Planning of Subsurface Investigation and Interpretation of Test Results for Geotechnical Design

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ABSTRACT: There have been many case histories of cost overrun and construction failures related to insufficient, mis-interpreted and unreliable geotechnical information acquired from subsurface investigation and laboratory tests used in the design of the geotechnical works. In view of the importance of subsurface investigation, this paper presents the planning and selection of subsurface investigation works, 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 tests for foundation design is also discussed.

1 INTRODUCTION

We are all familiar with the cartoon of the leaning tower of Pisa, Italy with the engineer saying "*I skimped a bit on the subsurface investigation, but no-one will ever know*.". Despite so many case histories of failures reminding us of the importance of proper subsurface investigation and correct interpretation of test results, until today, engineers are still squeezed by insufficient budgets and impossible programmes for subsurface investigation. Some engineers still lack the understanding or have indifference attitude towards subsurface investigation and interpretation of the test results.

During planning of the subsurface investigation, Engineer shall always remember that majority of the unforeseen costs and failures are associated with construction 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 paper presents the planning and selection of subsurface investigation works, 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 geotechnical design is also discussed.

2 PLANNING OF SUBSURFACE INVESTIGATION

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

<u>Stage 1 : Preliminary S.I.</u>

Preliminary S.I. aims mainly to confirm a proposed layout and its formation levels in relation to safety, cost and time for geotechnical work. Hence, a general subsoil profile together with its preliminary soil parameters and water table are obtained for preliminary designs and cost estimates.

<u>Stage 2 : Detailed S.I.</u>

Detailed S.I. is usually carried out after optimum layout has been selected and confirmed. It aims to obtain refined soil profile and properties for safety and optimum designs. The detailed S.I. is concentrated in critical areas of concern such as :

- Areas with difficult ground conditions such as very soft soils, suspected limestone cavities and pinnacles.
- Areas of major cut and fill.
- Locations with structures having high retaining walls and major columns.

The planning of the subsurface investigation 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 includes review 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 should be checked 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 tests. Information of adjacent development like types of structures and foundation system is also very useful for design and to prevent the proposed project affecting the serviceability of adjacent structures. If the subsoil information of adjacent site is available, it will also help the design engineer to optimise the S.I. required for the project.

(e) *Requirements of the Proposed Structures or Foundations*
In order to plan proper and cost effective S.I., the design engineer should have sufficient information on the requirements of the completed structures and their tolerance of differential movements.

3.2 Site Reconnaissance

The purpose of the site reconnaissance is to confirm information obtained in desk study and also to 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 design 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 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 subsurface information
- Geological formation and features
- Variability of subsoil and groundwater
- Proposed structures and platforms
- Adjacent properties and their conditions

<u>Preliminary S.I.</u>

(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 (more lines are needed for a large area)

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

(c) *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 7 times consecutively and at least one borehole coring into rock. In limestone area, continuous coring into solid rock for 10m is required to detect cavities.
- boring and probing in fill area of a formation
- Figures 1 and 2 show some theoretical guidelines for the extent of S.I. work. However in an actual field work depth of boreholes is usually deeper because the foundation system is yet to be decided and the cost of going deeper is not significant as compared to cost of mobilisation.

(d) *Geophysical Survey*
Geophysical survey should be used for large area and to determine the general bedrock profile and characteristics

<u>Detailed S.I.</u>

(a) *Spacing*

- There is no hard and fast rules but generally 10m to 30m for structures. The spacing can be increased for alluvial subsoil with more consistent layers (as interpreted from preliminary S.I.) or where 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.

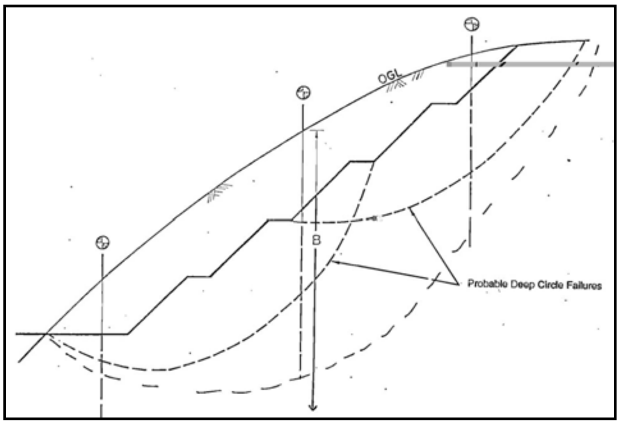


Figure 1 : Depth of Field Tests for Stability Analysis

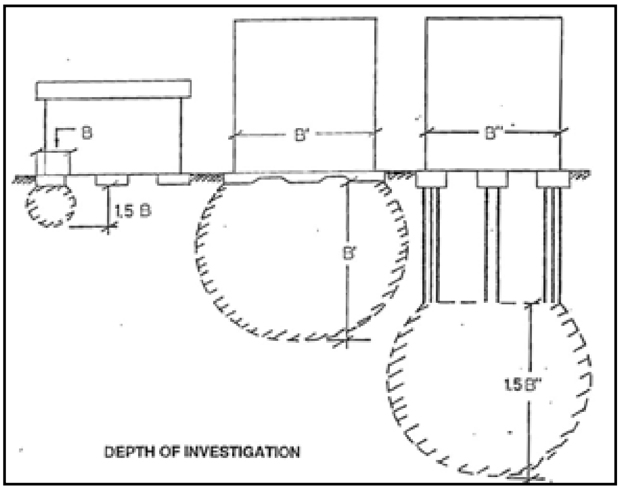


Figure 2 : Depth of Field Tests for Foundation Design

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. The commonly used field testing methods in subsurface investigation are :

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

3.4.1 Light Dynamic Penetrometer (JKR or Mackintosh Probe)

JKR or Mackintosh probe is usually used in preliminary S.I. to supplement boreholes and to identify subsoil variation between boreholes particularly in areas of very soft soils. It assists in interpolation between boreholes or piezocones. Figure 3 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.

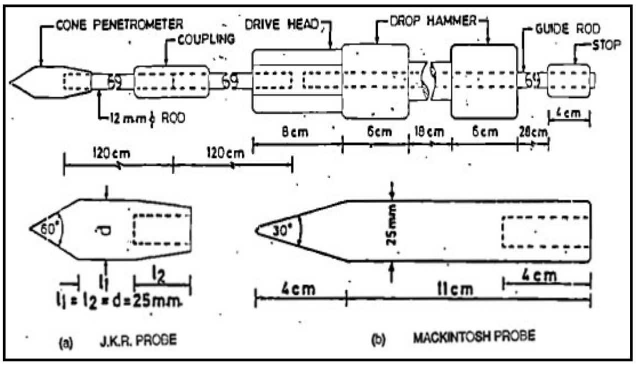


Figure 3 : Mackintosh and JKR Probes

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 are :

- 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 and rock coring.

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 4 shows the typical drilling rig for borehole.

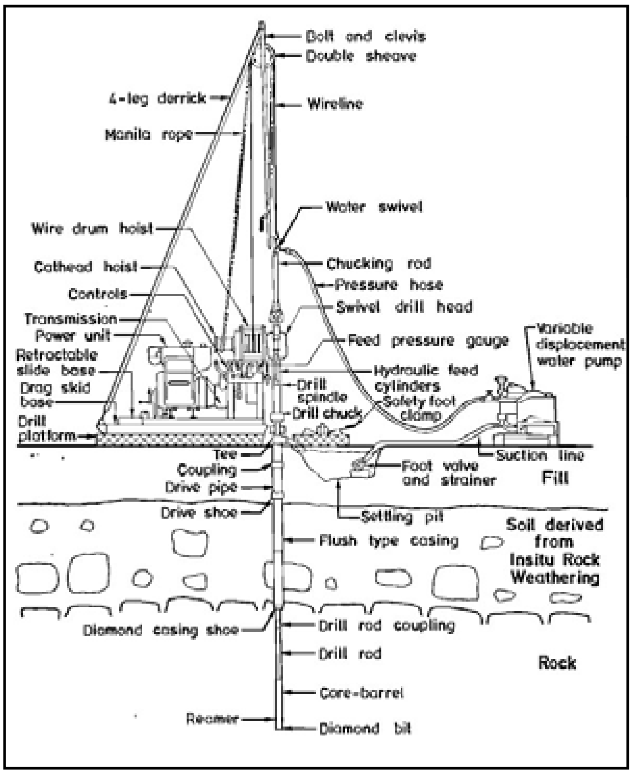


Figure 4 : Typical Rotary Drilling Rig (from Geoguide 2, 1987)

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 penetration 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. Figure 5 shows the equipment for Standard Penetration Test.

SPT is generally carried out at 1.5m depth interval. 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 inside the casing.
- Base of borehole must be properly cleaned.
- 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.

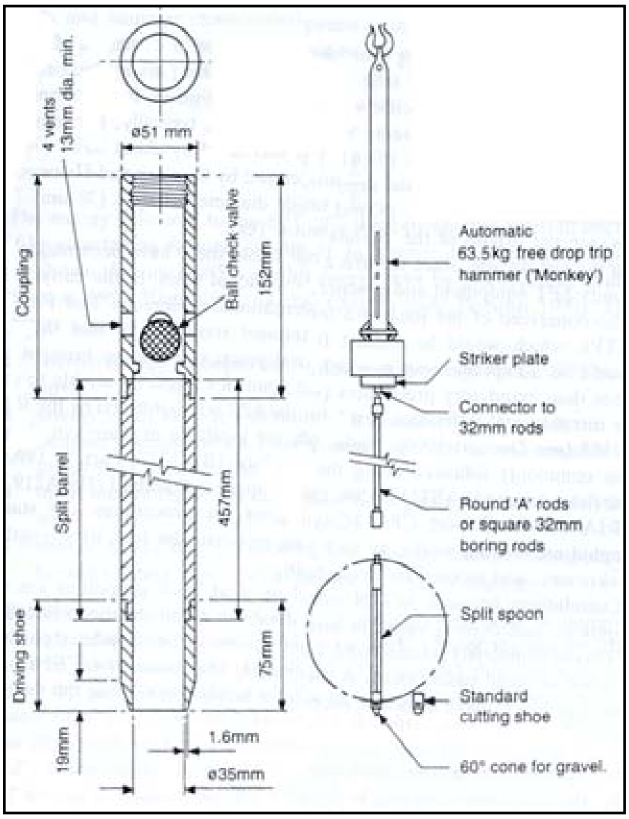


Figure 5 : Equipment for Standard Penetration Test (after Clayton 1995)

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 6 is used for very soft to soft cohesive soil which is easily disturbed. Usually this layer has SPT'N' < 2 or hammer weight. For cohesive soils from soft to firm consistency (SPT'N'<10) and free from large particles (e.g. marine deposits), thin wall sampler as shown in Figure 7 can be used. 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.

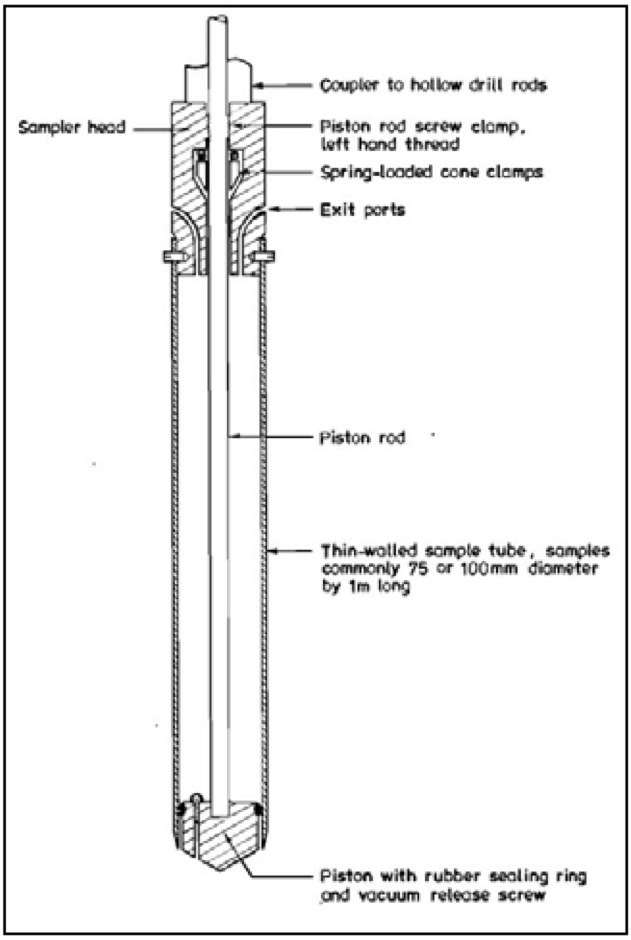


Figure 6 : Piston Sampler

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 8 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 72mm. Most of the hill-site development or foundation in stiff soils require the use of Mazier sampler. For best recovery of samples, foam drilling should be used.

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)
- Samples should be properly stored and packed for transport to prevent disturbance during transportation.

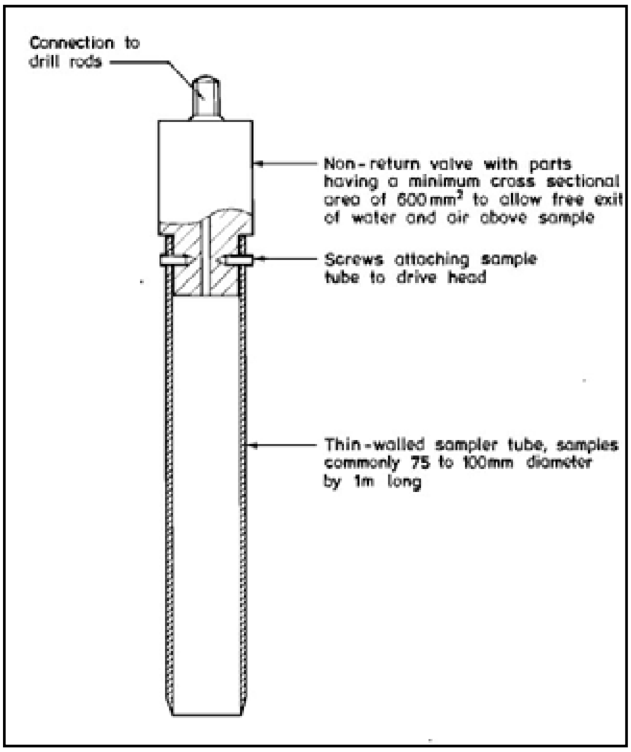


Figure 7 : Thin Wall Sampler

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.

Normally for the design of foundation into bedrock (other than limestone), the minimum rock coring is 3m. In granitic formation or large boulders are suspected, coring of 5m may be needed. In limestone, borehole usually terminate after 10m coring into cavity free rock or about maximum 15m coring into rock. If there is thick hard / dense overburden overlying the limestone, the maximum coring into rock may be reduced.

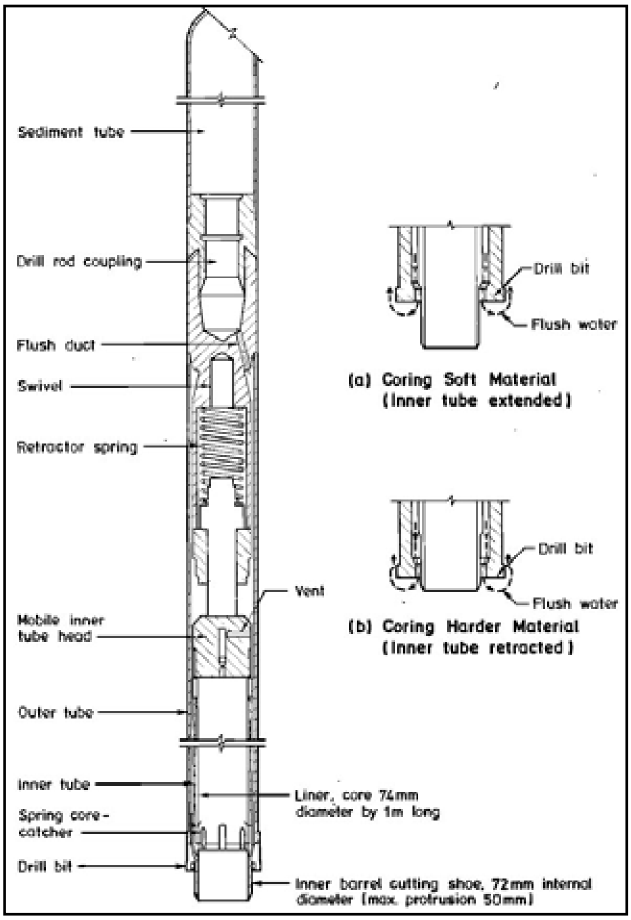


Figure 8 : Mazier Sampler

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

Field Vane Shear Test is suitable to obtain in-situ undrained shear strength of very soft to firm clay. However the results will be misleading if tested in peats or in clays containing laminations of silt, sand, gravels or roots. The field vane shear test is used to obtained 'undisturbed' peak undrained shear strength, and remoulded undrained shear strength thus sensitivity of the soil. Figure 9 shows the equipment details.

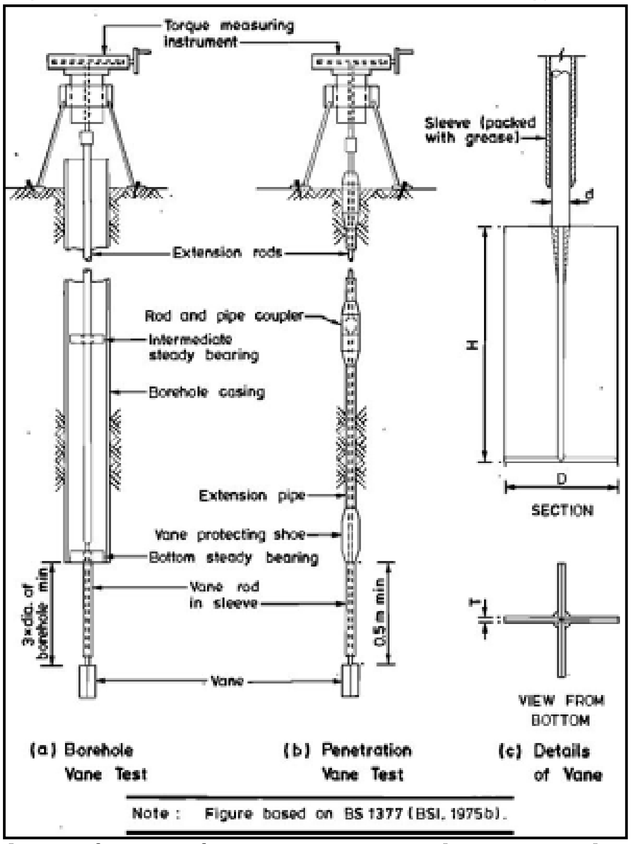
Following are some reminders when using field vane shear tests :

- The vane must be rotated soon (within 5 minutes) after insertion into the depth to be tested as delays may lead to overestimation of strength by 10% to 20%.
- 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 : Field Vane Shear Devices

3.4.4 Piezocone (CPTU)

Figure 10 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 is measured. In order to measure the pore water pressure and its dissipation, porous filter (e.g. porous plastic, ceramic or sintered stainless steel) which allow rapid movement of extremely small volumes of water is used 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 and deaired so that reliable pore pressure measurement can be obtained. All the data are captured electronically on computer therefore reduces human error. Figure 11 shows the rig for piezocone testing.

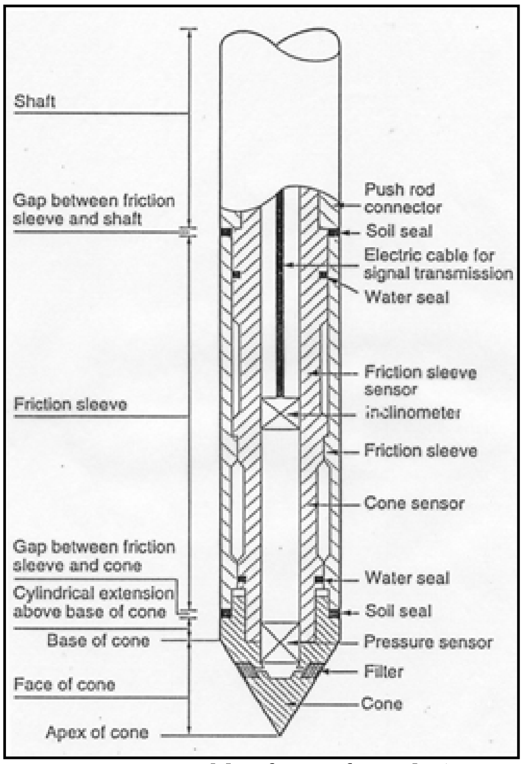
Piezocone (CPTU) has three main applications :

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

Figure 10 : Detailed Terminology of Piezocone

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 and obtain consolidation parameters.
- Has been widely used and many correlations available.

3.4.5 Pressuremeter

Pressuremeter tests can be carried out both in soils and in rock. The pressuremeter is a cylindrical device designed to apply horizontal uniform pressure to the ground via a flexible membrane. It is connected by tubing or cabling to a control unit at the ground surface. The aim of a pressuremeter test is to obtain the stiffness, and in weaker soils on the strength by measuring and interpreting the relationship between radial applied pressure and the resulting deformation. Figure 12 shows the typical pressuremeter.

Pressuremeter has two main purposes :

- To estimate in-situ earth pressure at rest, K_o
- To estimate geotechnical parameters for strength and stiffness.

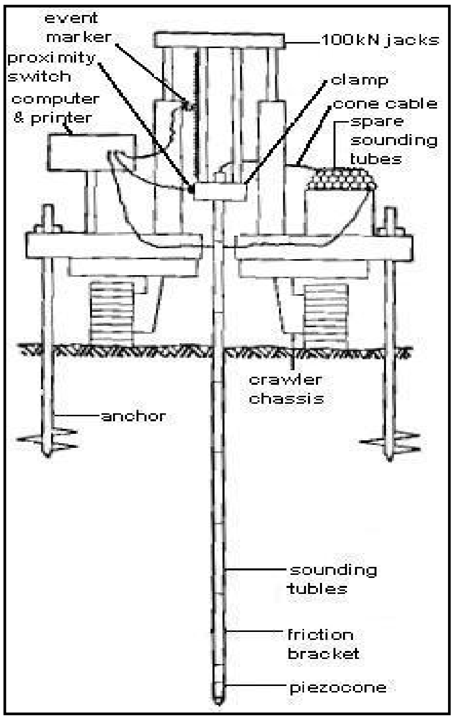
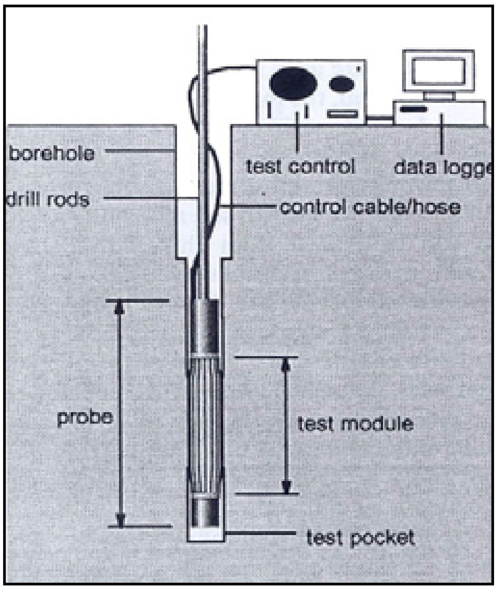


Figure 11 : Typical Piezocone Rig

The borehole pressuremeter (commonly called Menard Pressuremeter), originally developed by Menard is commonly used in Malaysia. A borehole is formed using drilling rig capable of producing a smooth-sided test hole in the stiff soils. The pressuremeter which has a slightly smaller outside diameter than the diameter of the hole, can be lowered to the predetermined level in the borehole before being inflated for testing.

When drilling of the borehole for pressuremeter test, rotation of the drill bit should be very slow (less than 60rpm). The circulation of the drilling mud should also be very slow (no bubbles or big ripples on the return to the mud pit). Because of the slow mud flow, some of the cuttings will not come back up all the way to the mud pit and will settle back at the bottom of the borehole once the mud flow is stopped. Therefore, it is recommended to drill 1m past the selected depth of testing to allow the cuttings to settle at the bottom of the hole without filling the portion of the hole where the test is to be carried out. It is very important to calibrate the pressuremeter before use. The details of this test can be found in Clarke (1995) and Briaud (1992).

Figure 12 : Definition of a Pressuremeter



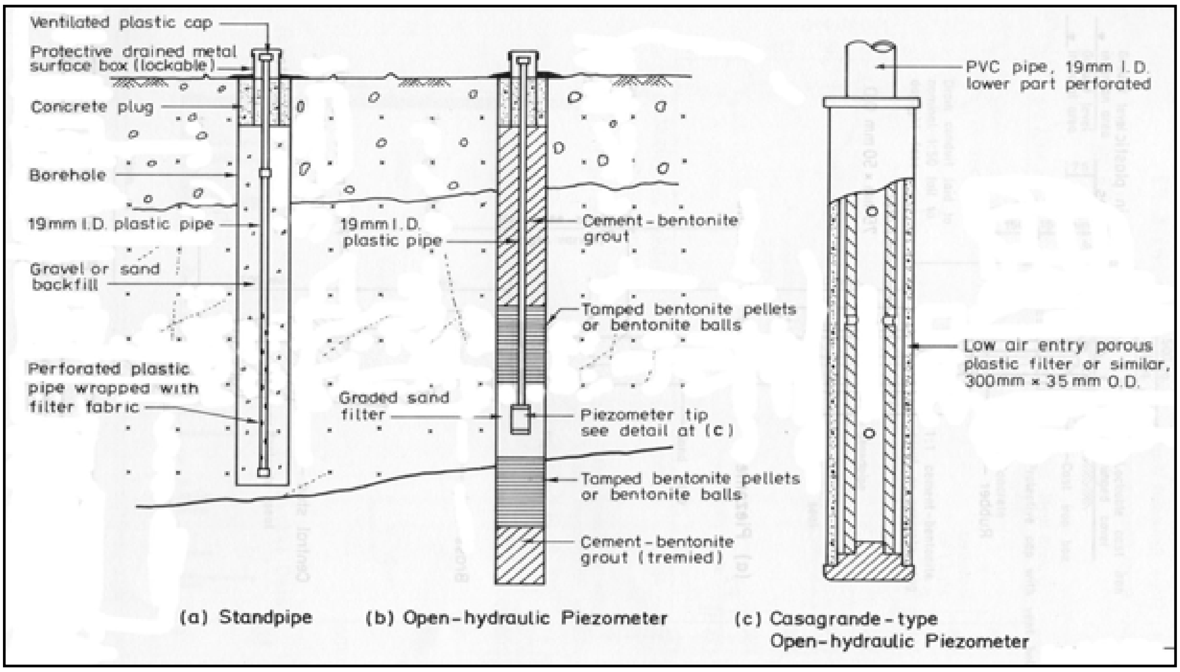


Figure 13 : Typical Standpipe and Open-Hydraulic Piezometers (after Geoguide 2, 1987)

3.5 Determination of Groundwater

The information 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. The groundwater level for cut slopes also plays a very important role in influencing the factor of safety against slip failure.

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. For cut slopes, standpipe piezometers can also be installed in the slope to monitor the long term ground water level so that the slope stability design can be validated.

Standpipe and open-hydraulic piezometers installed in borehole are shown in Figure 13. 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.

Usually vibrating wire piezometers are installed in basement excavation works to measure the changes of pore water pressure during excavation and in long term after completion of the basement. It is particularly useful where automatic recording of readings is needed.

4 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, retaining wall, embankment and slopes, particularly cut and fill slopes. Consolidation parameters allow engineer to evaluate deformation of the subsoil when there is changes of stress in the subsoil.

SOIL CLASSIFICATION TEST	TEST FOR MECHANICAL PROPERTIES
1. Particles Size Distribution : - Sieve Analysis (for content of sand and gravels) and Hydrometer Tests (for content of silt and clay)	1. One Dimensional Consolidation Test (Oedometer Test) :- to obtain compressibility and consolidation parameters for settlement analysis.
2. Atterberg Limits :- Liquid Limit, Plastic Limit & Plasticity Index (to be used in Plasticity Chart for soil classification)	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))
3. Moisture Content	
4. Unit Weight	
5. Specific Gravity	
CHEMICAL TEST	
1. pH Test	
2. Chloride Content Test	
3. Sulphate Content Test	
4. Organic Content Test	3. Compaction Test

Table 1 : Laboratory Testing

The main purpose of chemical tests on the subsoil except organic content is to detect any chemicals that are detrimental to concrete and other materials used and buried inside the ground.

5 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 of this paper are to illustrate the importance of correct interpretation and show methods of compiling results and recognising errors. The following section will cover the brief interpretation of commonly used field and laboratory tests in Malaysia and they are :

<u>(A) Field Tests :</u>
- Light Dynamic Penetrometer (JKR or Mackintosh Probe)
- Standard Penetration Test (SPT)
- Field Vane Shear Test
- Piezocone (CPTU)
- Pressuremeter

<u>(B) Laboratory Tests :</u>
- 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)

JKR or Mackintosh probe is used for :

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

Figure 14 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 are wrong computation of spring factor and if the clay contains organic materials (e.g. sea shells, decayed woods, peat, etc).

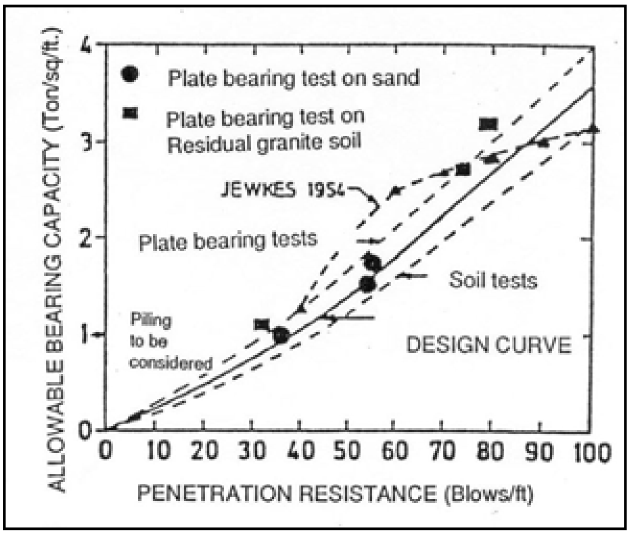


Figure 14 : Correlation of JKR Probe Resistance to Allowable Bearing Capacity (after Ooi & Ting, 1975)

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 as shown in Figure 15. 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.

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 using a modified dimensionless time factor, T^* as given in Table 3, and is 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 16 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

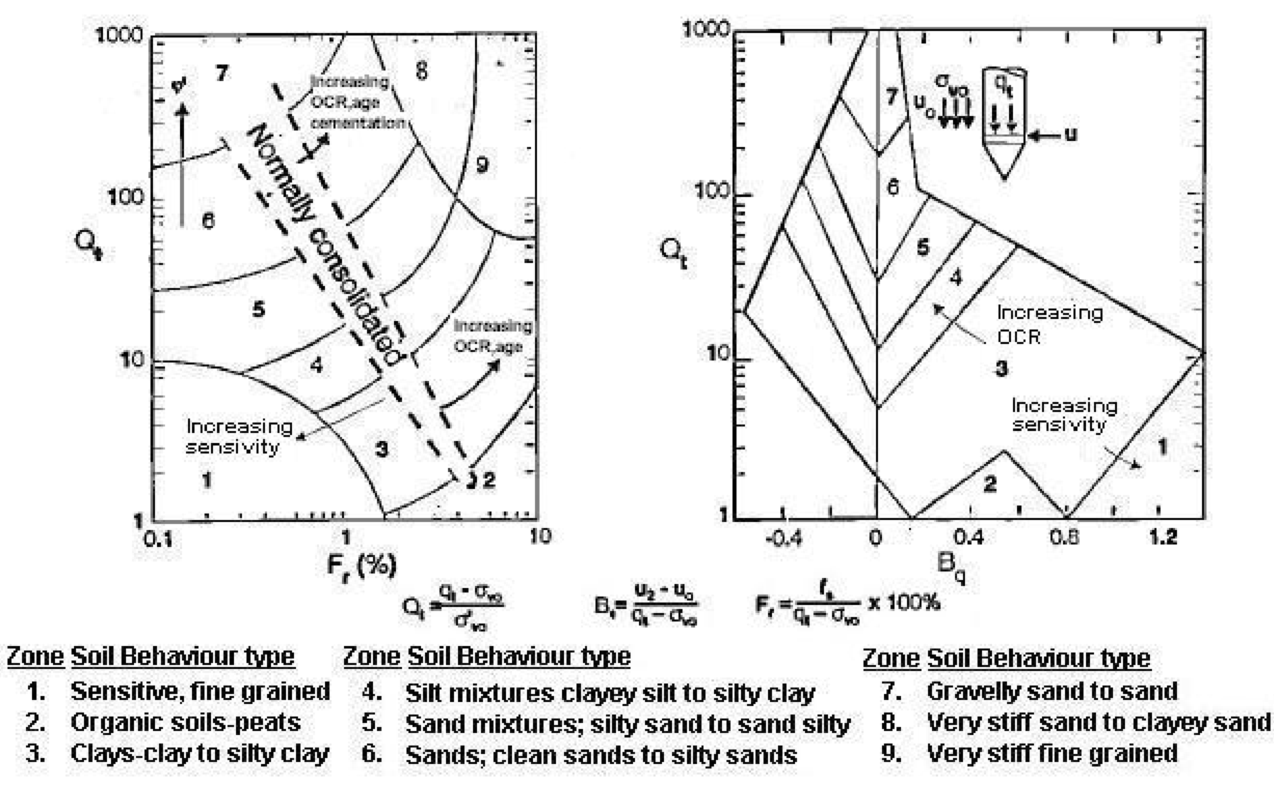


Figure 15 : Soil Classification Chart (after Robertson, 1990)

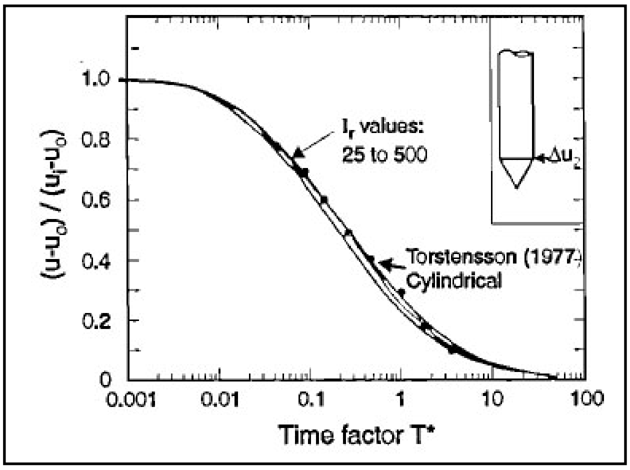


Figure 16 : Normalized Pore Pressure Dissipation vs T* (after Teh & Houlsby, 1991)

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

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 :

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.

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. Figure 17 shows the 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.

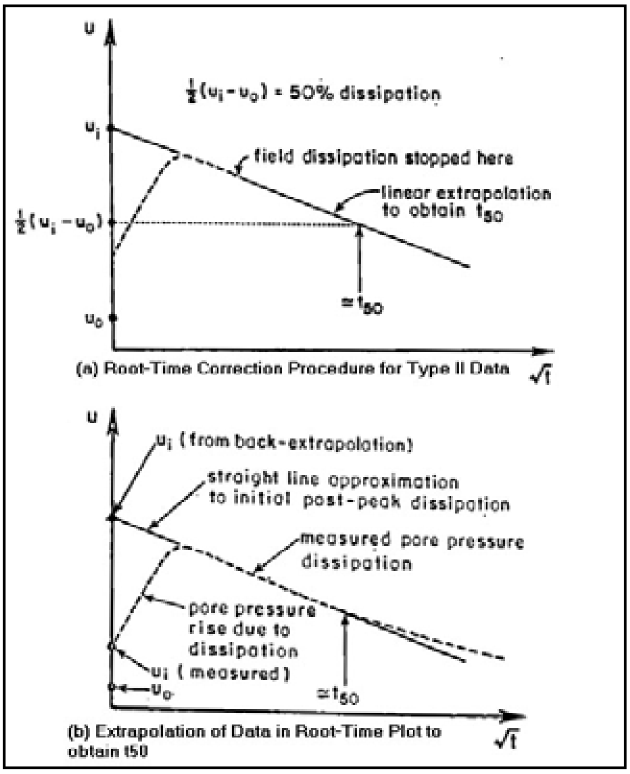


Figure 17 : Correction for Overconsolidation with Root Time Plot

5.1.5 Pressuremeter

The soil parameters normally obtain from the pressuremeter tests are as follows :

- Coefficient of earth pressure at rest, K_o
- Undrained shear strength, s_u of cohesive soil
- Effective Angle of Friction, ϕ' for cohesionless soil
- Young's Modulus, Es'
- Shear Modulus, G

In view of the extensiveness of the methods to obtain these parameters, they will not be discussed in this paper. For details please refer to Clarke (1995) and Briaud (1992).

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

<u>Total Stress</u>

Total stress strength parameters of undrained shear strength, s_u for cohesive soils can be obtained directly or indirectly from laboratory tests. The laboratory tests that can provide 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).

<u>Effective Stress</u>

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).

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 18):

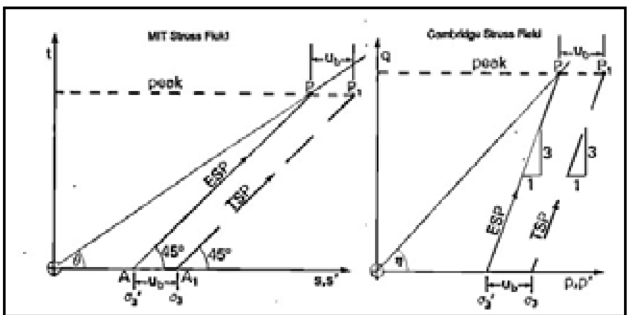


Figure 18 : MIT and Cambridge Stress Path Plot

<u>(a) MIT Stress Path Plot, (t - s Plot)</u>

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 :

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

<u>(b) Cambridge Stress Path Plot (q - p Plot)</u>

Developed by Roscoe, Schofield and Wroth (1958) at the University of Cambridge, England.

The vertical axis :

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

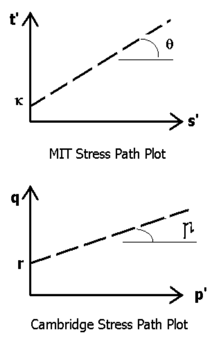
The horizontal axis :

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

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

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

Figure 19 shows the interpretation of Mohr-Coulomb failure envelope in compression from the MIT and Cambridge stress path plots respectively.



$\text{Tan}\ \theta = t' / s$
$\text{Tan}\ \theta = \text{Sin}\ \phi'$
$K = c'\ \text{Cos}\ \phi'$

$\text{Tan}\ \eta = q / p'$
$\text{Sin}\ \phi' = (3\ \text{Tan}\eta) / (6 + \text{Tan}\eta)$
$r = c'\ (6\ \text{Cos}\ \phi') / (3 - \text{Sin}\ \phi')$

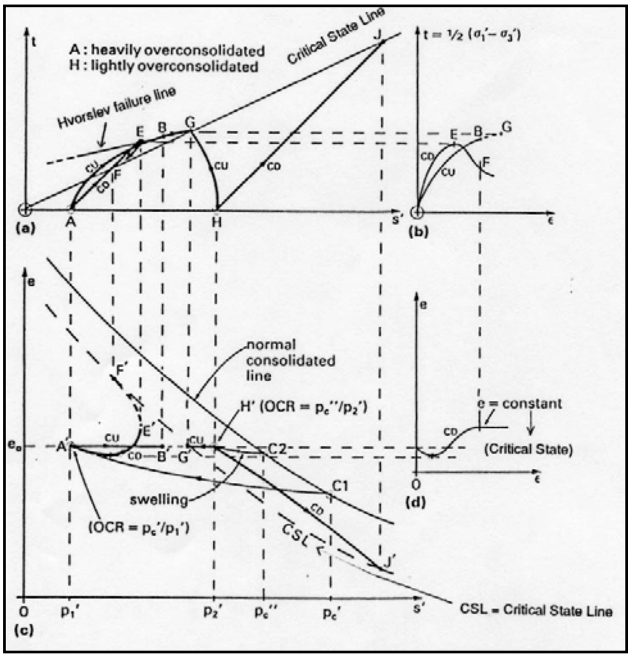
Figure 19 : Interpretation of Mohr Coulomb Failure Criteria

<u>*Stress path and Critical State Soil Parameters*</u>

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 20), the failure (peak strength) follow the relationship found by Hvorslev (1937) and can be termed as "Hvorslev Failure Surface/Line"

Figure 21 shows the stress path of consolidated triaxial undrained test on normally 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.



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 20 : Stress Path Interpretation

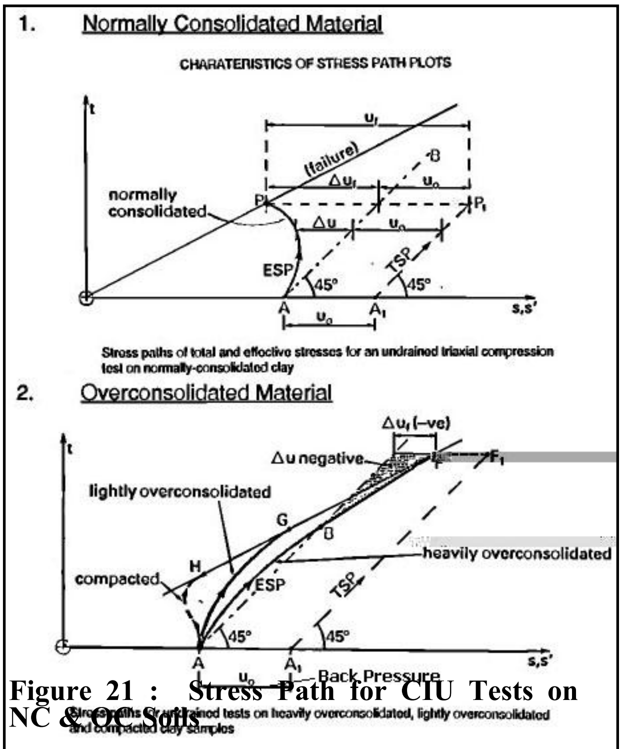


Figure 21 : Stress Path for CIU Tests on NC & OC Soils

Correlation of Effective Angle of Friction (ϕ')

For preliminary assessment of the effective angle of friction, correlations shown in Figures 22 and 23 can be used.

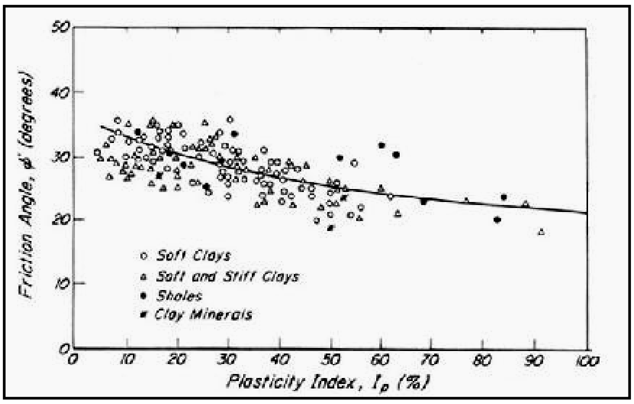


Figure 22 : Values of □' for Clays of Various Compositions as Reflected in Plasticity Index (from Terzaghi et. al.,1996)

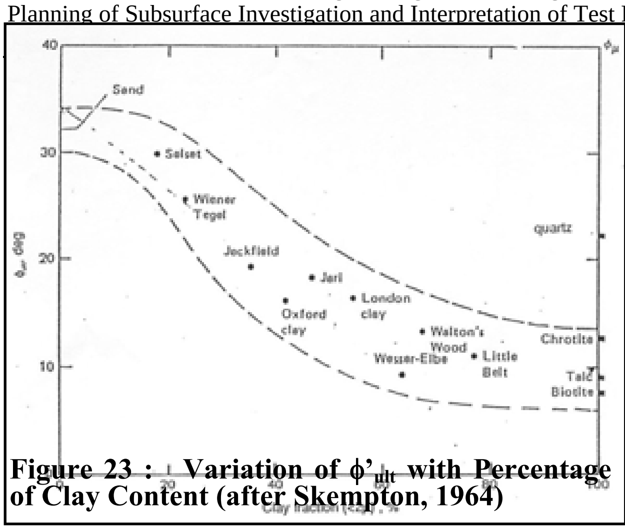


Figure 23 : Variation of ϕ'_{ult} with Percentage of Clay Content (after Skempton, 1964)

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).

Where C_c = Compression Index
LL = Liquid Limit.

The recompression index, C_r, is typically ranges from 0.015 to 0.35 (Roscoe *et. al*. 1958) and for preliminary assessment, often assumed to be 5% to 10% of C_c

6 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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