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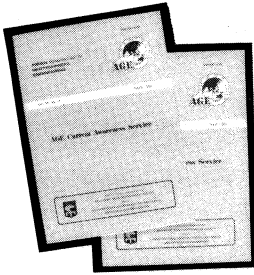
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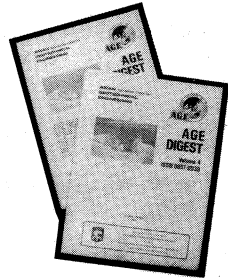
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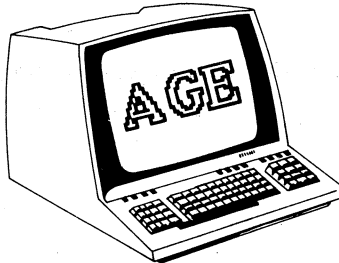
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CORRELATIONS OF INSITU TESTS IN BANGKOK SUBSOILS

**D.T. BERGADO¹, M.A. KHALEQUE²,
R. NEEYAPAN² & C.C. CHANG²**

SYNOPSIS

Six types of insitu tests were performed in the Bangkok subsoils, namely: LLT Pressuremeter, Field Vane, Screw Plate, Plate Load, Dutch Cone, and Standard Penetration tests. In the laboratory, a series of Direct Shear tests were performed on sand samples, and another series of One-Dimensional Consolidation tests was performed on clay samples to obtain the preconsolidation pressure. The sampling and field tests were carried out on the campus of the Asian Institute of Technology (AIT) wherein the four uppermost subsoil layers consist of about 2 m weathered clay, followed by about 6 m of soft clay, underlain by about 5 m of stiff clay and about 10 m of dense to very dense sand. The average laboratory and insitu soil parameters obtained were correlated. The correlations obtained have been verified, compared, and discussed in relation to earlier findings.

INTRODUCTION

It has been found that the action of sampling causes significant disturbance to soil, due to both mechanical deformation and the inevitable difference in stress history between a sampled element of the soil and a similar element in the field. Certain soils in the natural state cannot be sampled satisfactorily or prepared properly for laboratory tests regardless of the care exercised. A growing awareness of these problems has led to an increasing interest in all forms of insitu testing where the disturbance of the soil structure is minimized. For this purpose, a variety of empirical testing procedures have been created like the standard penetration test, the vane shear test, the plate load test, the Dutch cone test, and, recently, the screw plate test and the pressuremeter test.

The purpose of this study is to establish correlations between the soil parameters obtained from LLT pressuremeter tests and screw plate tests with those obtained from the conventional insitu tests, namely: plate load, vane shear, Dutch cone, and standard penetration tests. For weathered clay, soft clay, and stiff clay layers, a program of insitu testing was performed at four sites on the Asian Institute of Technology (AIT) campus, namely: near the

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ET Building, near the Football Field, near Korea House, and near the New Zealand Housing Area. The subsoil profile as shown in Figure 1 consists of about a 2 m thick weathered clay layer, underlain by about 6 m of soft marine clay, followed by about 5 m of stiff clay, and about 10 m of dense to very dense sand below the stiff clay layer.

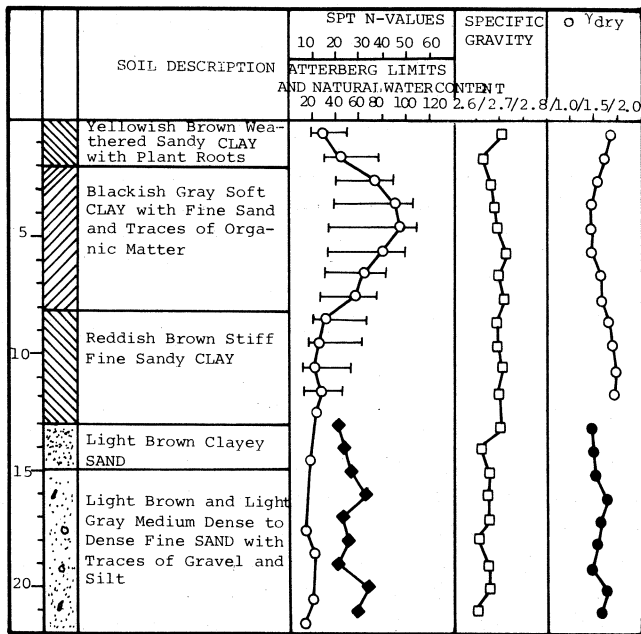


Fig. 1. Soil Profile and Properties at AIT Campus.

At each site, pressuremeter tests, vane shear tests, and Dutch cone tests were done in five locations about 10 m apart (Figure 2a). Another series of insitu tests involving screw plate tests, plate load tests and pressuremeter tests were carried out in three places (near the ET Building, near the Football

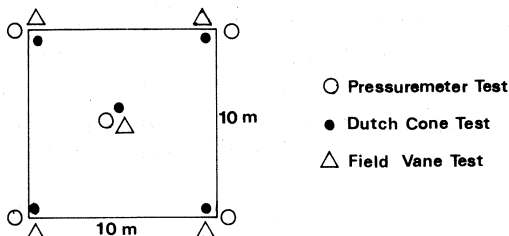


Fig. 2a. Location of Test Holes for the Weathered, Soft and Stiff Clay Layers.

INSITU TESTS

Location of Tests

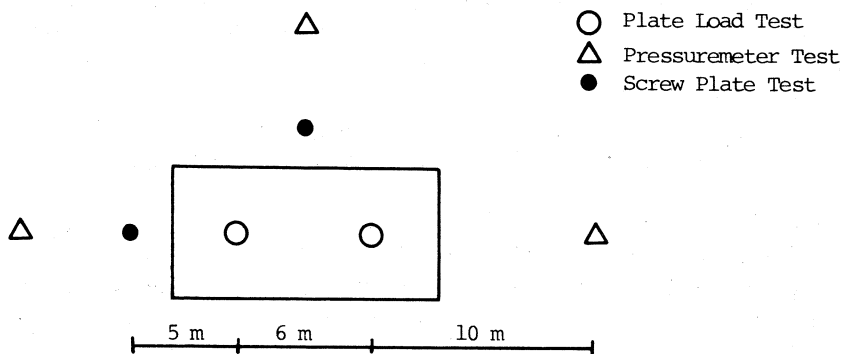


Fig. 2b. Location of Tests for Weathered and Soft Clay Layers.

Field and near Korea House). At each site, the pressuremeter tests were done in three locations while both the screw plate and plate load tests were done in two locations (Figure 2b). For the stiff clay and dense sand layers, the pressuremeter, Dutch cone, and standard penetration tests were performed also at four sites, namely: near the ET Building, near the Football Field, near Korea House, and near the Tennis Courts. At these sites, the Dutch cone test was done in five locations while the pressuremeter and standard penetration tests were done in three locations (Figure 2c). In addition, index tests were done on both clay and sand samples, a series of direct shear tests on sand samples was undertaken, and one-dimensional consolidation tests were done on clay samples.

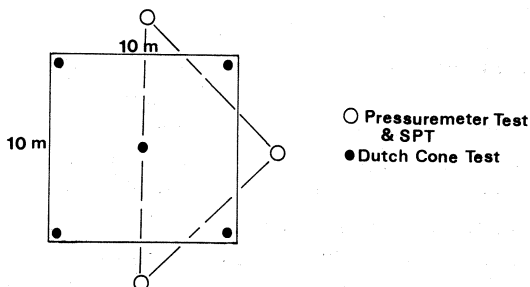


Fig. 2c. Location of Test Holes for the Stiff Clay and Dense Sand Layers.

Most of the data discussed in this paper was obtained from the Master's Thesis research of KHALEQUE (1984), NEEYAPAN (1985) and CHANG

(1985) under the guidance of the first author. The results of the insitu tests in the weathered, soft and stiff clay layers in the Bangkok subsoils will be presented by BERGADO & KHALEQUE (1986) emphasizing the pressuremeter test results. The results from the stiff clay and sand layer insitu tests are presented by BERGADO et al (1986).

THE OYO PRESSUREMETER AND METHOD OF TEST

Since 1956, when the first pressuremeter was introduced by MENARD (1956), the pressuremeter has been widely used. In Japan, the Oyo Corporation produced its first pressuremeter in 1966 which was called the Lateral Load Tester (LLT). The LLT pressuremeter is based on the same principle as the Menard pressuremeter. The only difference between the two devices is that the LLT is a monocell device with a large measuring cell while Menard's pressuremeter operates with three cells, a smaller measuring cell and two guard cells in both ends. OHYA et al (1982) have found that both pressuremeters are alike in terms of accuracy in the resulting soil parameters and that the overall expansion pattern of the monocell probe is closer to the cylindrical pattern. However, the monocell is more reliable because of the large contact area with the surrounding soil (YOSHII, 1979; TANAKA, 1979). In fact, CAPELLE (1982) has presented new types of monocell pressuremeters and concluded that they are not only simple but also economical to use.

The LLT pressuremeter probe used in this study was a model 4165 type M, with cell diameter of 70 mm and length of 60 cm. A detailed diagram of the instrument is given in Figure 3. Although the monocell has a larger measuring cell than the tricell pressuremeter, they may be thought of as nearly alike for practical purposes (OHYA et al, 1982).

Three different methods were used initially to advance the borehole in the pressuremeter test (HUANG, 1980), namely: hand augering, thin-walled tube, and wash boring methods. It was found that the thin-walled tube method yielded the best results and that the hand augering method yielded poor results. However, in a subsequent work by SURYA (1981), the wash boring method proved to be economical and efficient, producing good results especially with respect to the value of the deformation modulus, E_p , even in the soft clay layer. Using the wash boring method, the time required for advancing the borehole is considerably reduced since the borehole can be kept open by the drilling mud pressure. In this method, the hole is drilled by means of a special core barrel with wings (Figure 4) to minimise disturbance of the borehole wall during the circulation of drilling mud.

INSITU TESTS

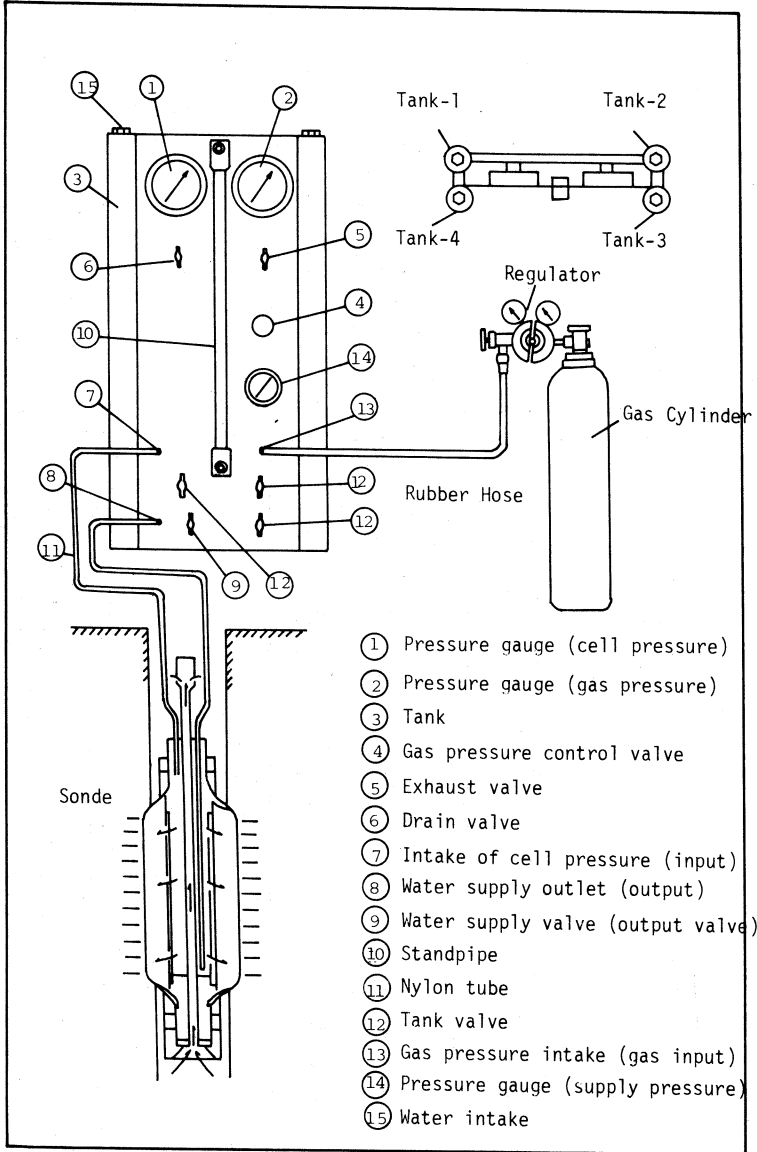


Fig. 3. Schematic Diagram of LLT Pressuremeter (after HUANG, 1980).

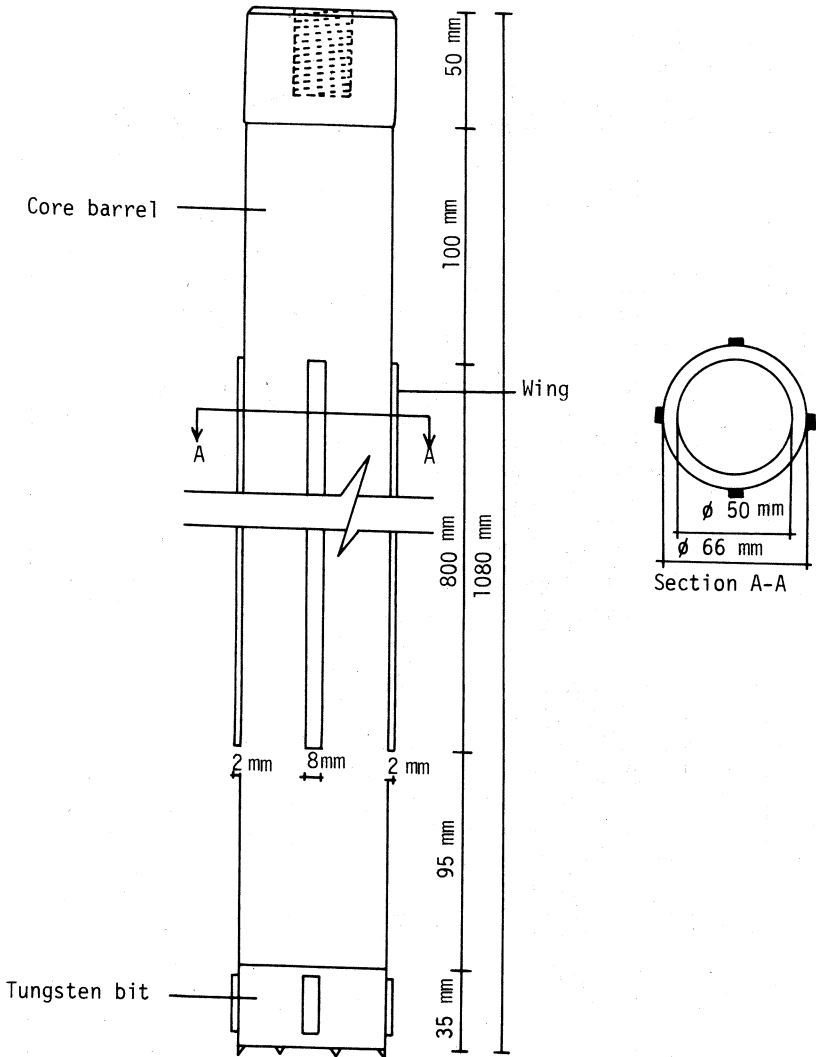


Fig. 4. Core Barrel with Wings.

INSITU TESTS

PARAMETERS OBTAINED FROM THE PRESSUREMETER TESTS

Typical results from the pressuremeter test are plotted as effective pressure (P_e) versus radius (r) of the probe, and as effective pressure versus deformation velocity (ΔH) curves as shown in Figure 5. These curves directly yield three parameters: horizontal earth pressure at rest (P_o), yield pressure (P_y), and limit pressure (P_l). The effective pressure is the cell pressure corrected for rubber reaction and hydrostatic pressure. The deformation velocity is the ratio of deformation to time during the time interval of 30 s to 120 s after loading at every loading stage. This is the deformation that goes on with the lapse of time at a particular pressure due to the viscoelastic behavior of the ground. Utilizing the theories developed for cavity expansion in an ideal elastoplastic material (GIBSON & ANDERSON, 1961), the undrained strength (S_{up}), the angle of internal friction ($\phi'p$), and the undrained Young's modulus (E_{up}) can be computed from the pressuremeter curves.

Horizontal Pressure At Rest (P_o)

P_o is the pressure which corresponds to a state at which the probe is in contact with the borehole wall and is keeping balance with the static earth pressure in the ground. In the P_e vs. r curve, the starting point of the pseudo-elastic zone represents P_o (Figure 5). A plot of P_o with depth for all boreholes in the weathered clay, soft clay, stiff clay, and dense to very dense sand layers is given in Figure 6. Each point in this plot is the average at that depth at each site. Starting from 1.5 m, the P_o values increased with depth at 3 different rates of increment, corresponding to each distinguished layer in the clay. CHANG (1985) obtained similar results in a subsequent investigation. In the sand layer, the P_o values were found to be scattered.

Yield Pressure (P_y)

The pressure corresponding to a point at the end of the linear portion of the P_e - r curve is called the yield pressure (P_y). At this point the soil moves from the elastic to plastic states. Figure 7 shows the averaged values of P_y with depth for the weathered clay, soft clay, stiff clay, and dense to very dense sand layers. In the topmost weathered clay layer, the values slightly decreased with depth. The P_y values are approximately constant in the soft clay layer while the values increased with depth in the stiff clay and dense sand layers.

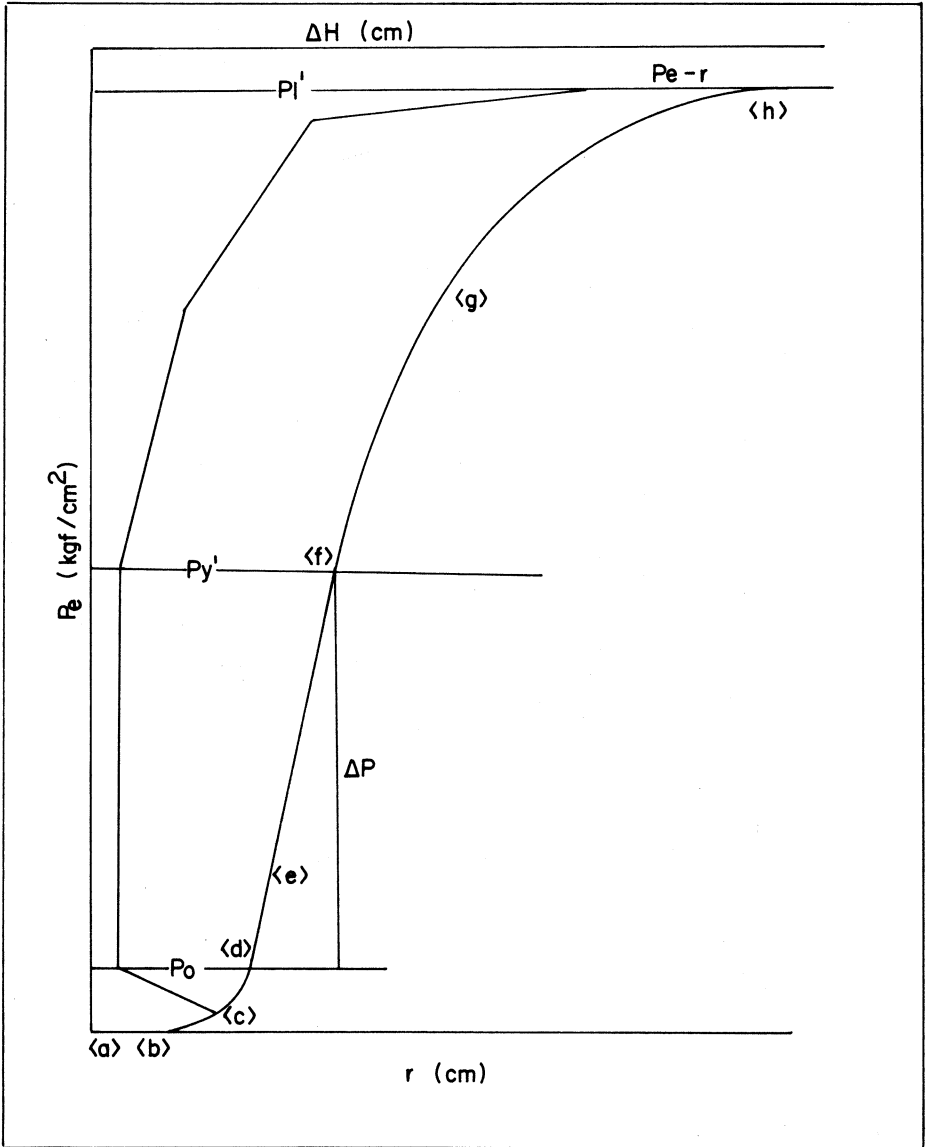


Fig. 5. Schematic Diagram of LLT Test Results.

INSITU TESTS

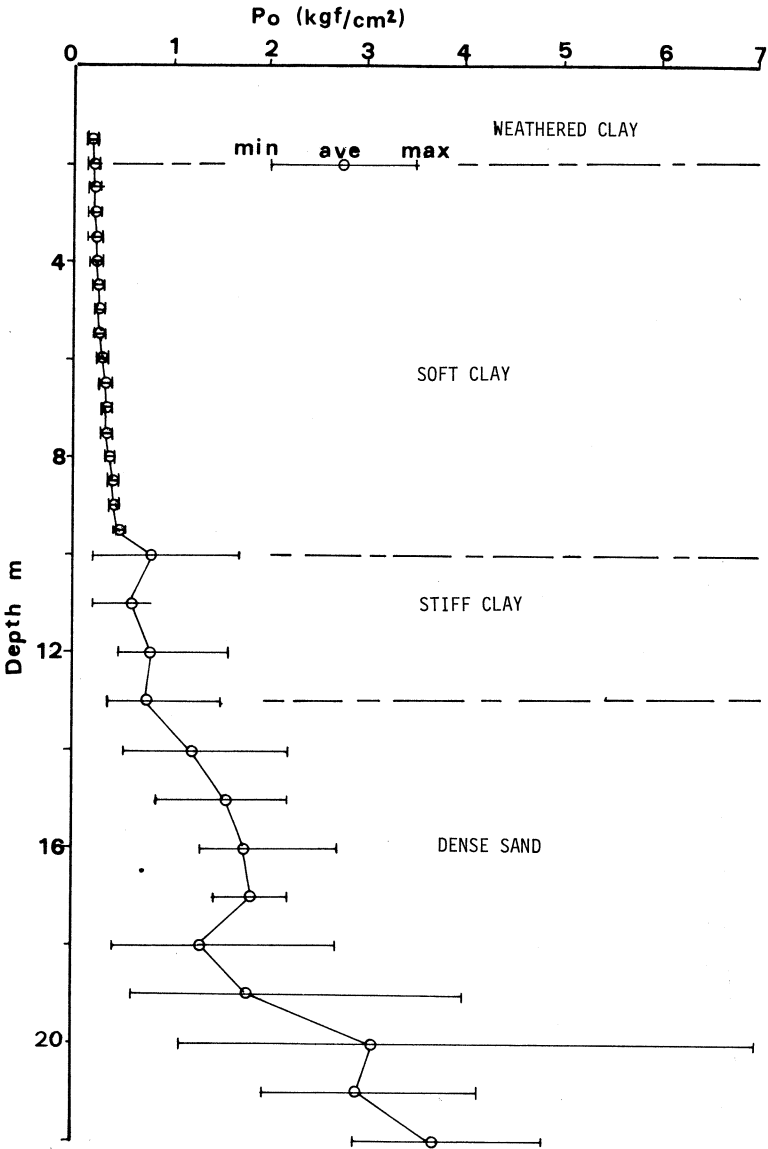


Fig. 6. Variation of Horizontal Earth Pressure at Rest with Depth.

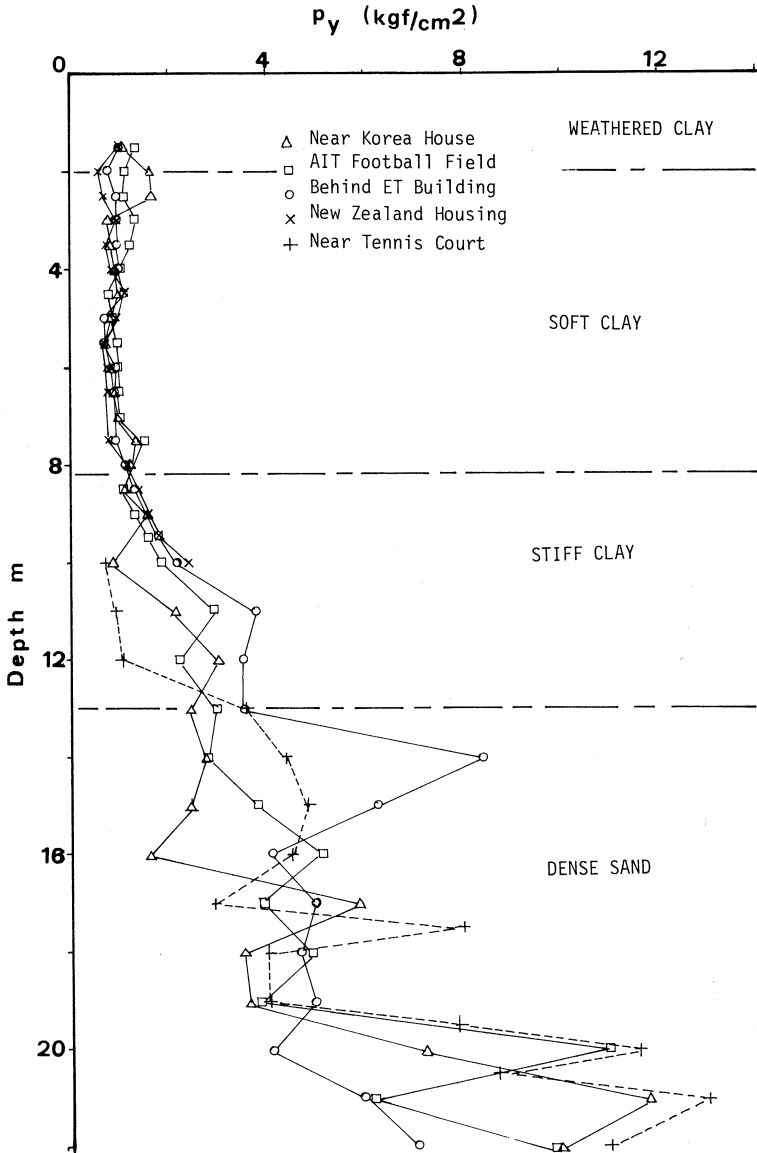


Fig. 7. Variation of Yield Pressure with Depth.

INSITU TESTS

Limit Pressure (P_l)

The pressure at which the cavity has doubled its initial volume is defined as the limit pressure (P_l). In some cases, the test cannot be carried out to obtain the limit pressure due to the limitation in the volume of water in the reservoir and also because of the risk of bursting or damaging the tubing and the probe. For this condition, the pressuremeter curve has to be extrapolated (GIBSON & ANDERSON, 1961) to estimate the limit pressure. Figure 8 shows the averaged P_l^* values as plotted with depth for the weathered clay, soft clay, stiff clay, and dense sand layers. The P_l^* values, equal to P_l minus P_o , were found to follow the same trend as the P_y values with depth but at about twice the magnitude.

Undrained Shear Strength (S_{up})

In this study, the calculation method of GIBSON & ANDERSON (1961) of obtaining the pressuremeter undrained strength (S_{up}) for the clay layers was used. In this method, the undrained shear strength of the cohesive soil is calculated based on the ideal elasto-plastic behavior of the soil as follows:

$$S_{up} = \frac{P_l - P_o}{1 + \ln \left[\frac{E_{up}}{2 S_{up} (1 + \nu)} \right]} \quad \dots\dots\dots (1)$$

where the terms have been defined previously. The Poisson's ratio (ν) was taken as 0.50 corresponding to the undrained, incompressible case. This method yielded reasonable values of undrained strengths, close to the values obtained by other field tests (HUANG, 1980; CHANG, 1985; WROTH & WINDLE, 1975). Other methods of calculating S_{up} resulted in much higher values (PALMER, 1972; LADANYI, 1972). The S_{up} values are plotted in Figure 9 as averaged values with depth for the weathered, soft, and stiff clay layers. It can be observed that in the soft clay layer the S_{up} values were in the range of 3.0 to 4.5 t/m². In the stiff clay the strengths increased with depth. Figure 13 shows the mean curve from the S_{up} data at all four sites compared to S_{uv} data.

Angle of Internal Friction (ϕ'_p)

The effective angle of internal friction (ϕ'_p) was computed using the method suggested by CENTRE D'ETUDES MENARD (1970) as follows:

$$P_l - P_o = 2.5 \left(\frac{\phi'_p - 24}{4} \right) \quad \dots\dots\dots (2)$$

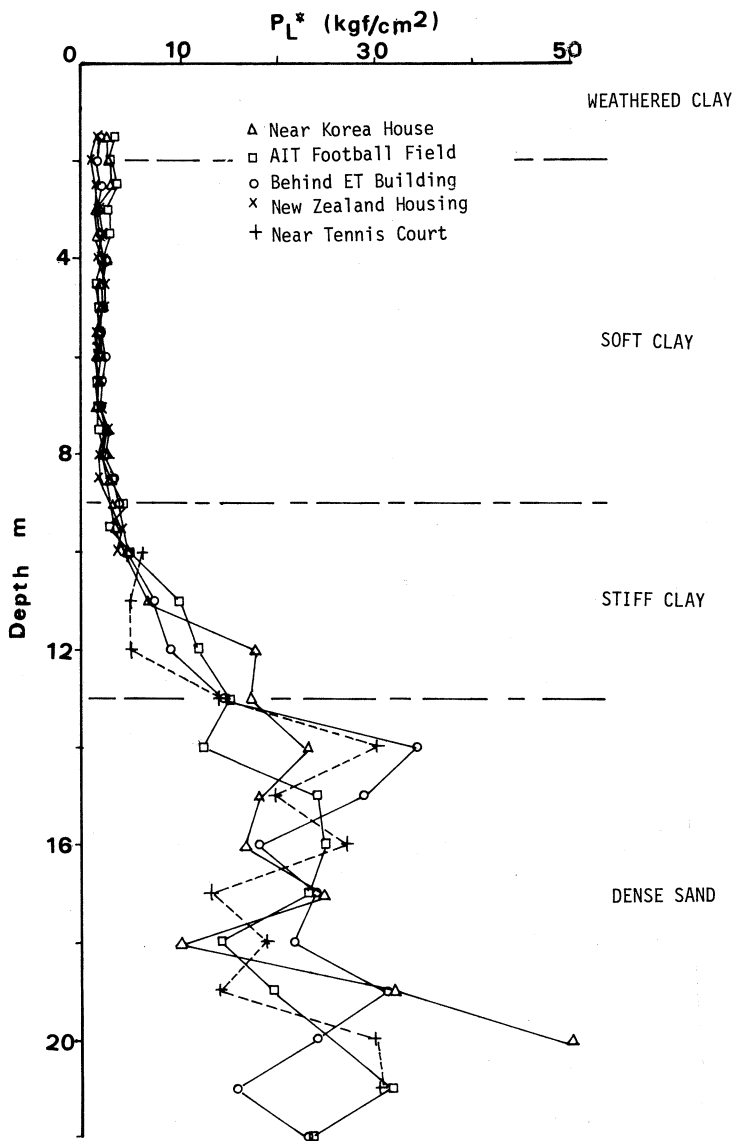


Fig. 8. Variation of Net Limit Pressure with Depth.

INSITU TESTS

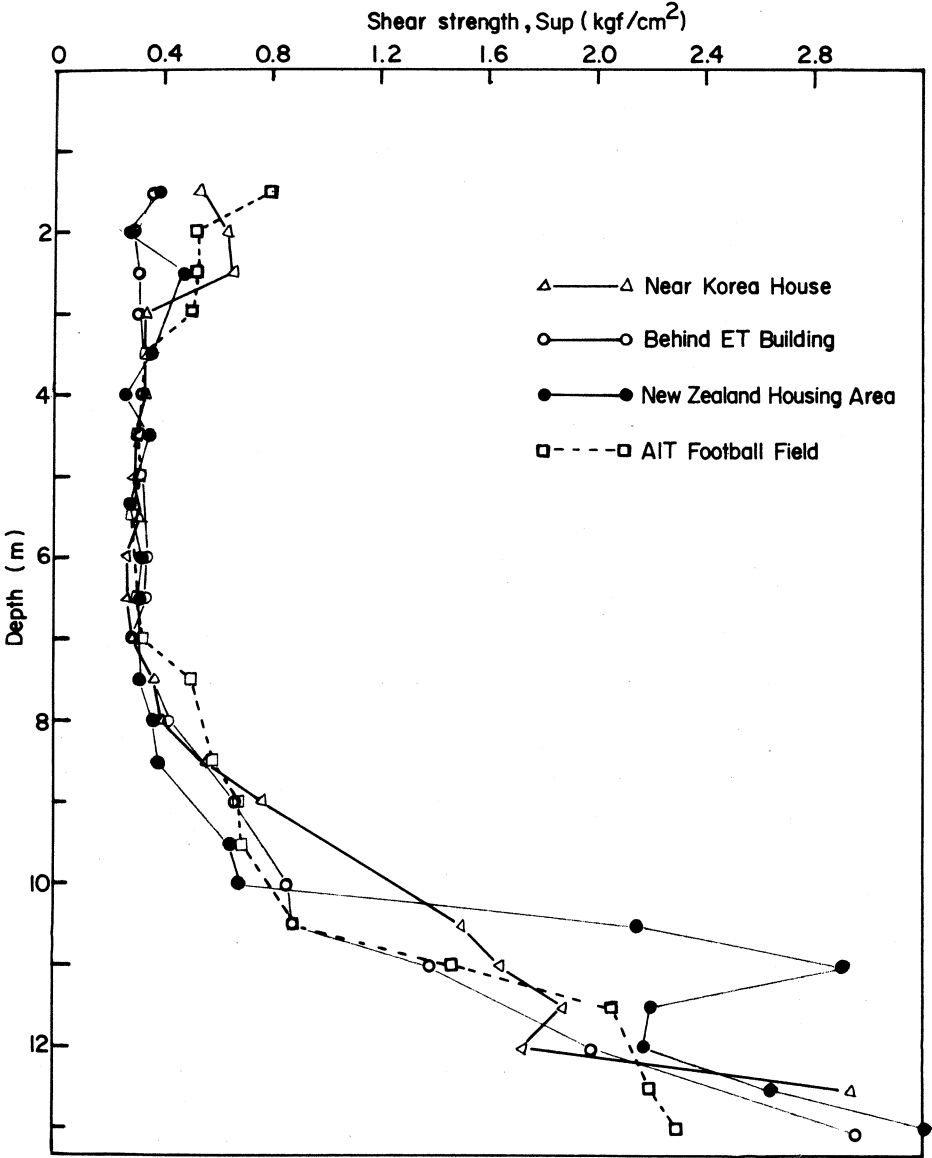


Fig. 9. Variation in Shear Strength with Depth Obtained from Pressuremeter Test.

which is given in a graphical form. The method suggested by GIBSON & ANDERSON (1961) was found to be not applicable for the data in this study. The average ϕ'_p values obtained from Equation 2 are plotted in Figure 10 together with the average ϕ'_d values from the laboratory direct shear test. The ϕ'_p values ranged from 30° to 38°.

Undrained Young's Modulus (Eup) and Pressuremeter Modulus (Ep)

The undrained Young's modulus (Eup) has been calculated as follows: (GIBSON & ANDERSON, 1961):

$$E_{up} = \frac{2\Delta P (1 + \nu)}{(\Delta V/V_o)} \dots\dots\dots (3)$$

- where $\Delta P = P_2 - P_1$ (linear part of the pressuremeter curve)
- $V_o =$ volume corresponding to P_1
- $V =$ volume corresponding to P_2
- $\Delta V = V - V_o.$

The calculated values of Eup are plotted in Figure 11. The values are scattered since this parameter is quite sensitive to the disturbance of the borehole wall (BAGUELIN et al, 1972; MORI, 1981). In the soft clay, Eup varied between 20 kg/cm² and 60 kg/cm². These values have been confirmed by CHANG (1985). The Poisson's ratio (ν) was taken as 0.50 corresponding to the undrained, incompressible case. For the stiff clay and sand layers the pressuremeter modulus (Ep) was computed using a Poisson's ratio (ν) of 0.33 using Equation 4 (BAGUELIN et al, 1978).

$$E_p = (1 + \nu) (K_m) (r_m) \dots\dots\dots (4)$$

- where $K_m = \frac{\Delta P}{\Delta r}$ (linear zone)
- $r_m =$ mean radius (linear zone).

Figure 11 also plots the pressuremeter modulus values for the stiff clay layer and the sand layer. The values of Ep in the stiff clay varied between 40 kg/cm² and 230 kg/cm². In the sand layer Ep varied between 50 kg/cm² and 375 kg/cm².

INSITU TESTS

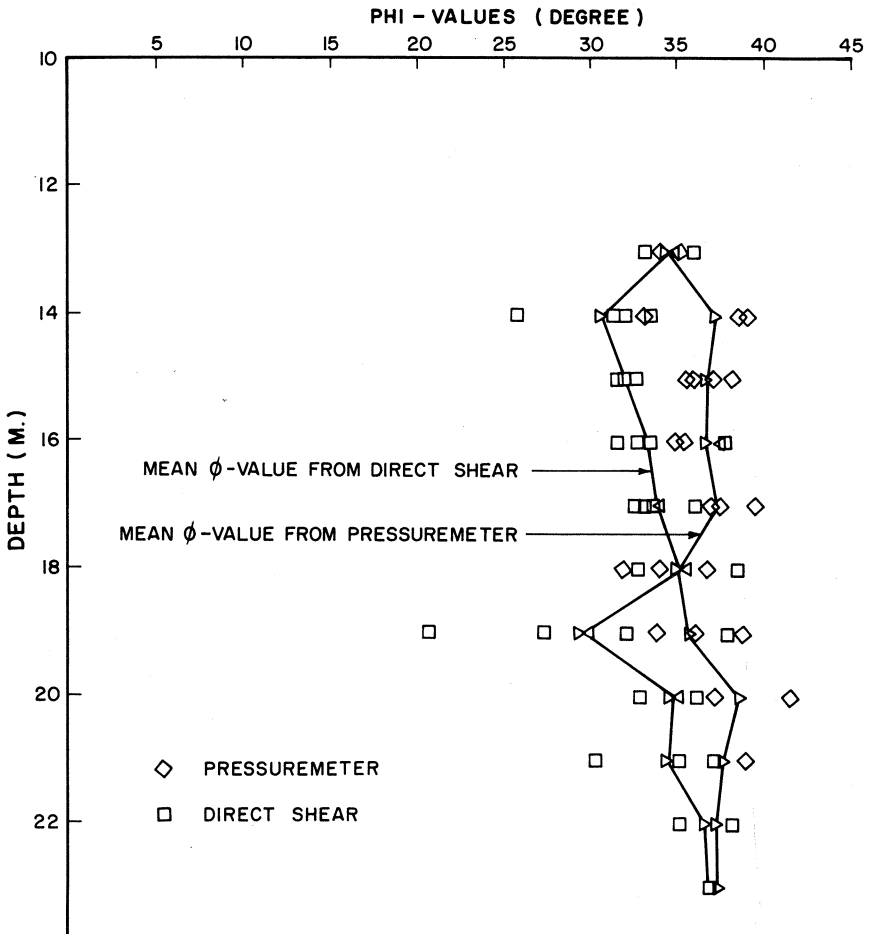


Fig. 10. Friction Angles with Depth from Pressuremeter and Direct Shear Tests.

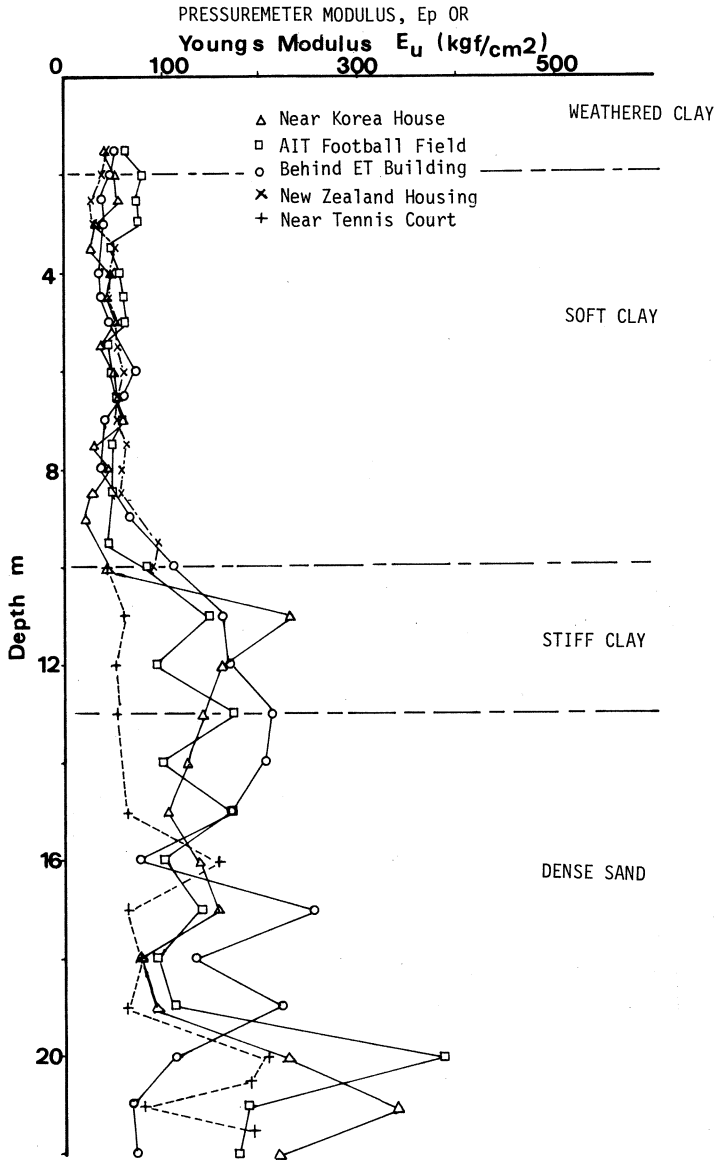


Fig. 11. Variation of Soil Modulus with Depth.

INSITU TESTS

THE FIELD VANE TEST

A Geonor vane apparatus with blade dimensions of 130 mm high and 65 mm wide was used. In performing the test, an assembly of the apparatus with the blade inside a protective shoe was pressed vertically down into the ground 50 cm above the testing depth. The vane was then pushed out of the shoe and rotated at a strain rate of 6 degrees per minute up to the maximum torque. Measurements of the vane strengths were made at 0.50 m depth intervals.

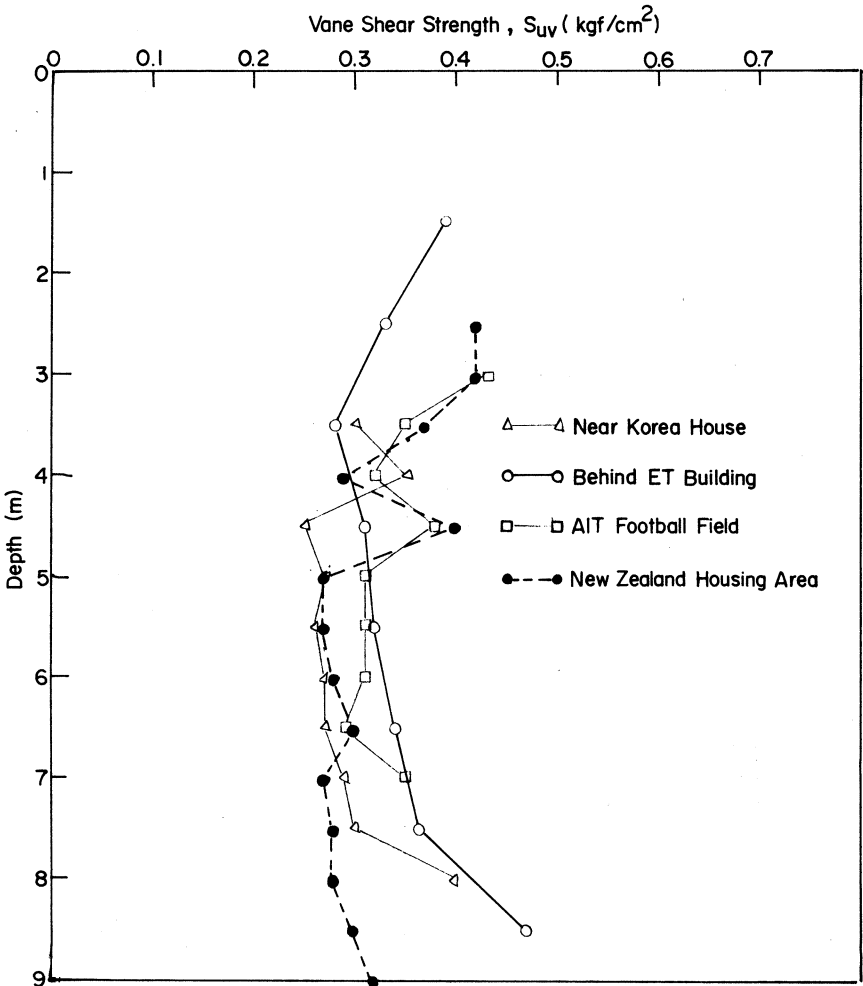


Fig. 12. Variation in Vane Shear Strength with Depth.

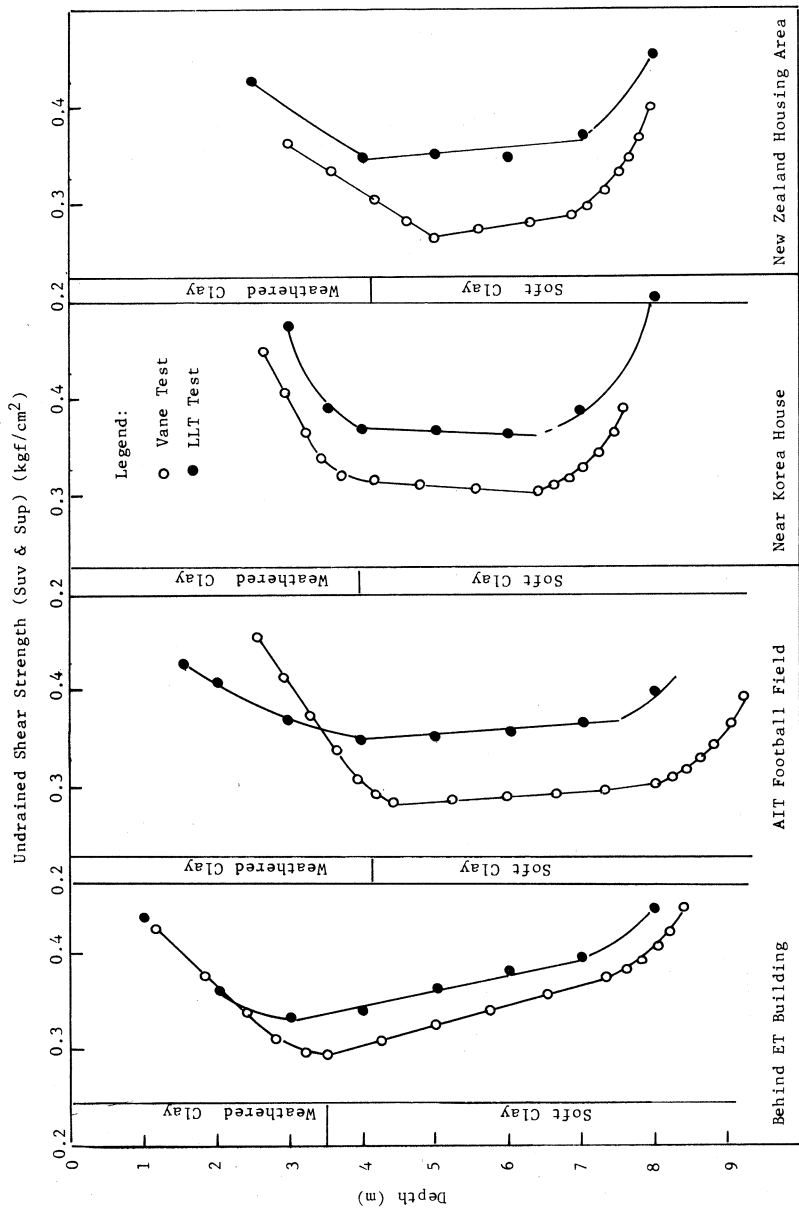


Fig. 13. Comparison of the Undrained Shear Strength Obtained by Field Vane and Pressuremeter Tests.

INSITU TESTS

The results of the vane shear strength measurements from the four sites in AIT Campus are plotted as averaged values with depth in Figure 12. It can be seen that in the soft clay zone the strengths are slightly increasing with depth, ranging from 0.25 kg/cm² to 0.35 kg/cm². Figure 13 shows the mean curves fitted to the vane shear strengths in each of the four sites. It can be seen that near Korea house the weathered zone extended to a depth of 3.0 to 3.5 m and the soft clay ended at 7 m depth. Behind the ET Building, the weathered layer extended to 2.0 to 2.5 m depth and the soft clay ended at 8 m depth. The other two sites are within these ranges.

THE DUTCH CONE TEST

A standard Dutch cone penetrometer of 2 ton capacity (2,000 kg) was utilized, having a cone diameter of 35.7 mm with an apex angle of 60 degrees. The friction sleeve above the conical tip has a standard area of 150 cm². The cone and the rod assembly were first pressed down to a testing depth. Readings were taken at every 20 cm interval at a rate of penetration of 2 cm/s. The average test results from four sites are plotted in Figures 14 and 15 corresponding to the local friction and cone resistance, respectively. Both data sets showed similar trend with depth. In the weathered zone both values decreased with depth, while in the soft clay layer both values were nearly constant. In the stiff clay layer both values increased rapidly with depth.

STANDARD PENETRATION TEST (SPT)

The standard penetration test consisted of the measurements (blows per foot) of the resistance of the soil to the penetration of a standard split spoon having 5.08 cm outside diameter and 3.49 cm inside diameter by the free-fall of a 63.60 kg weight through a height of 76.2 cm. The standard spoon was lowered to the bottom of the borehole and was given 15.24 cm initial penetration. Then, the number of blows (designated as N) for the next 30.48 cm was recorded. The averaged N values are plotted with depth in Figure 16 for the four sites in the stiff clay and dense to very dense sand layers. Figure 17 shows the range of grain sizes of the sand layer. These results were corrected for overburden pressure using the method of GIBBS & HOLTZ (1957). The averaged N values varied between 10 and 40, increasing with depth except near 18 m depth. Near this depth, the particle sizes passing the no. 200 sieve sometimes exceeded 35%.

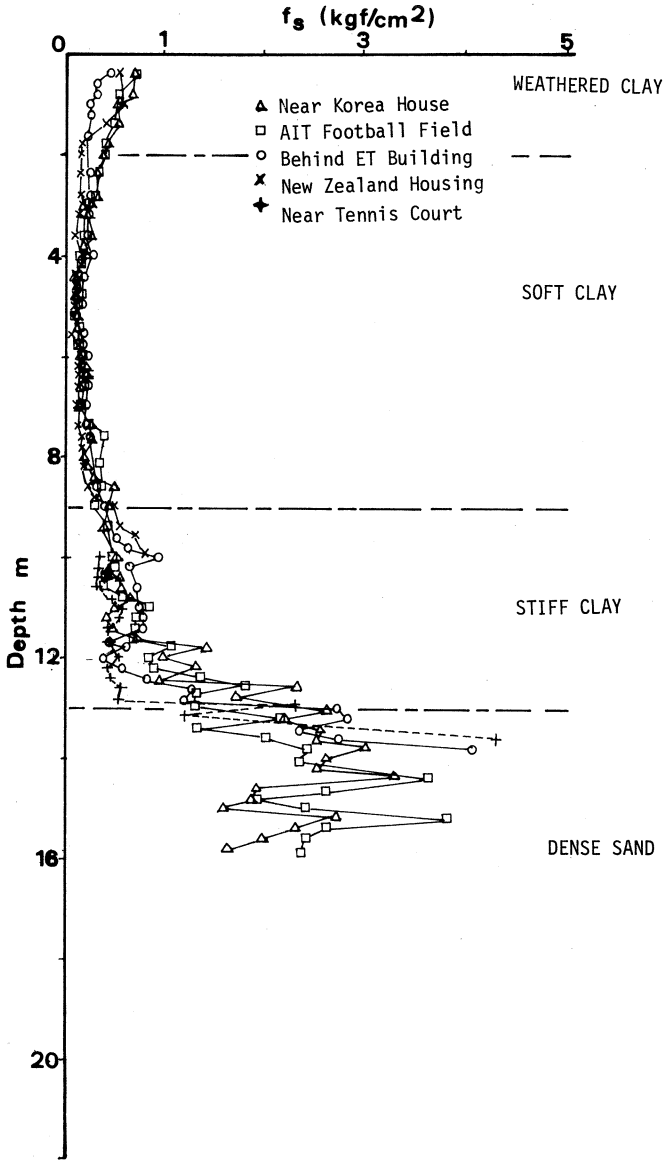


Fig. 14. Variation of Local Friction Value with Depth.

INSITU TESTS

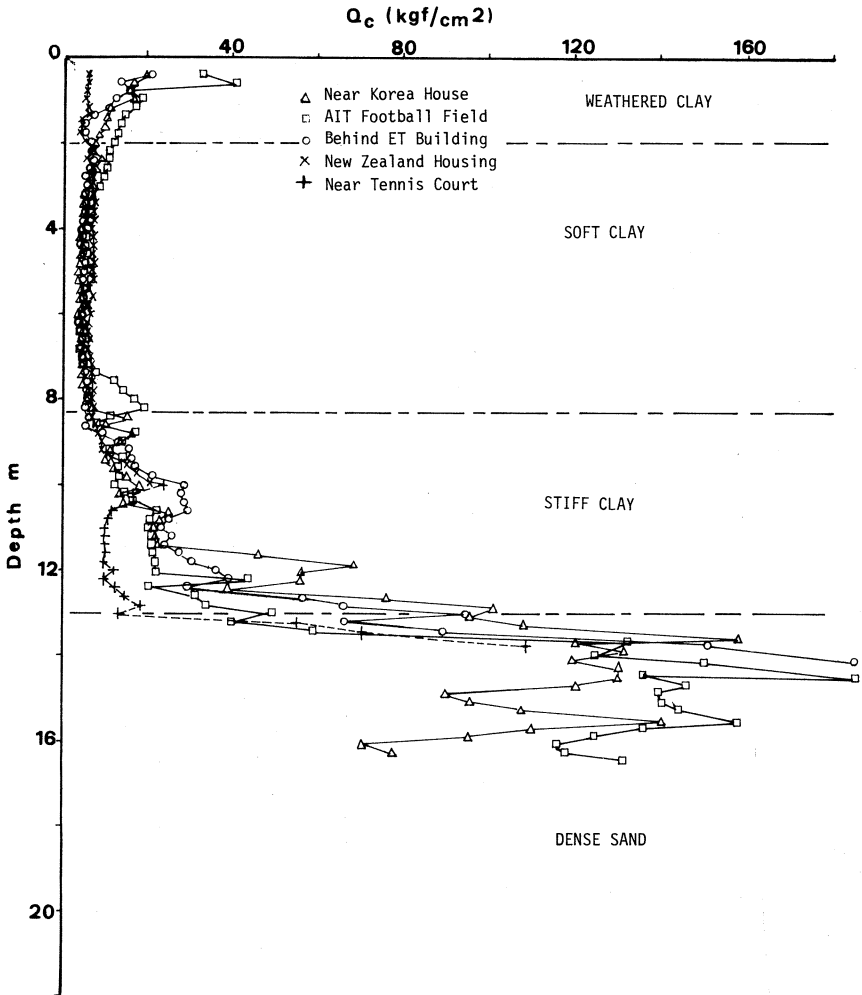


Fig. 15. Variation of Cone Resistance with Depth.

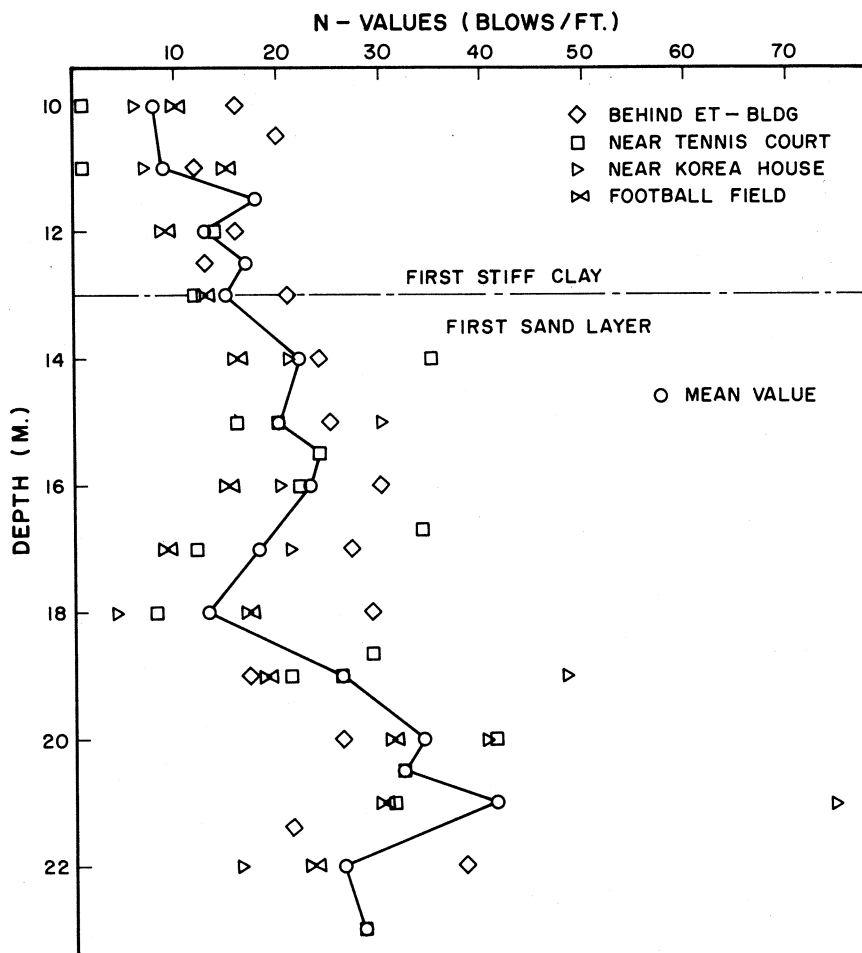


Fig. 16. Corrected SPT N-Value with Depth.

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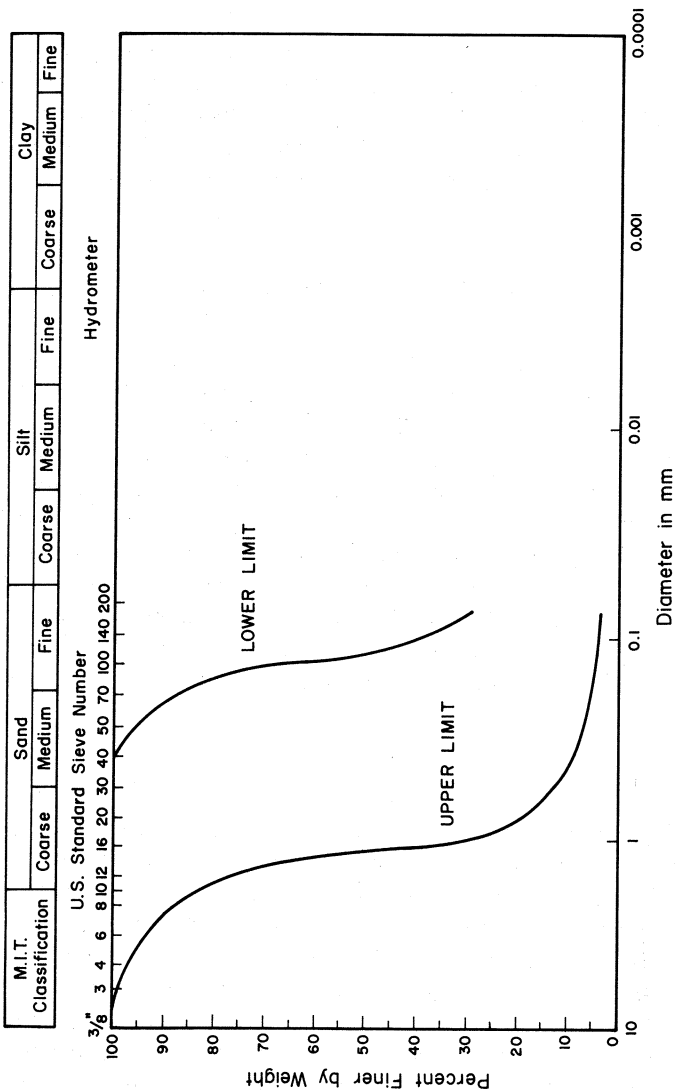


Fig. 17. The Range of Grain Sizes for the Dense to Very Dense Sand Layer.

PLATE LOAD TEST (PLT)

A 10 ton capacity hydraulic jack and a 10 ton load cell were used for applying and measuring, respectively, the load applied to a 24 inch (60.96 cm) diameter plate. A steel I beam with eight steel anchors was used as the reaction and a steel truss with three dial gauges (accuracy of 0.01 mm) was used as the reference beam. The procedures recommended by ASTM D1194-72 (ASTM, 1985) were followed for this test. Results of the tests plotted with depth are given in Figure 18 for the undrained modulus and in Figure 19 for the undrained strength.

SCREW PLATE LOAD TEST (SPLT)

The apparatus of the screw plate load test as shown in Figure 20 was made by AIT based on the instrument used by KAY & AVALLE (1982). A 10 ton capacity hydraulic jack and a 10 ton calibrated proving ring were utilized to apply and measure the load, respectively. The screw plate was a single cycle of a helical auger with diameter of 10 inches (25.4 cm), thickness of 6 mm, and pitch of 3 cm. A steel loading frame used both as reaction beam and advance controlling device was supported and anchored on four anchor piles. Two dial gauges with accuracy of 0.01mm attached to a reference beam were used to measure the settlement of the plate. The test procedures were similar to the plate load test. Results of the test are plotted in Figures 18 and 19 for the undrained modulus and undrained shear strength, respectively.

LABORATORY TESTS

For clay samples, index tests were performed and the results are plotted with depth in Figure 1 together with the soil profile. The consolidation tests were performed following the procedure of BOWLES (1978). The consolidometers used were the conventional type with lever arm ratio of 1 to 11 and a load increment ratio of 1. In the soft clay the preconsolidation pressure ranges from 7 to 16 t/m². All disturbed split-spoon samples of sand were air-dried, broken into pieces and tested for grain size, minimum and maximum dry densities, specific gravity and direct shear strength parameters. For the determination of the minimum and maximum density, the method suggested by the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE, 1979) was used, which was also applied by SAMANWONGTHAI (1983). These densities were used in the specimens for the direct shear tests.

INSITU TESTS

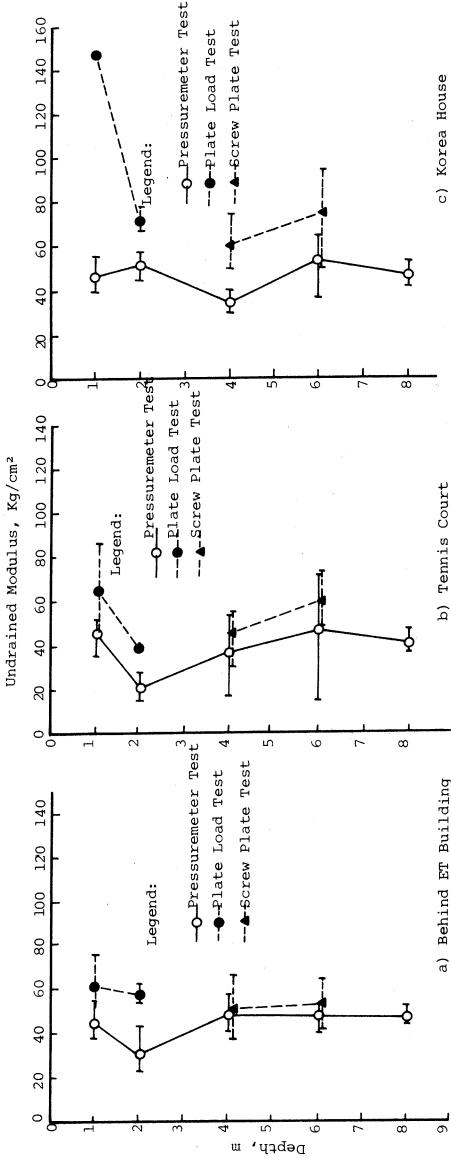


Fig. 18. Comparison of Undrained Modulus from Pressuremeter, Plate Load and Screw Plate Tests.

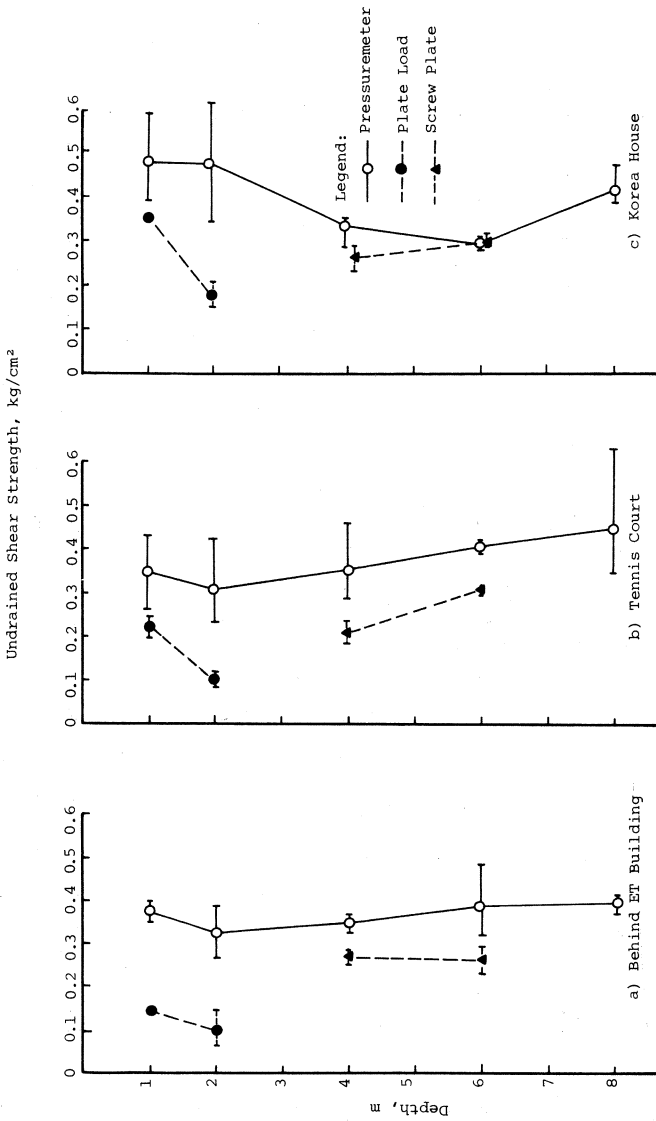


Fig. 19. Comparison of Undrained Shear Strength from Pressuremeter, Plate Load and Screw Plate Tests.

INSITU TESTS

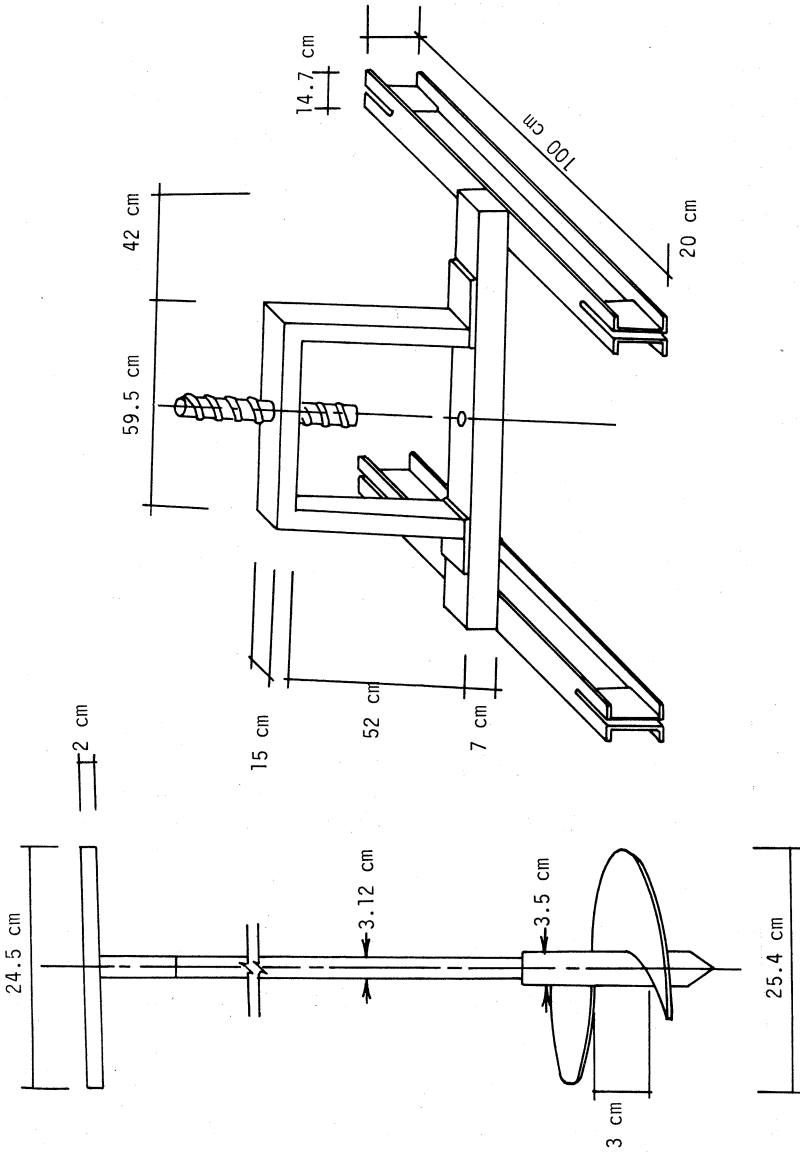


Fig. 20. Details of Screw Plate Apparatus.

The grain size limits of the sand are plotted in Figure 17. The fraction smaller than the no. 200 sieve varied from 4 to 30%. The relative densities derived from field data were determined using the method of GIBBS & HOLTZ (1957), COFFMAN (1960) and SCHULTZE & MENZENBACH (1961). The estimated dry densities in the field based on N-values ranged from 1.40 to 1.80 gm/cm³. The specific gravity ranged from 2.1 to 2.7. The rate of strain used in the direct shear tests was the same as that used by BRAND (1971), 0.061 cm per minute. The mean values of internal friction angle, ϕ'_d , from the direct shear tests are plotted with depth in Figure 10 together with the corresponding values from the pressuremeter test. The ϕ'_d values ranged from 30 to 40 degrees.

RELATIONSHIP BETWEEN THE VANE AND PRESSUREMETER TESTS

Correlation of Undrained Shear Strengths

Figure 13 shows the comparison of the mean relationships of undrained shear strengths from the vane test (S_{uv}) and from the pressuremeter test (S_{up}) with depth. By regression analysis the S_{up} values in the soft clay layer were found to be 10% to 25% higher than the S_{uv} values (KHALEQUE, 1984). The average relationship between S_{up} and S_{uv} in Bangkok soft clay is as follows:

$$S_{up} = 1.20 S_{uv} \dots\dots\dots (5)$$

Similar results were obtained by other investigators in London clay (HUGHES et al, 1975) and at different sites in France (AMAR et al, 1975). Other investigators have obtained S_{up} values up to 40% higher than S_{uv} values (MARSLAND & RANDOLPH, 1977; EDEN & LAW, 1980).

Correlation of S_{uv} and E_{up}

The undrained Young's modulus (E_{up}) determined from the pressuremeter test using Equation 3 is correlated with the undrained vane shear strength (S_{uv}) for both layers of weathered and soft Bangkok clay as given by the following equation:

$$E_{up} = 145 S_{uv} \dots\dots\dots (6)$$

The values obtained in this study agree well with those obtained by HUANG (1980) and TSAI (1982). The data of other investigator (BJERRUM, 1964) were found by TSAI (1982) to be too large for Bangkok clay.

INSITU TESTS

Correlation of S_{uv} with P_y and P_l

The correlation between the vane shear strength (S_{uv}) with the yield pressure (P_y) and limit pressure (P_l) yielded the following relationships:

$$P_y = 3.15 S_{uv} \quad \dots\dots\dots (7)$$

$$P_l = 5.90 S_{uv} \quad \dots\dots\dots (8)$$

The above correlation was found to be in good agreement with the works of HUANG (1980), LUKAS & DE BUSSY (1976), and CALHOON (1970) in the case of the limit pressure (P_l). In the case of the yield pressure (P_y), the main interest in a correlation with the vane shear strength lies in its relationship with the preconsolidation pressure of the clay as given in Equation 28.

RELATIONSHIP BETWEEN THE PRESSUREMETER AND DUTCH CONE TESTS

Correlations of S_{up} with q_c and f_s

The cone resistance (q_c) and local friction (f_s) have shown a good correlation with the undrained shear strength from the pressuremeter test (S_{up}) in both the soft clay and the stiff clay layers. By regression analysis, the following relationships were found:

$$q_c = 16.4 S_{up} \text{ (soft clay)} \quad \dots\dots\dots (9)$$

$$q_c = 19.4 S_{up} \text{ (stiff clay)} \quad \dots\dots\dots (10)$$

$$f_s = 0.53 S_{up} \text{ (soft clay)} \quad \dots\dots\dots (11)$$

$$f_s = 0.61 S_{up} \text{ (stiff clay)} \quad \dots\dots\dots (12)$$

These cone factors agree well with the results of past investigators (PHAM, 1972; KHALEQUE, 1984; THOMAS, 1965) using the undrained shear strength from the vane shear test adjusted to the undrained strength from pressuremeter test using Equation 5. For the Bangkok clay values of q_c/S_{up} between 14 and 19 have been reported, as have values of f_s/S_{up} from 0.47 to 0.60 (BRAND et al, 1974; WIROJANAGUD, 1974). It can be seen that the values in Equations 9 to 12 are within these ranges.

Correlation of q_c and E_{up}

The undrained Young's modulus from pressuremeter tests (E_{up}) was correlated with the cone resistance (q_c) from Dutch cone tests for the weathered and soft marine clays. By regression analysis the following relationships were obtained:

$$E_{up} = 8.85 q_c \text{ (soft clay)} \dots\dots\dots (13)$$

$$E_{up} = 8.48 q_c \text{ (weathered clay)} \dots\dots\dots (14)$$

For the stiff clay, Equation 15 below was obtained which is similar to the earlier results reported by WAMBEKE & HEMRICOURT (1982) and the subsequent work of NEEYAPAN (1985).

$$E_{up} = 4.5 q_c \text{ (stiff clay)} \dots\dots\dots (15)$$

A value for E_{up}/q_c of 4.5 was obtained by NEEYAPAN (1985).

Correlations of q_c with P_y and P_l

Correlations were made between the cone resistance (q_c) and the yield pressure (P_y) and limit pressure (P_l), respectively. By regression analysis, the following relationships were obtained for the soft clay layer.

$$q_c = 7.4 P_y \text{ (soft clay)} \dots\dots\dots (16)$$

$$q_c = 3.36 P_l \text{ (soft clay)} \dots\dots\dots (17)$$

In an earlier work by HUANG (1980), a ratio of q_c/P_l was found to be 3.5 for the soft Bangkok clay. NEEYAPAN (1985) obtained a ratio of q_c/P_y of 14 and a ratio of q_c/p_l^* of 3.24 in the stiff clay layer. BAGUELIN *et al* (1978) have found that for stiff to very stiff clay the value of q_c/p_l^* was between 2.5 and 3.5, and for very stiff to hard clay the ratio was between 3 and 4, where p_l^* is the net limit pressure which is equal to P_l minus P_o .

RELATIONSHIP BETWEEN THE PRESSUREMETER AND THE STANDARD PENETRATION TEST

Correlations of N-value with P_y and E_p

By regression analysis, the following relationship was obtained between the undrained pressuremeter modulus (E_p) and SPT N-value in the stiff clay.

$$E_p = 8.3 N \text{ (stiff clay)} \dots\dots\dots (18)$$

Similar results were obtained by TSUCHIYA & TOYOOKA (1982) in alluvial clayey soil. The relationship between the N-value and the yield pressure (P_y) for the stiff clay in this study is given as :

$$N = 6.86 P_y \text{ (stiff clay)} \dots\dots\dots (19)$$

For a clayey soil, OHYA *et al* (1982) found the ratio of N/P_y from 2 to 3.

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In the sand layer, the relationship between the N-value and the undrained pressuremeter modulus (E_p) was found to be:

$$E_p = 37 N^{0.45} \text{ (Bangkok sand) } \dots\dots\dots (20)$$

TSUCHIYA & TOYOOKA (1982) found slightly different values in a slightly different soil types as follows :

$$E_p = 7.85 N^{0.84} \text{ (sandy soil) } \dots\dots\dots (21)$$

$$E_p = 16.74 N^{0.66} \text{ (gravelly soil) } \dots\dots\dots (22)$$

The relationship between N-value and yield pressure (P_y) for the sand layer is as follows:

$$N = 5.44 P_y \text{ (Bangkok sand) } \dots\dots\dots (23)$$

OHYA et al (1982) obtained data for sandy soil in close agreement with the results of this study.

Correlation of SPT N-Value with Net Limit Pressure (P_l^)*

The relation between SPT N-value and the net limit pressure (P_l^*) for the stiff clay layer is given as follows:

$$N = 1.37 P_l^* \text{ (stiff clay) } \dots\dots\dots (24)$$

The relationship between the SPT N-value and the net limit pressure (P_l^*) for the dense to very dense sand layer was obtained by regression analysis as follows:

$$N = 1.03 P_l^* \text{ (dense to very dense sand) } \dots\dots\dots (25)$$

RAMASWAMY et al (1982) obtained a ratio of N/P_l^* of 1.0 for both loose and medium dense to coarse sand in Changi, Singapore.

Correlation of the Angle of Internal Friction Values from SPT and Pressuremeter Test

The values of the internal friction (ϕ'_N) estimated from the averaged N-values following the method of PECK et al (1974) were correlated with the corresponding values obtained from pressuremeter test (ϕ'_p). The following relationship was obtained.

$$\phi'_N = 0.97 \phi'_p \dots\dots\dots (26)$$

It can be observed that the calculated values for angle of internal friction from both tests are almost equal.

RELATIONSHIP OF PRESSUREMETER AND DIRECT SHEAR TESTS

Correlation of the Angle of Internal Friction Values

The angle of internal friction values obtained from laboratory direct shear (ϕ'_d) and the angle of internal friction values calculated from the method of MENARD (1970) in the pressuremeter tests (ϕ'_p) were correlated. By regression analysis, the following relationship has been found.

$$\phi'_d = 0.92 \phi'_p \dots\dots\dots (27)$$

It can be seen that the angle of internal friction obtained from the pressuremeter test is slightly higher than that obtained from the direct shear test.

RELATIONSHIP OF PRESSUREMETER AND ONE-DIMENSIONAL CONSOLIDATION TESTS

Correlation of Yield Pressure (Py) and Preconsolidation Pressure (Pc)

For the soft clay layer a correlation between yield pressure (Py) and preconsolidation pressure (Pc) obtained by regression analysis can be expressed by the following equation:

$$P_y = 0.96 P_c \dots\dots\dots (28)$$

The correlation in Equation 28 is found to be in good agreement with the results obtained by MORI & TAJIMA (1964) and the findings of LUKAS & DE BUSSY (1976).

RELATIONSHIPS OF PRESSUREMETER, PLATE LOAD, AND SCREW PLATE TESTS

Comparison of Undrained Modulus (Eup, Eusp, and Eup1)

The undrained moduli determined from the pressuremeter, plate load, and screw plate tests are plotted against depth in Figure 18 for three different sites. It can be seen that the values from the pressuremeter tests are about 40% lower than the results of the plate load test in the weathered clay layer and about 10 to 40 percent lower than the screw plate test in the soft clay layer. The lower modulus from the pressuremeter test may be attributed to the disturbance of the thin zone neighboring the borehole wall (MORI, 1981). The disturbance is due to the advancement of the borehole, a temporary lack of confinement in the sides of the borehole, and finally due to the insertion of the probe into the borehole (WROTH & HUGHES, 1973). Moreover, the difference in the modulus values may be due to the fact that both the plate

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load test and the screw plate test have vertical deformation, while the pressuremeter test has horizontal deformation. In this study, the values of undrained modulus obtained from both the plate load and screw plates tests are the values of secant modulus over the stress range from zero to one-half the bearing capacity.

Comparison of Undrained Strengths (S_{up} , S_{usp} , and S_{up1})

A comparison of the undrained strengths determined from the pressuremeter, plate load, and screw plate tests is shown in Figure 19, where the values are plotted with depth for each of the three sites. It can be seen that the undrained strengths from the pressuremeter test computed using the method of GIBSON & ANDERSON (1961) are one to three times greater than the corresponding values from the plate load test using a bearing capacity factor of 7.2 in the weathered clay layer. MARSLAND & RANDOLPH (1977) found that the undrained strengths determined from the pressuremeter test ranged from one to more than three times the corresponding values from the large insitu plate tests in London clay. In their theoretical studies, MARSLAND & RANDOLPH (1977) found that the ratio of the dimensions of the volume of soil tested to the distance between fissures in the clay can seriously affect the measured shear strengths. In the soft clay layer, the undrained shear strengths from the screw plate tests using a bearing capacity factor of 9 are about 75% to 85% of the corresponding values derived from the pressuremeter tests. KAY & PARRY (1982) obtained similar relative values of undrained strengths from the pressuremeter test and screw plate test in Gault clay, using a screw plate with diameter of 10 cm.

CONCLUSIONS

The pressuremeter, screw plate, plate load, field vane, Dutch cone, and standard penetration tests were performed in the weathered, soft, and stiff clay layers, as well as in the dense to very dense sand layers of Bangkok subsoils. In the pressuremeter test, the Oyo Corporation Lateral Load Tester (LLT) monocell probe was utilized together with the wash boring technique of advancing the borehole. The pressuremeter test yielded the horizontal earth pressure at rest (P_o), yield pressure (P_y), limit pressure (P_l), undrained modulus (E_{up}) or pressuremeter modulus (E_p), undrained shear strength (S_{up}), and angle of internal friction (ϕ'_p) for the sand layer. The field vane test yielded the vane shear strength (S_{uv}) and the Dutch cone test gave the cone resistance (q_c) and local friction (f_s). The screw plate test yielded the undrained modulus

(Eusp) and undrained strength (Susp) while the plate load test gave also the undrained modulus (Eup_I) and undrained shear strength (Sup_I). The standard penetration test yielded the number of blows (N) and the angle of internal friction (ϕ'_N). In addition, direct shear tests and index tests were performed in the laboratory for the sand samples to yield the angle of internal friction (ϕ'_d), grain size curve, dry density, relative density and specific gravity. Another series of one-dimensional consolidation tests was also performed on clay samples to obtain data on the preconsolidation pressure in the soft clay layer.

The sampling and field tests were carried out on the campus of the Asian Institute of Technology where the uppermost four subsoil layers consist of about 2 m weathered clay, underlain by about 6 m of soft clay, followed by about 5 m stiff clay, and about 10 m of dense to very dense sand. The pressuremeter, field vane and Dutch cone tests were done in both the weathered and soft clay layers at five locations in each of four sites. The pressuremeter, Dutch cone and standard penetration tests were performed in both the underlying stiff clay and dense to very dense sand layers at four sites (three to five locations in each site). The plate load and screw plate tests were done in the weathered clay layer and soft clay layers at two locations in each of four sites. The following correlations were obtained from the averaged soil parameters derived from these tests.

For the weathered clay layer (0 to 2 m depth):

$$q_c = Eu/8.48$$

For the soft clay layer (2 to 8 m depth):

$$\begin{aligned} Sup &= q_c/16.4 = f_s/0.53 \\ Suv &= Sup/1.20 = Eup/145 \\ Suv &= Py/3.15 = P_I/5.90 \\ q_c &= Eup/8.85 = 7.4 Py = 3.36P_I \\ Py &= 0.96 Pc \end{aligned}$$

For the stiff clay layer (8 to 13 m depth):

$$\begin{aligned} Sup &= q_c/19.4 = f_s/0.61 \\ q_c &= Eup/4.5 = 14 Py = 3.24 P_I^* = 3.86 N \\ N &= 6.86 Py = 1.37 P_I^* = Ep/8.3 \end{aligned}$$

For the dense to very dense sand layer (13 to 23 m depth):

$$\begin{aligned} N &= 5.44 Py = 1.03P_I^* \\ Ep &= 37 N^{0.45} \\ \phi'_p &= \phi'_d/0.92 = \phi'_N/0.97 \end{aligned}$$

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In addition, the undrained modulus values obtained from the pressuremeter test were found to be about 40% lower than the corresponding values obtained from the plate load test in the weathered clay layer. In the soft clay layer, the undrained modulus values from the pressuremeter test were about 10 to 40% lower than the corresponding values obtained from the screw plate tests. Moreover, the undrained strengths obtained from the pressuremeter tests were one to three times the corresponding values from the plate load test in the weathered clay. In the soft clay layer, the undrained shear strengths from the pressuremeter tests were about 20% higher than those obtained from the screw plate tests.

The correlations obtained in this study have been verified, compared, and discussed with the results of past investigators. It was found that the results in this study agreed well with the earlier results.

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DEEP EXCAVATIONS FOR BASEMENT CONSTRUCTION IN HONG KONG

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SYNOPSIS

Deep excavations for basement construction are common in urban areas of Hong Kong, and instances of the collapse of the support system to the faces of excavations have occurred, causing considerable inconvenience and financial loss. This paper discusses the main design requirements and describes some of the methods employed to overcome the common problems associated with basement excavation in Hong Kong.

INTRODUCTION

The Territory of Hong Kong consists of Hong Kong Island and the Kowloon Peninsula together with the New Territories (Figure 1). The New Territories comprises a small piece of the Chinese mainland north of Kowloon and more than 200 small islands, the largest of which is Lantau. The northern part of Hong Kong Island and the Kowloon Peninsula form the rim of a natural basin which constitutes Victoria Harbour.

Historically, the area around Victoria Harbour has been the location of extensive development. Over 80 percent of the five and a half million people of the Territory of Hong Kong live in the immediate vicinity of Victoria Harbour. Much of the natural terrain, and in particular Hong Kong Island, is very hilly with many hills rising steeply from the sea to heights exceeding 400 m. The steep hills are deeply weathered and are prone to landslides during the heavy rainfall experienced in the summer months. Rainfall intensities can be high, with 24-hour rainfall of more than 250 mm and one-hour rainfall in excess of 50 mm occurring fairly frequently. A rainfall intensity of about 70 mm/hr appears to be the threshold value above which landslides occur (BRAND et al, 1984). There is now an effective system of control of all slope works in Hong Kong and a full summary of the state-of-the-art of landslide prevention and control is given by BRAND (1984). There is very little flat land (0 to 5° slope) available for building development in this area, and platforms for construction of buildings were formed by significant modifications to the topography by large cut and fill site formation works, along with major reclamation schemes. As much as one third of the flat land for building in this region is reclaimed from the sea.

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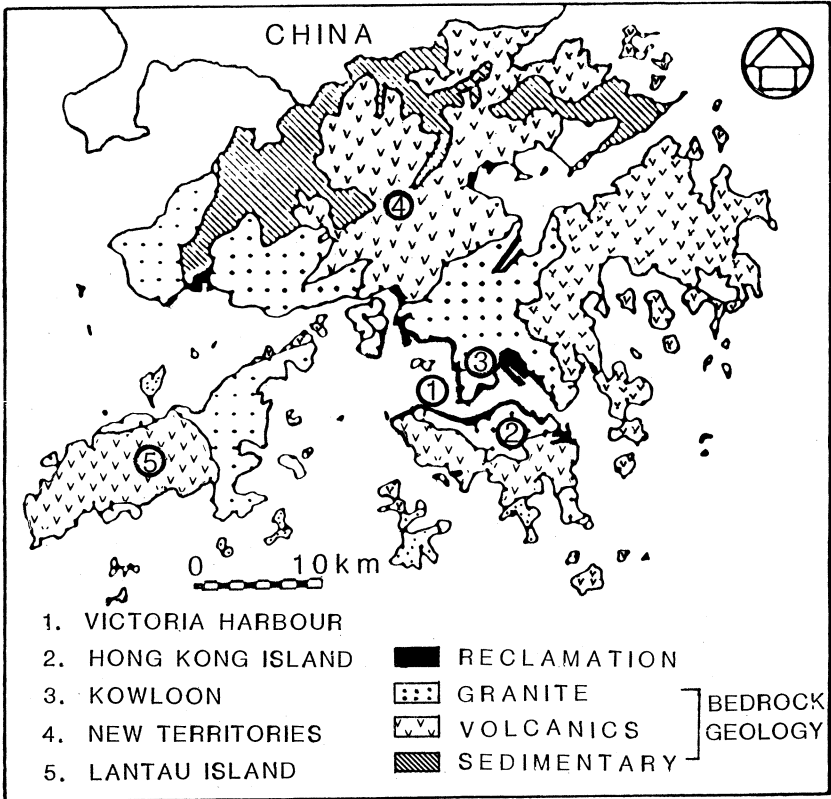


Fig. 1. Territory of Hong Kong and its Geology.

Originally, the northern part of Hong Kong Island was only a narrow strip of poorly drained swampy coastal lowland, and reclamation has enabled intensive development since the mid 19th century. The Kowloon Peninsula originally consisted of a group of severely eroded low granitic hills and swampy lowlands. Development of Kowloon Peninsula commenced in the late 19th century with the removal of materials from the low hills for land-based fills and reclamation around the harbour foreshores. Reclamation along the harbour foreshores is still carried out to meet the land requirement of Hong Kong. New building developments in these areas, due to the increase in land costs and the height restriction imposed by the proximity of the airport, have resulted in deep basement construction. For better utilisation of land, pre-war buildings in this area are also being replaced by new developments involving deep basements.

DEEP EXCAVATIONS

Excavation depths of about 10 m are common. The deepest excavations are about 40 m and these have been in connection with the construction of the underground Mass Transit Railway. Most of these deep basement constructions are in heavily built up areas, adjacent to heavily trafficked highways, within reclaimed land along the coastal fringes of the Kowloon Peninsula and the northern regions of Hong Kong Island.

GEOLOGY

Bedrock

The geology of the territory of Hong Kong has been described by RUXTON (1960) and ALLEN & STEPHENS (1971). The regional geology of Hong Kong Island and Kowloon Peninsula can be described simply as a sequence of faulted, folded, mildly metamorphosed volcanoclastic rocks that are extensively intruded by younger igneous rocks and partially overlain by a variety of superficial deposits. Granitic batholith predominates in the heavily developed areas around the harbour and volcanic rocks mainly occur as roof pendants on the hills in this area.

Climatic conditions in Hong Kong favour the formation of a deep weathering profile. The weathering is largely a chemical process that transforms rock to soft soil. The depth of weathering is largely controlled by joint spacing and the rate of erosion.

The granite rocks are extensively weathered, with a maximum weathered depth in excess of 80 m. Weathering depths in excess of 30 m commonly occur with large corestones in the matrix. These corestones have been a source of problems in the installation of support systems for the deep excavations required in basement construction.

The volcanic rocks are more resistant to weathering than the granite, with the residual mantle extending to about 20 m depth.

Investigation into the weathering process and description of the weathered profile in Hong Kong was pioneered by RUXTON & BERRY (1957, 1959, 1961). The weathering profile is logged using a system of weathering description and classification based on the work of Ruxton & Berry. Additional index tests have been proposed by HENCHER & MARTIN (1982) to identify various grades of weathering. A general review of site investigation practice and the weathering grades used in Hong Kong is given by BRAND & PHILLIPSON (1984).

Superficial Materials

Superficial materials are commonly encountered in deep basement excavation. These materials could be classified as mass wasting deposits (colluvium), terrestrial alluvial deposits, marine deposits and made ground.

Superficial deposits of the Hong Kong Harbour area have been described by BERRY (1957) and HOLT (1962), and more recently a review of superficial deposits and weathering in Hong Kong has been made by BENNETT (1984). Colluvium was recognised and described by BERRY (1957), and BERRY & RUXTON (1960). Criteria for recognising colluvium and distinguishing it particularly from the residual soils have been given by HUNTLEY & RANDALL (1981).

Colluvium is an extremely heterogeneous material and most commonly consists of boulders, cobbles and gravel in a matrix of sand, silt and clay. It is often in a loose state on the lower part (base) of slopes, and is commonly encountered in excavations on hill sides. However, in recent excavation work associated with the construction of an underground railway station on Hong Kong Island, colluvium is reported to underlie the upper marine—upper alluvial deposits in the Wanchai area to a depth of 8 m below Principal Datum and some 30 m offshore from the original shoreline (WILLIS & SHIRLAW, 1983).

Alluvial deposits and marine deposits are commonly encountered in deep excavations in the reclaimed areas. These sedimentary deposits constitute the extensive low-lying coastal plains and also occur offshore, and comprise the superficial sea floor cover, mantling weathered bedrock and residual soils. The succession of deposits described by YIM (1983), YIM & LI (1983) and DICKSON (1983) indicate the presence of upper and lower marine and alluvial deposits. Excavations for basements in urban areas have indicated the presence of only the upper marine-alluvial sequence. The alternating nature of these deposits is most satisfactorily explained as the result of marine transgression and regression during the Quaternary, and the gradual drowning and subsequent silting-up of previously low-lying ground under conditions ranging from truly alluvial to wholly marine in character (BERRY, 1962; BENNETT, 1984). Recognition of these deposits depends on sedimentological, physical and chemical criteria and, importantly, on the presence of shell and plant remains which also provide the basis for assessing the age of the deposits. The marine deposits tend to be fine grained silty clay with silt and sand seams and partings. The alluvial deposits are coarser and more compact. The engineering properties of marine deposits are described by LUMB (1977)

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and FUNG et al (1984). The upper marine deposits are of Holocene age and the underlying deposits are of late Pleistocene age. The upper alluvium is believed to have been deposited between 8,000 and 33,400 years BP during a predominantly glacial period (YIM & LI, 1983). These coarse alluvial deposits are more permeable than the marine deposits.

DEEP EXCAVATION DESIGN

General Requirements

The data required for adequate design of a deep excavation primarily depends on the depth of the excavation. TERZAGHI & PECK (1967) distinguished between shallow cuts with depth less than about 20 ft (6.1 m) and deep cuts (with greater depth).

The Building Authority (BA) is responsible for the control of all private building developments in Hong Kong. The geotechnical aspects of the building works, including basement excavations, are checked by the BA on the advice of the Geotechnical Control Office (GCO). The BA checks the lateral support plans for basement excavations and imposes the standard of qualified site supervision to be provided during the construction of the basement. When depth of basement excavation exceeds 7.5 metres, the BA requires a geotechnical assessment to be made of the adequacy of the building proposals at an early stage, when general building plans are submitted for checking of fundamental planning and design criteria. This permits geotechnical involvement when developer's planning is still flexible, and thus avoids delays due to objections or constraints when detailed engineering plans are submitted.

Excavation design in Hong Kong's steep hillsides for building development has been dealt with by FLINTOFF & COWLAND (1982). The design of shallow excavations is more or less standardised. This paper deals only with deep excavations for basement construction.

Soil Pressure Distribution

In the past, deep excavations were designed on the assumption that earth pressure increases with depth. However, behaviour of braced excavations has shown that this assumption is rarely justified. PECK (1969) in his state-of-the-art report states that the envelopes or apparent pressure diagrams (Figure 2) were not intended to represent the real distribution of earth pressure at any vertical section of the cut, but instead constituted a hypothetical pressure

diagram from which strut loads could be calculated. These loads might be approached, but would not be exceeded in the actual cut. Field observations show that the actual pressure distribution varies from section to section depending on many construction variables. Thus, distribution of pressure on a strutted excavation is complex, and it is normal to use a pressure envelope covering the expected range of pressure distributions. However, the apparent pressure distribution for cohesionless soils (sand) proposed by TERZAGHI & PECK (1967) is being widely used in Hong Kong to determine the strut loads. It is important to note that Terzaghi & Peck's method assumes the water table is below the bottom of the excavation, and the sand is assumed to be drained with zero pore pressure. This assumption is not true for Hong Kong conditions where groundwater level is high. It has therefore been the practice in Hong Kong to use the submerged density of soil with hydrostatic water pressure in determining the equivalent "Peck" envelope for the design of the struts.

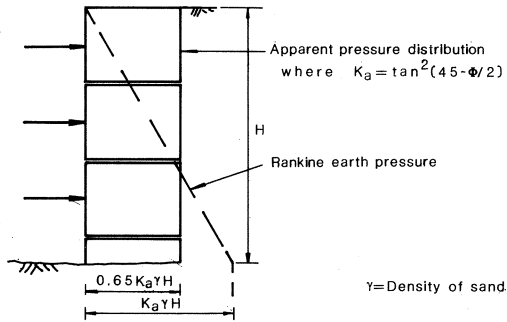


Fig. 2. Apparent Pressure Distribution for Completed Excavation in Sand for Computing Strut Loads (PECK, 1969).

It is also important to realise that the magnitude and distribution of active and passive earth pressures on the walls of an excavation are affected by the seepage of groundwater into the excavation due to dewatering. The groundwater seepage increases the effective density in the downward flow regions and decreases it in the upward flow region, as shown in Figure 3.

Groundwater Pressure Distribution

The effect of groundwater flow on the design of deep excavations has been described by KAISER & HEWITT (1982) amongst others. Figure 4a shows the idealised water pressure commonly used in the design. This could vary

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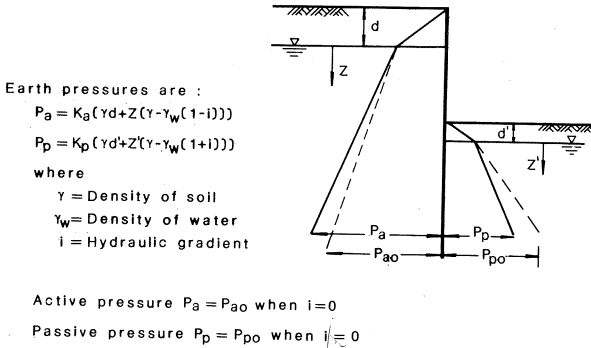


Fig. 3. Seepage Effects on Earth Pressure Distribution.

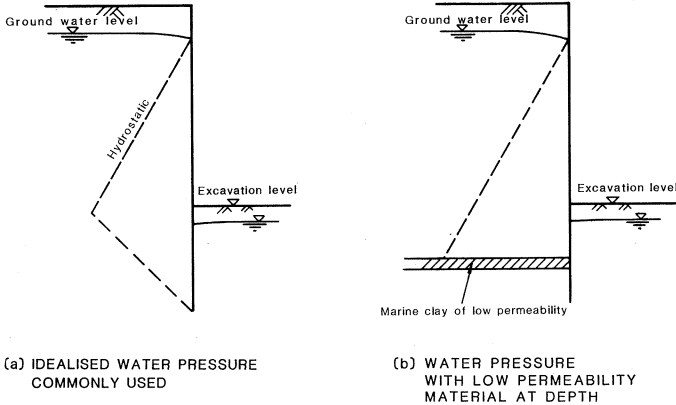


Fig. 4. Water Pressure Distribution.

considerably depending on the geological variations encountered at the site. Figure 4b shows the water pressure distribution in a reclaimed site where low permeability material such as marine clay exists. This generally results in high resultant water loads on the structure supporting the faces of the excavation. The resultant water load may vary within the range of ± 50 percent of the uniform isotropic case (fixed groundwater table) for an excavation 15 m deep with groundwater table 6.7 m below ground surface, due to boundary conditions, anisotropy, and stratification of the soil (KAISER & HEWITT, 1982).

Groundwater Control

Drawdown of groundwater level due to dewatering during excavation is a major factor contributing to ground movement in Hong Kong. The majority

of settlements of buildings and ground associated with the construction of cut and cover underground stations for the Mass Transit Railway in Hong Kong were a result of diaphragm wall installation or dewatering during station excavation (MORTON et al, 1980 a, b). Control of drawdown of groundwater is therefore vital to ensure stability of ground and buildings on shallow footings adjacent to deep basement excavations. This is normally achieved in Hong Kong by:

- (i) providing a sufficient depth of sheeting wall below the final excavation level to act as cut-off;
- (ii) confining the excavation/dewatering to selected areas within the closed perimeter sheeting walls; and
- (iii) monitoring of the groundwater/settlement.

If the monitoring system indicates that the drawdown of groundwater is excessive, then the dewatering is further controlled by:

- (i) restricting the excavation/dewatering so as to provide sufficient time for the groundwater to recover due to natural recharge,
- (ii) providing additional cut-off with a grout curtain below the toe of the sheeting wall, and
- (iii) recharging the ground with water.

A pre-draining system is now being commonly used in Hong Kong. This consists of an automatic submersible pump installed at the bottom of a steel casing with graded filter at about a metre below the final excavation level. Keeping the excavation dry by this means increases the strength of the soil at the bottom of the excavation and thus facilitates the use of heavy equipment for the excavation works.

Chemical grout curtains and recharge wells have been used in Hong Kong to control groundwater during deep excavations for basement construction. MORTON & LEONARD (1980) have described the effectiveness of chemical grouting in residual soils to limit groundwater drawdown during the construction of underground railway stations in Hong Kong. MORTON & TSUI (1982) have reported on the use of recharge wells to control groundwater drawdown at a deep excavation site on Hong Kong Island where penetration of steel sheet piles below the final excavation level was limited.

Piping and Heave

Seepage into an excavation may produce piping or heave in the soil. To avoid this situation Hong Kong practice has been to provide a depth of

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penetration of the sheeting sufficient to give a factor of safety greater than 1.5 against piping and heave. Excavations in marine clay may be subject to base heave failure due to shear. The factor of safety, F , with respect to base heave is given by

$$F = \frac{N_b c}{\gamma H + q}$$

where N_b is the stability factor, c is the average undrained shear strength, H is the depth of excavation, γ is the bulk density, and q is the surcharge. Values of N_b may be obtained for example from the publication by JANBU et al (1956).

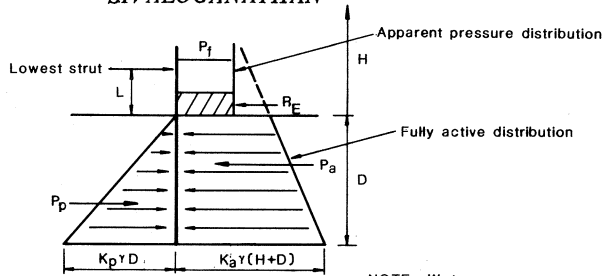
Studies by MANA & CLOUGH (1981) indicate that there is a well defined relationship between factor of safety against base heave and maximum lateral wall movements for cuts in clay. This relationship indicates that lateral wall movement increases very rapidly below a factor of safety of 1.5 against base heave.

Strut Loads

The load carried by each strut in a braced excavation is generally estimated by assuming that the sheet pile wall is simply supported between struts, and that a reaction below the base of the excavation exists. This reaction is provided by the passive resistance of the soil below the base. The depth of penetration of a sheet pile wall below the base should be sufficient to provide this reaction. In computing the required depth of penetration for the sheet pile into medium dense to dense decomposed granite, it is assumed that a triangular active pressure distribution exists below the base of the excavation to satisfy the force equilibrium on the horizontal plane, as shown on Figure 5. In determining the passive resistance, a factor of safety of not less than 1.5 is generally used to limit excessive movement of the wall.

When a relatively weak soil layer such as marine deposit is encountered below the level of excavation, little passive resistance can be expected from it. Under these conditions it is economical to design the sheeting as a cantilever, with the lowest strut taking up the extra load. In such situations, little is gained from driving the sheeting further than that required to prevent heave or piping, provided the moment of resistance of the sheeting is not exceeded by the cantilever action (GOLDBERG et al, 1975).

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NOTE : Water pressure .
omitted for clarity

In computing P_p provide a
F of S > 1.5
 $R_E = P_f \times L/2$

Compute depth of penetration
D from
 $P_p = P_a + R_E$

Check moment at bottom strut
assuming the sheet pile as
cantilever below the lowest strut.
Required depth of penetration = 1.2D

Fig. 5. Depth of Penetration to Satisfy Force Equilibrium.

CONSTRUCTION

Deep excavations follow the conventional sequence of installation of a sheeting wall (i.e. steel sheet piles, diaphragm wall, bored piles, etc.) and mass excavation with erection of temporary struts. This is followed by the construction of pile caps and removal of the temporary struts as floor slabs and screen walls are cast.

Support of deep excavations in Hong Kong was reported by HUNTER (1983). The temporary supports to the sheeting wall are generally provided by horizontal steel frames placed transversely across the site. The horizontal frames are supported by vertical steel piles driven below the excavation level. The photograph at Figure 6 shows the complete arrangement of a typical shoring system.

Steel Sheet Pile Walls

Steel sheet piling with multilayered struts is the most common form of lateral support system for excavation in flat areas. The presence of corestones in residual soils, boulders in fill, and old foundations are major sources of obstructions for the installation of sheeting walls. Eighty percent of the deep excavations in Hong Kong suffer from this problem (MALONE, 1982). The obstructions preventing the driving of the steel sheet piles are normally removed in advance of the excavation by means of a local coffer dam, and then re-driving the steel sheet piles to the required depth of penetration.

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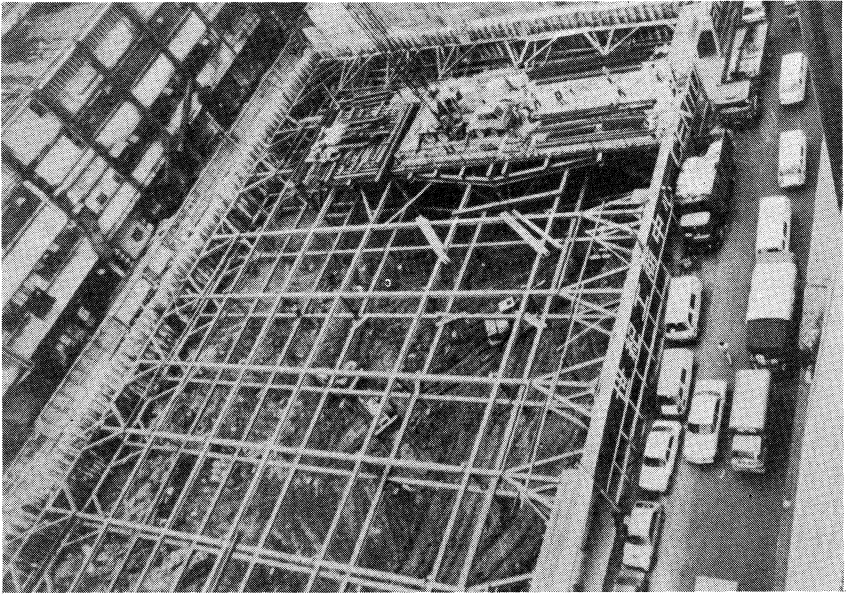


Fig. 6. Typical Arrangement of a Shoring System for Deep Basement Excavation in Hong Kong.

The problem of obstructions has often been underestimated when proposing steel sheet pile walls for deep excavations. This has resulted in considerable delay and modification to the support system. At the design stage, contractors usually express their confidence that the use of a powerful pile hammer and heavier steel sheet pile sections will be able to either split or push aside the boulders, and will penetrate the hard completely weathered layer of granitic rock. This has proved unsuccessful in many cases in Hong Kong. "Special measures" are then resorted to in order to achieve the required penetration of the steel sheet piles. At one site the "special measures" involved provision of a grout curtain from just above the toe of the existing steel sheet piles, followed by hand excavation of a 3 m long trench along the line of the sheet piles to a

depth of about 3 m below the general excavation level. The steel sheet piles were then redriven and the trench backfilled. This procedure was repeated throughout the length over which the steel sheet piles were obstructed by corestones. Although this was a time consuming process it proved to be successful at the particular site where corestones were widespread.

Diaphragm Walls

The use of diaphragm walls for basement excavation in Hong Kong has become popular in recent years (OPENSHAW & MARCHINI, 1980) particularly for excavations deeper than 18 m. The first major use of a diaphragm wall for basement excavation was the construction of the New World Centre building located on the Kowloon Peninsula, as described by TAMARO (1981).

Diaphragm walling frequently provides the best solution to many underground construction problems, and the technique can usually be adjusted to cope with most conditions by adjustment of the panel length, properties of slurry, level of slurry in the trench and groundwater (DAVIES, 1982). Obstructions encountered during the excavation of the slurry trench are first broken by heavy chisel or chopping devices and removed by grab buckets. Use of heavy devices to remove obstructions has resulted in overbreak and excessive settlement, of the order of 100 mm, adjacent to the slurry trench excavation. To facilitate the removal of obstructions and thus limit such overbreak, 200 mm diameter holes are drilled through the obstructions at 500 mm centres along both sides of the trench walls before the use of the chopping devices. This method is successfully being used at the site of the new headquarters of the Bank of China.

A method of dealing with loose boulder fill has been described by CRAFT (1983). The technique consisted of pre-trenching on the line of the diaphragm wall and backfilling it with lean concrete mixed with bentonite, and subsequently digging out by clamshell in bentonite slurry. The lean concrete mix should be of low permeability, sufficiently wet to be tremied, and strong enough to stabilise the boulders to be extracted subsequently by clamshell.

An examination of the observed ground settlement in Hong Kong indicates that only very small movements occur during the basement excavation, and the bulk of the settlement occurs during slurry trench excavations for the construction of the diaphragm walls (DAVIES & HENKEL, 1980; OPENSHAW, 1981). Significant ground and building settlements have been measured during slurry trench excavations (COWLAND & THORLEY, 1984). As a result of

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the study of the diaphragm wall construction at the Chater Station for the underground railway, it has been suggested that the settlement problems encountered in Hong Kong with slurry trench excavations were associated with swelling and softening of the decomposed granite (DAVIES & HENKEL, 1980). This problem has, however, been controlled by increasing the excess slurry head supporting the trench (DAVIES, 1982). The excess slurry head is defined as the difference in level between the slurry inside the trench and the piezometric level of the external groundwater. This excess slurry head could be obtained by raising the guide walls on the site or by dewatering the surrounding ground. If this dewatering is not controlled, it could lead to unacceptable ground settlement. To avoid the complication of using raised guide walls on the site of the new headquarters of the Hongkong and Shanghai Banking Corporation, an excess slurry head of approximately 3.5 m was obtained by dewatering the surrounding ground, where the water levels were within 2 to 3 m of the surface (FITZPATRICK & WILLFORD, 1985).

Pipe Pile Walls

A novel system of construction referred to as “pipe pile walling” has also been used to overcome the difficulties in driving steel sheet piles through hard strata of decomposed granite (SPT > 60) with corestones and through old foundations. Pipe pile walls consist of a series of 140 mm diameter slotted mild steel pipes set in place at centres of about 280 mm by drilling through

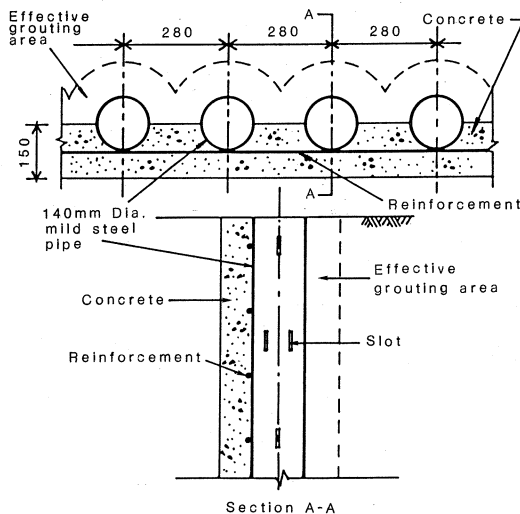


Fig. 7. Steel Pipe Pile Wall.



Fig. 8. Construction of a Pipe Pile Wall in Stages.

various formations using an eccentric reamer. The soil around the pipes is grouted through sleeved valve tubes inside the slotted pipes using bentonite cement and chemical grout to reduce the permeability of the soil behind the wall. The pipes form the structural elements of a retaining structure which is faced with concrete and braced with walings and struts, as excavation proceeds in stages of about 1.5 m. Figure 7 shows details, and the photograph in Figure 8 illustrates the progress of the wall construction in stages.

Caisson Walls

Hand-dug caisson walls are widely used in Hong Kong. These can be constructed without temporary soil cuts or shoring, close to existing structures, to overcome obstructions such as boulders and corestones without much difficulty (POPE, 1983).

Caissons in Hong Kong are usually dug by a husband miner and wife winch operator in stages of about one metre depth. Each stage of excavation is lined with a minimum of 75 mm thickness of well-mixed 20 grade concrete

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using a tapered steel form suitably braced and designed for ease of striking. The shutter remains in place, providing support for the fresh lining and surrounding ground, whilst the next stage of excavation proceeds. Submersible electric pumps are commonly used for dewatering within caissons. Excavation through corestones and other obstructions is carried out by pneumatic drilling. Hand-dug caissons encounter difficulties in soft clays and sands (FABER, 1981) which are present in material such as colluvium, alluvium, or marine deposits. The difficulties may include collapse of the caisson side, heavy flow of water, piping and heaving in the base, and these could lead to settlement of surrounding area. The HONG KONG INSTITUTION OF ENGINEERS (1982) has produced a booklet **Guidance Notes On Construction and Safety of Hand-dug Caissons**, which goes a long way in overcoming these difficulties and providing safety requirements for construction.

Contiguous hand-dug caisson walls have been used to support deep basement excavations in areas of high groundwater, and a cheaper system of hand-dug caissons and concrete lagging has been adopted where groundwater is low (Figure 9). Although hand-dug caissons to support deep excavation may initially be more expensive than a steel sheet pile wall, they allow the



Fig. 9. Hand-Dug Caisson Wall with Concrete Lagging.

excavation to proceed without delay. They have also in some cases been incorporated as part of the permanent wall of the basement structure.

Ground Anchors

Temporary ground anchors have been used to support walls of deep basement excavations (Figure 10). The use of ground anchors instead of the usual cross-bracing allows excavation to proceed rapidly without the obstructions normally posed by the lateral support system. Basement excavation often covers the whole site, and therefore the anchors have to extend into the adjacent properties. This has restricted the use of anchors on sites adjacent to underground structures such as basements, tunnels and public utilities. In a recent 40 m deep excavation for a basement structure, rock anchors and rock bolts have been used to support the faces of excavation in granitic rock (MORTON et al, 1984). The upper portion of this excavation, in soil and decomposed rock, was supported by strutting.

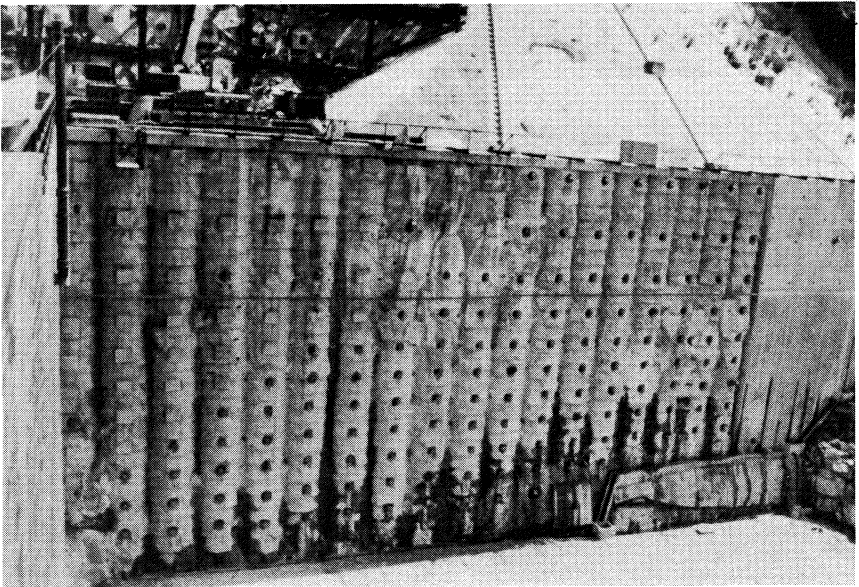


Fig. 10. Hand-Dug Caisson Wall with Temporary Ground Anchors.

Corrosion effects and long term integrity of anchors have been problems in Hong Kong. To improve this situation, the Geotechnical Control Office

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has issued a **Model Specification for Prestressed Ground Anchors** (BRIAN-BOYS & HOWELLS, 1984), which in addition to providing a specification also gives general guidance on the standards of installation of ground anchors.

Temporary Side Slopes

When excavation is on sloping ground, or when the ratio of excavation width to depth is large, it becomes convenient to provide temporary support to the steel sheet piles by side slopes, while raker supports are being installed to support the top waling. During the excavation works, the side slopes are kept dry by a well-point dewatering system, which might set up negative pore pressure and maintain the side slope at an angle even steeper than ϕ' of the soil. In such a situation, the usual wedge type stability analysis is often less helpful than experience. However, if gaps are left in the steel sheet pile wall due to the presence of obstructions, then excessive seepage might occur which could result in local failure of the side slope. The photograph in Figure 11 shows such a failure.

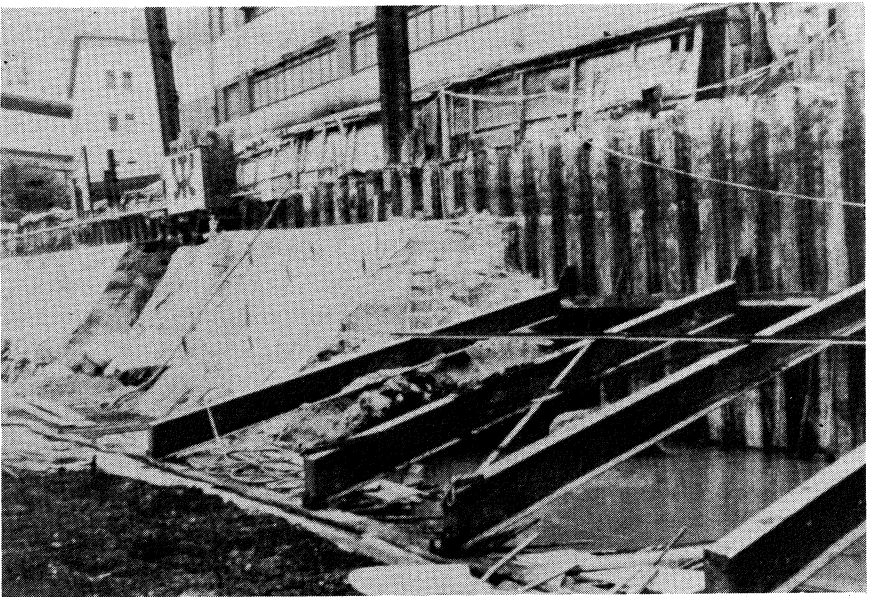


Fig. 11. Inadequate Toe Supports to Raking Struts; Left Side of Photograph Shows Collapse of the Side Slope in Front of the Steel Sheet Piling.

The side slopes should generally be provided with a top berm of about one metre width. This ensures that a definite height of slope is maintained. A 50 mm thick chunam surface plaster is also applied over the side slope to ensure the maintenance of negative pore pressure and to prevent erosion during heavy rainstorms.

The raker struts, with their bracing system, should be installed above the side slope so as to avoid cutting into it. The structural support to the raker struts should be adequate to take up both horizontal and vertical reactions. Often, horizontal support to the raker struts is obtained from pile caps which are not designed to resist such high horizontal forces. This problem can in most cases be overcome by the use of a ground beam between the pile caps, as shown in Figure 12.

Interim Construction Conditions and Workmanship

Since 1978 a number of steel sheet pile wall collapses have occurred in Hong Kong, and these have been illustrated by MALONE (1982). These failures have reinforced the belief (SOWERS, 1967) that the planning, design and construction of a lateral support system for deep excavations requires more than a knowledge of the mechanics of earth pressure and the design of structural systems.

A study of the collapse of steel sheet pile walls in Hong Kong indicates that they were caused by unrealistic design assumptions, insufficient details on working drawings and poor site supervision. The design assumptions often neglect or underestimate groundwater pressures due to leakage or breakage of water mains or storm water pipes, and over simplify the bracing system required for raker struts. Lack of adequate details of the joint between the raker struts and the walings to support the raker struts, insufficient description of the required construction sequence on the working drawings, and limited site supervision have led to material short cuts and premature removal of struts in an anxiety to speed up the construction. To avoid these, plans showing the lateral support details should indicate:

- (a) The sequence of installation of the lateral support system as the excavation proceeds; if the lowest strut cannot be conveniently located to avoid pile cap and basement slab construction, then it will be necessary to concrete it in with the basement structure.
- (b) The order in which the struts should be dismantled as the basement construction moves upwards.

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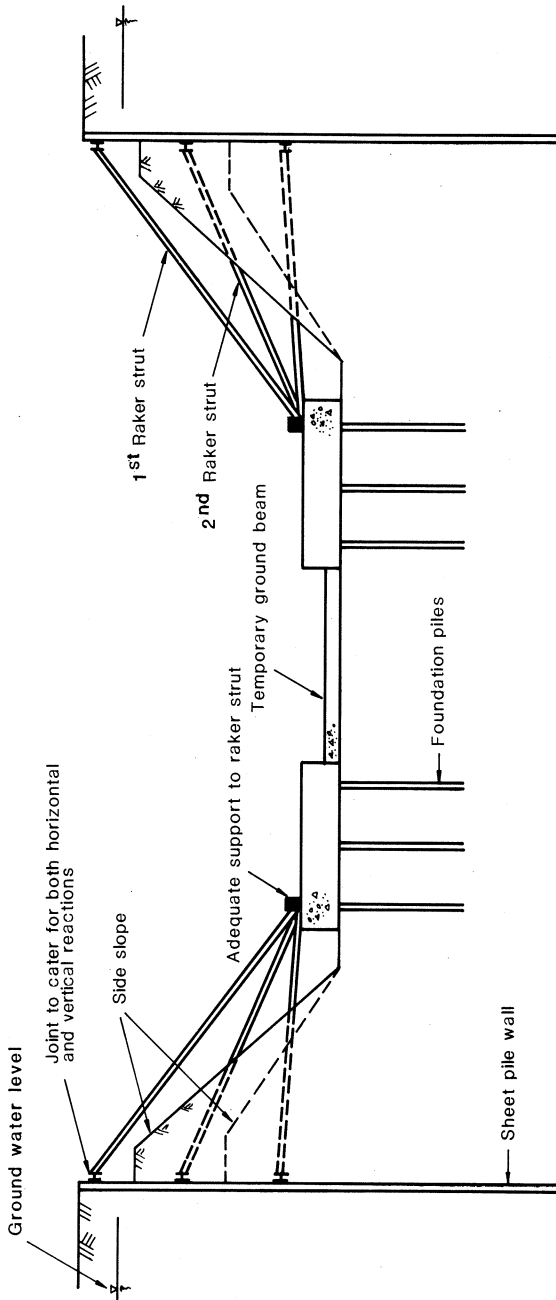


Fig. 12. Raker Supports with Temporary Side Slope.

- (c) Appropriate means of posting the struts and walings.
- (d) Lacing of struts.
- (e) Provision of wedging or jacking to maintain tight contact for all bracing members and to provide for uniformity of distribution of loads to struts and walings.

MONITORING

The primary problem associated with basement construction is excessive ground movement in the proximity of the site. Theoretical assessment of this movement however gives no more than an order of magnitude of the movement. It is therefore a normal procedure in Hong Kong to set up a monitoring system during the construction stage to guard against the occurrence of undesirable ground movement. Hence, any sign of excessive movement is revealed and remedial works and precautionary measures can be taken in due time.

As result of monitoring, precautionary measures have been adopted on a number of sites in Hong Kong to limit excessive movement on adjacent sites. Observed settlements of buildings and land adjacent to deep excavations for the Hong Kong underground railway stations were reported by MORTON et al (1980a, 1980b).

Monitoring of a Site in Kowloon

A typical monitoring system for a 13 m deep basement excavation site of about 70 m by 40 m at Hung Hom, on the Kowloon Peninsula, consisted of 26 settlement check points and four standpipe piezometers installed outside the four corners of the lot (Figure 13). The lot was bounded on all four sides by major roads carrying a number of utilities, including a 600 mm diameter fresh water main. The site is located in a typical reclaimed area, consisting of a sequence of 5 to 7 m of fill, 2 to 6 m of marine deposits and 2 to 8 m of alluvial deposits overlying decomposed granite.

The proposed lateral support system for the excavated faces at the Hung Hom site consisted of FSP III steel sheet piles driven to a depth of 19 to 21 m, a cross-bracing system with five layers of horizontal struts and preloading of the first four layers to half of their design load. Since the site investigation indicated the presence of a dense layer of decomposed granite with the possible presence of corestones, it was envisaged that the steel sheet piles may not reach the required penetration to provide sufficient depth of cut-off to prevent

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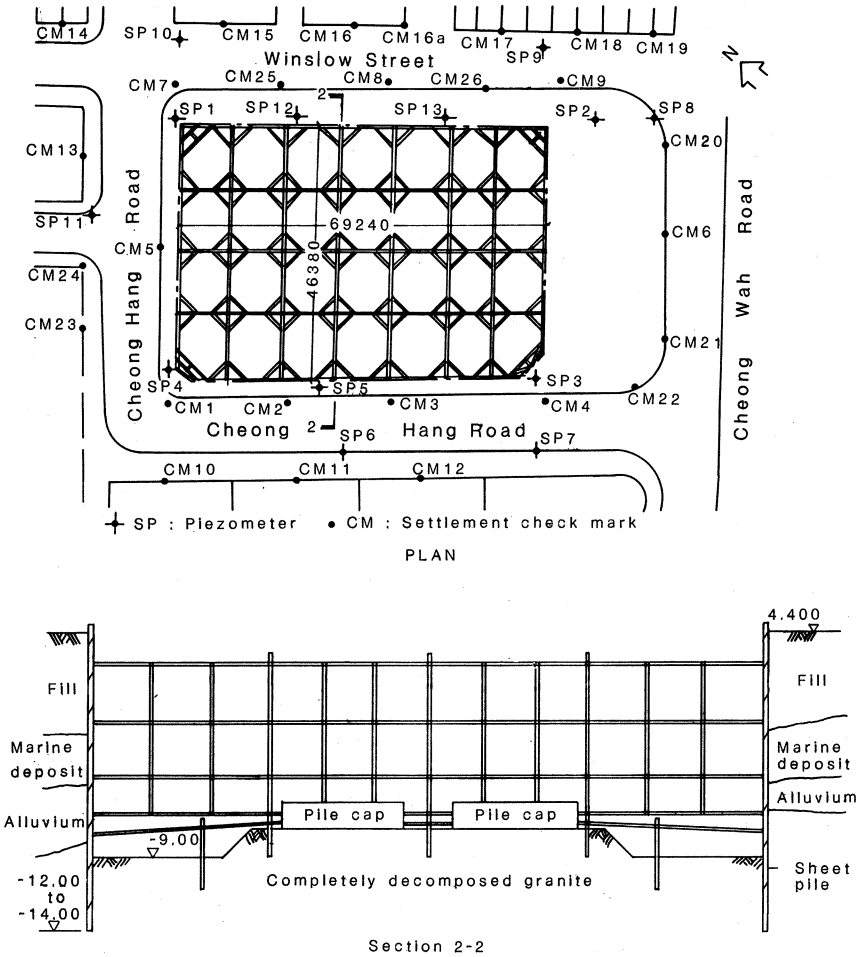


Fig. 13. Shoring and Monitoring System used at Hung Hom Site.

excessive drawdown of groundwater. Provision for a grout curtain below the toe of the steel sheet piles without adequate penetration was therefore included in the proposal.

Initial driving of the steel sheet piles achieved maximum penetration of only 16 to 18 m instead of the required value of 19 to 21 m. The efforts of

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the contractor to redrive the piles to greater depth were not successful. He concluded after some hard driving that the piles had toed into the hard decomposed granite, and hence there was no need for the provision of the grout curtain as originally envisaged. He then proceeded to dewater the site through four deep wells, and to excavate and provide shoring in the usual manner.

When excavation reached a depth of about 9 m, groundwater drawdown reached about 4 m and a settlement of 20 mm was observed. This excessive drawdown caused concern, and excavation works were temporarily suspended and remedial measures commenced. The immediate remedial measures included:

- (i) abandonment of dewatering through the four deep wells;
- (ii) limiting the excavation to the central portion of the site and dewatering through shallow sump pumps to keep the site workably dry;
- (iii) localised bentonite/chemical grouting of seven suspected areas of leakage through the steel sheet piles; and
- (iv) installation of two multi-level piezometers and five standpipe piezometers.

These immediate remedial measures, although effective in stabilising the groundwater level, did not arrest the settlement and drawdown along Winslow Street. In order to limit the settlement and drawdown along that street, which carries the 600 mm water main, curtain grouting was provided from one metre above the toe of the sheet piles to a depth of about 22.5 m. Under these remedial measures, the groundwater recovered, settlement was arrested and only one settlement check mark showed a further settlement of 2 mm. Figures 14 and 15 show the monitoring records and the increase in ground settlement, along with the drawdown of the groundwater level and its subsequent recovery with the implementation of the remedial measures.

CONCLUSIONS

The major factors which influence the design, construction and control of deep basement excavations in urban areas of Hong Kong have been described. Ground conditions in Hong Kong are very variable and obstructions in the form of corestones, boulders and old foundations are a major source of problems for the installation of lateral support systems for deep excavations. The assessment of site conditions should include the collection of data on the geological strata and past history of the site sufficient to make decisions on

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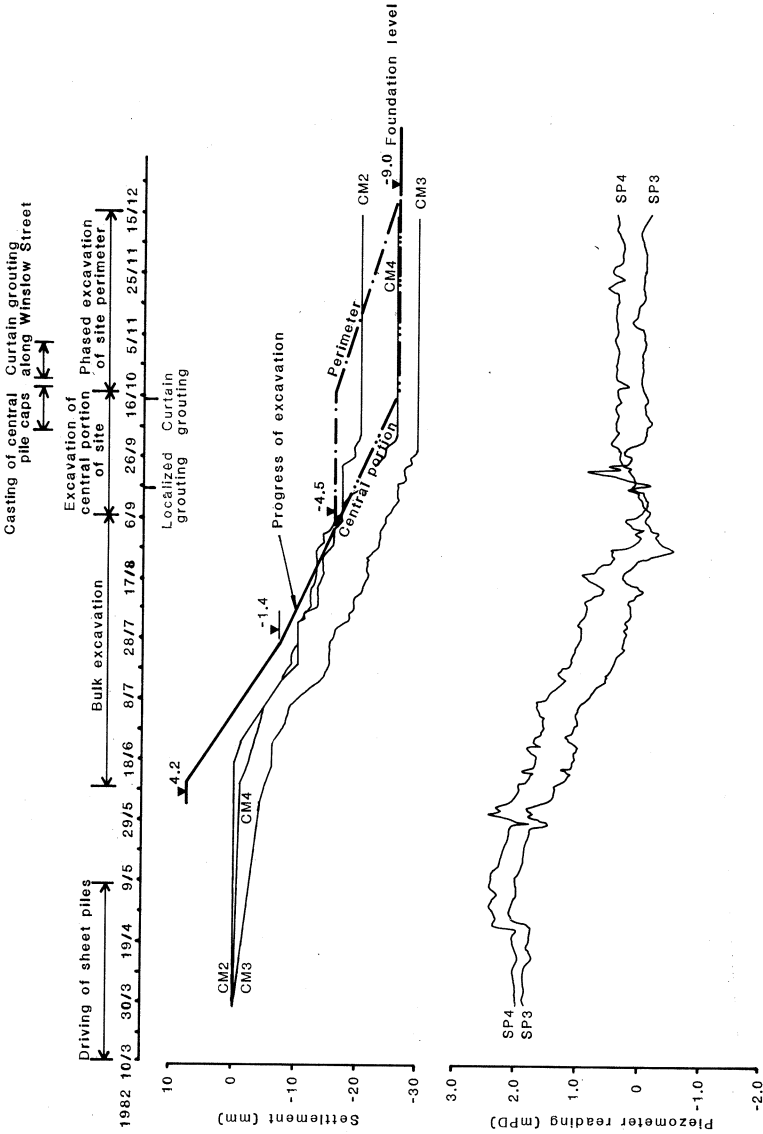


Fig. 14. Monitoring Records along Cheong Hang Road.

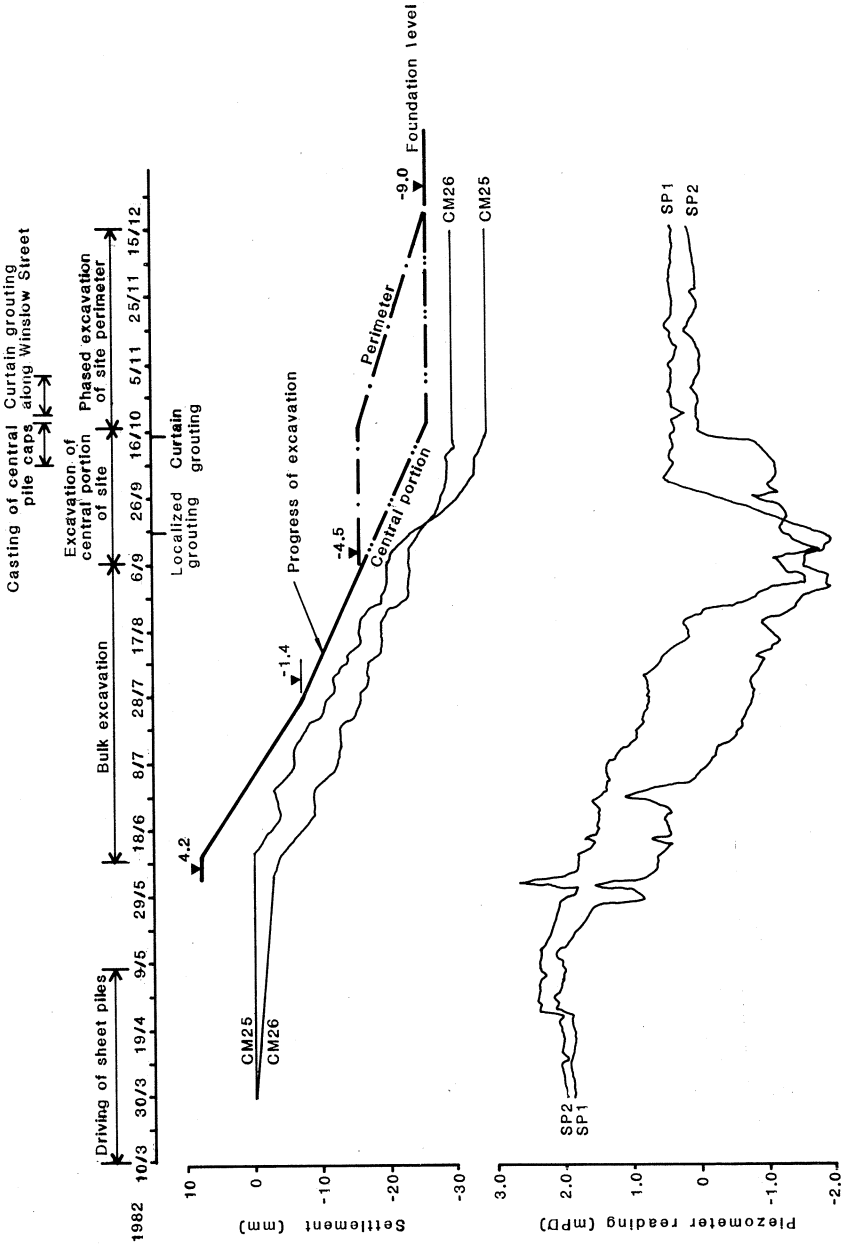


Fig. 15. Monitoring Records along Winslow Street.

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the type of lateral support that could be installed to the required depth of penetration. The site investigation should be aimed at determining the extent of the actual variation in the soil strata and the groundwater conditions.

Knowledge of the groundwater conditions and the location of water carrying utilities is of paramount importance to the design of a lateral support system. Bursting of water mains adjacent to deep excavations and collapse of the lateral support systems has occurred. It is therefore prudent for the designer, when the site is adjacent to water mains which cannot be conveniently isolated, to assume the groundwater level to be above the observed value to ensure that any leakage from the water pipes does not cause total collapse of the support system.

The cut-off provided by the support system should be adequate to prevent excessive groundwater drawdown which results in ground movements. It is often not possible to provide adequate depth of penetration with steel sheet piling in dense ground or in areas with obstructions. In such situations a grout curtain extending below the toe of the steel sheet piles has arrested excessive groundwater drawdown.

The detailing on the working drawings for the temporary lateral support system for deep excavations may not be to the same standard as the permanent works. In many cases, the details of the connections and support to the raker struts are left to the site staff. This situation is undesirable on a deep excavation site where the risk is high and where the site staff, in their anxiety to execute the work to the programme, cut corners without realising the consequences. To avoid such a dangerous situation, it is important for the designer to clearly detail the working drawings and to insist that any variations be agreed prior to execution.

Instrumentation has become an inseparable part of the design of a support system for a deep excavation where the high complexity of the geological and geotechnical data make it impossible to fully predict its behaviour at the design stage. Instrumentation serves two main purposes; firstly, to monitor safety and provide warning for precautionary measures to be taken during construction, and secondly, to check the behaviour for possible revisions to the design. A simple monitoring system consists of a sufficient number of ground and building settlement markers and piezometers. The ground settlement markers should be placed well into the soil (extending below the existing pavement) to reflect the true ground settlements. Multi-level piezometers are suggested as being necessary in areas of complex geology to identify

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the seepage pattern. Inclined meters are useful to detect lateral movements of steel sheet piles and diaphragm walls in soft material.

Practically no data is available in Hong Kong on the loads transferred to the struts in deep basement construction. There is wide scope for greater monitoring of strut loads and pore water pressures encountered with deep excavations.

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A STUDY OF THE PROPERTIES OF SURFACE SOILS IN KUWAIT

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SYNOPSIS

The surface soils of Kuwait and large areas of the Arabian Peninsula consist of fine windblown dune sand which varies in depth. Many foundations and earth structures are placed in this deposit and to determine the geotechnical properties of these soils in Kuwait a detailed laboratory testing program was carried out. Samples were taken from nine sites along a 35 km long, 2 km wide corridor extending from Salmiya to Andalus in the east-west direction. Laboratory work included determination of physical properties, chemical composition, compaction, direct shear and California Bearing Ratio (CBR) tests. The effect of soaking on strength and CBR value was assessed. The results indicate uniformity of soil conditions at the test sites and some loss of strength due to saturation. Based on test results, recommendations are given for preliminary design of foundations and earth structures placed in this soil deposit.

INTRODUCTION

The rapid development of Kuwait in the last ten years has resulted in major engineering projects and construction work. Among the projects carried out were construction of housing, schools, hospitals, office buildings, light industries, and a new highway system. With this major work, interest in the geotechnical properties and behavior of subsurface soils intensified because of the importance of soil conditions in the selection, design, and construction of foundations for various structures. Different construction and earthwork activities such as excavation, lateral support, dewatering, and foundation installation techniques depend on the type and properties of ground soils at the site and the depth of the ground water table.

Recent studies indicated that the ground is in general a flat, gently undulating desert plain with occasional low hills, escarpments, and depressions (AL-SALEH & KHALAF, 1982). The surface and near surface formations are of sedimentary origin ranging in age from Eocene to Recent. The soil profile typically consists of a surface layer of windblown dune sand extending to a depth of 7 m. It is underlain by a more competent marine silty sand containing between 5 and 35% of fines, which is usually cemented. This latter

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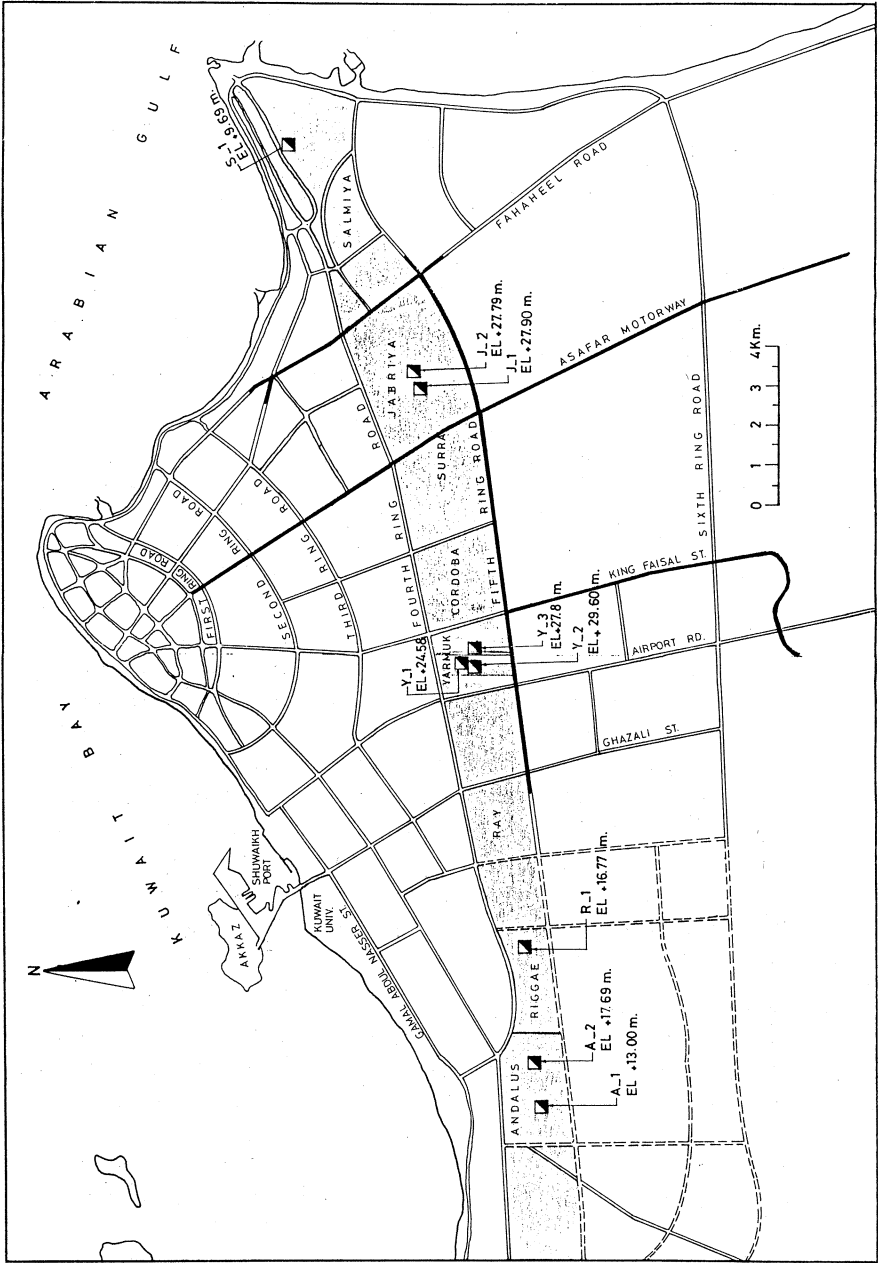


Fig. 1. Site Locations along Corridor.

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deposit, known locally as "gatch", extends to a great depth over limestone bedrock (ALLISON, 1969).

The arid environment of Kuwait and the excess of evaporation over rainfall leads to upward movement of groundwater and the concentration of soluble materials at or near the ground surface, enriching the soil with gypsum and carbonates and leading to the formation of crusts of cemented soils (FOOKES & HIGGINBOTTOM, 1980). The carbonate material enclosing the mainly quartz soil particles consists of calcite and/or dolomite. In some cases gypsum (calcium sulphate) exists in the soil matrix and acts as a binder.

To determine the geotechnical properties of surface soils, a study area was selected along a corridor extending longitudinally from Salmiya in the east to Andalus in the west and bounded laterally by the fourth and the fifth ring roads, Figure 1. The corridor is approximately 35 km long by 2 km wide. Samples were recovered from the upper windblown dune sand at nine locations in the area for laboratory testing. This paper presents a summary of the geotechnical properties including boring logs, penetration test results, basic physical properties, and compaction, strength, and California Bearing Ratio (CBR) test results. The effect of soaking on the strength parameters and CBR values is also reported.

SAMPLING AND TESTING PROGRAM

Nine sites were selected along the corridor for sampling as shown in Figure 1. These sites are located within five major areas, namely, Andalus, Riggae, Yarmouk, Jabriya and Salmiya. A total of 24 disturbed samples were taken from various depths ranging between 0.5 and 6 m. These samples were taken from the side walls of excavations for foundation construction, or from auger borings in conjunction with Standard Penetration Tests (SPT). They were used for classification tests including visual classification and mechanical analyses. Atterberg limit tests were carried out on the fraction passing the No. 40 U.S. sieve.

In addition to the above mentioned samples, five 0.4 to 0.5 m cube undisturbed block samples were cut from the bottom of excavation pits at depths from 1.0 to 1.5 m using long knives and saws. The samples were taken from five selected sites, namely A-1, R-1, Y-1, J-1, and S-1 where the soil displayed very slight cementation which facilitated the cutting of large blocks. These samples were carefully wrapped in large plastic bags and transported to the laboratory for testing. The testing included determination of natural mois-

ture content, unit weight, specific gravity, relative density, compaction and permeability. Direct shear and CBR tests were performed on samples at their natural moisture content and after soaking to determine strength parameters and CBR values, and the effect of increased water content on these parameters. Detailed chemical analyses were performed on samples from all sites. Table 1 shows the locations and details of the samples taken in this testing program.

SOIL PROFILE AND BASIC PROPERTIES

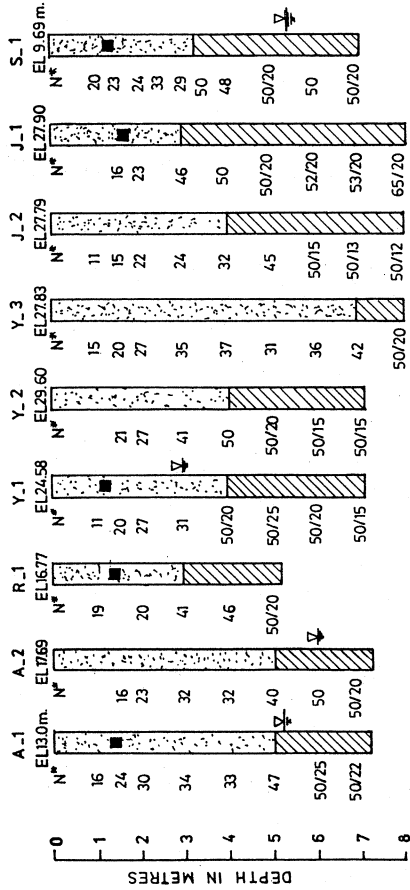
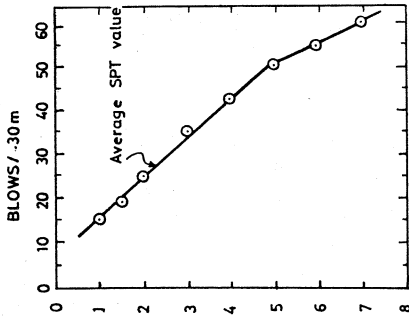
The soil conditions at the test sites along the corridor are shown in Figure 2. The soil profile typically consists of a surface layer of windblown sand underlain by a more competent marine deposited silty fine to medium

Table 1. Details of Sample Locations.

Area	Site	Block/Plot	Sample No.	Depth (m)	Remarks
Andalus	A-1	3/70	1	1.5	Block sample*
			2	2.0	
			3	3	
			4	4	
	A-2	5/103	5	1.0	
			6	2.0	
			7	4.0	
Riggae	R-1	Clinic	8	1.0	Block sample
			9	1.5	
Yarmouk	Y-1	1/278	10	1.0	Block sample
			11	1.5	
	Y-2	2/103	12	1.0	
			13	1.5	
			14	2.0	
			15	3.0	
	Y-3	2/139	16	1.0	
17			1.5		
Jabriya	J-1	5/26	18	1.0	Block sample
	J-2	5/363	19	1.0	
Salmiya	S-1	Girls Secondary School	20	0.5	Block sample
			21	1.0	
			22	1.5	
			23	2.0	
			24	2.5	

*All block samples were taken from the bottom of excavation pits at a depth of 1.0–1.5 m.

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- WIND BLOWN SAND : BROWN CALCAREOUS DRY FINE SAND WITH TRACES OF SILT & COARSE SAND (SP-SM)
- BROWNISH GREY CALCAREOUS SILTY FINE TO MEDIUM CEMENTED SAND WITH TRACES OF COARSER FRACTIONS (SM-SC)
- BLOCK SAMPLING LOCATIONS
- * SPT VALUE (BLOWS PER 30 cm. UNLESS OTHERWISE INDICATED)

Fig. 2. Soil Profiles at the Test Sites.

Table 2. Summary of Classification Test Results.

Location	Depth of Upper Layer (m)	% Passing U.S. Sieves					Soil Description (ASTM D422)			Coeff. of Unif. C _u	Coeff. of Curv. C _z	Unified Soil Classification
		# 4 4.76 mm.	# 10 2.00 mm.	# 16 1.18 mm.	# 40 0.425 mm.	# 100 0.150 mm.	# 200 0.075 mm.	CS	MS			
A-1	1.5	100	94.4	89.3	86.0	62.8	15.2	5.6	8.4	70.8	15.2	SM
	2.0	100	94.6	88.1	84.8	59.5	15.5	5.4	9.8	69.3	15.5	SM
	3.0	100	97.4	90.4	81.8	56.7	9.7	2.6	15.6	72.1	9.7	SP-SM
	4.0	100	98.9	90.8	79.1	54.6	12.6	1.1	19.8	66.5	12.6	SM
A-2	1.0	100	95.2	90.4	86.8	52.9	8.6	4.8	8.4	78.2	8.6	SP-SM
	5.0	100	95.6	89.8	84.3	55.6	11.3	4.4	11.3	73.0	11.3	SP-SM
R-1	4.0	100	98.1	89.7	82.5	58.4	11.9	1.9	15.6	71.6	10.9	SP-SM
	1.0	100	96.7	93.6	89.4	52.6	6.1	3.3	7.3	83.3	6.1	SP-SM
Y-1	1.5	100	94.9	90.7	85.7	54.7	9.1	5.1	9.2	76.6	9.1	SP-SM
	1.0	100	97.7	86.3	81.1	50.5	8.9	2.3	16.6	72.2	8.9	SP-SM
Y-2	1.5	100	98.6	89.3	79.3	52.9	11.0	1.4	19.3	68.3	11.0	SP-SM
	4.0	100	98.0	90.1	87.9	42.8	14.8	2.0	10.1	73.1	14.8	SM
Y-3	1.5	100	96.5	90.7	87.2	56.5	10.5	3.5	9.3	76.7	10.5	SP-SM
	2.0	100	97.4	85.7	76.6	45.6	6.9	2.6	20.8	69.7	6.9	SP-SM
	3.0	100	98.4	88.6	82.2	47.4	10.7	1.6	16.2	71.5	10.7	SP-SM
	7.0	100	96.8	89.2	81.4	52.1	10.4	3.2	15.4	71.0	10.4	SP-SM
J-1	1.5	100	98.9	91.8	85.8	51.2	10.6	1.1	13.1	75.2	10.6	SP-SM
	4.0	100	97.5	87.7	79.0	47.6	5.2	2.5	18.5	73.8	5.2	SP-SM
J-2	1.0	100	97.8	92.1	88.3	55.6	7.2	2.2	9.5	81.1	7.2	SP-SM
	3.0	100	91.9	84.2	79.9	54.8	10.8	8.1	12.0	69.1	10.8	SP-SM
S-1	1.0	100	93.1	82.4	78.0	54.0	9.7	6.9	15.1	68.3	9.7	SP-SM
	1.5	100	95.5	84.5	79.9	51.2	9.3	4.5	15.6	70.6	9.3	SP-SM
	2.0	100	95.6	82.0	74.4	43.2	9.2	4.4	21.2	65.2	9.2	SP-SM
	2.5	100	96.3	82.5	78.1	48.5	8.1	3.7	18.2	70.0	8.1	SP-SM
Average	Range	100	91.9-98.9	93.6-82.0	74.4-89.4	42.8-62.8	5.2-15.5	1.1-8.1	7.3-20.8	65.2-83.3	5.2-15.5	
	Average	100	96.5	88.3	82.5	52.6	10.1	3.5	14.0	72.4	10.1	

Legend: CS-Coarse Sand (4.75-2mm) MS-Medium Sand (2-425 mm)
 FS-Fine Sand (.425-.075 mm) S&C-Silt & Clay (< .075 mm)

Remarks: All samples non-plastic; linear shrinkage = 0%.

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sand known as "gatch". The gatch, which is usually cemented, is relatively impervious causing perched water to appear over its surface at some locations. The ground elevations at the test sites along the corridor were taken from the Ministry of Public Works projects, and airphoto maps provided by Kuwait Municipality. The ground is highest at the Yarmouk area in the center of the corridor. It slopes gently toward the direction of the coasts. The lowest elevations are at the Salmiya sites where ground level is approximately 10 m above Kuwait Land Datum (KLD).

The soil profiles shown in Figure 2 indicate that the thickness of the upper layer varies between 3 m and 7 m at the test sites along the corridor. The standard penetration resistance "N" values in this layer range between 15 and 35, and they generally increase in magnitude linearly with depth. The ground water level is marked on the borehole logs, Figure 2. As shown, it is located below the surface layer in the majority of the boreholes.

Static Cone Tests (CPT) were carried out at the sites A-1, R-1, Y-1, and S-1. At least three cone tests were performed in the vicinity of each boring. The cone used had the standard 10 cm² base area, and an apex angle of 60°. The friction sleeve, located above the conical tip and of the same diameter, had a standard area of 150 cm². The cone had a built-in load cell that record continuously the end resistance (q_c) in kg/cm² and side friction (f_s). An electric cable connected the cone with the recording equipment at the ground surface.

The results indicate that the average ratio of $\frac{q_c}{N}$ in the upper layer varies from 4.2 for the R-1 site to 5.6 for the A-1 site, with an overall average ratio of 4.8 for all sites. This relationship for the local surface soils may be useful for prediction of foundation settlement employing methods based on the Static Cone Tests at locations where only SPT test data are available.

A summary of the classification test results on the 24 samples taken from the various depths of the upper layer is given in Table 2. The material is predominantly fine sand with no gravel whatsoever and very little fines. The percent of fines passing the No. 200 U.S. sieve ranges between 5 and 15%, and on average it constitutes 10% of the total composition. The upper and lower bounds of the grain size distribution curves for all samples are shown in Figure 3. Noting the great length of the corridor, this range of variation is considered to be very narrow indeed.

The index properties determined on the fraction passing the U.S. No. 40 sieve indicated that all samples were non-plastic. The Unified Soil Classifica-

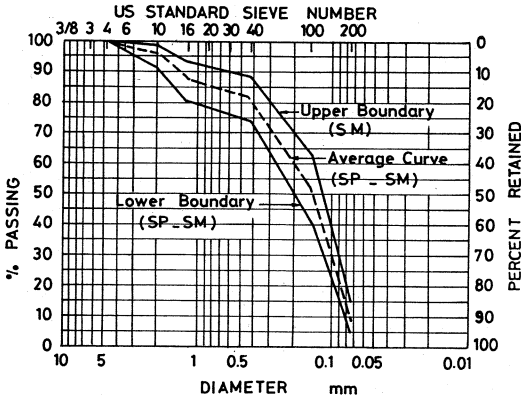


Fig. 3. Grain Size Distribution Curves.

tion designation of the majority of samples is SP→SM (dual classification of poorly graded sand to silty sand).

The physical properties of the five samples selected for detailed testing are summarized in Table 3, which includes natural moisture content, bulk and dry unit weights, specific gravity, median grain diameter, void ratio, degree of saturation, minimum and maximum dry densities, relative density, and the coefficient of permeability. Examination of Table 3 reveals the similarity of the soil properties at the different sites. The subsoils are in a relatively dry condition with natural moisture contents below 2%, and degrees of saturation between 4 and 8%. The median grain diameter is nearly constant at 0.14 to 0.16 mm, and the specific gravity varies between 2.67 and 2.72. The relative density averages nearly 70%, indicating medium dense to dense soil. The coefficient of permeability values were in the range 10^{-3} to 10^{-4} cm/sec, indicating a free draining sandy soil.

Complete chemical analyses on selected samples from different sites are given in Table 4. As indicated, quartz is the principal component. The amount of carbonates varies between 10 and 20% mostly in the form of calcite (calcium carbonate). With the exception of one site, A-2, in Andalus, the concentration of sulphates SO_3 in the samples was below 1%.

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Table 3. Summary of Physical Properties, 1-1.5 m Depth.

Location	Moisture Content w (%)	Bulk Unit Weight γ_b (kg/m ³)	Dry Unit Weight γ_d (kg/m ³)	Specific Gravity of Solids G_s	D ₅₀ (mm)	Void Ratio e	Degree of Saturation S_r (%)	Minimum Density γ_d min (kg/m ³)	Maximum Dry Density γ_d max (kg/m ³)	Relative Density R_d (%)	Coefficient of Permeability ($\times 10^{-3}$ cm/sec)
A-1	1.8	1717	1687	2.72	0.15	0.612	8.0	1521	1789	65.7	2.0
R-1	1.0	1727	1710	2.69	0.14	0.591	4.6	1541	1769	76.7	1.73
Y-1	1.1	1734	1715	2.72	0.15	0.586	5.1	1574	1783	70.1	0.93
J-1	0.9	1719	1704	2.72	0.16	0.596	4.1	1564	1792	64.6	0.26
S-1	1.5	1766	1740	2.67	0.16	0.563	7.2	1577	1779	72.4	0.64

*As per ASTM D 2049 using the dry method for maximum density.

Table 4. Chemical Composition of the Soil Samples, 1-1.5 m Depth.

Site	pH	Composition (%)													
		SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Org. Matter	CO ₂	CO ₃	CaSO ₄	CaCO ₃	MgCO ₃	Loss in Ignition	SO ₃ ²⁻	Cl-
A-1	7.65	71.86	9.30	1.04	5.62	2.82	*	8.41	11.46	1.16	10.03	6.81	*	0.850	0.018
A-2	*	64.36	8.88	0.88	12.26	2.42	*	9.63	13.13	*	21.89	*	*	1.60	0.020
R-1	8.0	63.56	11.44	0.80	11.76	0.25	1.16	9.24	10.62	0.75	17.70	*	*	8.95	0.44
Y-1	8.2	71.00	9.32	0.88	8.61	1.25	1.19	7.62	10.39	*	15.38	2.1	*	8.84	0.052
Y-2	8.5	66.84	8.16	0.88	12.12	2.09	*	8.25	11.25	*	18.75	*	*	0.025	0.014
J-1	*	73.12	6.48	0.72	7.98	1.50	0.073	6.27	8.47	*	14.26	*	*	*	0.018
J-2	*	66.74	8.22	1.12	12.22	2.60	*	8.42	11.48	*	19.13	*	*	0.036	0.016
S-1	8.7	76.90	5.62	0.52	6.30	1.08	0.78	6.93	9.45	1.01	11.25	3.06	*	0.025	0.021

*Not measured

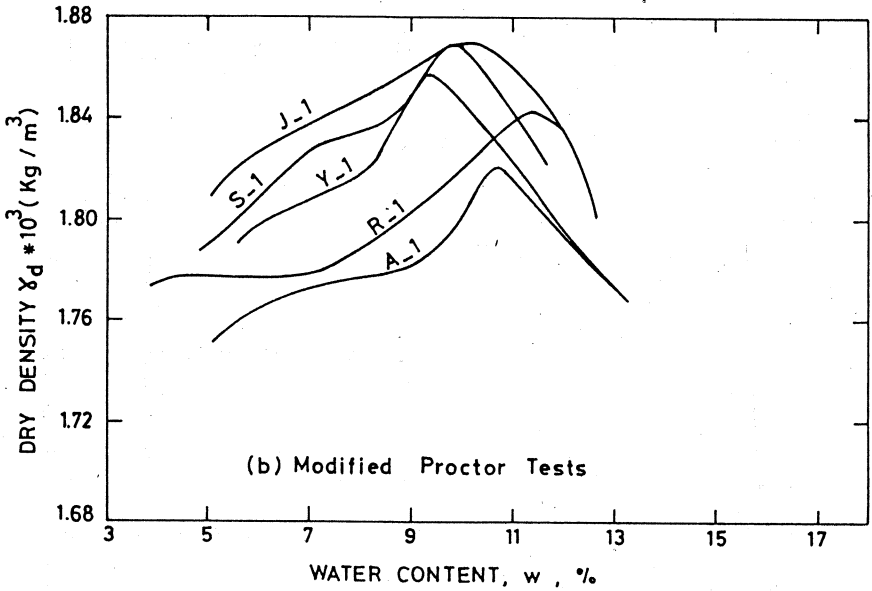
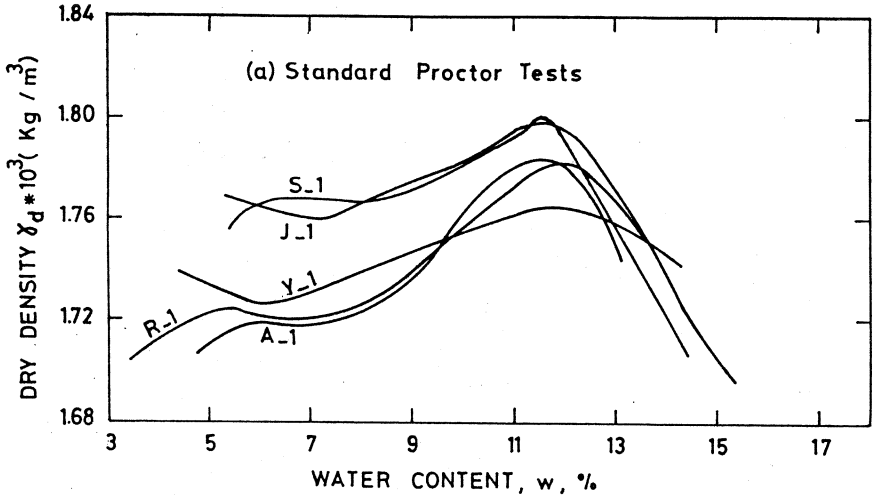


Fig. 4. Compaction Curves for Samples from Five Sites.

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COMPACTION CHARACTERISTICS

Compaction tests were conducted using both the standard and modified Proctor methods. The results are shown in Figure 4 in the form of dry density (γ_d) versus moisture content curves. Figure 4a shows the results of the standard (2.5 kg rammer) tests. An initial reduction in density is obtained at low moisture content followed by a gradual increase to the peak values. This phenomenon, known as bulking (LAMBE & WHITMAN, 1969), is a characteristic of cohesionless soils where capillary forces resist rearrangement of the grains at low moisture contents. Compaction curves using the Modified Proctor (4.5 kg rammer) method given in Figure 4b show a small increase in density at low moisture contents. A sharp fall in density occurs on the wet side of optimum in both cases. In general, flat curves of this nature indicate a poorly graded soil.

A summary of the compaction test results is given in Table 5. The maximum dry density and the corresponding optimum moisture contents from the Standard Proctor tests are 1765 to 1800 kg/m³ and 11.5 to 12.6 % respectively. The corresponding values for the Modified Proctor tests are 1820 to 1870 kg/m³ and 9.5 to 11.5%. As expected, this represents an increase in density at lower optimum moisture content as compared to the Standard Proctor values. The effect of compaction effort on soil improvement is demonstrated in Figure 5 for the Yarmouk Y-1 site. The maximum dry density increased by 5% using the 4.5 kg rammer against the 2.5 kg rammer, with a corresponding decrease of more than 2% in optimum moisture content.

Table 5. Compaction Characteristics of Surface Soils, 1-1.5 m Depth.

Sample	Standard Proctor (2.5 kg rammer)		Modified Proctor (4.5 kg rammer)	
	Optimum Moisture Content w_{opt} (%)	Maximum Dry Density γ_d max (kg/m ³)	Optimum Moisture Content w_{opt} (%)	Maximum Dry Density γ_d max (kg/m ³)
A-1	11.5	1785	11.0	1820
R-1	12.0	1785	11.5	1840
Y-1	12.0	1765	9.5	1855
J-1	11.5	1800	10.5	1870
S-1	11.5	1800	10.0	1870

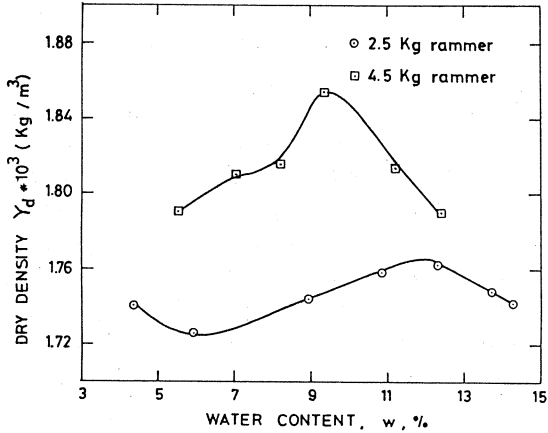


Fig. 5. Comparison of Soil Improvement by Different Compactive Effort.

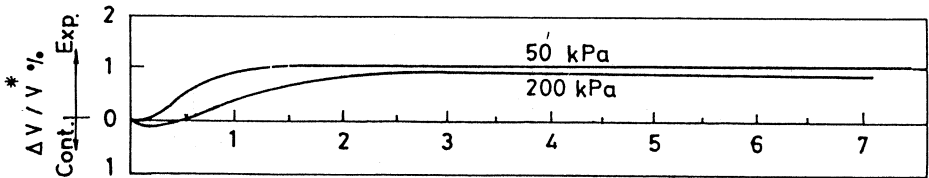
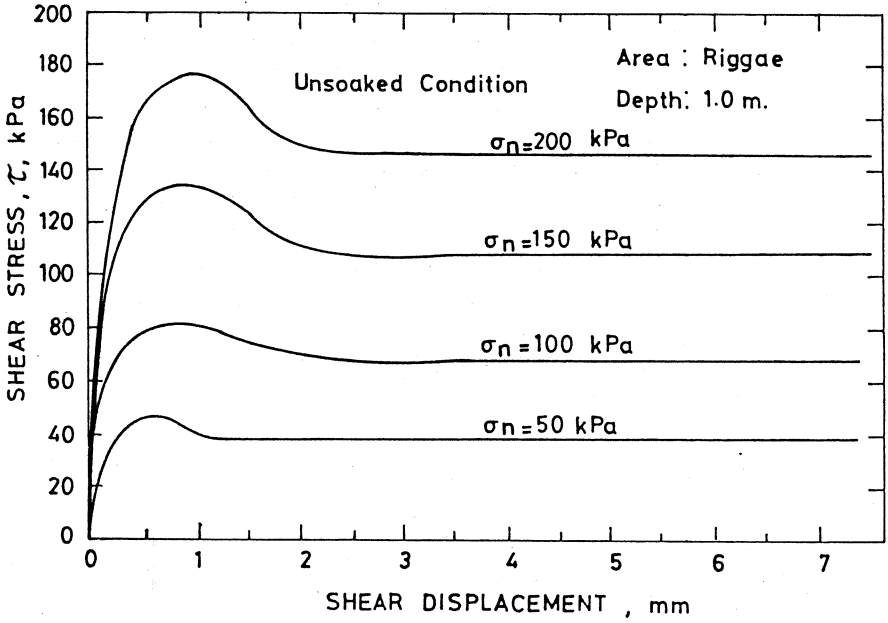
The results in Figure 4 and Table 5 reveal the similarity of the surface soils at the different sites as reflected by the narrow range of variation in the maximum dry density and optimum moisture content. It is also apparent that the specification of 95% relative compaction for fills using this soil may not be appropriate, as it yields densities below the minimum indicated by the compaction curves (Figure 4) and corresponds to low relative densities. It is therefore advisable to specify higher relative compaction, in the 98% to 100% range, to achieve proper compaction with these cohesionless soils.

DIRECT SHEAR TESTS

Two sets of direct shear tests were conducted following ASTM D-3080 on undisturbed samples at both insitu moisture content and after soaking for 24 hours. Soaking resulted in full saturation of laboratory samples (degree of saturation, $S_r = 100\%$). The moisture content of these samples increased 20% during soaking (e.g. from 2% to 22%). All tests were performed at small rates of strain to ensure total dissipation of pore water pressure during shear.

Stress-displacement curves and volume change-displacement curves are plotted for R-1 sand in Figure 6 for the insitu moisture condition. As shown, the stiffness and peak strength increase with increased normal loading. The volume change data indicate the development of strong dilation at small dis-

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* $\Delta V / V = \Delta H / H$

Fig. 6. Stress-Displacement & Volume Change Curves from Direct Shear Tests—Unsoaked Condition.

placements particularly with small normal pressures. The stress-displacement data in Figure 6 show a ductile failure mode having no significant drop in strength after reaching peak. However, if a comparison is made between the shear stress-displacement data for unsoaked and soaked specimens at the same normal pressure, it will appear that soaked specimens have a much more ductile failure than samples tested at insitu moisture content (Figure 7). It is also evident that soaking leads to a reduction in shear strength. Figure 7 indicates that the shear strength is higher and occurs at smaller displacements for samples tested at insitu moisture content. The results were similar for the other sites. All the samples tested were undisturbed samples trimmed from block samples. They possessed weak cementation or bonding which could easily be destroyed by slight finger pressure.

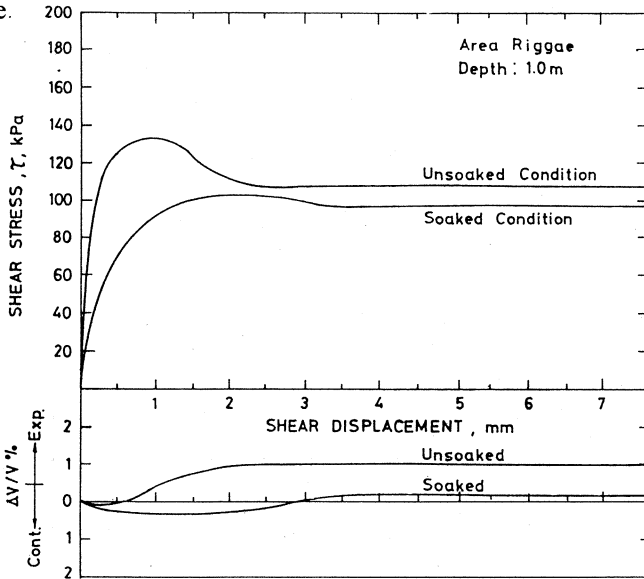


Fig. 7. Comparison of Stress-Displacement & Volume Change for Soaked and Unsoaked Specimens ($\sigma_n = 150$ kPa)

A summary of the peak strength parameters for the unsoaked and soaked conditions is given in Table 6 for four sites, along with the residual unsoaked parameters and the values predicted from the empirical relation between the SPT "N" values and angle of friction ϕ given by PECK *et al* (1974) and reproduced in Figure 8. For the fifth site (A-1) samples could not be trimmed properly to the required size, and they crushed prior to placement in the shear box.

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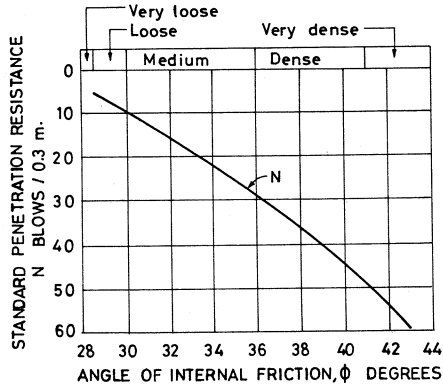


Fig. 8. Curve Showing the Relationship Between N and ϕ (after PECK et al, 1974).

The presence of a cohesion intercept of the order of 4 to 24 kPa is due to slight bonding or interparticle cementation which exists at some locations. This may explain why windblown sand may stand with steep or even vertical slopes in excavations at some locations. The soaked specimens show no cohesion, and an average angle of friction of 35° compared to 40° for unsoaked conditions.

Plots of the peak shear stress versus normal stress showing the Mohr-Coulomb envelopes for the test soils in the unsoaked and soaked conditions are shown in Figure 9. The loss of strength due to soaking implies loss of bearing capacity and reveals the sensitivity of surface soils to increasing moisture and the possible deterioration of steep slopes if subjected to heavy rain during the winter season. In such instances full saturation ($S_r = 100\%$) is likely to occur until drainage and partial drying take place.

Table 6. Summary of Direct Shear Strength Parameters for Surface Soils.

Site	SPT N Blows/ 0.3 m	Peak Unsoaked		Residual Unsoaked		Peak Soaked		Predicted ϕ after PECK et al (1974)
		c (kPa)	ϕ (deg)	c (kPa)	ϕ (deg)	c_s (kPa)	ϕ_s (deg)	
R-1	20	4	40.5	0	36	0	35.5	33.5
Y-1	20	0	40.9	0	37	0	35.5	33.5
J-1	18	4	41.7	0	36	0	35.8	33.0
S-1	23	24	40.4	0	35	0	35.5	34.0

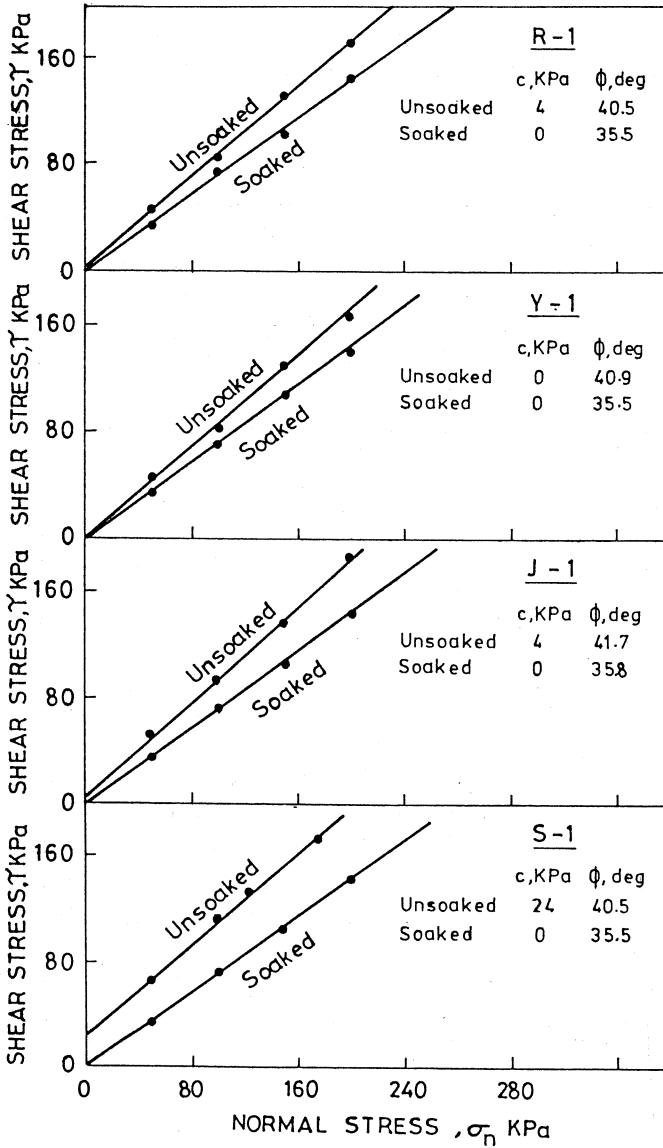


Fig. 9. Mohr-Coulomb Envelopes for Soaked and Unsoaked Specimens.

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The predicted angles of friction shown in Table 6 range between 33° and 34° and average 33.5°. They are smaller than the measured soaked value by 2° and as such, they are on the conservative side. Residual strength parameters for unsoaked conditions are similar to peak soaked parameters. This may be explained by noting that small interparticle cementations are destroyed once the peak strength is reached (SAXENA & LASTRICO, 1978; CLOUGH et al., 1981) usually at small displacement levels, and soaking will have little extra effect in further reducing the strength.

CALIFORNIA BEARING RATIO TESTS

A total of four sets of CBR tests were conducted on samples from the A-1, R-1, Y-1, J-1 and S-1 sites according to ASTM D-1883. All samples were compacted at optimum moisture content to the maximum density, employing both the standard and modified Proctor rammers. Tests were performed on unsoaked and soaked specimens. The period of soaking was 96 hours.

Test results are summarized in Table 7. The values measured indicate no major variations between the chosen sites. The average CBR values using 2.5 kg and 4.5 kg rammers tested in unsoaked conditions are 27 and 33 respectively, while the corresponding values after soaking are 19 and 24 – a reduction of 27 to 30% on average. There was a considerable variation in the loss of CBR from sample to sample, from 2 to 14 in actual values. In percentage change terms, the loss ranges from 7 to 42%. The increase in compactive effort by using the 4.5 kg rammer instead of the 2.5 kg rammer method has resulted, on average, in a 22 to 26% increase in the CBR values.

Table 7. Summary of CBR Values under Unsoaked and Soaked Conditions.

Sampling Site	Standard Proctor (2.5 kg rammer)		Modified Proctor (4.5 kg rammer)		Remarks
	Unsoaked	Soaked	Unsoaked	Soaked	
A-1	27	18	30	28	1. Specimens compacted at optimum moisture content to maximum dry density.
R-1	26	15	32	20	
Y-1	25	19	31	23	
J-1	28	20	36	22	
S-1	27	22	35	27	
	2. Soaking continued for 96 hours before testing.				
Average	27	19	33	24	

Observations of swelling were made during soaking under a surcharge of 2.5 kPa. As expected with granular soils, no swelling was recorded during soaking. This implies that the windblown sand can be used satisfactorily as a base, or a sub base material in road construction, provided that it is properly compacted and confined. The design in all cases should be based on the soaked CBR values.

The loss of some strength and bearing capacity due to soaking results from the destruction of weak interparticle cementation bonds. These bonds exist because of the presence of carbonates aided by hot temperatures and extreme desiccation. These bonds dissolve upon wetting, resulting in a loss in the frictional strength component. In Kuwait, wetting of the surface soils occurs due to heavy showers in the winter season, leaking of underground conduits, and surface water from irrigation of plants and private gardens.

DESIGN AND CONSTRUCTION CONSIDERATIONS

Based on the properties and behavior of the soils described, it appears that isolated footings at a shallow depth of 1.0 to 3.0 m will be suitable for most applications. Where a combination of heavy loads and low relative density is encountered, raft foundations may be considered. In all cases, strength and bearing parameters should be based on soaked conditions. Suitable drainage systems should be provided to prevent prolonged wetting of the site due to rain or other water.

Since the standard penetration resistance usually varies between $N = 15$ and 35 in this surface deposit, the allowable pressure based on 25 mm total settlement normally ranges between 1 and 3 kg/cm² (100 to 300 kPa) according to TERZAGHI & PECK (1967) and PECK *et al* (1974). Exact recommendations, however, should only be made after a proper site investigation is carried out. For important structures, and in cases where high allowable pressures are required, additional tests may be recommended such as plate bearing tests, Static Cone Tests, and other detailed laboratory tests.

An appropriate angle of shearing resistance ϕ for this soil ranges between 33° and 35°. These values may be used for preliminary or feasibility studies. For the design of retaining walls, and lateral support systems, the coefficients of earth pressure in the active and passive Rankine states may be taken as $K_A = 0.30$ and $K_P = 3.7$ respectively.

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Grading characteristics indicate that the surface soils are predominantly sands, which are classified as SP→SM according to the Unified Soil Classification system. As such, they can be used successfully for backfilling around foundations and walls, and below ground floor slabs. However, problems may occur due to lack of proper specifications for field compaction and field supervision. These problems may worsen because of the sensitivity of these soils to increasing water content and resulting settlements if improper compaction is carried out.

To achieve proper compaction the specifications should require high relative compaction ranging from 98% to 100%, or the density achieved should exceed a specified relative density. It is particularly important that good field supervision should be maintained during compaction.

In case proper field compaction is difficult to achieve, such as in chalky soils occasionally encountered at some local sites or where heavy loads are to be placed over compacted ground, the use of cement stabilization should be considered. This is particularly important in view of the possible deformation resulting from wetting. Recent studies on surface soils from three areas in Kuwait (ISMAEL, 1984) indicated the successful use of Portland cement as an additive to obtain a superior quality fill. In these cases the addition of 2% cement (by weight) at optimum moisture content, mixed and compacted in the normal manner, yielded strengths well above those normally required for structural fills.

The measured coefficients of permeability in the range 10^{-3} cm/sec suggest a free draining cohesionless soil. Dewatering of excavations can usually be carried out using a suitable well point system.

Because of the very low natural moisture content which ranges from 1 to 2%, underground electric cables may be subject to unfavourable conditions. This is due to the high thermal resistivity (reciprocal of conductivity) of the soil around the cable which occurs at low moisture content, and possible overheating. It is thus important to determine the thermal properties of the surface soils accurately and assign a proper value for the thermal resistivity of the soil based on laboratory and field test results. If necessary, the use of concrete or soil cement around the cable may be specified to prevent thermal instability.

To protect foundations against sulphate attack, the use of sulphate resisting cement should be required for all foundations and slabs in contact with soil. The measured sulphate contents in this study area show some

high sulphate concentrations up to 1.6 %. At other locations, especially close to the shoreline or where the water level is high, much higher sulphate concentration have been recorded (JERAGH & ISMAEL, 1984). The use of a rich mix and adequate concrete cover of approximately 75 mm for steel reinforcement should be required. Additionally, the use of an adequate protective coating of inert material such as an asphalt or bituminous emulsion may be required depending on the concentration of sulphates in the soil and the ground water in the vicinity of the foundations.

CONCLUSIONS

An investigation of the properties of surface soils in Kuwait has been carried out. The study area selected included a corridor extending 35 km longitudinally between Salmiya on the east and Andalus on the west. The area is bounded laterally by the fourth and fifth ring roads.

By means of field and laboratory soil testing at nine sites along the corridor, the soil profile and properties were determined for use as a guide for preliminary design and construction of foundations and earth structures. The properties determined were for the surface layer which consisted mainly of windblown dune sands.

Classification tests, penetration tests and basic physical properties were determined initially, followed by compaction characteristics. Direct shear tests and California Bearing Ratio (CBR) tests were performed for determination of the strength and bearing values. These tests were repeated after soaking with water to assess the effects of wetting.

Based on the test results it was concluded that the surface soils possess similar characteristics at the different sites, and that they are sensitive to soaking. An average ratio of 4.8 between the Cone Penetration Resistance q_c and the SPT N value was determined based on insitu tests. This may be used for prediction of footing settlement. Recommendations were made for the design and construction of foundations and other earth structures placed in this soil. These recommendations are intended as a guide for feasibility studies, initial designs, and the identification of anticipated construction problems.

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FACTORS AFFECTING BEHAVIOUR OF EXPANSIVE SOILS IN THE LABORATORY AND FIELD—A REVIEW

S. T. EL SAYED* & S. A. RABBAA*

SYNOPSIS

This paper discusses the different factors and environmental conditions which affect the behaviour of an expansive soil in the field, and compares them with those prevailing in the laboratory when a soil specimen is tested. The paper also presents a brief review of some of the common methods used for determination of heave and swelling pressure in the laboratory, and evaluates the compatibility of these methods with the boundary and environmental conditions existing in the field.

INTRODUCTION

Although the different factors which control the behaviour of expansive soils have been intensively investigated by many research workers, the magnitude of insitu heave and pressures generated by such soils when they take up water are still a puzzling problem for both geotechnical and structural engineers, due to the great discrepancy between what is measured in the laboratory and what is observed in field. GUBTA et al (1983) stated that measurements of the swelling pressure on remoulded specimens give a deceptive idea about the maximum swelling pressure that could build up in a field situation. The actual values, starting from the dry field situation, are very much lower than the laboratory values primarily on account of the intensive cracking of the ground and the capacity of cracks to absorb lateral ground movements.

PECK et al (1974) stated that the results of all swelling tests are at best rough approximations, partly because of inevitable changes in water content and structure of the soil during drilling, sampling and handling in the laboratory. Moreover, many of the variables which govern the behaviour of an expansive soil in the field such as stress path, climate, drainage path, confinement, etc. are difficult to simulate or account for in the laboratory. Therefore, in prediction of the behaviour of an expansive soil in the field by interpretation of laboratory results, such variables should be taken into consideration.

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This paper discusses some of the above mentioned variables and factors which affect the behaviour of expansive soils in the laboratory and field. The paper also presents a brief review of some of the most common methods used for measuring heave and swelling pressure in the laboratory. The paper investigates to what extent such methods simulate the boundary and environmental conditions, whether those already prevalent in the field during the construction or those which exist later during the service life of the structure. The present work is an attempt to better understand and interpret the laboratory test data by highlighting the differences between the boundary and environmental conditions which control the behaviour of an expansive soil in the field and those which govern the behaviour of a specimen of such soil in the laboratory. It is an attempt to assist the designer dealing with structures which are to be founded on expansive soils, so that the danger of the problem is not underestimated or overestimated.

FACTORS CAUSING DISCREPANCY BETWEEN LABORATORY RESULTS AND FIELD BEHAVIOUR

There are some factors which have an important influence on the swelling characteristics of expansive soils (such as type of clay mineral, type of exchangeable cation, clay content, etc.) but these factors have the same influence on the swelling characteristics whether in the laboratory or in the field. Therefore they will not be discussed herein. However, there are other factors which control the behaviour of expansive soils in the field that are difficult to simulate or account for in laboratory, such as stress path, stress distribution, scale effect, confinement, etc. and these are discussed in the following sections.

Stress Path

Stress Path is used herein to describe the variation in stresses acting on the foundation soil from commencement of the excavation until construction is complete and the soil comes in contact with a source of water, allowing the swelling process to start. When excavation for the foundation is commenced, the overburden pressure decreases to zero at foundation level. The pressures in the bulb area below the excavation also decrease according to the theory of stress distribution. With the commencement of construction, the application of dead load increases the bulb pressures until they reach the original state of the overburden pressure. Then, the increase in the bulb pressures continues until the construction has been completed and live loads are applied by occupancy of the structure (see Figure 1).

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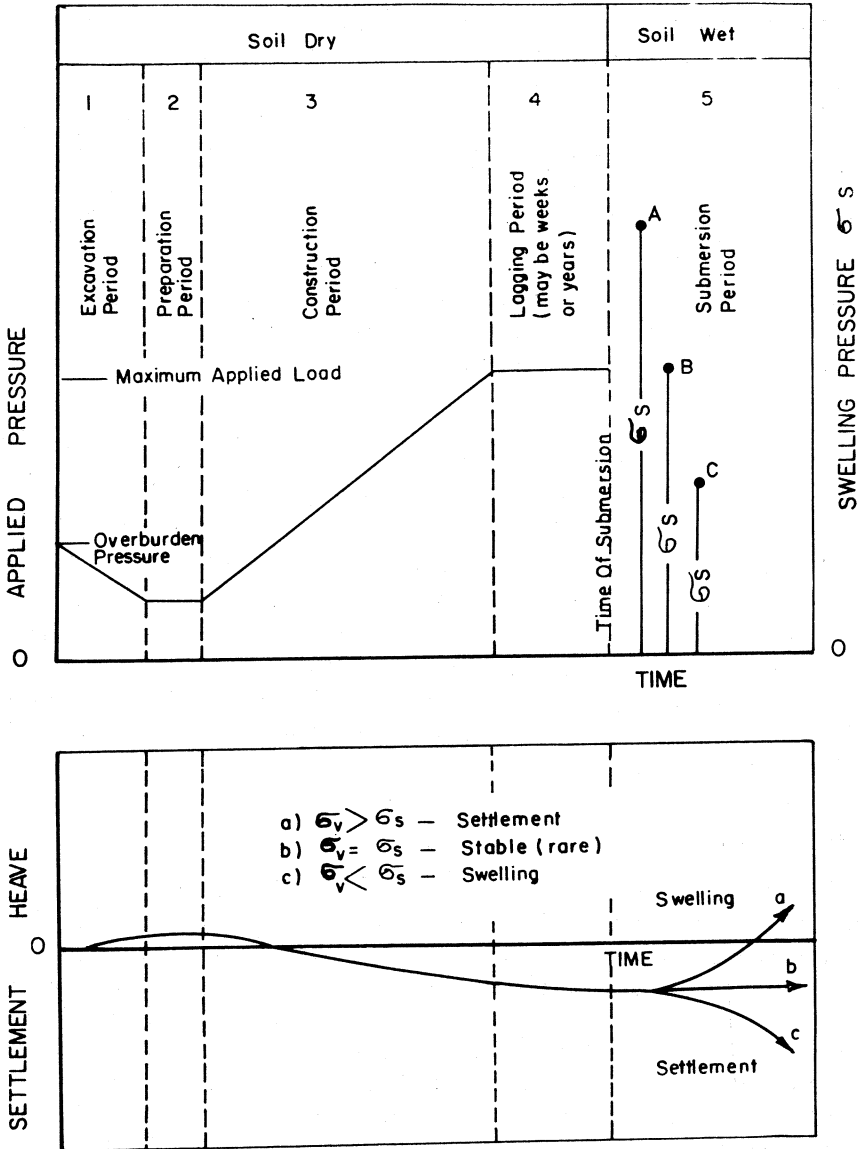


Fig. 1. Variation in Applied Pressure and Movement of Foundation Soil with Time.

This state of stress may continue for a long or short period, until the soil comes in contact with water. Within this period the soil may undergo compression from the effect of the applied loads. When the soil comes in contact with water, upward swelling pressures are generated. As a result, the structure may suffer upward movement (heave) if the upward swelling pressures are greater than the downward loads of the structure, or it may suffer from some increase in compression due to wetting if the swelling pressures are less than the applied loads. In the very rare case, it may remain stable if the exerted swelling pressures are almost equal to the applied loads.

On the other hand, when a sample of this soil is extracted, the overburden pressure acting on it diminishes until it becomes zero. During the sampling process the sample may or may not experience some compression, as will be discussed later. When the sample is tested, it is subjected to a relatively small load in some testing methods, and is then allowed to absorb water until it reaches full expansion and comes to equilibrium. After that, the sample is compressed to its original volume following the same procedures as in the consolidation test. Alternatively, as in some other testing techniques, the sample may first be loaded in the dry state, and then submerged and allowed to take its full expansion under the effect of the applied loads.

From the above, it is obvious that the stress path of a sample of swelling soil in the laboratory is different from that of the soil in the field. Many investigators have found that the amount of compression or expansion depends greatly on the stress path (KOMORNIK & DAVID, 1969; BURLAND, 1975; MITCHELL, 1976; and others).

Stress Distribution

This term, as known, describes the stress conditions in the area of influence of the foundation. It is noted that the soil in this area is under the influence of imposed stresses which decrease with depth and with distance from the centre line of the foundation according to the known distribution of the pressure bulb (see Figure 2). In addition, the overburden pressure increases with depth. Therefore, when the soil comes in contact with water, the absorption process (and consequently the swelling) occurs under stress conditions which differ from one point to another under the foundation. Thus the foundation movement is, in fact, the final resultant of the movements of different soil elements. In other words, the vertical movement of the soil could be assumed to occur under an average resultant pressure derived from the bulb pressure and the overburden pressure (Figure 2).

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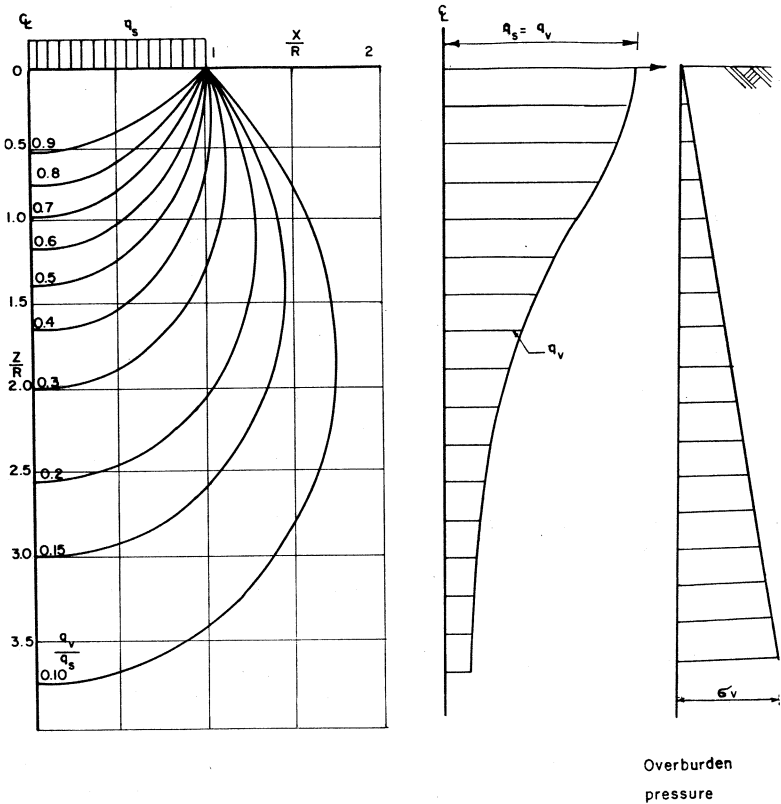


Fig. 2. Typical Distribution of Vertical Pressure under Circular Foundation.

A specimen in the laboratory however is subjected to vertical stresses which are constant throughout the relatively small thickness of the specimen (see Figure 3). Since the swell is highly dependent on the pressure applied, it may be expected that the swell in the field is different than that obtained in the laboratory if the soil specimen is tested under the structural pressure exerted at foundation level.

Scale Effect

A common problem which the geotechnical engineer may face is the wide variation in soil properties vertically and horizontally even within the boundaries of one structure. Swelling soils often exist as layers interbedded with other soils like sands or silts. Such soils have no swelling potential and even more they may tend to compress when submerged in water. The presence

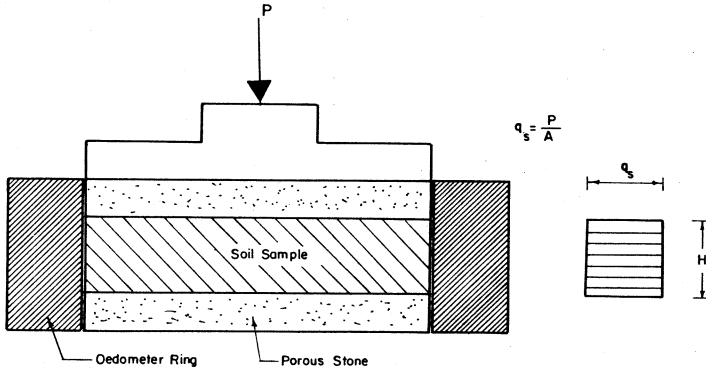


Fig. 3. Distribution of Vertical Pressure through the Sample Thickness.

of a non-swelling layer will reduce the swelling of the clay layer (KATTI & KULKARNI, 1967; KATE & KATTI, 1983). This implies that the final movement, which is a resultant of the swelling of the clay layer and the compression of the sand or silt layers, will be less.

Comparing this with the case of an expansive soil specimen, we find the matter is completely different. Firstly, swelling tests are usually conducted on a limited number of relatively small specimens selected from layers which are expected to have the highest swelling potential. Secondly, the non-swelling sand or silt layers have not been represented in the chosen specimens. This difference between the two situations, field and laboratory, means that the amount of heave or the value of the swelling pressure measured in the laboratory may be much higher than those that actually occur in field. Of course this will lead to conservative design, and it may inflate the assessment of probable danger and consequently lead to unnecessary costs.

On the other hand, the size of the tested specimen taken from a particular expansive soil has an influence on the laboratory results. For instance, it has been found that the measured swelling pressure depends to some extent on the diameter of the tested specimen (SING, 1967; CHEN, 1975; TAREK, 1980). Also, it has been noted that the specimen height has an influence on the swelling percentage and swelling pressure (PALIT, 1953; JENNINGS & KERRISH, 1962; SING, 1967; UPPAL & PALIT, 1969; EL RAYES et al, 1979; TAREK, 1980). Some investigators have reported that both swelling percentage and swelling pressure are independent of specimen height if the effect of side friction is eliminated (SALAS & SERRATOS, 1957;

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RANGANATHAM & SATYANARAYANA, 1965; RABBAA, 1975; CHEN, 1975).

From the above discussion, it appears that there is a severe need to standardise the size of the specimen (height and diameter) in swelling tests. Also, the results obtained should be reasonably interpreted taking into consideration the overall behaviour of the soil in the field.

Confinement

In the field the movement of moisture as well as the volume change takes place in three dimensions, but in the laboratory the lateral confinement in the oedometer makes these movements occur in only one dimension. KASSIF & BAKER (1969) stated that the principal stress ratio in this case is affected and does not properly reflect the actual field conditions. The lateral confinement in the oedometer leads to higher vertical swelling and swelling pressure. For instance, SING (1967) reported that a laboratory model test indicated a swelling pressure of only 9.25 t/m^2 for a circular test plate of 6.35 cm diameter placed centrally over the unsurcharged surface of a block of black cotton soil 45 cm by 45 cm deep. For the same soil, a laterally confined specimen exerted a pressure on the order of 174 t/m^2 .

Also, if the zone surrounding the test area is confined by surcharge, the swelling pressure increases as the area of confinement increases (UPPAL & PALIT, 1969). On the other hand, the side friction resulting from confinement of the laboratory specimen has an influence on the measured swelling pressure. CHEN (1975) has observed that the side friction that can develop due to swelling is about 15% of the zero volume change pressure.

Disturbance

Any clay sample brought to the laboratory for testing undergoes, to some extent, disturbance which changes the characteristics and properties of the sample. Sampling method is considered a prime factor influencing sample disturbance. For example, samples taken by driving a thick walled spoon into the ground may be denser than the soil in place and consequently may swell more than the undisturbed material. Sample disturbance results also from inadequate sealing of the specimens, transportation, storage and preparation operations, as well as relief of stress and relaxation between sampling and testing. All these disturbances are irreversible and cannot be accounted for. However, trials to overcome these problems by attempting to measure the swelling characteristics in place are still difficult. A few research workers

have made such trials (e.g. EL RAMLI & EL DEMIRI, 1973; OFER et al., 1983; and GUBTA et al, 1983).

Pattern of Moisture Distribution

The moisture content is considered one of the main factors which govern the amount of swell and the magnitude of swelling pressure. Therefore, a great error associated with measurement of swelling characteristics is likely to arise from a difference between the initial water content of the sample and the natural moisture content in the field.

The soil is assumed to have a moisture pattern before erection of a building which is in equilibrium with the existing insitu boundary conditions of evapotranspiration forces, depth to water table, etc. When a sample of soil is taken to the laboratory it is possible to make provisions to keep its natural moisture content unchanged. However in the field as soon as the site is covered by the building a new set of boundary conditions is developed under the building due to a change in temperature, the prevention of evaporation, etc. This new condition leads to a steady increase in the moisture content until equilibrium has been reached and a new pattern of moisture distribution established (DONALDSON, 1965). This increase in the moisture content makes the amount of heave that could occur if the soil later came in direct contact with water to be much less than that predicted from the laboratory results.

Mode of Watering

Mode of watering is meant herein to be the path which water follows to reach the soil under the foundation, the timing of the watering, and the rate. These factors are considered of great importance in governing the distribution of soil movement under the building and consequently the distribution of cracks and damage between the different elements of the building.

In the field, the source of moisture may be surface water (such as rain or garden water), a rise in the ground water table, or it may be due to some accidental influences such as broken drains, leakage of water pipes, septic tanks, etc. In all these cases the timing of watering is usually after the structure is in service and the maximum loads (dead loads and partial or full live loads) have been applied. According to the water source available, the rate of absorption will vary and the time necessary for the soil to produce detrimental heave may extend to several years or may be a matter of a few weeks. Of

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course, whether the building will be able to accommodate the heave or not will depend, to some extent, on the swelling rate.

In the laboratory the mode of watering is different. Firstly, the specimen is submerged directly with water. The time of submersion and consequently commencement of swelling may be before, during or after application of loads according to the testing method used, as will be discussed in the next section. It has been found that both swelling percent and swelling pressure depend greatly upon the timing of watering, i.e. submersion before, during or after the application of load.

Water Properties

Many research workers have indicated that the water properties and soluble salts present have an influence on the swelling characteristics of expansive soils (BOLT, 1956; LADD, 1959; LAMBE & WHITMAN, 1959; RANGANATHAM, 1961; EMERSON, 1963; THOMSON & ALI, 1969; MITCHELL, 1976; MOAFI, 1978; and others). Their results indicate that in general the amount of swell and consequently the swelling pressure is inversely proportional to the salt concentration in the absorbed water.

Of course water in the field reaches the soil under the foundation mixed with the soluble salts present. In the laboratory the sample is often submerged in distilled water, or sometimes in drinking water. As a result there is a possibility of overestimating the swell characteristics from the laboratory data.

TESTING TECHNIQUES FOR EXPANSIVE SOILS

Techniques available for quantitative measurement of expansive soil characteristics fall into three categories, namely, oedometer tests, soil suction tests and empirical methodology. Among these techniques the oedometer tests are capable of simulating some of the factors which affect the swelling characteristics of an expansive soil. No doubt this technique suffers from a serious limitation, though, as insitu the movements of moisture as well as volume change frequently take place in three dimensions rather than in one dimension as imposed in the oedometer. However, for its simplicity the oedometer testing technique has become popular and extensively used.

In the following section, some of the methods which use the oedometer will be reviewed and investigated to show to what extent such methods are capable of simulating the environmental and boundary conditions prevalent in the field.

Different Pressures Method

The Different Pressures Method has been adopted by many investigators (EL-RAMLI, 1965; MYSLIVEC, 1969; SING, 1967; EL-RAMLI & EL-DEMIRI, 1973; DAVID et al, 1973; and others). In this method several identical samples (three or more) are loaded with different loads near the expected swelling pressure. The samples are left until they reach equilibrium in the dry state, then they are submerged and the swell is recorded. The samples are again left until they reach equilibrium and the swelling practically ceases. The vertical movements (swelling or compression) are plotted against the applied pressure and the pressure corresponding to zero volume change is taken as the swelling pressure (see Figure 4).

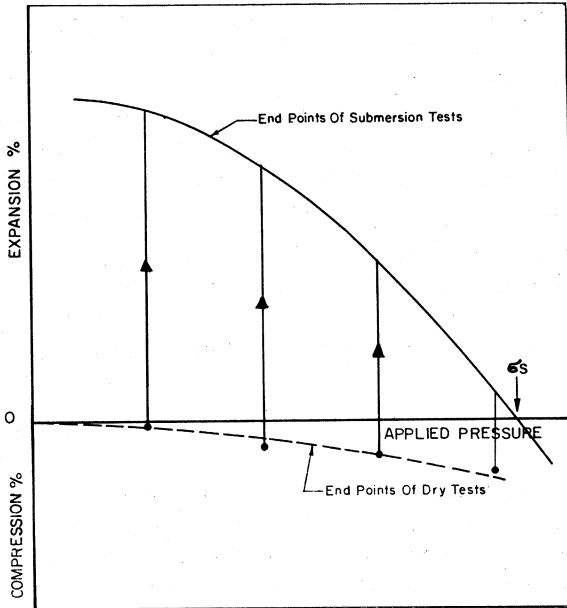


Fig. 4. Determination of Swelling Pressure by Different Pressures Method.

It can be considered that the Different Pressures Method has an important advantage with respect to other methods. The loading-wetting events in this method follow the same sequence as in the field: the loads are firstly applied to an unsaturated soil, the soil undergoes compression under these loads, then after a period of time (which may be as long as years or as short as weeks) the soil comes in contact with a source of water and an upward or downward movement starts. Swelling or compression is observed accord-

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ing to the magnitude of the applied load relative to the swelling pressure exerted.

The data obtained by this method can provide much information useful for design such as the swelling pressure corresponding to zero volume change, the vertical movement that is to be expected under the loads imposed by the structure, and the loads that could be applied to develop a certain swelling within the tolerable limits. On the other hand this method requires a number of (three or more) identical undisturbed samples as mentioned, which may sometimes be difficult to obtain. But from another point of view, testing three samples or more may be considered a merit since it makes the swelling characteristics measured by this method more realistic and more representative of the soil in the field than those obtained by testing only one sample.

The Different Pressures Method has some other limitations which make it unrepresentative of the field situation. For instance, the sample in the consolidometer is laterally confined and suffers from side friction. The sample is also subjected to a uniform pressure through its thickness, opposite to what happens insitu under the foundation, as explained in the previous sections. However, other methods suffer from these limitations also.

Swell-Consolidation Method

In the Swell-Consolidation Method an undisturbed sample is allowed to absorb water under a relatively small load (0.1 kg/cm^2) and is left until it fully expands and reaches equilibrium, then it is consolidated by increasing the applied pressure in intervals following the procedure of the conventional consolidation test. The consolidation continues until the sample reaches its initial volume or even shows some decrease in volume. The stress-strain relationship is plotted as shown in Figure 5. The pressure corresponding to zero volume change is taken as the swelling pressure. Many investigators have used this method to obtain a relationship between swell and applied pressure, and to determine the swelling pressure (HOLTZ & GIBBS, 1956; YOUSEFE et al, 1957; RAMARISHNA et al, 1972; SRIDHARAN & VENKATAPPA, 1972; ZACHARIAS & RANGANATHAM, 1972; JOHNSON et al, 1973; CHEN, 1975; and others).

The most serious criticism of this method is that it does not represent the normal sequence of load-submersion. The soil in the field will not absorb water and swell first with the structural loads applied later, but rather vice versa. Moreover, a period of time which may extend to years after the struc-

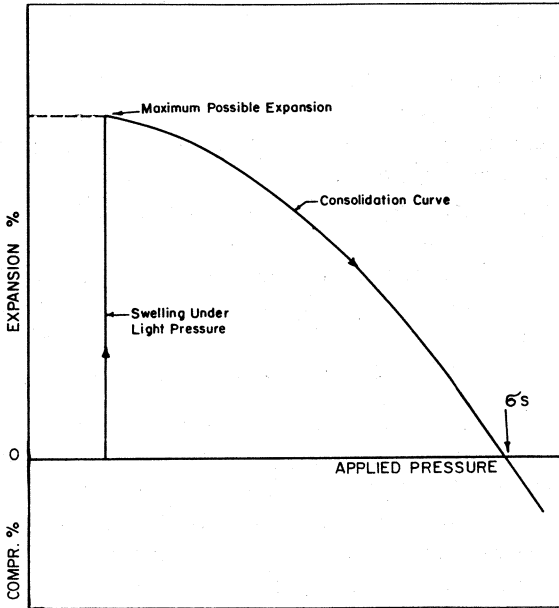


Fig. 5. Determination of Swelling Pressure by Swell-Consolidation Method.

ture is in service may pass before the soil comes in contact with water. The swell-consolidation path which is followed in this method tends to make the pressure required to compress the pre-swelled sample higher than those obtained by other methods (e.g. Different Pressures Method) and consequently the produced swelling pressure is higher (see Table 1). The higher pressures required to consolidate the pre-swelled sample can be attributed to the great energy required to expell the absorbed water.

Table 1. Swelling Pressures Determined by Various Testing Methods using Identical Specimens (Rabbaa, 1975).

Method	Swelling Pressure (kg/cm ²)
Different Pressures	18.0
Swell-Consolidation	22.0
Double Oedometer	34.0
Constant Volume	19.0

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Double Oedometer Method

The Double Oedometer Method was proposed by JENNINGS & KNIGHT (1975) and adopted by many investigators to study the swell-pressure relationship of an expansive soil (BURLAND, 1965; WILLIAMS, 1965; BRACKLEY, 1975; CHEN, 1975; RABBAA, 1975; TAREK, 1980; and others). In this method two identical samples are compressed under different pressures: the first in the dry state (i.e. with natural water content) and the second after absorbing water and swelling under very small pressure followed by consolidation using the same procedure as the swell-consolidation method. For the two samples the applied pressures are plotted against the vertical strain and two curves are obtained. The pressure corresponding to the intersection point of the two curves is taken as the swelling pressure (see Figure 6).

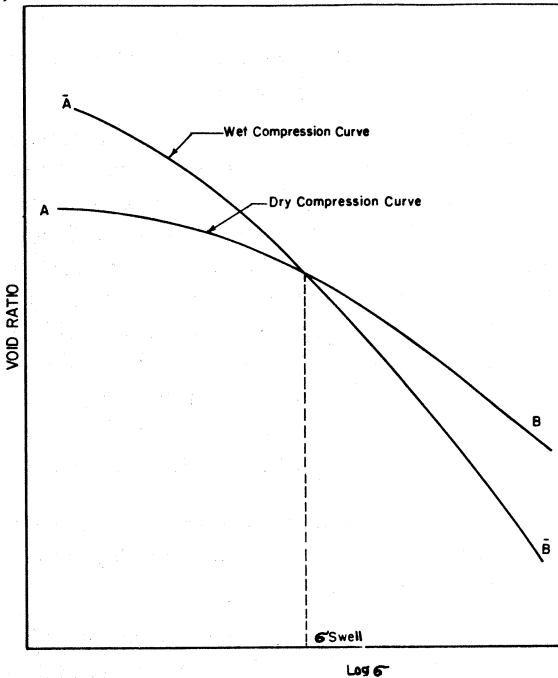


Fig. 6. Determination of Swelling Pressure by the Double Oedometer Method (Burland, 1965).

This method usually produces relatively high swelling pressures, even higher than the Swell-Consolidation Method (see Table 1). This is of course due to the fact that the pressure taken as the swelling pressure is not that required

to bring the sample to its initial volume, but to its volume after being compressed in the dry state by a pressure equal to the swelling pressure (see Figure 6). This method also suffers from the same limitations as the Swell-Consolidation Method.

Constant Volume Method

The specimen in the Constant Volume Method is allowed to absorb water without any increase in volume by increasing the applied pressure as the test proceeds until the sample reaches equilibrium. The increase in applied pressure may be carried out two ways. The first is by using the oedometer, where more load is added to keep the sample volume constant while the sample absorbs water (HOLTZ & GIBBS, 1956; BALDOVIN & SANTOVITO, 1973; DAVID et al, 1973). The second is by using a rigid proving ring to do the same thing (PALIT, 1953; ALPAN, 1957; SING 1967; SANKARAN & DUKSHANA, 1972). The swelling pressure can be determined by plotting the applied pressure against time (Figure 7).

This method also does not simulate the insitu condition, where the applied load after the structure is in service does not change with time, and therefore when the swelling process occurs a constant pressure acts rather than different pressures which increase with time. Also, this method does not present some information which may be necessary for design, such as the amount of heave which could be expected under application of a certain load (e.g. the load exerted by the structure) or the load which could be applied such that the heave developed is tolerable. Finally, the method needs human control for an uninterrupted and relatively long period.

CONCLUSIONS AND RECOMMENDATIONS

From the present study of various factors and environmental conditions which are prevalent in the laboratory and field, the following conclusions and recommendations can be presented:

- (1) The insitu stress path is certainly different from that of a soil sample tested in the laboratory for most of the common methods used in testing of expansive soils. This leads to discrepancies between results obtained in the laboratory and those observed in the field.
- (2) The stress distribution, which is practically uniform within the relatively small thickness of a soil sample, results in expansion and swelling pressures determined in the laboratory which are surely different from

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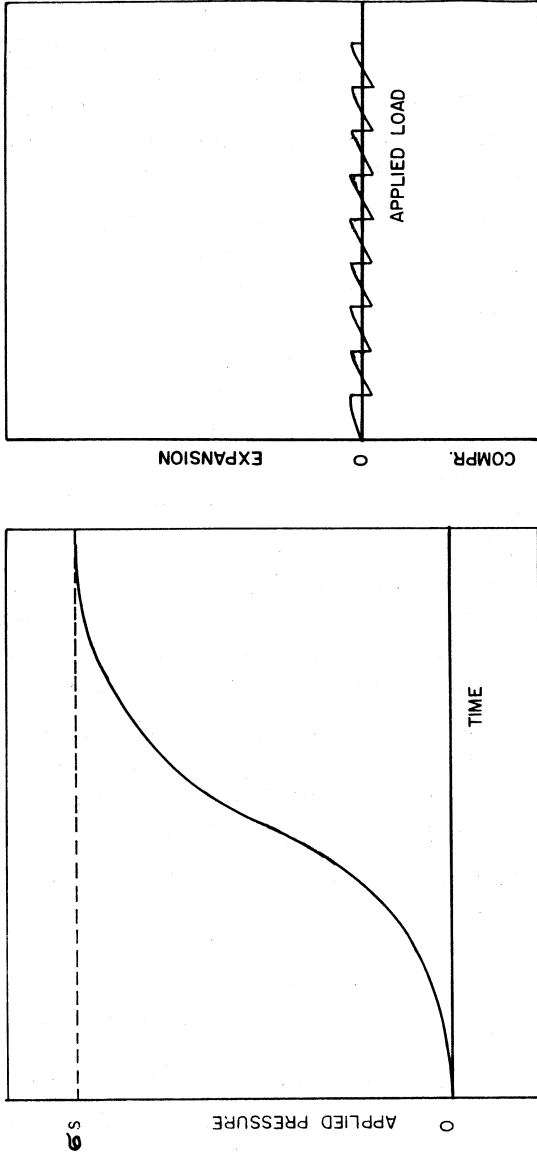


Fig. 7. Determination of Swelling Pressure by the Constant Volume Method.

those produced in the field under the combined effects of the overburden pressures and the bulb pressures.

- (3) The non-swelling layers of sand and silt which are often interbedded with an expansive soil layer (or which lie between it and the foundation, as in the case of expansive soil present at depth) lead to expansion and swelling pressures in the field that may be several orders of magnitude smaller than those measured in the laboratory.
- (4) Confinement and side friction have a great influence on laboratory results. Measured expansion and swelling pressures vary according to the size of the sample. Different results can be obtained by using samples of different heights and diameters.
- (5) The inevitable disturbance which occurs during sampling leads to laboratory results which are absolutely different from those observed insitu.
- (6) The manner, timing and rate of wetting in the field differ from those prevalent in the laboratory for most of the common methods used in the testing of expansive soils.
- (7) The properties of the water and soluble salts available in the field differ from those of the water usually used in the laboratory, e.g. distilled water. This also leads to measured expansion and swelling pressures which may be higher than those that actually occur insitu.
- (8) Most conventional methods of testing do not simulate the field situation well. However, from this point of view, the Different Pressures Method may be considered the most suitable for predicting the insitu behaviour of an expansive soil from laboratory results.
- (9) A new standard test method is needed with the following attributes:
 - Partial confinement as in the field.
 - Partial loading.
 - Expansion under constant load (design swelling pressure).
 - A sample of suitable size which represents the various layers of the soil system.
 - A water content for testing which takes into consideration the change in the pattern of moisture distribution after construction of the building.
 - Mode of watering which can be estimated from the conditions of the site and local experience.

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- Good sampling and transportation to minimize the disturbance of the specimens, with the sampling done by using air flush or from open test pits.
- (10) The designer should be aware of the different variables and environmental conditions which are prevalent in the laboratory and on site, to be able to make comprehensive interpretation of, and sound judgement on, the test results.

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BEHAVIOUR OF PILE GROUPS CONTAINING PILES OF UNEQUAL LENGTH

H.G. POULOS¹ & C.M. HEWITT²

INTRODUCTION

Situations may arise in which additional piles are added to an existing pile group in order to increase its ultimate load capacity or increase its stiffness. This Note considers the case of a hypothetical two-pile group in stiff clay, and examines the effect of adding two further piles whose length may be different from that of the original piles. The effect of the length of the additional piles is examined in relation to:

- i) the static load capacity of the group;
- ii) the settlement and load distribution under static loading;
- iii) the load capacity after cyclic loading.

ANALYSIS

The analysis of pile groups containing dissimilar piles has been described by POULOS & HEWITT (1986). This analysis involves the use of the boundary element method, with each pile in the group being discretised into a number of elements. The surrounding soil is assumed to be an elastic continuum and each element is acted upon by an interaction stress or traction at the pile-soil interface (shear stress for shaft elements, normal stress for base elements or elements at diameter discontinuities). Vertical displacement compatibility between the pile and the soil displacements at each element is considered, with allowance also being made for the possibility of pile-soil slip at shaft elements, and bearing failure at pile base or discontinuity elements. Limiting values of the stress at each element are specified as input data into the analysis, together with the soil deformation parameters (Young's modulus, Poisson's ratio) and the stiffness of the pile element.

Under cyclic loading, cyclic degradation of the limiting skin friction at shaft elements is assumed to be described by the model proposed by MATLOCK & FOO (1979). Degradation occurs at an element if the reversal of slip occurs at the pile-soil interface of the element. In the

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solutions described below, the maximum possible reduction of skin friction due to cyclic loading has been taken as 50% of the original static value.

RESULTS FOR STATIC LOADING

Figure 1 shows the problem analysed and is meant to represent the case of a group of bored piles in stiff clay.

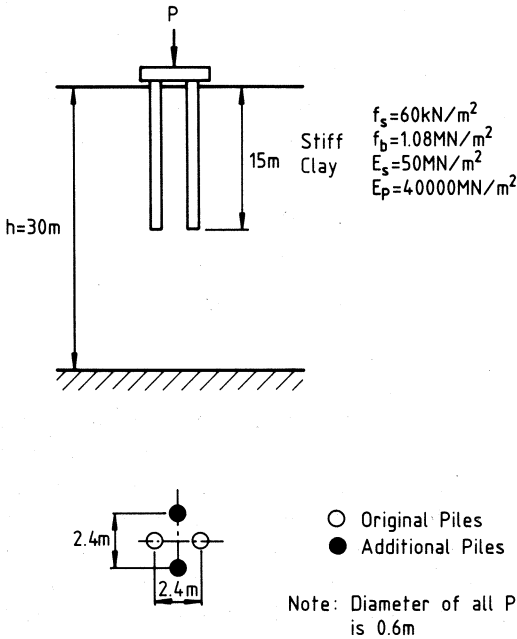


Fig. 1. Problem Analysed.

Figure 2 shows the effect of length of the added piles on the static load capacity of the group, and Figure 3 illustrates the effect of added pile length on group settlement and load distribution. Clearly, very little benefit is gained from the additional piles unless they are at least half the length of the original piles.

Figure 4 shows the load distributions in the original and additional piles, and reveals that the short added piles are subjected to negative friction along at least part of their length, thus explaining why such piles do not improve the group performance.

UNEQUAL PILES

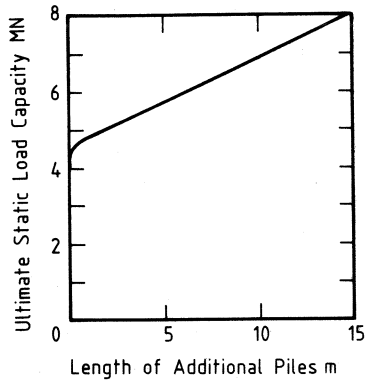


Fig. 2. Effect of Additional Piles on Ultimate Static Load Capacity.

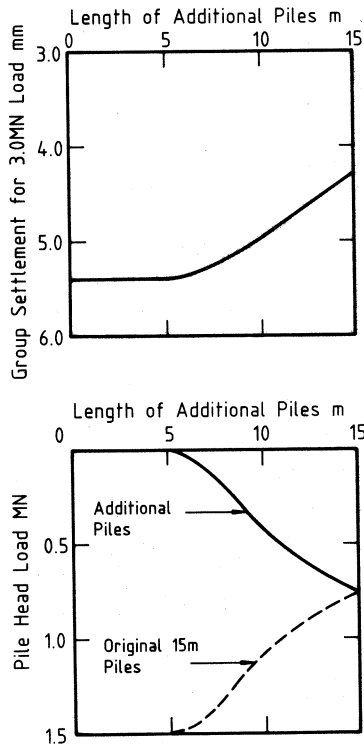


Fig. 3. Settlement and Load Distribution in Pile Group for Total Load = 3.0 MN, Static Loading.

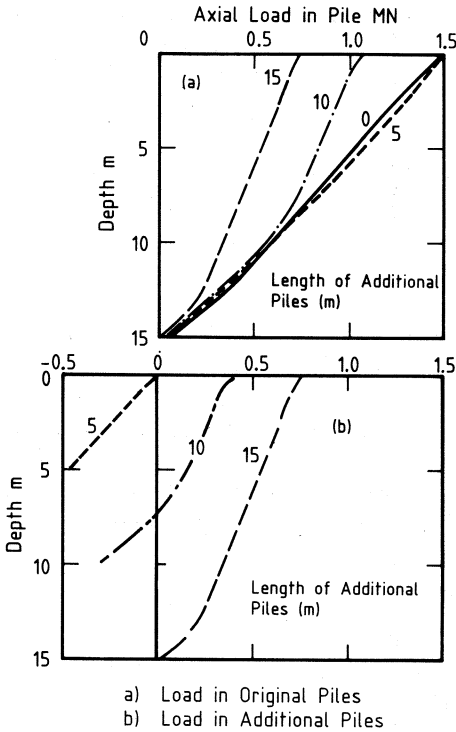


Fig. 4. Static Load Distribution in Piles, Total Load = 3.0 MN.

RESULTS FOR CYCLIC LOADING

Figure 5 shows the influence of the amplitude of cyclic load on the group load capacity after cyclic loading. For any particular length of pile, a "critical" cyclic load can be found at which the group fails during cycling, after 10 cycles of load. As shown in Figure 6, for groups of piles of equal length, this critical cyclic load is about 70% of the static load capacity. However, when the group contains piles of unequal length, the critical cyclic load falls to about 50% of the static load capacity.

Figure 7 summarises the effect of length of added piles on the safety factor under both static and cyclic loading. Clearly, the use of short added piles does not increase the safety factor under cyclic loading to the same extent as under static loading. However, in this case, the use of additional piles of

UNEQUAL PILES

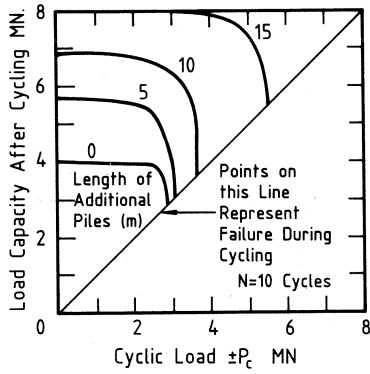


Fig. 5. Variation of Load Capacity after Cycling with Cyclic Load Amplitude.

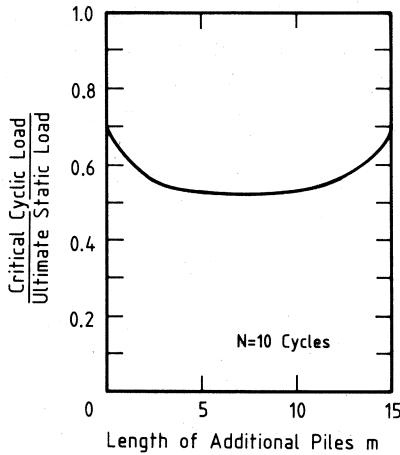


Fig. 6. Effect of Length of Additional Piles on Critical Cyclic Load.

length equal to the length of the original piles (15 m) results in the same factor of safety under both static and cyclic loading, since no degradation of skin friction occurs then.

CONCLUSIONS

For the particular case considered, analysis described herein suggests that, under static working loads, the use of short additional piles results in very little improvement in the performance of a pile group. This occurs because

POULOS & HEWITT

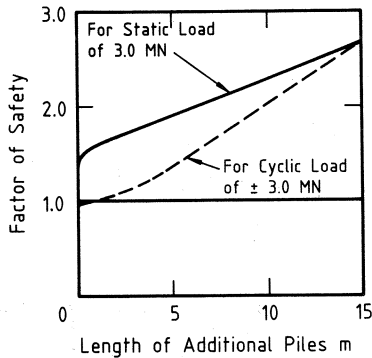


Fig. 7. Effect of Additional Piles on Factor of Safety.

the long piles carry almost all the load, while the short piles may be subjected to negative friction. Under cyclic loading, short additional piles result in a smaller increase in safety factor than under static loading, and the critical level of cyclic load (relative to the static load capacity) is less than for groups with equal-length piles. The overall conclusion is that groups with piles of equal length perform more efficiently than groups containing different length piles.

REFERENCES

MATLOCK, H. & FOO, S.C. (1979). Axial Analysis of a Pile Using a Hysteretic and Degrading Soil Model. *Proc. Conf. Numerical Methods in Offshore Piling*, Inst. Civ. Engrs, London, pp. 165-185.

POULOS, H.G. & HEWITT, C.M. (1986). Axial Interaction Between Dissimilar Piles in a Group. *Proc. 3rd Int. Conf. Numerical Methods in Offshore Piling*, Nantes, France.

CLOSURE

THE EFFECT OF SAMPLING DISTURBANCE ON UNDRAINED STRENGTH OF COHESIVE SOILS[†]

T. Kimura¹ & K. Saitoh²

The Authors are deeply indebted to Mr. M.D. Howat for his valuable discussion.* Before proceeding, the Authors wish to express an apology for errata in Tables 1 and 2 of their paper. The percentage of clay fraction under 2 μm (%) for M50 in Table 1 should read 29.4 for 39.4. The second and fourth columns of the bottom three rows in Table 2 should read as follows:

	$(\sigma'_{vc})_c$ (kN / m ²)	$ u_1 / (\sigma'_{vc})_c$
M30	61	1.33
	80	0.99
	85	1.04
	91	1.01
M20	83	1.07
	85	1.06
	93	1.01
	96	0.86
M10	93	0.81
	77	0.87
	75	0.88
	69	1.04
	77	1.09

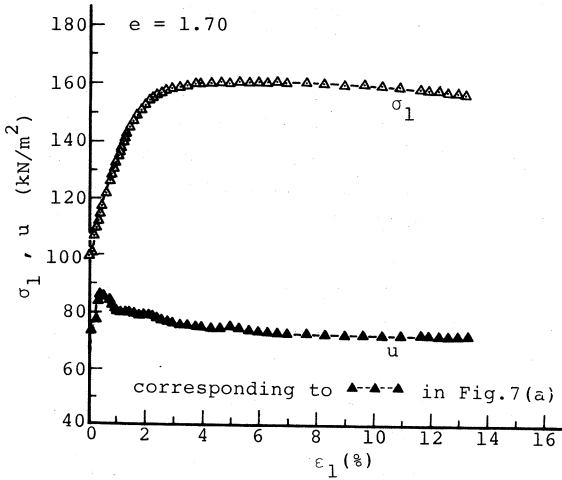
The results of unconsolidated undrained (UU) tests on the trimmed samples, for which the effective stress paths were given in Figure 7 of the paper, are replotted in Figure 12 in terms of axial stresses and pore water pressures against axial strains. As pointed out by the Discussor, "collapse" seems to have taken place in some samples. The Authors agree with the Discussor in

[†]Published in Vol. 15, No. 1, 1984

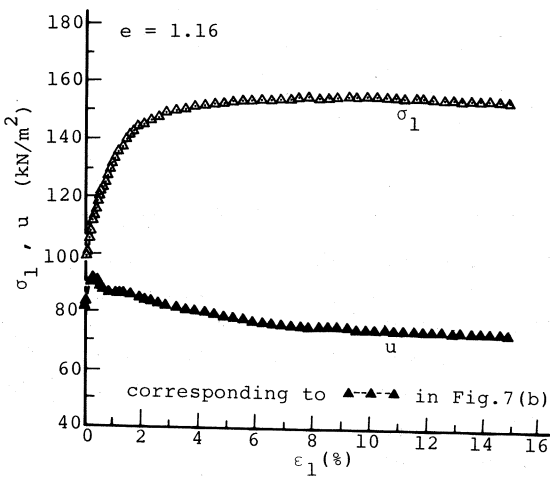
*Published in Vol. 16, No. 1, 1985

¹Professor and ²Research Associate, Tokyo Institute of Technology, Dept of Civil Engineering, Tokyo, Japan.

interpreting this as collapse of metastable structure developed during the process of consolidation from slurry. In the samples which the Authors dealt with the development of metastable structure could be accelerated by leaching because clay was remoulded by adding fresh water.

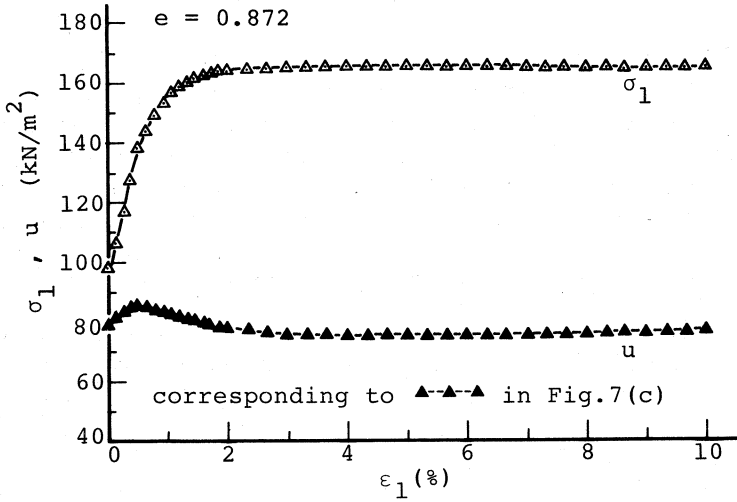


(a) M50

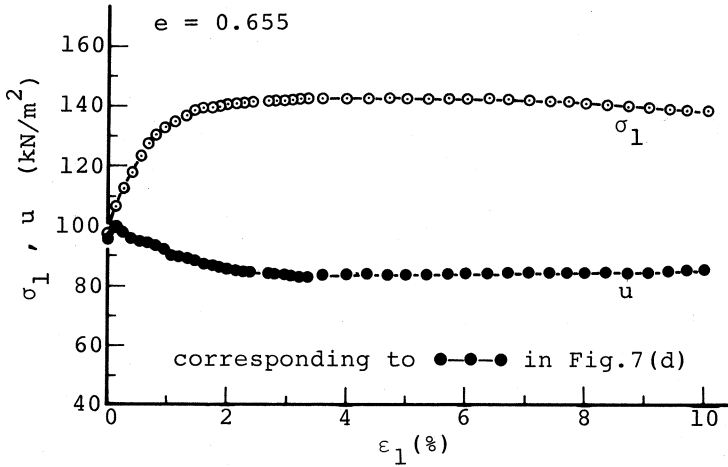


(b) M30

SAMPLING DISTURBANCE



(c) M20



(d) M10

Fig. 12. Results of UU Tests; Axial Stresses (σ_1) and Pore Water Pressures (u) versus Axial Strains (ϵ_1); $\sigma_3 = 98 \text{ kN/m}^2$.

KIMURA & SAITOH

On the question of saturation degree of the samples, the Authors consider that the degree is fairly high because *B* values are predominantly above 0.95 as can be seen from Figure 12. The Discusser calculated K_0 values by assuming that the initial pore water tension is equal to the spherical effective pressure reduction. The Authors cannot accept this assumption, because $|u_1|$ values given in Table 2 of the paper are absolute values of negative pore water pressure observed immediately after unloading the vertical consolidation pressure. No separation was observed at this stage in the experiments between soil and the wall of the consolidometer, which may justify the calculation attempted by Mair (MAIR, 1979).

The coefficients of consolidation for M50, M30, M20 and M10 are listed in Table 5. Since in the Authors' experiments shearing tests were carried out on all the samples less than 60 minutes after termination of consolidation, it is difficult to separate the effective stress loss due to sample age from the total loss observed.

Table 5. Coefficients of Consolidation.

Sample	C_v^* (cm^2/min)
M50	6×10^{-2}
M30	8×10^{-2}
M20	9×10^{-2}
M10	5×10^{-1}

*For average consolidation pressure of 98 kN/m^2

CONFERENCE NEWS

International Conference on Reliability of Methods for Engineering Analysis, Swansea, U.K., July 9-11, 1986. All enquiries to: Prof. D.R. J. Owen, Dept. of Civil Engineering, University College of Swansea, Singleton Park, Swansea SA2 8PP, U.K.

International Symposium on Natural and Man-Made Hazards, Rimouski & Quebec City, Canada, August 3-9, 1986. All enquiries to: Dr. Mohammed El-Sabh, Dept. d'oceanographie, Universite du Quebec a Rimouski, 310 avenue des Ursulines, Rimouski, Quebec G5L 3A1, Canada.

Symposium on Intermontane Basins, Chiang Mai, Thailand, August 4-7, 1986. All enquiries to: Dr. T. Thanasuthipitak, Dept. of Geological Sciences, Chiang Mai University, Chiang Mai 50002, Thailand.

Interpretation of Field Testing for Design Parameters, Adelaide, Australia, August 18-19, 1986. All enquiries to: Conference Dept., The Institution of Engineers, Australia, 11 National Circuit, Barton, A.C.T. 2600, Australia.

Third U.S. National Conference on Earthquake Engineering, Charleston, South Carolina, U.S.A., August 24-28, 1986. All enquiries to: Dr. J.E. Beavers, Martin Marietta Energy Systems, Inc., Bldg. 9733-4, M/S 2, P.O. Box Y, Oak Ridge, Tennessee 37831, U.S.A.

International Symposium on Large Rock Caverns, Helsinki, Finland, August 25-28, 1986. All enquiries to: Dr. Kari Soari, Technical Research Center of Finland, Lehtisaarentie 2, SF-00340 Helsinki, Finland.

39th Canadian Geotechnical Conference, Ottawa, Canada, August 27-29, 1986. All enquiries to: Dr. G.E. Bauer, Dept. of Civil Engineering, Carleton University, Ottawa K1S 5B6, Canada.

ISRM International Symposium on Rock Stress and Rock Stress Measurements, Stockholm, Sweden, September 1-3, 1986. All enquiries to: Rock Stress, CENTEK Conference, University of Lulea, S-951 87 Lulea, Sweden.

International Conference on Deep Foundations, Beijing, China, September 1-5, 1989. All enquiries to: Deep Foundations Institute, 120 Charlotte Place, Englewood Cliffs, New Jersey 07632, U.S.A.

8th European Conference on Earthquake Engineering, Lisbon, Portugal, September 7-12, 1986. All enquiries to: 8ECEEE, a/c Laboratorio Nacional de Engenharia Civil, Av. do Brasil 101, 1799 Lisboa Codex, Portugal.

Planning and Engineering Geology, Plymouth, U.K., September 8-11, 1986. All enquiries to : The Geological Society, Burlington House, Piccadilly, London W1V 0JU, England.

Third International Conference on Numerical Methods for Non-Linear Problems, Dubrovnik, Yugoslavia, September 15-18, 1986. All enquiries to Dr. E. Hinton, Dept. of Civil Engineering, University College of Swansea, Singleton Park, Swansea SA2 8PP, U.K.

2nd International Conference on the Bearing Capacity of Roads and Airfields, Plymouth, England, September 16-18, 1986. All enquiries to: Conference Organiser, 1986 Roads and Airfields Conference, P.O. Box 18, Fishponds, Bristol BS16 4NX, England.

Prediction Symposium on the Performance of a Reinforced Earth Embankment, London, England, September 17-18, 1986. All enquiries to: R.H. Bassett, Dept. of Civil Engineering, King's College, London WC2R 2LS, England.

VIII Danube European Conference on Soil Mechanics and Foundation Engineering, Nuremberg, F.R. Germany, September 24-26, 1986. All enquiries to: Prof. U. Smoltczyk, Deutsche Gesellschaft für Erd-und-Grundbau, e.V., Kronprinzenstrasse 35A, D-4300, Essen, 1, F.R. Germany.

5th IAEG Congress, Buenos Aires, Argentina, October 20-26, 1986. All enquiries to: Dr. M. Primel, L.C.P.C., 58, Boulevard Le febvre, 75732 Paris Cedex 15, France.

Geotech 4, Boston, MA, USA, October 27-29, 1986. All enquiries to: ASCE, 345 East 47 St., New York, New York 10017, USA.

International Symposium on Engineering in Complex Rock Formations, Beijing, China, November, 1986. All enquiries to: ECRF Secretariat, Institute of Geophysics, Academia Sinica, P.O. Box 928, Beijing, China.

International Conference on Coastal Engineering, Taipei, Taiwan, November 9-14, 1986. All enquiries to: ASCE, 345 East 47 St., New York, New York 10017, USA.

Mining Latin America, Santiago, Chile, November 17-21, 1986. All enquiries to: The Institution of Mining and Metallurgy, 44 Portland Place, London W1N 4BR, England.

The Role of Geology in Urban Development in Southeast Asia (Landplan III), Hong Kong, December 15-20, 1986. All enquiries to: Conference Secretary, Geological Society of Hong Kong, c/o Dept. of Geography and Geology, University of Hong Kong, Pokfulam Road, Hong Kong.

Indian Geotechnical Conference, New Delhi, India, December 18-20, 1986. All enquiries to: Prof. S.K. Gulhati, Civil Engineering Dept., Indian Institute of Technology, New Delhi 110 016, India.

8th Symposium on Earthquake Engineering, Roorkee, India, December 29-31, 1986. All enquiries to: Prof. B.V.K. Lavania, Dept. of Earthquake Engineering, University of Roorkee, Roorkee 247667, India.

Second International Conference and Short Course on Constitutive Laws for Engineering Materials in Theory and Application, Tucson, Arizona, USA, January 5-10, 1987. All enquiries to: Dept. of Civil Engineering and Engineering Mechanics, University of Tucson, Tucson, Arizona 85721, USA.

International Conference on Computational Plasticity, Barcelona, Spain, April 6-10, 1987. All enquiries to: Prof. D.R.J. Owen, Dept. of Civil Engineering, University College of Swansea, Singleton Park, Swansea SA2 8PP, U.K.

Drillex 87, Stoneleigh, Warwickshire, England, April 7-10, 1987. All enquiries to: The Institution of Mining and Metallurgy, 44 Portland Place, London W1N 4BR, England.

International Symposium on the Engineering Geological Environment in Mountainous Areas, Beijing, China, May 3-7, 1987. All enquiries to: Prof. Wang Sijing, Institute of Geology, Academia Sinica, P.O. Box 634, Beijing, China.

Symposium on Coastal Lowlands: Geology and Geotechnology, Hague, The Netherlands, May 25-27, 1987. All Enquiries to : c/o Congrex, Keizers gracht 610, 1017 EP Amsterdam, The Netherlands.

Fifth International Conference on Applications of Statistics and Probability in Soil and Structural Engineering, Vancouver, Canada, May 25-29, 1987. All enquiries to: Institute of Risk Research, University of Waterloo, Waterloo, Ontario N2L 3G1, Canada.

VIII Asian Regional Conference on Soil Mechanics and Foundation Engineering, Kyoto, Japan, July 20-24, 1987. All enquiries to: 8ARC SMFE, c/o Kyoto International Conference Hall, Takara-ike, Sakyo-ku, Kyoto, 606, Japan.

Geotechnical Engineering of Soft Soils, Mexico City, Mexico, August 12-13 1987. All enquiries to: Manuel J. Mendoza, Instituto de Ingenieria-Unam, Apdo, Postal 70-472, 04510, Mexico.

8th Panamerican Conference on Soil Mechanics and Foundation Engineering, Cartagena, Colombia, South America, August 16-21, 1987. All enquiries to: J. Montero-Olarte, P.O.Box 057045, Bogota, D.E. Columbia.

6th International Congress on Rock Mechanics, Montreal, Canada, August 30-September 3, 1987. All enquiries to: ISRM Congress 1987, 400-1130 Sherbrooke St. W., Montreal, H3A 2M8, Quebec, Canada.

Ninth European Conference on Soil Mechanics and Foundation Engineering, Dublin, Ireland, August 31-September 3, 1987. All enquiries to: Dr. Trevor Orr, Secretary General, IX ECSMFE, Civil Engineering Department, Trinity College, Dublin, 2, Ireland.

9th African Regional Conference on Soil Mechanics and Foundation Engineering, Lagos, Nigeria, September 15-18, 1987. All enquiries to: 9 ARC, Nigerian Geotechnical Association, P.O. Box 2100, Lagos, Nigeria.

Fourth International Conference on Polypropylene Fibers and Textiles, Nottingham, U.K., September 23-25, 1987. All enquiries to: M.D. Shuttleworth, The Plastics and Rubber Institute, 11 Hobart Place, London SW1W 0HL, U.K.

9th Southeast Asian Geotechnical Conference, Bangkok, Thailand, December 7-12, 1987. All enquiries to: 9th SEAGC, c/o Division of Geotechnical and Transportation Engineering, Asian Institute of Technology, GPO Box 2754, Bangkok 10501, Thailand.

Asian Mining'88, Kuala Lumpur, Malaysia, March 8-11, 1988. All enquiries to: The Institution of Mining and Metallurgy, 44 Portland Place, London W1N 4BR, England.

Tunnelling'88, London, England, April 17-22, 1988. All enquiries to: The Institution of Mining and Metallurgy, 44 Portland Place, London W1N 4BR, England.

Vth International Symposium on Landslides, Lausanne, Switzerland, July 10-15, 1988. All enquiries to: Cristophe Bonnard, P.O. Box 83, CH-1015, Lausanne 15, Switzerland.

International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka City, Japan, October, 1988. All enquiries to: Prof. N. Miura, Dept. of Civil Engineering, Saga University, Saga 840, Japan.

28th International Geological Congress, Washington D.C., U.S.A., July, 1989.

12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, 1989. All enquiries to: P.A. Neme, Associacao Brasileira de Mecanica dos Sols, P.O. 7141, Sao Paulo 01000, SP, Brazil.

ANNOUNCEMENTS

Dr. Chai Muktabhant Receives Honorary Doctorate Degree from AIT

Dr. Chai Muktabhant, a founding council member of the Southeast Asian Geotechnical Society, recently received an Honorary Doctorate Degree from the Asian Institute of Technology. Dr. Chai is well known in Thailand and in the region as an expert in soil mechanics and foundation engineering, and above all he was one of the key persons in the earlier years of the establishment of the SEATO Graduate School of Engineering -- the predecessor institution of what is presently the Asian Institute of Technology. As a Thai participant of the working group of the SEATO Graduate School, Dr. Chai was actively involved in planning the curriculum, faculty, finance and staff of the Institute in the very difficult period of its formative years. That the Asian Institute of Technology even exists at all, or exists as it does today, is naturally the result of the efforts of a large number people, and Dr. Chai, a devoted academic and a professional engineer is one of the few who made contributions which are truly significant.



Dr. Chai obtained his Bachelor's and Master's degrees in Civil Engineering from the Chulalongkorn University and an LL.B degree from Thammasat

University. He then proceeded to the U.S.A. where he earned his Master's Degree from Yale University and the Doctor of Technical Science from the University of Michigan, Ann Arbor. The credit for introducing soil mechanics and foundation engineering in the undergraduate curriculum of the Chulalongkorn University and indeed in Thailand should go to Dr. Chai. He joined Chulalongkorn University in 1959 as an Assistant Professor and gradually worked up the ladder and became the Dean of Engineering in 1973. Dr. Chai received an honorary doctorate degree from Chulalongkorn University in 1967 for his outstanding contribution as an educator in engineering in Thailand. This is also the period the Asian Institute of Technology and its predecessor, the SEATO Graduate School of Engineering, was housed in the Chulalongkorn University.

In addition to his academic and professional contributions, Dr. Chai has also been very active in many aspects of community service and served as a member of the Task Group to preserve the Temple of Dawn and the Marble Temple. As a Fellow of the Applied Science of the Royal Science Institute of Thailand, and the President in the Engineering Field of the Industrial Research Division of the National Research Council of Thailand, Dr. Chai has often been requested to advise the Royal Thai government on numerous development projects.

As a founding Council member of the Southeast Asian Geotechnical Society, Dr. Chai has always been unanimously elected to the Council from 1967 to date. His contribution and wisdom in maintaining the harmony among the members of the Society, and his personal contribution to the advancement of the subject, particularly in Thailand and in S.E. Asia in general, is worthy of praise. To be able to contribute significantly in geotechnics and indeed in any profession demands the virtues of good sense and sound judgement; both are possessed abundantly by Dr. Chai. Those who know Dr. Chai Muktabhant very closely always admired his extreme modesty, with a total absence of pretension. Such qualities are rare and when combined with the love and affection he has shown to students both at the Chulalongkorn University and the SEATO Graduate School of Engineering have made Dr. Chai a unique and respected educator. In recognition of his outstanding contribution and services to the entire Thai community, Dr. Chai has been awarded the Knight Grand Cordon (Special Class) of the Most Noble Order of the Crown of Thailand.

We look forward to the continuous contribution and support from Dr. Chai Muktabhant in the future, as well to the development of geotechnics in Thailand and S.E. Asia.

LANDPLAN III- A Symposium on the Role of Geology in Urban Development in Southeast Asia

The LANDPLAN III Symposium will be held at the University of Hong Kong on December 15-20, 1986. The Symposium continues the series of conferences on the impact of soils, geology and land forms on land use planning in developing countries that commenced in 1982 under the aegis of the Association of Geoscientists for International Development (AGID).

Themes of the symposium include all aspects of geology related to urban development including:

- Land Use Planning
- Construction Materials
- Site Investigations
- Slope Failures and Ground Subsidence
- Marine Studies for Harbors, Reclamations and Foundations
- Use of Underground Space
- Waste Disposal
- Hydrogeology and Water Supply
- Coastal Management
- Seismic, Volcanic and other Hazards
- Education of Geologists for Urban Planning and Construction

In addition, workshops and training courses will be held during the symposium.

Correspondence should be directed to:

Conference Secretary
Geological Society of Hong Kong
c/o Dept. of Geography and Geology
University of Hong Kong
Pokfulam Road
Hong Kong

Eighth Asian Regional Conference on Soil Mechanics and Foundation Engineering

The Eighth Asian Regional Conference on Soil Mechanics and Foundation Engineering will be held in Kyoto, Japan, July 20-24, 1987. The object of the conference is to provide an opportunity for engineers and scientists working in the field of soil mechanics and foundation engineering to meet and present new ideas, achievements and experiences. There will be a special emphasis on practical applications, and papers dealing with engineering practice. The conference themes are the following:

- 1) Development of theory and practice in geotechnical engineering
- 2) Problems of regional soils
- 3) Soil dynamics and geotechnical aspects of earthquake engineering
- 4) Tunneling and excavation in soils
- 5) Deep and shallow foundations
- 6) Embankments and slope stability

The official languages of the conference will be English and Japanese. Simultaneous translation will be provided for oral presentations. The papers submitted should, however, be written in English. All correspondence regarding the conference should be directed to:

T. Adachi, Secretary of the 8th ARCSMF
Kyoto International Conference Hall
Takara-ike, Sakyo-ku, Kyoto 606
JAPAN

During the week preceding the conference a short course on "Construction in Soft Ground" will be held at the Kobe Convention Center in Kobe, Japan on July 17-18, 1987.

Ninth Southeast Asian Geotechnical Conference

The Ninth SEAGC is to be held December 7-12, 1987 at Bangkok, Thailand in conjunction with the 20th anniversary celebration of the Southeast Asian Geotechnical Society. The Conference theme is "Geotechnical Engineering in Southeast Asia" with specific emphasis on:

- (i) Engineering Behaviour of Soils and Rocks
- (ii) Site Investigation and Insitu Tests
- (iii) Ground Improvement
- (iv) Shallow and Deep Foundations
- (v) Settlement of Structures
- (vi) Earth and Earth Supported Structures

Papers relating to this theme are invited. Summaries of papers, not exceeding 300 words in length, should be sent to the Hon. Secretary by July 30, 1986. All enquiries should be addressed to:

The Hon. Secretary, 9th SEAGC,
c/o Division of Geotechnical &
Transportation Engineering,
Asian Institute of Technology,
P.O. Box 2754
Bangkok 10501,
Thailand.

XII International Conference on Soil Mechanics and Foundation Engineering

The Brazilian Society of Soil Mechanics, on behalf of the International Society of Soil Mechanics and Foundation Engineering, takes pleasure in inviting the members of the International Society, their companions, and others who are interested, to attend the XII International Conference on Soil Mechanics and Foundation Engineering to be held in Rio de Janeiro, Brazil in 1989. Bulletins containing information on conference themes, submission of papers, and the conference program will be distributed through the National Societies during the 1986-1989 period. The official languages of the International Society are English and French. At all sessions, simultaneous translation will be provided in English and French. An interesting and exciting social program is being organized. The official travel agent is preparing packages with the purpose of offering to the participants the opportunity to see some of the most interesting attractions of Brazil. In connection with the conference, an exhibition will be arranged to display recent developments in geotechnical engineering equipment and techniques. Brazil is a country of continental dimensions with an area of more than 8.5 million square kilometers. Its population is over 130 million. Rio de Janeiro was founded in 1565 and served as the capital of Brazil from 1763 to 1960 when the seat of government moved to Brasilia. More information can be obtained from:

Organizing Committee
XII ICSMFE, Caixa Postal 1559
2000 Rio de Janeiro, RJ
BRAZIL

Golden Jubilee Commemorative Volume of the Southeast Asian Geotechnical Society

Geotechnical Engineering in Southeast Asia, a commemorative volume of the Southeast Asian Geotechnical Society, is now available from Balkema Publishers. This volume contains 11 contributions from Southeast Asia ranging from the behaviour of piles in soft organic clays to landslides and their control. It also includes bibliographies of books, journals, conference publications, theses, etc (c. 2,200 titles) and a directory of consultants and contractors in Southeast Asia. This 352 page document (US\$ 40) can be obtained from:

A.A. Balkema Book Distributors
P.O. Box 1675, NL-3000 BR Rotterdam
Netherlands

Residual Soil Sampling Review Volume

Sampling and Testing of Residual Soils : A Review of International Practice, Edited by E.W. Brand and H.B. Phillipson, is now available from Scorpion Press. This volume was produced by a Technical Committee of the International Society for Soil Mechanics and Foundation Engineering, and includes reviews of practice in 19 countries, as well as a comparative review of international practice. This 194 page volume (US\$ 25 including surface postage of US \$ 30 including airmail postage) is available from:

Scorpion Press
P.O. Box 90674
Tsimshatsui Post Office
Hong Kong

General Committee Members of SEAGS

The Governing Committee of the Southeast Geotechnical Society for the period 1985-1987 is as follows:

Prof. A.S. Balasubramaniam (President)
Dr. Ting Wen Hui (Immediate Past President)
Dr. E.W. Brand (Past President)
Dr. Za-Chieh Moh (Founder President)
Dr. Tan Swan Beng (Past President)
Prof. Chin Fung Kee (Past President)
Mr. D.R. Greenway (Editor)
Dr. Chai Muktabhant
Prof. Seng Lip Lee
Mr. A.A. Beattie
Dr. Ooi Teik Aun
Mr. Yu Cheng
Mr. Jose Rolando R. Santos

SEAGS Newsletter

All correspondence related to the SEAGS Newsletter should be addressed to:

Dr. Dennes T. Bergado,
Southeast Asian Geotechnical Society
c/o Division of Geotechnical and Transportation Engineering,
Asian Institute of Technology,
P.O. Box 2754
Bangkok, Thailand 10501.

SI UNITS AND SYMBOLS

The following list of quantities, SI (Système International) units and SI symbols, are recommended for use in Geotechnical Engineering.

Quantities	Units	Symbols
Length	kilometre	km
	metre	m
	millimetre	mm
	micrometre	μm
Area	square kilometre	km^2
	square metre	m^2
	square millimetre	mm^2
Volume	cubic metre	m^3
	cubic millimetre	mm^3
Mass	tonne	t
	kilogramme	kg
	gramme	g
Density ρ (mass density)	tonne per cubic metre	t/m^3
Unit weight γ (weight density)	kilogramme per cubic metre	kg/m^3
Force	kilonewton per cubic metre	kN/m^3
	meganewton	MN
Pressure	kilonewton	kN
	newton	N
	megapascal	MPa
	kilopascal	kPa
Energy	megajoule	MJ
	kilojoule	kJ
	joule	J
Coefficient of volume compressibility or swelling m_v	1/megapascal	MPa^{-1}
	1/kilopascal	kPa^{-1}
Coefficient of consolidation or swelling c_v	square metre per second	m^2/s
	square metre per year	m^2/year
Hydraulic conductivity k (formerly coefficient of permeability)	metre per second	m/s

NOTES: The term specific gravity is obsolete and is replaced by relative density. The former term relative density $(e_{\text{max}} - e)/(e_{\text{max}} - e_{\text{min}})$ is replaced by the term density index, I_D .

NOTES FOR THE GUIDANCE OF AUTHORS

Manuscripts of original papers, technical notes, and discussions should be submitted to the **Editor, Geotechnical Engineering, P.O. Box 2754, Bangkok, Thailand**. *Three copies* of the manuscript, in English, are required (*two copies* for technical notes and discussions) together with the original drawings.

Papers submitted will be reviewed and accepted on the understanding that they have not been published elsewhere prior to their publication in **Geotechnical Engineering**. It is suggested that a maximum of 12,000 words be used as a guide for the length of contributions.

The format which must be followed for the preparation of manuscripts is that adopted in this issue of the Journal. The following notes are for guidance:

1. The manuscript should be typewritten on one side of the paper, with double-line spacing and wide margins.
2. The author's full name, position and affiliation should appear on the first page of the manuscript together with a *Synopsis* of not more than 200 words. A *Synopsis* is not required for technical notes.
3. Main headings should be upper case in the centre of the page. Sub-headings should be lower case, except for the first letter of each word, without indentation and underlined. Subdivisions should be lower case, indented five spaces and underlined, with text following on immediately.
4. Figures should be drawn boldly in black ink on one side of good quality tracing paper. Photographs should be black and white glossy prints with good contrast. All illustrations should not be more than twice the size of the final reproduction, which will be a maximum width of $4\frac{1}{2}$ in. (11.3 cm). No captions should be placed on figures but these should be listed on a separate sheet.
5. Formulae should be expressed as simply as possible, keeping in mind the difficulties and limitations encountered in typesetting.
6. Symbols should be defined when they first appear and should conform as far as possible with the list published in Volume III of the Proceeding of the Fifth International Conference on Soil Mechanics (Paris, 1961).
7. References should appear in the text as the author's name in capitals followed by the year of publication. A list of references should be given at the end of the text in alphabetical order of authors' names. Do not abbreviate bibliographic references other than those for volume, number, and page.

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