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Pressuremeter and Static Cone Penetration Tests in Obhor Sabkha, Saudi Arabia

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ABSTRACT. The results of 19 pressuremeter tests (PMTs) at 6 locations, and those of static cone penetration tests (CPTs) at 7 locations in the clayey parts of Obhor Sabkha are presented. Comparison of the results of these PMTs and CPTs with those of field vane shear tests (FVTs) and Mackintosh probing shows that they are equally suitable for site investigation in a clayey sabkha. Based on the results obtained so far, tentative correlations are proposed between the different design parameters obtained from these *in situ* tests. Calculations show that for equal loads and footing sizes for light structures, settlements based on CPT or PMT parameters are generally smaller than those based on consolidation test results.

Introduction

A number of recent studies (*e.g.* Sabtan and Shehata 1982, Ali 1985, Ali *et al.* 1985, Ghazali *et al.* 1985, Dhowian *et al.* 1987, Ali and Hossain 1988, have focussed on the presence and geotechnical nature of the sabkhas along the western coast of Saudi Arabia. The problems of applying the routine site investigation methods to these sabkhas are being increasingly recognized (*e.g.* Hossain and Ali 1988) and the most suitable technique (or techniques) is yet to be settled. Hossain and Ali (1990) discussed the use of two *in situ* tests, namely, the field vane test (FVT) and Mackintosh probing (Chan and Chin 1972) in the clayey areas of one of these sabkhas located in Obhor, North of Jeddah (Fig. 1) and showed that Mackintosh probing is less affected than FVTs by interference of the gypsum crystals and shells, and they suggested a tentative correlation between the results of the two tests. The suitability of other advanced *in situ* techniques like the static cone penetration test (CPT) and pressuremeter test (PMT) for the above sabkhas has not been examined so far. So it was natural for the authors to try these tests, and this paper presents their results from the clayey part of Obhor Sabkha and it attempts to correlate the results of these tests with those from other *in situ* tests such as FVT and Mackintosh probing.

Nature of Obhor Sabkha

According to Hossain and Ali (1988), the Obhor Sabkha occurs in two main zones roughly parallel to the Red Sea coast (Fig. 1). In its landward zone a surface crust of 0.6-0.8 m thickness overlies a layer of soft to firm grey sandy silty clay (CL to CH) about 2.0 m thick that constitutes the main body of the sabkha layer and rests on a layer of stiff brown sandy silty clay (CH). The seaward zone is underlain by sandy sabkha deposits resting on limestone.

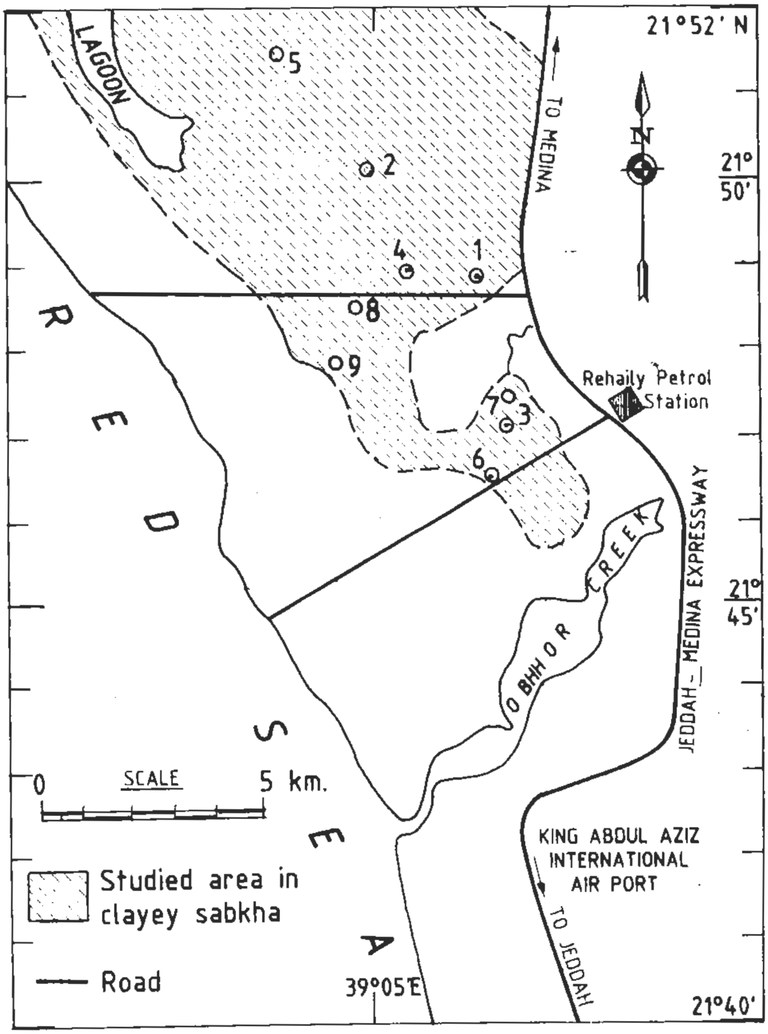

FIG. 1. Location of Obhor Sabkha.

Table 1 (from Hossain and Ali 1988) summarizes the important geotechnical characteristics of the landward zone deposits which are the concern of the present paper. The water table in this zone was at depths of 0.6-2.0 m below the ground surface during the 1984-85 period. Typical data on the chemical characteristics of soil and groundwater were reported by Ali and Hossain (1988) who indicated groundwater salinities in the range of 55-155 g/l.

TABLE 1. Summary of geotechnical characteristics of clayey soil units of Obhor Sabkha (Hossain and Ali 1988).

Property	Range of values for		
	Top crust	Main body of soft clay	Stiff brown clay
Natural water content, w_n (%)	15–40	40–78	25–31
Liquid limit, w_L (%)	20–50	36–60	50–56
Plastic limit, w_P (%)	16–40	18–30	25–33
Plasticity index, I_P (%)	5–20	20–32	24–30
Undrained shear strength by field vane shear test, s_{uv} (kN/m^2)	20–60	12–45	>50
Sensitivity, S_t	3.5–22	1.3–11	1–2
M-value from Mackintosh probing	15–140	3–40	50–300
Cone resistance from CPT, q_c (kg/cm^2)	7–12	3–5	15–60
N-value from SPT	2	1	9–24
Coefficient of volume compressibility, m_v (m^2/MN)	0.5–0.7	0.52–1.47	0.17–0.22
Compression index, C_c	0.37–0.42	0.40–0.88	0.17–0.31
Overconsolidation ratio	4–37	1.3–2.4	6–11

Equipment and Techniques

A) The Pressuremeter Tests

In the present study a commercial pressuremeter designated as type GA manufactured by erstwhile M/s Techniques Louis Menard of France (no company with this name exists now) and owned by the Faculty of Earth Sciences (FES) was used. It is essentially of the basic Menard design with two parts namely, the probe that is lowered into a pre-drilled hole and the pressure and volume control unit (C.P.V.) that remains on the ground and is connected to the probe by connecting tubes (Fig. 2). As usual, the probe has three independent cells, of which, the central cell is called the measuring cell and the two outer cells are called the guard cells. The authors' probe is 60 mm in diameter with a calibration volume of 535 cm^3. In order to reduce the errors in volume readings, which would result from the dilation of the tubing connected to the central cell, this tubing in GA pressuremeter is run co-axially through the tubing connected to the guard cells. During a test, the central cell is inflated by water applying a maximum pressure of 2500 kPa while the guard cells are inflated by a gas. The CPV has a differential valve which is used to maintain a differential pressure between the central cell and the guard cells, the higher pressure being applied in the central cell. The general procedure of PMT is available elsewhere (*e.g.* Baguelin *et*

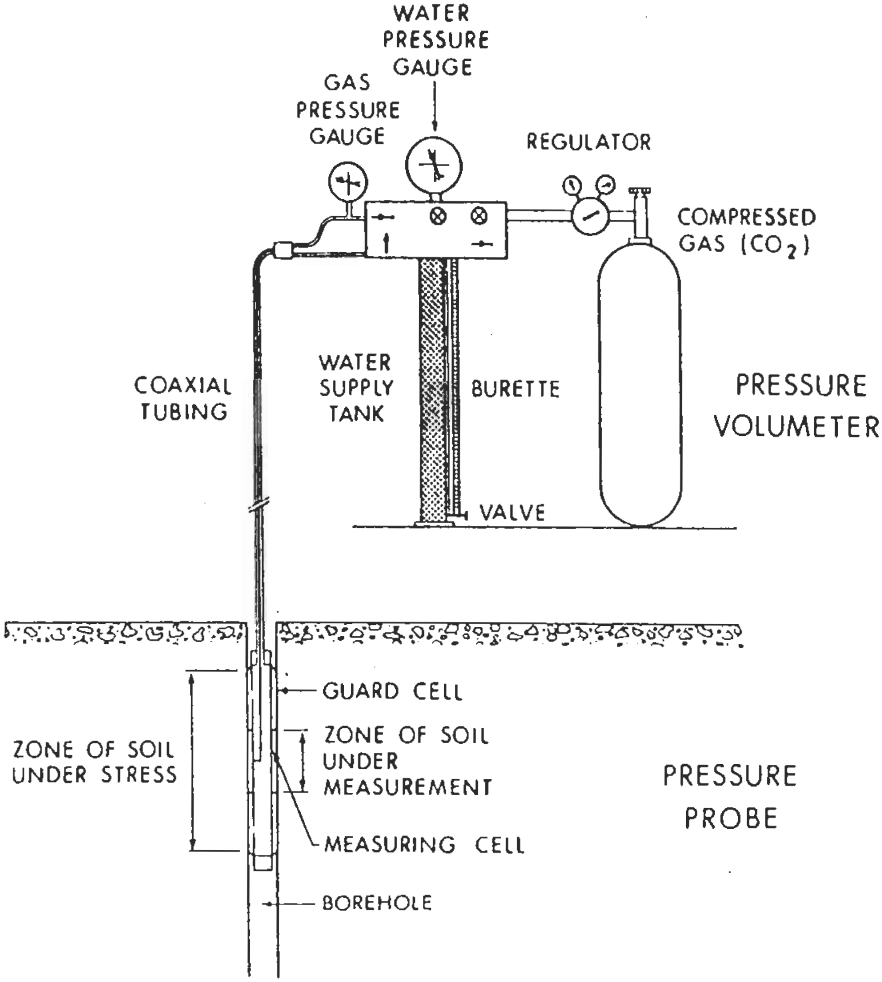

FIG. 2. Schematic view of pressuremeter test setup (Ladanyi 1972).

al. 1978) and only some important points are noted here.

The probe was installed in the ground by lowering it in a borehole, which was pre-drilled by a special hand auger 62 mm diameter supplied by the manufacturer. In order to minimise the time gap between drilling and testing at a particular level, the drilling and testing were undertaken successively at increasing depths. A total of 19 PMTs were conducted at 6 different stations (marked as stations 1 through 6 in Fig. 1) with the number of tests at one station varying from 1 to 5 and the maximum depth of testing being 3.25 m. At the first few stations, the hole diameter was 62 mm from top to bottom. However, on two occasions difficulty was experienced in lifting the probe after the last test in those holes due to partial closing up of the upper part of the hole. This difficulty was avoided in the later part of the testing program by having each of the tests, except the last one, in a hole followed by lifting the probe and increasing of the hole diameter in the upper pre-tested section to 152 mm using an ordinary post-hole auger.

Initially, multiple tests at one station were made in one hole. This did not allow more than two tests in the grey, soft to firm clay layer occurring at the test locations. However, in order to have a more detailed picture of the soil characteristics variation vertically within the top few meters with the PMTs, the later tests were conducted at small depth intervals (roughly equalling to the length of the probe which was 70 cm). For this purpose, two sets of tests were made at a station using a pair of holes 2-3 meters apart horizontally and staggering the test depths in the two holes.

The thicknesses of the top crust and the following soft clay layer (noted earlier) at a station were ascertained by conducting Mackintosh probing and/FVT prior to the PMTs at that location. Attempts were made to have a PMT wholly in a particular layer (soft clay or stiff clay) in order to: i) avoid unequal expansion of the two halves of the probe located within two layers of distinctly differing strengths and, ii) select the appropriate magnitude of the pressure increments (generally 8 to 12 approximately equal sized increments in a test) in a particular layer.

B) Cone Penetration Test (CPT)

A CPT machine, commonly referred to as a Dutch cone of 20 ton capacity owned by the FES and supplied by M/s Goudsche Machinefabrik of Holland was used in this study. It has a mechanical cone with a friction jacket of standard dimensions. The four pickets allowed the frame to be anchored to the ground before the start of CPT at a location.

The CPTs were conducted by following the standard procedure of BSI (1981) with a penetration rate of 2 cm per second. Within every 20 cm of cone penetration, a reading of the resistance of cone plus jacket and another of the cone alone were recorded. This allowed calculation of cone resistance, q_c and friction ratio, R_f for every 20 cm depth.

CPTs were conducted at 7 different stations, marked 1, 3, 4 and 6 through 9, and the maximum depth penetrated was 4 m. Greater depths could not be penetrated due to the limited anchoring capacity of the pickets whose length were insufficient to reach the lower stiff layer.

Results of Pressuremeter Tests

A) Pressuremeter Curves

A conventional pressuremeter curve is drawn by plotting the raw field data of the pressure applied on the wall of the borehole, p against the expansion of the cavity represented by the volume of water injected into the measuring cell of the probe, V_m. Three such curves from tests made within the soft clay and 3 within the stiff clay at different locations are shown in Fig. 3 and 4 (Fig. 3 also includes the calibration curve of the particular probe used). They show the typical shape of the pressuremeter curves. However, it appears from Fig. 4 that the limit pressure, P_l (the pressure cor-

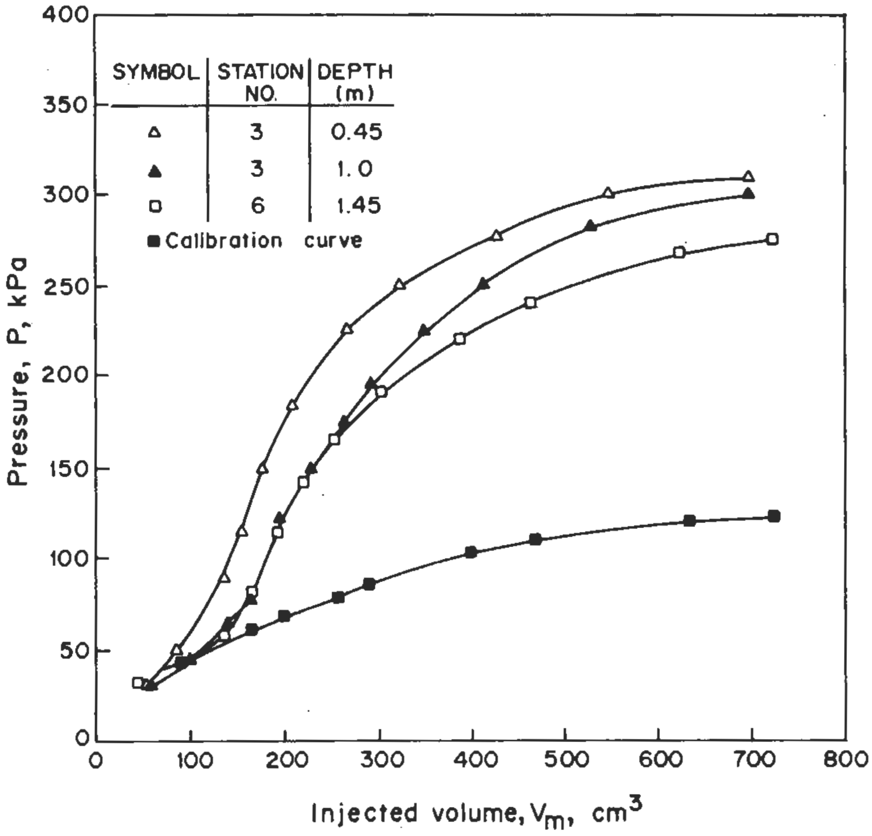

FIG. 3. Typical pressuremeter curves for soft clay.

responding to a volume twice the initial volume of the cavity) was not reached in some of the tests with the maximum injected volume of 725 cm^3 which is the limit of the currently available equipments and it confirms observation of Baguelin *et al.* (1978).

B) Limit Pressure, P_l and Pressuremeter Modulus, E_p

From each of the pressuremeter curves of Fig. 3 and 4 and from others (not presented herein), the usual limit pressure P_l, and pressuremeter modulus, E_p, were determined. Appropriate corrections for (i) the difference in elevation of the pressure gauge (mounted on the C.P.V. panel) and the centre of the measuring cell during a test, and for (ii) the strength of the probe membrane were applied to obtain the corrected limit pressure, P_l. The net limit pressure P_l^* was calculated by subtracting the estimated horizontal earth pressure, p_0, at the test depth. No attempts were made in this study to evaluate the strain-dependent secant moduli of the type discussed by Briaud *et al.* (1983).

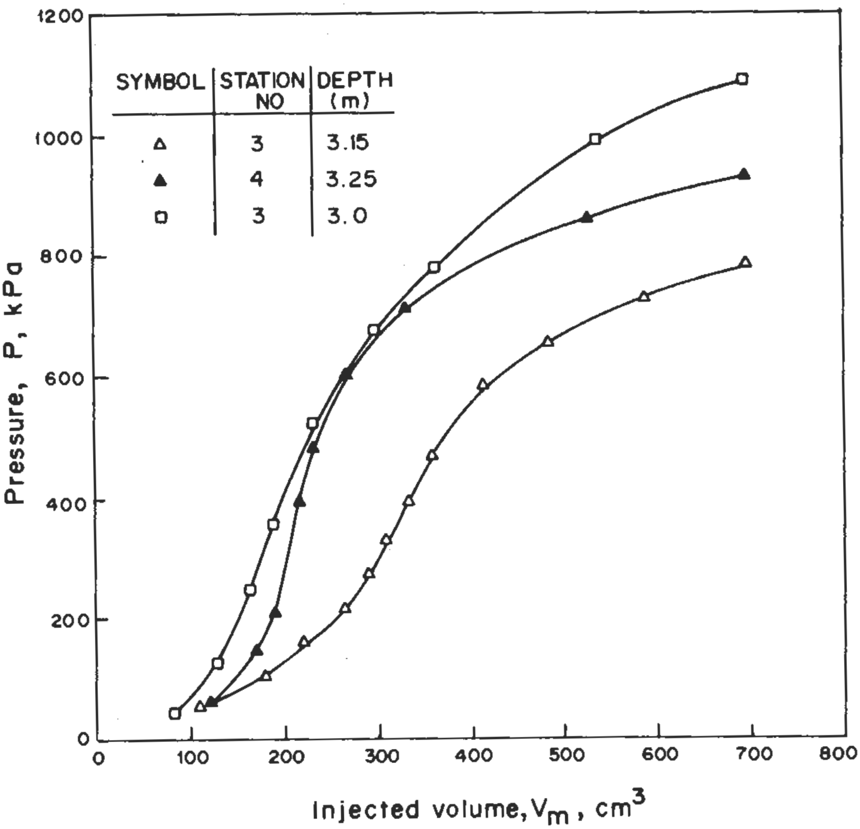

FIG. 4. Typical pressuremeter curves for stiff clay.

The values of E_p and P_l^* determined above are listed in Table 2. It is observed that E_p and P_l^* – values are in the ranges 860-3200 kPa and 92-342 kPa, respectively, for the soft clay, and in the ranges of 2700-11700 kPa and 360-1002 kPa, respectively, for the stiff clay. The E_p values are possibly affected to some extent by the presence of a disturbed annulus and initial unloading discussed, among others, by Sayed and Hamed (1988). However, these effects depend on the thickness of the disturbed annulus as well as on the degree of its change from the original undisturbed state and some disturbance is unavoidable with this type of pressuremeter and attempt at minimizing this was the primary factor in the development of self-boring pressuremeter. Disturbance effects are presumed to be within tolerable range in the present PMTs because the boring was made carefully by the special hand auger, and the time gap between boring and testing at any depth was small (say 10 to 15 minutes).

Figures 5 and 6 show variation of P_l^* with depth at two of the tested locations (*i.e.*, stations 3 and 6) along with the variation of undrained shear strength, s_{uv} determined by FVT and *M*-value from Mackintosh probing. These figures show that PMT, FVT and Mackintosh are all suitable in delineating the transition from the top soft layer to

TABLE 2. Summary of results of PMT, FVT and Mackintosh probing and their correlations.

Clay Type	Station no.	Depth (m)	$s_{u(pm)}$ (kPa)	s_{uv} (kPa)	$s_{u(pm)} / s_{uv}$	E_p (kPa)	P_l^* (kPa)	M blows 10.3m	P_l^* / M	P_l^* / s_{uv}	P_{lr} (kPa)	P_{lr} / s_{uv}
Soft	1	0.80	54	32	1.69	860	122	10	12.2	3.81	155	4.84
		1.80	86	24	3.58	2100	186	34	5.47	7.75	240	10.0
	2	1.00	210	60	3.50	3200	342	39	8.77	5.7	470	7.83
	3	0.45	59	36	1.64	2100	191	30	6.36	5.30	209	5.80
		1.00	71	20	3.55	1550	176	12	14.7	8.8	202	10.1
		1.45	75	28	2.68	1700	174	20	8.7	6.21	240	8.57
		1.92	101	33	3.06	1100	109	22	4.95	3.30	157	4.76
	4	1.5	54	21	2.57	990	93	13	7.15	4.43	129	6.14
	6	0.45	67	32	2.09	2800	127	30	4.23	3.97	139	4.34
		1.00	79	42	1.88	1280	160	18	8.89	3.81	234	5.57
		1.45	62	58	1.07	1680	147	20	7.35	2.53	193	3.33
		2.0	53	17	3.12	970	92	21	4.38	5.41	135	7.94
Stiff	1	2.6	360	> 160		2700	375	142	2.64	2.34	480	3.0
	2	2.0	381	> 160		7400	714	> 300			1080	
		2.7	337	> 160		8500	890	> 300			1200	
	3	3.15	435	140		6000	640	> 220		4.57	800	5.71
	4	3.25	265	> 160		11700	820	>300			1020	
	5	3.0	442	> 160		7200	1002	>300			1370	
	6	2.6	238	140		2100	360	100	3.60	2.57	480	3.43

the underlying stiff layer. However, the correspondence between P_l^* and s_{uv} or M-value at different depths within the soft layer at station 3 is not so good as it is at station 6. As argued by Hossain and Ali (1988), this is considered to be partly due to the raise of some of the s_{uv}-values locally by the interference on the vane rotation by the large gypsum crystals or shells present in these sabkha clays.

B) Estimation of Undrained Shear Strength

Gibson and Anderson (1961) proposed a theory to calculate the undrained shear strength of a saturated clay by using a semi-log plot of pressure against volumetric strain of a PMT and both Palmer (1972) and Ladanyi (1972) suggested modification of the method. In this modified version, the applied pressures and volume changes in a PMT are recalculated by referring to p_0 and the corresponding injected volume, V_{m0}, respectively. This corrected pressure, p_c is plotted against $\ln(\Delta V / V)$, $(\Delta V / V)$ being the volumetric strain. The greatest slope of such a plot gives the undrained shear strength designated here as $s_u(pm)$. Thus, the corrected pressure and volumetric strains are given by

$$p_c = p_{c_1} - p_0 \text{ and} \tag{1}$$

$$\Delta V / V = \text{the current volumetric strain} \tag{2}$$

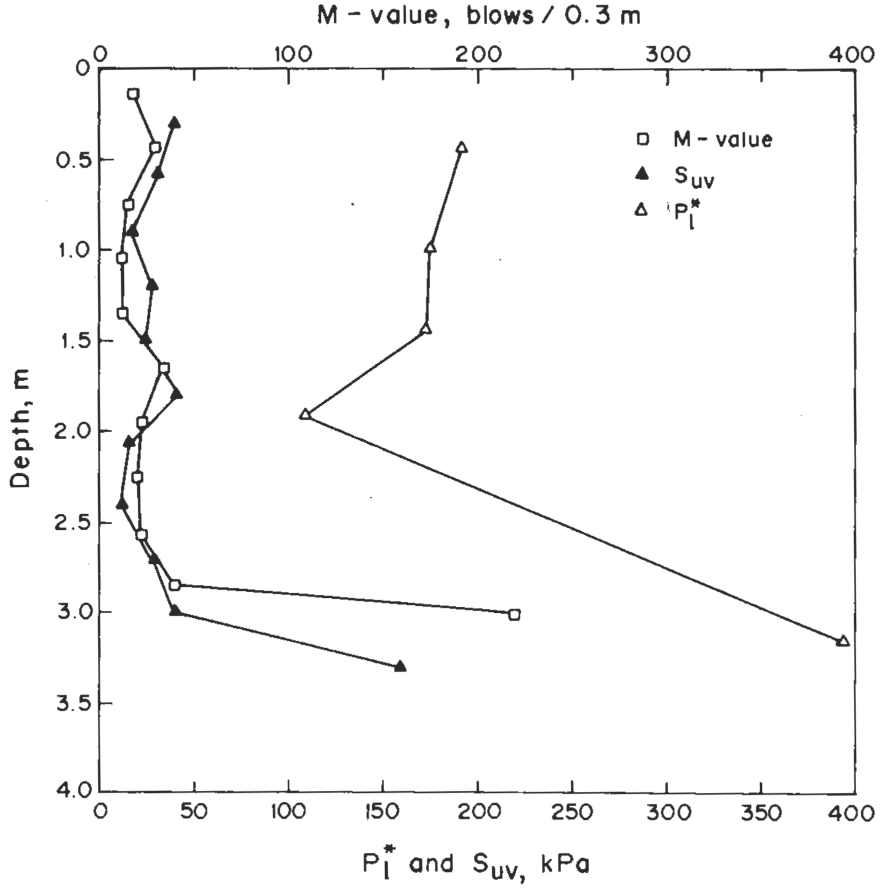

FIG. 5. Variation of P_l^*, s_{uv} and M-value with depth at station 3.

in which

p_{c1} = applied pressure corrected for membrane resistance and for the difference in elevation between the centre of measuring cell and the pressure gauge,

$$V = V_c + V_m$$

$$\Delta V = V_m - V_{m0}$$

where

V_m = the total volume of water injected into the measuring cell from the start of pressure application.

V_{m0} = the volume of water injected into the probe at $p_{c1} = p_0$

V_c = calibration volume of the probe (= 535 cm^3 for the authors' probe)

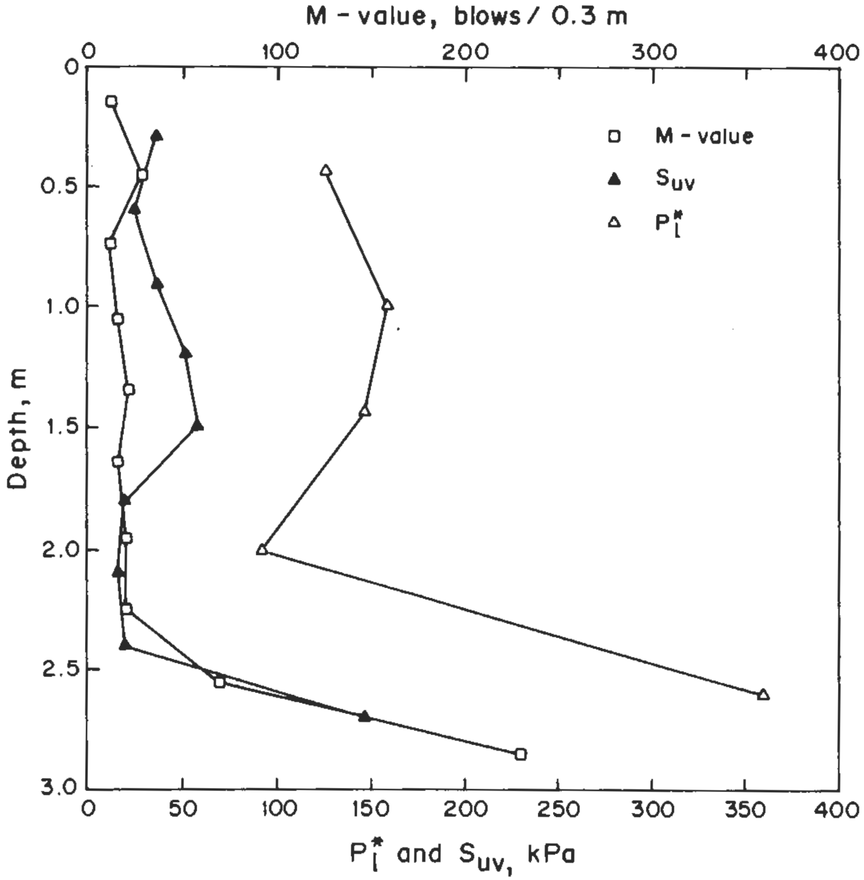

FIG. 6. Variation of P_l^*, s_{uv} and M-value with depth at station 6.

Plots of the above form were made for all the authors' tests and some of these for the soft clay and the stiff clay are presented in Fig. 7 and 8, respectively. The curves so obtained are mainly of two types. The curves of one group have the characteristic S-shape suggesting a drop in shear stress after a peak while those of the other group end in almost straight lines indicating absence of a peak. One example of an unusual shape is obtained from the test at depth 0.45 m at station 6 (Fig. 7) which suggests that there was a thin layer stronger than the rest of the soil mass, and this yielded half-way during the test so that further expansion was resisted by the softer material. This is the case at some locations of this sabkha where a thin highly salt encrusted layer occurs within the top half meter depth.

The values of $s_u(pm)$ obtained by the above method are listed on Table 2 along with the measured s_{uv} values. It is observed that, in common with the the findings of other investigators (*e.g.*, Windle and Wroth 1977, Battaglio *et al.* 1981 for other clays, the $s_u(pm)$ values for the soft clay are consistently higher than the correspond-

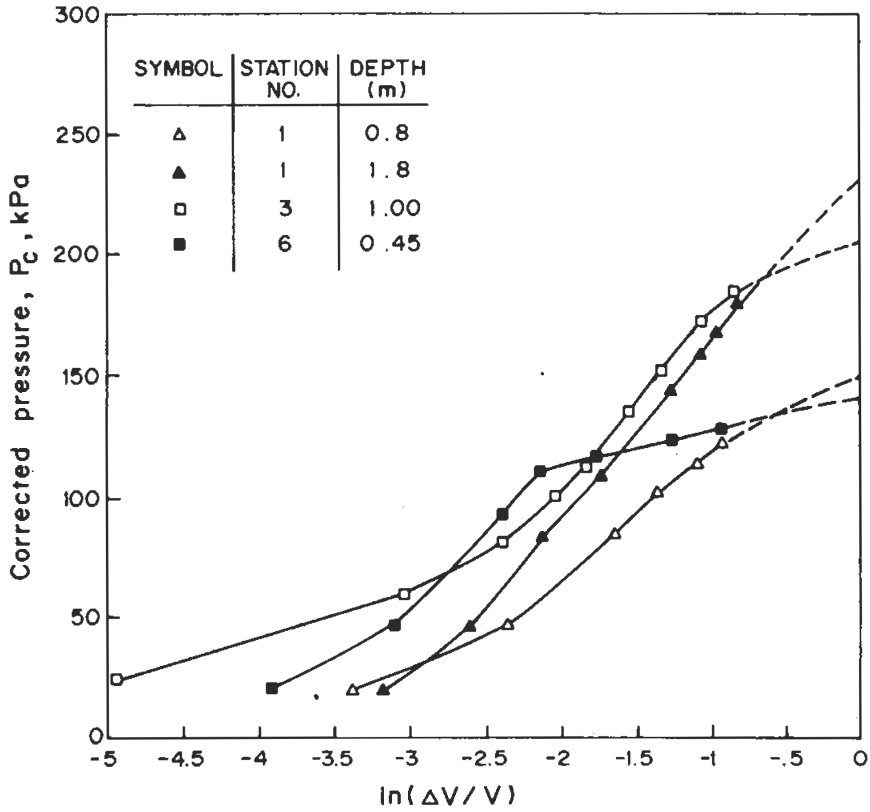

FIG. 7. Typical p_c vs. ln($\Delta V/V$) plots for soft clay.

ing s_{uv} values leading to $s_u(pm)/s_{uv}$ values in the range of 1.07 to 3.58 with a mean value of 2.54 and a standard deviation of about 0.82 as shown in Table 3. As explained by other researchers (*e.g.*, Baguelin *et al.* 1978), this difference is due to disturbance during boring prior to testing. Similar correlation could not be attempted for the stiff clay layer because definite s_{uv} values (which were mostly greater than 160 kPa which was the capacity of the vane used) in that layer was unknown.

C) Shear Strength-Limit Pressure Correlation

Baguelin *et al.* (1978) points out that the application of the theories of Bishop *et al.* (1945), Hill (1950), and Salencon (1966) to PMT leads to a relation between the undrained shear strength, s_u of a clay to its net limit pressure, P_l^* of the form

$$s_u = P_l^* / B \tag{3}$$

where

B = a constant whose value is estimated to be in the range of 5.2 to 7.5.

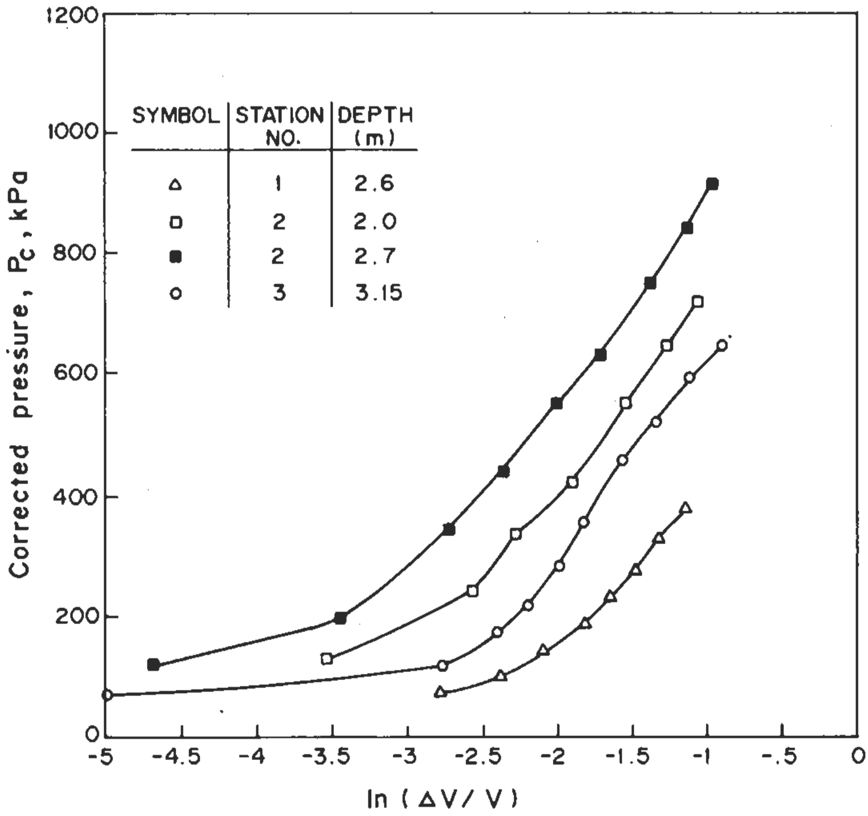

FIG. 8. Typical p_c vs. $\ln(\Delta V/V)$ plots for stiff clay.

TABLE 3. Ranges of correlation factors between the results of CPT, PMT, FVT and Mackintosh probing in Obhor Sabkha.

Factor	No. of data	Range	Mean	Standard deviation	Clay type
$s_{u(pm)}/s_{uv}$	12	1.07 - 3.58	2.54	0.82	Soft
P_l^*/M	12	4.23 - 14.7	7.76	3.04	Soft
P_l^*/s_{uv}	12	2.53 - 8.80	5.09	1.76	Soft
P_{le}/s_{uv}	12	3.33 - 10.1	6.60	2.14	Soft
s_{uv}/M	12	0.70 - 3.20	1.66	0.74	Soft
P_{le}/P_l^*	19	1.09 - 1.51	1.33	0.11	(Soft + stiff)
E_p/P_l^*	19	5.83 - 14.3	9.54	1.95	(Soft + stiff)
N_k (i)	32	4.30 - 31.0	15.3	7.2	Soft
(ii)	12	10 - 34.0	22.6	8.4	Stiff

The P_l^* values of the 12 PMTs within the soft clay are plotted against the corresponding s_{uv} values in Fig. 9. The value of B ($= P_l^*/s_{uv}$) obtained from this figure are in the range of 2.53 to 8.8 with an average of 5.09 and a standard deviation of 1.76. The relatively lower B-values are considered to be due to some of the measured s_{uv} values raised through interference on the FVT by the gypsum crystals and shells. Such interference effects would be smaller on the P_l^* value than on the s_{uv} because the thickness of soil stressed by the 70 cm long pressuremeter is much greater than that stressed by a 13 cm long vane and also possibly due to the difference in mode of failure in the two tests. Although both FVT and PMT are quasi-static in nature, the failure surface in case of FVT is predetermined, whereas it is not so in case of PMT. However, the observed range of the B-values is quite encouraging and proves the suitability of PMT for sabkha clay investigation and dependability of the resulting limit pressures.

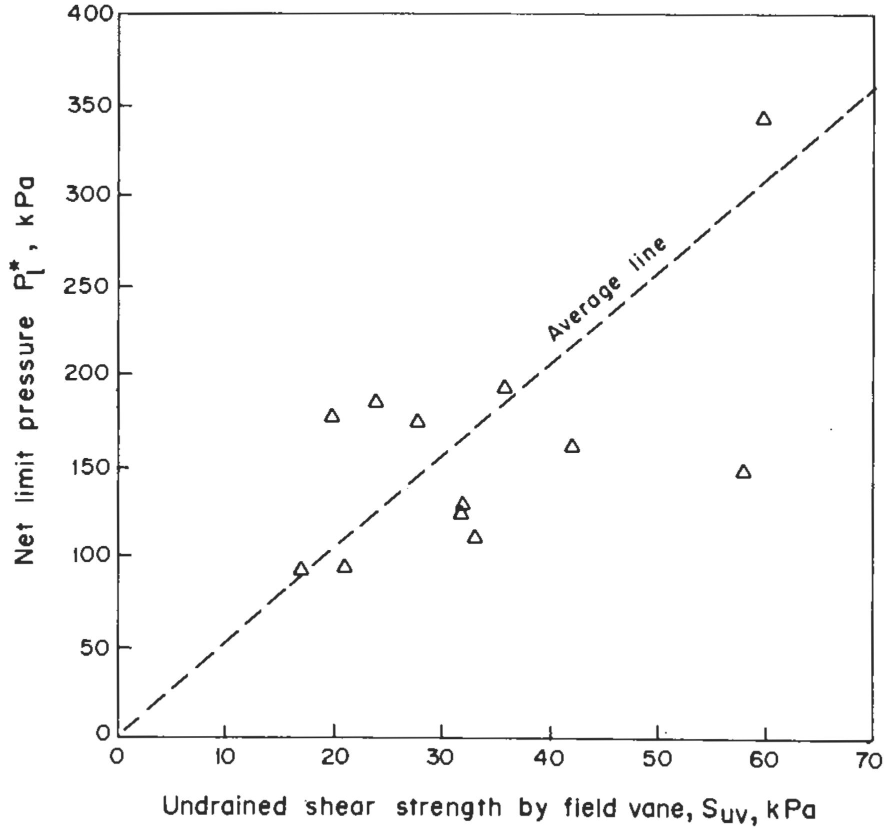

FIG. 9. Net limit pressure (P_l^*) vs. undrained shear strength (s_{uv}) plots for soft clay.

D) M-Value Limit Pressure Correlation

There is no published relation of M-value of Mackintosh (*i.e.*, blows/0.3 m of penetration) with E_p or P_l from a PMT for any soil. Figure 10 shows a plot of P_l^* (in kPa) against the corresponding M-value for the Obhor soft clay layer. This suggests that the P_l^*/M ratio varies from 4.23 to 14.7 (10 of the 12 values being in the range 4.23 to 8.89) with a mean of 7.76 and a standard deviation of 3.04 as listed on Table 3. Considering the fact that Mackintosh is a light weight dynamic tool of small diameter whose M-values may be affected more than P_l^* of PMT by local non-homogeneity due to the presence of large crystals and shells, these ratios are considered encouraging.

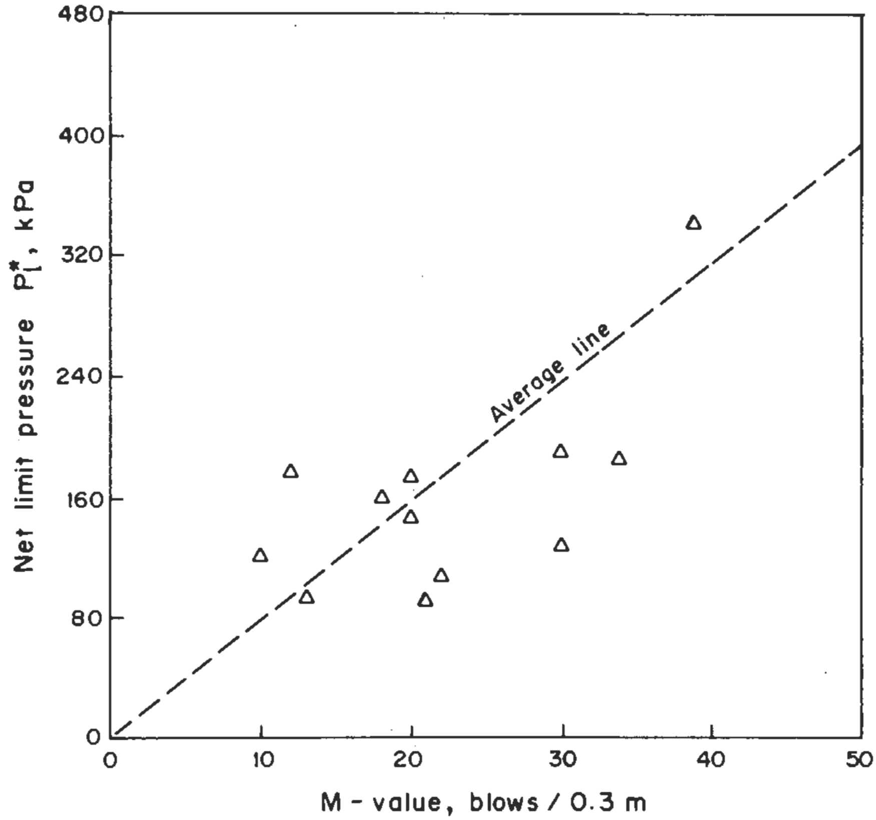

FIG. 10. Net limit pressure (P_l^*) vs. M-value plots for soft clay.

E) E_p/P_l^* Relation

Foundation settlement calculations by the Menard (1975) method based on PMT results require an estimate of the ratio E_p/P_l^*, and its range varies with the consistency of the clays. A plot of E_p against P_l^* for both the soft clay and the stiff clay of the studied sabkha together in Fig. 11 suggests E_p/P_l^* ratio in the range of 5.83 to 14.3 with a mean of 9.54 and a standard deviation of 1.95 and the ranges for the two clays are found to be overlapping.

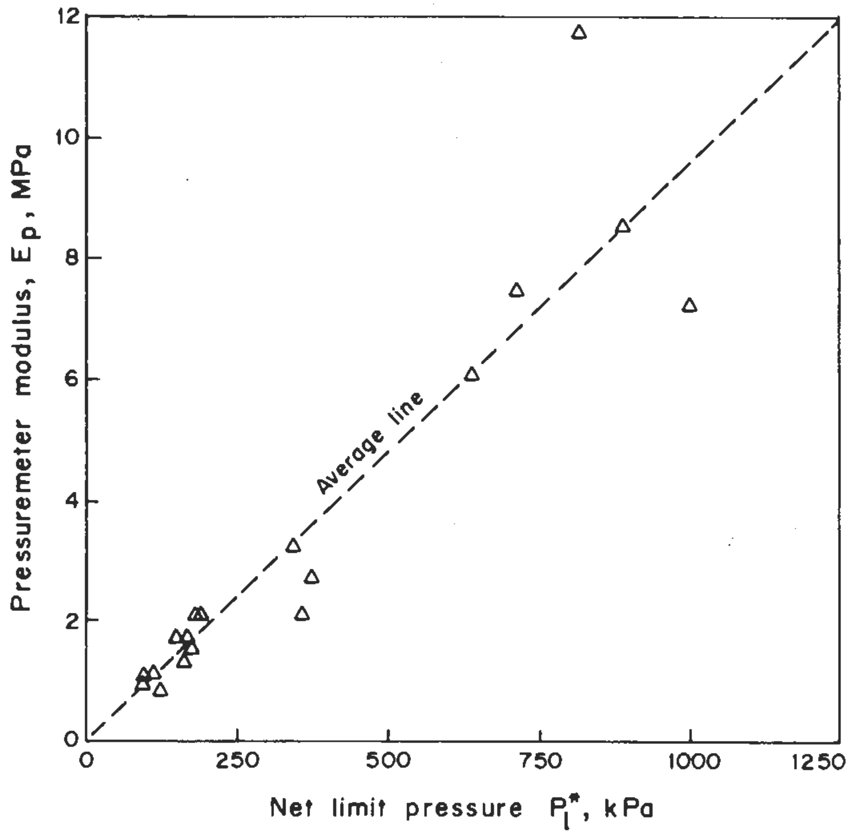

FIG. 11. Pressuremeter modulus (E_p) vs. net limit pressure (P_l) plot for soft and stiff clays.

F) Extrapolated Limited Pressure

Considering that the pressuremeter used in this investigation did not allow continuing the PMTs to reveal the real P_l^* values, the most probable values of P_l^* designated simply as P_{le} were extrapolated from the p_c vs. ln $(\Delta V/V)$ plots (*e.g.*, Fig. 7 and 8) following the method of Ladanyi (1972). These values are listed on Table 2 and compared with P_l^* obtained earlier. It is observed that the P_{le}/P_l^* ratios for both the soft and the stiff clay are in the range 1.09 to 1.51 with a mean of 1.33 and a standard deviation of 0.11 (Table 3).

CPT Results

A) Cone Resistance Variation with Depth

Cone penetration tests provide data to calculate the cone resistance, q_c, and jacket friction q_f of the penetrated soil at successive depths. From this q_c, the net cone resistance $(q_c\text{-}P_{v0})$, in which p_{v0} is the total *in situ* vertical pressure at the test depth, can be evaluated. Figures 12 and 13 show the variations of net cone resistance (q_c-p_{v0}) from CPT and s_{uv} from FVT with depth at two stations within the studied clayey sabkha. It is observed that both the tests reveal the end of the soft layer as well as that of the slightly stronger crust at the top. Some local fluctuations in both (q_c-p_{v0}) and s_{uv} observed with the soft clay at station 3 (Fig. 13) is again considered to be due to the interference of the gypsum crystals and shells noted earlier and partly due to the usual random variations in strength of the clays.

B) Undrained Shear Strength from CPT

Undrained shear strength was calculated as $q_c/14$ as per Begemann (1965) for the authors' CPTs and this is designated as s_{uc}. Figure 14 shows typical variations of s_{uc} and jacket friction q_f (also from CPT) with depth along with that of s_{uv}.

It is observed that s_{uv} is mostly between q_f and s_{uc} (with $q_f < s_{uc}$). This is in line with the observation of Schmertmann (1975) who suggests taking q_f as the lower boundary of undrained shear strength.

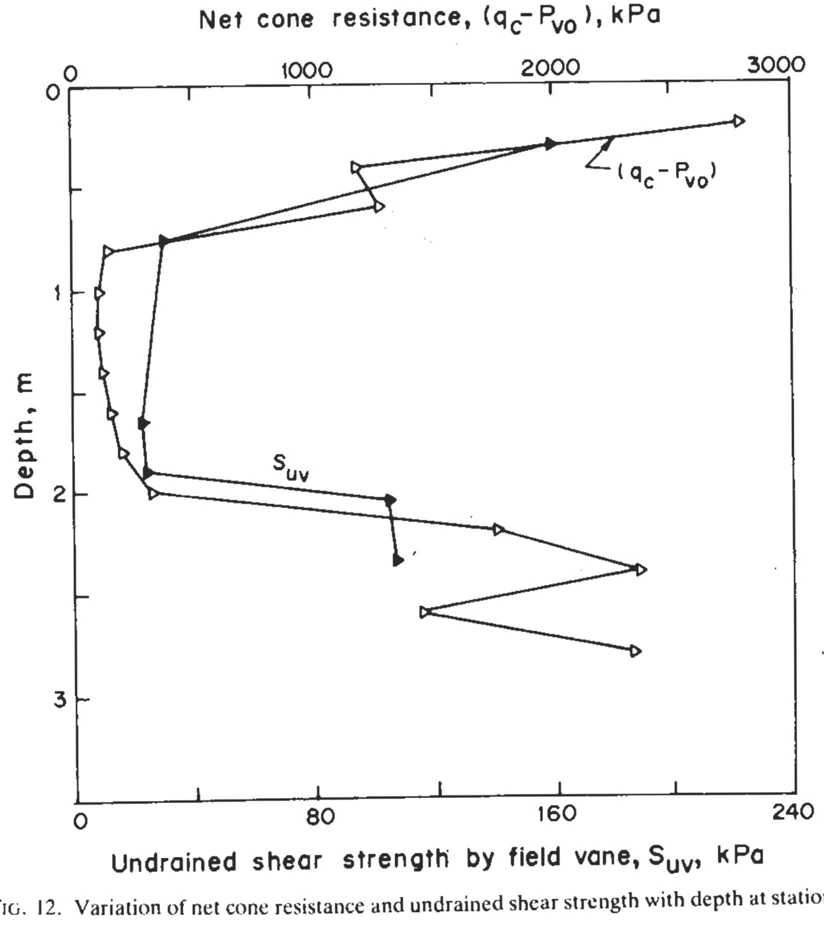

FIG. 12. Variation of net cone resistance and undrained shear strength with depth at station 1.

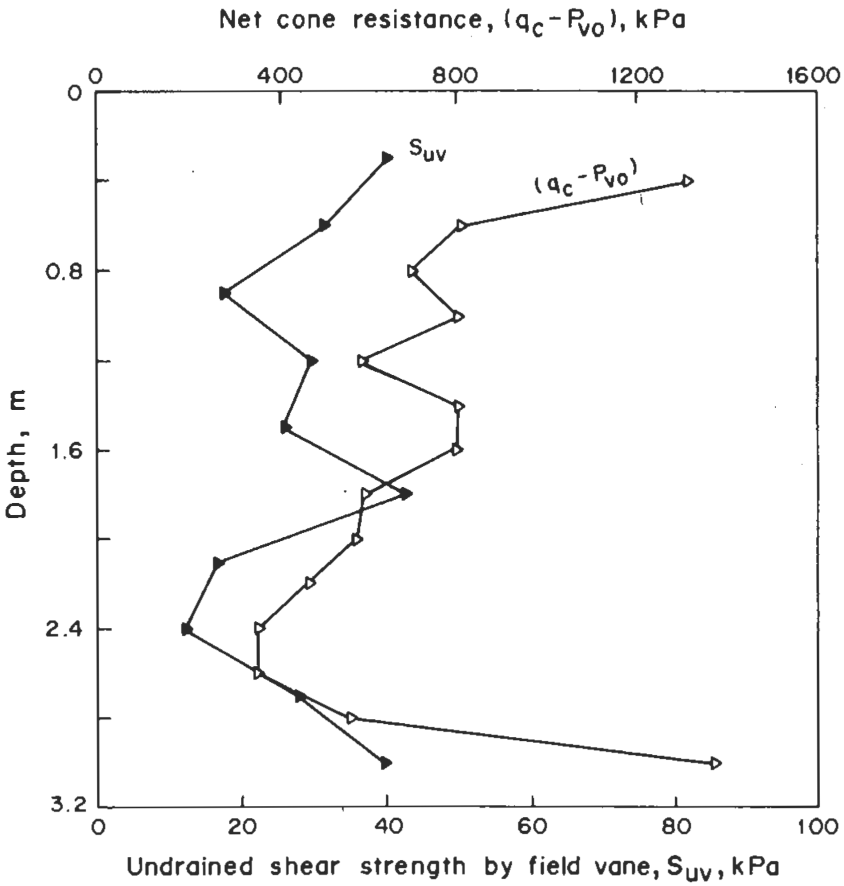

FIG. 13. Variation of net cone resistance and undrained shear strength with depth at station 3.

In order to examine the range of variation of the correlation between q_c and s_{uv}, the values of the net cone resistance (q_c-p_{v0}), are plotted against the corresponding s_{uv} in two groups, namely, (i) the soft clay excluding the top crust, and (ii) stiff clay plus the crust in Fig. 15 and 16. They suggest that the values of cone factor, N_k (= $(q_c - p_{v0})/s_{uv}$) for soft clay are in the range of 4.30 to 31 with a mean value of 15.3 and a standard deviation of 7.2 and those for the other group are in the range of 10 to 34 with a mean value of 22.6 and a standard deviation of 8.4. Although there is appreciable scatter of the data points, specially in case of the soft clay, the relatively higher mean value of N_k for the stiff clay is in line with observations elsewhere. For example, De Ruiter (1982) noted typical N_k values in the range of 10-15 for normally consolidated clays and 15-20 for overconsolidated clays, but concluded that there is no universal value for all types of clays. In case of Obhor Sabkha, Hossain and Ali (1988) showed that the soft clay is normally consolidated to lightly overconsolidated, and the stiff clay and the crust are overconsolidated, and hence the pattern of the N_k values are in order (although slightly on the higher side).

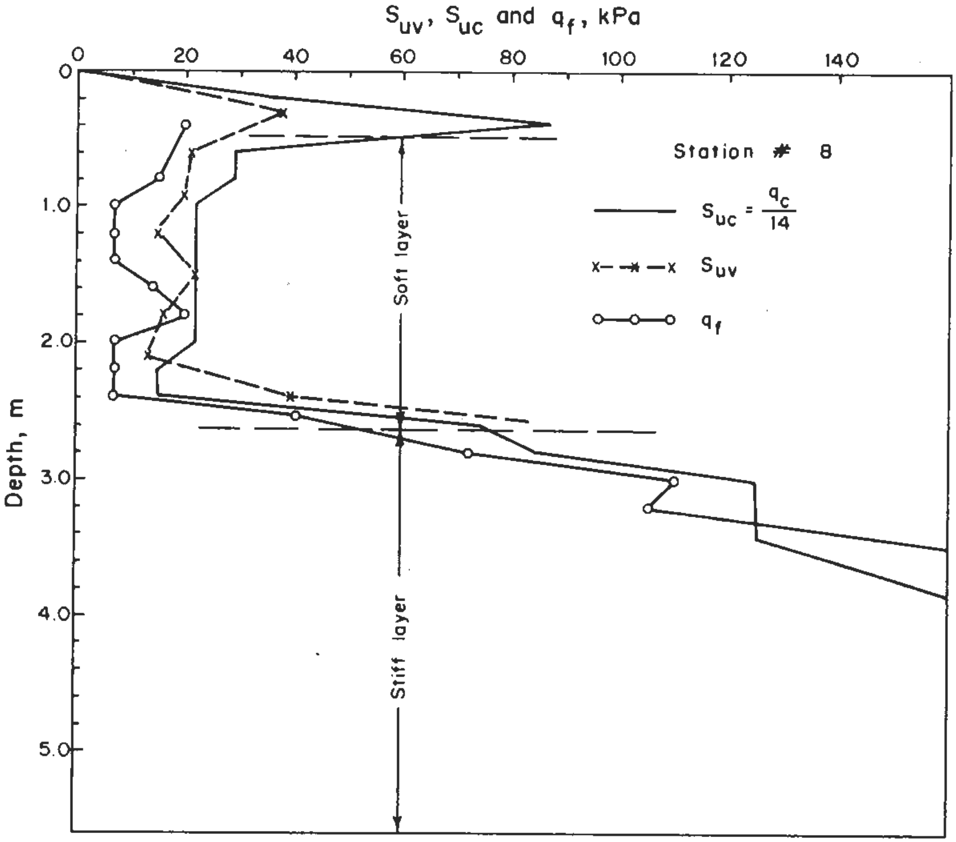

FIG. 14. Comparative variation of undrained shear strengths from field vane shear test and static cone penetration test.

Foundation Design Based on CPT and PMT Parameters

Calculations were made for the presumed bearing value (and hence net allowable soil pressure) and settlement for a few lightly loaded column footings at the tested locations using the P_l^* and E_p and following the method outlined by Baguelin *et al.* (1978) and also alternative ones using the s_{uv} from FVT and settlement parameters from laboratory consolidation tests made on undisturbed sabkha clays. Some data on the consolidation characteristics of Obhor sabkha clays are reported by Hossain and Ali (1988) and are summarized in Table 1. It is observed from the present calculations that while the presumed bearing values from the two methods are comparable, the settlements from PMT parameters are generally smaller than those based on consolidation test results. Table 4 illustrates this for square footings of two sizes founded on the soft clay at a depth of 0.5 m.

Similar calculations based on CPT results and following the method outlined by Balasubramaniam and Brenner (1977) showed that settlements so obtained are

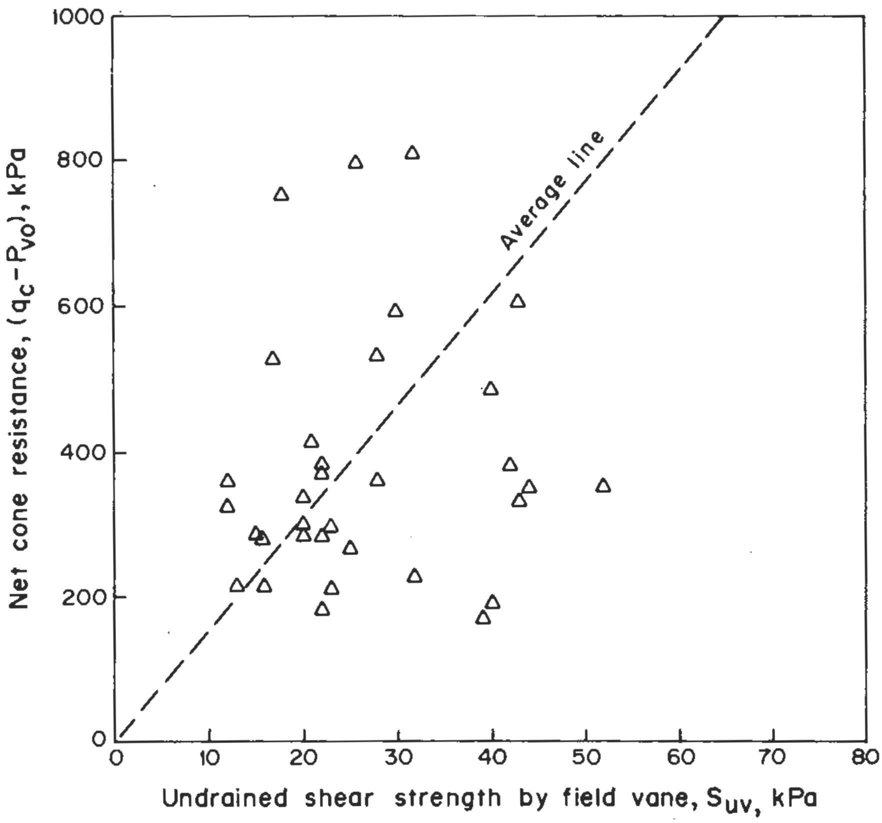

FIG. 15. Net cone resistance vs. undrained shear strength plot for soft clay.

TABLE 4. Typical foundation design summary.

Station	Footing		Presumed bearing value (kPa) based on		Settlement (cm) based on	
	Width	Depth				
	(m)	(m)	PMT	S_{uv}	PMT	Consolidation test
3	1.5	0.5	60	44	1.41	5.6
3	2.0	0.5	58	40	1.82	6.7

smaller than those based on consolidation test results. For example, the settlement predicted using the CPT results for a square footing 2.5 m wide at a depth of 0.6 m at station 3 and supporting a column load of 300 kN is about 3.9 cm compared to 8.4 cm obtained by using the consolidation data. Relatively greater disturbance during undisturbed sampling and specimen preparation for consolidation testing (compared to *in situ* tests like PMT or CPT) is considered to be the cause of such differences.

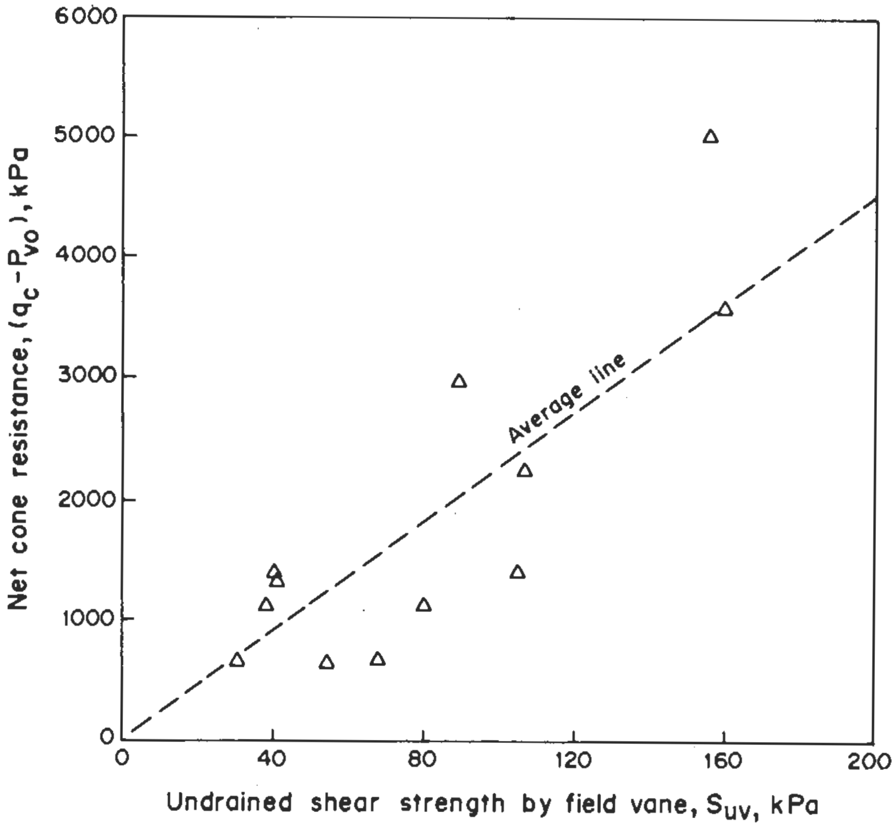

FIG. 16. Net cone resistance vs. undrained shear strength plot for stiff clay.

Discussion

The present study involving the use of pressuremeter test and static cone penetration test in a clayey sabkha is the first of its kind and it forms the initial stage of a continuing research at King Abdulaziz University. The experience gathered so far shows that both the above tests are technically feasible in site investigation in clayey sabkha areas. However, some careful planning and preparation before starting the field work and care during the actual testing are considered necessary. For example, the CPT machine as well as the towing vehicle may run the risk of getting stuck if taken to a weak sabkha area, especially after a rain. Longer pickets are necessary to provide the required anchoring if the lower stiff clay is to be penetrated to substantial depths. In case of PMT, it is better to use a membrane of lower stiffness for testing the soft clay layer, and a stronger one for the stiff clay. Questions may be raised about the possible presence of a "critical depth" in case of a PMT at a small depth such as the one at 0.45 m included in the study. However, Briaud and Shields (1981) conducted PMTs at depths smaller than this and did not observe any "critical depth" for clays.

It is considered that additional CPTs and PMTs in Obhor and other sabkhas could increase confidence of the practising engineers in the use of these tests. Further, some plate loading tests or loading of instrumented footings at the locations of the CPTs and PMTs could shed more light on the differences observed in the settlements calculated by the different methods.

Conclusion

Based on the investigations reported herein the following conclusions are drawn:

1) Both pressuremeter test (PMT) and static cone penetration test (CPT) can be used in investigating a clayey sabkha, provided safe access is ensured.

2) Both PMT and CPT successfully delineate the transition between two soil layers of appreciably different strengths in the same way as it is possible with field vane shear tests (FVTs) or Mackintosh probing.

3) Reasonable correlations are obtained between the undrained shear strength from FVT and net limit pressure from PMT and cone resistance from CPT.

4) The undrained shear strength of soft sabkha clay obtained by applying Palmer (1972) and Ladanyi (1972) method to PMT data exceeds that from PVT due to disturbance.

5) For equal loads on lightly loaded column footings of similar sizes and depths, settlements calculated using PMT or CPT parameters are generally smaller than those based on laboratory consolidation test results.

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فحوص جهاز قياس الضغط وجهاز غرز المخروط الساكن في سبخة أبحر بالمملكة العربية السعودية

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المستخلص . تُبرز هذه الدراسة نتائج ١٩ فحصًا لجهاز قياس الضغط في ستة مواقع في الجزء الطيني لسبخة أبحر . وبمقارنة هذه النتائج مع نتائج جهازي القص المروحي وجس ماكنتوش ، نجد أن نتائج جهازي قياس الضغط وغرز المخروط الساكن مناسبة وبنفس الدرجة للاختبارات الحقلية . واستنادًا إلى هذه النتائج ، فقد تم اقتراح علاقات مؤقتة بين مختلف معايير التصميم . كما أوضحت الحسابات أن الهبوط المستحصل من نتائج هذه الأجهزة أقل بشكل عام من نتائج فحص الاندماجية لنفس الأحمال ونفس أحجام القواعد في المنشآت الخفيفة .