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# Study of in-situ test methods in deltaic silt

## Etude des méthodes d'essai in-situ dans un limon deltaïque

J.-M. KONRAD, Research Officer, Division of Building Research, National Research Council Canada, Ottawa, Canada  
 M. BOZOUK, Research Officer, Division of Building Research, National Research Council Canada, Ottawa, Canada  
 K. T. LAW, Research Officer, Division of Building Research, National Research Council Canada, Ottawa, Canada

**SYNOPSIS** A comprehensive in-situ testing program using a field vane, electric cone penetrometer with friction sleeve and piezometer tip, dilatometer and a self-boring pressuremeter, was carried out in deltaic clayey silt of low plasticity, below 15 m of sand. Soil strengths measured with the pressuremeter were 62% more than the vane shear strengths which in turn were greater than those measured with the dilatometer. In-situ horizontal stresses measured with the pressuremeter provided an average  $K_0 = 0.56$ . It also provided a measure of the rigidity index used to study the piezocone test results.

The present study revealed that empirical correlations developed for clay soils were not directly applicable to the clayey silt formation. Rate effect during cone penetration differed also from that generally observed in clays.

### INTRODUCTION

The design and construction of safe, economical oil and gas exploration and production structures for developing Canada's offshore energy resources require the measurement of engineering properties of the foundation material. Various in situ tests that have been performed in the marine soils of the Beaufort Sea were repeated in similar fine-grained deltaic soil deposits at the mouth of the Fraser River in Vancouver. The test equipment consisted of an electric cone with a friction sleeve and a piezometer tip (piezocone), a flat plate dilatometer, a field vane and a Cambridge self-boring pressuremeter. The paper discusses the results of the test program related to the clayey silt deposits underlying 15 m of fine sands.

### DESCRIPTION OF SITE AND SOIL CONDITIONS

The selected test area was at the northern tip of Sea Island, located at the mouth of the north arm of the Fraser River in Vancouver. The island is about 5.6 km from east to west, 4.1 km from north to south and is the site of the Vancouver International Airport. The subsoils are deltaic sands overlying silts, extending to more than 100 m deep. The ground surface is very flat with an elevation of +1.6 m and is protected from flooding by dykes. The depth of the groundwater table was about 1 m, and it fluctuated daily about 1 m due to the effect of ocean tides. Piezometric measurements showed that the pore water pressures were hydrostatic with depth.

The soil profile is made up of 0.3 m of organic topsoil underlain by silty clay and fine sand

to a depth of 3 m. Between this depth and 15.3 m are loose to dense deltaic sands ranging in grain size from medium to coarse containing about 4% silt. Typical of most deltaic deposits, numerous thin deposits of wood, some beach pebbles and occasional marine shells were found. This was followed by 2 m of grey fine silty sand overlying the deep deposit of layered grey silt and clay. The variations in the soil profile were reflected in the cone penetrometer test results (Figure 1).

The soil formation from 15.3 to 17.4 m was fine sand with silt and clay layers. A composite grain size analysis indicated about 60% sand, 30% silt and 10% clay sizes. The plasticity index varied from 5 to 10 and the natural water content of 38% exceeded the liquid limits by about 10-15%.

Soil tests on samples obtained from 17.3 to 24 m indicated about 10% sand, 70% silt and 20% clay sizes. The liquid limits averaged 35%, slightly less than the average water content of 36%. The plasticity index of 10% is typical of cohesionless fine grained soils. The sensitivity was 5 using the field vane and 3 using the Swedish Laboratory cone. These results indicate the soil is an inorganic clayey silt of low plasticity.

The saturated density determined from Shelby tube samples and from  $G$ ,  $e$  and  $w$  relations was  $18.8 \text{ kN/m}^3$  for the clayey silt below 15 m,  $20 \text{ kN/m}^3$  for the overlying sand and  $18.4 \text{ kN/m}^3$  for the silty clay above the 3 m depth.

The geological history of these deltaic soils indicates that the deposit should be normally consolidated (Campanella et al, 1981). Valid consolidation tests could not be performed because the Shelby tube samples were badly

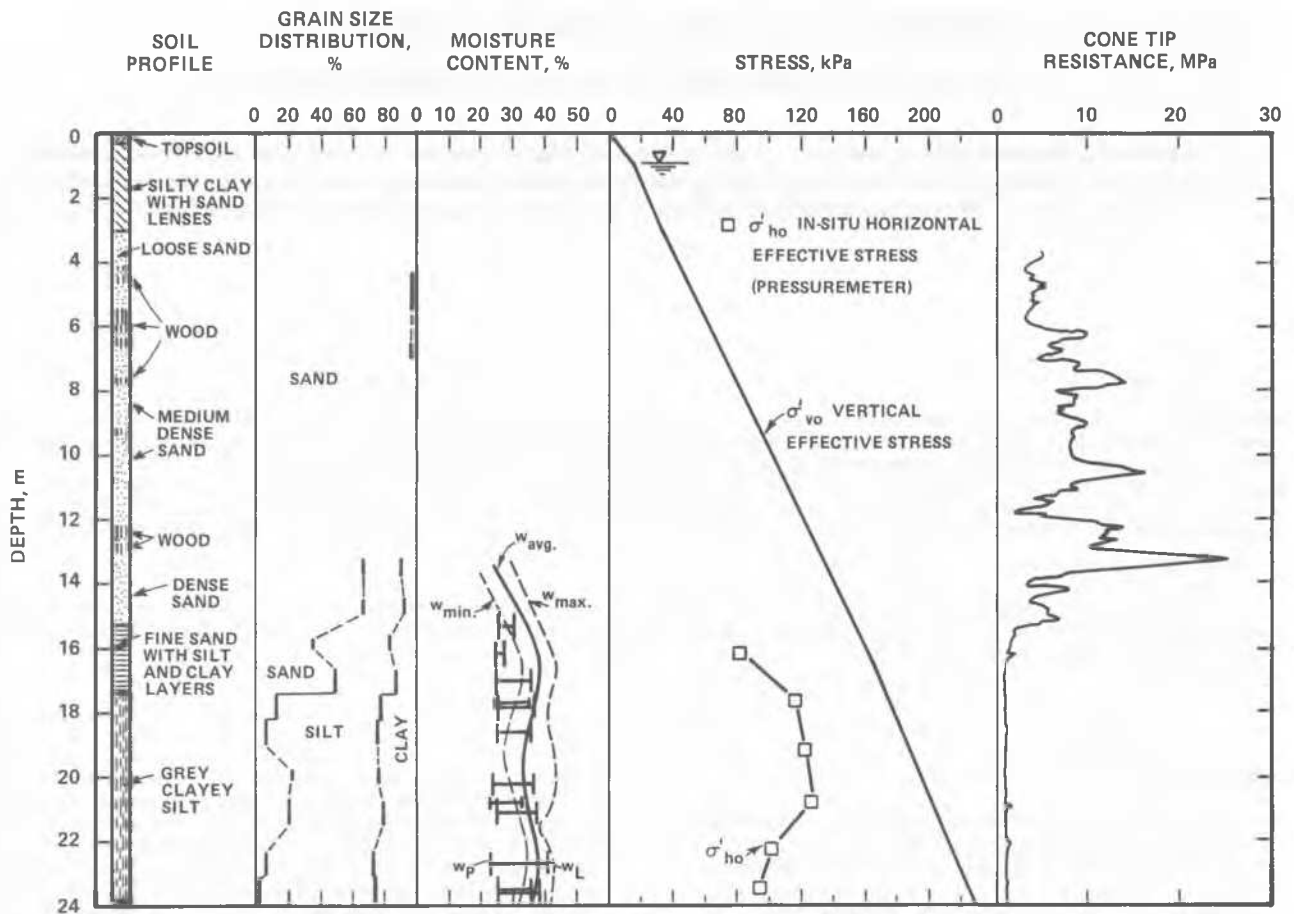


Figure 1. Soil profile, grain size distribution, soil classification, effective stress and cone penetration profile.

disturbed. Triaxial undrained strength tests on specimens isotropically consolidated to  $0.8 \sigma'_{vo}$  gave an average  $\phi'$  of  $36.5^\circ$ .

TEST RESULTS

Field Vane and Pressuremeter Tests

(a) Soil strengths measured in situ and in the laboratory include the Cambridge self-boring pressuremeter, the NGI field vane, triaxial CIU and Swedish cone tests (Figure 2). Vane sizes of 110 mm x 55 mm and 86 mm x 43 mm were used at V-1 and V-2, respectively. Both vane profiles, although 9 m apart, were in good agreement except at 24 m depth where a hard layer was encountered at V-2. The vane strengths ( $S_{uv}$ ) were about equal to the CIU values and considerably greater than those obtained with the Swedish cone on Shelby tube soil samples. Both vane profiles indicate a slight strength increase with depth, a mean  $S_{uv}$  of 74.5 kPa and an average  $S_{uv}/\sigma'_{vo}$  of 0.33.

The pressuremeter strength ( $S_{up}$ ) was determined by the method proposed by Ladanyi (1972) which assumes soil homogeneity around the pressuremeter probe but does not require prior assumption of a stress-strain model.  $S_{up}$  was about 62% greater than  $S_{uv}$  (Figure 2). A summary of the results of six pressuremeter tests is given in Table I.

Table I. Summary of Pressuremeter Test Results

Test No.	Depth m	$\sigma'_{ho}$ kPa	G MPa	$S_{up}$ kPa	$I_R$	$K_O$
10	16.1	230	23.0	94.8	227	.50
11	17.6	280	18.4	89.1	233	.67
12	19.1	300	23.9	95.7	210	.65
13	20.7	320	18.3	108.7	190	.63
14	22.2	310	26.7	119.6	170	.47
15	23.4	315	27.1	134.8	150	.42

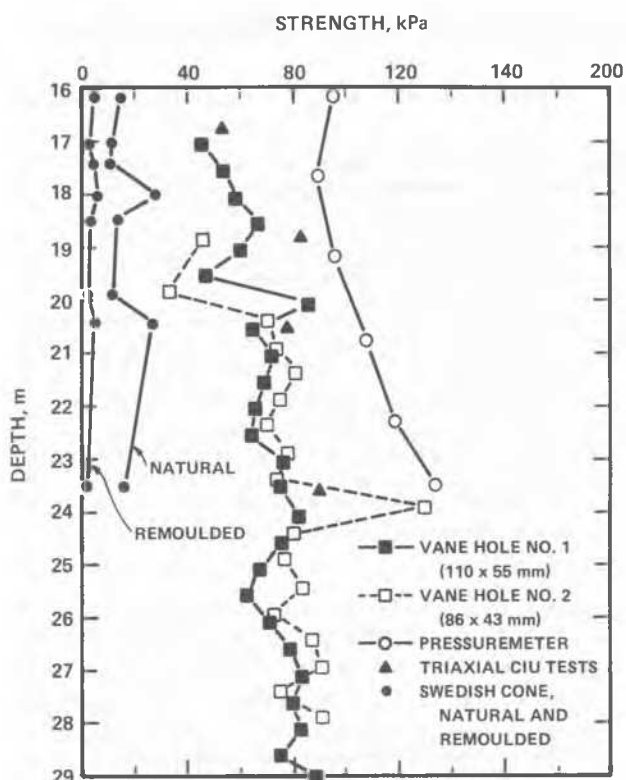


Figure 2. Comparison of soil strengths obtained with the NGI vane, pressuremeter, triaxial CIU and Swedish cone in the clayey silt formation.

The fact that the pressuremeter measures a higher strength than the field vane has been well documented (Baguelin et al, 1972; Eden and Law, 1980; Lacasse et al, 1981). The relationship between  $S_{up}/S_{uv}$  and the overconsolidation ratio (OCR) is shown in Figure 3 for a number of clays with plasticity indices between 25% and 40%. Although there appears to be a correlation between the strength ratio and the OCR, the data for Sea Island clayey silt ( $I_p = 10\%$ ) does not fall into this relationship.

In clay soils, the trend of decreasing  $S_{up}/S_{uv}$  with increasing OCR is reasonable. Natural soils at high OCR are commonly fissured and under high in situ horizontal stress. The vane measures high intact strengths in spite of fissures, and the higher in situ horizontal stresses also lead to higher vane strengths. This was demonstrated from a slope stability analysis of a natural slope involving clay soils with OCR = 3.8 where the vane measured a much higher strength than was available in the slope (Eden and Jarrett, 1971). The pressuremeter, however, imposes less constraint on the fracture surface and often stresses the soil horizontally beyond the preconsolidation

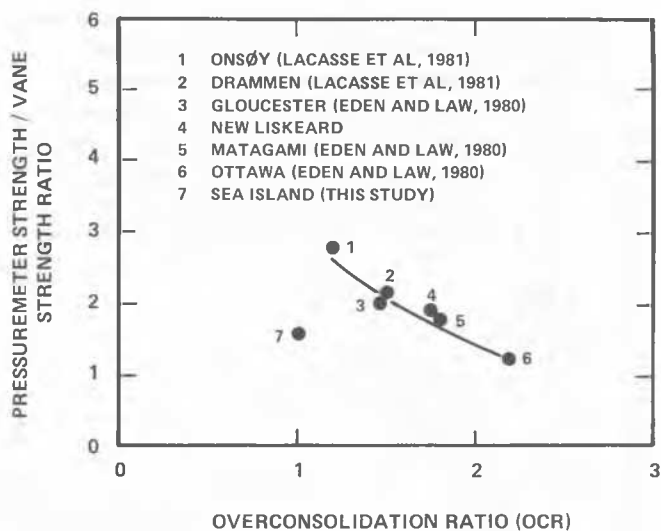


Figure 3. Correlation of pressuremeter/vane strength ratio and the overconsolidation ratio of soils.

pressure. Consequently, the effects of high OCR leading to high undrained strength is less significant for the pressuremeter test and the ratio  $S_{up}/S_{uv}$  decreases with increasing OCR.

(b) The in situ horizontal stress,  $\sigma_{ho}$ , is obtained by noting the lift-off pressure at the beginning of the pressuremeter test. This pressure which corresponds to the pressure surrounding the probe may be different from the original in situ value due to the effects of disturbance introduced by the insertion of the probe. For the present silty soil, the effect of disturbance is small and the lift-off pressure is therefore close to the in situ horizontal pressure. From the results shown in Figure 1 the coefficient of earth pressure at rest,  $K_o$ , varied from 0.42 to 0.67 with an average of 0.56.

#### Piezo-Friction-Cone

(a) The piezometer in the piezo-friction-cone is located above the conical tip where water pressure can act over a part of the base area of the cone inside the penetrometer and on both ends of the friction sleeve. The measured total stress  $q_c$  will therefore be reduced. For the 10 cm<sup>2</sup> GMF piezo-friction penetrometer used in this study:

$$q_T = q_c + 0.34 u_T \quad (1)$$

where  $q_T$  = corrected total stress and  $u_T$  = measured total pore water pressure. To correct the friction sleeve measurements, consider the total pore water pressure acting

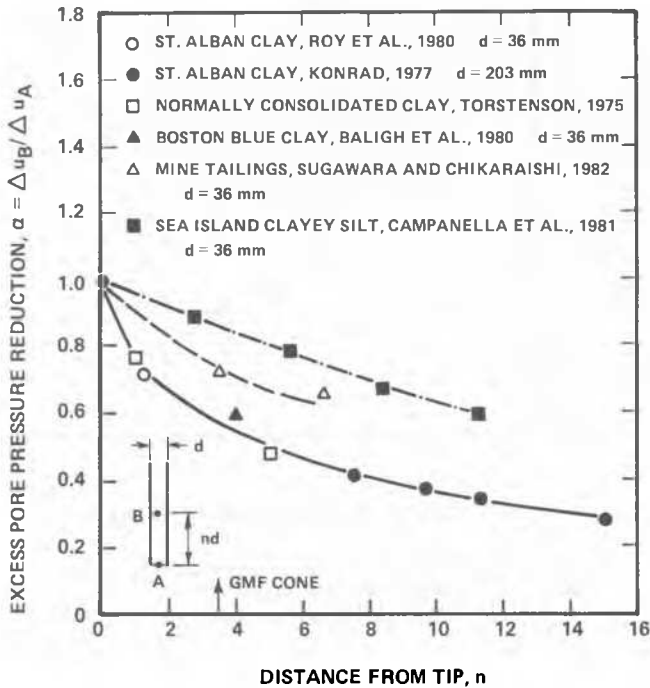


Figure 4. Distribution of excess pore water pressure in saturated soil along a penetrating cylindrical pipe.

at both ends of the sleeve. Figure 4 shows that the distribution of excess pore water pressure in saturated soil along a shaft of a penetrating cylindrical pipe is a function of the distance from the base. It may be influenced by soil type, but it is quite insensitive to variations in diameter of the pipe. The correction for the penetrometers used in the study will be:

$$f_s(\text{corrected}) = f_s(\text{measured}) - \beta u_o - \gamma \Delta u_A \quad (2)$$

where  $\beta = \frac{A_A - A_B}{A_S}$

$$\gamma = \frac{A_A \alpha}{A_S}, \quad \alpha = \frac{\Delta u_B}{\Delta u_A}$$

- $A_S$  = surface area of sleeve
- $A_A$  = cross-sectional area of sleeve at bottom end
- $A_B$  = cross-sectional area of sleeve at upper end
- $\Delta u_A$  = measured excess pore water pressure at bottom end of friction sleeve
- $\Delta u_B$  = excess pore water pressure at upper end of friction sleeve.
- $u_o$  = average hydrostatic pore water pressure in the soil around the tip of the cone.

It must be stressed that equal cross-sectional end areas of the sleeve will still require a

correction of the measured friction given by:

$$f_s(\text{corrected}) = f_s - (1-\alpha) \frac{A_A}{A_S} \Delta u_A \quad (3)$$

These are extremely important when dynamic pore water pressures are generated during penetration in clays or silty soils but are practically negligible in cohesionless soils because the tests are performed under practically drained conditions.

(b) Four soundings were performed at adjacent locations spaced from 3 m to 4 m apart. At the first, measurements of cone tip resistance  $q_c$  and skin friction  $f_s$  were made continuously using a standard penetration rate of 2 cm/sec. The effect of varying the rate of penetration was studied at the other three soundings where measurements of  $q_c$ ,  $f_s$  and total pore water pressure  $u_T$  were obtained for penetration rates of 2, 1 and 0.5 cm/sec.

The results of the cone tests are shown in Figures 5 to 7. There was no zero shift for  $q_c$  and  $u_T$  after each sounding; however, there was a slight shift in  $f_s$ . The change was less than 5% of the average measured friction and thus no further zero correction was made.

The piezo-friction-cone penetrometer is very sensitive to changes in the stratigraphy. There is a sharp increase in cone tip resistance, sleeve friction and an abrupt reduction in pore pressure at 20.5 m in test C3 only, probably due to the presence of a sand lens. Slight increase in  $q_T$  with a concomitant decrease in  $u_T$  are observed at depths 22, 24 and 26 m indicating possible layers of siltier soil (Figure 1).

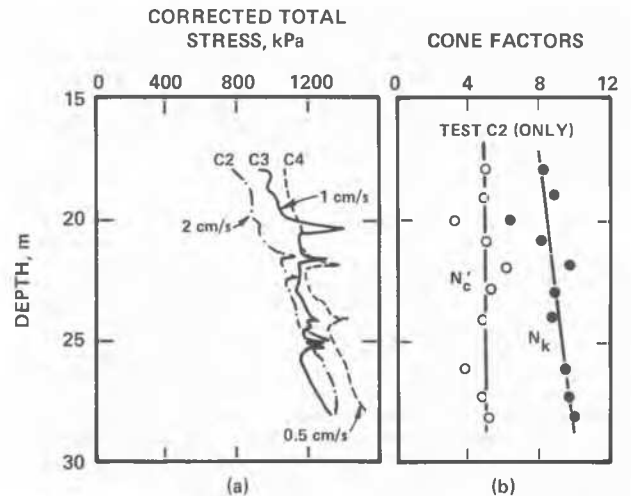


Figure 5. Corrected cone tip resistance profiles and cone factors.

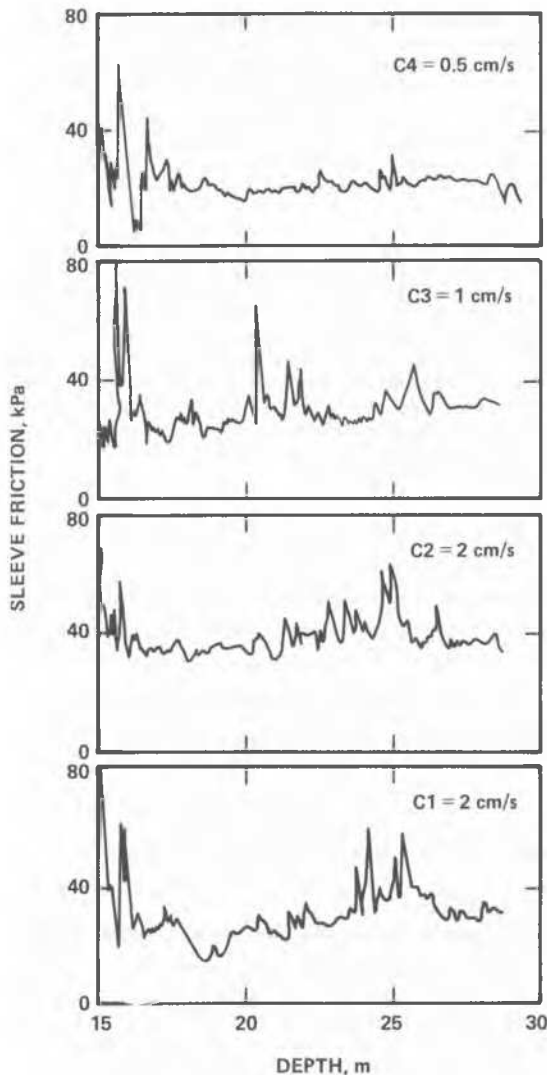


Figure 6. Measured sleeve friction for various cone penetration rates.

The presence of these various silt layers and sand lenses at different depths in the soft deposit, typical of most deltaic formations, results in some stratigraphic variations in the different cone logs, although the test holes were closely spaced. Therefore, the effect of different penetration rates are analysed using the average values of tip resistance, sleeve friction and total pore water pressure.

The total tip resistance  $q_T$  was penetration rate dependent in the clayey silt formation. Its average value increased about 10% each time the penetration rate was reduced 50% from 2 to 1 to 0.5 cm/s. The cone tip resistance is dependent upon the behaviour of the soil and the effective stress conditions around the cone. One significant factor is the pore pressure distribution around the tip. This

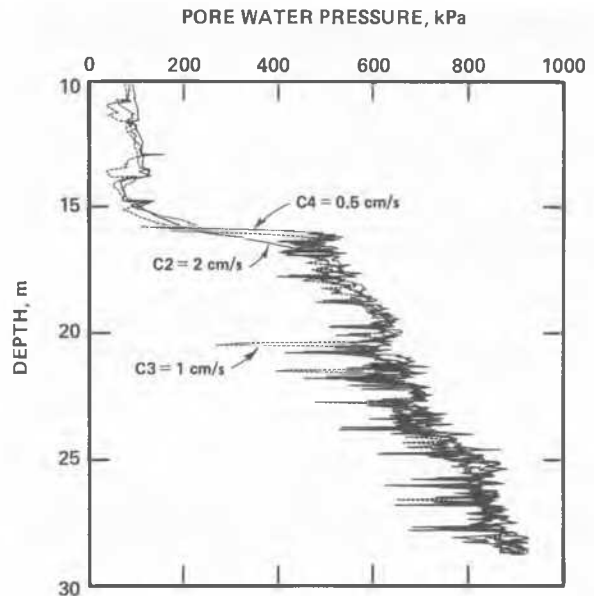


Figure 7. Excess pore water pressure profiles obtained at three different rates of penetration.

increase of  $q_T$  at low rates is thought to be associated with the development of partially drained conditions induced by pore pressure dissipation in all directions ahead of the tip. The pore water pressures have been measured at the base of the shaft and were not appreciably affected by the different rates of penetration (Figure 7). It must be stressed, however, that these pore pressures do not reflect the actual pore pressures below the cone, owing to the different consolidation conditions around the penetrometer. Since the consolidation ahead of the tip is spherical and around the shaft it is cylindrical, the time factor for the latter pore pressure dissipation is much larger than in the soil ahead of the cone. Hence, the pore pressures generated below the penetrometer will dissipate much faster, especially in the more permeable silty soils.

The measured sleeve friction was also affected by rate of cone penetration. As shown in Figure 6, the sleeve friction decreased with decreasing rate of penetration. To illustrate, consider the profile obtained in the uniform silt below a depth of 27 m. The measured friction was 30 and 35 kPa at 2 cm/s, 30 kPa at 1 cm/s and 24 kPa at 0.5 cm/s. Note that by applying pore water pressure corrections (Equation 3) and using  $\alpha = 0.85$ , these corrected values are respectively 22 and 27 kPa, 22 kPa and 17 kPa.

#### Cone Factors

(a) Cone tip resistance, skin friction and pore pressures generated during penetration depend upon the effective stresses developed in the soil surrounding the cone and on the fundamental soil properties. The cone tip

resistance may be empirically correlated to an undrained shear strength by the relationship:

$$q_T = S_u N_K + \sigma_{vo} \quad (4)$$

where  $N_K$  = cone factor.

$S_u$  is not a unique soil property but depends on the stress path used to reach failure with different measurement techniques.

The effective value of tip resistance,  $q' = q_T - u_T$  is also related to  $S_u$  by the relation proposed by Senneset et al (1982):

$$q' = N'_C S_u \quad (5)$$

where the average  $N'_C = 9 \pm 3$ .

For saturated cohesive soils typical values of  $N_K$  are 10 to 15 for normally consolidated and 15 to 20 for overconsolidated clays (De Ruiter, 1982).

Cone factors  $N_K$  and  $N'_C$  evaluated with respect to the undrained shear strength from the field vane  $S_{uv}$  are shown in Figure 5(b) for the standard rate of penetration.  $N_K$  increases with depth and its average value is 9.  $N'_C$  is fairly constant with depth and has a value of about 5.

Approximate theoretical evaluation of the cone factor can be obtained by spherical cavity expansion failure (Vesic, 1972) from:

$$N_C = \frac{4}{3} (\ln I_R + 1) + 2.57 \quad (6)$$

where  $N_C$  is the theoretical cone factor.

For  $I_R$  between 60 and 250,  $N_C$  varies from 9.4 to 11.2. Back-calculated cone factors from:

$$q_T = N_C S_u + \sigma_{oct} \quad (7)$$

where  $\sigma_{oct} = \frac{1}{3} (\sigma_{vo} + 2\sigma_{ho})$  range between 6.8 and 10.2 which agrees fairly well with Vesic's simplified approach.

(b) Konrad (1977), Baligh et al (1980) suggested that the pore pressures measured immediately behind the tip during cone penetration could be used to determine the undrained shear strength and to indicate consolidation history of fine grained soils. Figure 7 shows the pore pressure profiles obtained from three cone tests. In the clayey silt formation below 15 m, the average pore pressure generally increased with depth. Table II presents average values of various proposed pore pressure ratios reported in current literature. As mentioned above,  $u_T/q_T$  varies with consolidation history. Work by Smits (1982) and Lacasse et al (1981) shows  $u_T/q_T$  varying between 0.6 and 0.8 in normally consolidated soils which seems to agree with the present study for clayey silt.

Table II. Summary of Pore Water Pressure Data

Depth (m)	$u_T/q_T$	$\Delta u/q_T$	$\Delta u/(q_T - \sigma_{vo})$	$\Delta u/\sigma'_{oct}$	$\Delta u/S_{uv}$
20-25	0.65	0.45	0.73	3.3	6.2
25-28	0.70	0.55	0.93	3.9	7.0

Between 20 and 25 m, the ratio of the change in excess pore pressure with respect to the change in total vertical stress is 0.73 for the field conditions and 0.66 in CIU tests. The larger field value is due to a change in total horizontal stress during the penetration of the cone. The undrained shear strength may also be determined semi-empirically from the pore pressure measurements using the following equation proposed by Vesic (1972) for a spherical cavity expansion:

$$\frac{\Delta u}{S_u} = \frac{4}{3} \ln I_R + 0.67 (3A_f - 1) \quad (8)$$

The selection of representative parameters  $I_R$  and  $A_f$  is of paramount importance for adequate determination of  $S_u$ . The average rigidity index of the soil layer between 20 and 25 m is 70 (from CIU tests) and 170 (from pressuremeter tests at 10% shear strain). Theoretical values of  $\frac{\Delta u}{S_u}$  from Equation (8) vary

from 6.3 to 7.4, which compare with the observations given in Table II using uncorrected field vane strengths ( $\Delta u/S_{uv}$ ).

#### Dilatometer (DMT) Tests

Figure 8 presents a typical DMT test profile in terms of lift off pressure,  $p_o$ , and  $p_1$ , pressure at 1 mm deformation through the clayey silt below a depth of 16 m. The recorded profile is consistent with the cone, although less detailed. Nevertheless, the larger  $p_o$  and  $p_1$  at depths of 21 m and 26 m correspond to the sandy layers detected with the cone. Two DMT soundings showed almost identical results, indicating a fair repeatability with this tool. Figure 8 shows  $p_o$  and  $p_1$  increasing uniformly with depth. The soil type index  $I_d$  defined as  $\Delta p/(p_o - u_o)$  is relatively constant at about 0.18. According to Marchetti's (1980) soil classification this indicates a silty clay, whereas the actual clayey silt should have an  $I_d$  value of about 0.6.

The determination of  $K_o$ , OCR and  $S_u$  is obtained from the horizontal stress index  $K_D$ , defined as  $(p_o - u_o)/\sigma'_{vo}$ . It was relatively constant in the clayey silt with an average  $1.8 \pm 0.2$ . This is not, however, within the range of Marchetti's empirical relationships which start at  $K_D > 1.5$  for determining  $K_o$  and  $K_D > 2$  for determining the OCR. At  $K_D = 2$ ,  $K_o = 0.5$  and OCR = 1, and for  $K_D < 2$  the OCR < 1, which is unrealistic. Consequently, further research

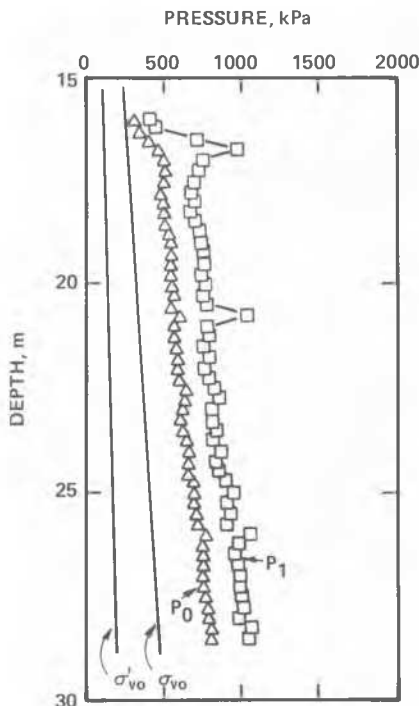


Figure 8. Dilatometer test results for DMT-6.

is required in soils where  $K_D < 2$  in order to define OCR reasonably accurately.

The determination of the undrained shear strength  $S_u$  is therefore affected because it is calculated from the following equation given by Mesri (1975).

$$\frac{S_u}{\sigma'_{vo}} = 0.22 \times \text{OCR} \quad (9)$$

From the DMT tests in the clayey silt, the empirically determined strength ratio  $S_u/\sigma'_{vo} = 0.19$  which considerably underestimated the average value of 0.33 obtained in situ with the NGI vane.

#### CONCLUSIONS

(1) In situ tests with the piezo-friction-cone, self-boring pressuremeter and the flat plate dilatometer demonstrated that empirical correlations developed for clay soils were not directly applicable to the low plasticity silt formation ( $I_p = 10$ ) at Sea Island.

(2) The self-boring pressuremeter measured 62% greater soil strengths than the field vane. In turn, the measured field vane strengths were a little less than the triaxial CIU values, significantly greater than those measured in situ with the dilatometer and at least double those obtained with the Swedish cone on tube samples.

(3) The in situ horizontal stress measured with the self-boring pressuremeter indicated a  $K_0 = 0.56$  for the clayey silt formation.

(4) The rate of cone penetration was more important on the tip resistance and friction sleeve than on the pore pressure response. In silt soils a penetration rate of 1 cm/s is suggested.

(5) The piezocone is very sensitive to the stratigraphic changes in soil profile.

(6) The flat plate dilatometer does not produce good empirical correlations for  $K_0$ , OCR and  $S_u$  in silt soils when the horizontal stress index  $K_D$  is less than 2.

(7) Samples of the clayey silt formation obtained with thin walled stainless steel Shelby tubes were considerably disturbed.

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