# SITE INVESTIGATION AND IN SITU TESTING

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Moh and Associates International Ltd.

*Preprint of a paper*

*prepared for the*

*Commemorative Volume in Celebration of*

*50th Anniversary of the ISSMFE,*

*Southeast Asian Geotechnical Society, 1985*

# Site Investigation and In Situ Testing

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## INTRODUCTION

On the ocassion of the fiftieth anniversary of the International Society for Soil Mechanics and Foundation Engineering, the Southeast Asian Geotechnical Society is compiling a commemorative volume of articles describing the various geotechnical activities and practices in the countries covered by the SEAGS. This article describes some of the highlights of the site investigation practices and types of in situ testings carried out for determination of the in situ characteristics of soil deposits. Due to limitation of space, the article cannot be comprehensive in its scope and coverages. Majority of the test results presented in the paper are based on work carried out by the author's firm in various countries in the region.

## SITE INVESTIGATION

For the design of any engineering constructed facility, the responsible engineer must play two important roles. They are : Prediction and Judgement & Decision. In order to arrive at a good prediction, one has to firstly acquire the necessary information and data. Site investigation is the first and foremost important item of work for any civil engineering project. Although the importance of site investigation, particularly its quality in the region has long been recognized by geotechnical engineers, it has not received sufficient attention from the governments, private developers and even the architectural and engineering profession in general. Many governmental agencies and private developers usually have the sense of " false economy " and are reluctant to spend the necessary amount of monies on investigation and data collection. This attitude has prevented the engineers to exercise a satisfactory level of supervision on site investigation work and has inhibited the site investigation industry to upgrade their standards of work. Site investigation work in most of the countries within the Southeast Asian Geotechnical Society's region are carried out by site investigation contractors. Until recently, say in the last four or five years, few of the site investigation contractors had professionally qualified civil or geotechnical engineers or engineering geologists on their staff. The traditional way of operation of the site investigation contractor does not require this type of personnel. Due to the high mobility of drilling crew and lack of proper training, improvement of site investigation standards was greatly hindered. The situation, although has been improving in the past few years due to the gradual recognition of the importance of the geotechnical engineering profession in general but the speed of upgrading and improving has been slow. Up to the present, few of the countries in the region has qualification requirements for the drilling crew. Most of the site investigation works, whether governmental or private, were awarded on price competitive basis. Due to the lack of quality requirements of the work and shortage of good trained personnel, price competition further reduced the chances of improvement in the region because of cost and non-existence of governmental regulatory requirements. Very few site investigation works were supervised by qualified engineers/engineering geologists, even the work was undertaken under the general supervision of consulting engineers.

Although there is no official registration of professional geotechnical engineers exists in Hong Kong, the Hong Kong Government now requires geotechnical submissions be made by organizations which are known to have adequate geotechnical professional resources. In the Republic of China, recent revision of the Classification of Professional Engineers includes geotechnical engineering as one of the branch of engineering which requires registration. It is anticipated that the standards of site investigation will improve rapidly in these two countries. In the Southeast Asian region, geotechnical formations vary greatly, from the very soft marine clays, heterogeneous residual soils, heavily decomposed rocks to fresh rocks. Some fairly sophisticated methods of site investigation and testing techniques are often needed. For advancing boreholes, percussion drilling and rotary drilling are the two most commonly used methods. Until about four or five years ago, percussion drilling has been a very popular type of drillign method, even in very soft soil deposits, in Malaysia and Singapore. Rotary drilling is nearly always employed for major projects. Wash boring techniques are used to advance holes in soil deposits and rotary drilling with core barrels are used in boulders and rocks in all countries in the region. Spilt spoons with or without inner linings are used to take representative soil samples and thin walled tube drive sampler or piston sampler are used to obtain undisturbed samples. Denison sampler, double or triple tube core barrels are used to take samples in dense granular deposits, residual soils and rocks. Special sampling techniques have been adopted for special requirements, such as undisturbed samples of sand for liquefaction study and air foam sampling for residual soils.

There is so far no standards or codes of practice on site investigation and sampling existing in any of the countries in the Southeast Asian region. Majority of the site investigation work follows some of the following three standards or codes of practice :

(i) BS 5930 : Code of Practice for Site Investigation (BRITISH STANDARDS INSTITUTION, 1981).

(ii) ASTMD 1586 - 67 : Penetration Test and Spilt Barrel Sampling of Soils (ASTM, 1979)

(iii) JIS A 1219 - 1961 : Method of Penetration Tests for Soils (JIS, 1973)

In Hong Kong, the Gentechnical Control Office of the Government of Hong Kong has published a Geotechnical Manual for Slopes (GEOTECHNICAL CONTROL OFFICE, 1984) which gives guidelines for site investigation in addition to the BS Code of Practice. In the Republic of China, the Bureau of Standards in cooperation with the Geotechnical Engineering Committee of the Chinese Institute of Civil and Hydraulic Engineering is in the process of drafting standards for site investigation. There are a number of papers published which describe the site investigation practices and sampling techniques used in different countries, such as EIDE (1977), TING (1972), BRAND AND PHILLIPSON (1984).

HIGH QUALITY SAMPLING OF RESIDUAL SOILS

Some of the most difficult types of soils to be sampled are residual soils and colluvium. The presence of boulders, cobbles and gravels within the colluvium often add to the difficulty of successfully obtaining acceptable samples. In connection with a major geotechnical study of the Mid Levels area of the Hong Kong Island, the Geotechnical Control Office of the Hong Kong Government carried out a detail drilling trial study for high quality drilling and sampling. Three types of core barrels were used including a 63mm diameter standard triple tube core barrel with retraction shoe and spilt steel liners (HMLC), a 83mm diameter non-retractable triple tube core barrel incorporating a wireline mechanism for withdrawing the liner barrel (PQ-Wireline), a 73/84mm diameter Mazier triple tube retractor barrel (Mazier/T2-101), and a 102mm large diameter HMLC triple tube barrel (4C-MLC). The other major operational variable studied in the trial was the flushing media. Four types of flushing media were tested; they include the conventional fresh water, drilling mud,air and air foam. Air foam is very much like shaving cream in appearance and requires a low uphole velocity to clean cuttings. The liquid element of the foam was consumed at a rate of about 2.4 liter per min, which compares favorably with the typical consumption of drilling water of 20 - 40 liter per min using standard technique.

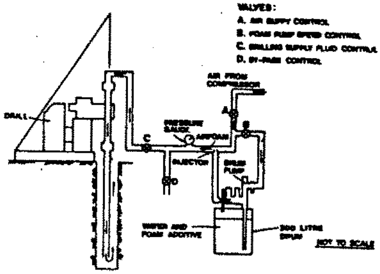


Fig. 1 Air Foam Mixing and Flushing System

Figure 1 shows the air foam mixing and flushing system used in the trial. Based on results of linear core recovery and core quality, the study concluded that the best results were achieved by using the large 4C-MLC barrel with air foam as the flushing medium. High quality cores were obtained with a mean recovery of over 95 per cent. In addition to the optimum choice of core barrel and flushing medium, PHILLIPSON AND CHIPP (1982) have pointed out that there are many other factors relating to equipments and techniques which will influence the success of high quality sampling. The more important factors are drilling rig, flushing medium circulation system, casing and drilling rods, drilling techniques, drilling crew, core handling, specifications and supervision. The study report has also recommended important points to be included in specifications for high quality site investigation drilling which particularly emphasizes the importance of the contractor's qualifications and supervisors. (PHILLIPSON AND CHIPP, 1981).

STANDARu PENETRATION TEST

Standard Penetration Test is the most common in situ test for soil identification. It is almost invariably carried out in every site investigation work along with borings. Although the test specifies a standard procedure by allowing a standard hammer, 63.5 kg (140 lb) in weight, to drop freely over a distance of 76 cm (30 in). The number of blows required to cuase a 50.8 mm (2.0 in) diameter spilt spoon sampler to penetrate the soil for 30 cm (12 in) is called the Standard Penetration Resistence N. In fact, there are many variables which affect the reproducibility and reliability of Standard Penetration Test including personnel, equipment and procedures. The variables affecting the SPT, which are summarized by KOVACS et al (1981), directly or indirectly affect the energy that is transferred through the drill rods to the sampler, and thus the N value. SCHMERTMANN AND PALACIOS(1979) concluded that the energy reaching the sampler is inversely proportional to the blow count N. By measuring the energy passing through the drill rods, SCHMERTMANN AND PALACIOS (1979) found that the actual energies for the SPT using a number of different drill rigs varied from 30 to 85 per cent of the free fall energy. A very comprehensive study on this problem was carried out (KOVACS et al, 1983) in the USA. They concluded that due to the large number variables it was impossible to make a statistically significant estimate of the reference energy which is of average energy delivered in the US practice.

Since the SPT-N values are often used for empirical characterization of soils and for preliminary geotechnical analyses and designs, it is therefore important to realize the accuracy of the N value before any empirical relations are to be used. A study was carried out by MOH AND ASSOCIATES INC (1984) in conjunction with a site selection study for future nuclear power plant. The energies imparted by the standard 63.5 kg hammer in SPT were measured. Two methods of releasing the hammer, rope cathead and free-fall, which are commonly employed by the drilling workers in Taiwan, were determined. In the conventional rope-cathead method, a rope is connected to the hammer and wrapped around the cathead with 1.8 turns. The hammer is lifted to the standard height of 76 cm and the rope is then released to let the hammer dropped onto the anvil. In the free-fall method, the same rope and cathead system is used but a small trap hammer is utilized to connect the hammer and the rope. After the hammer is lifted to the desired height, the hammer is suddenly released from the trap hammer by a trigger. In this way, friction between the rope and cathead is eliminated and the height of fall is maintained at 76 cm.

As shown in Figure 2, the average energy ratio, i.e. the ratio of energy measured to the theoretical energy of SPT, obtained from the free-fall method is about 2 per cent higher than that from the rope cathead method. In real practice, the difference is expected to be larger, around 5 to 10 percent, since in this test series, the rig operators were more conscious about the operation than they usually would do. The important fact revealed by these tests results is that the actual energy imparted in the SPT is much lower than the theoretical SPT energy. The 73 - 75 percent energy ratio should be considered as the upper bound of energy ratios in SPT work in Taiwan.

CONE PENETRATION TEST (CPT)

Cone Penetration Testing is a method of soil investigation as well as in situ testing. Its basic principle arises from a concept that the resistence to penetration of different types of soil will be different, the resistence will be higher if the soil is firmer. Depending upon the method of advancing the cone penetrometer into a soil deposit, the CPT can be classified into static, dynamic and static-dynamic penetrometers. There are many different types of static cone penetromenters of varying sizes and shapes being used around the world. Basically, the static cone penetrometers commonly in use can be classified into three types, i.e. the friction sleeve type, the movable cone tip type and the fixed cone type, as illustrated in Figure 3.

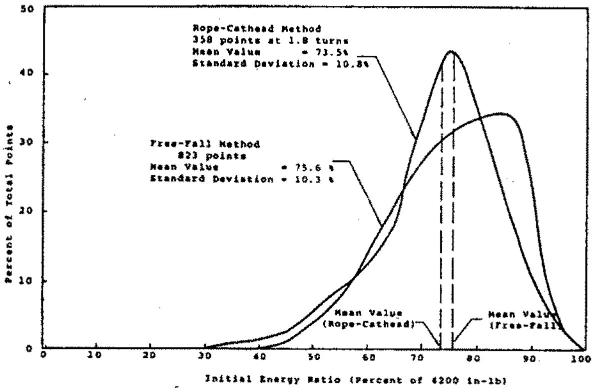


Fig. 2 Statistic Results of SPT Energy Measurement

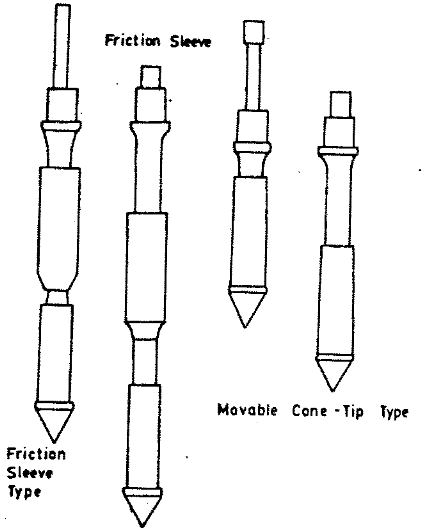


Fig. 3 Types of Static Cone Penetrometer

Among the over hundred static penetrometers used, the one developed in the Netherlands, commonly referred to as the Dutch Cone Penetrometer, is the most popular one. It has been used extensively in the Netherlands for over 40 years for site investigation and for prediction of pile behavior, with about 80,000 CPT tests performed yearly (BEGEMANN et al, 1982). In the Southeast Asian countries, the CPT method is gaining interest in recent years, but the application of the CPT results is generally limited to the identification of soil profiles.

The Dutch cone has an apex angle of 60 degrees and a cross-sectional area of 10 sq cm. Loads are transferred to the cone through metal rods. With friction sleeve, the cone resistence and local friction can be determined. Results of the CPT tests are usually presented graphically as a plot of cone resistence and local friction against depth of soil profile. The ratio of the cone resistence and local friction expressed in percentages is called friction ratio which is a characteristic of the soil type.

The cone resistence and the local friction can be measured in two different ways. They are known as the mechanical and electrical methods. With the machanical method, the force exerted on the inside rods to push the cone out its tip gives the cone resistence which can be measured above ground level through a pressure cell. Measurement of the force required to push the cone together with the friction sleeve is done similarly. In the electrical method, the cone resistence as well as the local friction are measured by means of electrical strain gauges placed near the spot where the soil forces are acting, thus eliminating the effects of rod friction. Due to the inherent difference in the nature of the measuring equipment and methods of measurements, the two types of cone will give different test results. However, due to the relatively low cost of the equipment and simple in operation, the mechanical cone is still in wide use. On the other hand, the electrical cone is more sophisticated and requires more care and knowledge to operate and the cost of the equipment is much higher, but it tends to give more accurate results of the true soil behavior.

The magnitude of resistence which is felt by a penetrating cone depends on many factors. For granular soils, the cone as well as the friction resistence are generally determined by characteristics such as density , angle of shearing resistence, rigidity of the layers and the existing stress conditions. In saturated cohesive soils, the cone resistence is determined to a great extent by the undrained shear strength. In addition to the soil types and the method of measurement, the cone resistence and also the friction resistence are affected by the size and shape of the cone and rate of penetration. It is thus important to standardize the type of cone apparatus and method of testing if cone penetration results are to be used for geotechnical design purpose.

The CPT results can be used for both qualitative and quantiitve interpretation of the following :

a) determination of the stratification of the subsoils,

b) determination of the undrained shear strength of cohesive soils,

c) determination of other engineering characteristics of soils,

d) estimation of end bearing and frictional resistence of piles.

## (1) Determination of Soil Stratification

Due to its high dissolving capacity particularly when continuous records of cone penetration and local friction resistence are obtained, the CPT can identify the presence of thin layers of different soil strata which the conventional boring method often misses. It has been found that the friction ratio has a definite relationship with the type of soil. Since the physical properties of the soil deposit, which are reflections of geological history and weathering process of the formation, may vary considerably from place to place, the application of cone resistence and friction ratio for classification and identification of soil stratification depends greatly upon the amount of data accumulated at any particular locality of region. In other words, empirical relationships have to be established for positive identification of soil strata. BEGEMANN et al (1982) has presented graphs relating cone resistence, friction ratio and soil type for soil deposits in the Netherlands as shown in Figure 4. These graphs were developed over many years on the basis of a large number of CPT test results.

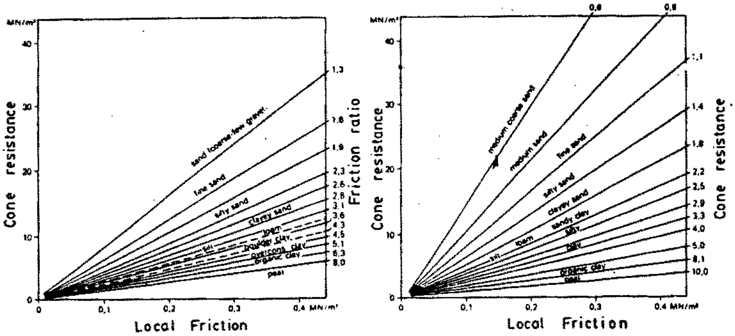


Fig. 4 Relationship Between Friction Ratio and Soil Type in The Netherlands

SANGLERAT (1972) has gathered results from many investigations and concluded that :

i) Generally the friction ratio for clays is in the order of 5 to 6 per cent. For peat and soft soils, the ratio is in the order of 8 per cent and may go up as high as 15 per cent when the local friction is close to the value of the undrained shear strength of the soil;

ii) For sands of varying compactness, the friction ratio is in the order of 0.5 to 2 per cent.

In Thailand, some studies have been carried out to use the CPT in site investigation work. On the basis of some limited results obtained from the CPT tests with mechanical cone and soil borings in Bangkok and its vicinity, WIROJANAGUD (1972) suggested a soil classification systen for the soft Bangkok Clay deposit by using Dutch CPT results, as shown in Table 1.

Table 1: Soil Classification by Dutch CPT Results

| Cone Resistence $kN/m^2$ | Undrained Shear Strength $kN/m^2$ | Friction Ratio % | Soil Type |
|---|---|---|---|
| 2 - 10 | - | 4 - 8 | Highly weathered Clay crust |
| 1 - 5 | Less than 25 | 1 - 5 | Very soft clay |
| 5 - 8 | 25 - 50 | 1 - 5 | Soft clay |
| 10 - 80 | 50 - 100 | 1 - 6.5 | Stiff clay |
| 130 - 200 | - | 1 - 3 | Fine to medium sand |

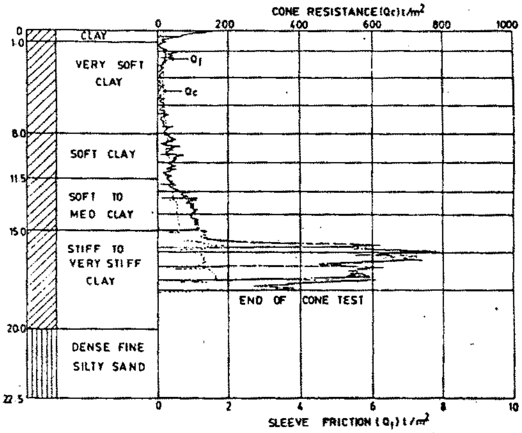


Fig. 5 Soil Profile and CPT Results in Soft Clay Deposit - Nong Ngu Hao, Thailand

Figure 5 shows a comparison of Dutch CPT results with a soil profile identified from boring and laboratory testing. The data are one of the many test results obtained at Nong Ngu Hao, a site located at about 20 km east of Bangkok City. It is interesting to note that the CPT results clearly show the presence of several thin sand lenses at various depths which were not detected in the boring record but could only be identified by careful examination of cut sections of continuous samples. Figure 6 shows CPT results obtained at the Taichung Harbor in Taiwan (LEE et al, 1984). The CPT results indicate that the subsoil profile in that region does not have very distinct stratifications, consisting primarily medium dense fine sands or silty sand.

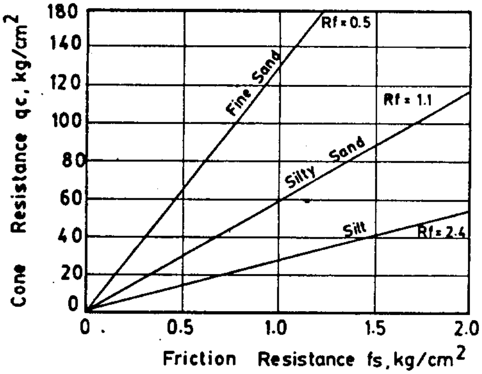


Fig. 6 Soil Classification and CPT Results In Taichung Harbor Area, Taiwan

But the sand layer contains interstrafied thin seams of soft and compressible silty clays and sandy silts which are revealed by the low cone resistence and high friction ratio peaks on the CPT results. Based on the study results, a soil classification chart for soil deposits in the Taichung Harbor area by using CPT results was established as shown in Figure 7.

(2) Correlation Between Cone Resistence With Undrained Shear Strength of Saturated Cohesive Soils

Cone penetration testing in clay should be considered as a quick test under undrained conditions. Based on conventional theories, it can be expected that the cone resistence $q_c$ relates directly to the undrained shear strength $s_u$

$$q_c = N_k \cdot s_u \quad \ldots\ldots(1)$$

where $q_c$ = cone resistence,
$s_u$ = undrained shear strength
$N_k$ = cone factor

The value of the $N_k$ depends on the shape of the cone and the method of testing as well as the soil characteristics, particularly its compressibility and sensitivity. BEGEMANN et al (1982) reported that for normally consolidated clays in the Netherlands, $N_k$ equals to 13 to 16 when the test is performed with a mechanical penetrometer with discontinuous measurement, and the $N_k$ value is equal to 8 to 12 for electrical penetrometer. On the basis of theoretical analysis of results from a number of sources, SANGELRAT (1972) suggested that approximately the cone factor varies between 15 to 18 for normally consolidated clays and between 22 and 26 for overconsolidated clays.

For soft clay deposits in the Bangkok area, the following cone factors were found :

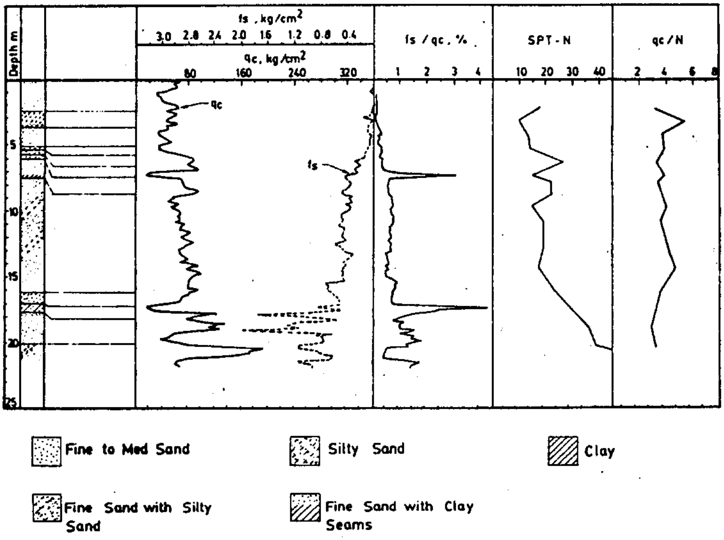


Fig. 7 Soil Profile and CPT Results in Sandy Deposit - Taichung Harbor, Taiwan, ROC.

(i) For soft Bangkok Clay with mechanical cone (PHAM, 1972)

$N_k = 14$

(ii) In soft Nong Ngu Hao Clay with mechanical cone (BRAND et al, 1974)

$N_k = 19$

(iii) In the Nong Ngu Hao clays with electrical cone (ENGINEER FOR SBIA, 1984)

$N_k = 12.5$ (soft clay above 11 m)

$N_k = 15.0$ (medium clay, 11 - 15 m)

(3) Correlation Between Cone Resistence With Standard Penetration Resistence

Up to the present, the Standard Penetration Test (SPT) is still the most common test being carried out in almost every site investigation project. The Standard Penetration Resistence N values give a good indication of the relative density of cohesiveless soils and consistency of cohesive deposits. Many empirical relations have been developed between the N value and the other engineering characteristics of the soil. Engineers have been using these empirical relations as guidelines for foundation design. Since CPT tests are much simpler and quicker to be carried out and with high resolutions and repeatability, the potential of using CPT results for preliminary design or design guide is very high. However, just like the use of SPT results, empirical relationships will have to be established. It is therefore worthy to correlate the relationships between these two types of penetration tests. LEE et al (1984) reported that for the sandy deposits in the Taichung Harbor region definite correlations were found between the CPT cone resistence $q_c$ and the SPT N value. The correlations are dependent upon the grain size of the deposit as pointed out by other researchers. Table 2 lists the correlation factor along with results reported by other researchers. Similar to the Terzaghi's (TERZAGHI AND PECK, 1967) classification of cohesiveless soilsby using N value, the CPT results can also be used as shown in Table 3. It should be noted that the N values quoted in Tables 2 and 3 were obtained with the normal procedures commonly used in Taiwan without any correction for energy difference.

Table 2: Relationships between $q_c$ and N

| Type of Soil | $q_c/N$ | | | |
|---|---|---|---|---|
| | Lee et al (1984) | Schmertmann (1978) | Simons | Lacroix |
| Medium to fine sand or silty fine sand | 4.0 | 3 - 4 | 5.5 | 4 - 6 |
| Silty fine sand | 3.7 | 3 - 4 | 4 | 4 - 6 |
| Fine sandy silt and silt | 3.0 | 2 | 2.5 | 2 - 4 |

Table 3: Relationship between $q_c$, N and Relative Density

| Soil Condition | Relative Density, % | N | CPT $q_c$, $kg/km^2$ |
|---|---|---|---|
| Very Loose | < 20 | < 4 | < 20 |
| Loose | 20 - 40 | 4 - 10 | 20 - 40 |
| Medium Dense | 40 - 60 | 10 - 30 | 40 - 120 |
| Dense | 60 - 80 | 30 - 50 | 120 - 200 |
| Very Dense | > 80 | > 50 | > 200 |

PORE PRESSURE PROBE TEST

Pore Pressure probe is a device developed recently for determining the geological profile of soft soils. The system utilizes the pore pressure response for soil identification when the probe penetrates the soil at a constant rate. The generated pore pressure is mainly a function of the varying permeability of the soil layers. In normally consolidated clays, high excess pore pressures are generated. In more permeable soils, such as sand and silt, the penetration of the pore pressure probe normally generates only small excess pore pressures. The probe should have extremely rapid response to changes in the generated pore water pressure. This type of probe is specially suitable for the identification of thin seams of sandsilt or clay embedded in a soft soil deposit. Since the presence of thin seams of permeable soils in an otherwise relatively impervious soft soil deposit has strong influence on the rate of consolidation of the soil under laoding, positive identification of the presence of these seams is of great importance in analysis and design. This is particularly critical if soil improvement schemes are to be implemented. By combining the pore pressure probe with a cone penetrometer, the system becomes a very powerful tool for soil investigation work.

For the soil investigation work for the master plan and preliminary design of the Second Bangkok International Airport, a series of pore pressure probe test (or Piezoprobe Test) was carried out in conjunction with Dutch Cone Penetration Tests. The pore pressure probe system developed by BAT of Sweden was used. This system consists of a set of pore pressure probe assembly, Figure 8, with adaptor and an automatic recorder which can provide continuous record of pore pressure variations as a function of penetrated depth.

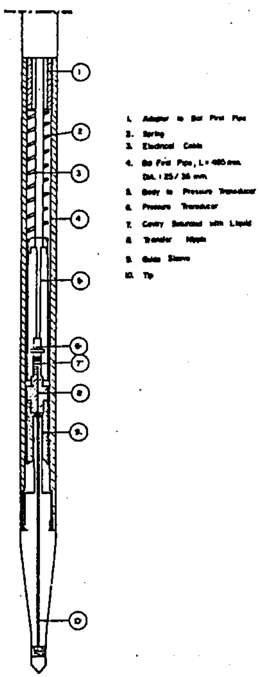


Fig. 8 Pore Pressure Probe

Figure 9 shows a typical record of pore pressure probe test along with the Dutch CPT results. The soil profile comprises of a 15m thick layer of very soft to soft clay which is underlain by a layer of medium clay and then stiff clay, stiff sandy clay and dense sand. The top 1.5 m of the soft clay has been subjected to weathering and therefore has lower moisture content and higher shear strength. The soft clay layer has a very high moisture content, varying from 60 to 110 per cent, which is close to or sometimes even higher than the liquid limit, and contains thin layers of high silt content and sands. The presence of these more permeable layers in the soft clay strutum is clearly shown by the pore pressure probe results. The design of a sand drain-surcharge soil improvement scheme for the project depends greatly upon the identification of the presence of these thin permeable layers.

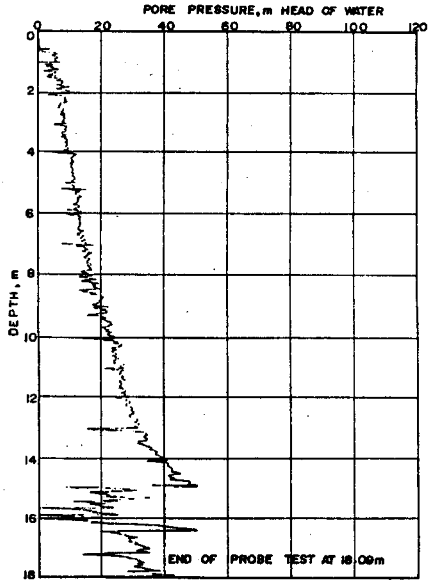


Fig. 9 Pore Pressure Probe
Test Results

By interrupting the penetration of the pore pressure probe and studying the rate of excess pore pressure dissipation, additional information about the geotechnical characteristics of the different soil layers can be obtained. The diagram in Figure 10 shows the result of a pore pressure dissipation test taken at the depth of 4.02m in the soil profile shown in Figure 9.

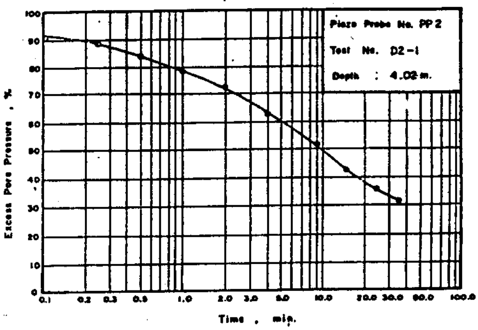


Fig. 10 Pore Pressure Dissipation Test Results

From the dissipation test results, the coefficient of consolidation in the horizontal direction $c_h$ can be calculated by measuring the time required for 50 per cent pore pressure dissipation. Figure 11 shows that the values of the $c_h$ determined in situ from pore pressure dissipation tests are always higher, some 2 to 5 times, than that determined from laboratory oedometer tests. This means that the actual dissipation of excess pore pressure in the field generated by loading is much faster than that predicted by using laboratory determined $c_h$ values. These data have been used in the design of the soil improvement scheme and the results compare favorably with field measured performance of test sections (ENGINEERS FOR SBIA, 1984).

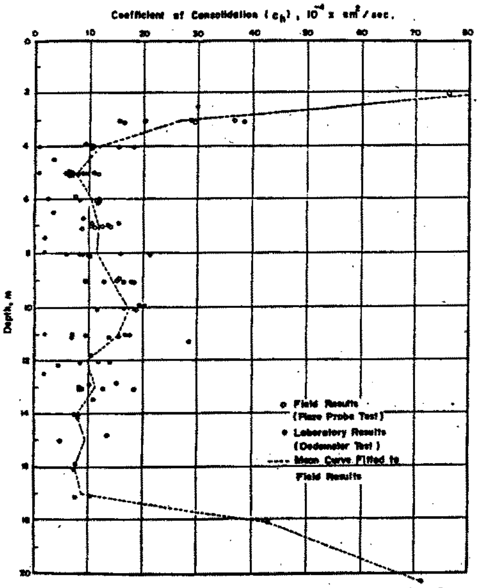


Fig. 11 Comparison of $c_h$ Values Determined from Pore Pressure Probe Tests and Laboratory Tests

PRESSUREMETER TEST

Pressuremeter is a test instrument which can be used to determine the load-deformation characteristics of soils and rocks in situ. The test results are frequently used in the determination of deformation modules and earth pressure at rest. It has also been used in foundation design to evaluate settlement and failure load of foundations. The pressuremeter test has been widely used in Europe, Canada, Japan and Isreal. Only in recent years, this type of test is gradually gaining recognition and popularity in the Southeast Asian countries. The pressuremeter test has several inherent advantages over laboratory tests. They are :

(i) The test is carried out in situ and therefore more representing the actual field characteristics of the soil ;

(ii) The soil to be tested is subjected to less disturbance ;

(iii) Application of stress in the test more similuates the actual behavior of foundations such as piles subjected to lateral loads ; and

(iv) The test can be carried out in soil deposit where undisturbed sampling is particularly difficult.

The pressuremeter unit consists of three components : the probe, the control unit and connecting tubing. There are two types of pressuremeter probe which are commonly in use. They are the Menard Pressuremeter and Single Cell Pressuremeter. The Menard Pressuremeter probe is constructed of a steel cylindrical tube surrounded by two flexible membranes. The inner menbrane forms the measuring cell and the exterior membrane connects two guard cells at the top and bottom ends of the probe. These guard cells are used to reduce end effects on the measuring cell in order to ensure uniformity of stress and deformation conditions around the central measuring cell during test. The most commonly used sizes of Menard probe are 60 mm and 44 mm in diameter. Figure 12 shows the details of a 60 mm probe of the Menard G A Pressuremeter. The single cell pressuremeter was first developed by Professor M. Fukuoka in Japan in the fifties, and was later marketed by the OYO Corporation under the name of Lateral Load Tester (LLT). The main difference between the single cell LLT and the Menard's tricell pressuremeter is the number of cells. Theoretically, the end effect of a single cell probe causes deformation of the surrounding soils different from the plane strain condition which the tricell probe produces. By comparing the test results obtained from the two types of probes, OYO concluded that the values of the coefficient of horizontal subgrade reaction deduced from the test results obtained with a tricell probe are generally about 7% larger than that from monocell probe test results. A model 4120 LLT is shown diagrammatically in Figure 12. The control unit is a system for controlling the pressure and monitoring the volume change of the probe. It performs its function by supplying and regulating pressure from a pressure source, i.e., compressed gas bottle, to the probe and then monitoring volume change of the measuring cell.

The pressuremeter test is usually performed in a borehole. After clearing the borehole to the required depth, the probe is lowered into the hole and pressure is applied tp the borehole by inflating the rubber membrane of the measuring cell. In general, the test is carried out in stress-control condition. Equal increments of pressure are applied to the probe and maintained for a fixed length of time, usually one to two minutes. The test is usually carried out in 8 to 14 increments to reach the limit pressure. Changes in the volume of the measuring cell are recorded at several time intervals after application of each increments of pressure. Since the test is carried out in a relatively short period of time, in cohesive soils, it can be considered as an undrained test.

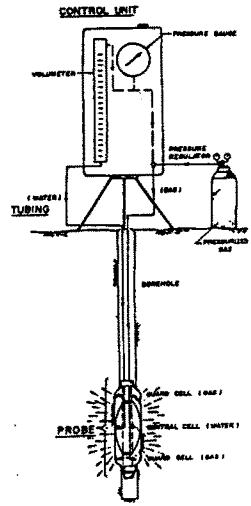


Fig. 12a Menard's Tricell Probe

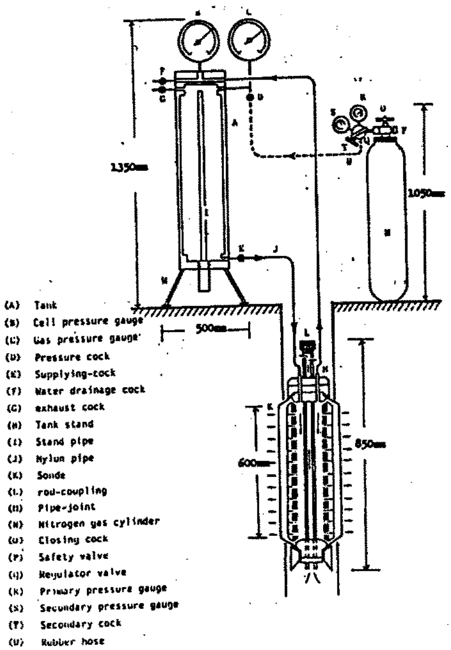


Fig. 12b OYO Model 4120 Monocell Probe

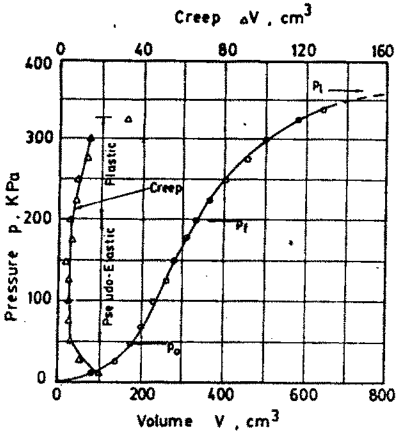


Fig. 13 Typical Results Obtained from Pressuremeter Test

Figure 13 illustrates typical results obtained from a pressuremeter test, which is a plot of readings of the volumeter vs increment of pressure after corrections for hydrostatic pressure in the system tubing, effect of menbrane resistence, volume change of water in the tubing and the tubing itself. From the shape of the stress-deformation curve, there are three characteristic zones and pressures which can be defined.

(a) Contact pressure $p_o$ - The initial part of the curve OA occurs when the probe pushes the side of the borehole back to the original position before the hole was made and before the soil yielded inwards. At point A (Fig. 12), the pressure at rest condition in the ground is considered to be reestablished and at that point, the initial size of the cavity is defined and measured.

(b) Yield Pressure $p_y$ - A linear stress-strain behavior of soil will begin soon after the probe is in contact with the surrounding soil and the pressure $p_o$ is reached. This zone, AB, is the "pseudo-elastic " zone of the soil and the pressure at point B is known as the

" creep pressure " or " yield pressure ".

(c) Limit Pressure $p_l$ - Beyond the pseudo-elastic zone the soil enters a plastic state, where the deformation increases substantially with increase in pressure. This plastic zone starts at Point B and eventually becomes asymptotic to the abscissa at a large deformation. A pressure called limit pressure $p_l$ is defined as the pressure required to expand the cavity to reach a volume of two times of the original size.

In carrying out the pressuremeter tests, there are a number of factors which will affect the test results. They include size of borehole, position of probe at the boundary of two different soil layers, properties of the membrane, imperfection of boreholes and soil inhomogeneity. Some of these factors attributed to boreholes can be eliminated by the use of newly developed self boring pressuremeter. However, the complicated electronics of this type of equipment and difficulty in operation have so far restricted its use to reaearch works.

(1) Application of Pressuremeter Test Results

Based on the straight line pseudo-elastic part of the pressuremeter curve, the Pressuremeter Modulus $E_m$ is defined as (MENARD 1975) :

$$E_m = 2(1 + \upsilon) \cdot \bar{V}_m \cdot \frac{\Delta p}{\Delta V} \quad .....(2)$$

where $\upsilon$ = Poisson's ratio

$\bar{V}_m$ = dimensional coefficient of the probe

$\frac{\Delta p}{\Delta V}$ = slope of pressuremeter curve in the pseudo-ealstic range

GIBSON AND ANDERSON (1963), PALMER (1972), LADANYI (1972) and BAGUELIN et al (1972) have proposed various methods to evaluate the undrained shear strength from pressuremeter tests results.

The settlement beneath a deep foundation can be predicted using the semi-empirical formula proposed by MENARD AND ROUSSEAU (1962) which was later modified by MENARD (1975). The formula utilizes the concept that the total settlement is the sum of volumetric or consolidation and deviator or shear deformations. The bearing capacity of a foundation for a specified allowable settlement can be calculated based on the value of pressuremeter modulus (MENARD, 1975).

The lateral resistence of a deep foundation, from the concept of an infinite beam embedded in an elastic medium, can be determined by solving the differential equation of :

$$\frac{d^4x}{dz^4} = k_s \cdot x \quad ....(3)$$

where z = length of the pile or caisson

x = lateral movement

$k_s$ = coefficient of horizontal subgrade reaction.

Solutions of the differential equation have been presented by DAVISSON AND GILL (1963) for constant $k_s$ and $k_s$ in stepped variation, by MATLOCK AND REESE (1960), and DAVISSON (1970) for $k_s$ varying linearly wit depth. The coefficient of horizontal subgrade reaction $k_s$ can also be derived from pressuremeter modulus by formula described in MENARD (1975).

(2) Some Results of Pressuremeter Tests

CHIANG AND HO (1980) reported the use of pressuremeter tests for foundation design in Hong Kong. Pressuremeter tests by using a Menard G A type pressuremeter were performed in a completely weathered granite soil deposit which is commonly found in northern and central part of Hong Kong Island, and in the southern part of Kowloon. In the Hong Kong granitic formation, the grading of the soil changes from clayey sand near the ground surface through silty sand to coarse sand at greater depth, as the degree of weathering decreased with depth. Figure 14 shows the general properties of the granitic soils tested. The Standard Penetration Resistence N value increased from about 10 to 130 within the completly weathered zone.

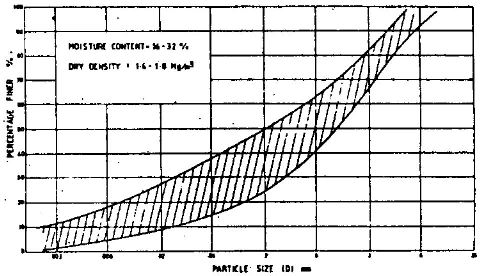


Fig. 14 General Properties of Completely Weathered Hong Kong Granitic Soils

The SPT test with its ease of operation and extensive correlation with soil parameters has been used extensively in foundation design in Hong Kong. It is logical to correlate the parameters obtained from pressuremeters with the N values. Figure 15 presents such correlations. The coefficient of correlations between the Limit Pressure $p_l$ and N is 0.885 by least square method.

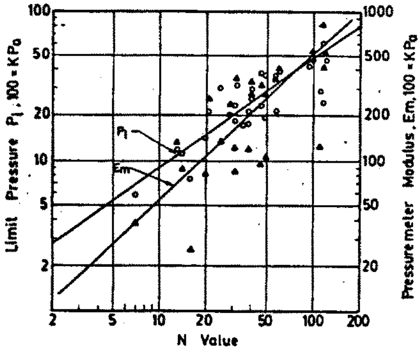


Fig. 15 Correlation of Limit Pressure and Pressuremeter Modulus with N-value for Hong Kong

Values of the coefficient of modulus variation, $n_h$ for calculating the lateral resistance capacity of foundation caissons embedded in completely weathered granite were determined from the pressuremeter tests as shown in Table 4.

Table 4: Coefficient of Modulus Variation in Completely Weathered Hong Kong Granite

| Relative Density | Medium Dense (N <50) | Dense (N >50) |
|---|---|---|
| Range of $n_h$ | 2 - 5 | 5 - 16 |
| Range of $n_h$ | 3.8 | 7.8 |

Attempt was made to use the pressuremeter test results for estimating bearing capacity of caissons. It was found that the bearing capacity value calculated from pressuremeter results was 2 to 3 times larger than that calculated by other conventional methods such as TERZAGHI and PECK (1967) and MEYERHOF (1956). Full scale load test will need to be performed to verify the validity of the use of pressuremeter tests in foundation design.

In conjunction with geotechnical investigation works for the proposed Taipei Railway Underground, Hsin Sheng North Road vehicular overpass and Chih Ching Building projects in Taipei, Moh and Associates Inc. (MAA Inc., 1981, 1980, 1979 and CHEN, 1980) has carried out a number of pressuremeter tests in the Taipei Silt Formation to determine the applicability of pressuremeter tests for geotechnical design use. MOH and OU (1979) has described the engineering characteristics of the Taipei Silt. A typical soil profile is shown in Figure 16. Many major structures in the Taipei area are founded in the so called Layer 4 of the deposit which is located at a depth of about 6 to 30 m below the ground surface. This layer of soil is a grey color silty clay with less than 10 per cent sand content. The soil contains numerous sea shells, micas and marine insects. The natural moisture content of this soil is near or slightly higher than its liquid limit and the sensitivity is in the range of 4 to 6. Majority of the pressuremeter tests were carried out in this soil stratum by using OYO's Model LLT 4120 single cell pressuremeter.

(a) Coefficient of earth pressure at rest - From the pressuremeter test results, the coefficient of earth pressure at rest, $K_o$, can be computed from the contact pressure $p_o$ and the effective overburden pressure. In calculating the effective overburden pressure, it is important to measure the actual piezomteric pressure since there is a considerable under pressure (i.e. below static pressure distribution) in the groundwater below the depth of about 20 m below the ground surface due to deep well pumping. It was found that the $K_o$ value above 20 m depth varied from 0.49 to 0.58 with an average of about 0.55. Below 20m depth, the $K_o$ value varied considerably due to difference in the under pressure at different localities with most of values varying between the range of 0.38 to 0.48.

(b) Relationship between pressuremeter parameters and SPT N values - Based on 30 sets of test results, CHEN (1980) reported linear correlations between the pressuremeter parameters $p_l$, $k_m$ and $E_m$ with N as follows :

$$p_l\ (\mathrm{kg/cm^2}) = 1.38\ N^{0.744}\ ;\ \text{correction factor} = 0.67$$

$$k_m\ (\mathrm{kg/cm^3}) = 2.01\ N^{0.922}\ ;\ \text{correction factor} = 0.64$$

$$E_m\ (\mathrm{kg/cm^2}) = 14.15\ N^{0.848}\ ;\ \text{correction factor} = 0.68$$

(c) Relationships between pressuremeter parameters with maximum deviator stress - By using laboratory unconsolidated undrained triaxial tests, the following relationships were obtained (CHEN, 1980) :

$$p_l = 7.37\ (\sigma_1 - \sigma_3)\max\ ;\ \text{correlation factor} = 0.89$$

$$k_m = 16.99(\sigma_1 - \sigma_3)^{1.477}\max\ ;\ \text{correlation factor} = 0.66$$

$$E_m = 90.2\ (\sigma_1 - \sigma_3)\max\ ;\ \text{correlation factor} = 0.76$$

The ratio of 7.37 between $p_l$ and $(\sigma_1 - \sigma_3)$max is somewhat higher than the values reported by other investigators on other types of soils.

Pressuremeter tests were carreid out at a reclaimed land site in Singapore (MOH AND ASSOCIATES (S), 1980). The subsoils at the site consists of a 1.5 m to 8.5 m thick layer of loose sand fill over a layer of very soft marine clay with thickness varying between 0.5 to over 15 m. Underlying the marine clay are layers of dense clayey sand, stiff silty clay and slightly cemented very dense silty sand of varying thickness. A total of 23 Pressuremeter Tests with Menard G A Type pressuremeter were carried out in the various subsoil layers. It is interesting to note that despite its variation in the soil type, unit weight and mositure content, good correlations were obtained bewteen $p_l$ and N as well as $E_m$ and N as shown in Figure 17.

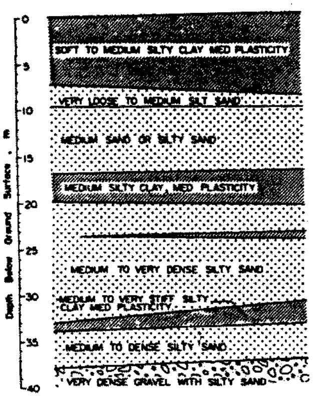


Fig. 16 Typical Subsoil Profile of Taipei Silt Formation

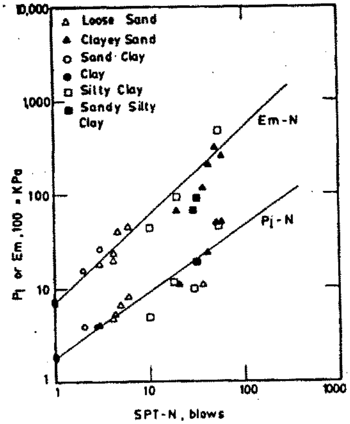


Fig. 17 Correlations of Pressuremeter Modulus, Limit Pressure and N-value for a Singapore Reclaimed Site

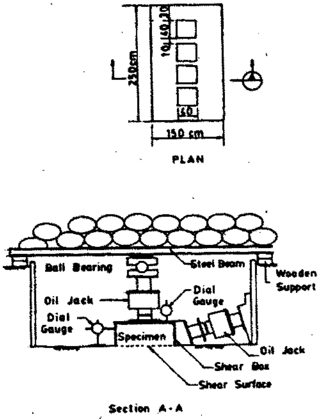


Fig. 18 Set-up for In Situ Direct Shear Test

## IN SITU DIRECT SHEAR TEST

The shear strength of soils are commonly determined in the laboratory by unconfined compression tests and triaxial compression tests. However, for residual soils and colluvium deposit, it is difficult to determine the shear strength of the deposits due to their heterogenity. Furthermore, the presence of gravels and rock fragments in these types of soils virtually make small diameter sampling, usually 75 to 100 mm, impossible. For rock formations with joints, fractures and stratifications, the stability of the rock is usually dictated by the in-fill material in the joints. It is very difficult to obtain good undisturbed sample of these materials for laboratory testing. For all these difficult soil deposits, in situ direct shear test has its special merit. BRENNER et al (1978) has developed a portable equipment for use in Thailand on sloping ground composed of residual soils. Their apparatus derived its normal and shear reactions from a light steel frame loaded with sand bags. Normal loads of up to 10 kN could be applied by means of a hydraulic jack. The shear force was applied by a hand driven CBR screw-jack and proving ring assembly. A strain rate of 0.4 per cent was adopted for their tests. BRAND et al (1983) reported that the development of a similar but improved in situ direct shear test apparatus for Hong Kong residual soils.

For design of cut and fill slopes in connection with development of hilly areas in Taiwan, MOH AND ASSOCIATES has constructed a direct shear apparatus which is portable and easy to assemble on site. As shown in Fig. 18, the assembly consists of a steel box 400 x 400 x 200 mm in size, two hydraulic jacks and a light steel frame. At the location where in situ sirect shear tests are to be carried out, a test pit of minimum size of 2.50 x 1.50 m is dug to the required depth to leave columns of soil which are then trimmed to the internal dimension of the shear box, i.e. 400 x 400 x 200 mm. Normally, four test specimens are prepared for each series of test. The shear box is actually only one-half of a normal direct shear box since in these in situ tests, the tests are carried out with the bottom of the test pit serving as the shearing surface. The sides of the pit are supported with wooden planks and the steel frame for vertical dead load is erected outside the pit. Vertical normal laod is applied by means of hydraulic jack against the sand bag dead weight of about 7 tons (70 kN). After consolidation and soaking, if any, are completed, the specimen is sheared by means of a hydraulic jack acting against the side of the pit. This equipment has been used on many different sites with different soil types.

Several series of in situ direct shear tests were carried out at the site of a large luxury residential development on the outskirt of Taipei city. First phase of the project covers an area of over 50 hectares of hilly land. The larger scale direct shear tests were carried out to determine the shear strength characteristics of colluvium deposits and highly weathered sandstone-shale rock formations. Figures 19 and 20 illustrate the strength envelopes obtained by in situ direct shear tests on colluvium and highly weathered sandstone-shale formation, respectively (MAA, Inc., 1981).

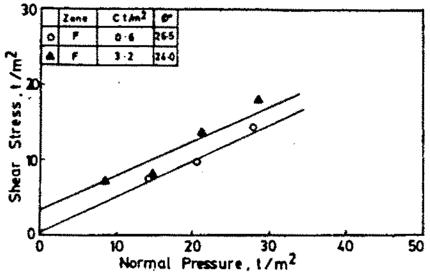


Fig. 19 Results of In Situ Direct Shear Tests on Colluvium Deposit.

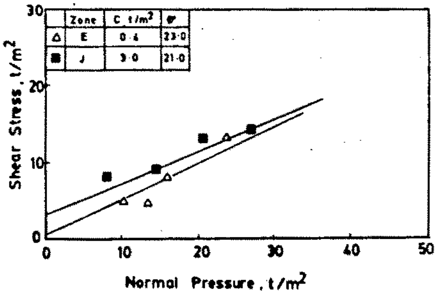


Fig. 20 Results of In Situ Direct Shear Tests on Highly Weathered Sandstone Shale Formation

The test results showed fairly consistent trend with low scatter for tests run at the same site. however, considerable variations were observed when tests were carried out at different locations which reflect the heterogeneity of these types of deposit. The results shown in Fig. 20 indicate that for highly weathered sandstone-shale formation below watertable, the snear strength can be lower than that of a soil deposit. Figure 21 shows that water saturation does not appear to have significant effect on the strength characteristics of a shale formation (MAA Inc., 1983).

Direct shear tests were carried out to determine the strength characteristics of a clayey in-fill material as shown in Fig.22 (UGI, 1984). The in situ shear strength values of the clay in-fill are higher than that determined by laboratory triaxial compression tests. This largely reflects sampling problem and the inevitable selectivity that must be employed in preparing satisfactory specimens for laboratory testing.

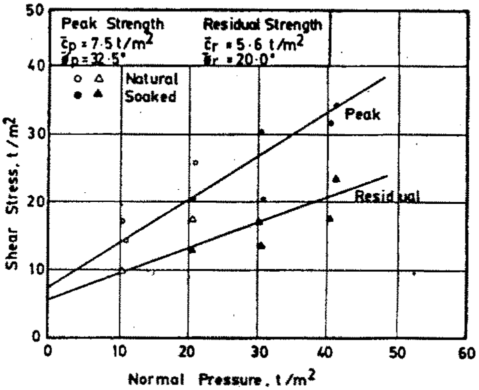


Fig. 21 Comparison of Peak and Residual Strength of a Shale Formation

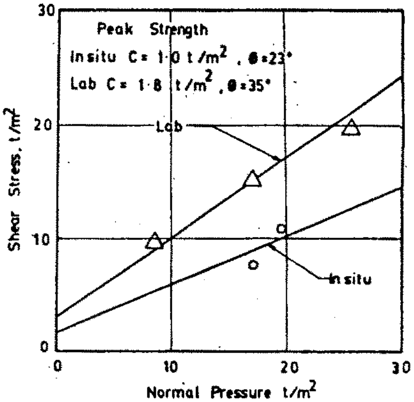


Fig. 22 Comparison of Laboratory and In Situ Strength Results of Rock In Fill Material

ACKNOWLEDGEMENTS

The writer wishes to acknowledge assistances given by the following organizations and individuals who have either helped in collecting or supplying information contained in this paper : Dr E. W. Brand of the Geotechnical Control Office in Hong Kong; Mr Benjamin Chi of Ret-Ser Engineering Agency and Professor T. H. Ong of the National Taiwan University of the Republic of China : Dr Chai Muktabhant and Professor A. S. Balasubramaniam of the Asian Institute of Technology in Thailand ; Dr Ting Wen Hui of Malaysia; and Dr C. D. Ou, Dr S. M. Woo, Mr K. Yu and Dr T. C. Kao of the Moh and Associates Group.

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