical cut failure in Welland clay. Stability methods such as critical height ($H = 4c_u/\gamma$) and conventional slip circle methods (Fig. 4.16) were used in the back-calculations. The shear strength values used in these calculations were obtained from triaxial quick tests, and the results of these calculations are shown in Table 4.5.

Stability analyses based on vane strength

Stability analyses were conducted in this study using the critical height equations and the simplified Bishop method. The results of these computations are shown in Table 4.5. The shear strength profile of the subsoil as determined by vane testing (Kwan 1971) was used in Bishop's simplified method. According to Bjerrum's (1972) correction approach, no correction on the vane shear strength values is required because the plasticity index of the clay at Welland is 20% (Fig. 1.1). This method yielded a factor of safety of 0.61 showing that this limit equilibrium method does not seem to give an appropriate estimate of the failure of the Welland vertical cut.

The critical height equation ($H = 4c_u/\gamma$) was also used to evaluate the stability of the cut. An average vane strength of 47.9 kPa and a weighted average of the unit weight of the soil, within the range of the 9.6 m cut, were employed in the analysis. Consequently, a critical depth of 10 m was obtained and this yielded a satisfactory safety factor of 1.04. On the other hand, a safety factor of 0.99 was obtained when the Fellenius modified version of this equation ($H = 3.85c_u/\gamma$) was used.

<u>Stability analyses based on c_u=0.22σ'_p</u>

The stability of the Welland vertical cut was also analyzed using the critical height method with the vane shear strength profile replaced by the profile evaluated from the expression c_u=0.22σ'_p. This profile is shown in Fig. 4.17. The preconsolidation pressures used here were based on the values given by Kwan (1971).

The c_u=0.22σ'_p profile exhibits a different shape than the vane strength profile (Fig. 4.17). The shear strength decreases from 57 kPa at the top of the cut to 24 kPa, 3.7 m below; then the strength increases drastically with further depth. The average c_u=0.22σ'_p value obtained throughout the 9.7 m cut is 47.4 kPa.

The factor of safety evaluated by means of the critical height equation was 1.01; meanwhile the modified critical height equation yielded a value of 0.98. The same profile was also used in conjunction with Bishop's simplified method

(Fig. 4.16). The factor of safety obtained from this method was 0.64.

4.5 DISCUSSION

In this chapter, several case histories dealing with failures under different loading and unloading conditions have been presented. The following is a brief discussion of each of the case histories.

4.5.1 Bangpli embankment

As mentioned in section 4.2.1.5, Brand et al. (1976) have conducted a stability analysis of the test embankment at Bangpli. They obtained a relatively high factor of safety using an average field vane strength value and concluded that the vane strength value overestimated the mobilized shear strength by 17%. Based on Bjerrum's correction approach, the overestimation of the mobilized shear strength at Bangpli (I_p=69%) would be 30%. This implies that the Bjerrum correction factor is more conservative than that of Brand et al. (1976). But in fact, the stability analyses based on Bjerrum's corrected vane strength profile yield satisfactory results showing that the mobilized shear strength overestimation was about 30% instead of 17%.

In the Bangpli test program, a number of Dutch cone tests were also conducted. From the analyses using these values, Brand et al. (1976) found that the shear strength obtained by the Dutch cone also overestimated the mobilized shear strength of Bangkok clay. Two extreme values for the factor of safety were obtained. They are 0.6 and 16, depending on the type of resistance measured, i.e. either based on cone resistance or local friction. Therefore, in order to employ the results obtained from this in situ testing method, further calibration is necessary.

4.5.2 Bearing capacity failures

Both the New Liskeard and Vankleek Hill silo failures gave an opportunity to investigate the applicability of the $c_u=0.22\sigma'_p$ method to shallow foundation loading. The results show that the $c_u=0.22\sigma'_p$ expression is quite adequate to represent the mobilized shear strength of the subsoil.

It seems that, in case of New Liskeard silo failure, the desiccated crust did not have much influence on the stability calculations. A possible explanation is that almost two thirds of the crust on the silo site was excavated in order to construct the foundation base ring.

Also at New Liskeard site, because the desiccated crust is relatively thin with respected to the underlying soft clay, it is therefore a perfect case to show the relationship between mobilized shear strength and preconsolidation pressure of the subsoil. Fig. 4.18 consists of a profile of $\mu c_u/\sigma'_p$ ratios showing that the average value of $\mu c_u/\sigma'_p$ is approximatly 0.23. This indicates the expression $c_u=0.22\sigma'_p$ can be adequately used to describe the mobilized shear strength of New Liskeard varved clays at failure.

It was found that in both silo failure analyses, the limit equilibrium methods gave relatively low factors of safety compared to the bearing capacity equations. This discrepancy can be attributed to different assumptions between the two types of methods.

4.5.3 Vertical cut at Welland

In the analysis of the Welland canal test failure, the limit equilibrium methods gave very conservative factors of safety, whereas the critical height equations, both the original and the modified ones, gave consistently realistic results. This indicated that critical height equations for the analyzed cases are more suitable for vertical trench excavation analyses than are limit equilibrium methods.

One possible explanation for the low factors of safety obtained by the limit equilibrium methods is that incorrect positions of the tension cracks were recorded. As indicated by Kwan (1971), all the measurements were done along the northern end of the vertical slope (Fig. 4.15), and they were only roughly measured following the failure. As a result of this, the actual precise depth and position of the cracks are unknown.

After the failure of Welland vertical cut, Kwan (1971) observed that the closest distance of the final tension crack was equal to 2.4 m (based on the measurements of the distance in the northern side-trench). However, in Fig. 4.15, an average value of 4.1 m, which reflects the distance of the crack along the full length of the cut can be observed. According to Terzaghi (1941), in estimating the distance of the final tension crack from the vertical cut, the maximum tension stress behind the vertical surface is located at a distance equal to half of the critical depth. Therefore, tension cracks are more likely to occur at this location. Accordingly, in the Welland canal failure, the location of the crack should have been about 4.9 m away from the vertical face of the cut. Comparing this with the average distance observed (Fig. 4.15), the difference is about 15 per cent.

In 1846, Alexandre Collin observed that failure surfaces in most cohesive soils take the shape of a cycloidal curve instead of an inclined plane as suggested by Coulomb (Collin 1846). A numerical treatment assuming this kind of rupture surface has been derived by Ellis (1973) to analyze the stability of vertical and near vertical slopes in homogenous and isotropic soils. In the analysis of the Welland canal failure, the potential and the actual slip surfaces were compared with the curves generated by the following cycloidal function (Fig. 4.19):

$$x = a\,(\theta - \sin\theta)$$

$$y = a\,(1 - \cos\theta)$$

where 'a' is the functional coeffient and θ is the angle to the function created by the circle with the radius 'a' (Fig 4.20). A good correspondance was found between the potential slip surface described by Kwan (1971) and the generated curve (Fig. 4.19). For the actual slip surface (Fig. 4.21), only the bottom two thirds of the slip surface matched the cycloidal curve. The upper part of the curve was found to intersect the top surface at a distance approximate 5 m behind the vertical face of the cut (a=40 and 41, Fig.4.21), which is very close to the maximum distance of the crack (5.3 m), see Fig. 4.15. These results would seem to indicate that the use of cycloidal arcs as possible failure surfaces in vertical cuts would give more realistic results.

TABLE 4.1

COMPUTED FACTORS OF SAFETY OF BANGPLI EMBANKMENT

FILL BEHAVIOUR ASSUMPTIONS	CIRCLE	VANE STRENGTH PROFILE WITH DIFFERENT ASSUMPTIONS OF CRUST STRENGTH: MINIMUM	MID-DEPTH	FULL	$c_u = 0.22\sigma'_p$
$\phi = 37°$	C	1.16	1.39 (0.97)	1.57 (1.10)	0.93
	F	1.23	1.47 (1.03)	1.66 (1.16)	0.96
FISSURE	C	1.26	1.49 (1.04)	1.64 (1.15)	0.97
	F	1.30	1.57 (1.10)	1.77 (1.24)	1.01

C = CRITICAL CIRCLE

F = OBSERVED FAILURE ARC

(   ) = CORRECTED VALUE BASED ON BJERRUM'S (1972) METHOD

TABLE 4.2

COMPUTED FACTORS OF SAFETY OF THE NEW LISKEARD SILO

METHOD OF ANALYSIS	AVERAGE VANE STRENGTH	VANE STRENGTH WITH DIFFERENT CRUST STRENGTH ASSUMPTIONS				$c_u = 0.22\sigma'_p$
		MAX. CRUST STRENGTH		MINI. CRUST STRENGTH		
BISHOP METHOD	0.94	1.31	(1.12)	1.26	(1.06)	1.02
CIRCULAR ARC(2)	0.94(1)	1.07	(0.97)	1.03	(0.88)	0.89
SKEMPTON (1951)	0.97(1)	1.14	(0.97)	1.10	(0.94)	0.94
SKEMPTON (1942)	1.10(1)	1.16	(0.99)	1.12	(0.95)	0.96
MEYERHOF (1951)	1.09(1)	1.27	(1.08)	1.23	(1.05)	1.04

(1) = TAKEN FROM EDEN AND BOZOZUK (1962)

(   ) = CORRECTED VALUE BASED ON BJERRUM'S (1972) METHOD

(2) = STABILITY METHOD WITHOUT SLICES

TABLE 4.3

COMPUTED FACTORS OF SAFETY WITH AVERAGE SHEAR STRENGTH VALUES, VANKLEEK HILL SILO

METHOD	UNDRAINED SHEAR STRENGTH	
	FIELD VANE	$c_u = 0.22\sigma'_p$
SKEMPTON (1951)	1.10* (0.95)	1.08
SKEMPTON (1942)	1.13* (0.97)	1.10
MEYERHOF (1951)	1.24* (1.07)	1.20

* TAKEN FROM BOZOZUK (1972)

(   ) CORRECTED VALUE BASED ON BJERRUM'S (1972) METHOD

TABLE 4.4

COMPUTED FACTORS OF SAFETY OF THE VANKLEEK HILL SILO

	VANE SHEAR STRENGTH WITH DIFFERENT CRUST STRENGTH ASSUMPTIONS			$c_u = 0.22\sigma'_p$
	MAXIMUM	MID-DEPTH	MINIMUM	
BEARING CAPACITY EQUATIONS:				
SKEMPTON (1951)	1.52 (1.31)	1.12 (0.96)	0.80 (0.68)	1.05
SKEMPTON (1942)	1.56 (1.34)	1.25 (1.08)	0.82 (0.57)	1.07
MEYERHOF (1951)	1.71 (1.47)	1.37 (1.18)	0.89 (0.76)	1.17
LIMIT EQUILIBRIUM METHODS:				
CIRCULAR ARC(1)	1.38 (1.19)	1.12 (0.96)	0.75 (0.64)	0.96
BISHOP	1.59 (1.37)	1.24 (1.07)	0.76 (0.66)	1.05

(   ) = CORRECTED VALUE BASED ON BJERRUM'S (1972) METHOD

(1) = STABILITY METHOD WITHOUT SLICES

TABLE 4.5

COMPUTED FACTORS OF SAFETY OF WELLAND EXCAVATION

	SHEAR STRENGTH		
	TRIAXIAL QUICK TEST (Kwan 1971)	FIELD VANE	$c_u = 0.22\ \sigma_p'$
BISHOP	0.81	0.61*	0.64*
CRITICAL HEIGHT METHOD:			
$H = \frac{4\ c_u}{\gamma}$	0.80	1.04	1.01
$H = \frac{3.85\ c_u}{\gamma}$	0.77	0.95	0.98

*TENSION CRACK IS TAKEN INTO ACCOUNT.

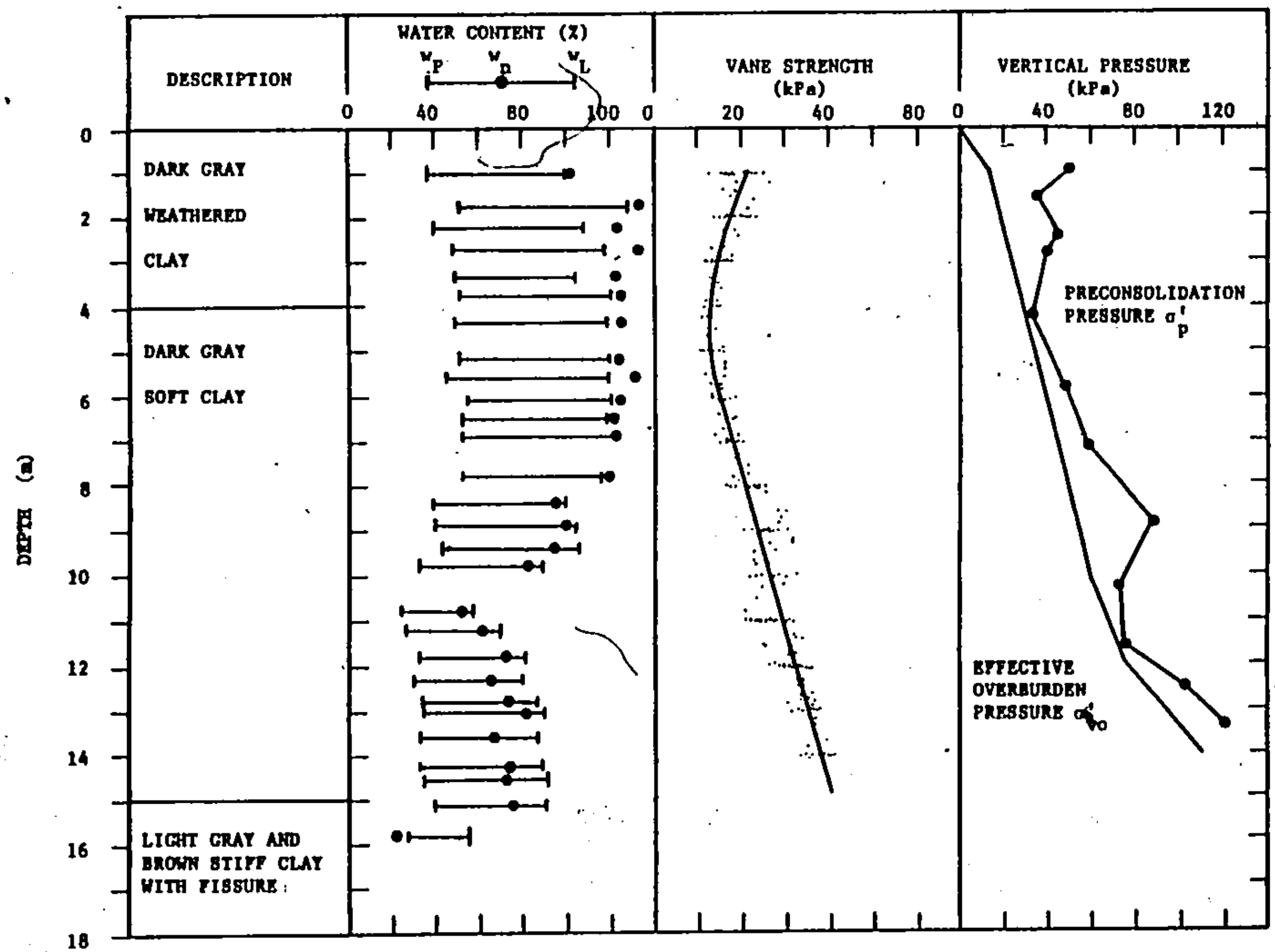


FIG. 4.1 SOIL PROFILE AT BANGPLI

(after BRAND ET AL. 1976)

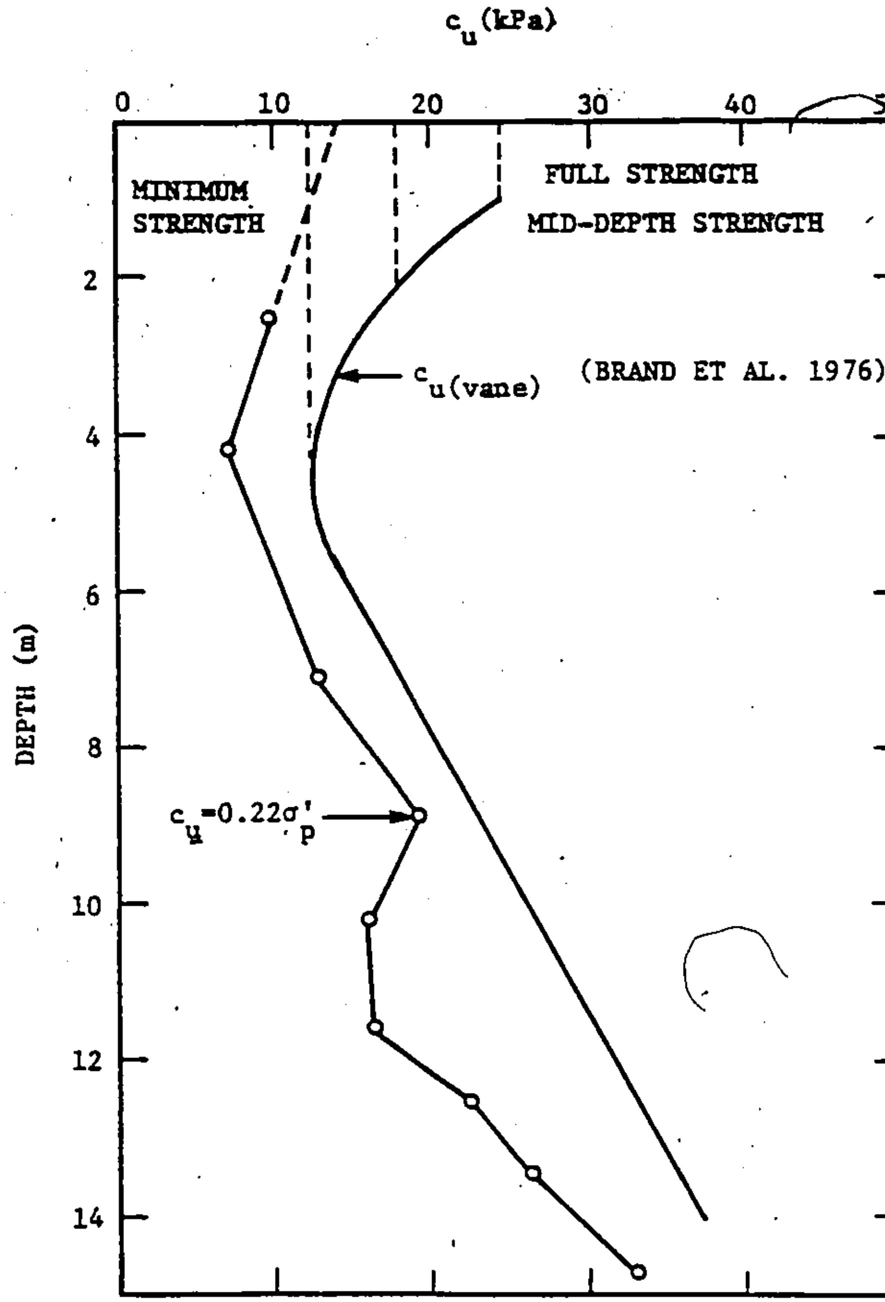


FIG. 4.2 STRENGTH PROFILES AT BANGPLI

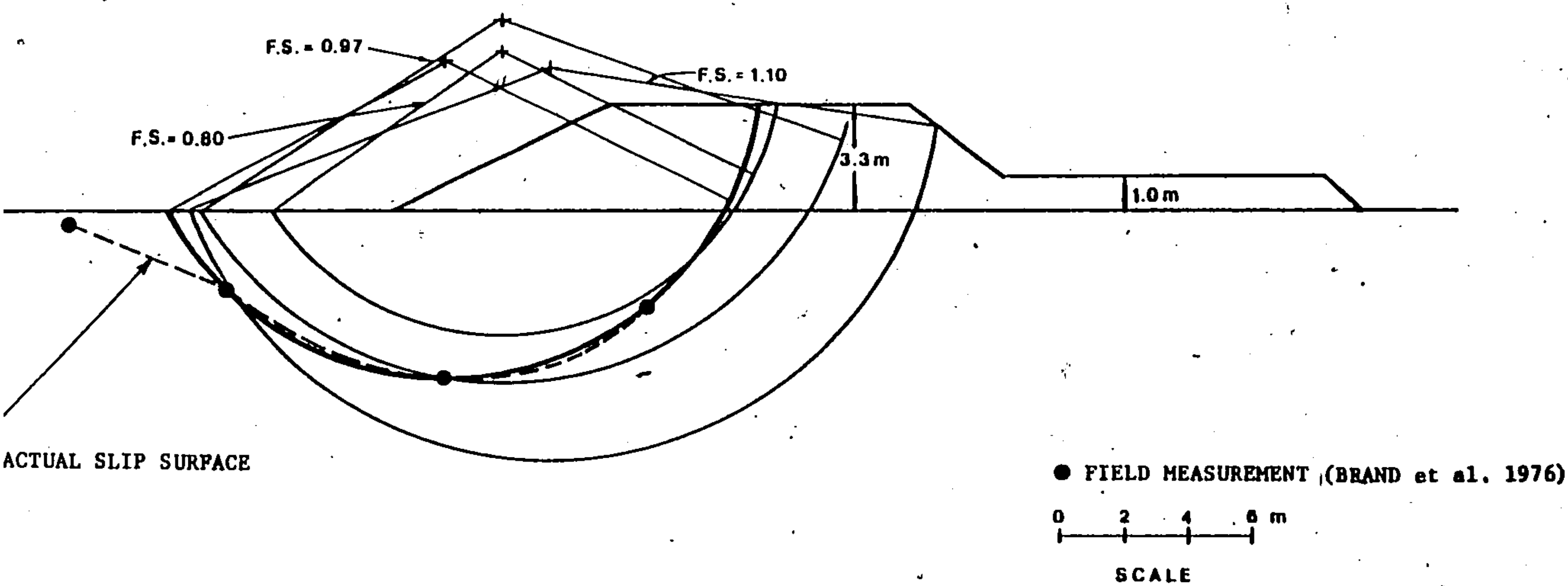


FIG. 4.3 CRITICAL CIRCLES DETERMINED USING VANE STRENGTH PROFILE, BANGPLI EMBANKMENT

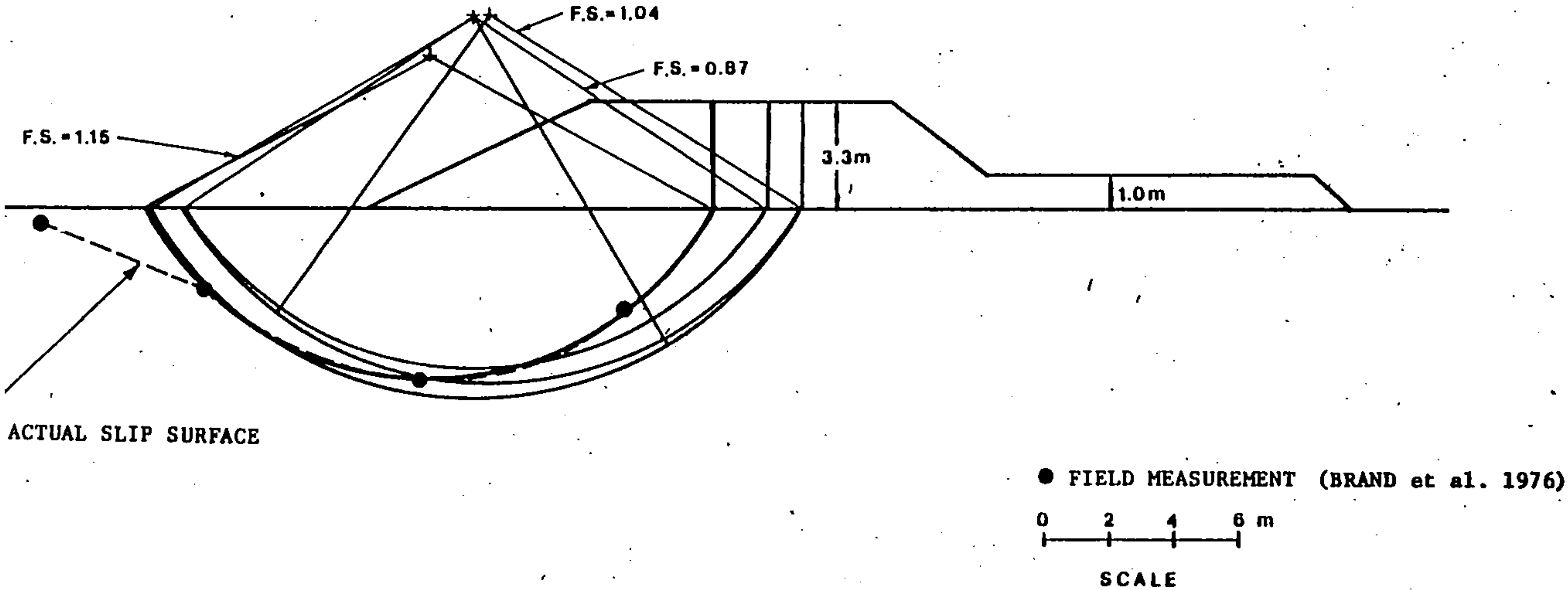


FIG. 4.4 CRITICAL CIRCLES DETERMINED USING VANE STRENGTH PROFILE WITH TENSION CRACK ASSUMPTION, BANGPLI EMBANKMENT

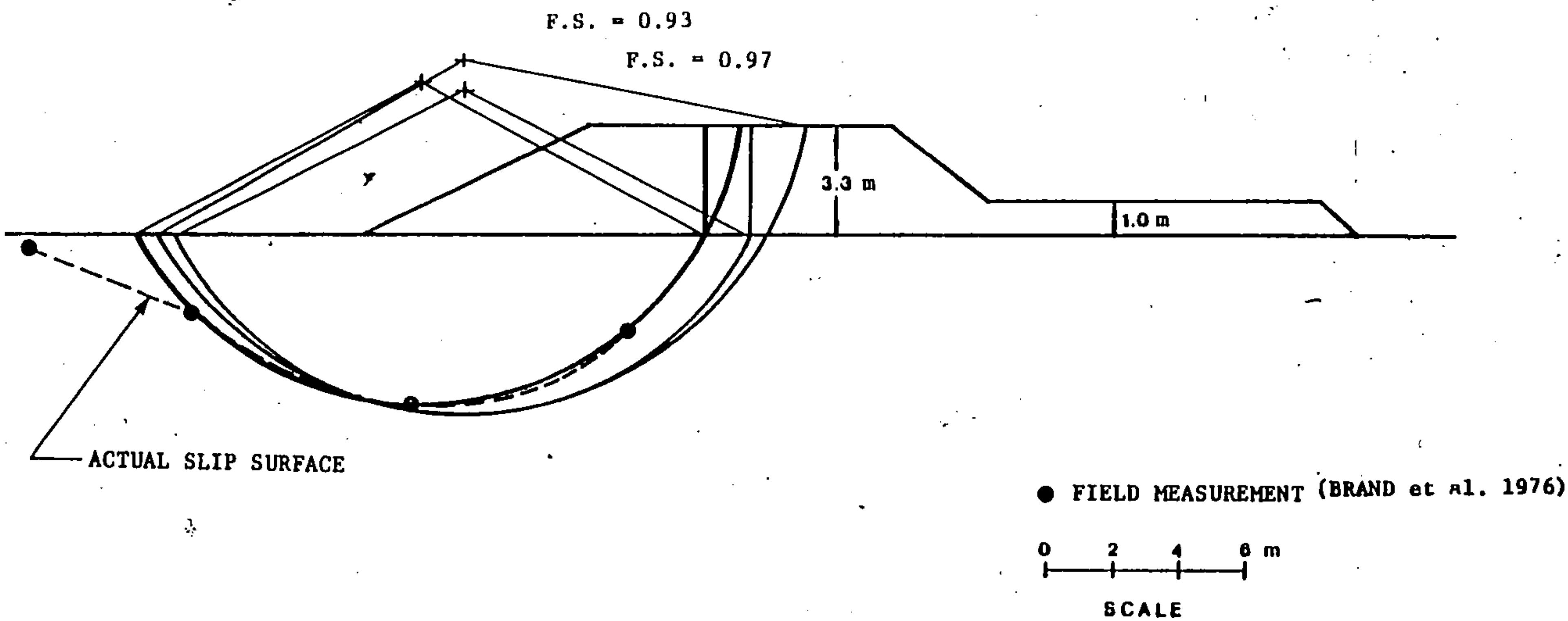


FIG. 4.5 CRITICAL CIRCLES DETERMINED USING $c_u = 0.22\sigma_p'$, BANGPLI EMBANKMENT

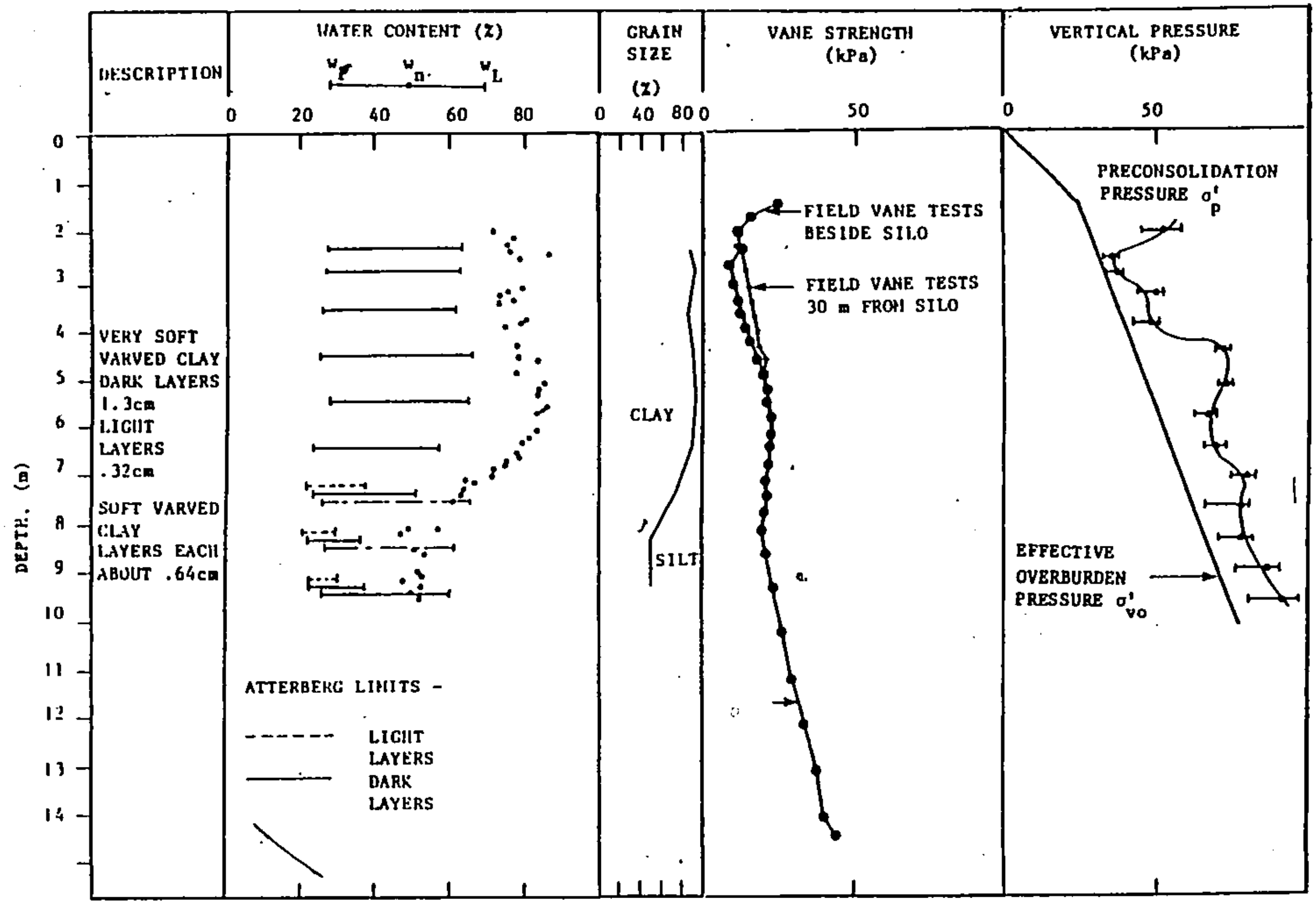


FIG. 4.6 SOIL PROFILE AT NEW LISKEARD

(after EDEN & BOZOZUK 1962)

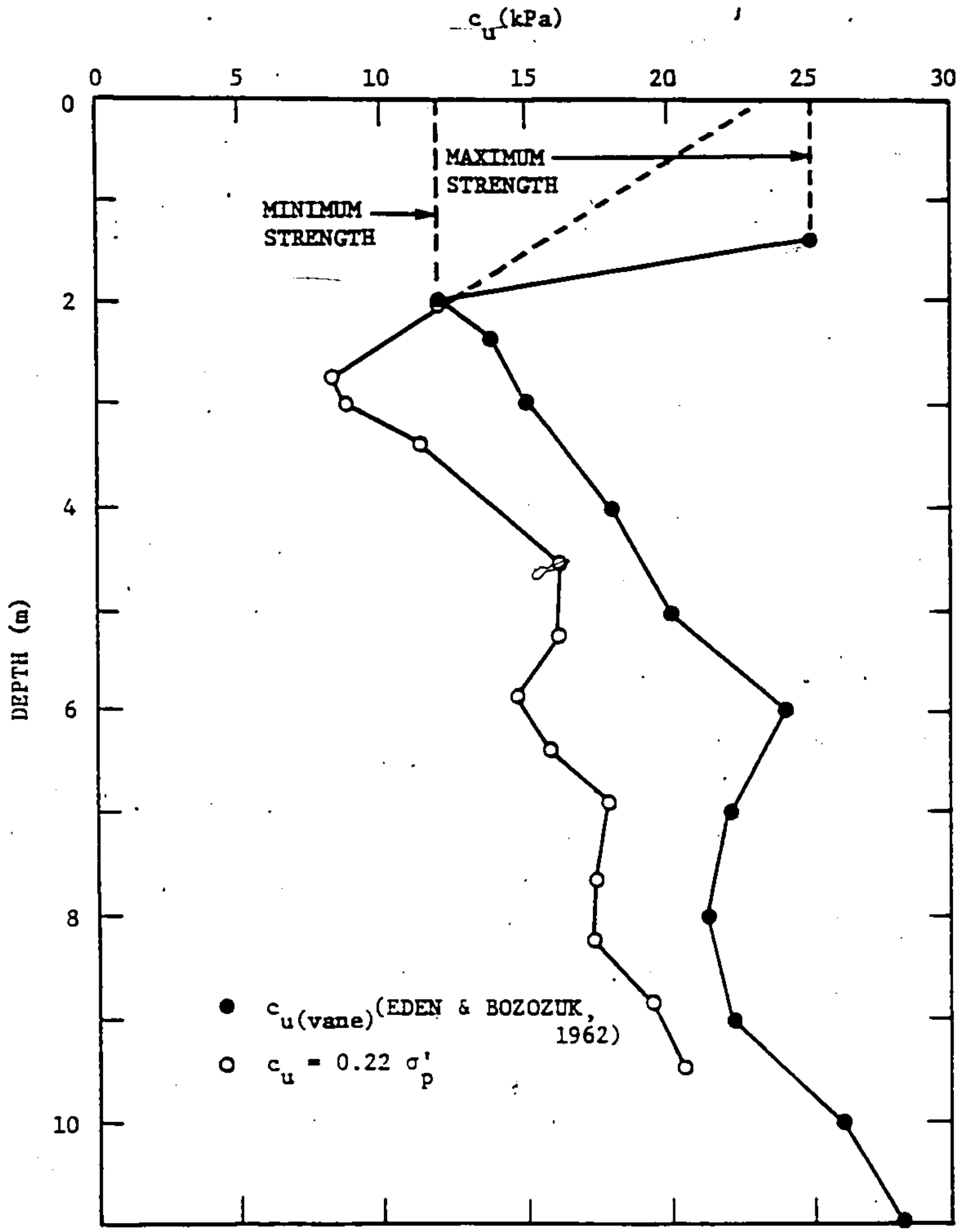


FIG. 4.7 STRENGTH PROFILES AT NEW LISKEARD

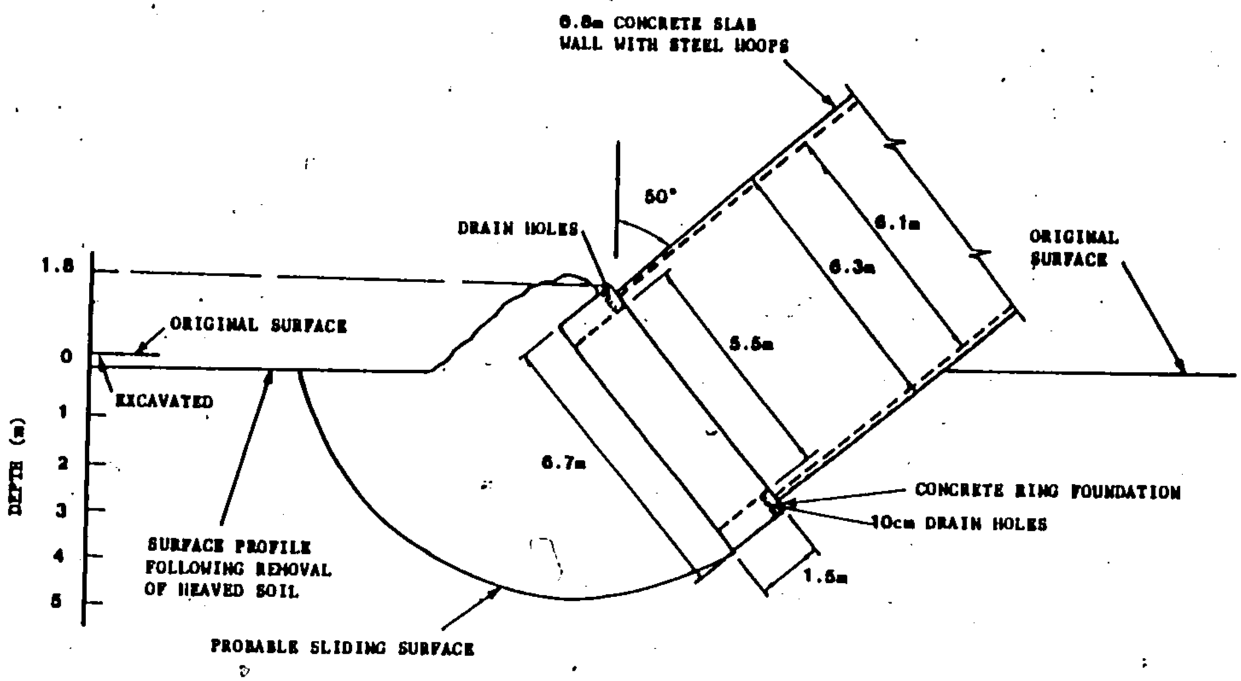


FIG. 4.8 DETAILS OF THE SILO BASE AFTER FAILURE, NEW LISKEARD

(after EDEN & BOZOZUK 1962)

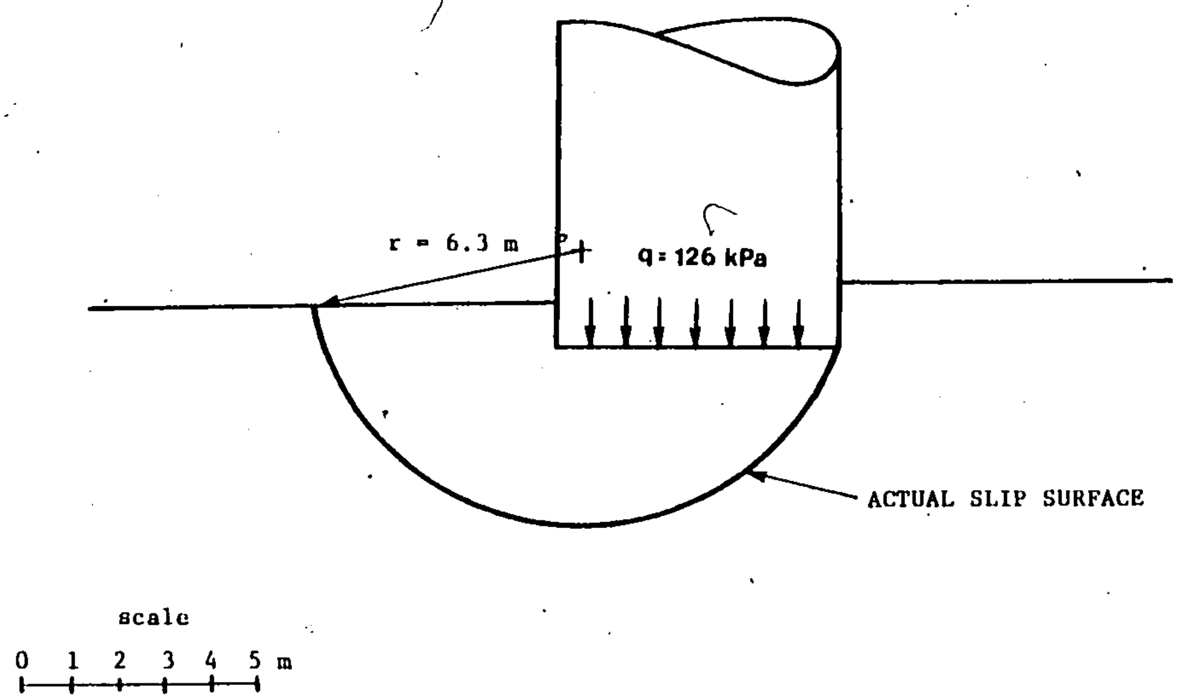


FIG. 4.9 SLIP CIRCLE ANALYSIS, NEW LISKEARD SILO

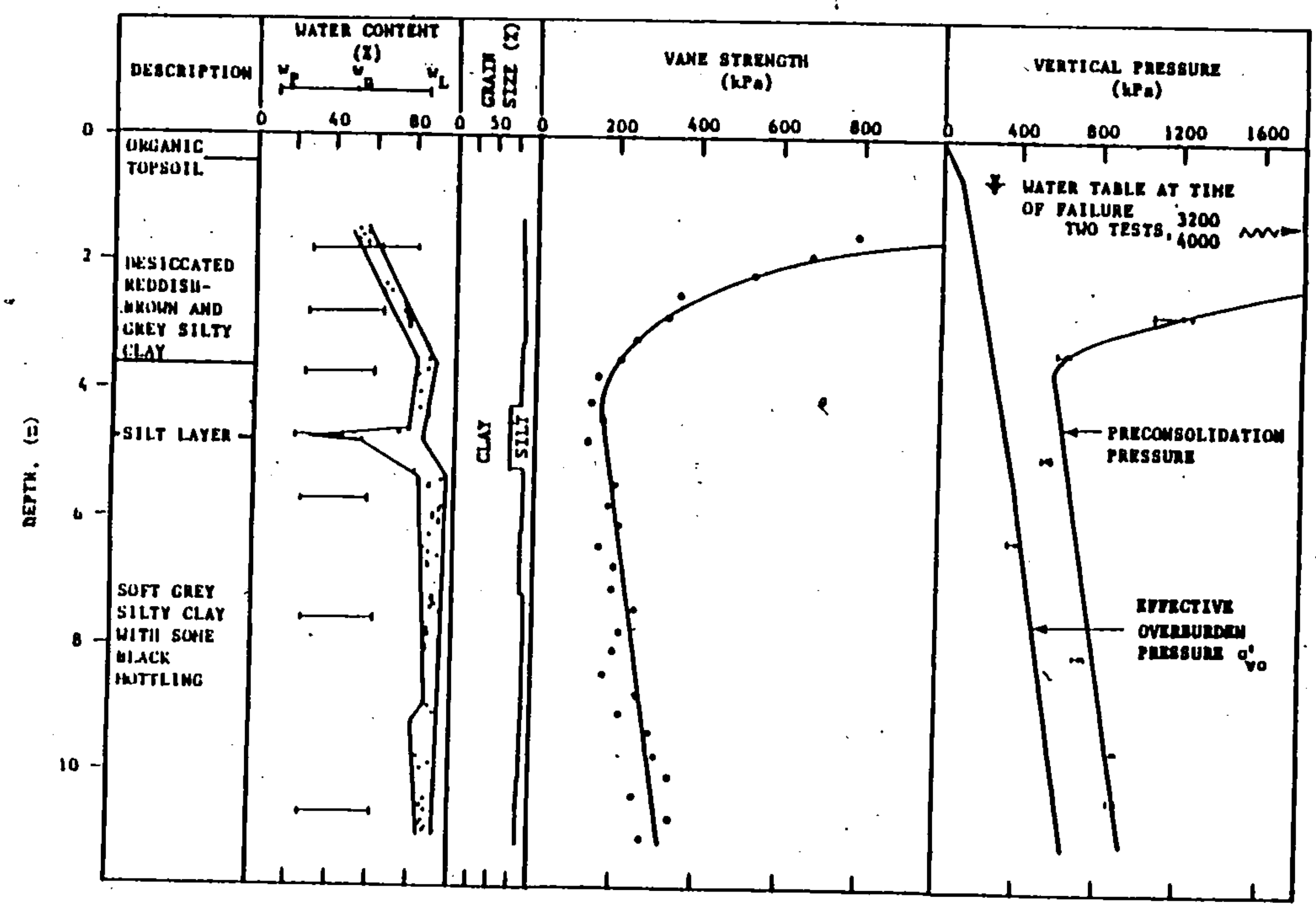


FIG. 4.10 SOIL PROFILE AT VANKLEEK HILL

(after BOZOZUK 1972)

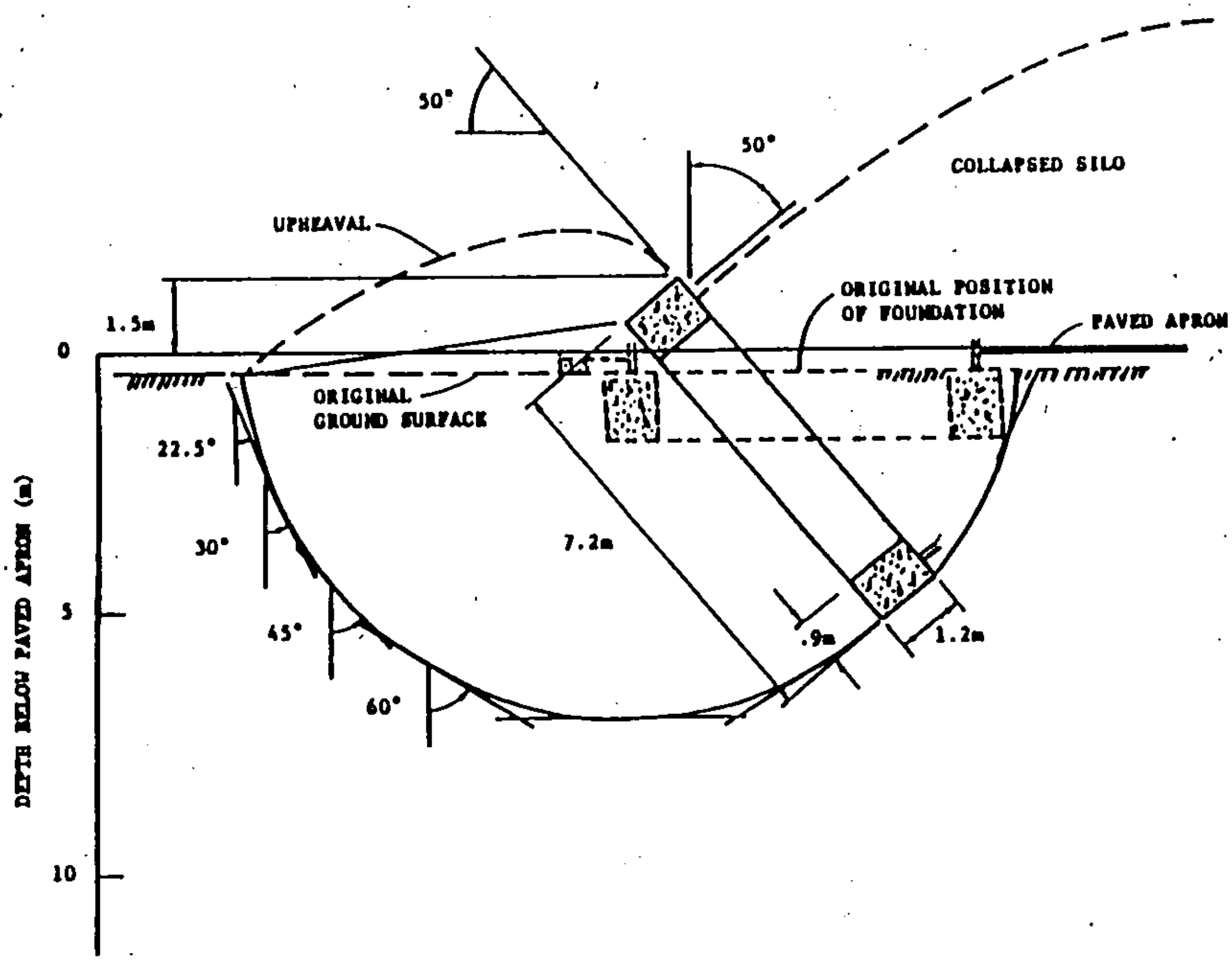


FIG. 4.11 ATTITUDE OF SILO FOUNDATION AFTER FAILURE, VANKLEEK HILL

(after BOZOZUK 1972)

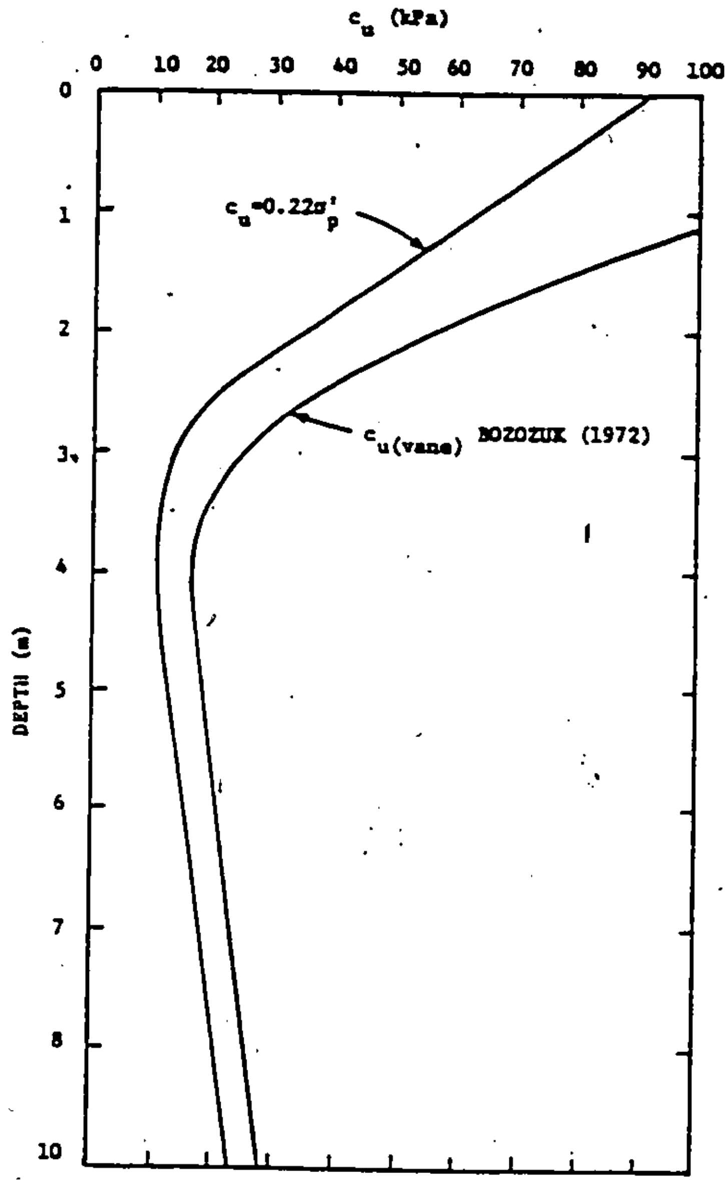


FIG. 4.12 STRENGTH PROFILES AT NEW LISKEARD

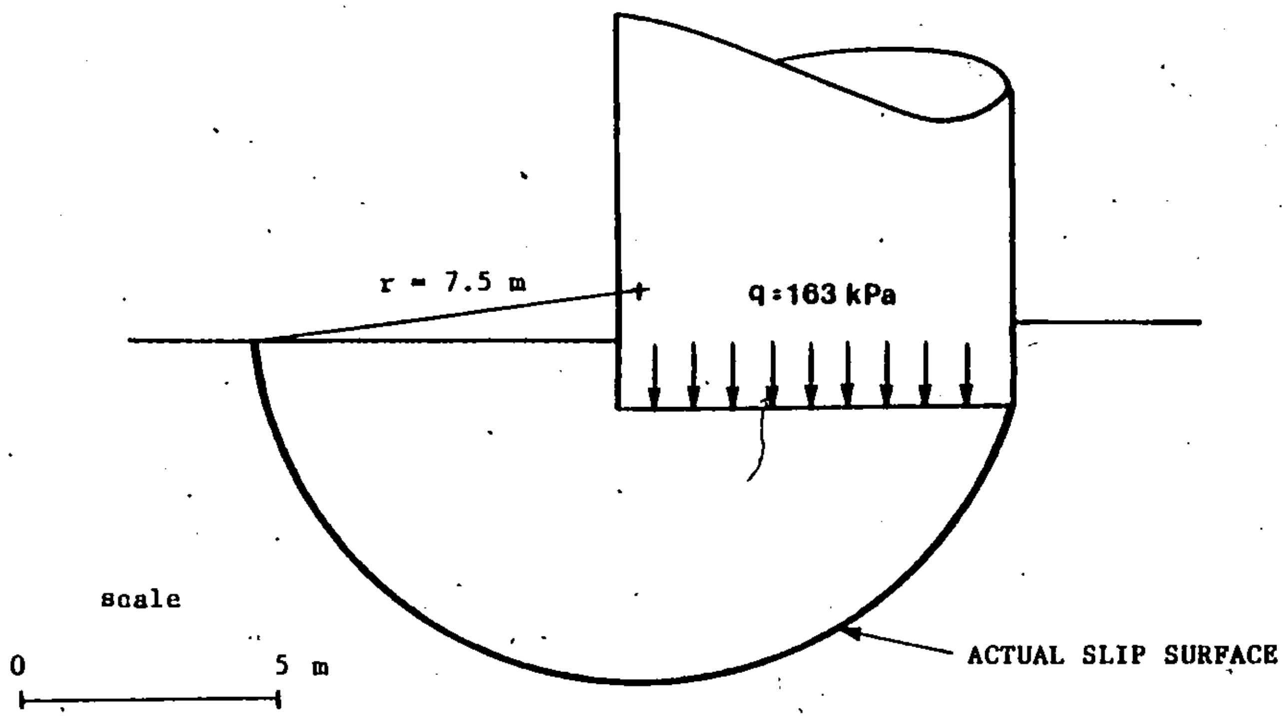


FIG. 4.13 SLIP CIRCLE ANALYSIS, VANKLEEK HILL SILO

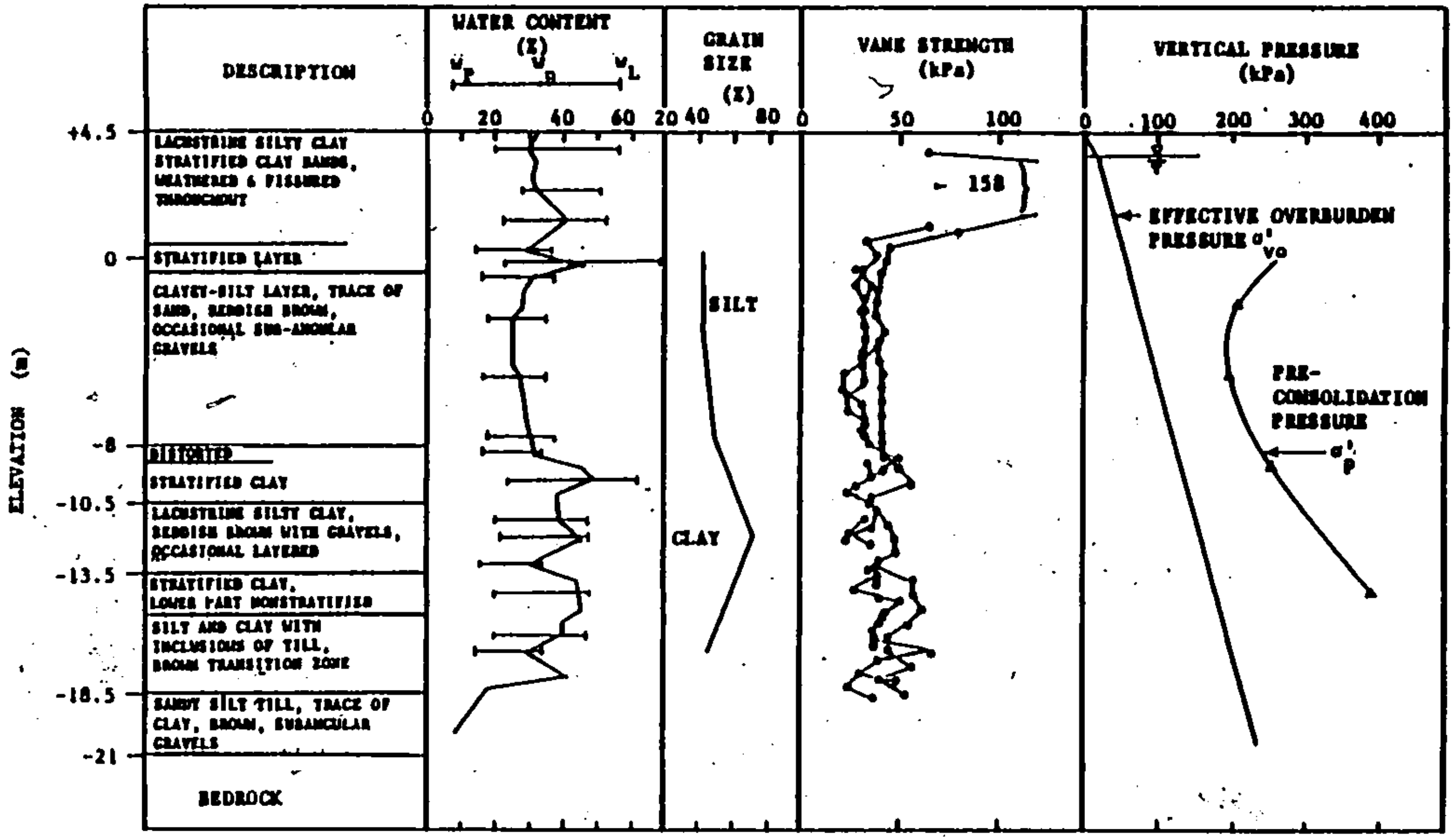


FIG. 4.14 SOIL PROFILE AT WELLAND

(after KWAN 1971)

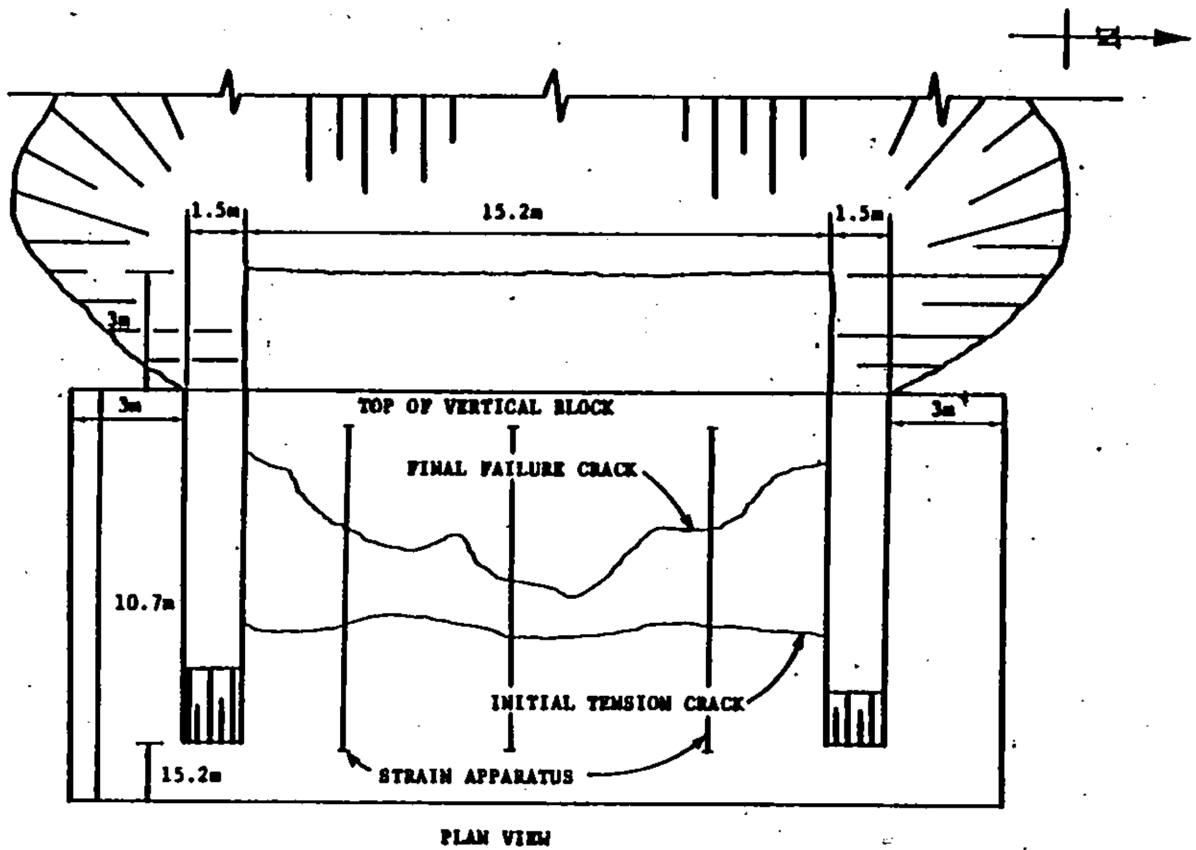


FIG. 4.15 LOCATION OF TENSION CRACKS, WELLAND VERTICAL CUT

(after KWAN 1971)

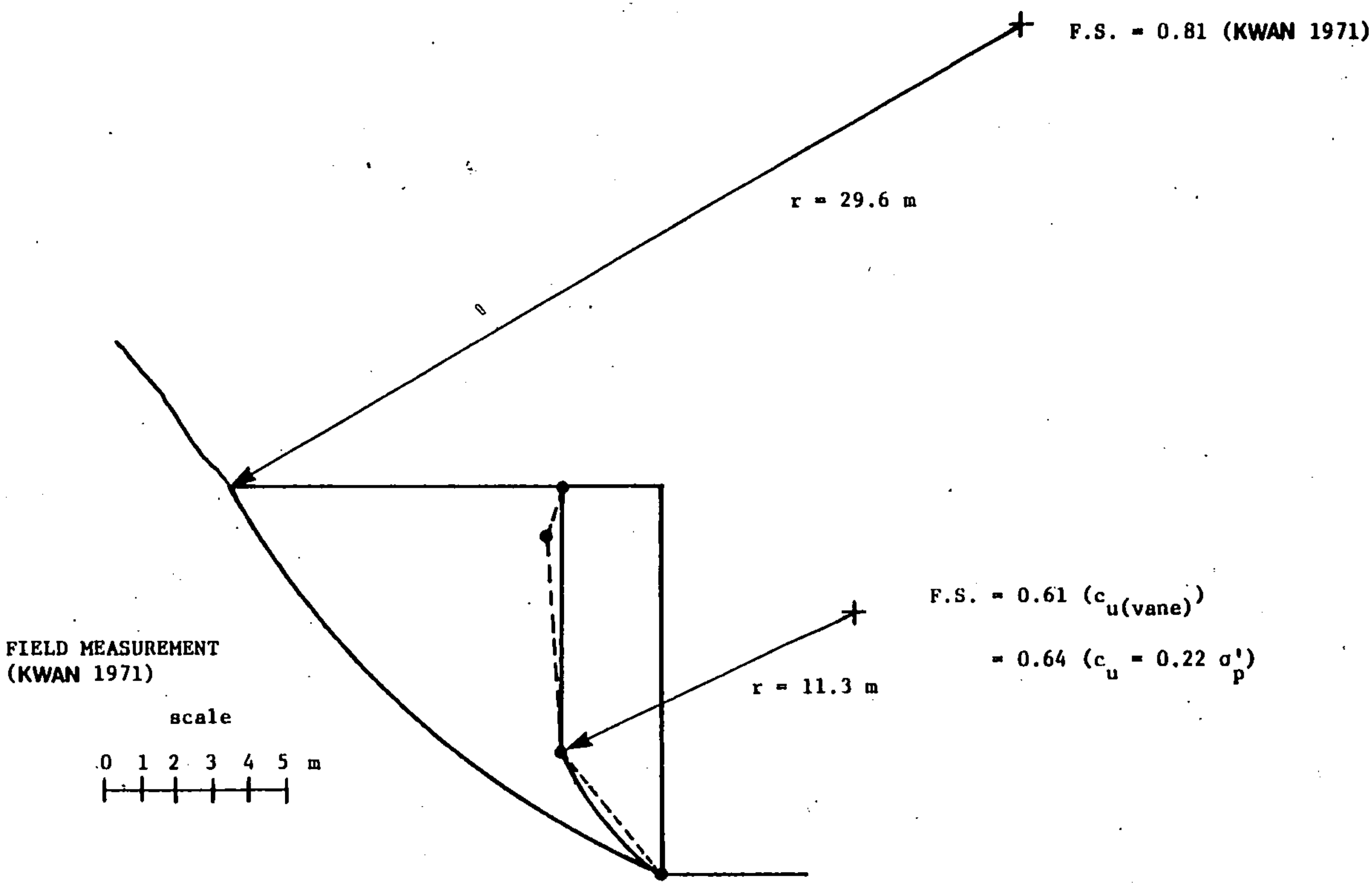


FIG. 4.16 SLIP CIRCLE ANALYSIS, WELLAND VERTICAL CUT

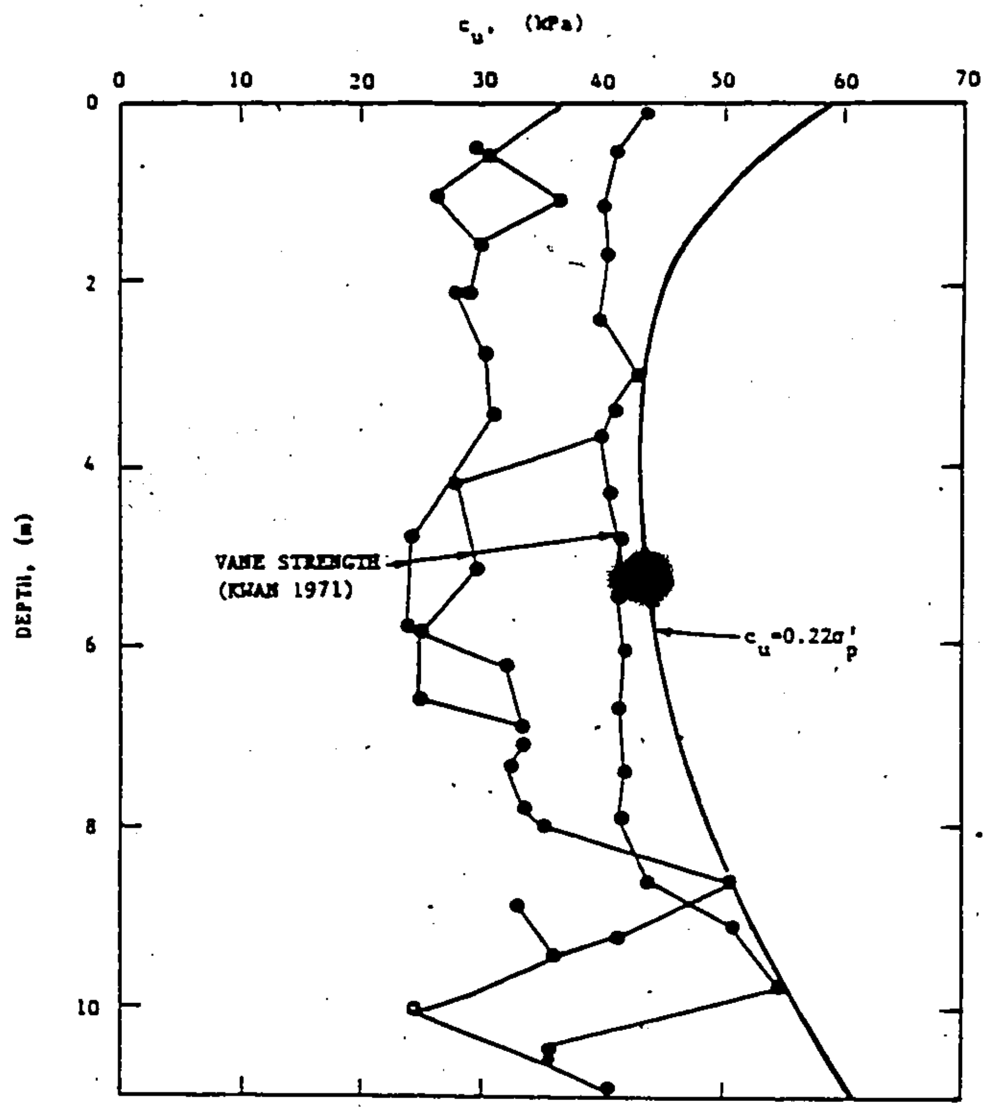


FIG. 4.17 SHEAR STRENGTH PROFILES OF WELLAND CLAY

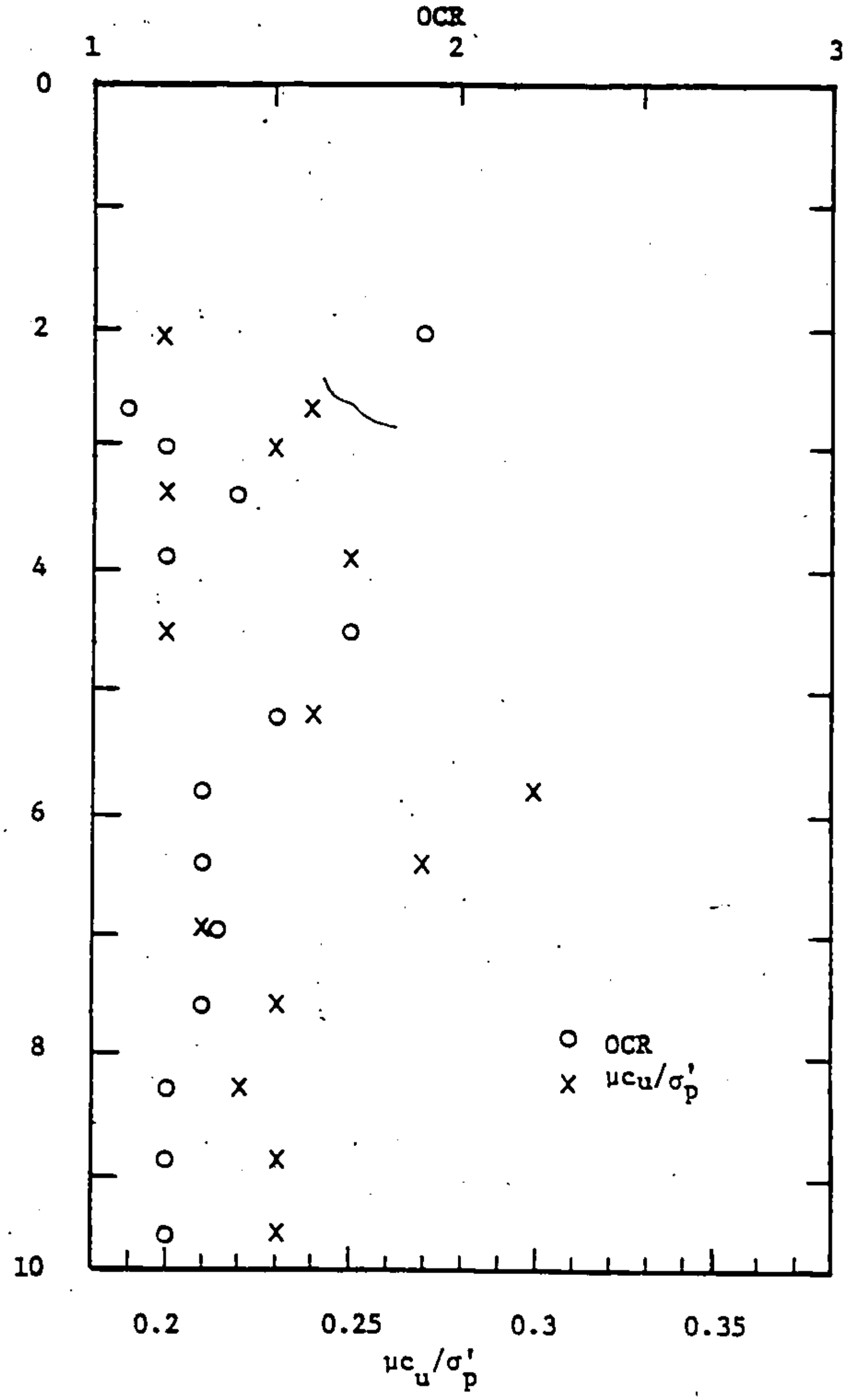


FIG. 4.18 $\mu c_u/\sigma_p'$ RATIO AND OCR PROFILES AT NEW LISKEARD

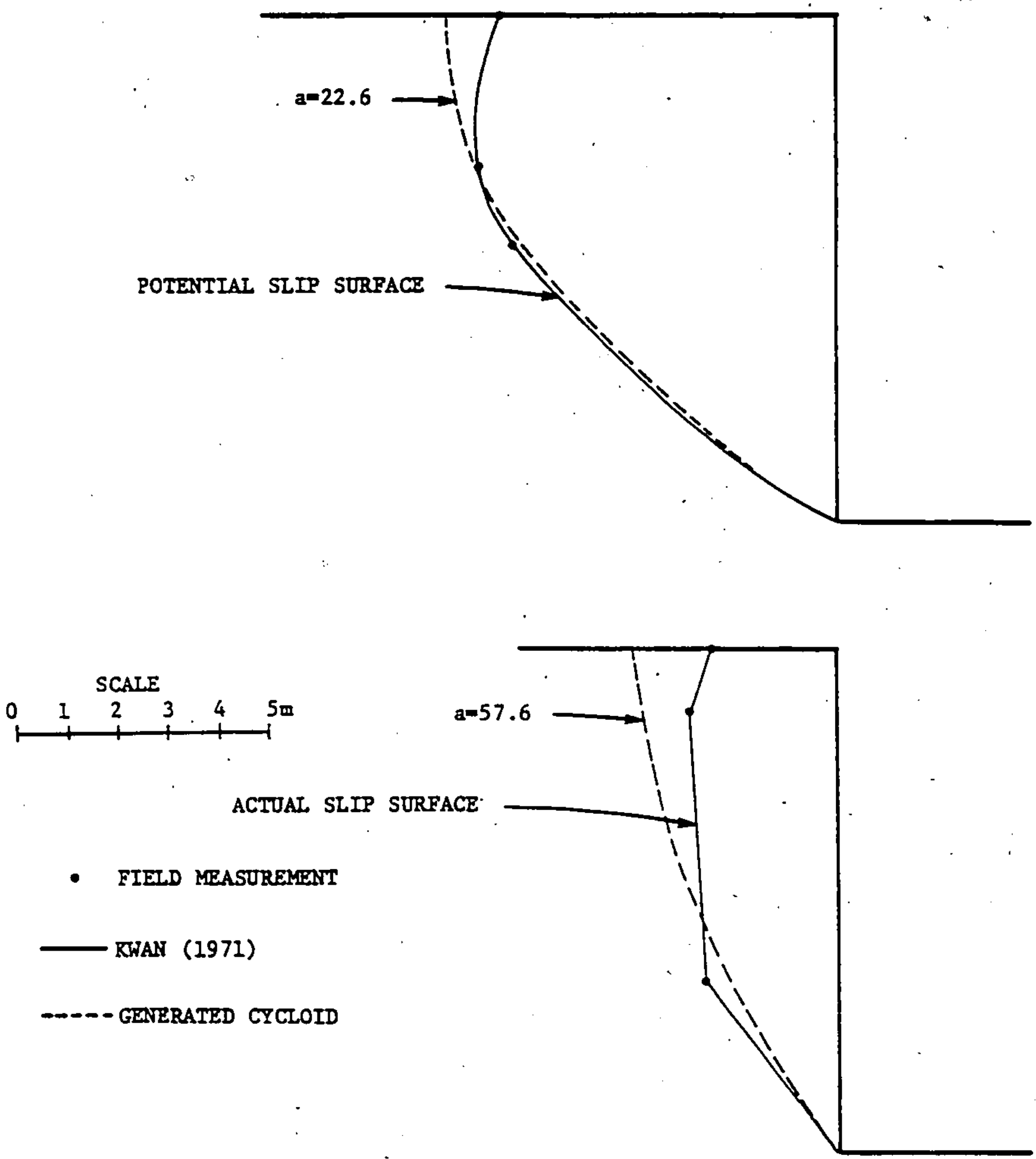
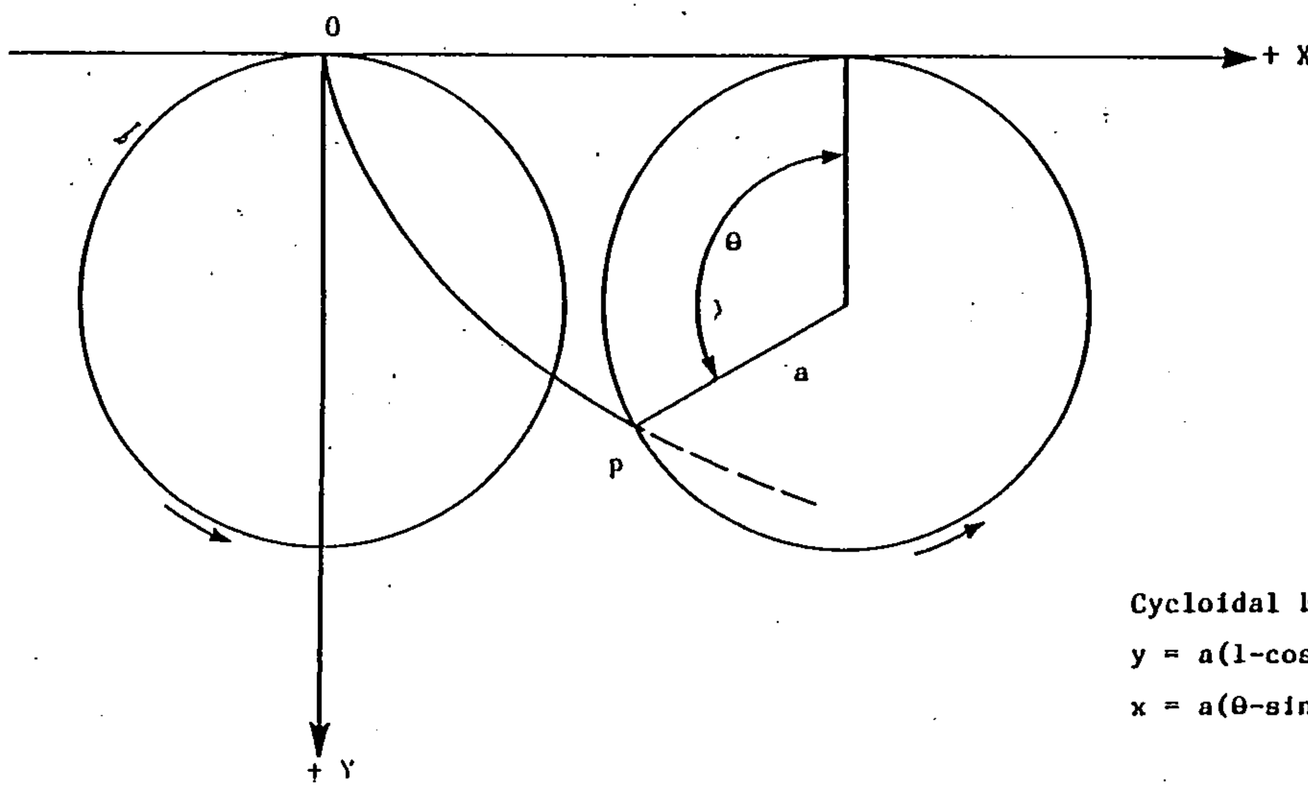


FIG. 4.19 POTENTIAL AND ACTUAL FAILURE SURFACES, WELLAND VERTICAL CUT



Cycloidal Function

$y = a(1-\cos\theta)$

$x = a(\theta-\sin\theta)$

FIG. 4.20 GENERATION OF CYCLOIDAL ARC

(after ELLIS 1973)

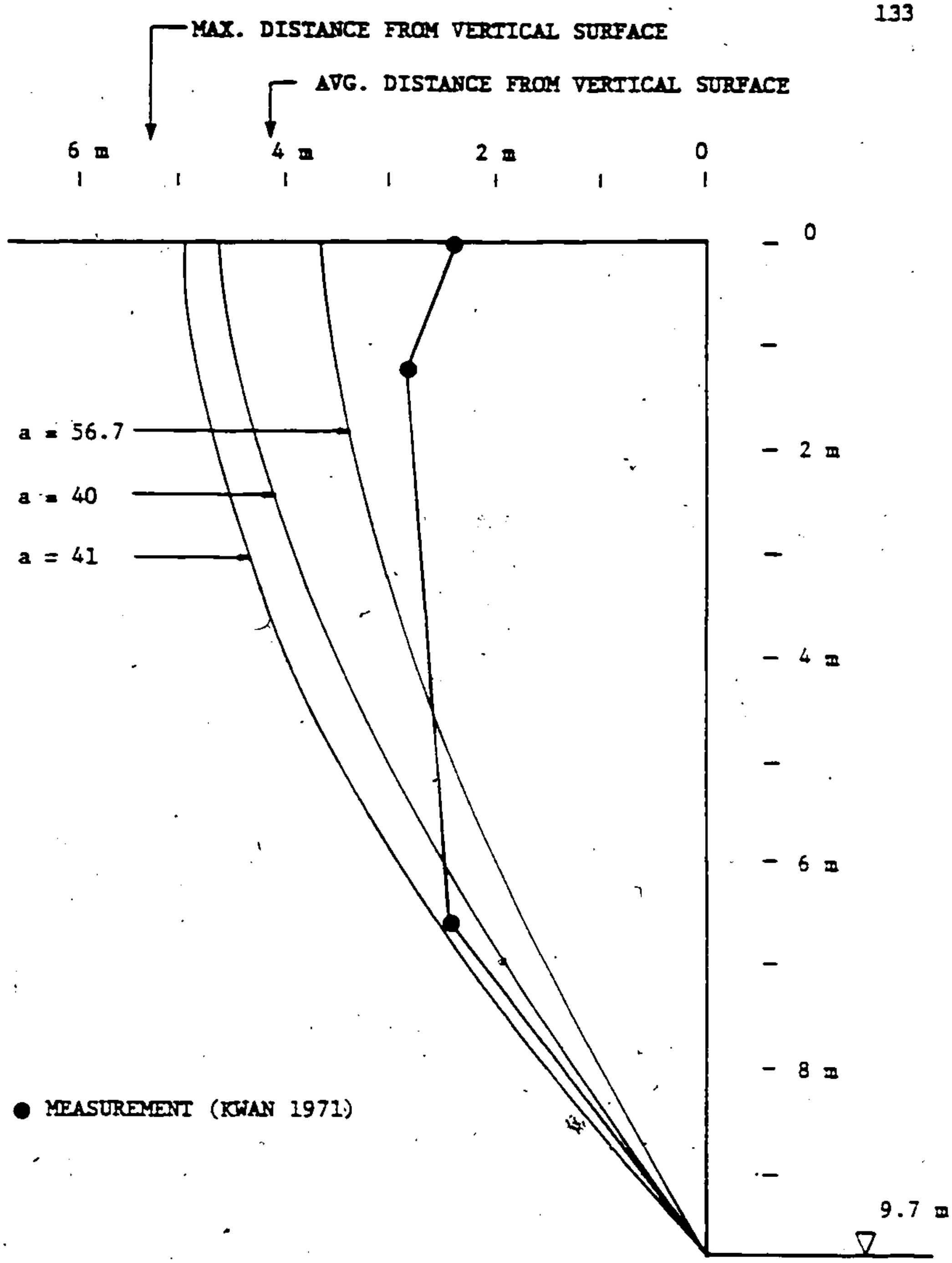


FIG. 4.21 COMPARSION BETWEEN ACTUAL FAILURE SURFACE AND SURFACES GENERATED BY CYCLOIDAL FUNCTIONS

Chapter V

CONCLUSIONS

5.1 GENERAL

In this chapter, a brief review of the study is presented and the major conclusions are given. In addition, topics for further research are suggested.

5.2 SUMMARY OF THE STUDY

The main object of this study has been to investigate the applicability of Mesri's expression, $c_u=0.22\sigma'_p$, for different types of sensitive clay under different conditions of loading and unloading. In order to achieve that objective, stability analyses were carried out for an embankment failure, two silo failures, and a vertical cut failure. All the case studies were obtained from recently published articles. Only two different types of clay were examined: the soft sensitive clays of Eastern Canada and the Bangkok clay of Thailand.

In the analyses of the failures, both the vane strength, and the shear strength obtained from the $c_u=0.22\sigma'_p$ expression were employed. Because high shear strength values are often measured in the desiccated clay crust, three types of crust strength assumptions were made in the analyses when using the results of vane strength measurements. The three assumed values were: the full, the mid-depth and the minimum crust shear strengths. When the maximum and the mid-depth crust strength assumptions were used, the shear strength profile of the subsoil was corrected by means of Bjerrum's (1972) correction factor.

In order to be able to make comparisons with the results obtained by the original authors, the same methods of analyses were used for the case histories in this study. These methods include the limit equilibrium methods, such as Fellenius' and Bishop's methods; the bearing capacity equations, such as Skempton's (1942, 1951) and Meyerhof's equations; and the critical height equations modified as well as unmodified.

5.3 MAJOR CONCLUSIONS

The following is a summary of the major conclusions resulting from the study.

1. Analyses have shown that the c_u=0.22σ'_p expression adequately describes the mobilized shear strength at failure of the soft sensitive clay in Eastern Canada and that it is possible to apply this expression to other types of soft clay.
2. The c_u=0.22σ'_p expression can be used to estimate the mobilized shear strength under loading and unloading conditions such as embankments, shallow foundations and excavations.

3. It was found that it is more difficult to obtain a representative mobilized shear strength profile using the field vane tests with Bjerrum's correction approach than by the c_u=$0.22\sigma'_p$ method. In Bjerrum's method, no consideration is made to deal with the selection of shear strength profiles for the clay crust, whereas, a crust strength profile may be obtained by simply extrapolating the $0.22\sigma'_p$ values at the base of the desiccated clay crust.
4. According to the results of the study of the New Liskeard silo failure, the limit equilibrium analyses using the uncorrected vane strength values gave factors of safety closer to unity than the ones using the corrected vane strength values. The opposite situation was encountered when the bearing capacity equations were used; i.e. in this case, satisfactory results were obtained when using the corrected values of vane strength. This case study also showed that generally good factors of safety can be obtained using the c_u=$0.22\sigma'_p$ profile, regardless of the method of analysis selected.
5. In the bearing capacity analyses of Vankleek Hill silo failure based on the corrected vane strength values, the safety factors obtained from Meyerhof's equation were higher than unity. This overestimation of bearing capacity indicates that the

Meyerhof equation may not be adequate for the design of foundations on soft sensitive marine clays.

6. In the analysis of the Welland canal failure, it was found that the limit equilibrium methods of slices, such as Bishop's method, did not give satisfactory results. However, this could have been caused by misinterpretation of the position of the tension crack. In contrast, factors of safety obtained by using the critical height equations generated values close to unity.
7. The shapes of both the actual and the potential rupture surfaces of the Welland canal failure were found to be best represented by curves generated by a cycloidal function. This probably holds true for all excavation cases.
8. Also in the Welland canal failure analysis, the average location of the final rupture surface was found to be consistent with Terzaghi's (1941) finding, stating that the tension crack would be located at about a distance equal to half of the critical height away from the vertical face of the cut.

5.4 RECOMMENDATIONS

This study showed that the $c_u=0.22\sigma'_p$ expression is applicable to the four case studies investigated. This extended the work of Trak et al. (1980) on embankment failures and suggests that it would be fruitful to conduct further research along these lines for shallow foundations and excavations. Also, further research may show whether the range of application of the expression, $c_u=0.22\sigma'_p$, can be extended to clays other than the ones investigated thus far, especially other soft sensitive clays outside of Canada.

The use of a cycloidal curve to simulate the shape of a slip surface in limit equilibrium analysis might possibly give interesting results. Unfortunately, at the present time, no such numerical treatment is available, except for the one developed by Ellis (1973), for vertical and near vertical slopes in homogenous and isotropic soils. One possible way to include a cycloidal function in a stability analysis is to apply it with non-circular stability methods such as those developed by Janbu (1954) and Spencer (1967). Since many computer programs have already been written using these methods, only a slight modification would be required to produce an analytical method that could simulate a cycloidal slip surface.

REFERENCES

Aas, G. 1965. "A study of the effect of vane shape and rate of strain on the measured values of in-situ shear strength of clay". Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, 1, pp. 141-145.

Aas, G. 1976(a). "Stability of slurry trench excavation in soft clay". Proceedings, 6th European Conference on Soil Mechanics and Foundation Engineering, 1(1), pp. 103-110.

Aas, G. 1976(b). Soil properties, their determination and use in stability analysis of clays. NIF - course, Gol, P. 69 (quoted in Helenelund 1977).

Aas, G. 1981. "Stability of natural slope in quick-clays". Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 3, Session 11, pp. 333-338.

Baligh, M. M. and Azzouz, A. S. 1975. "End effects on stability analysis of slopes". ASCE Journal of Geotechnical Engineering Division, 101, GT11, pp. 1105-1117.

Bishop, A. W. 1955. "The use of the slip circle in the stability analysis of slopes". Géotechnique, 5(1), pp. 7-17.

Bishop, A. W. and Bjerrum, L. 1960. "The relevance of the triaxial test to the solution of stability problems". Proceedings, ASCE Conference on Shear Strength of Cohesive Soils, Boulder, pp. 437-501.

Bjerrum, L. 1972. "Embankments on soft ground". Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, 2(1), pp. 1-54.

Bjerrum, L. 1973. "Problems of soil mechanics and construction on soft clays". Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, 3, pp. 111-160.

Bjerrum, L. and Flodin, N. 1960. "The development of soil mechanics in Sweden, 1900 - 1925". Géotechnique, 10(1), pp. 1-18.

Bjerrum, L. and Simons, N. E. 1960. "Comparison of shear strength characteristics of normally consolidated clays". Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, pp. 711-726.

Bözozuk, M. 1972. "Foundation failure of the Vankleek Hill tower silo". Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, 1(2), pp. 885-902.

Bozozuk, M. 1977. "Evaluating strength tests from foundation failures". Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1, pp. 55-59.

Brand, E. W., Moh, Z. C. and Wirojanagud, P. 1976. "Interpretation of dutch cone tests in soft Bangkok clay". Proceedings of the European Symposium on Penetration Testing, Stockholm, 2(2), pp. 51-58.

Burmister, D. M. 1951. "The application of controlled test methods in consolidation testing". Consolidation Testing of Soils, ASTM, STP 126, pp. 83-91 (quoted in Das 1983).

Cadling, L. and Ödenstad, S. 1950. "The vane borer". Swedish Geotechnical Institute, Proceeding No. 2, 88 p.

Casagrande, A. 1936. "The determination of the preconsolidation load and its practical significance". Discussion D-34, Proceedings, 1st International Conference on Soil Mechanics and Foundation Engineering, Cambridge, 3, pp. 60-64 (quoted in Holtz and Kovacs 1981).

Chen, W. F. and Scawthorn, C. R. 1970. "Limit analysis and limit equilibrium solution in soil mechanics". Soil and Foundation, 4(3), pp. 13-49.

Chowdhury, R. N. 1978. Slope Analysis. Elsevier Scientific Publishing Company, Developments in Geotechnical Engineering, 22, 423 p.

Collin, A. 1846. Recherches expérimentales sur les glissements spontanés de terrains argileux, accompagnées de considérations sur quelques principes de la mécanique terrestre. Carillan-Goeury, Paris.

Crawford, C. B. 1965. "Resistance of soil structure to consolidation". Canadian Geotechnical Journal, 2(2), pp. 90-115.

Das, B. M. 1983. Advanced Soil Mechanics. McGraw-Hill Book Co., 511 p.

Dascal, O., Tournier, J. P., Tavenas, F. and La Rochelle, P. 1972. "Failure of a test embankment on sensitive clay". Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, 1, pp. 129-158.

Eden, W. J. and Bozozuk, M. 1962. "Foundation failure of a silo on varved clay". The Engineering Journal, 45(9), pp. 54-57.

Ellis, H. B. 1973. "Use of cycloidal arcs for estimating ditch safety". ASCE Journal of Soil Mechanics Division, 99, SM2, pp. 181-198.

Fellenius, W. 1927. Erdstatische Berechnungen mit Reibung and Kohäsion (Adhäsion) und unter Annahme kreiszylindrischer Gleitflachen. Ernst Verlag, Berlin.

Fredlund, D. G. and Krahn, J. 1977. "Comparison of slope stability methods of analysis". Canadian Geotechnical Journal, 14, pp. 429-439.

Hansbo, S. 1957. "A new approach to the determination of shear strength of clay by the fall cone test". Swedish Geotechnical Institute, Proceeding No. 14, 47 p.

Helenelund, K. V. 1977. "Methods for reducing undrained shear strength of soft clay". Swedish Geotechnical Institute, Report No. 3, 59 p.

Holtz, R. D. and Kovacs, W. D. 1981. An Introduction to Geotechnical Engineering. Prentice-Hall, Inc., Englewood Cliffs, 733 p.

Holtz, R. D. and Wennerstrand, J. 1972. "Discussion to embankments on soft ground". Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, 3, pp. 59-60.

Janbu, N. 1954. "Application of composite slip surfaces for stability analysis". Proceedings European Conference on Stability of Earth Slopes, Stockholm, 3, pp. 43-49.

Kwan, D 1971. "Observations of the failure of a vertical cut in clay at Welland, Ontario". Canadian Geotechnical Journal, 9, pp. 283-298.

Lacasse, S. M. and Ladd, C. C. 1973. "Behaviour of embankment on New Liskeard varved clay". MIT Research Report R73-44, Soil Pub. No. 327.

Ladd, C. C. and Foott, R. 1974. "New design procedure for stability of soft clays". ASCE Journal of Geotechnical Engineering Division, 100, GT7, pp. 763-786.

Lam. T. M. and Trak, B. 1982. STAB: slope stability computer program for Bishop's and Fellenius' methods of analysis. User's Manual, Department of Civil Engineering, University of Ottawa, Ottawa, Ont..

Lambe, T. W. and Whitman, R. V. 1973. Soil Mechanics, SI Version. John Wiley and Sons, New York, 533 p.

La Rochelle, P., Trak, B., Tavenas, F. and Roy, M. 1974. "Failure of a test embankment on a sensitive Champlain clay deposit". Canadian Geotechnical Journal, 11, pp. 142-164.

LA ROCHELLE, P., SARRAILH, J., TAVENAS, F., ROY. M., and LEROUEIL, S. 1981. "Causes of sampling disturbance and design of a new sampler for sensitive soils". Canadian Geotechnical Journal, 18(1), pp. 52-66.

Larsson, R. 1980) "Undrained shear strength in stability calculation of embankments and foundations on soft clays". Canadian Geotechnical Journal, 17(4), pp. 591-602.

Lo, K. Y. and Stermac, A. G. 1965. "Failure of an embankment founded on varved clay". Canadian Geotechnical Journal, 2(2), pp. 234-253.

Mesri, G. 1975. "Discussion on new design procedure for stability of soft clays". ASCE Journal of Geotechnical Division, 101, GT4, pp. 409-412.

Meyerhof, G. G. 1951. "The ultimate bearing capacity of foundations". Géotechnique, 2, pp. 301-322.

Mieussens, C. and Pilot, G. 1971. "Etude du comportement d'un remblai sur sols mous peu avant la rupture". Journées Françaises de Mécanique des sols, Paris, mai 1971.

Morgenstern, N., Blight, G. E., Janbu, N. and Resendiz, D. 1977. "Slope and excavation". Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 2, pp. 547-604.

Moh, Z. C., Nelson, J. D. and Brand, E. W. 1969. "Strength and deformation behaviour of Bangkok clay". Proceedings, 7th International Conference of Soil Mechanics and Foundation Engineering, Mexico, 1, pp. 287-296.

Parcher, J. V. and Means, R. E. 1968. Soil Mechanics and Foundations. C. E. Merrill Publishing Co., 573 p.

Parry, R. H. G. 1971. "Undrained shear strength in clays". Proceedings, 1st Australia-New Zealand Conference on Geomechanics, Melbourne, pp. 11-15

Petterson, K. E. 1955. "The early history of circular sliding surfaces". Géotechnique, 5(4), pp. 275-299.

Pilot, G. 1972. "Study of five embankment failures on soft soils". Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structure, Lafayette, 1(1), pp. 81-100.

Schmertman, J. H. 1955. "The undisturbed consolidation behaviour of clay". Transactions, 102, pp. 1201-1233 (quoted in Das 1983).

Skempton, A. W. 1942. "An investigation of the bearing capacity of a soft clay soil". Journal of the Institution of Civil Engineers, Paper No. 5305, 18, pp. 307-321

Skempton, A. W. 1951. "The bearing capacity of clay". Proceedings, Building Research Congress, pp. 180-189.

Soderman L. G. and Quigley, R. M. 1965. "Geotechnical properties of three Ontario clays". Canadian Geotechnical Journal, 2(2), pp. 167-193.

Smith, G. N. 1978 Elements of Soil Mechanics for Civil and Mining Engineers. 4th edition, Granada Publishing Ltd., London, 424 p.

Sowers, G. F. 1979. Introductory Soil Mechanics and Foundations: Geotechnical Engineering. 4th edition, MacMillan Publishing Co., New York, 621 p.

Spencer, E. 1967. "A method of analysis of the stability of embankments assuming parallel interslice forces". Géotechnique, 17(1), pp. 11-26.

Statens Järnvägar 1922. Geotekniska kommission, 1914-1922. Slutbetänkande agivet till Kungl. Järnvägsstyrelsen den 31 maj, 1922. Statens Järnvägars Geotekniska Meddelanden 2, Stockholm.

Stermac, A. G., Lo, K. Y. and Barsvary, A. K. 1967. "The performance of an embankment on a deep deposit of varved clay". Canadian Geotechnical Journal, 4, pp. 45-61.

Tavenas, F. and Leroueil, S. 1977. "Effects of stresses and time on yielding of clays". Proccedings, 9th International Conference in Soil Mechanics and Foundation Engineering, Tokyo, 1, pp. 319-326.

Tavenas, F. and Leroueil, S. 1980. "The behaviour of embankments on clay foundations". Canadian Geotechnical Journal, 17(2), pp. 236-260.

Taylor, D. W. 1948. Fundamentals of Soil Mechanics. Wiley, New York.

Terzaghi, K. 1941. "General wedge theory of earth pressure". ASCE Transactions, 106, pp. 68-97.

Terzaghi, K. and Peck, R. B. 1967. Soil Mechanics in Engineering Practice. Wiley, New York, 729 p.

Trak, B. 1974. Contribution à l'étude de la stabilité à court terme des remblais sur fondations argileuses. Thèse de maîtrise, Département de Génie Civil, Université Laval, Québec.

Trak, B. 1980. De la stabilité des remblais sur sols mous. Thèse de Doctorat, Départment de Génie Civil, Université Laval, Québec.

Trak, B. 1981. "Discussion to session XI - Stability of natural slopes in quick clays". Proceedings, 10th International Conference in Soil Mechanics and Foundation Engineering,

Trak, B., La Rochelle, P., Tavenas, F., Leroueil, S. and Roy, M. 1980. "A new approach to the stability analysis of embankments on sensitive clays". Canadian Geotechnical Journal, 17(4), pp. 526-544.

Tschebotarioff, G. P. 1973. Soil Mechanics, Foundations, and Earth Structures. McGraw-Hill Book Co., Inc., New York, 655 p.

Appendix A

THE DETERMINATION OF PRECONSOLIDATION PRESSURE AND ITS RELATION TO THE SENSITIVITY OF SAFETY FACTORS

A.1 GENERAL

When using the expression $c_u=0.22\sigma'_p$ in stability analysis, the preconsolidation pressures are generally determined from the oedometer test. Since the advent of the oedometer, several methods have been proposed to determine the value of σ'_p. Different values of preconsolidation pressure evaluated by these methods might have significant influence in the determination of mobilized shear strength using the $c_u=0.22\sigma'_p$ expression. There are various ways to evaluate the preconsolidation pressure of soft soil, and their influence in the stability calculation are discussed here.

A.2 DETERMINATION OF PRECONSOLIDATION PRESSURE

The oedometer test is widely employed by geotechnical engineers, but the uncertainty in the preconsolidation pressure determined from this test is large. There are three factors which significantly influence the determination of σ'_p from laboratory tests. These factors are: (1) effect of sample disturbance, (2) effect of load increment ratio ($\Delta\sigma/\sigma$), and (3) effect of load duration.

Effect of sample disturbance on shear strength evaluation has been discussed in section 2.5. However, in consolidation testing, the curvature of the e-log σ' curve depends on the amount of disturbance occuring during and after sampling. The curvature becomes less pronounces with increasing disturbance (Fig. A.1(a)). Significant changes in the curvature can also occur when the ratio of load increment varies. Fig. A.1(b) shows the shape of the e-log σ' curve for various values of $\Delta\sigma/\sigma$. If $\Delta\sigma/\sigma$ is small, the ability of individual clay particles to readjust to their position of equilibrium is small, which results in a smaller compression as compared to that for larger $\Delta\sigma/\sigma$ (Das 1983). The other factor affecting the preconsolidation pressure value is the effect of loading duration or the time effect. As mentioned in section 2.2.3, the slower a material is loaded the lower its resistance becomes. This phenomenon is the same in consolidation testing. Fig. A.2 demonstrates that the apparent preconsolidation of a clay is reduced if the rate of loading is reduced in the oedometer test.

A procedure for the oedometer test was introduced in 1926 by Karl Terzaghi, and has been the normal way of finding the compression properties of soft soils. In this procedure, the loading is applied step by step, with the load increment in each step equal to the total load before that step, i.e. $\Delta\sigma/\sigma=1$. The duration of each load increment is 24 hours.

This type of test is called the conventional 24 hours oedometer test.

For determining the preconsolidation pressure from oedometer test results, the Casagrande (1936) method illustrated in Fig.A.2 has been widely used. This method involves the selection of the point corresponding to the maximum curvature (point A) along the e-log σ' curve. Two lines, one running horizontal and one tangent to the curve, are drawn from point A. A bisector is then constructed between these lines. The preconsolidation pressure is obtained by intersecting the bisector and the extension of the virgin compression curve (point B). Other procedures for determining σ'_p have been proposed by Burmister (1951) and Schmertmann (1955). Often, a range of probable values, rather than a single value, is defined. The upper limit is taken as the pressure at point D (Fig.A.2), and the lower limit is the pressure at the point of intersection of a horizontal line through the initial void ratio and the extension of the virgin compression curve. However, Casagrande's method is considered, by most engineers, to yield satisfactory results with a good quality sample and a properly conducted test.

An oedometer test is time consuming. Empirical methods for determination of the preconsolidation pressure have therefore been proposed as a complement to the test. In Sweden, an empirical expression was introduced by Hansbo.

(1957), and is widely used. The expression correlates preconsolidation pressure, vane shear strength value, and liquid limit.

$$\sigma'_p = \frac{c_u}{0.45 w_L}$$

A.3 SENSITIVITY OF FACTOR OF SAFETY

In stability analyses involving the use of the expression $c_u = 0.22\sigma'_p$, the magnitude of the shear strength computed depends totally on the σ'_p value used. Therefore it is very important to use the appropriate σ'_p value in the calculation. According to Trak et al. (1980), the σ'_p values determined by means of Casagrande's method from 24 hours oedometer test results should be used in order to obtain satifactory results in stability calculations. For example, Trak et al. (1980) report that, in their analysis of the embankment failure at Narbonne, the $c_u = 0.22\sigma'_p$ profile was determined from oedometer tests which were run at a rate of approximately one load increment every 10 days. This created difficulty in determining representative σ'_p values and subsequently led to unsatisfactory results.

The determination of the OCR in a desiccated crust is difficult, because of heterogeneity and of the presence of organic materials, such as roots and plant remains.

Therefore, an empirical extrapolation of the σ'_p values through the crust was adopted when the $c_u=0.22\sigma'_p$ expression is used. In St. Alban, Trak et al. (1980) found that the σ'_p values measured from samples obtained at the lower part of the crust can yield values that indicate the trend of the preconsolidation pressures in the entire crust. This approach was then applied to evaluate the $0.22\sigma'_p$ profiles for the stability analyses of embankments on different clay soils. Satifactory results were obtained indicating that this approach can adequately give the mobilized shear strength in the crust.

As mentioned in the last section, preconsolidation pressure can be evaluated by different methods. The σ'_p values determined from these methods can affect the results of a stability analysis employing the expression $c_u=0.22\sigma'_p$. In order to investigate the sensitivity of the analysis due to the variation of σ'_p values, the case history of New Liskeard silo failure was chosen. The reason for selecting this case study was that a constant c_u/σ'_p ratio was found throughout the subsoil deposit (Fig. 4.18).

In this sensitivity analysis, a number of preconsolidation pressure profiles were examined (Fig. A.3). These profiles are: (1) maximum and (2) minimum profiles, (3) an average profile evaluated from the maximum and minimum, (4) a profile based on Casagrande's approach, and (5) a profile determined using Hansbo's (1957) expression.

From these profiles, the shear strength values were calculated using the $c_u = 0.22\sigma'_p$ expression. Fig. A.4 shows the range of values based on oedometer test results and the ones determined using Hansbo's expression. The Hansbo expression gave a profile similar to the others. This may be due to the constant c_u/σ'_p ratio at the site.

Stability analyses similar to the ones in section 4.3.1.5 were carried out, and the results are shown in Table A.1. Generally, the percentage difference is within 10% of the factor of safety based on Casagrande's approach. The factors of safety computed using different analytical methods are almost identical for any σ'_p value used. The second best assumption for σ'_p value, other than the one based on Casagrande's approach, is the maximum possible σ'_p. The difference in factor of safety computed using this profile is about 5%. It is also of interest to note that when employing the $c_u = 0.22\sigma'_p$ expression with Hansbo's relationship, the mobilized shear strength of the subsoil can be expressed as 79% of vane shear strength measured ($0.22/0.45w_L = .79$; where $w_L = .62$). Comparing this value with Bjerrum's vane strength correction factor ($\mu = .85$), the difference is approximately 8%.

In conclusion, the variation of values was found to influence slightly the factor of safety in the analyses of the New Liskeard silo.

TABLE A.1

COMPUTED FACTORS OF SAFETY OF NEW LISKEARD SILO WITH

DIFFERENT $0.22\sigma'_p$ PROFILES

σ'_p PROFILE	BISHOP METHOD	CIRCULAR ARC METHODS(1)	SKEMPTON (1951)	SKEMPTON (1942)	MEYERHOF (1951)
CASAGRANDE (1936)	1.02	0.89	0.94	0.96	1.04
MAXIMUM	1.07	0.91	0.97	0.99	1.08
MINIMUM	0.91	0.82	0.87	0.88	0.96
AVERAGE(1)	0.99	0.87	0.92	0.94	1.02
HANSBO (1957)	0.92	0.82	0.87	0.89	0.96

(1) = AVERAGE OF MAXIMUM AND MINIMUM PRECONSOLIDATION PRESSURES

(2) = STABILITY METHOD WITHOUT SLICES

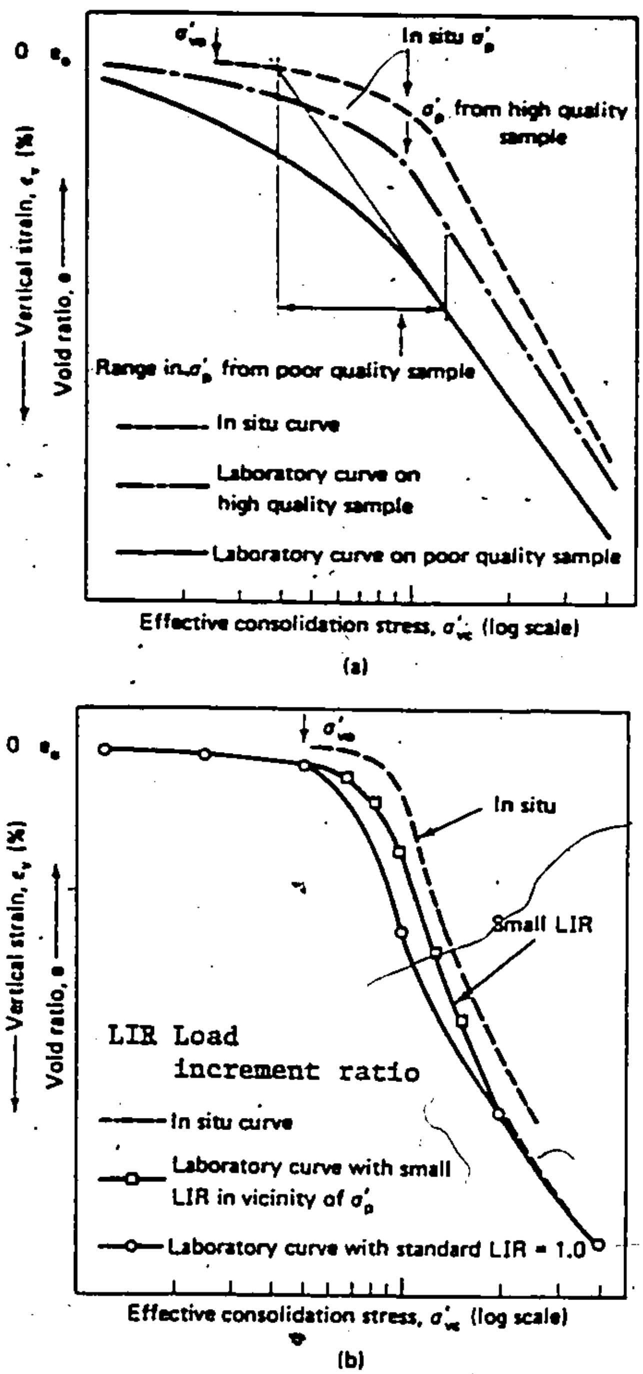


FIG. A.1 (a) EFFECT OF SAMPLE DISTURBANCE

(b) EFFECT OF LOAD INCREMENT RATIO

(after HOLTZ and KOVACS 1981)

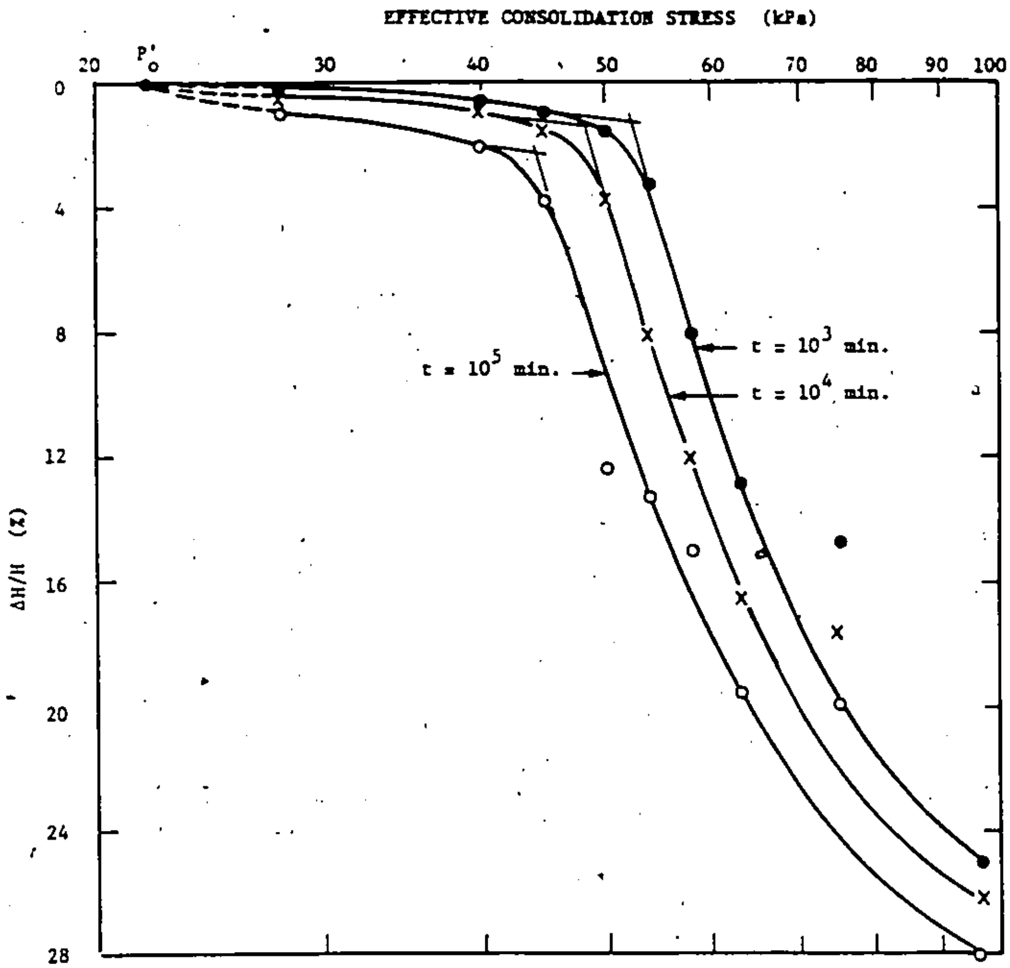


FIG. A.2 EFFECT OF TIME ON σ'_p IN OEDOMETER TESTS, ST. ALBAN 3 m

(after TAVENAS and LEROUEIL 1977)

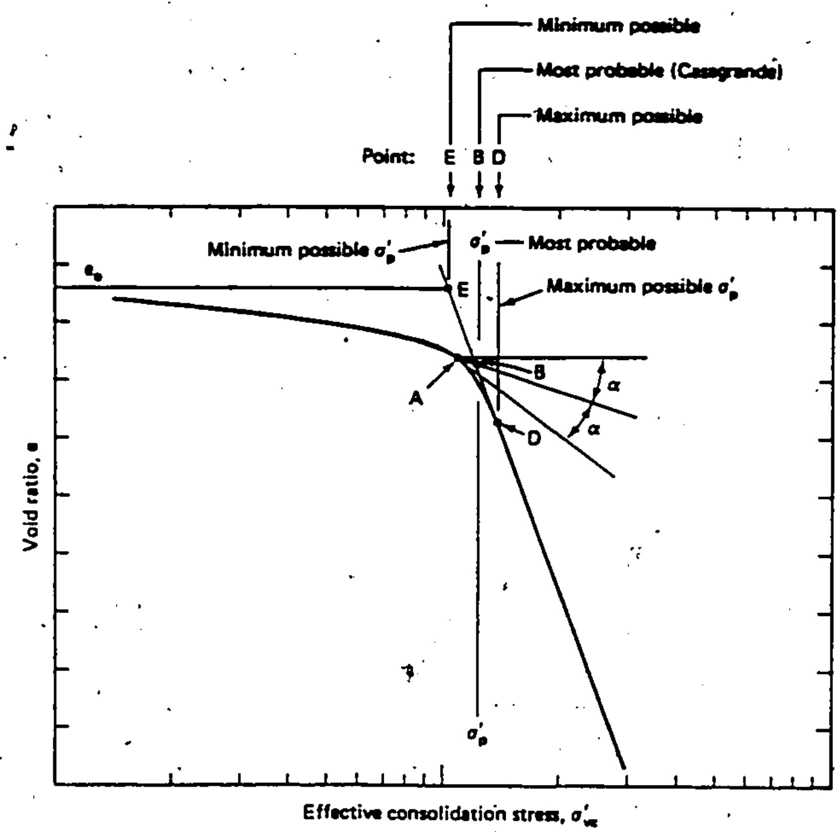


FIG. A.3 GRAPHICAL PROCEDURE FOR DETERMINATION OF PRECONSOLIDATION PRESSURES

(after HOLTZ and KOVACS 1981)

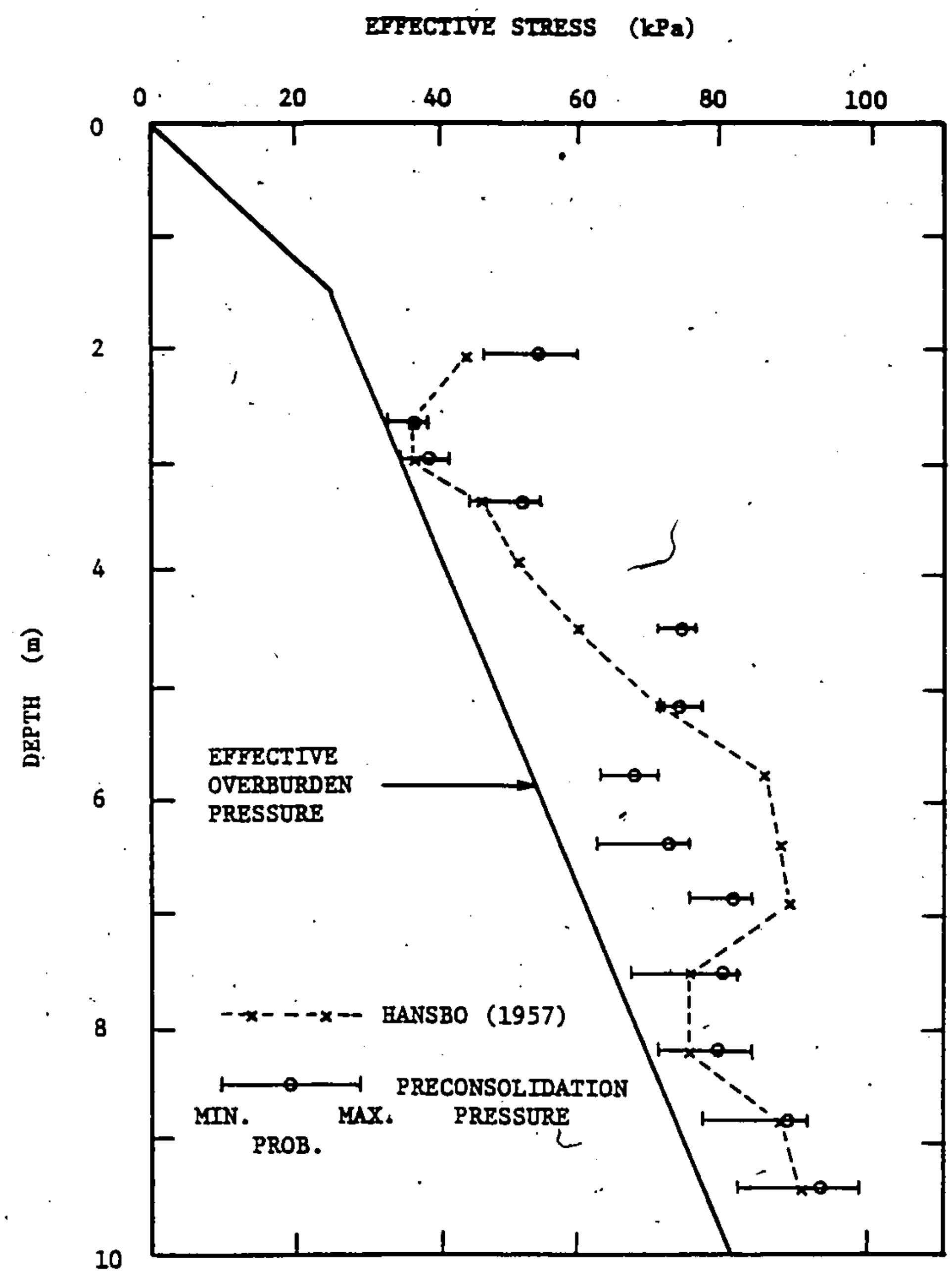


FIG. A.4 PRECONSOLIDATION PRESSURE PROFILES AT NEW LISKEARD

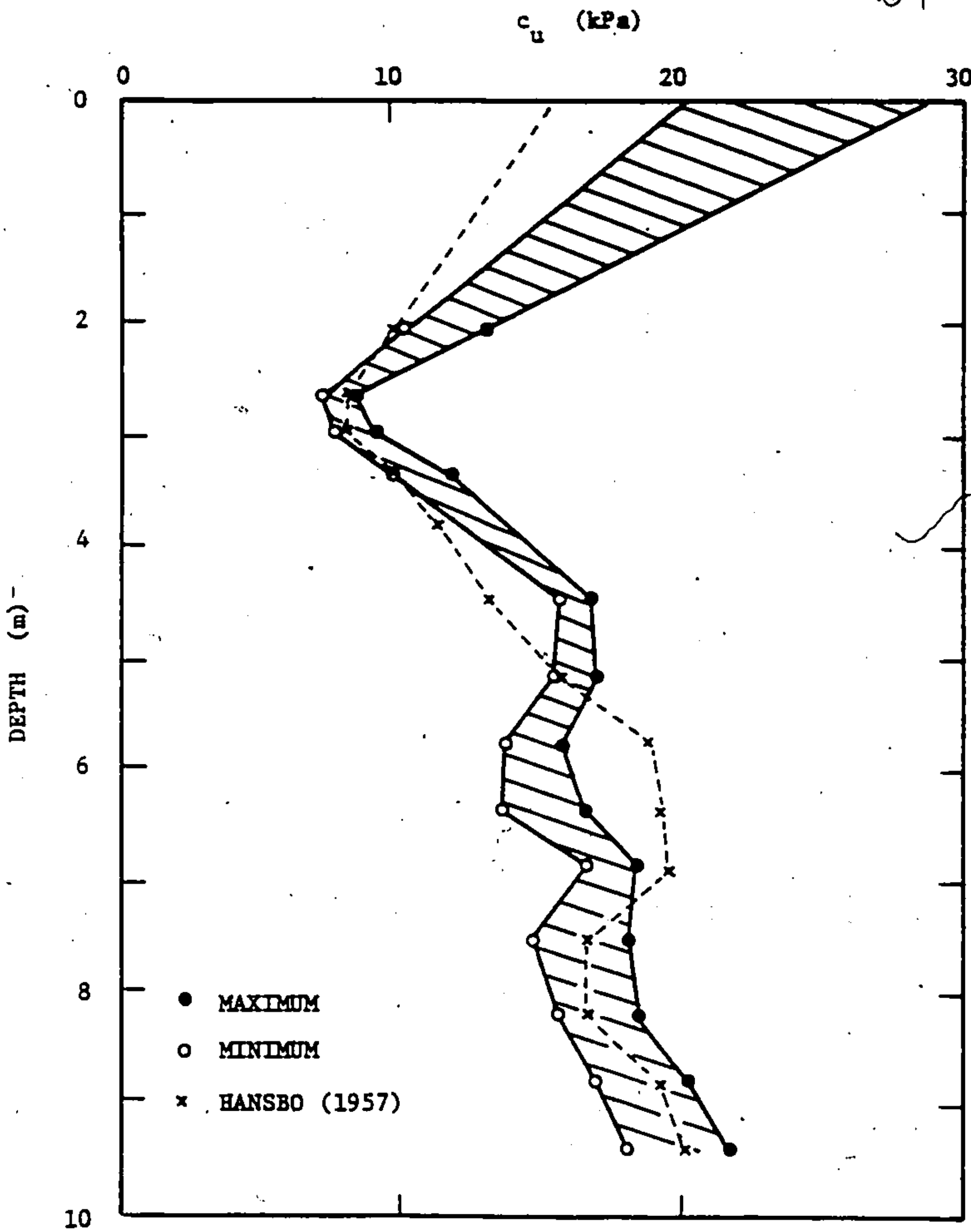


FIG. A.5 SHEAR STRENGTH PROFILES DETERMINED BY $c_u = 0.22\sigma'_p$, NEW LISKEARD