DETERMINATION OF DRAINED AND UNDRAINED SOIL PARAMETERS USING THE SCREW PLATE TEST

by

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ABSTRACT

An evaluation of the screw plate test for use in the determination of soil deformation behavior is presented. Current methods of analysis are used to interpret the screw plate data, and the limitations of each method are discussed. As a result of the extensive test programme, the screw plate is found to provide reproducible estimates of drained moduli in sands, and undrained shear strengths in clays.

Screw plate tests were performed at three research sites where clay, silt, and sand lithologies are encountered. Comparisons are made between deformation parameters obtained from the screw plate test, and those obtained from the pressuremeter, cone penetrometer, dilatometer, vane shear, and triaxial tests. Published correlations are used where applicable, and confirm the suitability of the screw plate test for field investigations in a variety of soil conditions.

The development of a unique test apparatus is also detailed. Special features of this system include the automation of plate insertion, the controlled application of rapid cyclic load histories, and the retrieval of the plate upon completion of the sounding. The adaptation of this system to a conventional hydraulic jacking unit is also described.

Based upon this systematic evaluation of the screw plate test, recommended test procedures and methods of analysis are also summarized. Suggested topics for future research are presented which would further enhance the applicability of the test to foundation design problems.

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1. INTRODUCTION

1.1. Test Principles

The screw plate usually consists of a single flight of a helical auger, with a cross-sectional area ranging from 250 cm^2 to 2000 cm^2. This plate is screwed down to the test depth and a load applied at the surface. The load-deformation behavior of the soil is then recorded, and interpreted to yield drained or undrained moduli, as well as consolidation characteristics and undrained shear strengths.

1.2. Historical Review

Use of the screw plate test appears to have originated approximately 25 years ago. Kummeneje (1956) used the screw plate test to predict settlements of petroleum tanks on sand. Kummeneje and Eide (1961) used the device to assess changes in soil porosity and settlements associated with soil densification through blasting. Gould (1967) presented favourable comparisons between screw plate tests and large plate bearing tests in granular deposits. Webb (1969) conducted screw plate tests in fine to medium sands well below the water table, and produced compressibility correlations for the cone penetration test using compressibility measurements obtained from the screw plate test. Schmertmann (1970) proposed a method of predicting settlements using the Cone Penetration Test, employing the screw plate test as a means of assessing the in-situ deformability of cohesionless soils. Janbu and Senneset (1973) also produced a significant contribution by presenting a detailed method of analysis of the screw plate test for determining deformability and consolidation characteristics of both cohesive and non-cohesive soils.

More recent studies have concentrated on the sophistication of interpretive methods, both in the correlation of test results with other in-situ tests, and their application to more specific foundation problems. Dahlberg (1975) used the test to determine the preconsolidation pressure in sands. Marsland and Randolph (1975) compared the results of screw plate tests with pressuremeter tests. Schwab (1976) and Schwab and Broms (1977) examined the time-dependent and time-independent behavior of silty clays. Selvadurai and Nicholas (1979) provide a comprehensive review of the theoretical assessment of the screw plate test in cohesive soils, and provide a framework within which the screw plate test can be compared with other in-situ tests in cohesive soils. Bodare and Massarch (1982) use a screw plate modified for impulse loading to study in-situ shear moduli. Kay and Parry (1982) evaluated the use of the screw plate test for the determination of moduli, shear strength and coefficient of consolidation in stiff clays. Recent developments include the measurement of installation torque using a torque load cell, and the application of rapid cyclic loads (Berzins and Campanella, 1981).

1.3. Purpose and Scope

The primary objective of this dissertation is to compare deformation parameters obtained from the screw plate test, with those derived from other in-situ and laboratory tests. Review of the screw plate data will be completed using existing analytical techniques, with an emphasis on determining moduli and shear strengths of cohesive and non-cohesive soils.

In order to provide a more comprehensive evaluation of factors affecting the test results and interpretation, an improved installation system was developed, and is discussed herein. Use of this installation system allowed the author to evaluate deformation behavior at a variety of research sites. These test results are reviewed, and suggested guidelines for test procedures and further research are identified.

2. THEORETICAL ANALYSIS OF SCREW PLATE DATA

Screw plate tests have been analyzed to provide data on the deformation behavior of soil. Analytical techniques are summarized in the following sections as they pertain to drained and undrained behavior.

2.1. Drained Parameters

The drained analysis of the test assumes that all consolidation-induced strains occur during the test. This is normally accomplished using incremental loading, with sufficient time allowed between load increments for pore pressure dissipation. The parameters which can be derived from drained analyses are the constrained modulus, M, Young's Modulus, E, and coefficient of consolidation for radial drainage, c_r.

Attempts have also been made to measure an in-situ bearing capacity in sands, (Berzins and Campanella, 1981). It was generally observed that high plate loads in sands resulted in considerable deformation of the plate itself, and that failure loads could not be achieved. Consequently, peak strengths could only be inferred from extrapolation of the test data using a hyperbolic relationship.

2.1.1. Drained Modulus; Janbu Analysis

A method of determining a drained constrained modulus is presented by Janbu and Senneset (1973). The constrained modulus is defined by the expression:

$$M = k_m \, p_a \left(\frac{p'}{p_a}\right)^{1-a} \qquad (2.1)$$

where M = constrained tangent modulus (Janbu, 1963)

k_m = modulus number

p_a = reference stress, (normally 1 bar)

p' = vertical effective stress

a = stress exponent = 1 for O.C. clays

= .5 for sand and silt

= 0 for N.C. clays

It should be noted here that intuitively the stress exponent 'a' should vary with the O.C.R. in overconsolidated clays; however this range is not documented.

Janbu and Senneset (1973) then use a construction as illustrated in Figure 2.1 to determine the modulus number k_m, using the following equation:

$$\delta = \frac{S}{k_m} \, \frac{p_n B}{p_a} \qquad (2.2)$$

in which: δ = plate deflection

S = dimensionless settlement number

k_m = modulus number

p_a = reference stress (normally 1 bar)

p_n = net stress on plate = $p - p'_o$

B = plate diameter.

p = applied stress on plate

p'_o = initial vertical effective overburden pressure.

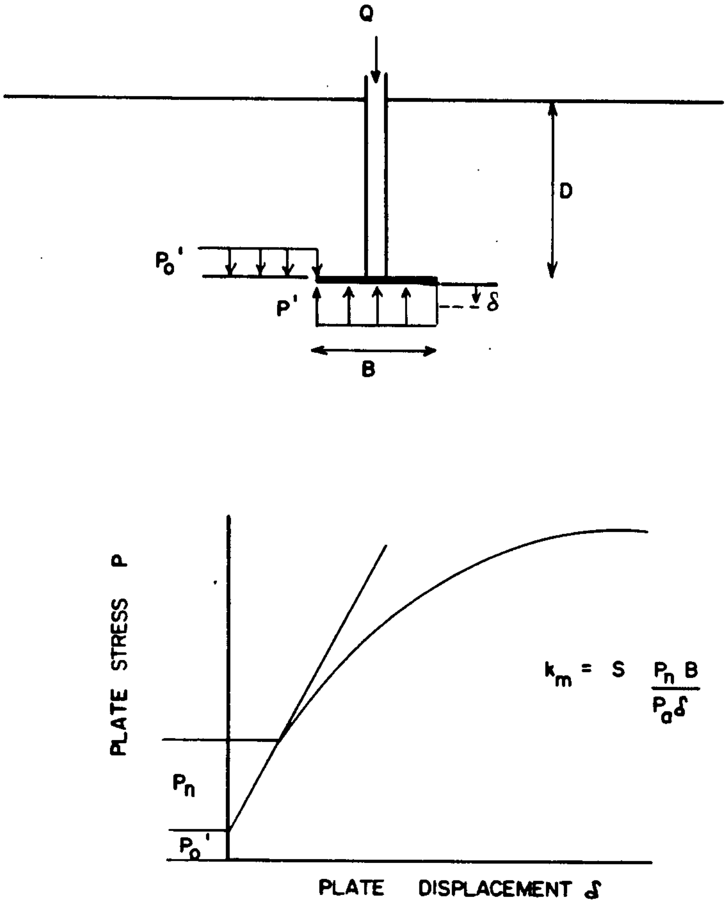


FIGURE 2·1 - DEFINITION OF JANBU MODULUS NUMBER k_m

By measuring the net plate stress 'p_n' and deflection 'δ' in the field, and using the solutions for 'S' given in Figure 2.2, a modulus number 'k_m' can be determined. From this, a constrained modulus can be calculated at each depth for a uniform soil.

Janbu's analysis assumes that the soil is homogeneous, elastic, isotropic, and that all strains are vertical. In addition, their theory does not compensate for variations in plate stiffness; consequently in dense sands the constrained modulus will be underestimated. An advantage of their method is that it recognizes that the modulus is very much dependent upon stress level; consequently, it provides a method whereby the variation of modulus with depth is accounted for.

In addition, the use of the initial tangent portion of the load curve, rather than subsequent unload-reload curves, reduces the compounding effects of continued disturbance. Since the modulus of the soil is very much dependent upon the stress level and history, the use of repeated loading curves can only introduce further uncertainty in the determination of a representative modulus. Janbu's method, on the other hand, utilizes the initial portion of the loading curve, which represents the soil's "least disturbed" state.

The primary uncertainty in defining the initial portion of the curve is related to the selection of an applied stress level which results in the in-situ "undisturbed" state. Arching of stresses immediately behind the plate, which generally occurs in sands, may effectively reduce the residual load on the back of the plate, hence the applied stress may not necessarily be superimposed upon the in-situ stress level.

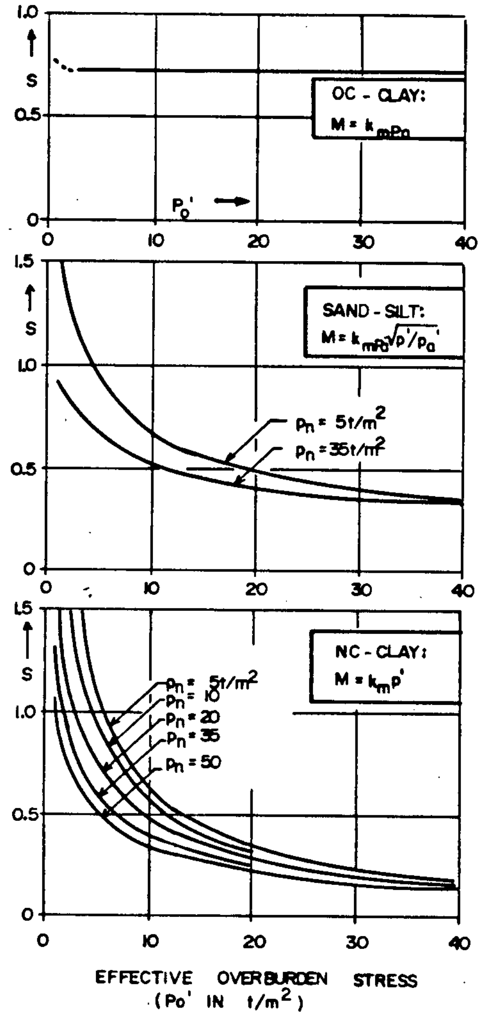


FIGURE 2·2 - VALUES OF JANBU'S SETTLEMENT NUMBER "S"

(AFTER JANBU AND SENNESET, 1973)

For the purpose of this study, it was assumed that arching did occur. Consequently, constrained moduli were obtained by drawing tangents to the test curve when the applied stress equalled the overburden stress.

2.1.2. Drained Young's Modulus; Schmertmann Analysis

Schmertmann (1970) presents a method whereby the elastic modulus "E_s" can be determined using a simplified strain influence factor. His analysis assumes a homogeneous, isotropic, elastic half space.

The Schmertmann (1970) analysis is derived from Ahlvin and Uhlery (1962) who define the vertical strain at depth beneath a uniform circular load by:

$$\varepsilon_z = \frac{p}{E}(1 + \nu) \lfloor (1-2\nu)A + F \rfloor \qquad (2.3)$$

where:
- ε_z = vertical strain at depth
- p = vertical stress on circular plate of radius 'r'
- E = elastic modulus
- ν = Poisson's ratio
- A, F = dimensionless factors which are a function of the point location.

By assuming "p" and E are constant, the vertical strain is shown to depend on a vertical strain influence factor, I_z, where:

$$I_z = (1+\nu) \lfloor (1-2\nu)A + F \rfloor \qquad (2.4)$$

The distribution of the influence factor is shown in Figure 2.3. This simplified distribution is based upon Schmertmann's review of theoretical and experimental analysis.

Schmertmann then proposes corrections to the strain influence factor which consider the effects of strain relief due to embedment and settlement due to creep.

The embedment correction applied to compensate for an assumption inherent in the elastic model; in which the overlying material can produce tensile stresses which reduce the strain felt at the bearing level. A linear correction factor is applied, whereby:

$$C_1 = 1 - 0.5\left(\frac{p_o}{\Delta p}\right) \qquad (2.5)$$

where C_1 = embedment correction factor

p_o = in-situ vertical pressure

Δp = $p-p_o$ = net foundation pressure increase.

The correction for creep can be first attributed to Nonveiler (1963), who suggested the following logarithmic relationship:

$$\rho_t = \rho_o[1 + \beta \log(t/t_o)] \qquad (2.6)$$

where ρ_t = settlement at time t

ρ_o = reference settlement at t_o

β = constant = 2 to 3.

Based on case histories, Schmertmann suggests that $t_o \cong 3.2 \times 10^6$

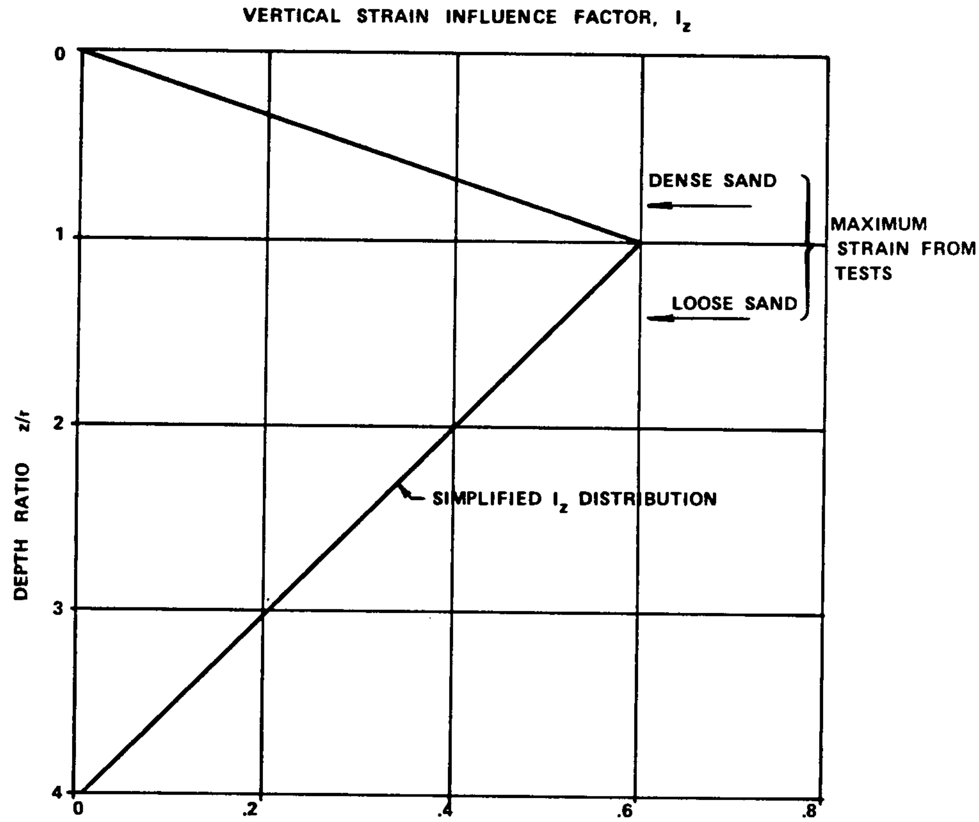


FIGURE 2·3 - SIMPLIFIED DISTRIBUTION OF VERTICAL STRAIN BENEATH LOADED CIRCULAR AREA

AFTER SCHMERTMANN (1970)

seconds, hence for a rapid plate load test in sand, the correction factor:

$$C_2 = 1 + \beta \log(t/t_o) \qquad (2.7)$$

does not converge. Consequently, creep cannot be considered a factor in rapid tests in sand, and $C_2 = 1$.

Application of the embedment correction to the general equation describing vertical strain, and integration over the depth of influence yields:

$$\rho = \int_o^\infty \varepsilon_z \, dz \simeq \Delta p \int_o^{2B} \left(\frac{I_z}{E_s}\right) dz$$

$$\simeq C_1 \, \Delta p \sum_0^{2B} \left(\frac{I_z}{E_s}\right) \Delta z \qquad (2.8)$$

The determination of E_s can be made by back calculation from the results of the screw plate test, and the above equation. An assumption of a constant modulus within the strain area beneath the plate yields the following expression from which E_s can be determined:

$$E_s = C_1 \frac{\Delta p}{\rho} 1.2 \, B \qquad (2.9)$$

where: E_s = equivalent Young's modulus

C_1 = embedment factor = $1 - (.5)(\frac{p_o}{\Delta p})$

Δp = applied plate stress

ρ = measured plate deflection

B = plate diameter.

The modulus can then be calculated by assuming a tangent or secant to the stress-deformation curve, depending upon the stress level of interest. It should also be noted that Schmertmann (1970) recommends modulus be determined over a stress range 1 to 3 tsf (≈ 1 to 3 kg/cm^2), which is a typical design value for shallow foundations. The method cannot be confidently applied at low initial plate stresses, particularly at depth since the method has been developed for a typical stress range.

2.1.3. Coefficient of Radial Consolidation

The screw plate test can also be used to determine the coefficient of consolidation (Janbu and Senneset, 1973, Kay and Avalle, 1982). Janbu and Senneset (1973) present a method whereby axisymmetric or one-dimensional consolidation theories can be applied to incremental load tests in cohesionless soil. By using the basic relationship:

$$c = \frac{Td^2}{t} \tag{2.10}$$

where: c = coefficient of consolidation
d = drainage path
t = time after load increase
T = dimensionless time factor,

they analyze the consolidation beneath the loaded plate, which is assumed to be governed by essentially radial drainage. By using the construction shown in Figure 2.5, a coefficient of radial drainage can be determined by using the following equation:

$$c_r = T_{90} \frac{R^2}{t_{90}} = .335 \frac{R^2}{t_{90}} \tag{2.11}$$

where: c_r = coefficient of radial consolidation
R = plate radius = length of drainage path "d"
t_{90} = time for 90% consolidation
T_{90} = time factor for 90% consolidation = 0.335.

Kay and Avalle (1982) propose a revised method for determining c_v, whereby more realistic drainage conditions are assumed. Their method is summarized in Figure 2.4.

The field measurement of the coefficient of consolidation requires incremental load tests of lengthy duration. As a result, verification of the aforementioned theories was not undertaken in this study. The analyses have been presented, however, for completeness.

2.2. Undrained Parameters

Undrained parameters are obtained from rapid load tests in fine silts and clays in which the rate of load application precludes significant pore pressure dissipation. Undrained analyses of screw plate data yields estimates of an undrained modulus and undrained shear strength.

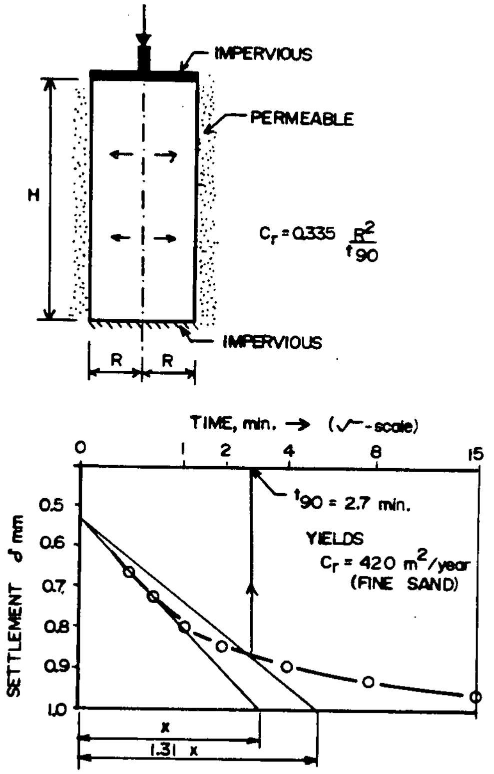
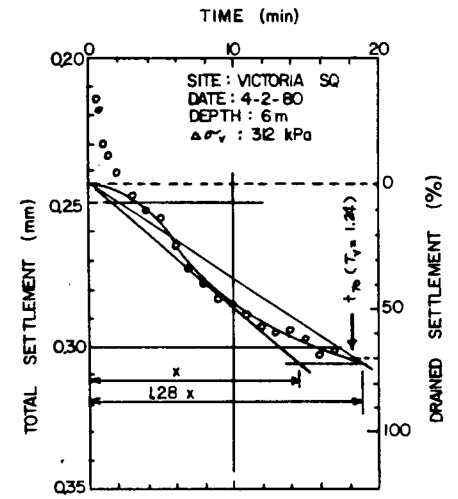


FIGURE 2·4 - DETERMINATION OF COEFFICIENT OF CONSOLIDATION

JANBU METHOD

(AFTER JANBU AND SENNESET, 1973)



D = PLATE DIAMETER

t_{70} = time to 70% dissipation

$c_v = \frac{.31\ D^2}{t70}$

FIGURE 2·5 - DETERMINATION OF COEFFICIENT OF CONSOLIDATION

KAY AND AVALLE METHOD, 1982

2.2.1. Undrained Elastic Modulus

Selvadurai and Nicholas (1979) provide a theoretical assessment of the screw plate tests in a homogeneous, isotropic, elastic medium. They present a number of closed-form solutions which consider the effects of plate rigidity and the plate-soil interface. The closed form solutions follow the basic relationship:

$$\frac{\delta}{pa/E_u} = \lambda \tag{2.12}$$

in which: δ = plate displacement

p = average stress on plate

a = plate radius

E_u = undrained, elastic Young's modulus

λ = modulus factor.

The modulus factor 'λ' is a function of Poisson's ratio and the degree of bonding with the plate. The results of a review by Selvadurai et al. (1979) are presented in Table 1.1. They conclude that the undrained modulus can be approximated by the expression:

$$\frac{\delta}{pa/E_u} = 0.60 \text{ to } 0.75 \tag{2.13}$$

The upper limit applies when the plate is partially bonded to the soil, which might be the case in a sensitive clay. Kay and Parry (1982) suggest that a value of .66 be adopted as a reasonable

TABLE 1.1. SUMMARY OF MODULUS FACTORS 'λ'.

Solution	$\frac{\delta}{pa/E_u} = \lambda$	Reference
(a)	0.630	Kelvin (1890)
(b)	0.589	Collins (1962, Kanwal and Sharma (1976), Selvadurai (1976)
(c)	0.750	Hunter and Gamblen (1974)
(d)	0.750	Keer (1975)
(e)	0.648	Selvadurai (1979a)
(f)	0.585	Selvadurai (1976)
(g)	0.730	Christian and Carrier III (1978) Pells and Turner (1978)
(h)	0.525	Christian and Carrier III (1978) Pells and Turner (1978)

a = radius of screw plate
p = average stress on screw plate = $\frac{P}{\pi a^2}$
E_u = undrained modulus

δ = plate displacement

Solution	Remarks
(a)	average displacement of uniform load
(b)	displacement of fully bonded rigid disc
(c)	displacement of smoothly embedded rigid disc
(d)	displacement of partially bonded rigid disc
(e)	central displacement of flexible disc
(f)	displacement of rigid spheroidal region
(g)	average displacement of deep borehole subjected to uniform load
(h)	displacement of rigid plate at base of deep borehole

(After Selvadurai et al. 1979)

approximation for partial bonding in most clays.

Selvadurai et al. (1979) studied the disturbance associated with plate installation by performing full scale model tests. They concluded that the disturbance induced by rotation of the plate through a clay was minimal. This finding is reflected in the analysis they propose, whereby the effect of stress relief is not specifically addressed. Selvadurai et al. (1979) attempt to eliminate this uncertainty by analyzing the unload-reload portion of the test curve for the modulus determination, rather than the initial tangent portion. The validity of this method cannot be verified theoretically, however Selvadurai as well as Kay and Parry (1982) found reasonable agreement between the initial tangent moduli and subsequent reload moduli.

The uncertainty in the determination of an undrained elastic modulus in an ideal media will be further complicated when any of Selvadurai et al.'s (1979) assumptions are violated. This is particularly true in many soil deposits which exhibit heterogeneity and strength anisotropy. The combination of stress relief, soil variability and preshearing during plate installation make an accurate determination of undrained modulus tenuous at best.

2.2.2. Undrained Shear Strength

The undrained shear strength can be determined from the screw plate test by using the expression of the bearing capacity of a deep circular footing, whereby:

$$c_u = \frac{p_{ult} - \sigma_{vo}}{N_K} \tag{2.14}$$

in which: c_u = undrained shear strength

p_{ult} = ultimate average plate stress

σ_{vo} = total overburden stress

N_K = bearing capacity factor.

Again the undrained shear strength depends upon boundary conditions including the soil-plate interface and plate stiffness. Selvadurai et al. (1979) reviewed classic theoretical and empirical solutions, and concluded that:

$$\frac{p_{ult}}{c_u} = N_K = 9.0 \text{ for partial bonding} \tag{2.15}$$

$$= 11.35 \text{ for full bonding}$$

It should be noted here that factors including strain rate effects, strength anisotropy and progressive failure are not specifically addressed in current methods of screw plate analysis.

Failure loads were achieved in the clays tested during this study. However, it is recognized that load limitations with a more conventional test apparatus may preclude the development of sufficient plate stress to cause failure, particularly in stiff clays. Kay and Parry (1982) propose a method whereby the load-displacement curve can be extrapolated to obtain the ultimate plate capacity, and hence undrained shear strength. By measuring the plate deflection at two specific points on the stress-displacement curve, they estimate the ultimate plate load using the following hyperbolic relationship;

$$p_{ult} = 2.54\ p_y - 1.54\ p_x \tag{2.16}$$

where:

p_{ult} = the ultimate plate stress,

p_x = the plate stress at a strain equal to 1.5% of the plate diameter (B)

p_y = the plate stress at a strain equal to 2% of B.

3. DEVELOPMENT OF TEST APPARATUS AND PROCEDURE

3.1. Development of Test Apparatus

Upon initiation of the research project, the available literature was collated in an effort to define the design criteria for the plate configuration and installation system. Development of an automated test apparatus concentrated upon the adaptation of the test to a hydraulically operated CPT rig developed at U.B.C. A schematic representation of the screw plate system is presented in Figure 3.1, and is elaborated upon in subsequent sections.

The system developed incorporated several features which extended its testing capability. The servo-controller system was used to apply cyclic loads to the plate, and could also be used to apply strain-controlled loading if a displacement transducer was used. Other special features included the measurement of torque during plate installation, and the capability to vary rates of rotation and thrust in a controlled fashion.

3.1.1. Screw Plate Configuration

Previous researchers studying the screw plate have utilized a single flight of helical auger, ranging in area from 250 to 2000 sq. cm. The aspect ratio, (i.e., the ratio of the half-pitch to diameter), is reported to be between 0.1 and 0.2. Thickness of the plate, hence its relative rigidity, has been discussed by Selvadurai and Nicholas (1979), but has not been extensively treated in published literature. In order to provide a pivotal point around which the helical auger can rotate, the screw plate has a conical

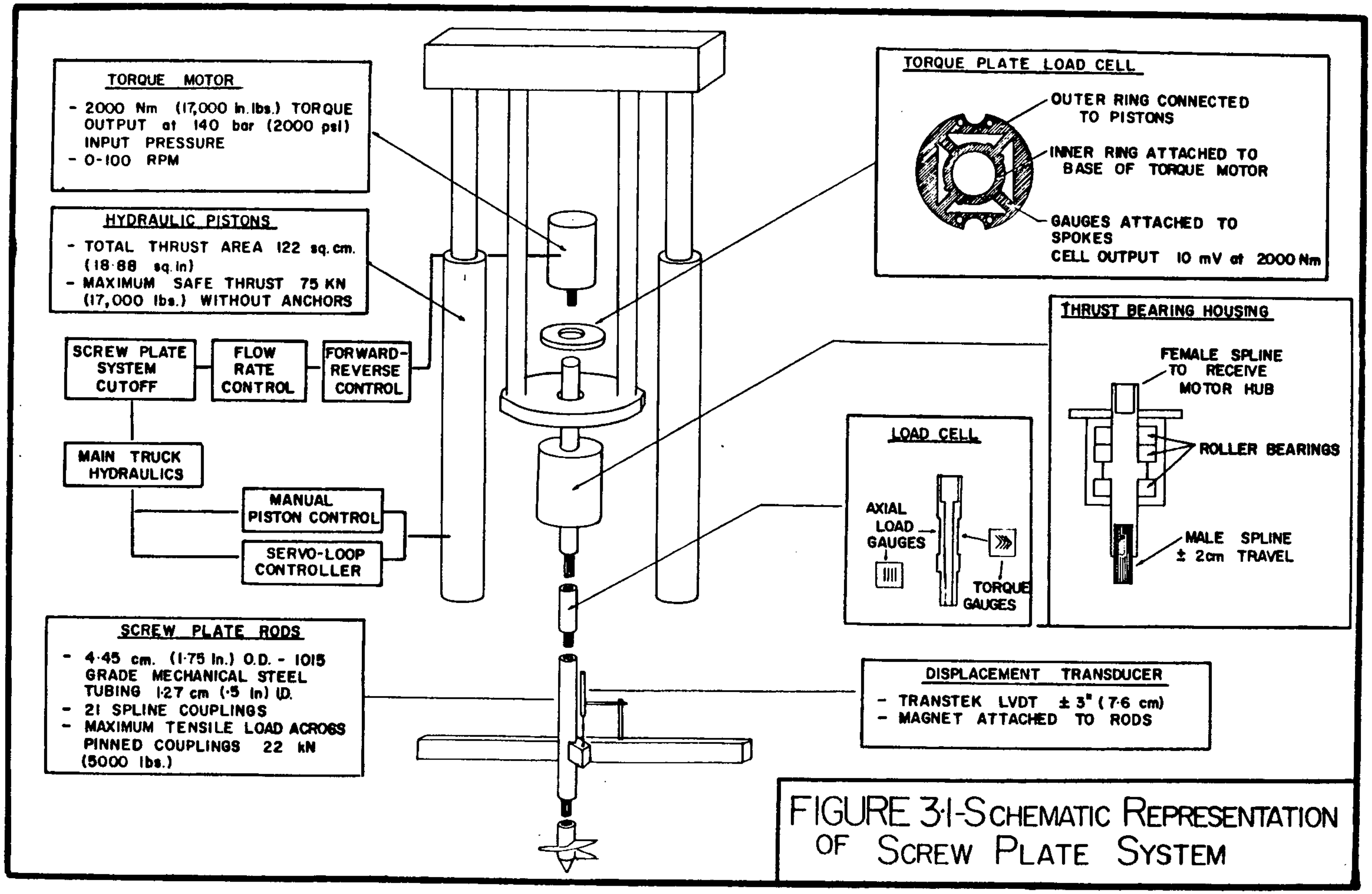


FIGURE 3·1-SCHEMATIC REPRESENTATION OF SCREW PLATE SYSTEM point in advance of the plate itself.

In order to assess the viability of a particular plate design, one must consider the various components of plate resistance. Preliminary calculations based upon the mechanisms of resistance, and available CPT data on the tip and sleeve resistance of various soil deposits revealed that the plate should satisfy the following criteria:

1) reduced friction between the plate surface and the soil,
2) reduced surface area on the leading (cutting) edge of the plate,
3) reduced edge area,
4) reduced diameter of the conical tip.

Preliminary calculations indicated that a single flighted auger would require approximately 50% less torque to install than a double helical plate. During the course of the field testing, however, it was found that plate size (for a single flighted auger), had a minimal effect on the total torque which was required during installation in most soils. The total installation torque became an important factor in dense sands, where the capacity of the system may be reached, and cyclic torsional loads required.

The plate stiffness is a key parameter in stiff soils, as increased flexibility results in higher measured deformations; hence an underestimation of the elastic modulus. The effect of plate rigidity is further discussed when the test data are presented.

It was generally observed that the double flighted plate was self-centering, and tended to "wander" less during installation. As a result, it is believed that the use of a double flighted plate leads to reduced soil disturbance, particularly in dense sands, and also permits symmetrical loading on the plate.

The "wandering" phenomena observed during the installation of the single-flighted plate was most likely the result of a tendency for the plate to rotate about a point located on the cutting edge, rather than the central conial tip. The eccentricity of the center of rotation increased the total torsional force required during installation, since the applied torque was no longer being used solely to overcome the bearing resistance of the cutting edge. The conical tip has to be quite large to overcome the tendency to wander; hence the zone of disturbance below the plate is increased considerably. The use of a self-centering, double-flighted plate allows the size of the conical tip to be reduced; consequently soil disturbance will be less.

Another important consideration in the selection of the plate is that it should be recovered at the end of the sounding. This was found to be quite important, as with the Swedish cast-iron plate, (see Fig. 3.2) which was found to have broken during a load test in dense sands. By recovering the plate, one can examine it for evidence of buckling or breakage, which would significantly alter the interpretation and hence reliability of the data. In order to accommodate this improvement during this study, splined rods with pinned connections were used to permit removal of the plate.

Figure 3.2 illustrates the double helical plates employed in this study.

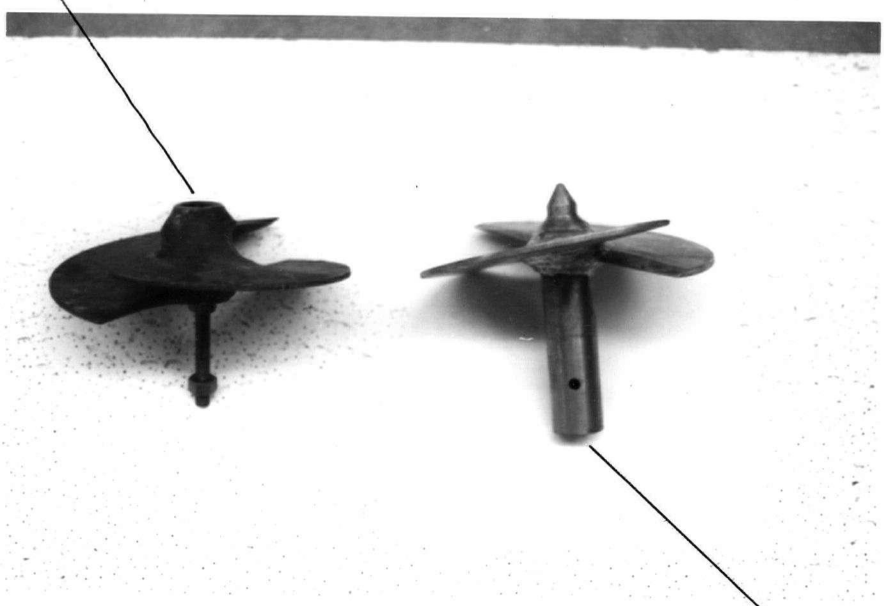


Figure 3.2 - Example of Double Pitched Screw Plates

3.1.2. Effect of Plate Stiffness

Selvadurai and Nicholas (1979) studied the relative effect of plate rigidity in determining an undrained modulus. By examining their analysis, we can gain some insight into the effect of varying plate stiffness. Selvadurai (1979) developed an expression for the behavior of a flexible plate in contact with an elastic medium:

$$\frac{\delta}{pa/E} = \frac{\pi(3-4\nu)(1+\nu)(144-60\ \xi_a + 90R)}{16(1-\nu)(64+90R)} = \lambda \tag{3.1}$$

where:
- δ = plate deflection
- p = average plate load
- a = plate radius
- E = soil modulus
- ν = Poisson's ratio for soil
- λ = modulus factor
- ξ_a = radius of rods behind plate

R is defined as the relative rigidity of the plate, where:

$$R = \frac{\pi(3-4\nu)(1+\nu)}{12(1-\nu_p)(1-\nu)} \frac{E_p}{E} \left(\frac{h}{a}\right)^3 \tag{3.2}$$

in which: h = plate thickness

$\{\nu_p\}$ and $\{E_p\}$ = elastic constants for plate.

The solution of this equation for varying plate stiffnesses yields

$$\lambda = \frac{\delta}{pa/E} = .589 \quad \text{for} \quad \log R = 2 \quad \xi_a = .25 \tag{3.3}$$

$$= .883 \quad \text{for} \quad \log R = 0$$

(After Selvadurai (1979))

This formula provides a basis whereby a stiffness correction factor can be applied to the test data. By re-arranging 3.3, the relationship becomes:

$$E = \lambda \frac{pa}{\delta} \tag{3.4}$$

From this, a relationship between moduli determined from stiff and flexible plates is determined, where

$$E_s = \beta E_f \tag{3.5}$$

where: E_s = modulus determined by a stiff plate test,

E_f = flexible plate modulus,

β = stiffness correction factor
= 1.5 for a flexible plate
= 1.0 for a stiff plate.

Admittedly, Selvadurai's solution is for undrained behaviour, consequently volume change due to shear as observed in sands is not accounted for. Nevertheless, the stiffness factor represents a method by which the screw plate moduli can be normalized for plate stiffness. An increase in plate rigidity, R, or the radius of the push rods, ξ_a, will reduce the correction factor and produce more consistent E determinations.

3.2. Installation System

Screw Plate Rods

In evaluating various alternatives for the screw plate rods, consideration was given to satisfying the forces required during installation of the plate and during load tests. The cross-sectional area of the loading rods chosen ensured a minimal amount of elastic compression during loading, (0.25 centimeters at 90 kN load for 20 metres of rod). In addition, in anticipation of conducting repeated torsional shear tests in-situ, the torsional twist was minimized. The connections between rods are splined in order to allow clockwise and counter-clockwise rotation during installation and removal of the plate. Set screws are employed in order to prevent accidental disengagement of the rods, and to permit the rods to be pulled up where the soil provides insufficient reaction during withdrawal of the plate. The rods are hollow in order to allow for future instrumentation of the plate itself, as well as permitting the use of internal rods in the future in order to apply the load directly at the plate, and thus eliminate the effect of rod friction.

Torque Motor

Preliminary calculations indicated that installation of the screw plate would require approximately 2000 Nm of torque. Selection of a suitable torque motor was governed not only by required power, but also by space limitations within the preexisting hydraulic pistons, as well as the input pressures provided by the truck hydraulics. The motor selected is capable of operating continuously at low rates (1 to 100 RPM) and high loads (up to 20 kN). Figure 3.3 illustrates the positioning of the torque motor.

Hydraulic Loading Pistons

The screw plate system was essentially designed 'around' the hydraulic pistons initially developed for use with the CPT test. Full details of the hydraulic system are given elsewhere (Campanella and Robertson, 1981). The system is limited to producing 75 kN of thrust, which is the reaction force provided by the truck. To be compatible with the screw plate system, the hydraulic pistons had to provide a variable rate of advance, with an additional control of pressures applied to the rods during installation. The advance of the pistons can be controlled either manually, or through a servo-loop system. Figure 3.4 illustrates the schematics of the hydraulic control system.

In order to enable the torque motor to apply torque to the screw plate rods, and at the same time apply an axial load, a thrust bearing was designed. This bearing permits the application of either a compressive or tensile force on the rods during screw plate advancement or withdrawal.

3.3. Data Acquisition System

A particular refinement of the measuring system over conventional screw plate systems was the design of a torque load

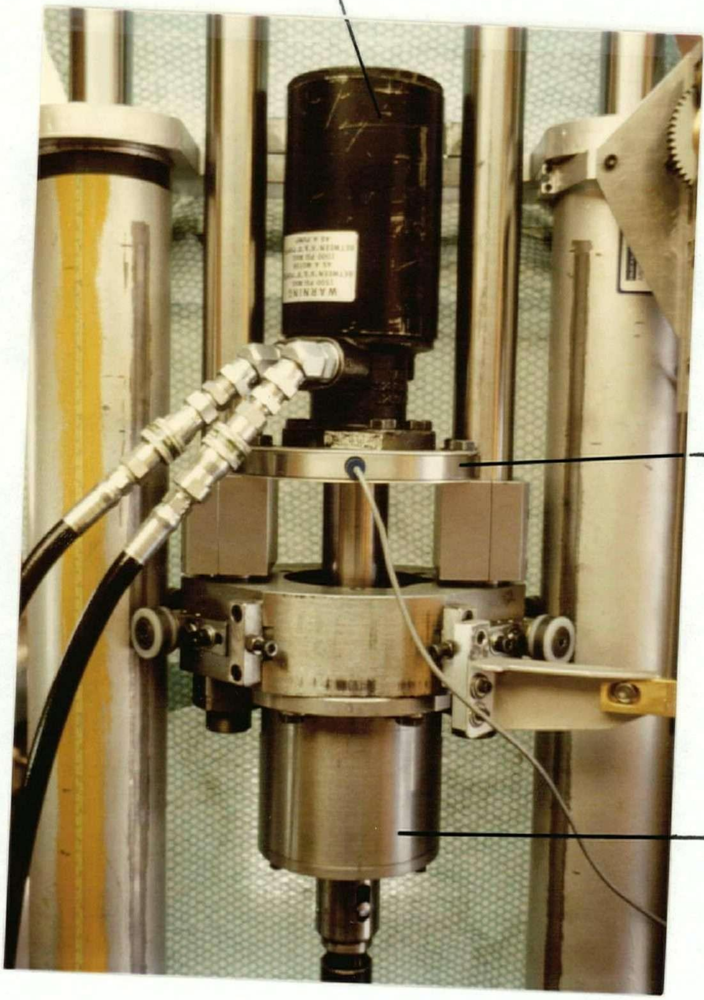


FIGURE 3.3 POSITION OF TORQUE MOTOR

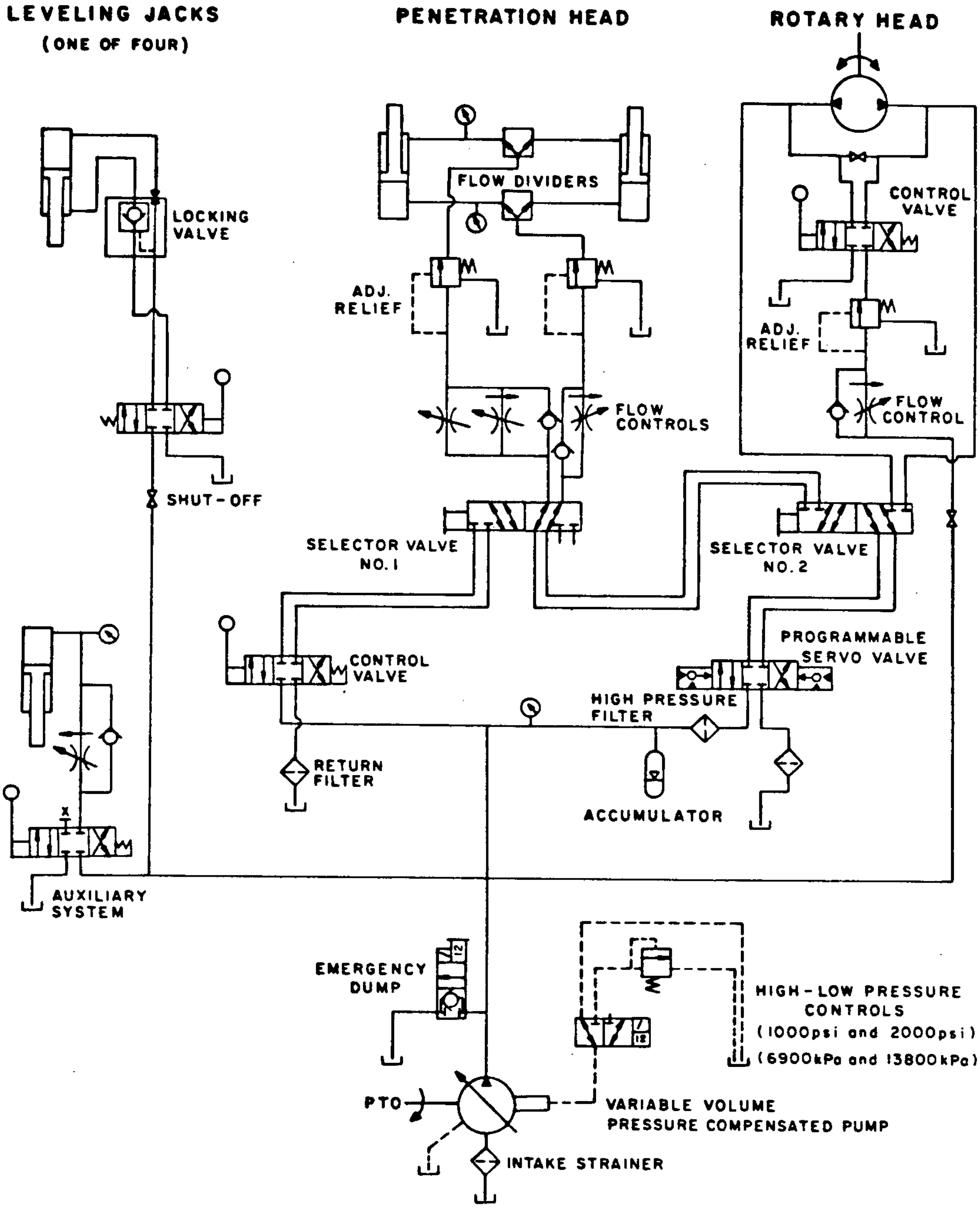


FIGURE 3·4 - SCHEMATIC OF HYDRAULIC CONTROL SYSTEM

after Campanella & Robertson (1981)

cell which enable the operator to measure installation torque during the advancement of the plate. Figure 3.5 shows the principle of the torque load cell.

Axial loads, hence the load on the plate, were measured using an axial load cell shown in Figure 3.6. Axial loads were also estimated using a pressure transducer which recorded the pressure in the hydraulic system. This pressure was multiplied by the cross-sectional area of the pistons to obtain an estimate of the total applied load.

Axial displacements were recorded using a direct current displacement transducer (DCDT) mounted on a reference beam (Figure 3.7). The reference beam was employed to ensure that changes in surface load beneath the truck pads did not influence the deformation measured at the rods.

The parameters measured during the test consist of:

1) installation torque
2) axial load
3) axial displacement

X-Y-Y' chart recorders were used to record the installation torque and the load-displacement curves. Loads and displacements were also routinely recorded on time plots as well. Figure 3.8 shows a typical layout of recording equipment.

The transducer outputs were also used as input signals to an MTS servo-controller. In this manner, strain or stress controlled tests could be performed, with variable amplitudes and magnitude. The hydraulic system could routinely provide 1 hz load cycles in dense sands. Lower frequency loading of 0.1 hz was used in silts

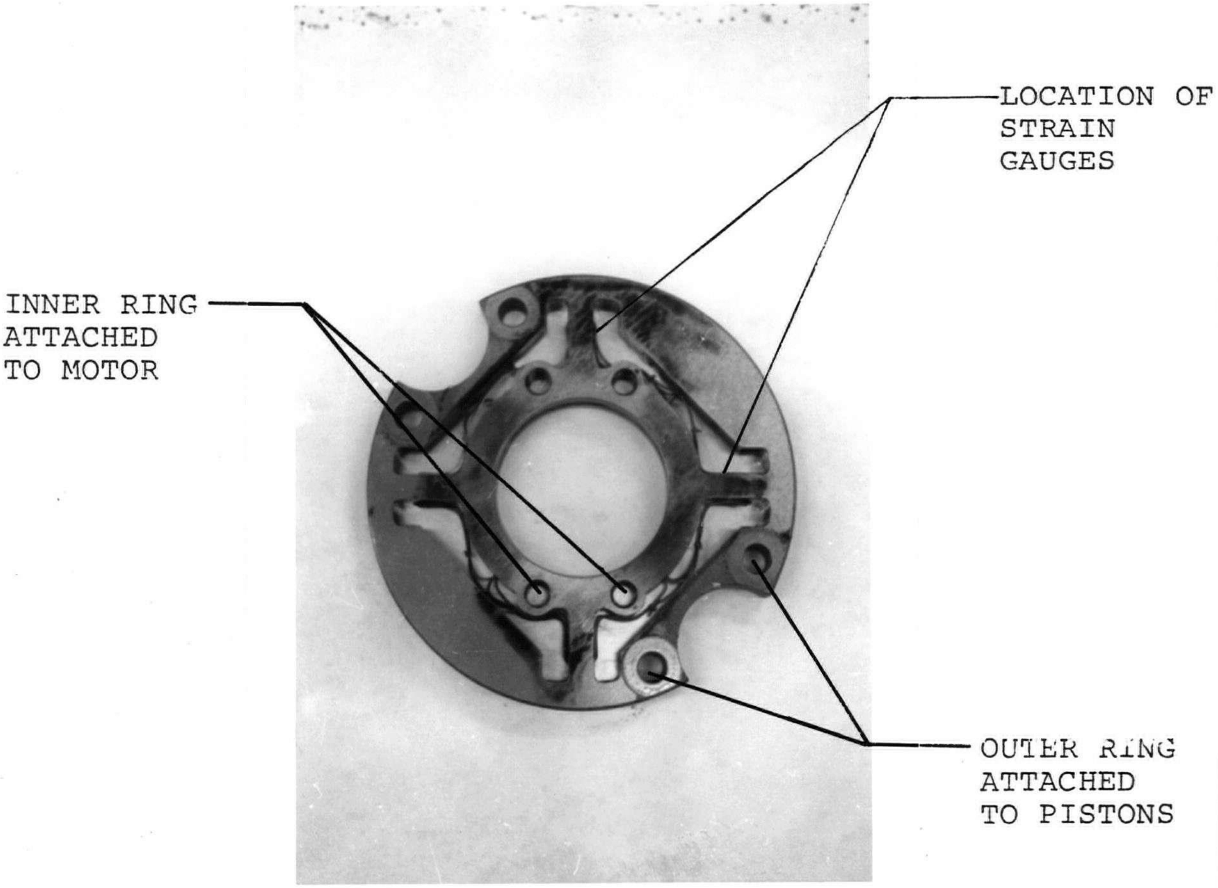


FIGURE 3.5 TORQUE LOAD CELL

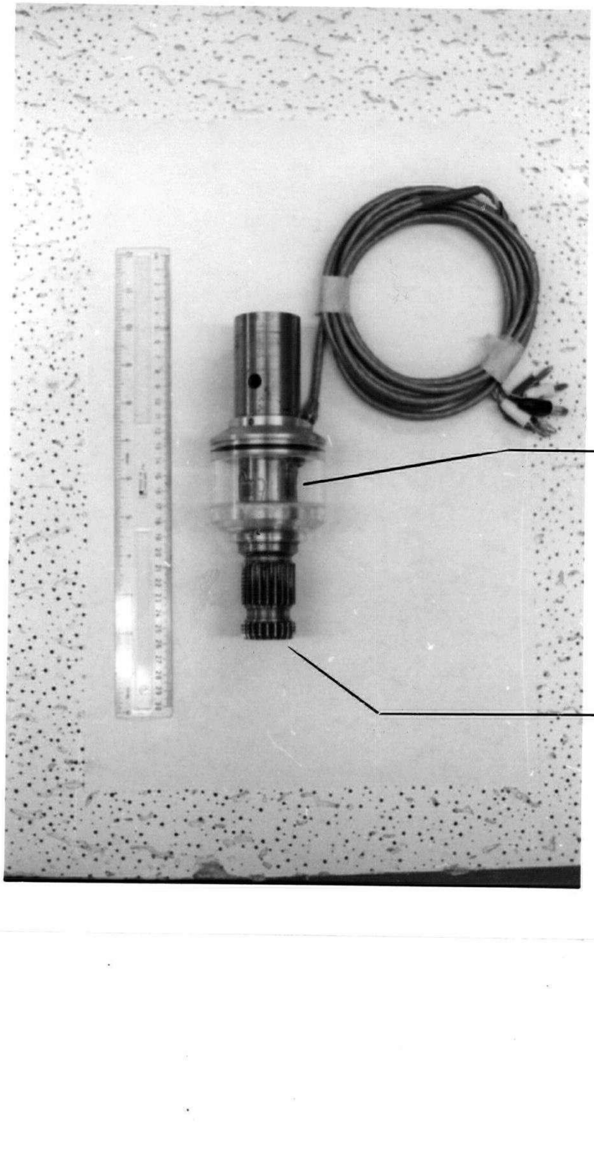


FIGURE 3.6 AXIAL LOAD CELL

FIGURE 3.7 REFERENCE BEAM IN POSITION

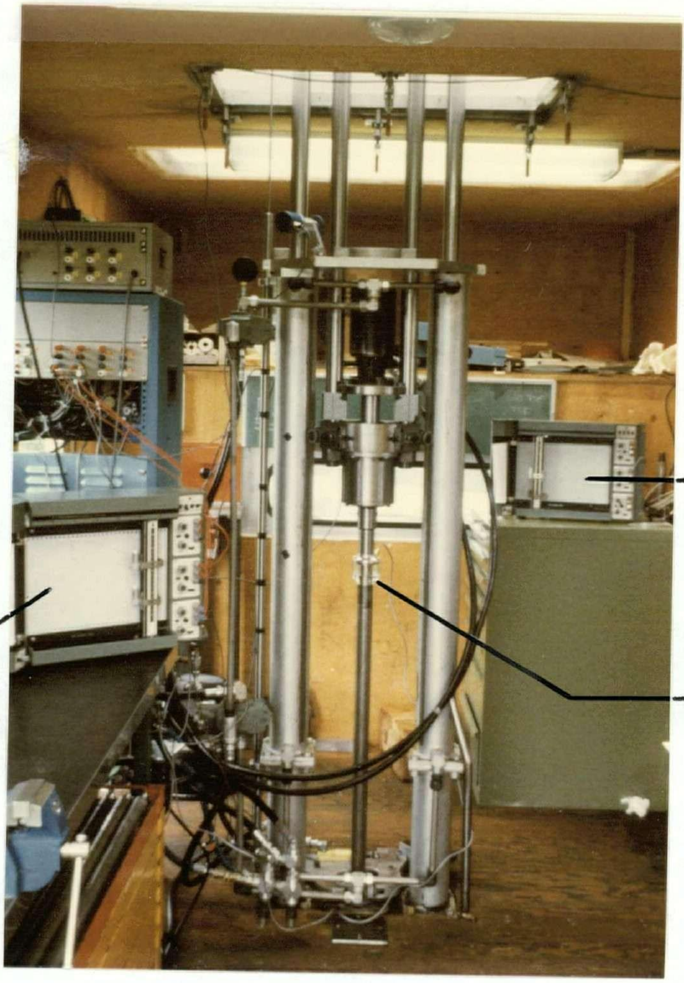


FIGURE 3.8 SCREW PLATE RECORDING SYSTEM

where large amplitude strain resulted in considerable fluid displacement in the pistons. Square, triangular or sinusoidal waveforms could also be applied.

3.4. Development of Test Procedures

3.4.1. Drained Tests in Sand

Gould (1967) identifies a suggested load procedure whereby loads are incrementally applied to the plate, with consolidation/compression permitted before each additional load increment is applied. Dahlberg (1975) specifies that a t_{90} be reached for each load increment, as the coefficient of consolidation can be determined during the load test using Eqn. 2.1.3.

Generally, the screw plate test in sands can be considered a fully drained test, hence the tests conducted during this study were done at a rate of 0.1 Hz. Cyclic stress amplitudes as well as maximum stress levels were varied periodically. The effect of these variations is discussed later.

The general test procedure adopted during this study is summarized below.

(1) Plate Installation - the plate was rotated into the soil under its own impetus, in other words it was allowed to "pull itself" downward. Efforts to advance it at a rate equal to the pitch times the rate of revolution resulted only in preloads on the plate which were detected during installation and the first load increment. The rate of revolution was approximately 10 revolutions per minute. The torque required during installation was generally not affected by the rate of

revolution.

(2) Load Application During Tests - loads were applied using a load cell which provided an analogue signal to a servo controller. Cyclic loads were applied at 0.1 Hz, generally to the maximum capacity of the testing vehicle. Higher frequency loading was found difficult to achieve given the limitation of the hydraulic system. Deformations were recorded using a displacement transducer.

Loads were generally applied until failure of the soil was observed. The failure stresses observed in this study during tests in cohesive and noncohesive soils are summarized in Table 3.1 below.

TABLE 3.1. Observed Plate Stresses at Failure.

	STRESS AT FAILURE (bars)
Silty Clay	4-6
Sensitive Clay	1-2.5
Fine Sand, Loose	8-14
Medium Sand, Dense	greater than 14

As mentioned previously, failure loads were not attained during plate load tests in dense sands.

(3) Repetition of Load Sequence - upon completion of the load test, the plate would be advanced one to two metres, and the test repeated. It was considered that a one metre (or 4 plate diameters) difference between test depths was sufficient to eliminate significant superposition of strain on successive tests.

(4) Upon completion of the profile, the plate rotation was reversed and the entire down hole apparatus retrieved. At this time, it was observed that invariably, the disturbed zone through which the plate passed was unable to support the weight of the plate plus rods. This seemed to confirm the assumption that rod friction was insignificant since it was less than the total weight of the rods. This observation was also made by Schmertmann (1970).

(5) The development of the aforementioned test equipment and procedures resulted in a system which proved to be a rapid investigative tool. A 15 metre profile, including torque measurements and cycled tests at 1 metre intervals could be completed in 8-10 hours; thereby providing valuable deformation parameters over the depth of influence beneath most conventional shallow foundations.

3.4.2. Undrained Tests in Clay

A similar test procedure was employed in clays to determine undrained shear strengths and moduli. Drained tests were not attempted because of the length of time it normally took to achieve 90% consolidation. Researchers studying consolidation parameters in clays generally conduct slow incremental load tests (Janbu and Senneset (1973), Kay and Parry (1982)).

3.4.3. Factors Affecting Test Procedure and Results

The contribution of rod weight to the initial plate load depends upon the amount of friction along the rods. When it was discovered that the rods had to be clamped at the surface during withdrawal of the plate, it was conservatively assumed that the friction on the rods was negligible, hence all load calculations included rod weight. The installation system reported by Janbu and Senneset (1973) utilizes a down-hole hydraulic piston to apply the load to the plate, thereby eliminating any possible effects on friction on the rods. A restriction of their system is that it precludes possible instrumentation of the plate through inner cables, and further complicates the installation procedure. Consequently, a simpler system using only exterior rods was adopted in this study. Further refinement of the test might involve the use of inner rods to apply the load to the plate, in a fashion similar to the mechanical friction cone. This should be considered if rod friction becomes excessive.

The effect of rod compression on measured plate deformation was also evaluated. Calculations showed that the screw plate rods would be compressed by .1 mm per metre at the full capacity of the loading system. At a depth of 10 metres, rod compression would typically contribute only 5% of the observed plate deformation at the maximum load, hence rod compression effects were neglected during modulus determinations. The effect of rod buckling could not be quantified. The use of inner rods on down-hole hydraulic pistons could reduce the possible influence on measured displacements.

The stiffness of the plate can also has a significant effect on deformations measured in stiff soils, as discussed in 3.1.2.

Several times during the installation of the plate in dense sands, cyclic torsional loads were required, in which the plate rotation repeatedly was reversed during installation. This was found to be the only way in which the plate could be advanced through the denser sands. This cyclic torsional loading undoubtedly pre-shears the soil immediately below the plate, and hence would tend to reduce the stiffness measured during the initial load cycle.

4. DESCRIPTION OF FIELD PROGRAMMES

The field testing programmes were conducted on three research sites shown on Figure 4.1. The generalized site geology is given in Table 4.1 below.

SITE	SOIL DESCRIPTION
McDonald Site - Sea Island	Sand and clayey Silt
Langley Clay Site	Sensitive Clay
Cloverdale Clay Site	Sensitive Clay

TABLE 4.1. General Lithology at Research Sites.

4.1. McDonald - Sand and Silty Clay Site

4.1.1. General Geology and Site Description

The McDonald site is situated on sea island, and is approximately 2 metres above sea level. The deposits represent various stages of development of the Fraser Delta. The general soil profile consists of:

0–2 m - clay; silty, soft, organic

2–13 m - sand; medium to coarse, variable density, concretionary layers at depth

13–15 m - fine sand; transition zone

15–+300 - silt; clayey, normally consolidated, soft

A typical cone profile is shown in Figure 4.2.

4.1.2. Description of Test Programme

A detailed site investigation has been carried out at the site as part of an on-going research effort. Details on equipment and

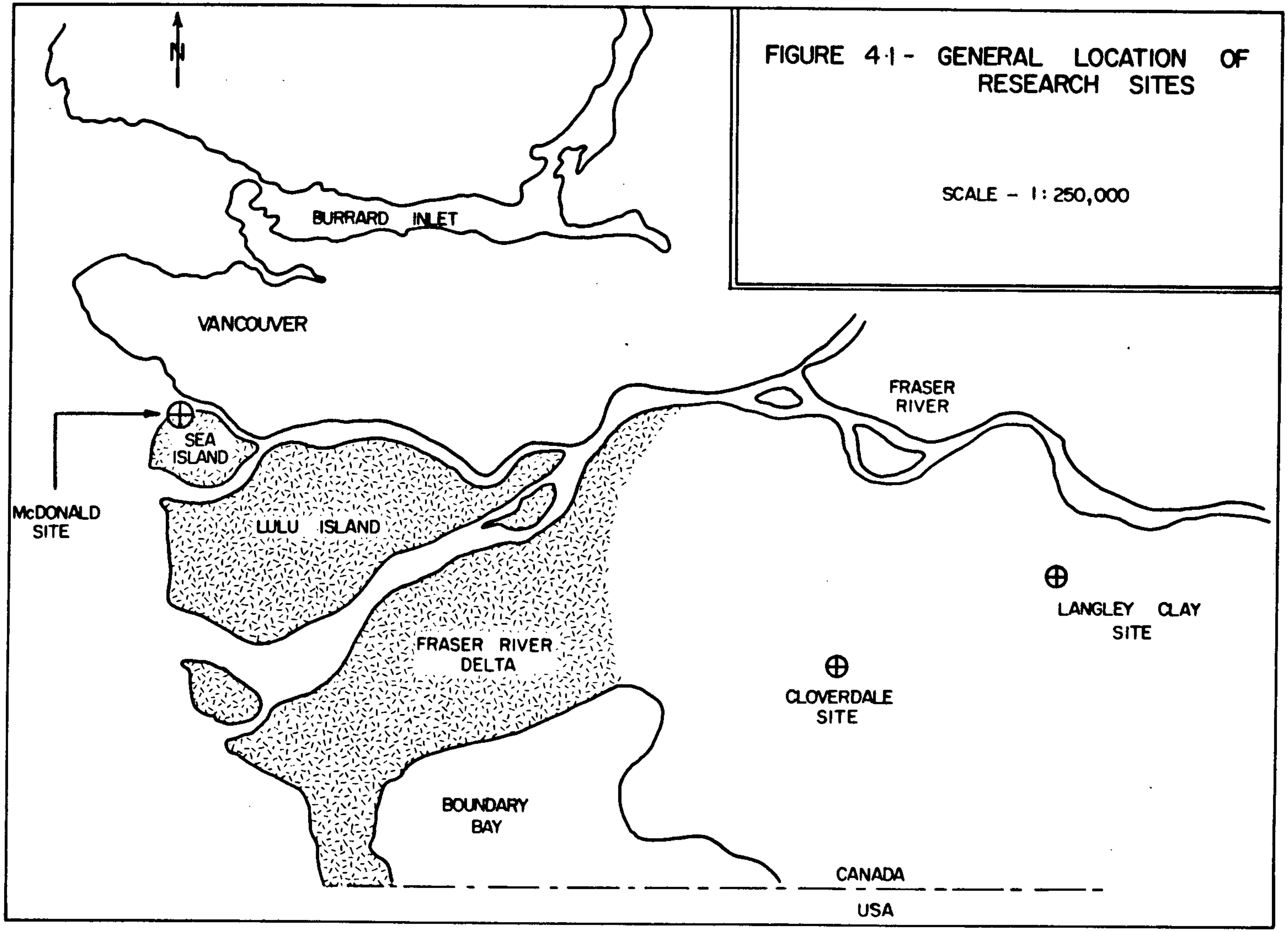


FIGURE 4·1 - GENERAL LOCATION OF RESEARCH SITES

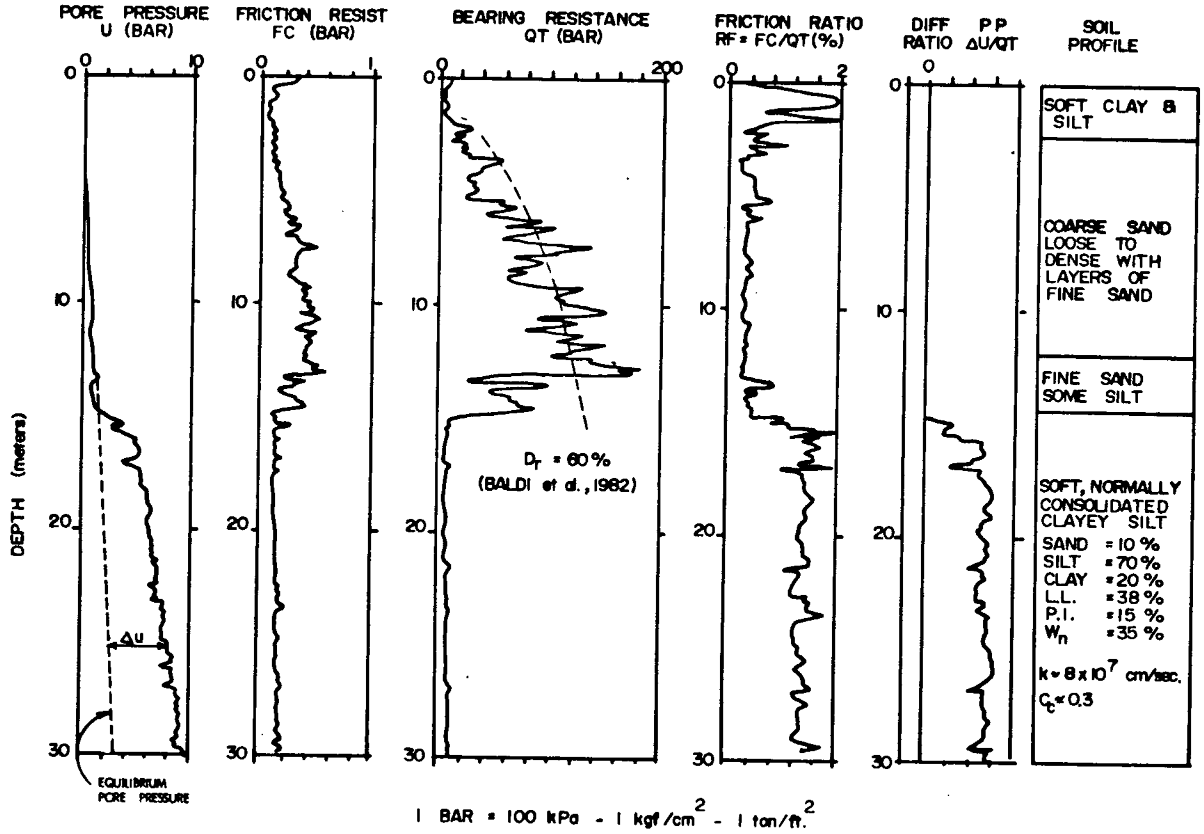


FIGURE 4·2 - TYPICAL CONE PROFILE AT McDONALD SITE
(AFTER CAMPANELLA, ROBERTSON AND GILLESPIE, 1983)

procedures are given in Robertson (1982). The tests under consideration in this study are summarized in Table 4.2 below.

	TEST TYPE	NO.
FIELD	Cone penetration profiles	6
	Standard penetration tests	13
	Self boring pressuremeter profile	3
	Dilatometer profiles	2
	push-in cone pressuremeter profile	2
	Screw plate profiles	12
LAB	cyclic triaxial tests	7
	triaxial compression tests	5

TABLE 4.2. Summary of Test Programme - McDonald Site.

A location plan is given in Figure 4.3.

4.1.3. Test Results - Drained Behaviour

4.1.3.1. Constrained Moduli in Sands

A typical screw plate load displacement curve is presented in Figure 4.4. These curves were analyzed using Janbu and Senneset's (1973) method of analysis, using the initial tangent modulus. The modulus numbers k_m within the McDonald sands are presented in Figure 4.5 and range from

$$k_m = 120 \text{ to } 550 .$$

The variation in k_m corresponds quite closely with the variation in cone bearing values at the site (Figure 4.6). Both profiles indicate peaks at approximately 9 to 10 metres, where the sand density appears greatest. The presence of the dense layer at 12 to 13 metres is not reflected in the screw plate tests because of the presence of the softer layer at depth.

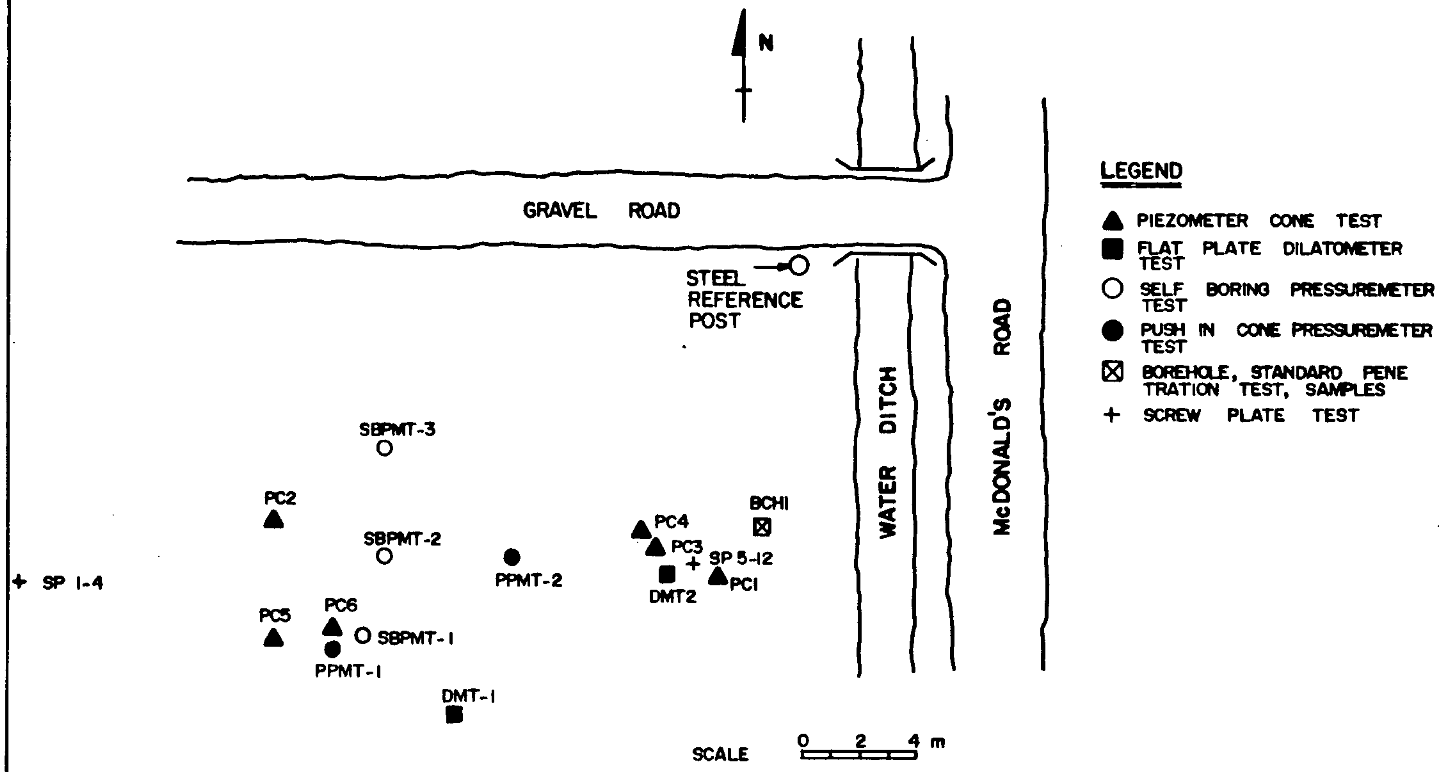


FIGURE 4·3 - LOCATION PLAN AT MACDONALD FARM SAND AND SILTY CLAY SITE

after Robertson (1982)

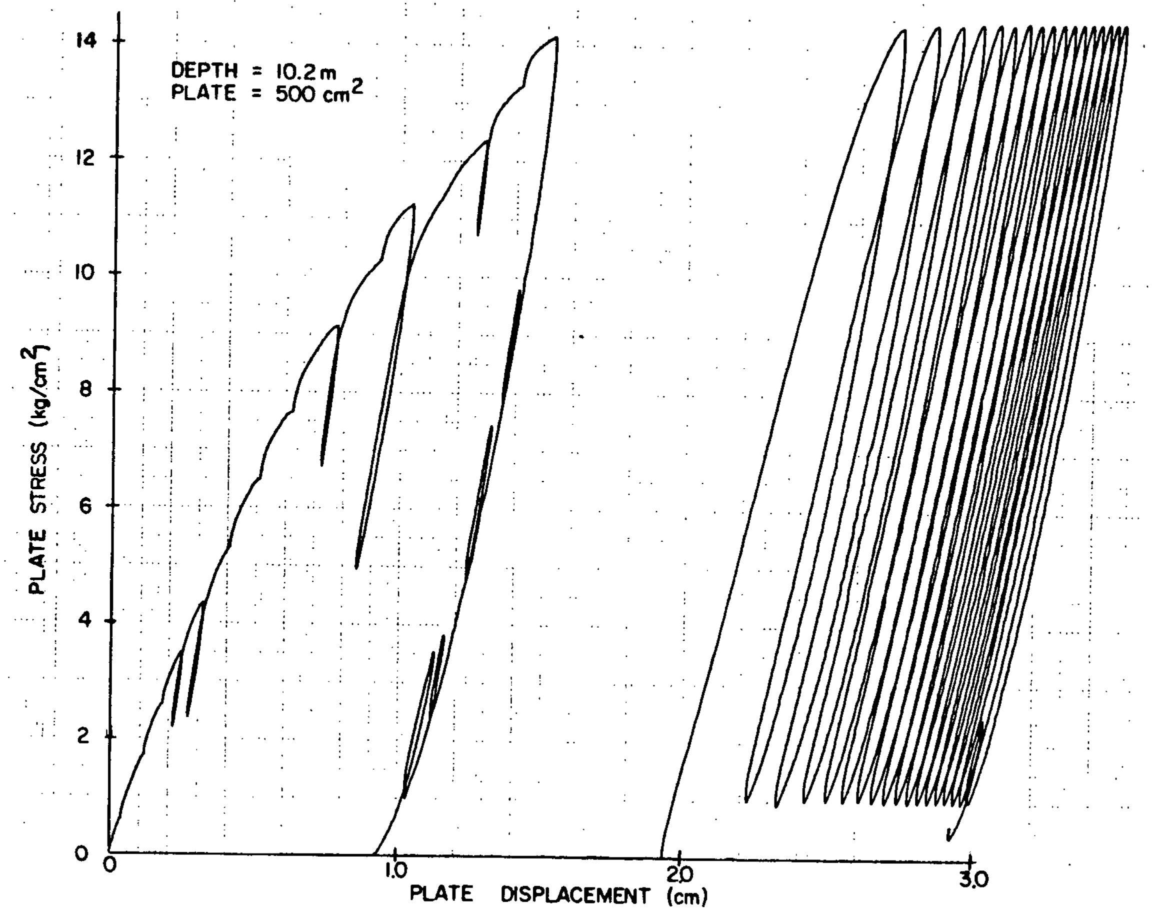


FIGURE 4.4 - TYPICAL LOAD DISPLACEMENT CURVE IN SAND

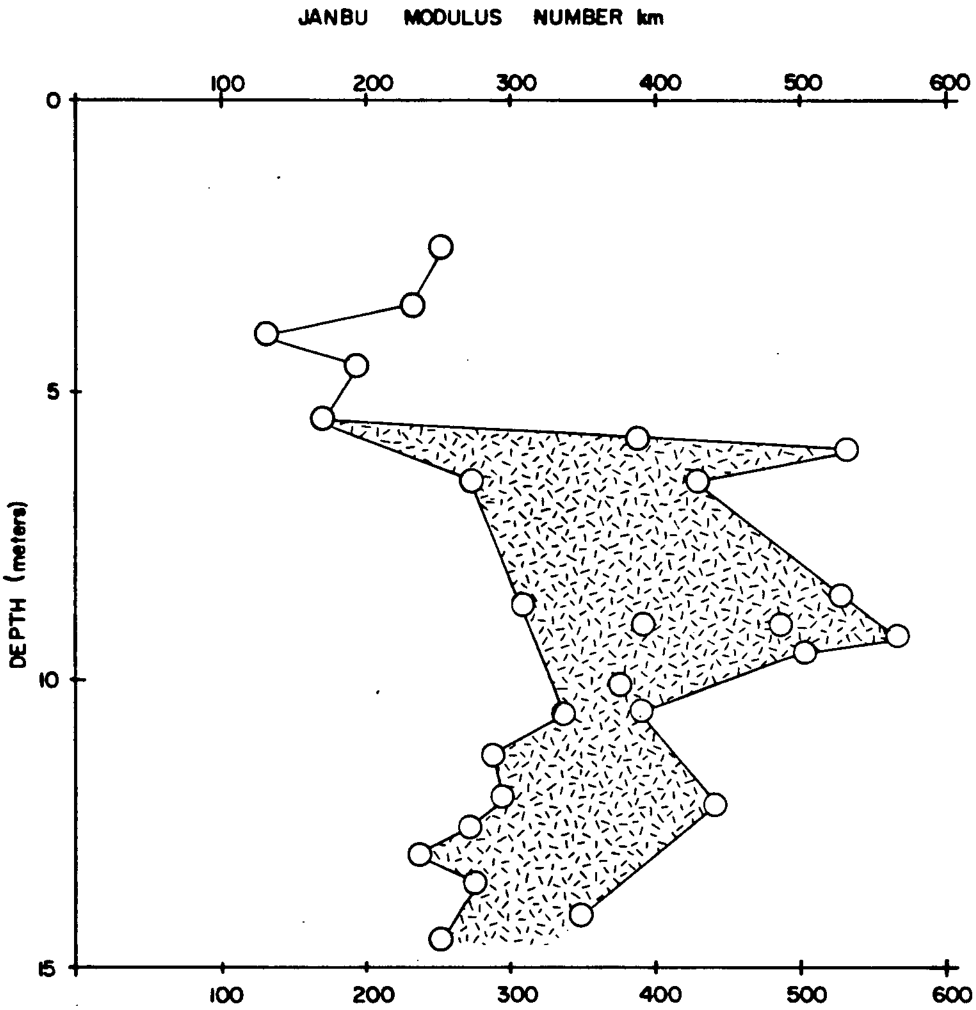


FIGURE 4.5 - RANGE OF JANBU MODULUS NUMBER km AT MACDONALD SITE

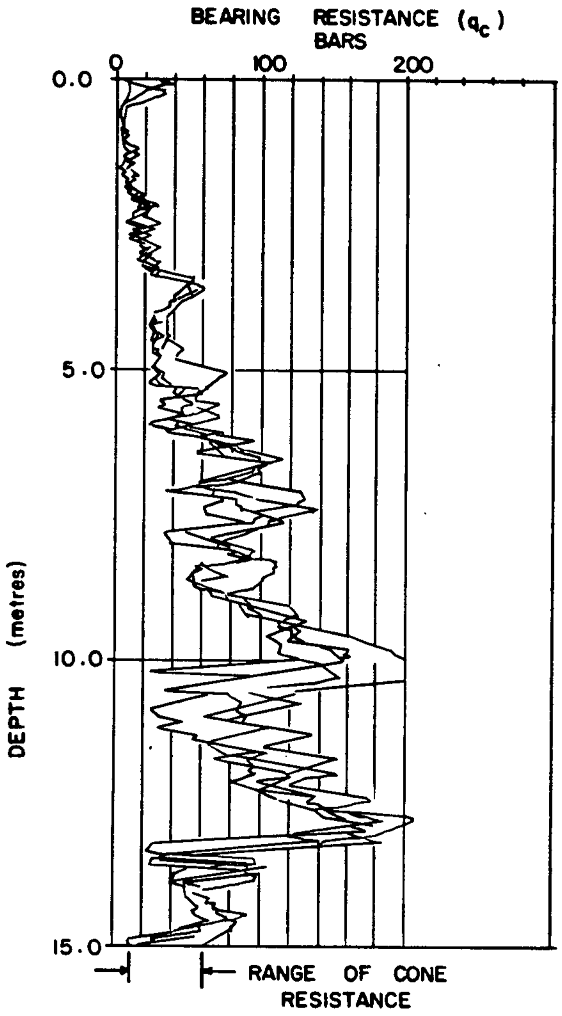


FIGURE 4.6 - RANGE OF CONE RESISTANCE AT MCDONALD'S FARM

after Robertson (1983)

This variation can be attributed to variations in lithology, occasional cementing within the beach-deposited sands (concretions), and localized high K_o values arising out of the deposition of the sand in a high energy environment.

4.1.3.2. Comparison with Laboratory and Pressuremeter Moduli

The screw plate k_m values are compared to laboratory $(k_E)_{lab}$ and pressuremeter $(k_E)_{pmt}$ values as reported by Robertson (1982) in Figure 4.7. The use of the nondimensional modulus number "k" eliminates the effect of stress variability. The laboratory k_E values were obtained from the initial tangent portion of the triaxial tests performed on 'undisturbed' samples. It is recognized that the measurement of deformation behaviour at low levels of stress and strain is difficult unless specialized procedures are used. The pressuremeter k_E was obtained from unload-reload tests performed at the site. When screw plate k_m values are corrected for plate rigidity, using equation 3.5, they compare favourably with laboratory and pressurementer values as shown in Fig. 4.8. It is recognized that the various moduli numbers represent different loading mechanisms and stress paths. However, for the purposes of discussion, the k_m values will initially be compared to the average k_E value. A differentiation between the various moduli numbers will be made after further discussion.

The apparent relationship between the various modulus numbers can be summarized as:

k_m = (.9 to 1.2) $(k_E)_{avg}$ in fine sand

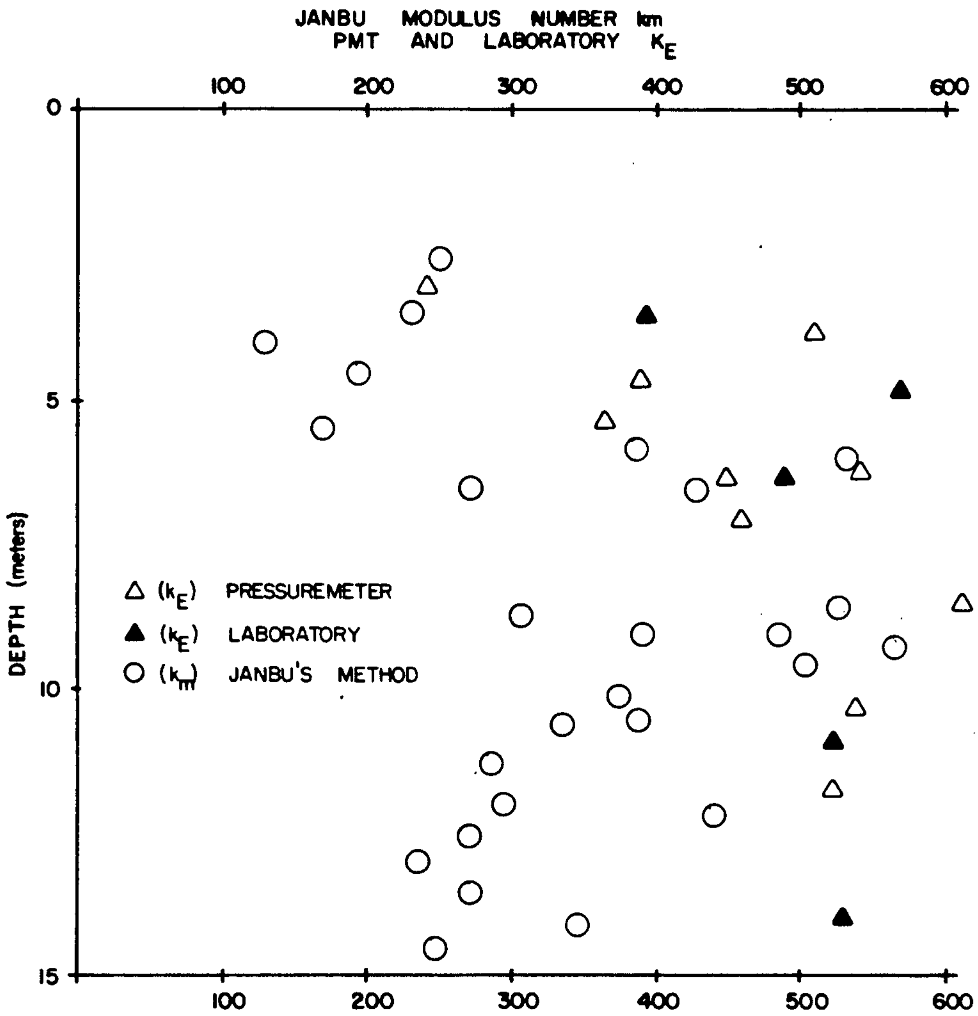


FIGURE 4.7 - COMPARISON OF SCREW PLATE, PRESSUREMETER AND LABORATORY MODULUS NUMBERS

$$k_m = (.75 \text{ to } 1.0)\,(k_E)_{avg} \text{ in medium dense sand.} \qquad (4.1)$$

Intuitively, one would expect that $(k_E)_{pmt}$ would be lower than k_m, unless the in-situ K_o value is high, because the pressuremeter measures a horizontal modulus, whereas the screw plate measures a vertical constrained modulus. Ladd et al. (1977) observed that the vertical modulus can be approximately twice the horizontal.

Similarly, if we examine the relationship between the constrained modulus and vertical elastic modulus, using:

$$M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \qquad (4.2)$$

in which: M = constrained modulus

E = elastic Young's modulus

ν = Poisson's ratio,

we would expect that $k_m \simeq (1.5 \text{ to } 2.1)\,(k_E)_{lab}$, provided that the laboratory k_E was obtained from a truly undisturbed sample. It should be recognized that the effects of sample disturbance generally result in an underestimation of the modulus determined from laboratory tests, consequently in practise one would expect that the in-situ k_m values would be even greater than twice the laboratory k_E.

The discrepancy in the modulus numbers observed in Figure 4.8 may arise out of a violation of the assumptions in Janbu's analysis. Factors which would reduce the measured stiffness of the soil, as discussed previously, include soil disturbance during installation,

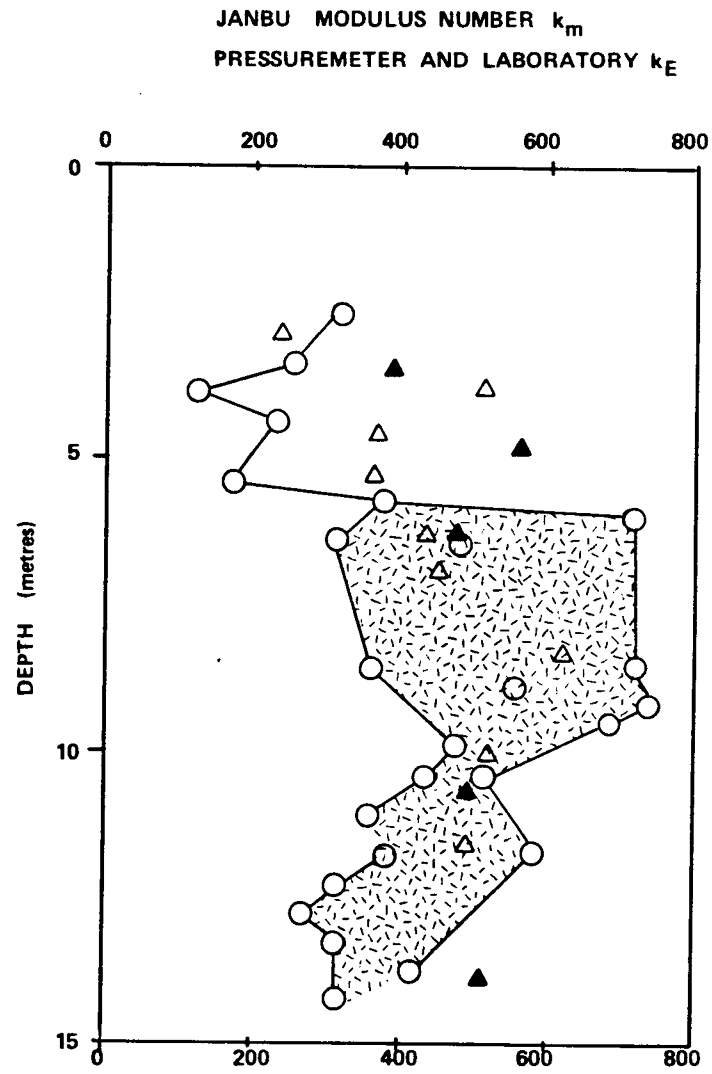


FIGURE 4.8 - CORRECTED k_m VS $(k_E)_{LAB}$ AND $(k_E)_{PRESSUREMETER}$

and lateral strains during loading which result in greater measured vertical displacements. These factors cannot be quantified without model studies or parametric finite element studies which would indicate the variation in modulus which is expected with various degrees of soil disturbance.

Robertson (1982) observed that the pressuremeter unload-reload modulus is approximately equal to the in-situ horizontal modulus. If we extend this observation to the screw plate data and assume that $(E)_{vertical} = 2(E)_{horizontal}$, then we would expect that:

$$k_m = 1.5 \text{ to } 2.1\ (k_E)_{vertical} \tag{4.3}$$

and

$$k_m = 3.0 \text{ to } 4.2\ (k_E)_{pmt}\ . \tag{4.4}$$

In order to obtain a more realistic vertical constrained modulus from Janbu's analysis, it is necessary to adjust the modulus number accordingly:

$$\frac{(k_m)_{true}}{(k_m)_{Janbu}} = (2.7 \text{ to } 3.6) \text{ in fine sand}$$

$$= (2.3 \text{ to } 3.0) \text{ in medium dense sand.} \tag{4.5}$$

The relative rigidity of the UBC plate was evaluated using equation 3.2, and it was concluded that the U.B.C. plate was very flexible. Consequently, the use of this plate leads to an underestimation of modulus in dense sands. To obtain a corrected modulus, E should be multiplied by $\beta = 1.5$ when a flexible plate is used. Based on this analysis, it can also be shown that an optimal (stiffer) plate design can be achieved by:

(1) increasing the plate thickness,

(2) decreasing the plate diameter, and

(3) increasing the diameter of the rods in contact with the plate.

These would be primary considerations in the optimization of plate design.

4.1.3.3. Modulus Determination from Cyclic Loads

Several researchers have indicated that elastic moduli can be determined from the unload-reload portion of in-situ tests. Hughes (1982), for example, presents a theoretical basis for the determination of shear moduli from reload tests using the self boring pressuremeter.

A number of cyclic screw plate load tests were performed in the sands at the site to obtain a preliminary assessment of the suitability of the screwplate test to determine a drained elastic modulus in a similar fashion. Figure 4.9 presents an example of the variation in a rebound modulus, E_R, with stress amplitude at a constant upper limit. E_R ranges from:

$$E_R = \{.73 \text{ to } 1.1\}\, E_i, \tag{4.6}$$

E_i = initial elastic modulus.

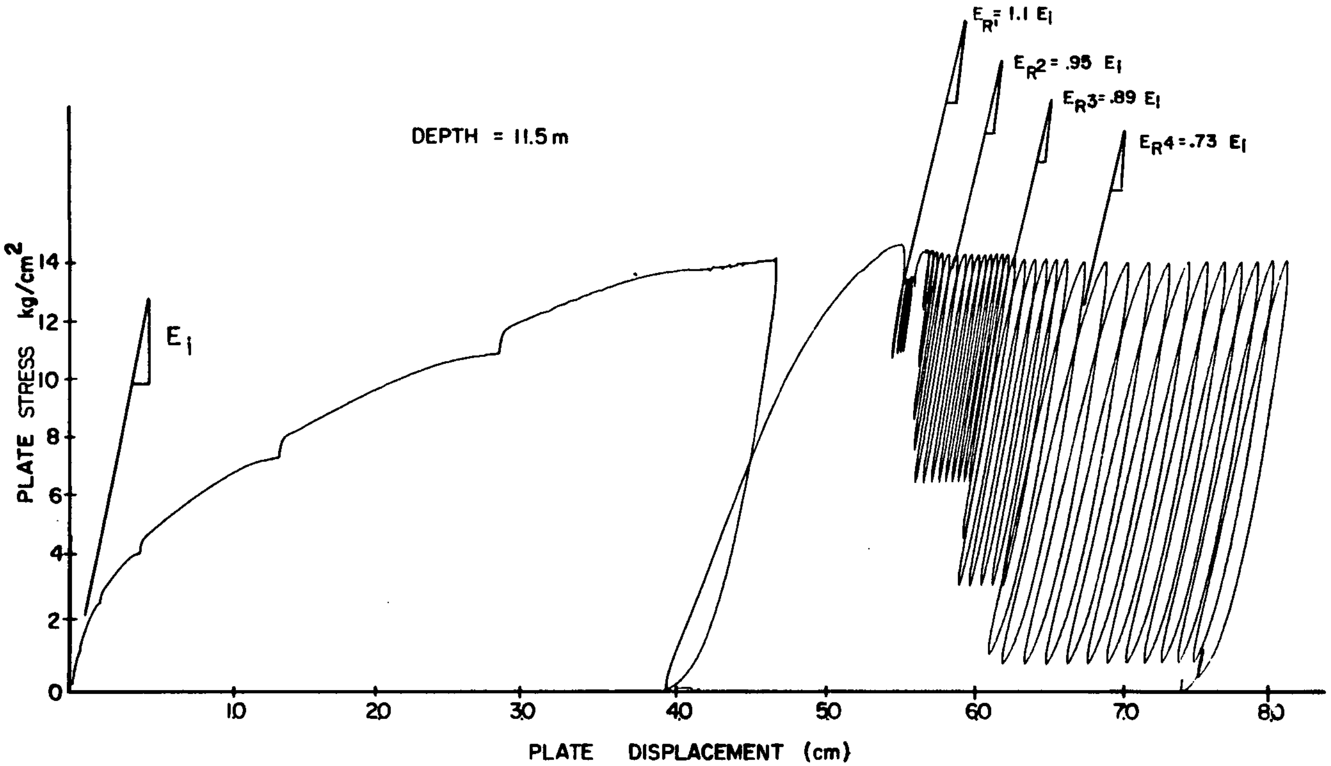


FIGURE 4.9 - VARIATION IN E_R WITH STRESS AMPLITUDE AT A CONSTANT MAXIMUM STRESS LEVEL

In this instance, E_i and E_R represent the slope of the stress displacement curve.

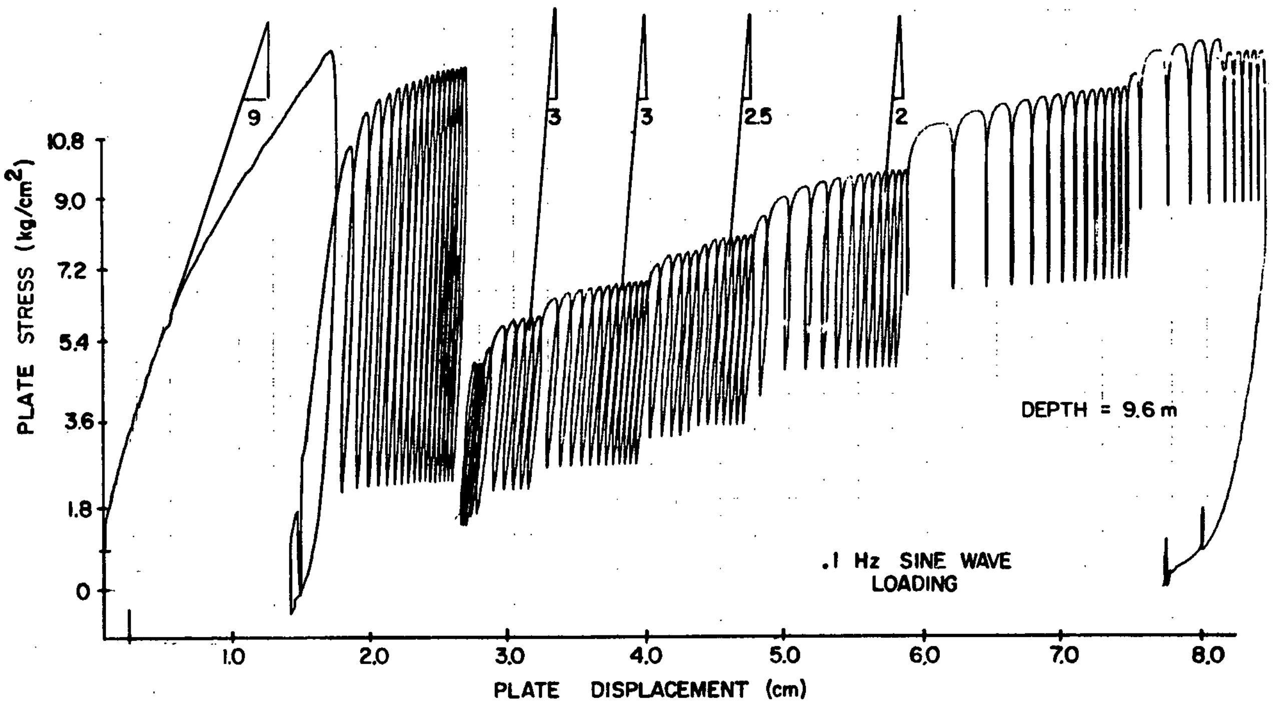
Figure 4.10, on the other hand, shows how the modulus also varies with maximum attained load for constant stress amplitudes.

This phenomena is summarized in Figure 4.11, where a considerable variation in rebound modulus is shown. An optimum level of plate load was found to be approximately 60% of the ultimate plate load. In practise, this plate load can be estimated from the cone resistance profile.

Considerable evidence exists of variations in screw plate rebound modulus, much more so that that observed in pressuremeter and laboratory tests. Perhaps the most important consideration is the boundary stress condition, including the load acting on the back of the plate during unloading. The resistance of the soil column above the plate would lead to less elastic rebound; hence an increase in the apparent rigidity of the soil. This increase would be dependent upon installation procedure as well as soil parameters, and has not been accounted for through closed-form solutions.

Similarly, E_R varies considerably with stress level at constant stress amplitudes, as was shown in Figure 4.10, and can be up to twice the initial value. This behavior is consistant with the observation of Makhlouf and Stewart (1965), which is shown in Figure 4.12. They found that sands were stiffer with increasing stress level, and with decreasing strain amplitude. Consequently, the determination of an "elastic" modulus is very much dependent upon the stress level and amplitude during the unload-reload portion of

FIGURE 4.10 EXAMPLE OF THE VARIATION IN E_R WITH STRESS LEVEL AT A CONSTANT STRESS AMPLITUDE



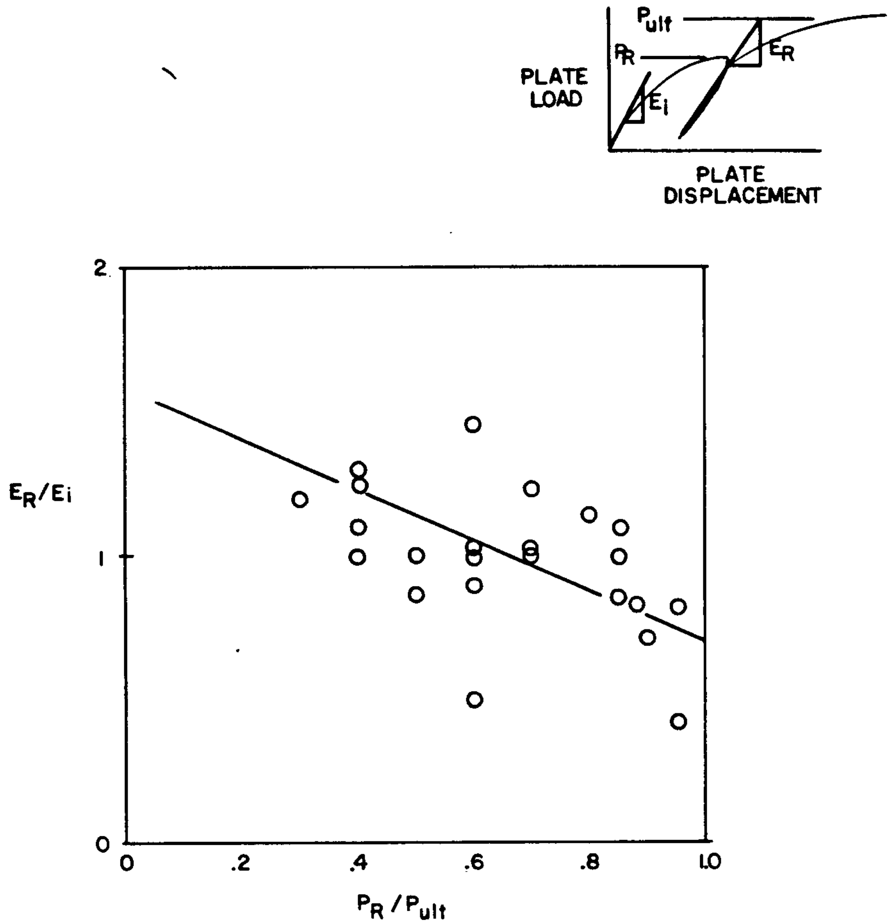
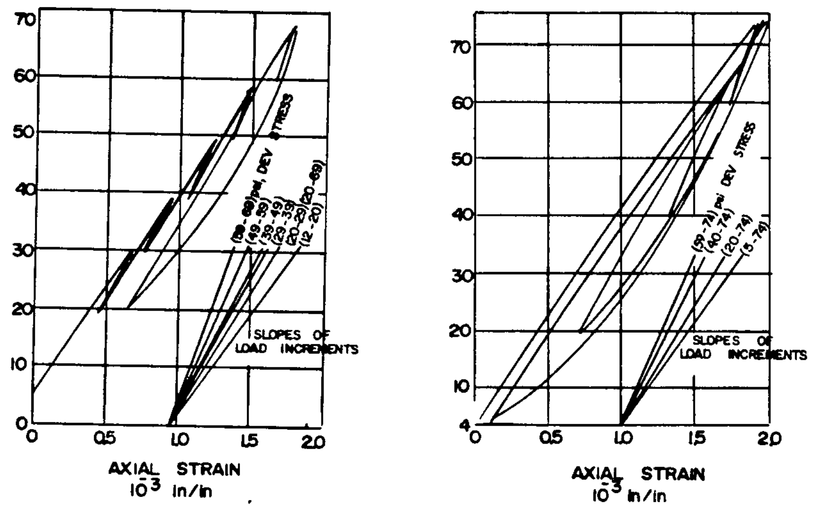


FIGURE 4·11 - RELATIONSHIP BETWEEN REBOUND MODULUS AND INITIAL MODULUS AT VARIOUS LEVELS OF PLATE LOAD

(data from all tests)



INFLUENCE UPON E OF THE LOWER LIMIT OF A CONSTANT RANGE OF DEVIATOR STRESS

INFLUENCE UPON E OF THE RANGE OF DEVIATOR STRESS WITH A CONSTANT UPPER LIMIT

FIGURE 4.12 INFLUENCE OF STRESS RANGE AND LEVEL ON LABORATORY MODULUS DETERMINATION

(AFTER MAKHLOUF & STEWART, 1965)

test. Consistent modulus determinations cannot be obtained by selecting arbitrary stress levels or amplitudes, hence a standard procedure should be adopted.

The Janbu analysis takes into consideration the effect of stress level with the modulus factor, k_m . As a result, it represents one of the better methods of determining an in-situ modulus, particularly at low stress levels. The use of an initial tangent to the loading curve reduces the possible errors inherent in methods which use the unload-reload curves.

4.1.3.4 Young's Moduli in Sands

The load-displacement curves were also analyzed using Schmertmann's (1970) method, and equation 2.8. Young's moduli were obtained by assuming a secant modulus at 2 bars ($\simeq$ 2 tsf) as suggested by Schmertmann, and neglecting the effects of compression due to creep in sands during a test of short duration. The results are presented in Figure 4.13.

The values obtained during this study exhibit a scatter similar to that found in the literature (Dahlberg (1975) and Schmertmann (1970)). Young's modulus, E_s, is found to vary by a factor of 2-3 at the site, again possibly due to the variation in the lithology at the site.

Perhaps the most significant observation here is the tendency for the McDonald site data to increasingly underestimate E_s in high q_c (stiff) sands. Returning again to the effect of plate stiffness discussed in Section 3.1, the appropriate correction factor has again been applied, whereby

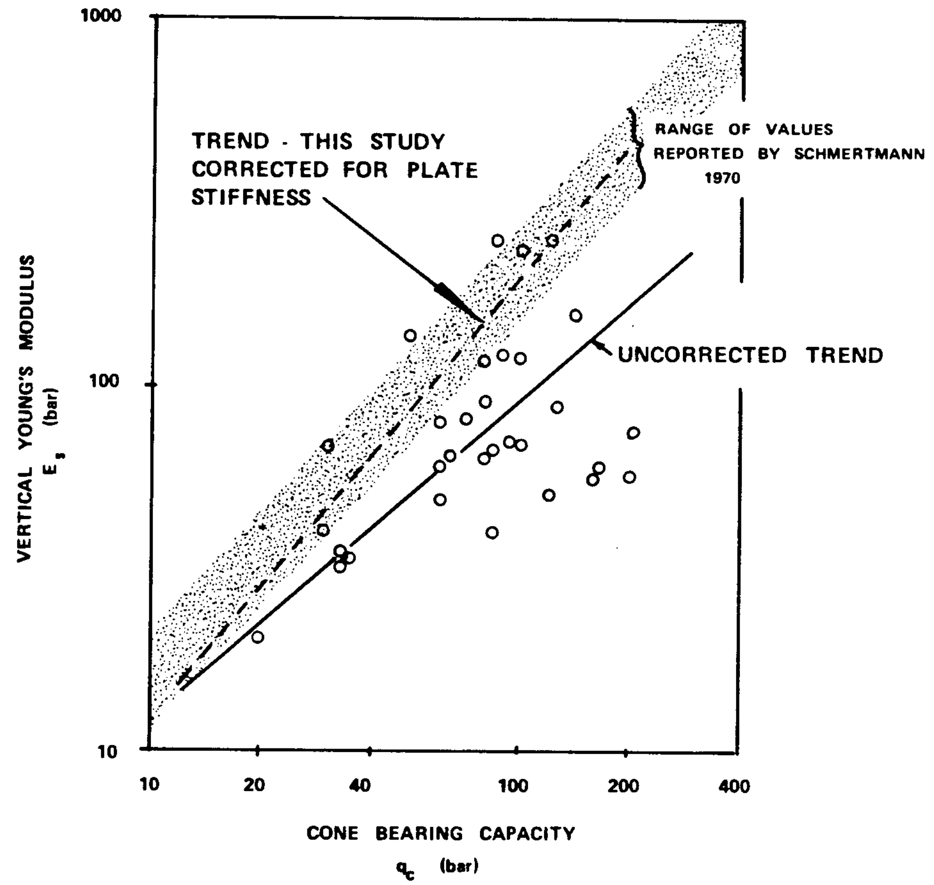


FIGURE 4·13 - OBSERVED RELATIONSHIP BETWEEN CONE BEARING q_c AND VERTICAL YOUNG'S MODULUS, E_s (SCHMERTMANN'S METHOD)

MCDONALD SITE

$$(E_s)_{corr} = E_s \times \beta \qquad (4.7)$$

where: $(E_s)_{corr}$ = Young's Modulus corrected for plate stiffness, and

E_s = uncorrected modulus, and

β = correction factor for plate stiffness,

= 1.5 for flexible plates

1 for rigid plates.

Application of this correction factor to the trend obtained in this study brings it well within the range reported in the literature, and highlights the importance of plate rigidity during the evaluation of test data.

4.1.3.5. Cyclic Loading for Liquefaction Assessment

Hughes et al. (1980) propose that repeated cyclic load tests can be performed with the pressuremeter to assess the liquefaction resistance. This assessment can be made through an estimate of the cyclic stress ratio to cause liquefaction (Robertson 1982). The constant stress amplitude portion of the load curve of Figure 4.10 has been reduced, and the results shown in Figure 4.14. Here the potential for using the screw plate test in a similar manner can be seen. Plots of cumulative or incremental strain can be used in a manner similar to that suggested for the pressuremeter. The maximum plate load reached during each cycle obviously affects the cumulative strain to a large extent, however standardization of test

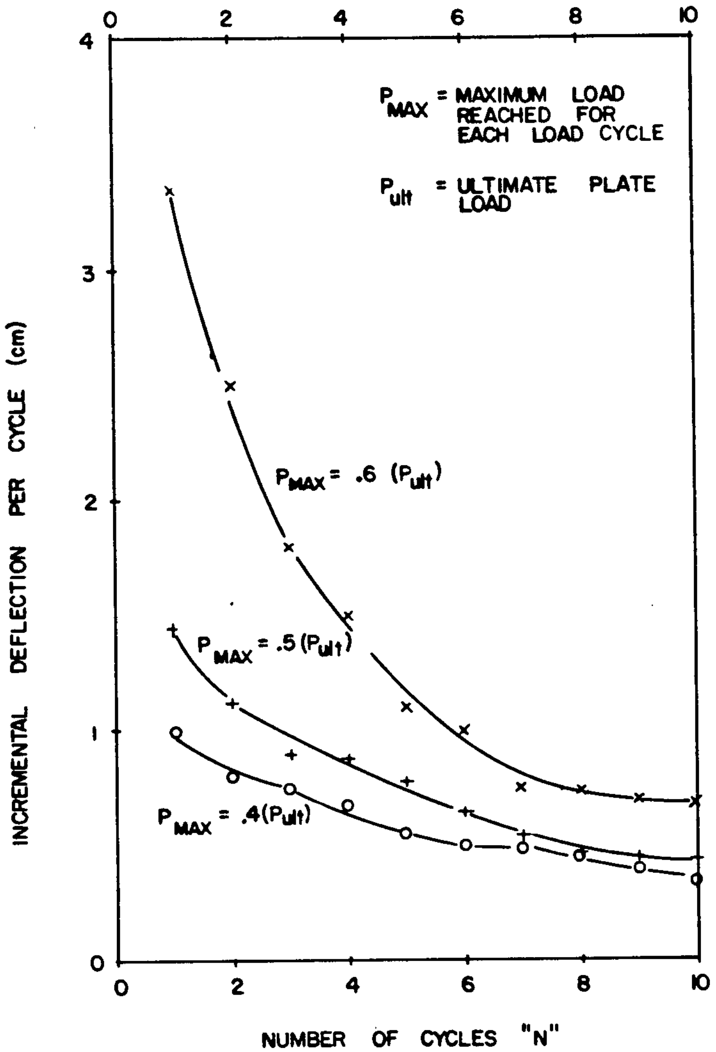


FIGURE 4.14 - INCREMENTAL PLASTIC DEFLECTION PER CYCLE

procedures could permit similar comparison to be made between the response in loose and dense sands.

It should be noted here that several cyclic tests were also done to compare results from various waveforms. No appreciable change was observed with either triangular, square or sinusoidal loading. Sinusoidal loading was generally adopted for the cyclic tests because it placed less strain on the hydraulic system.

4.1.4. Test Results - Undrained Behavior

4.1.4.1. Undrained Shear Strength

Undrained shear strengths were obtained using a variety of correlations with the screw plate, cone, pressuremeter and dilatometer data. Some disagreement between the various shear-strength determinations is to be expected, due to inherent differences in the orientation of failure planes, induced disturbance and strain rate. Nonetheless, comparisons are presented herein.

A summary of the undrained shear strength profile is presented in Figure 4.15. The strength values agree reasonably well, within a range of 10 kPa of the median value. An N_K value of 9 for the screw plate data appears to give good agreement with other in-situ strength determination. The cone factor $N_c = 17$ was selected using a rigidity index of $G/c_u = 300$, and the correlation developed by Baligh (1975).

There is some evidence of scatter in the data, which is briefly discussed in the next section. It is also important to note that the tests done in the silty clay at depth were done while the author

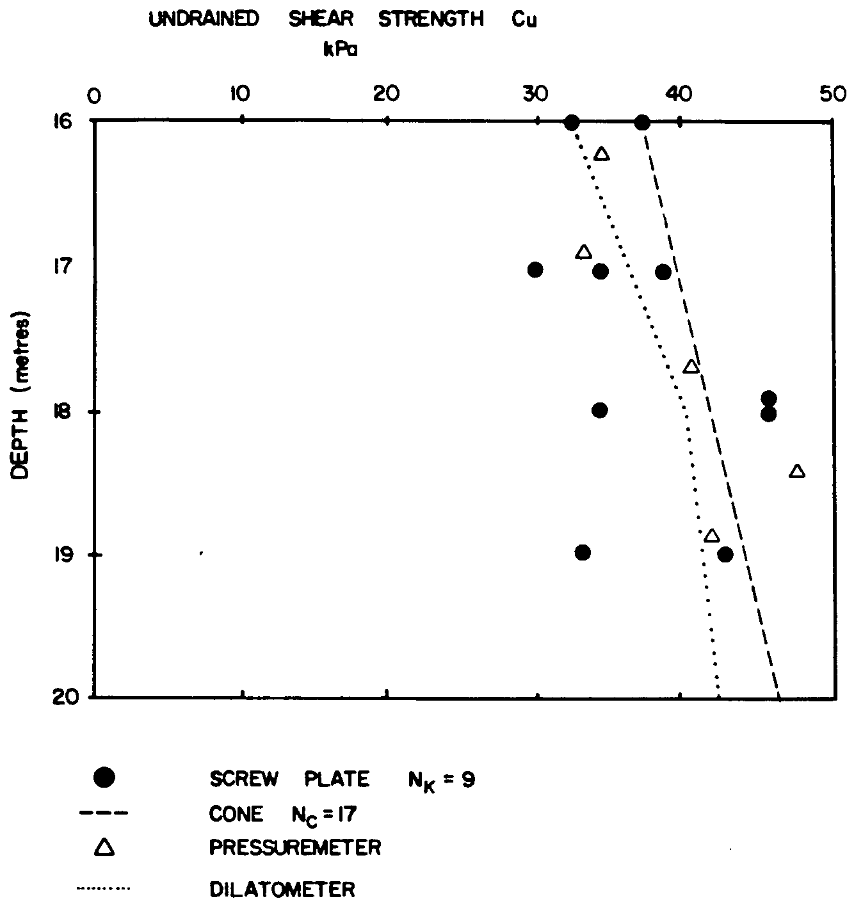


FIGURE 4·15 - COMPARISON OF UNDRAINED SHEAR STRENGTHS
MCDONALD SILTY-CLAY SITE

was still developing a suitable test procedure; consequently the possible effect of plate rotation during the application of load, which is discussed in a later section, was not specifically addressed. In addition, at this particular site the effect of rod friction within the overlying dense sands could be significant, and result in lower calculated values of c_u. A proper assessment would require field vane tests at depth, as well as modifications to the installation system, including inner rods which would eliminate the effect of rod friction.

4.1.4.2. Repeated Undrained Loading in Silt

During undrained cyclic load tests in silt, a significant reduction in ultimate plate capacity was noted with successive load increments. This behavior is typified in Figure 4.16, where strain softening appears to be occuring. Continued strain also occurs after the load is released, which would indicate that some consolidation may be taking place after the drop in the plate load.

There is little evidence in the other in-situ tests to suggest that this behavior should be expected. The effect of strain-softening on a determination of c_u would vary according to the degree of disturbance associated with each installation. Consequently, unless this phenomena is accounted for, the selection of a suitable N_K or N_c factor is somewhat arbitrary. This factor highlights the importance of carefully controlled installation of the plate , particularly in strain-softening materials.

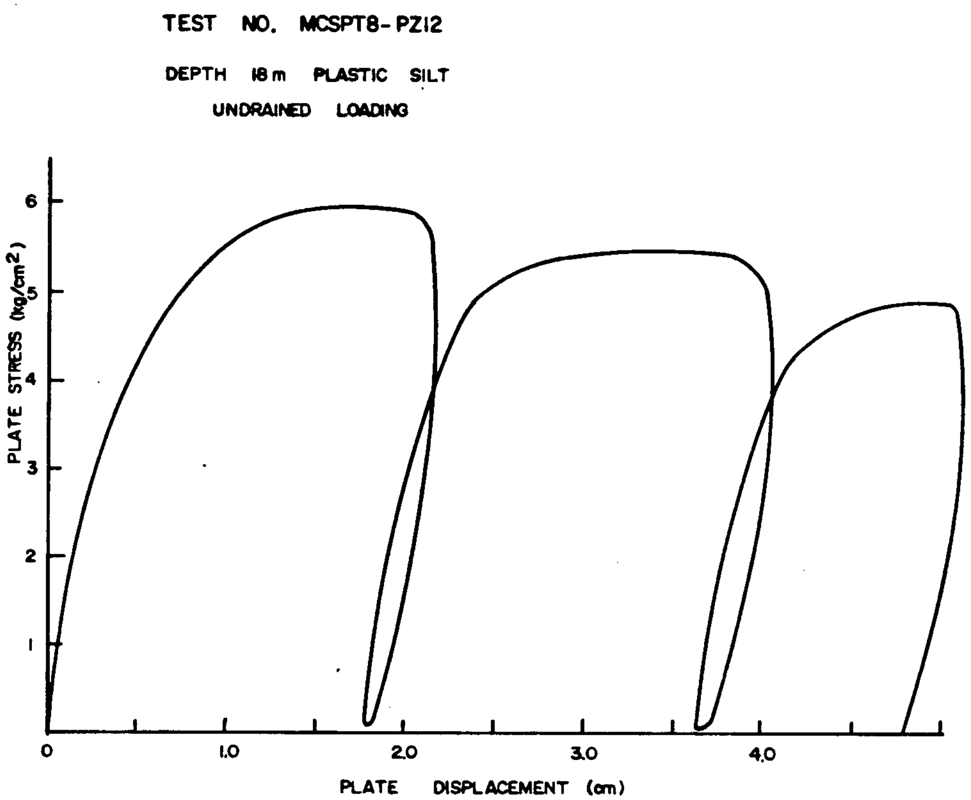


FIGURE 4·16 - REDUCTION OF STRENGTH DURING LOAD TESTS IN SILT AT MACDONALD SITE

4.1.5. Torsional Resistance during Plate Installation

Measurements of installation torque were made during several of the soundings at the site. Torque soundings for a 500 cm^2 plate and a 250 cm^2 plate are presented in Figure 4.17, along with the appropriate cone bearing profile.

The installation torque "T" appears to correlate quite closely with the cone bearing q_c, and varies according to the following approximation:

$$
\begin{aligned}
T/q_c &= 17.6 \quad \text{in silty clay} \\
&= 6.6 \quad \text{in sand}
\end{aligned}
\tag{4.8}
$$

where: T = torque in Nm

q_c = cone bearing in bars.

This correlation can be used to predict the site-specific installation torque required given the cone bearing profile. In addition, the torque record can be used to indicate when pre-determined test depths are reached, and provide a rough log of stratigraphy at the site.

4.2. Langley Sensitive Clay Site

4.2.1. General Geology and Site Description

This site is adjacent to the #1 Highway, and situated at the base of a 5 m cut for the road right of way. The water table is at 2.3 m depth. The general geology consists of Capilano sediments, raised marine and fluvial deposits. The site stratigraphy is:

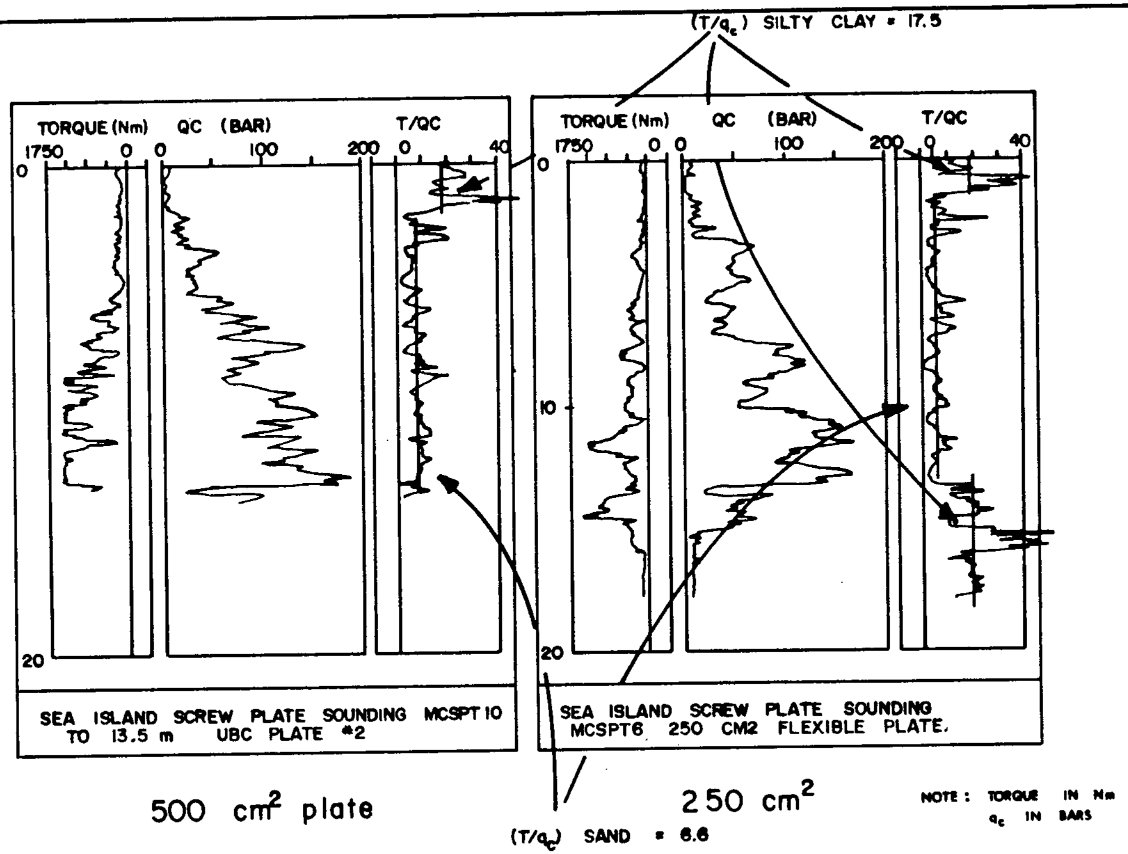


FIGURE 4·17 – RELATIONSHIP BETWEEN INSTALLATION TORQUE AND CONE RESISTANCE AT SAND AND SILTY CLAY SITE

0-2 m - gravel and clay fill

2-10 m - clay, over-consolidated with interbedded silty sand; sensitivity = 10-15 based on $S_t = \frac{15}{R_f}$ (R_f = friction ratio)

10-30 m - clay, slightly over-consolidated to normally consolidated.

A typical cone profile is presented in Figure 4.18.

4.2.2. Description of Test Programme

A detailed site investigation was carried out, and is summarized below:

TEST TYPES	NUMBER
Cone penetrometer	1
Dilatometer	1
Vane	1
Piezometer Cone	1
Cone Pressuremeter	1
Screw Plate	1

TABLE 4.2. Summary of Test Programme - Langley Site.

Location of the probings are shown in Figure 4.19.

4.2.3. Test Results

A typical screw plate load test in sensitive clay is presented in Figure 4.20. The achievement of a peak and residual load is more typical of the response which might be observed through laboratory testing, and has not previously been reported in literature covering the screw plate.

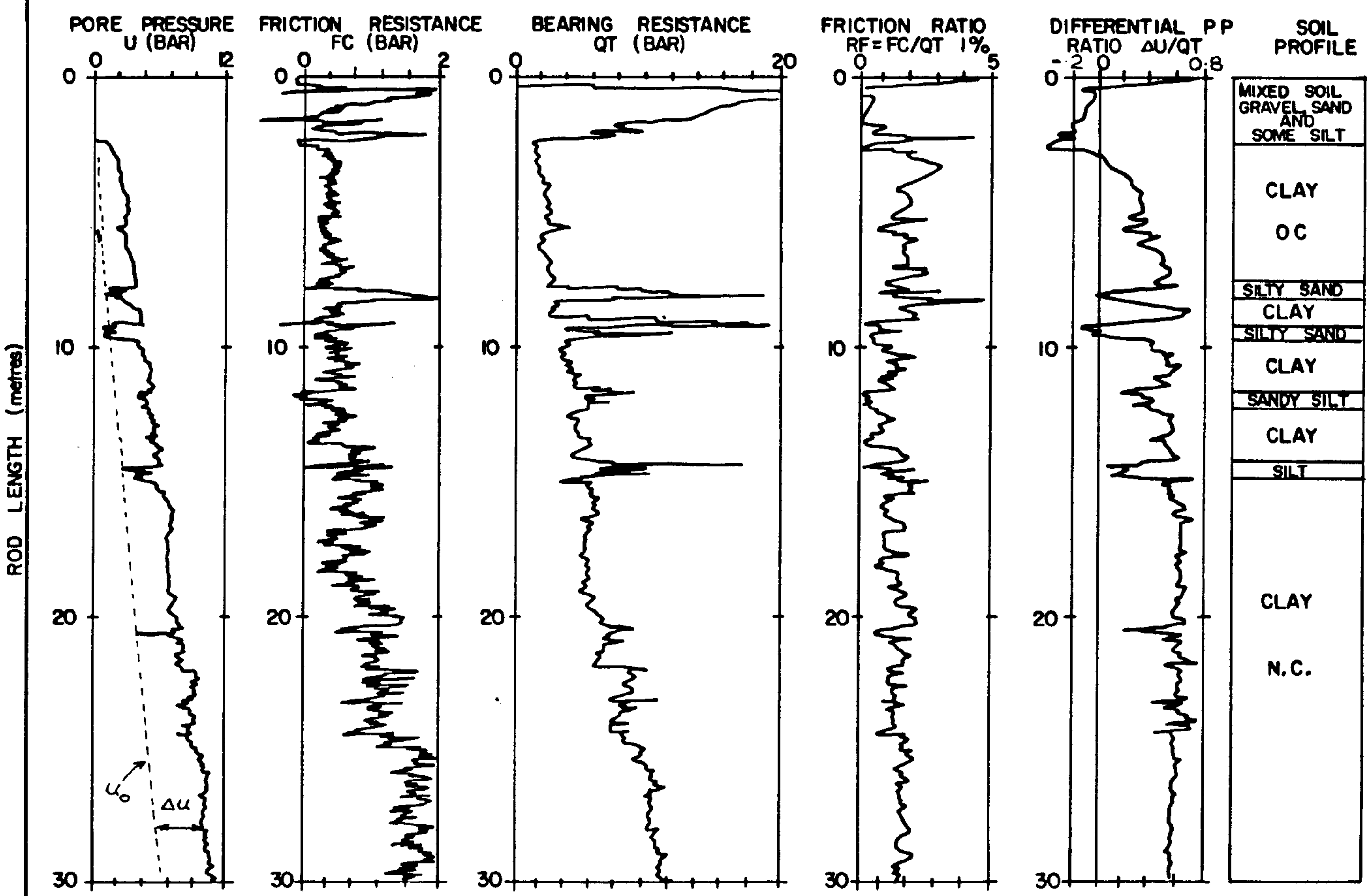
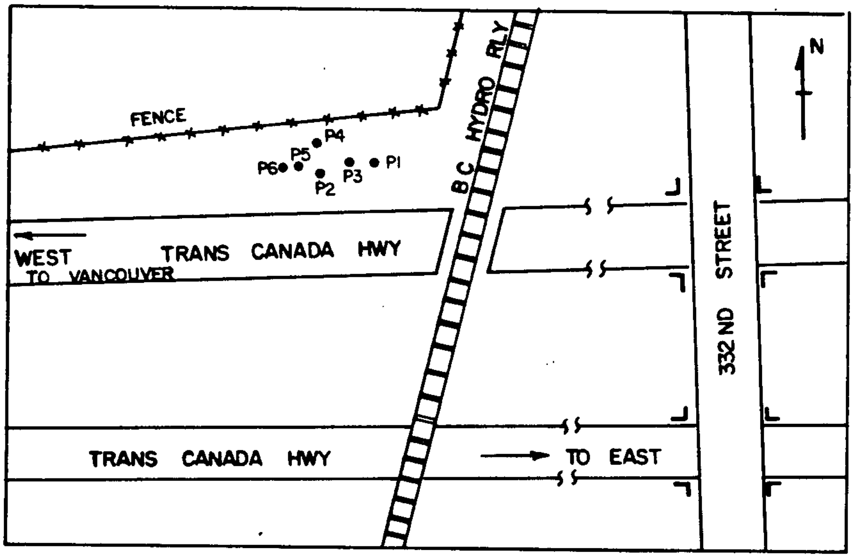


FIGURE 4·18 - CONE PROFILE AT LANGLEY CLAY SITE



NOT TO SCALE

LEGEND

P1 CPT 1 OCT. 14, 1982
P2 DILATOMETER NOV. 4, 1982
P3 VANE TEST NOV. 18, 1982
P4 PORE PRESSURE DISSIPATIONS NOV. 18, 1982
P5 PUSH IN CONE PRESSUREMETER NOV. 25, 1982
P6 SCREW PLATE DEC. 2, 1982

FIGURE 4.19 - LOCATION PLAN AT LANGLEY CLAY SITE

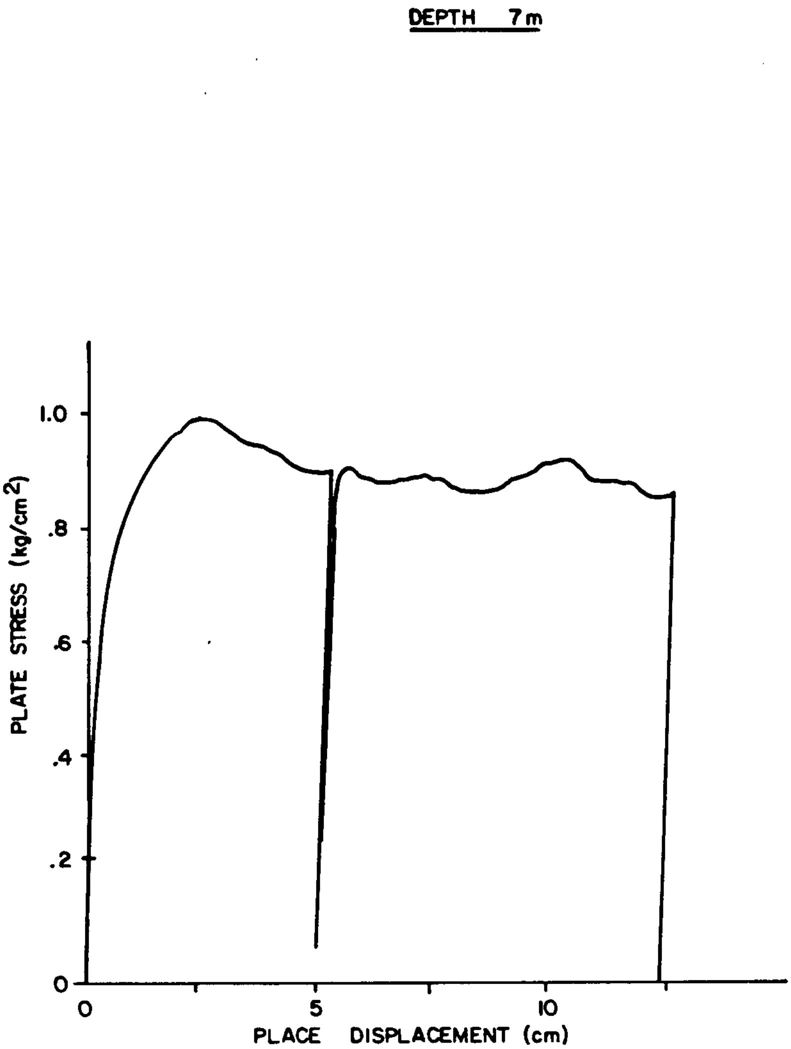


FIGURE 4·20 - LOAD TEST IN LANGLEY SENSITIVE CLAY

During the tests at this particular site, it was observed that the torque rods were rotating slightly during the application of load. When the rods were clamped as shown in Figure 4.21, the peak load was again achieved.

A possible explanation of this phenomena is that the frictional resistance of the thoroughly remolded soil in contact with the plate is negligible, hence the plate has a tendency to rotate during loading because of its screw pitch. This will tend to reduce the measured strength and modulus.

The irregularities in the loading curve beyond the peak are possibly related to failure planes which develop beneath the plate and result in a discontinuous load-deformation curve. Again, this phenomena has not been extensively treated in the literature.

Note also that the apparent undrained modulus does not appear to have been significantly affected by loading the plate past the peak or "failure" load. This is an important consideration when examining the relationship between initial and reload modulus values from undrained tests in clay.

4.2.4. Undrained Modulus

A comparison of undrained modulus values is shown in Figure 4.22. The screw plate moduli reported are E_{50}, taken as a secant modulus to a load of 50% of P_{ult}; and E_{loop}, which is taken from the first reload cycle. The screw plate moduli give modulus values approximately twice that estimated from the cone. This seems quite reasonable, since the cone modulus is based on laboratory correlations which generally underestimate the in-situ modulus.

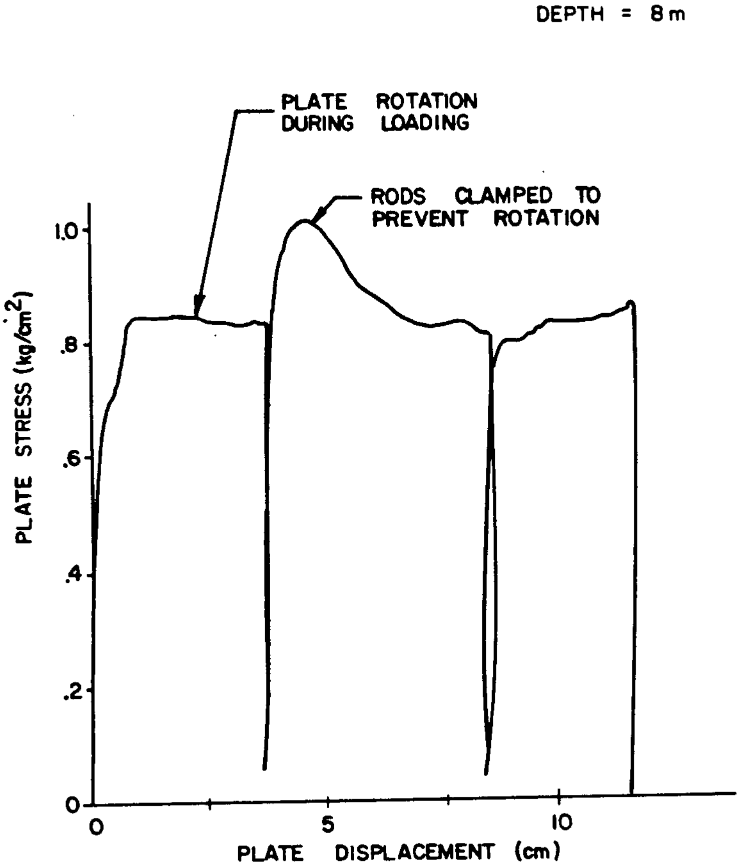


FIGURE 4·21 – EFFECT OF PLATE ROTATION DURING TESTING IN LANGLEY SENSITIVE CLAY

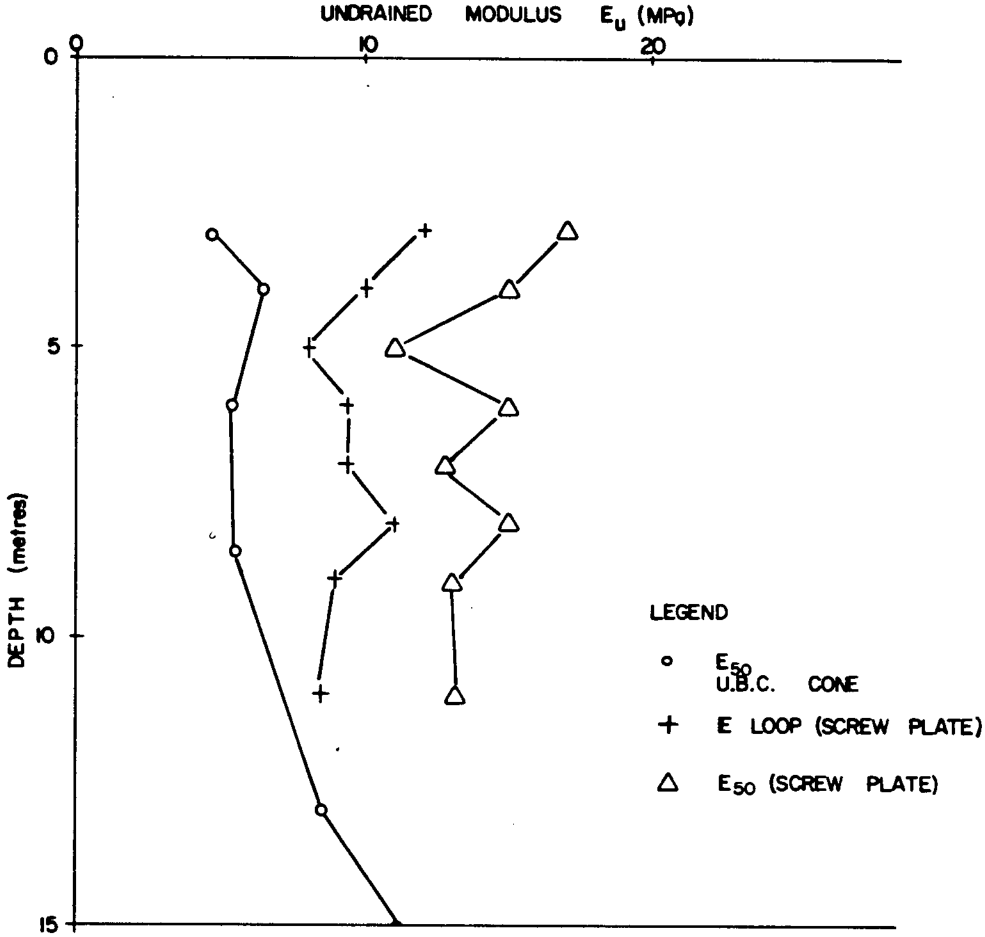


FIGURE 4·22 - COMPARISON OF UNDRAINED MODULI AT LANGLEY CLAY SITE

More test data is required before a conclusion can be drawn.

4.2.5. Undrained Shear Strengths

The general agreement between undrained strengths from a variety of tests (excluding the dilatometer) is evidenced in Figure 4.23. In view of the previously stated difference in the orientation of the failure planes associated with each test, this agreement is not necessarily expected. Naturally, the selection of bearing factors is somewhat arbitrary, nonetheless the general agreement beween various in-situ determination of c_u is encouraging. The value of $N_c = 17$ chosen for the cone was based on a rigidity index of $I_R = G/c_u = 200$, and Baligh's (1975) solution. Adoption of a single N_K or N_c for a particular site would require a more comprehensive laboratory and field programme. The effects of strain rate and strength anisotropy were not evaluated at this site; and would, result in variations in undrained shear strengths. A statistically valid number of tests would be required in order to make a confident assessment of these factors.

4.3. Cloverdale Sensitive Clay Site

4.3.1. General Geology and Site Description

The site is located on level ground approximately 2-3 metres above sea level. The site is underlain by silty clays and clays of the Cloverdale sediments situated on till. The specific lithology at the site is:

0-1.5 m - silt, sand and gravel fill

1.5-4.0 m - clay, slightly O.C., soft

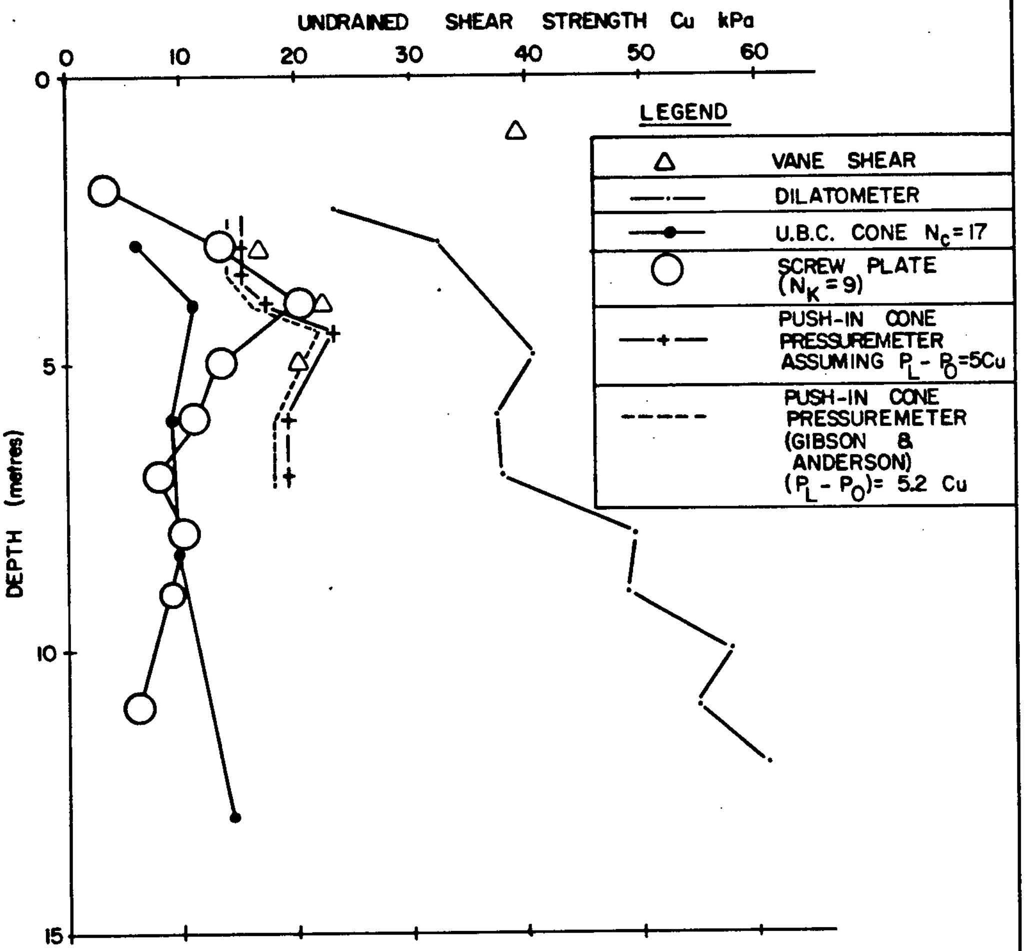


FIGURE 4·23 – COMPARISON OF UNDRAINED SHEAR STRENGTHS AT LANGLEY SENSITIVE CLAY SITE

4-14 m - clay, slightly O.C., very sensitive, S_t = 10 to 27

14-25 m - clay, N.C., soft silty

25-25.3 m - dense sand

25.3-27.7 m - clay, N.C.

27.7 → - glacial till

A typical cone profile is presented in Figure 4.24.

4.3.2. Description of Field Programme

A series of in-situ tests were completed at the site. The test programme is summarized in Table 4.3 below.

TEST DESCRIPTION	NUMBER
Cone penetration test	1
Dilatometer	1
Vane shear	1
Piezometer cone profile	1
Screw plate test profile	1

TABLE 4.3. Summary of Field Programme - Cloverdale Site.

Test locations are given in Figure 4.25.

4.3.3. Test Results

4.3.3.1. Undrained Elastic Modulus

A typical load displacement curve is presented in Figure 4.26. At this site the torque rods were not clamped, hence the true peak loads were probably not attained. Based upon the tests at Langley, this might lead to a 10-15% underestimation of the ultimate load. It is also evident here that the selection of a suitable modulus

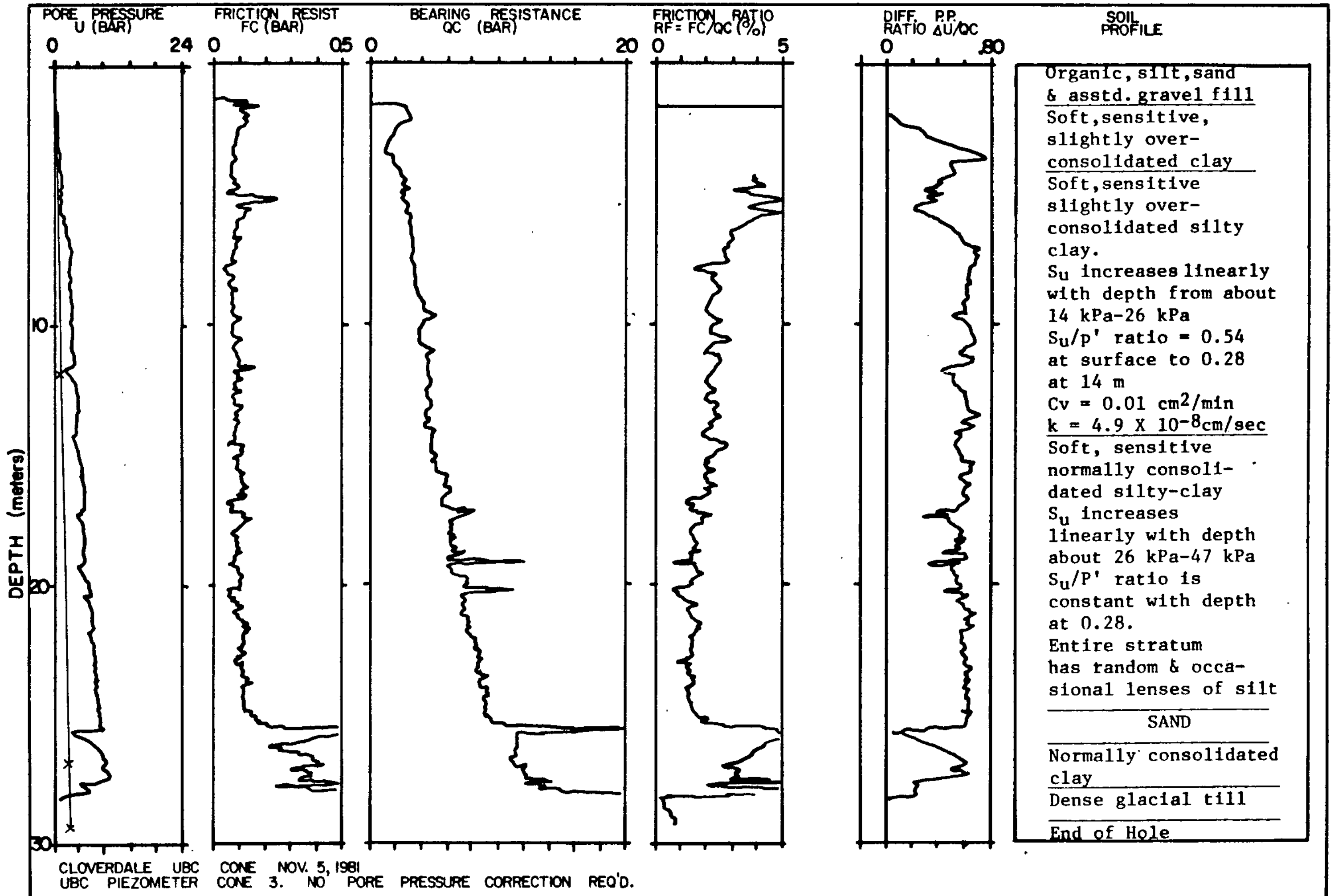


FIGURE 4·24 - TYPICAL CONE PROFILES AT CLOVERDALE SITE

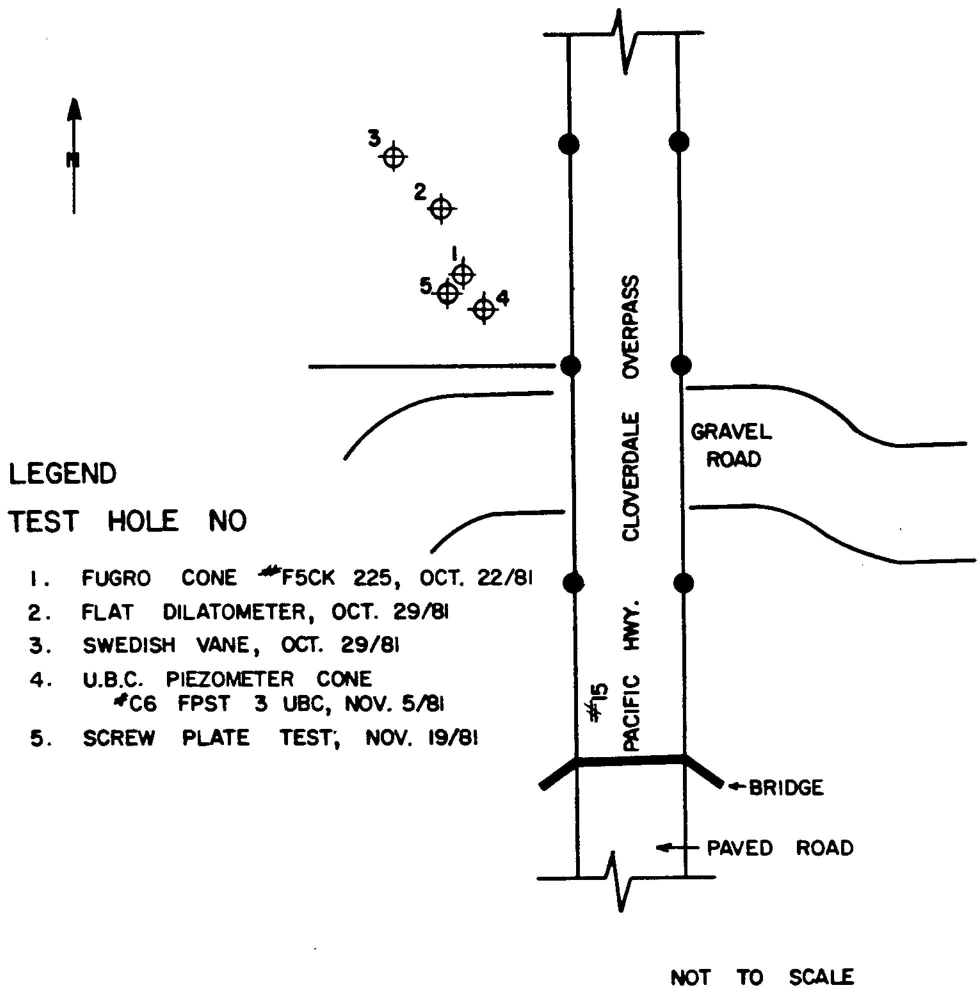


FIGURE 4·25 - LOCATION PLAN CLOVERDALE CLAY SITE

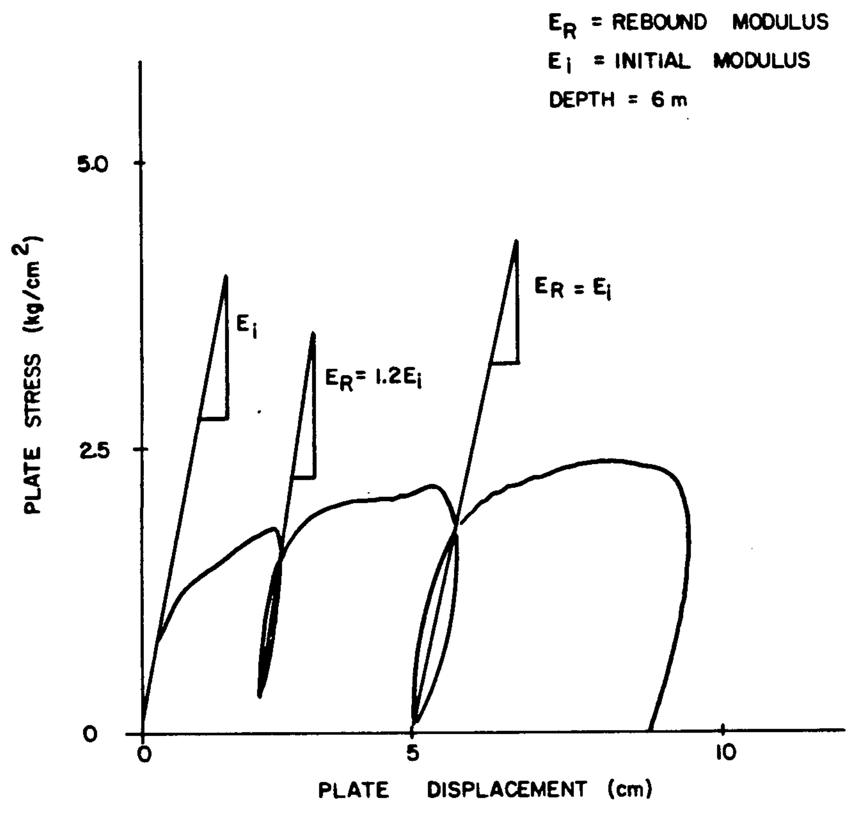


FIGURE 4·26 - LOAD TEST IN CLOVERDALE CLAY from a reload curve is very much dependent upon the amplitude of the plate loadings reached during each cycle.

Values of E_u are compared in Figure 4.27. It appears that except for E_{50}, the screw plate test produces higher values of the in-situ undrained modulus, as was the case at Langley. Again, this reflects the fact that the screw plate modulus may more closely correlate with the true in-situ modulus.

An important point here is that the tests in the Langley clay showed no change in modulus for a full unload-reload past failure. In contrast, the Cloverdale test shows that the modulus varies if full unload is not achieved.

4.3.3.2. Undrained Shear Strength

Undrained shear strengths are compared in Figure 4.28. A cone $N_c = 12$ was selected based on a best fit with the corrected vane strengths. Remarkably good correlation is seen between the various in-situ tests, with the dilatometer again giving slightly higher values of c_u in the sensitive clay. It should be noted, however, that at this site, plate rotation was not prevented, hence the ultimate load reached during the test may have been underestimated.

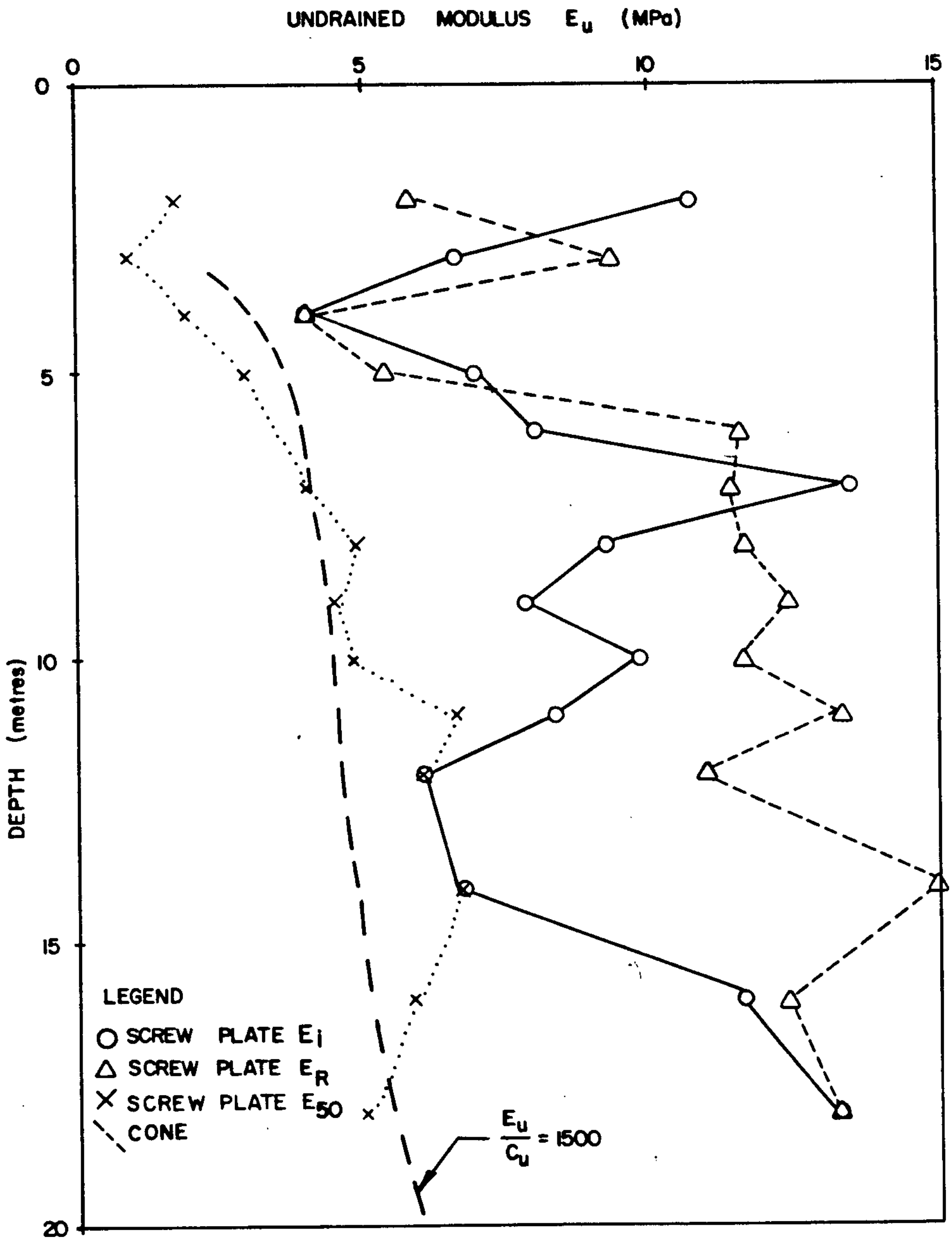


FIGURE 4·27 - COMPARISON OF UNDRAINED MODULI AT CLOVERDALE CLAY SITE

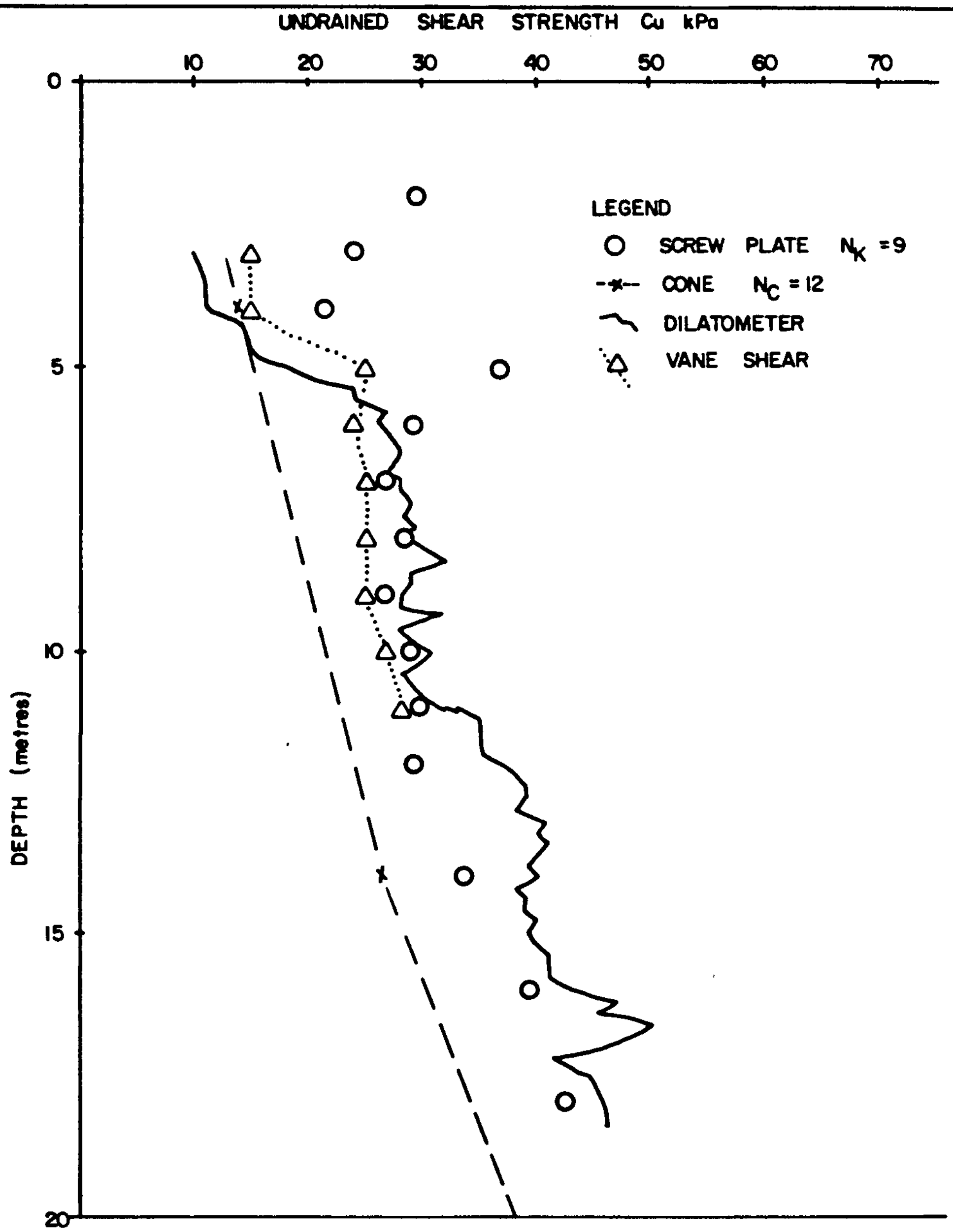


FIGURE 4·28 - COMPARISON OF UNDRAINED SHEAR STRENGTHS AT CLOVERDALE, CLAY SITE

5. SUMMARY AND CONCLUSIONS

5.1. Measurements of Drained Behavior

5.1.1. Constrained Modulus

The screw plate test appears to produce an underestimation of the in-situ constrained modulus, M. This underestimation arises because of several factors, most notably:

1) plate flexure which results in greater deflections,
2) Janbu's assumption of vertical compression,
3) disturbance associated with plate installation.

This combination yields values of k_m which are 25 to 30 percent of expected k_m values. The k_m determinations are consistent, however, and indicate that reproducible results can be obtained.

5.1.2. Young's Modulus

The determination of a Young's modulus, E_s, using Schmertmann's method is also sensitive to the effect of plate stiffness. The results shown herein indicate that the application of a modulus factor, λ, according to the relative stiffness of the plate, will result in E_s values similar to those published elsewhere. This highlights the applicability of the test in sands of varying densities.

5.2. Determination of Drained Modulus from Unload Reload Curves

Review of the cyclic loading curves indicates that the screw plate reload modulus is very much dependent upon stress level and stress amplitude. Determination of an absolute reload modulus is further complicated by the uncertain load condition acting on the back of the plate. It does appear, however, that careful selection of a maximum plate load and stress amplitude could yield a reasonably consistent modulus values.

5.3. Measurement of Undrained Shear Strength - c_u

The screw plate c_u determinations agreed very well with other in-situ tests. An N_K value of 9.0 was found to provide a reasonable estimate of c_u in the clays tested, however further site specific correlations are required. It was also discovered that plate rotation during load testing in sensitive clays can lead to an underestimation of c_u.

In general, in-situ determinations of c_u must address the issues of strain rate, strength-anisotropy, and progressive failure before confident correlation can be made.

5.4. Measurement of Undrained Modulus

The most apparent uncertainty of the screw plate test is its estimate of undrained modulus in sensitive clay. As with the determination of sand moduli, the plate boundary conditions including plate back load and torque-induced shear strain contribute to rather uncertain E_u determinations. Part of the inaccuracy of existing interpretive techniques also lies in their assumption of homogeneous, elastic, isotropic media.

The selection of a suitable undrained modulus from the screw-plate test has historically involved an arbitrary selection of the stress level at which an unload-reload modulus is determined. The data presented during this study highlights the necessity for further work into the resolution of whether initial tangent or reload moduli should be used.

6. SUGGESTED TEST PROCEDURE AND ANALYSIS

As with any test, the standardization of test procedures and analyses will improve the applicability of the test in a variety of soil conditions. Based on the results of this study, the following guidelines are proposed.

6.1. Test Equipment

6.1.1. Screw Plate

The screw plate should satisfy the following criteria:

(1) double-flighted

(2) cross-sectional area of 250 cm^2 for use in sand; 500 cm^2 in clay

(3) have a high relative stiffness, as defined in Section 3.1.2.

6.1.2. Installation System

The torque required during installation of the plate depends primarily upon the soil type. During this study, it was observed that the installation torque varied accordingly:

100 to 250 Nm - in clay and silt

250 to 2000 Nm - in sand.

The selection of a suitable torque motor will be governed by the observed cone resistance. Where a higher torque is required during installation, the plate may be advanced by cyclically reversing the direction of rotation; alternatively a smaller plate should be used.

The rate of advance of the plate can be up to a rate equal to the pitch times the rate of rotation. In general, a low friction bearing at the top of the rods should permit the unhindered advancement of the plate into the soil.

The screw plate rods should be pinned so that the plate may be removed upon completion of the profile. A larger rod cross-sectional area is required in order to improve the relative stiffness of the plate, perhaps at the expense of increased rod friction during loading.

6.1.3. Loading System

The loads required for failure in cohesive materials ranged from 1.0 to 2.5 bars. In sands, failure loads may not necessarily be achieved, as the resistance of the sand will increase during testing. Consequently, the loading frame should have a capacity of approximately:

12 kN in silt and clays

50 to 75 kN in sand.

6.1.4. Measurement System

The parameters which must be measured during the test are:

(1) plate load

(2) plate displacement.

The load may be measured using a load cell, or by measuring the piston pressure with a pressure gauge. The capacity of the load cell should be greater than the overall capacity of the loading frame.

The displacement may be measured using a displacement transducer or dial gauge. During this study it was observed that displacements were generally:

$$\delta_{Total} = 0 \text{ to } 2.0 \text{ cm in sand}$$
$$= 2.0 \text{ to } 5.0 \text{ cm in clays.}$$

The range and accuracy of the displacement gauges should be selected accordingly.

The use of automated recording equipment and electrical transducers allows the operator to apply rapid cyclic loads with a suitable loading frame.

6.2. Suggested Procedure for Plate Installation

After selection of a suitable plate and installation system, the installation of the plate itself is relatively straightforward. It can be rotated to the test depth at a variable rate, governed primarily by the ability of the frame torque motor to follow the plate downward. The plate should be permitted to "pull itself" downward without the application of vertical loads. Load tests should be separated by at least four plate diameters in order to reduce the possibility of superposition of strains.

Upon completion of the profile, the plate should be withdrawn and inspected. During withdrawal, it may be necessary to clamp the rods to prevent the apparatus from slipping downward, though it is unlikely that the rods would drop completely down the hole.

6.3. Suggested Test Procedure in Sand

Drained tests in sand should be carried on in the following manner.

1) An initial load equal to approximately 60% of the capacity of the loading frame should be applied, and the load displacement curve recorded.

2) The constrained tangent modulus is then obtained by:

$$M = k_m p_a \left(\frac{p'}{p_a}\right)^{1-a} \qquad (2.1)$$

Janbu and Senneset (1974)

where: M = constrained tangent modulus

k_m = modulus number

p_a = reference stress (1 bar)

p' = vertical effective stress

a = stress exponent = .5.

The modulus at the test depth can be estimated from a tangent drawn through the initial portion of the test curve

$$\delta = \frac{S}{k_m} \frac{p_n B}{p_a} \qquad (2.2)$$

where: δ = plate deformation, corrected for rod compression

B = plate diameter

p_n = net plate stress = applied stress + rod weight where necessary

S = dimensionless settlement number, S, (Figure 2.2).

3) The stiffness of the plate should be calculated using the stiffness factor R, whereby:

$$R = \frac{\pi}{12} \frac{(3-4\nu)(1+\nu)}{(1-\nu_p)(1-\nu)} \frac{E_p}{E} \left(\frac{h}{a}\right)^3 \qquad (3.2)$$

Selvadurai et al. (1979)

in which: h = plate thickness

a = plate radius

$\{\nu\}$ and $\{E\}$ = soil elastic constants

$\{\nu_p\}$ and $\{E_p\}$ = plate elastic constants.

The calculated modulus value should then be multiplied by the appropriate stiffness correction factor, β:

$\beta = 1$ for $\log R = 2$

$\beta = 1.5$ for $\log R = 0$.

4) Based on the limited data obtained during this study, a realistic vertical constrained modulus, M_{true}, can be estimated using:

$$\frac{M_{true}}{M_{Janbu}} = (2.3 \text{ to } 3.6) \text{ in sand.} \qquad (4.5)$$

5) Alternatively, Schmertmann's method of analysis can be applied to the data to obtain an equivalent Young's Modulus, E_s, for vertical compression. E_s is obtained by using a secant modulus over the range from 1 to 3 bars, and the relationship:

$$\rho = C_1 \, \Delta p \sum_{o}^{2B} \frac{I_z}{E_s} \Delta z \qquad (2.8)$$

where: ρ = plate settlement

Δp = $p - p_o$

p = applied plate stress

p_o = in-situ effective overburden pressure

B = plate diameter

I_z = strain influence factor (Fig. 2.3)

Δ_z = depth

C_1 = embedment correction

$= \{ 1 - .5(\frac{p_o}{p-p_o}) \}$

Any consistent units may be used. This modulus has been verified for a number of case histories, and is most applicable to shallow foundations which generally have design stresses of that order.

6) A complete unload-reload cycle should then be completed to the maximum capacity of the loading frame. The secant modulus during this load curve, should be approximately equal to the initial tangent modulus. A deviation from this observation probably reflects the effects of soil disturbance. A significant difference would indicate that the soil has been disturbed during installation of the plate, and that only the rebound modulus will be reliable.

6.4. Suggested Test Procedure in Clay

(1) Based on an understanding of the site stratigraphy, which can be obtained through cone penetration testing or more conventional investigative methods, the in-situ undrained shear strength should be estimated. The ultimate plate stress can be estimated using the following relationship:

$$p_{ult} = c_u N_K + \sigma_{vo} \qquad (2.15)$$

where: p_{ult} = ultimate plate capacity

c_u = undrained shear strength

N_K = bearing factor $\cong$ 9

σ_{vo} = in-situ vertical effective stress.

(2) A rapid undrained test should be performed, with the load applied to 60% of the estimated p_{ult}, and dropped to zero. An estimate of undrained modulus, E_u, can then be made using Selvadurai et al. (1979):

$$\frac{\delta}{pa/E_u} = \lambda \qquad (2.13)$$

Selvadurai and Nicholas (1979)

where λ = $\{0.60 \text{ to } 0.75\}$ = modulus factor (2.14)

p = plate load

a = plate radius

E_u = undrained elastic modulus

δ = plate displacement.

This modulus should be calculated using the initial portion of the load displacement curve.

(3) Care must be taken to prevent plate rotation during the test.

(4) A final load cycle should be applied to the failure load. From this load, c_u can be determined from

$$c_u = \frac{p_{ult} - \sigma_{vo}}{N_k} \tag{2.15}$$

where: c_u = undrained shear strength

p_{ult} = plate load plus rod weight

σ_{vo} = in-situ vertical stress

N_k = bearing factor = 9.

(5) Further load cycles should be applied to check for strain softening.

(6) The plate should be advanced at least one metre to the next test depth, and the test repeated.

(7) Upon completion of the profile, the plate should be withdrawn, and inspected for damage.

(8) In stiffer clays, where an ultimate load may not be achieved, the ultimate load may be estimated by:

$$p_{ult} = 2.54\ p_y - 1.54\ p_x \tag{2.17}$$

Kay and Parry (1982)

in which: p_{ult} = ultimate plate stress

p_x = plate stress at a strain equal to 1.5% of the plate diameter (B)

p_y = the plate stress at a strain equal to 2% of B.

(2) The constrained modulus number 'k_m' can be estimated in a fully drained test from the initial tangent portion of the curve and Janbu's formula:

$$\delta = \frac{S}{k_m} \frac{p_n B}{p_a}$$

where: δ = plate deformation

k_m = constrained modulus number

S = dimensionless settlement number (see Figure 2.2)

p_n = net plate stress

p_a = reference stress (= 1 bar)

B = plate diameter.

(3) Corrections should be made for rod compression and effect of plate rigidity. These corrections are detailed in 3.1.2.

7. SUGGESTIONS FOR FUTURE RESEARCH

Future research related specifically to the research sites discussed herein should be directed towards obtaining more field vane shear data to correlate with the screw plate estimates of undrained shear strength. This is particularly true of the silty clays at depth at the McDonald site, where no vane tests were completed. Through these correlations, the effects of variable strain rate and strength anisotropy can be examined. Furthermore, efforts should be made to eliminate rod friction and plate rotation in the additional screw plate tests at those depths.

Further research into the soil behavior in drained tests in sand should be directed at achieving a more fundamental understanding of the relationship between the screw plate - derived modulus number k_m, and the true k_E value of the soil. Parametric finite element studies, including a study of the effect of load on the back of the plate, would be most applicable here.

Additional field testing could be used to verify the predicted response, and directed towards improvements in the test apparatus, including plate geometry and stiffness.

As our fundamental understanding of soil behavior improves, more attention can then be directed towards interpretting the results of cyclic load tests performed using the plate. This could include relating the unload-reload curves to such parameters as in-situ moduli and sand densities, and their application to machine foundations.

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