SUBSURFACE INVESTIGATION AND INTERPRETATION OF TEST RESULTS FOR FOUNDATION DESIGN IN SOFT CLAY

by : Ir. Dr. Gue See Sew & Ir. Tan Yean Chin
Gue & Partners Sdn Bhd

1.0 INTRODUCTION

There are many different types of ground improvement techniques that can be used for construction on soft clay. The most common problem engineers face is whether ground improvement is needed or not before proceeding with the design. In order to decide, the engineers have to consider the following factors :

- Geological and geotechnical information of the site
- Types of structures to be constructed and the movements tolerance of the proposed development
- Time or duration allocated for the construction
- Construction cost and future maintenance cost.

During planning of the subsurface investigation, the Engineer shall always remember that majority of the unforeseen costs associated with construction on soft clay are geotechnical in nature. Additional costs are often attributed to inadequate planning of subsurface investigation and improper interpretation of the factual information and results of the field and laboratory tests. In view of the importance of subsurface investigation, this lecture presents the planning and selection of subsurface investigation works in soft clay, including field tests, sampling and laboratory tests, with emphasis on the practical aspect of the work. The interpretation of the test results either from field or laboratory testing for foundation design in soft clay is also discussed.

2.0 GEOLOGY OF SOFT ALLUVIAL CLAY IN PENINSULAR MALAYSIA

The behaviour of soft alluvial soils is influenced by the source of the parent material, depositional processes, erosion, redeposition, consolidation and fluctuations in groundwater levels. Generally alluvial deposits (materials transported and deposited by water action) consist of finest clays to very coarse gravel and boulders. Alluvial soils usually show pronounced stratification and sometime organic matter, seashell and decayed wood are present in the alluvial clay.

Raj & Singh (1990) presented an overview of the alluvial deposits of Peninsular Malaysia. Figure 1 shows the unconsolidated quaternary sediments in Peninsular Malaysia (After Stauffer, 1973).

3.0 PLANNING OF SUBSURFACE INVESTIGATION

In most projects, subsurface investigation (S.I.) is carried out in two stages namely Preliminary S.I. and Detailed S.I..

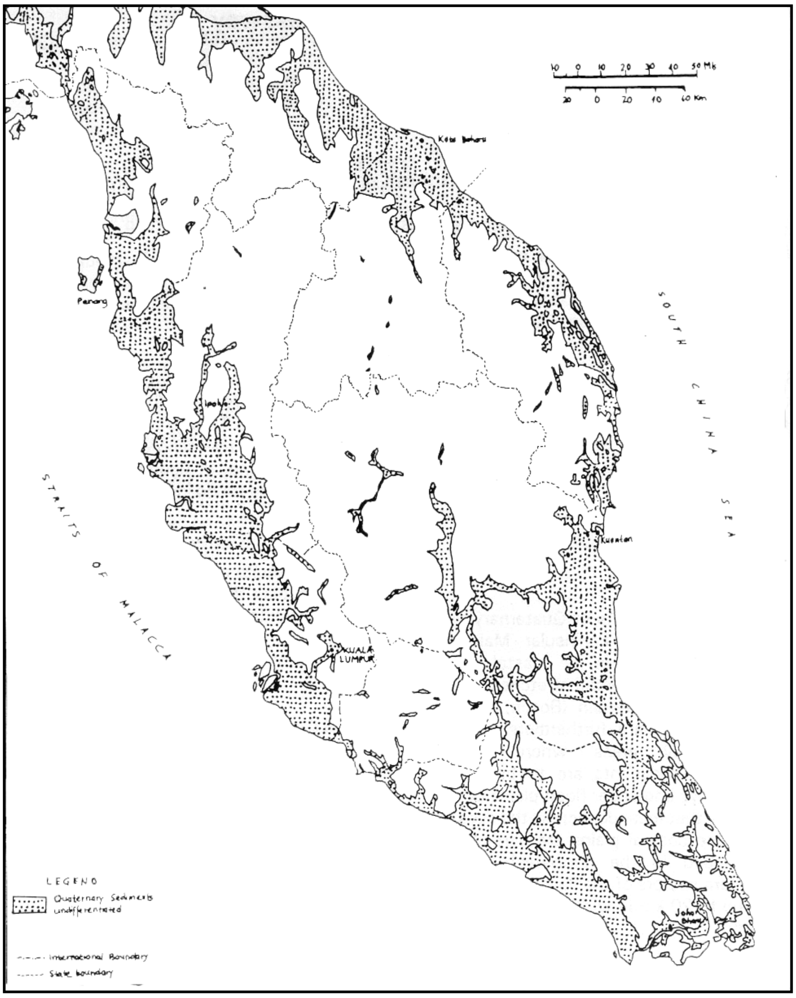


Figure 1 : Quaternary Sediments in Peninsular Malaysia (after Stauffer, 1973)

<u>Stage 1 : Preliminary S.I.</u>
Preliminary S.I. aims to achieve the following objectives :

- To obtain general subsoil profile for estimation of earthwork
- Preliminary or confirmation of layout and formation level
- Preliminary soil parameters and water level/table
- For conceptual designs and preliminary cost and time estimates

<u>Stage 2 : Detailed S.I.</u>
Detailed S.I. is usually carry out after optimum layout has been selected and confirmed. Detailed S.I. aims to achieve the following objectives :

- Plan for critical areas of concern
- Refine subsoil profile
- Obtain necessary soil parameters for detailed design of foundations
- At areas with difficult ground conditions (e.g. very soft soils, etc.)
- Major fill or cut areas that are more critical
- Locations with structures (e.g. retaining walls, areas with large loadings, etc.)

The planning of the subsurface investigation for soft clay can be divided into four major sections as follows :

- Desk Study
- Site Reconnaissance
- Extent of Subsurface Investigation
- Selection of Types of Field Tests and Sampling Methods.

3.1 DESK STUDY

Desk study including reviewing of the following information :

(a) Geological Maps and Memoirs
Reviewing geological maps and memoirs together with an understanding of the associated depositional process can enable a preliminary assessment of ground conditions to be made.

(b) Topographic Map
Use topographic map to examine the terrain, access and site conditions. The topographic map shall be confirmed through site reconnaissance.

(c) Aerial Photographs
Aerial photographs give an indication of geomorphology features, land use, problem areas and layout arrangement especially for highways and hill-site development.

(d) Site Histories and Details of Adjacent Development
The knowledge of the site histories like land use before the current development, tunnels, underground services are very important information to have before planning the field testing. Information of adjacent development like types of structures and foundation system is also very useful for design and to prevent affecting the serviceability of adjacent structures. If the subsoil information of adjacent site is available, it will help the Engineer to optimise the S.I. required for the project.

(d) Requirements of the Proposed Structures or Foundations
In order to plan proper and cost effective S.I., the Engineer shall have sufficient information on the requirements of the completed structures .

3.2 SITE RECONNAISSANCE

The purpose of the site reconnaissance is to confirm and obtain additional information from the site. This includes examining adjacent and nearby development for tell-tale signs of problems and as part of the pre-dilapidation survey. Site reconnaissance allows the Engineer to compare the surface features and topography of the site with data and information obtained from the desk study. The checking of the presence of exposed services markings and cut and fill areas is also essential. It is also very important to locate and study the outcrops to identify previous slips or collapse that will act as an indicator of stability of the site.

The study on the vegetation would give tell tale signs of localised very soft areas where additional subsurface investigation should be carried out. Very often failures occur in localised soft areas as reported in Gue & Chen (2000).

3.3 EXTENT OF SUBSURFACE INVESTIGATION

The extent of subsurface investigation depends on :
- Available information
- Geological formation and features
- Variability of subsoil and groundwater
- Proposed structures and platforms
- Adjacent properties

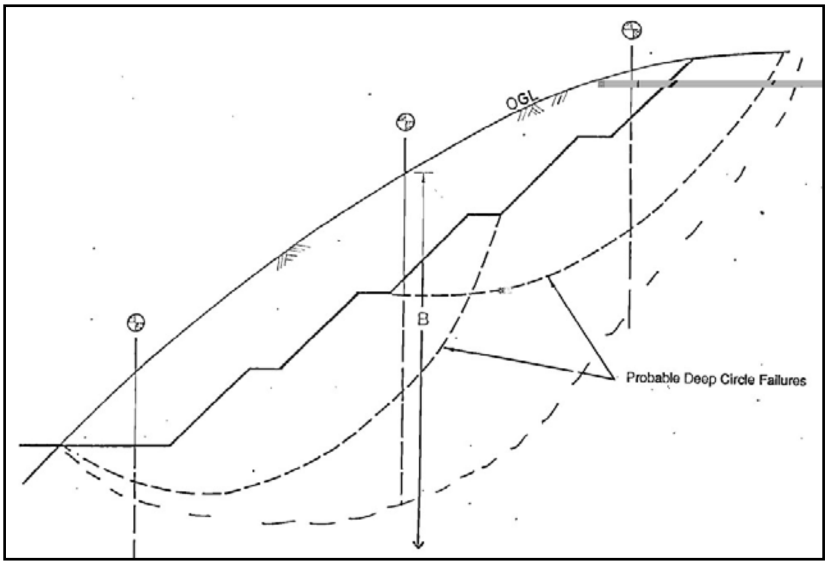


Figure 2 : Depth of Field Tests for Stability Analysis

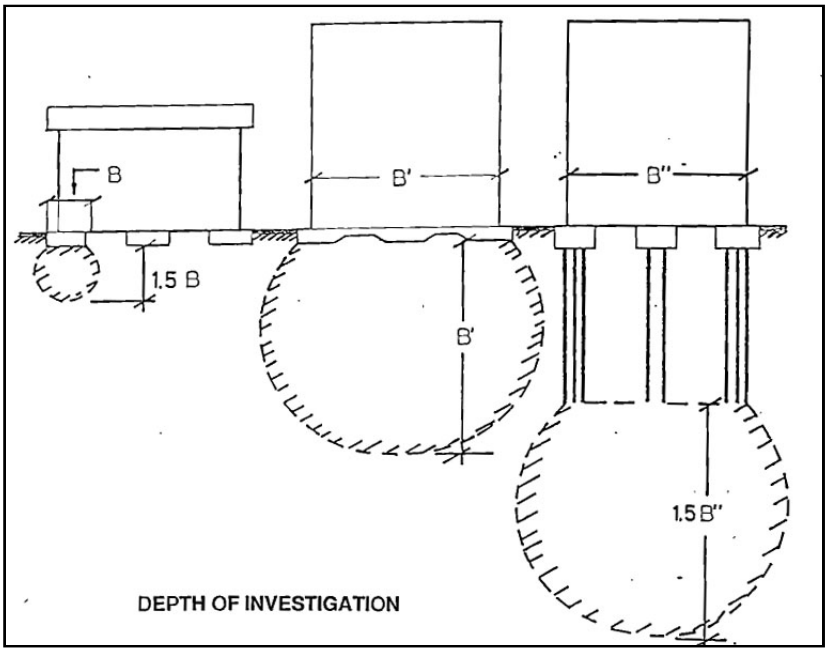


Figure 3 : Depth of Field Tests for Foundation Design

Preliminary S.I.

(a) **Number / Spacing** (Minimum Requirements)
- boring and probing in fill area of a formation
- boring and probing in a line for one of a typical cluster or cross-section of similar topography (large area a few more lines are needed)

(b) **Depth**
- In fill area, up to a depth with SPT'N' $\approx$ 50
- In cut area, up to a depth exceeding potential slip surface or when hard material is encountered.
- For deep foundation in soft clay, up to a depth with SPT'N' $\geq$ 50 for at least 5 to 7 times consecutively.
- boring and probing in fill area of a formation

(c) **Structures**
- Up to depth of soils where the pressure induced by structure has little or no influence.

(d) **Geophysical Survey :** For large area and to determine bedrock profile and characteristics (beneath the soft clay)

Figures 2 and 3 show some guideline for the extent of S.I. work.

Detailed S.I.

(a) **Spacing**

- No hard and fast rules but generally 10m to 30m for structures. The spacing can be increased for alluvial subsoil with more consistent layering (as interpreted from preliminary S.I.) or geophysical survey is used to interpolate or identify problem areas.
- Intensified ground investigation for problem areas and structures with heavy loading for safe and cost effective designs.
- At bridges generally one borehole at every pier or abutment.

3.4 SELECTION OF TYPES OF FIELD TESTS AND SAMPLING METHODS

The selection of types of field tests and sampling methods should be based on the information gathered from the desk study and site reconnaissance. There are many commonly used field testing methods for subsurface investigation in soft clay :

- Light Dynamic Penetrometer (JKR or Mackintosh Probes)
- Boreholes with Standard Penetration Tests (SPT), collection of disturbed and undisturbed soil samples.
- Field Vane Shear Tests (in borehole or penetration type)
- Piezocone (CPTU)

3.4.1 Light Dynamic Penetrometer (JKR or Mackintosh Probe)

JKR or Mackintosh probes are usually used in preliminary S.I. to acquire the undrained shear strength (indirectly through correlations) and consistency of the subsoil layering for very soft to soft soils. It assists in interpolation between boreholes or piezocones. Figure 4 shows the probe details. This method is also effective in identifying localised soft or weak materials or slip plane. However the major limitation of the method is shallow depth.

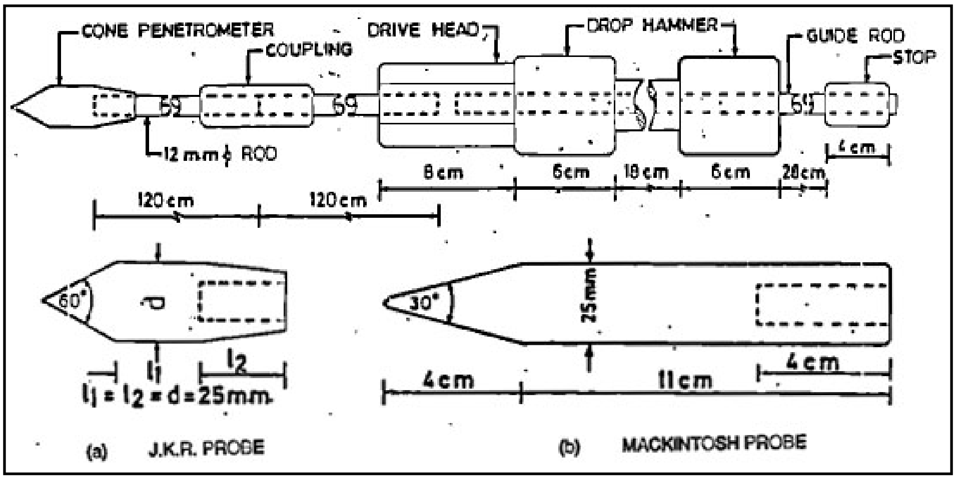


Figure 4 : Mackintosh and JKR Probes

Human errors are also prone in this method such as wrong counting, non-consistent drop height or exerting force to the drop hammer giving misleading results. When using light dynamic penetrometer, some of precautionary measures to prevent errors in testing :

- drop of hammer should be a free fall and consistent drop height
- components and apparatus properly washed and oiled

3.4.2 Boreholes with Standard Penetration Tests (SPT), collection of disturbed and undisturbed soil samples.

Boreholes is sometime called deep boring. The details of boring, sampling and testing are described in BS5930: 1981. Rotary open hole drilling by circulating fluid (water, bentonite or air foam) is the most common method. The other commonly used method is wash boring which utilises the percussive action of a chisel bit to break up materials and flush to the surface by water pumping down the hollow drill rods. Percussion method is not normally recommended due to large disturbance caused by the technique of advancing. Borehole usually includes boring through soil, coring through rock, sampling, in-situ testing and water-table observations. The depth usually do not exceed 100m.

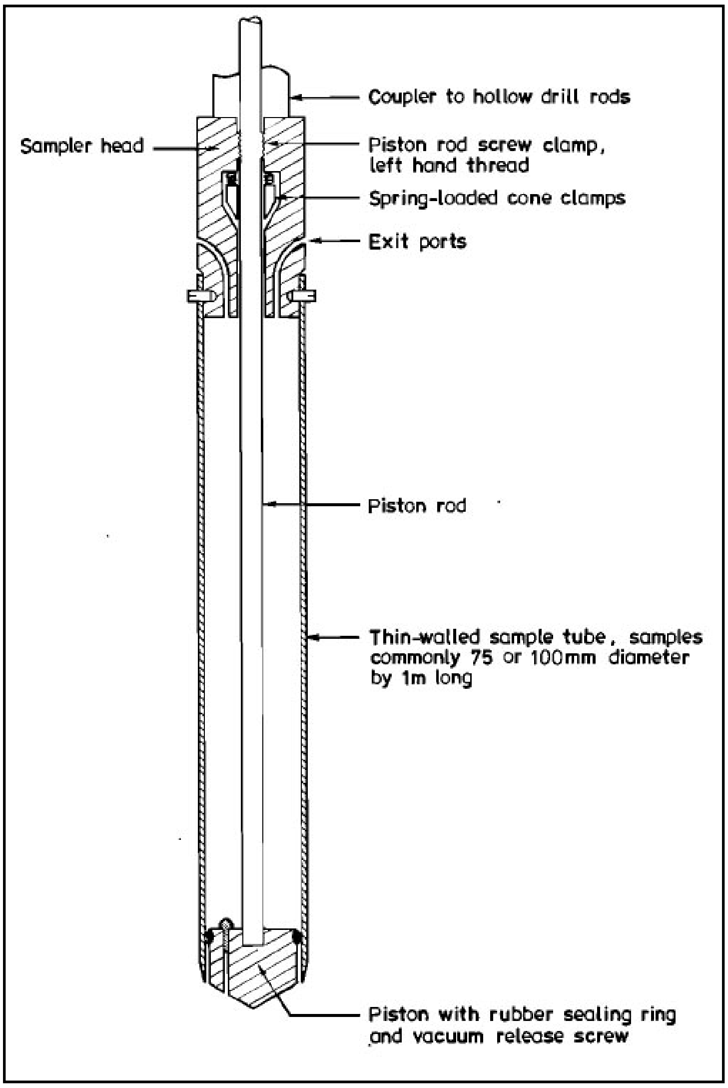


Figure 5 : Piston Sampler

Standard Penetration Test (SPT) is the most commonly used in-situ test in Malaysia. As per BS1377, the hammer weight is 65kg, with drop height of 760mm. Sampler is driven a total of 450mm into soils and the number of blows for the last 300mm of penetration is the SPT'N' value. The disturbed soil samples can be collected from the split spoon sampler.

SPT is generally carried out at 1.5m depth interval or larger interval depending on the undisturbed soil sampling schedule. At greater depth, the interval can be increased. The SPT test is simple and rugged however certain care are required :

- Dented driving shoe should not be used.
- Depth of test is important and no test shall be carried out in the casing.
- Base of borehole must be properly cleaned.

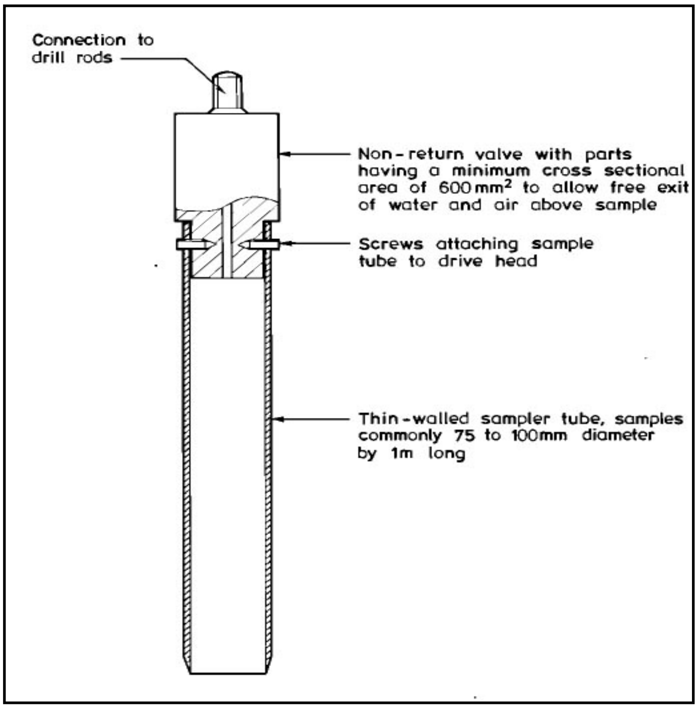


Figure 6 : Thin Wall Sampler

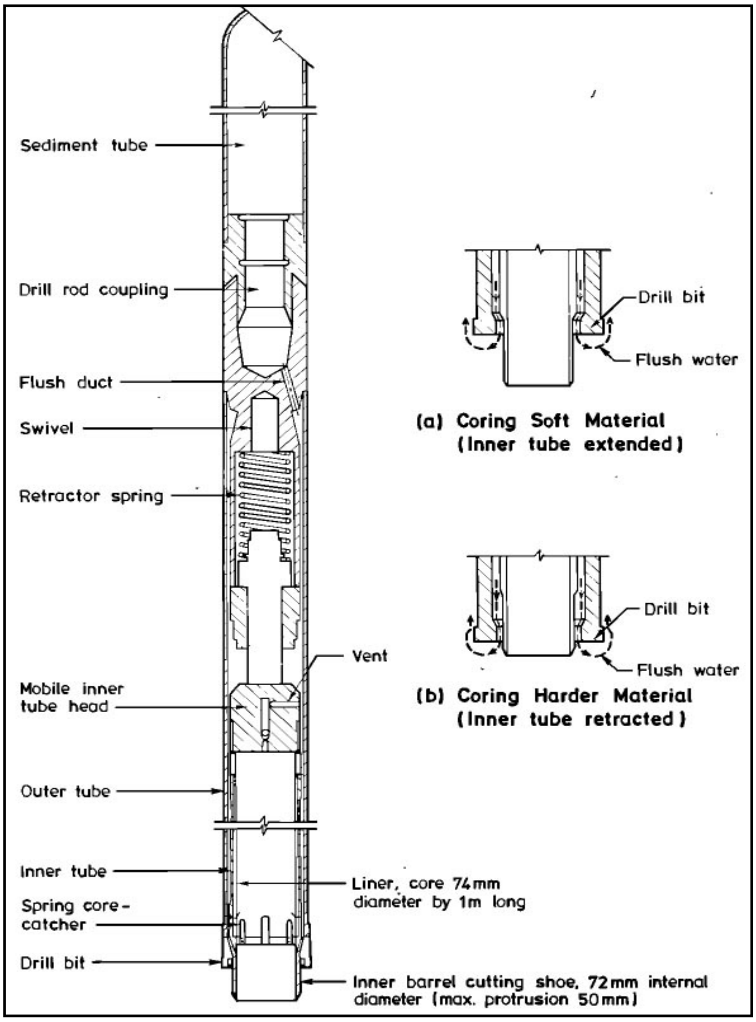


Figure 7 : Mazier Sampler

- Use counter to prevent counting error.
- Mark the penetration depth clearly.
- Always keep borehole water level as close to the natural ground water as possible (if the approximate ground water level is known) or else keep the borehole full of water.
- Prevent water level in the borehole dropping too fast and below natural ground water level during changing of assembly for SPT in silty and sandy soils to prevent boiling in the soils.
- Require close supervision.

Soil samples collected from the borehole are as follows :

- Wash Samples : from soil washed out from the borehole for soil strata description.
- Disturbed Soil Samples : from split spoon samplers after SPT.
- Undisturbed Soil Samples : using piston sampler, thin wall sampler, continuous sampler, mazier sampler, etc.

Piston sampler shown in Figure 5 is used for very soft to soft cohesive soil which is easily disturbed. Usually this layer is with SPT'N' < 2 or hammer weight. For cohesive soils from soft to firm consistency (SPT'N'<10) and free

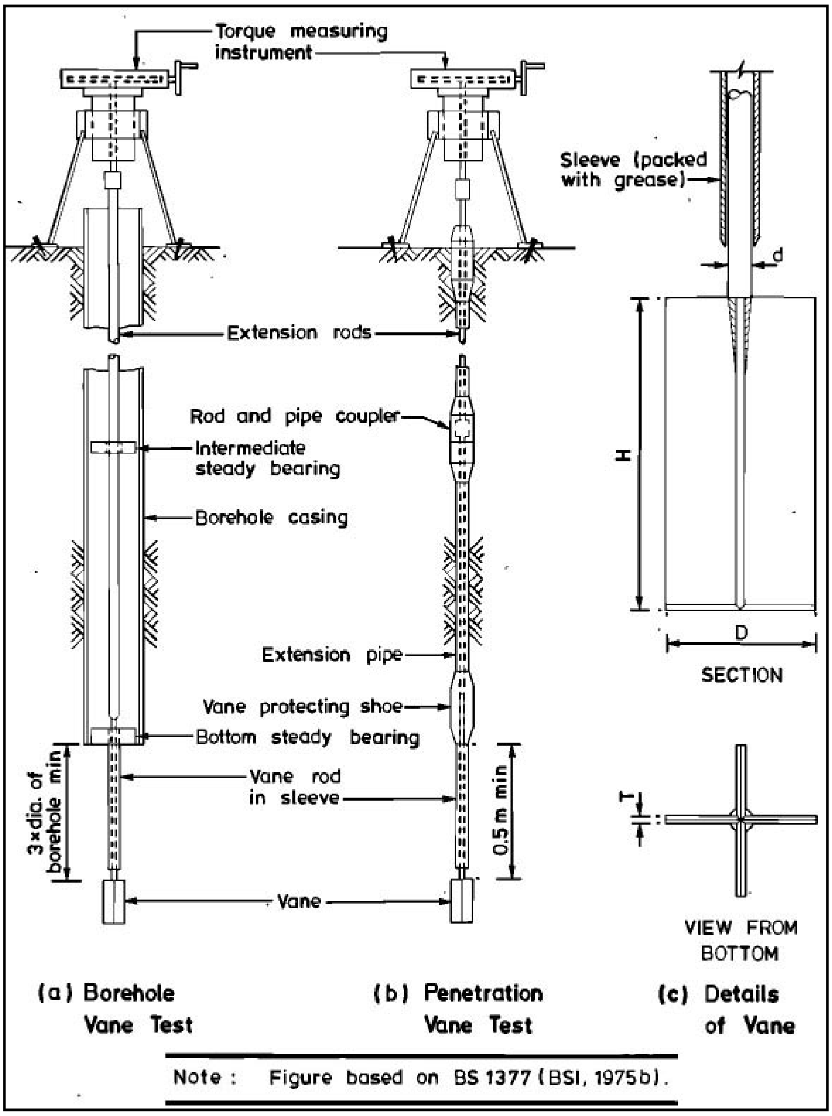


Figure 8 : Field Vane Shear Devices

from large particles (e.g. marine deposits), thin wall sampler as shown in Figure 6 can be used. The area ratio of this wall sampler is about 10%. The piston and thin wall samplers are commonly 75mm or 100mm diameter by 1m long. Continuous sampler is usually used for identifying sand lenses, description and classification tests in soft marine deposits.

When the subsoil is stiff, mazier sampler shall be used to obtain the undisturbed soil samples. Mazier Sampler (Triple-tube core-barrels) as shown in Figure 7 contains detachable liners within the inner barrel and is ideal for collecting undisturbed soil samples for triaxial tests as the diameter of core sampler is about 75mm.

When sampling soil from the boreholes, the following checklist shall be followed :

- Distorted or blunt cutting edge or dirty tubes should not be used.
- Check sizes of components and condition
- Sampler must be properly cleaned and greased.
- The soil samples collected should be properly sealed and labelled to prevent loss of water when preserving moisture content is required.
- Undisturbed soil samples should be sealed with a layer of grease, follow by non-shrink wax and tape to prevent loss of moisture (excess grease by the side of tube for wax placement must be removed to ensure good contact between wax and the inner side of sampler)
- Properly stored and packed for transport to prevent disturbance during transportation.

For rock coring, rotary core drilling is commonly used to advance the borehole and provide core samples for examination and testing. Some of the definitions related to rock core are :

Core recovery (%) = (Length of Recovered Core / Length of Run) X 100%

RQD (%) = ([Sum of recovered core in pieces > 100mm] / Length of Run) X 100%
Where RQD = Rock Quality Designation.

3.4.3 Field Vane Shear Tests (in borehole or penetration type)

Field Vane Shear Test is suitable to test very soft to firm clay. However the results will be misleading if tested in peats, sands, gravels, or in clays containing laminations of silt, sand, gravels or roots. The field vane shear test is used only to obtained 'undisturbed' peak undrained shear strength, and remoulded undrained shear strength thus sensitivity of the soil. Figure 8 shows the equipment details.

Following are some reminders when using field vane shear tests :

- The vane must be rotated soon (within 5 minutes) after insertion into the depth to be tested as delays of one (1) hour to one day may lead to overestimation of strength by 10% to 20% respectively.
- Standard rate of rotation is 6 degrees per minute.
- Correct calibration chart for the torque and different vane size for shall be used.

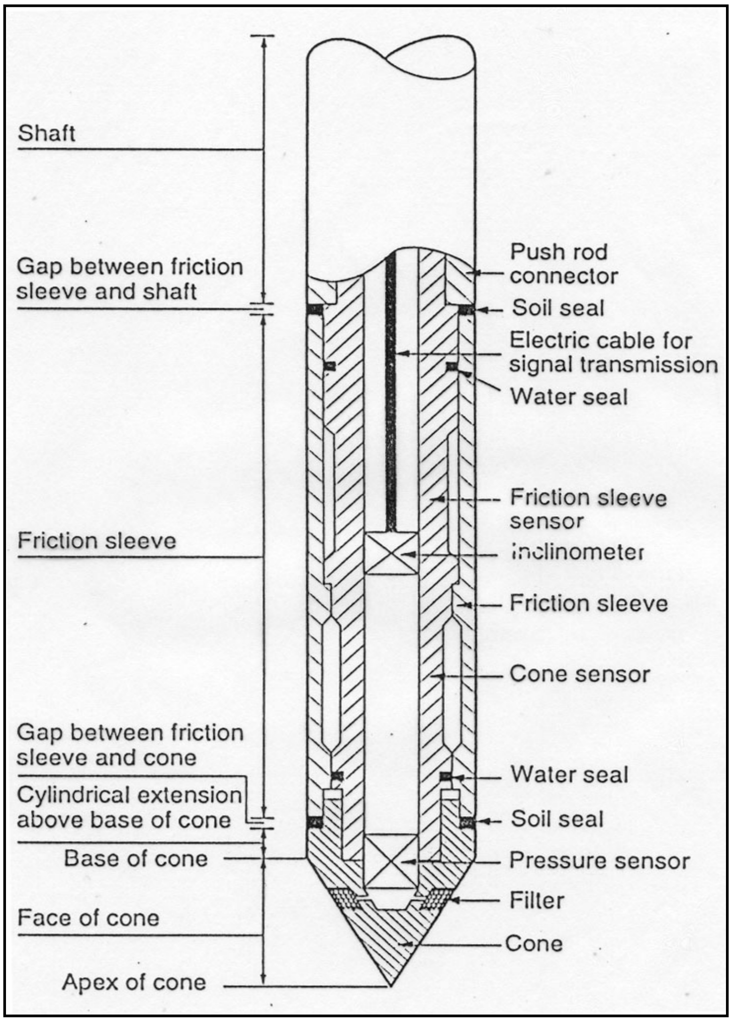


Figure 9 : Detailed Terminology of Piezocone

3.4.4 Piezocone (CPTU)

Figure 9 shows the detailed terminology and design features for a piezocone. The cone is the cone-shaped end piece of the penetrometer tip on which the end bearing is developed. The cone normally has a diameter of 35.7mm, area of 10cm^2 and cone angle of 60^o. The friction sleeve is the section of the penetrometer tip upon which the local side friction resistance to be measured is developed and has an area of 150cm^2, diameter of 35.7mm and is slightly larger than the cone. In order to measure the pore water pressure, porous filter (e.g. porous plastic, ceramic or sintered stainless

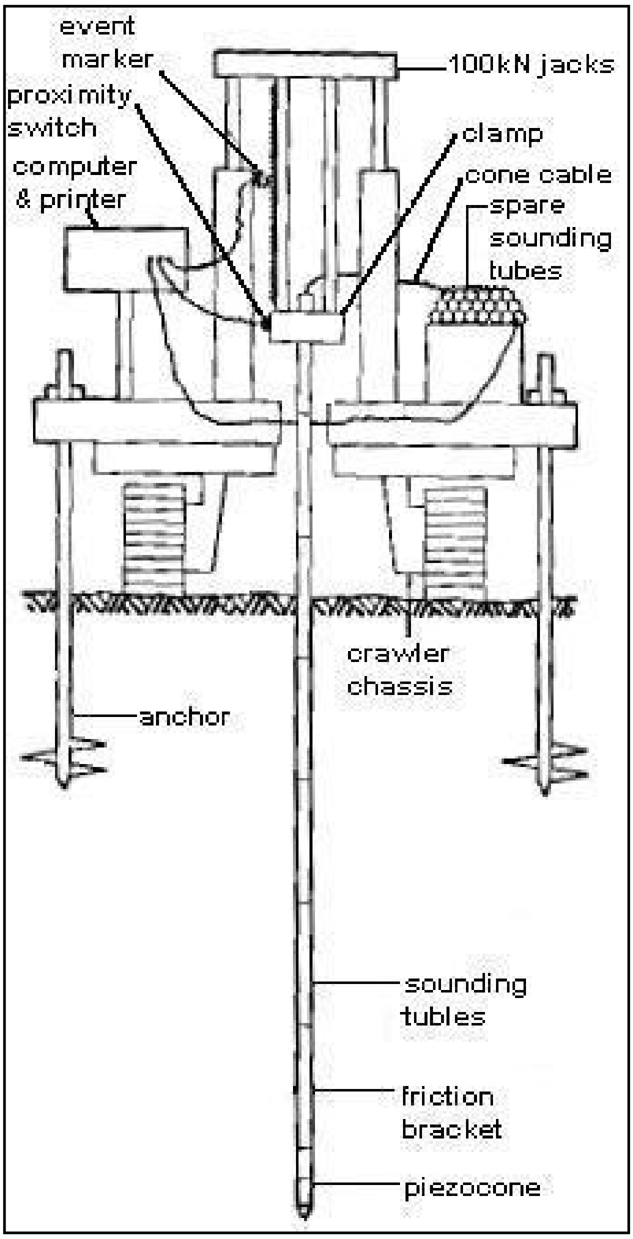


Figure 10 : Piezocone Rig

steel) which allow rapid movement of extremely small volumes of water to activate the pressure sensor.

The porous element is usually placed immediately behind the cone neck. It is very important that the pore pressure measurement system should always be fully saturated so that reliable pore pressure measurement can be obtained. All the data are captured electronically on computer therefore reduces human error. Figure 10 shows the rig for piezocone testing.

Piezocone (CPTU) has three main applications :

- To determine subsoil stratigraphy and identify materials present.
- To estimate geotechnical parameters.
- To provide results for direct geotechnical design.

The advantages of piezocone are as follows :

- Rapid and continuous (rather than intermittent) measures of soil profile and strength
- Faster than other S.I. tests
- Very suitable for soft soil (especially soft clay)
- Allow dissipation of excess pore water pressure (consolidation)
- Has been widely used and many correlations available

3.5 DETERMINATION OF GROUNDWATER

The knowledge of groundwater level, groundwater pressure and potential flooding is important in soft clay as they will affect the effective stress of the subsoil and also the design. Water level observation in completed boreholes and existing wells (if any) should be taken daily during the ground investigation, particularly in the morning. Rain in the preceding night must be recorded and the borehole protected against surface in flow of water that could cause misleading results to be obtained. However, in order to obtain a representative ground water level, measurement and monitoring of longer period of time is required and should include seasonal variation and tidal changes (if applicable). The main disadvantages of measuring groundwater level from the boreholes are slow response time and collapse of hole if not cased. Therefore, the use of proper piezometer is recommended.

Standpipe and open-hydraulic piezometers installed in borehole are shown in Figure 11. During installation, porous elements must be fully saturated and filled with deaired water or fluid such as glycerine. Response test is required by carrying out falling head test. If two piezometers are placed in a single borehole, proper seal is important to prevent confusing or misleading results. In order to measure the quick response of pore water pressure changes, vibrating wire piezometer can be used.

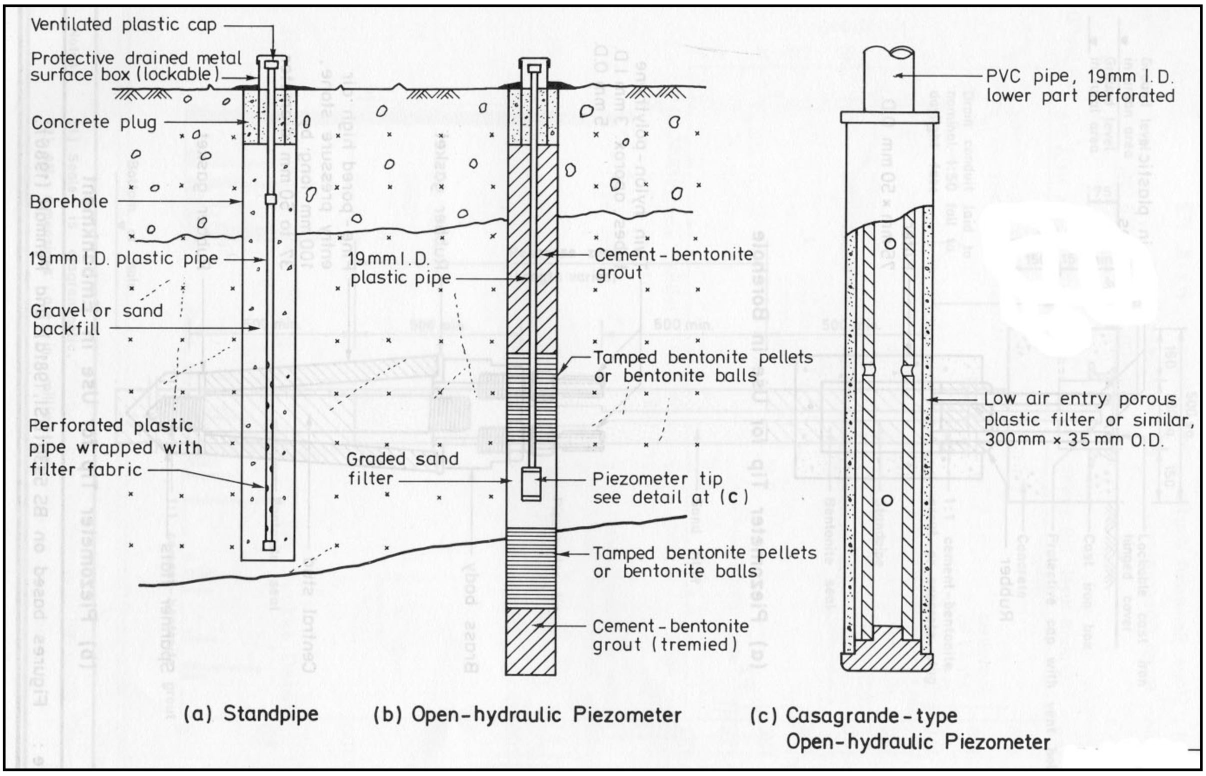


Figure 11 : Typical Standpipe and Open- hydraulic Piezometers (after Geoguide 2, 1987)

4.0 PLANNING OF LABORATORY TESTING

The types of laboratory test commonly used in Malaysia to determine soil classification, chemical and mechanical properties are summarised in Table 1. The total stress strength parameter like undrained shear strength, s_u is required for short term undrained stability analysis of embankment on cohesive soils and for foundation design (e.g. footing, pile, retaining wall) in cohesive soils. The effective strength parameters like c' and ϕ' are for long term stability analysis of foundation, embankment and slopes, particularly cut slopes. Consolidation parameters allow engineer to evaluate deformation of the subsoil when there is changes of stress in the subsoil. Main purpose of chemical tests on the subsoil except organic content are to detect any chemicals that are detrimental to concrete and other materials used and buried inside the ground.

<table>
<tr><th><u>SOIL CLASSIFICATION TEST</u></th><th><u>TEST FOR MECHANICAL PROPERTIES</u></th></tr>
<tr><td>1. Particles Size Distribution : - Sieve Analysis (for content of sand and gravels) and Hydrometer Tests (for content of silt and clay)</td><td>1. One Dimensional Consolidation Test (Oedometer Test) :- to obtain compressibility and consolidation parameters for settlement analysis.</td></tr>
<tr><td>2. Atterberg Limits :- Liquid Limit, Plastic Limit & Plasticity Index (to be used in Plasticity Chart for soil classification)</td><td rowspan="8">2. Shear Strength Test :
(a) For Total Stress :- Laboratory Vane, Unconfined Compression Test (UCT), Unconsolidated Undrained Triaxial Test (UU), Shear Box Test.
(b) For Effective Stess :- Isotropic Consolidated Undrained Triaxial Test (CIU), Isotropic Consolidated Drained Triaxial Test (CID). (Note : Side Drains <u>shall not</u> be used on samples to accelerate consolidation to prevent errors) (Gue (1984) and Tscheboutarioff (1951))</td></tr>
<tr><td>3. Moisture Content</td></tr>
<tr><td>4. Unit Weight</td></tr>
<tr><td>5. Specific Gravity</td></tr>
<tr><td><u>CHEMICAL TEST</u></td></tr>
<tr><td>1. pH Test</td></tr>
<tr><td>2. Chloride Content Test</td></tr>
<tr><td>3. Sulphate Content Test</td></tr>
<tr><td>4. Organic Content Test</td><td>3. Compaction Test</td></tr>
</table>

Table 1: Laboratory Testing

5.0 INTERPRETATION OF FIELD AND LABORATORY TEST RESULTS

The interpretation of field and laboratory test results is usually a neglected topic and only briefly covered in universities. It is very dangerous for engineer to use field test results directly without interpretation, and understanding of the usage and limitation of each test. The selection of design parameters and choice of values depend on knowledge and experience of the engineer. The objectives are to illustrate the importance of correct interpretation and show methods of compiling results and recognising of errors. The following section will cover the brief interpretation of commonly used field and laboratory tests in Malaysia and they are :

(A) Field Tests :

- Light Dynamic Penetrometer (JKR or Mackintosh Probes)
- Standard Penetration Test (SPT)
- Field Vane Shear Test
- Piezocone (CPTU)

(B) Laboratory Tests :

- Unconfined Compression Test
- Triaxial Test (UU, CIU and CID with pore pressure measurement)
- Consolidation Test

- Compaction Test

It is very important to interpret the results and compile the results in order that errors can be recognised.

5.1 INTERPRETATION OF FIELD TESTS

5.1.1 Light Dynamic Penetrometer (JKR or Mackintosh Probe)

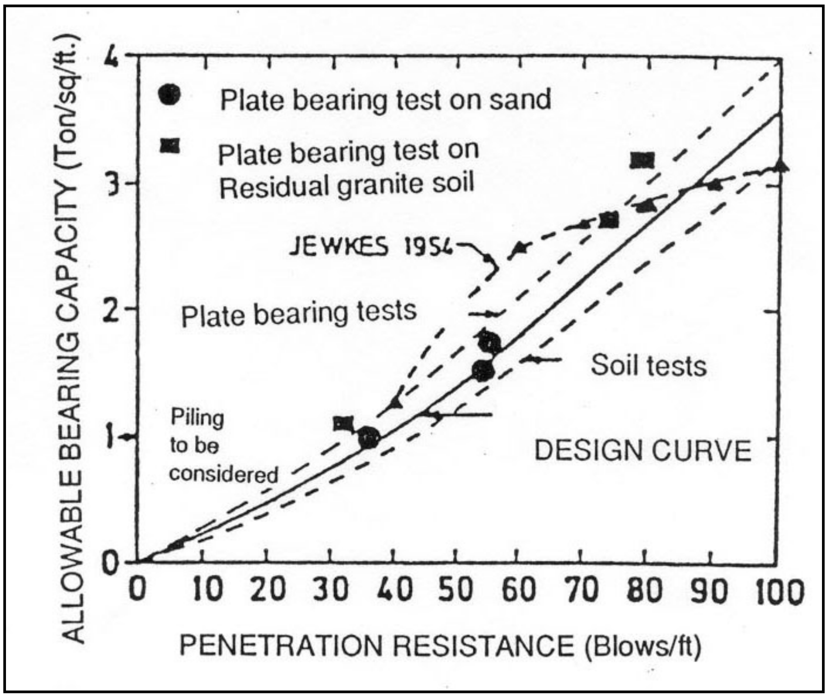


Figure 12 : Correlation of JKR Probe Resistance to Allowable Bearing Capacity (After Ooi and Ting, 1975)

JKR or Mackintosh probes are used for :

- Detection of weak or shear plane at shallow depth
- Determination of shallow bedrock profile
- For design of shallow foundation on natural ground with no recent fill and for structure of low risk. If in doubt use borehole instead.

Figure 12 shows the allowable bearing capacity versus JKR probe resistance. The ratio of JKR probe to SPT'N values are about 8.8 based on energy per unit area.

For shallow depth (less than 4m), the ratio of JKR probe to undrained shear strength in kPa is about 1. For larger depth, the ratio reduces significantly and often unreliable.

5.1.2 Standard Penetration Test (SPT)

Standard Penetration Test (SPT) is the most popular field test in Malaysia. The common errors of SPT are shown in Table 2. In cohesive soils, SPT'N' values are usually used to correlate with undrained shear strength, s_u and some of the correlations commonly used in Malaysia are as follows:

For SPT'N' > 5, (where N = SPT'N')

s_u = 4N to 6N (kPa) (Stroud & Butler, 1975)

For SPT'N' < 5

s_u = 5 + 7.5N (kPa) (Japanese Road Association, 1980)

The correlations above should be used with care and correlations with s_u obtained from field vane shear can be performed to verify the correlation used for each site.

ERRORS	CONCEQUENCE
Inadequate cleaning of borehole	(X) N, sludge trapped in sampler
Casing driven bottom of the borehole	(↑) N in sand & (↓) N in clay
Damage tip of sampling spoons	(↑) N
Loose joints on connecting rods	(↑) N
Not using guide rod	(↑) N, eccentric blows
Water level in borehole below ground water level	(↓) N especially sand at bottom of borehole, piping effect
Note : Where N = SPT'N' values, (↓) = Giving misleading lower value, (↑) = Giving misleading higher value, (X) = Wrong Results	

Table 2 : Some Common Errors of SPT

5.1.3 Field Vane Shear Test

The field vane shear tests are widely used to obtain the representative s_u profile of cohesive soils. The sensitivity, S_t of the material can also be obtained. The most common errors that occurred are wrong computation of spring factor and if the clay contains organic materials (e.g. sea shells, decayed woods, peat, etc).

5.1.4 Piezocone

Other than obtaining the continuous subsoil profile, commonly used soil parameters can be obtained from piezocone testing using correlations are as follows :

- Undrained shear strength, s_u
- Horizontal coefficient of consolidation, c_h through dissipation tests.
- Relative density (Dr) for granular soils
- Effective Angle of Friction, ϕ'
- Secant Young's Modulus, Es'
- Maximum Shear Modulus, G_{max}

Soil classification can be carried out using the Robertson (1990) chart shown in Figure 13. The undrained shear strength of cohesive soils can be estimated from piezocone data with reasonably accuracy.

$$s_u = \frac{q_c - \sigma_{vo}}{N_k} = \frac{q_T - \sigma_{vo}}{N_{kT}}$$

where : σ_{vo} = total overburden pressure
q_c = cone resistance
q_T = corrected cone resistance
N_k or N_{kT} = cone factor

Generally the cone factor, N_k is 14 ± 4 for Malaysian Clay (Gue, 1998). Robertson and Campanella (1988) recommended using $N_k = 15$ for preliminary assessment of s_u. Since N_k is sensitivity dependent, N_k should be reduced to around 10 when dealing with sensitive clay ($8<S_t<16$). In practice, the N_k or N_{kT} is determined empirically by correlation of cone resistance to undrained shear strength measured by field vane shear tests or laboratory strength tests.

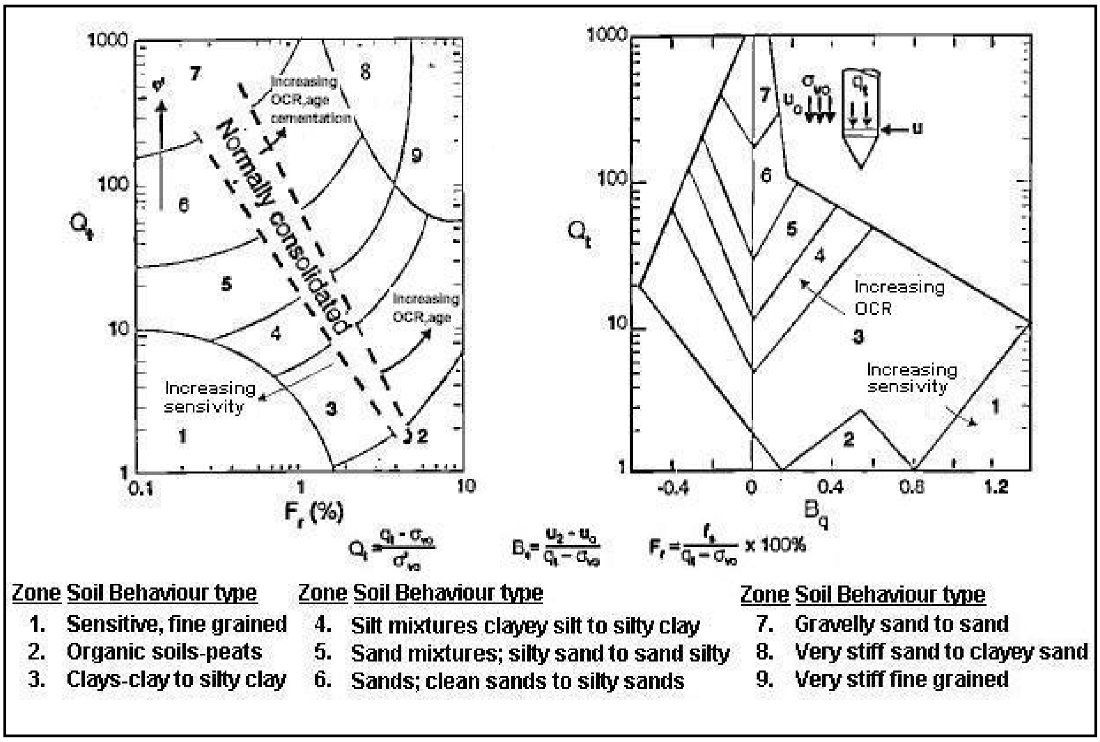


Figure 13 : Soil Classification Chart (after Robertson, 1990)

Horizontal coefficient of consolidation, c_h which is an important parameters for vertical drain design can be assessed from the dissipation of pore pressure with time after a stop in penetration during testing. Houlsby and Teh (1988) propose an interpretation which utilised a modified dimensionless time factor, T^* as given in Table 3, and defined as follows :

$$T^* = \frac{c_h \cdot t}{r^2 \sqrt{I_r}}$$

where c_h = horizontal coefficient of consolidation
r = radius of cone, typically 17.8mm
I_r = rigidity index, G/s_u
G = shear modulus
s_u = undrained shear strength

Figure 14 shows a simplified diagram that can be used to estimate c_h using the Houlsby and Teh (1988) solution. The normalized excess pore pressure, U, at time t, is expressed as :

$$U = \frac{u_t - u_o}{u_i - u_o}$$

where U = normalized excess pore pressure
u_t = the pore pressure at time t
u_i = initial pore pressure at t=0
u_o = insitu pore pressure before penetration

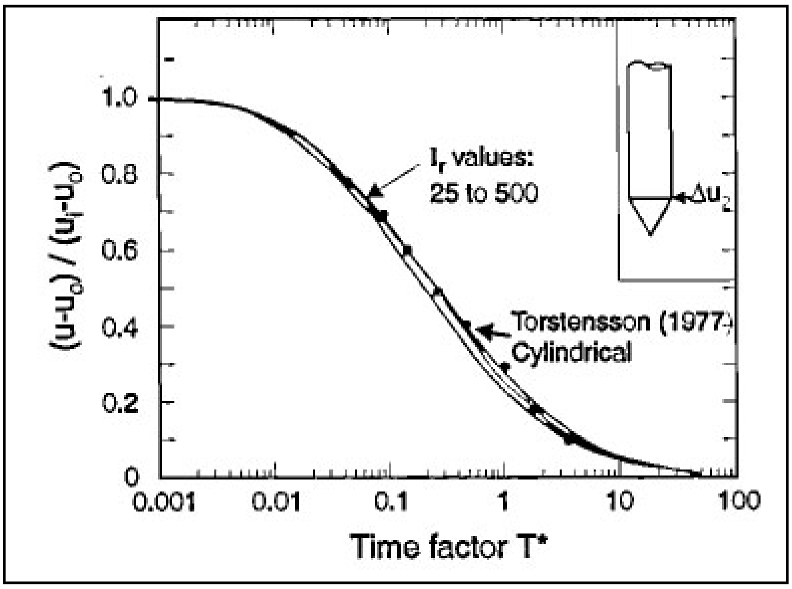


Figure 14 : Normalized Pore Pressure Dissipation vs T^* (after Teh & Houlsby, 1991)

Degree of Consolidation	T^*
20%	0.038
30%	0.078
40%	0.142
50%	0.245
60%	0.439
70%	0.804
80%	1.600

Table 3 : T^* values

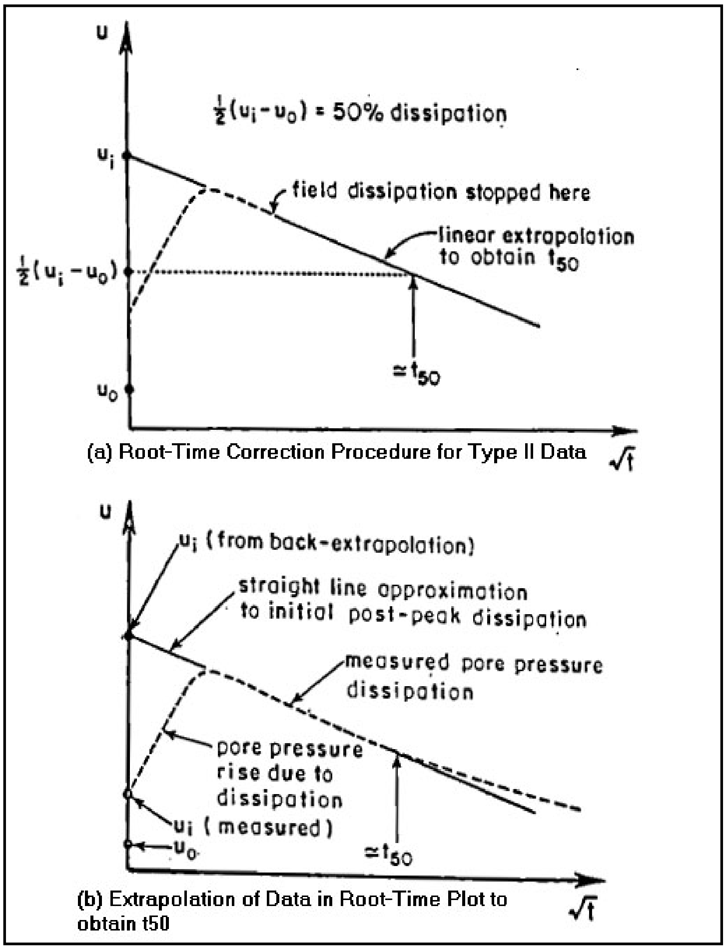


Figure 15 : Correction for Overconsolidation

Dissipation tests carried out in overconsolidated soils have shown that the pore pressures on stopping the penetration do not decrease immediately, instead they show an initial increase over a definite period of time before finally beginning to dissipate. If this occur, time correction methods proposed by Sully and Campanella (1994) can be used to carry out time correction for dissipation tests.

The two time correction method proposed are :

1. **Log-time plot correction**

The maximum pore pressure is taken as the peak value that occurs during the post-penetration increase and the time at which this peak occurs is taken as the zero time of the dissipation record and all other times adjusted according.

2. **Root-time plot**

In the root-time plot, the dissipation that occurs after the initial peak caused by redistribution of pore pressure, depicts a straight line which can be back-extrapolated to t=0 in order to obtain a u_I for the corrected dissipation curve. Refer to Figure 15 for details. The advantage of the root-time method is that the initial straight line portion can be extrapolated to 50% pore pressure reduction if short dissipation periods are used in the field and measured data to longer period are not available.

5.2 INTERPRETATION OF LABORATORY TESTS

The soil design parameters to be obtained from the laboratory tests can be divided into two (2) major categories :

(A) Strength parameters for stability and bearing capacity analyses of foundation.
(B) Stiffness and deformation parameters for prediction and evaluation of settlement, heave, lateral deformation, volume change, etc.

5.2.1 Strength Parameters

5.2.1.1 Total Stress

Total stress strength parameters of undrained shear strength, s_u for cohesive soils can be obtained directly or indirectly from laboratory testing. The laboratory testing that can indicate the s_u directly are :

- Unconfined Compression Test (UCT)
- Unconsolidated Undrained Triaxial Test (UU)
- Laboratory Vane Shear Test

If not enough undisturbed soil samples are collected, preliminary estimation of s_u can also be obtained indicatively by correlating to results of Atterberg Limit Tests as follows :

a) $s_u/\sigma_v' = 0.11 + 0.0037\ PI$
For normally consolidated clay, the ratio tends to increase with plasticity index (PI) (Skempton, 1957).

b) $s_{u(mob)}/\sigma_p' = 0.22$;
$s_{u(mob)}$ is the undrained shear strength mobilised on the failure surface in the field, and σ_p' is the preconsolidation pressure (yield stress) (Mesri, 1988).

5.2.1.2 Effective Stress

Effective stress strength parameters (e.g. c' and ϕ') for cohesive soils can be interpreted from the Mohr's Circle plot either from CIU, CID or shear box tests.

However there are advantages of obtaining the effective stress strength parameters through interpretation of stress paths. This stress paths method enables the field stress changes to be presented more realistically indicating the characteristic of subsoils and are generally plotted in total stress (Total Stress Path, TSP) and effective stress (Effective Stess Path, ESP).

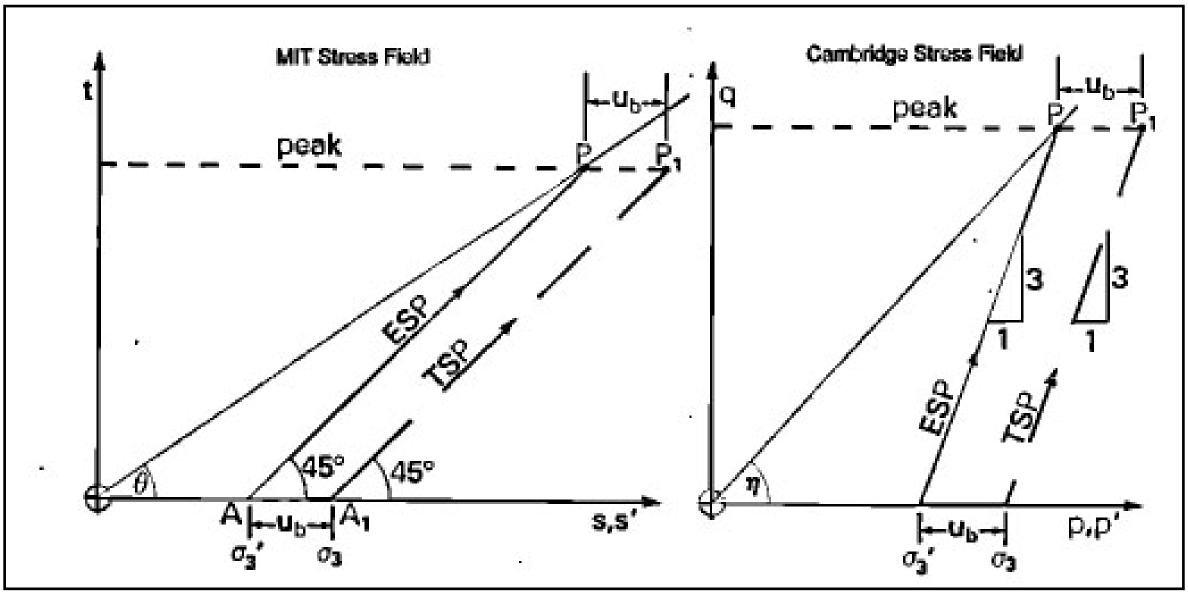


Figure 16 : MIT and Cambridge Stress Path Plot

There are two types of plot, namely MIT stress path plot and Cambridge stress path plot. The conventions used for these two stress path plot are as follows (see Figure 16):

(a) MIT Stress Path Plot, (t - s Plot)
Developed by T.W. Lambe of Massachusetts Institute of Technology (1967), USA.
The vertical axis :

$$t = (\sigma_1 - \sigma_3)/2 \quad = (\sigma'_1 - \sigma'_3)/2$$

The horizontal axis :

$$s = (\sigma_1 + \sigma_3)/2 \qquad \& \qquad s' = (\sigma'_1 + \sigma'_3)/2$$

(b) Cambridge Stress Path Plot (q - p Plot)
Developed by Roscoe, Schofield and Wroth (1958) at the University of Cambridge, England.
The vertical axis :

$$q = \sigma_1 - \sigma_3 \qquad = \sigma'_1 - \sigma'_3$$

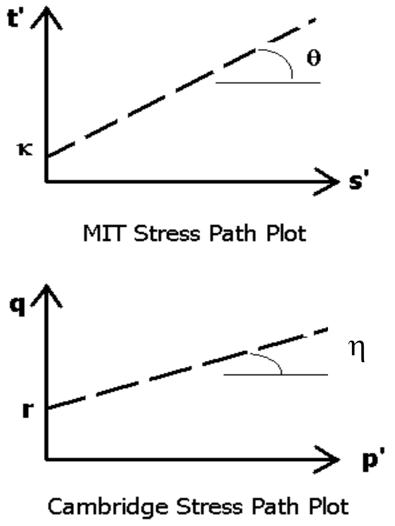
The horizontal axis :

$$p = (\sigma_1 + \sigma_2 + \sigma_3)/3 \qquad \& \qquad p' = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$$

for triaxial test, two of the principal effective stress are equal to the horizontal effective stress, therefore can be expressed as :

$$p = (\sigma_1 + 2\sigma_3)/3 \qquad \& \qquad s' = (\sigma'_1 + 2\sigma'_3)/3$$

Figure 17 shows the interpretation of Mohr-Coulomb failure envelope in compression from the MIT and Cambridge stress path plot respectively.



Tan θ = t' / s

Tan θ = Sin ϕ'

K = c' Cos ϕ'

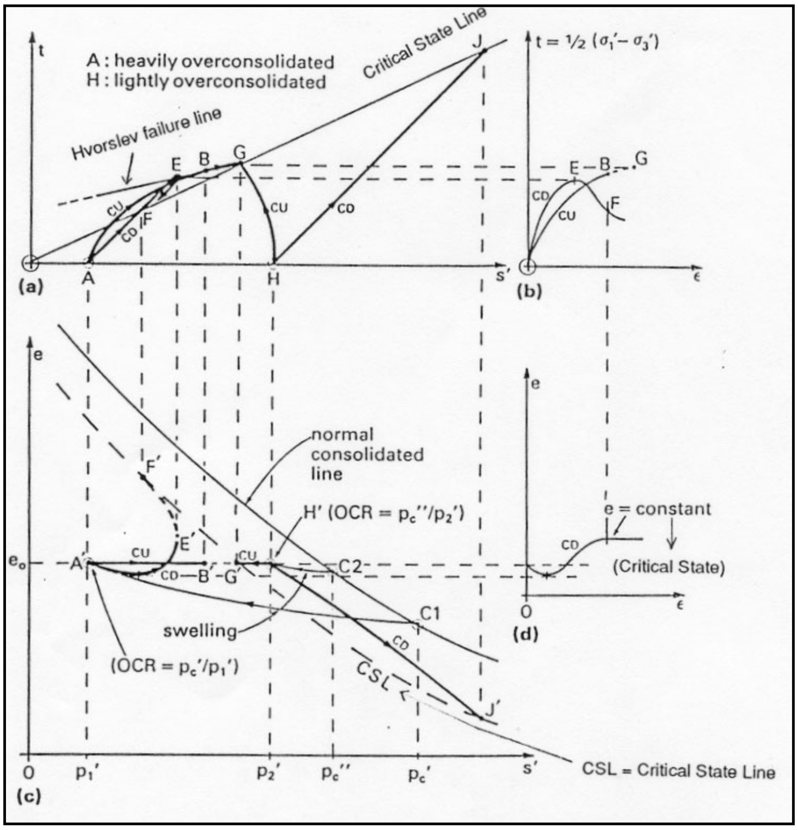
Tan η = q / p'

Sin ϕ' = (3 η) / (6 + η)

r = c' (6 Cos ϕ') / (3 – Sin ϕ')

Figure 17 : Interpretation of Mohr-Coulomb Failure Criteria

Stress path and Critical State Soil Parameters



Idealised stress paths related to critical state line for undrained and drained tests on over-consolidated clays :
(a) Stress paths in MIT field, (b) Stress-strain curves (t aginst strain) (c) Voids ratio against mean effective stress, s', with project stress paths . (d) voids ratio against strain

Figure 18 : Stress Path Interpretation

Critical state concept introduced by Roscoe, Schofield & Wroth (1958), relates effective stresses and void ratio. From the stress field, the surface where all effective stress paths reach or approach a line/surface, the "Critical State Line (CSL)" which is ultimate condition of soil (as in critical concept) in which the material deform (plastic shearing) at contact volume under constant effective stresses.

For highly overconsolidated material (as from Point A of the Figure 18), the failure (peak strength) follow the relationship found by Hvorslev (1937) and can be termed as "Hvorslev Failure Surface/Line"

Figure 19 shows the stress path of consolidated triaxial undrained test on normally

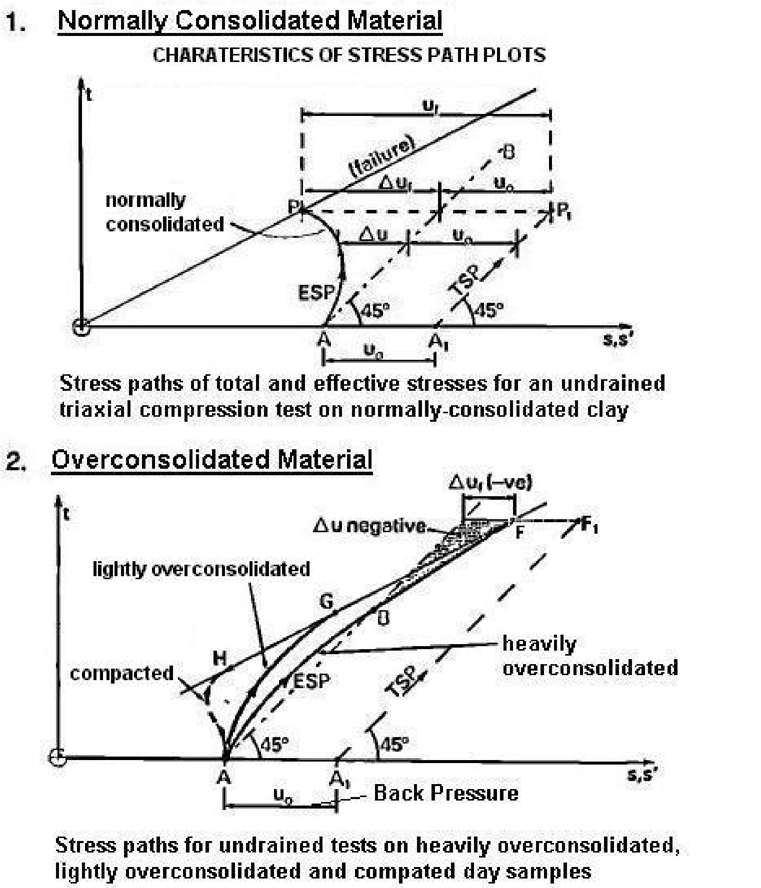


Figure 19 : Stress Path for CIU Tests on NC and OC Soils.

consolidated (NC) and overconsolidated (OC) materials respectively. It is observed that for normally consolidated material, pore pressure increases as the deviator stress is applied. As the overconsolidation ratio (OCR) of the material increases, the increase in pore pressure would reduce to a state where there is a reduction of pore pressure due to the effect of dilatancy (stress path curves to the right). Therefore, from the direction of stress path, the likelihood of material to be NC or OC can be known.

Correlation of Effective Angle of Friction (ϕ')

For preliminary assessment of the effective angle of friction, correlations shown in Figures 20 and 21 can be used.

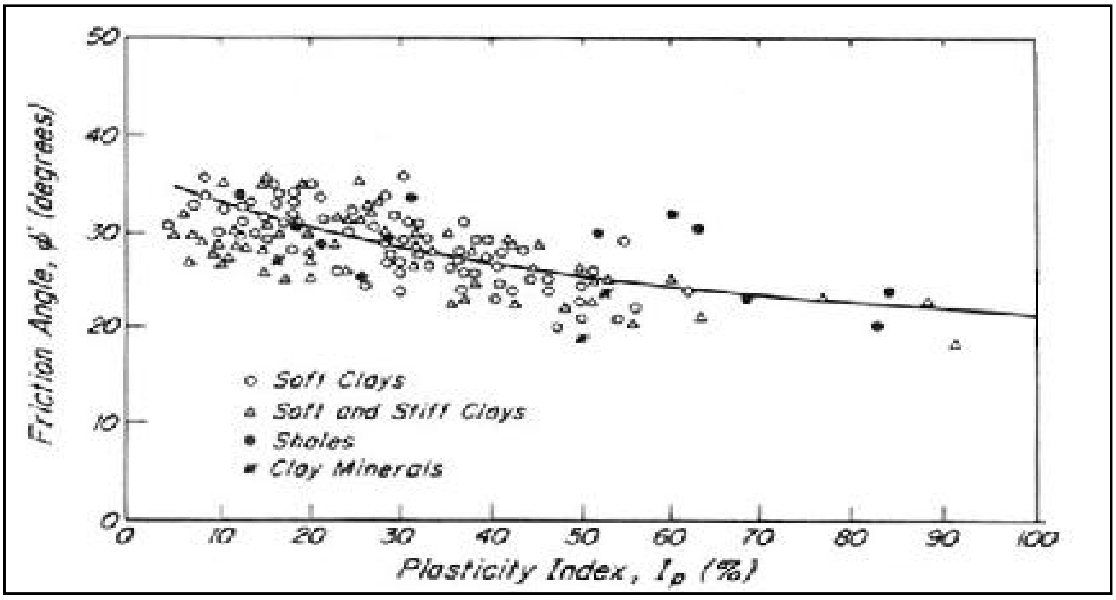


Figure 20 : Values of Effective Friction Angle ϕ' for Clays of Various Compositions as Reflected in Plasticity Index. (from Terzaghi et. al., 1996)

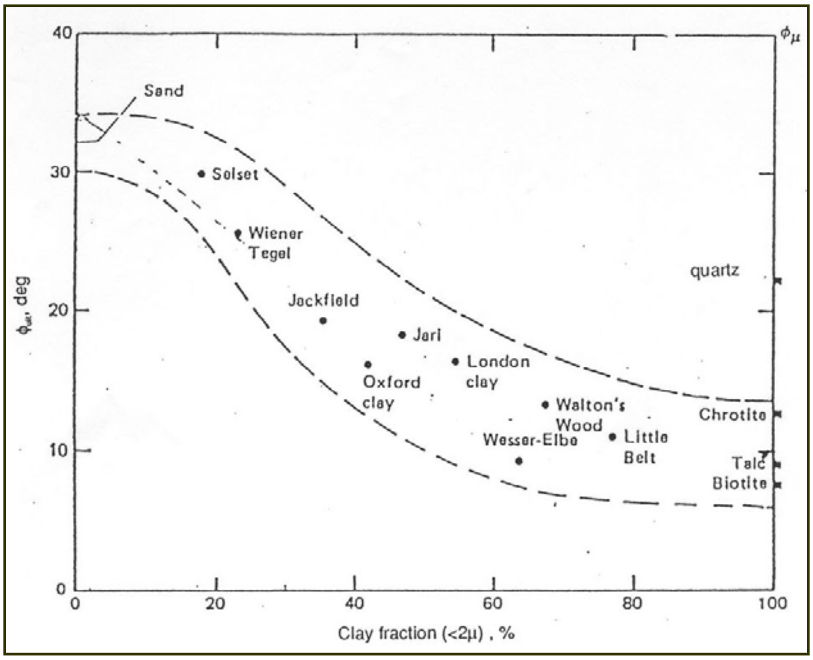


Figure 21 : Variation of ϕ'_{ult} with Percentage of Clay Content (after Skempton, 1964)

5.2.2 Stiffness and Deformation Parameters

The most commonly used deformation parameters for soft clay are obtained from consolidation test (Oedometer Test). The parameters are used to predict deformation (vertical) of the ground due to load , unload, water level changes, etc. and also the rate (time) required to achieve equilibrium (completion of settlement).

There is also indirect estimation of the consolidation parameters from Atterberg Limit tests as follows. However the parameters for detailed design should be obtained directly from consolidation tests.

a) C_c = 0.007 (LL-10%)
For normally consolidated clay, (Skempton, 1944).

b) C_c = 0.009 (LL-10%)
For clays of low and medium sensitivity, (Terzaghi & Peck, 1967).

6.0 CONCLUSION

A well planned and full-time supervised subsurface investigation (S.I.) is necessary to obtain reliable subsoil information and parameters for safe and economical designs. Although there may be an increase in awareness of the need for subsurface investigation, however this does not necessary means there is an increase in understanding of what subsurface investigation can achieve. Hence clients need to be made to understand that insufficient and unplanned subsurface investigation will lead to poor design and subsequently means higher cost and sometimes unsafe design for a project.

“ Without Proper S.I., Ground is a HAZARD !!! ”

“ You Pay for S.I. whether You Carry it Out Properly or Not !! ”

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