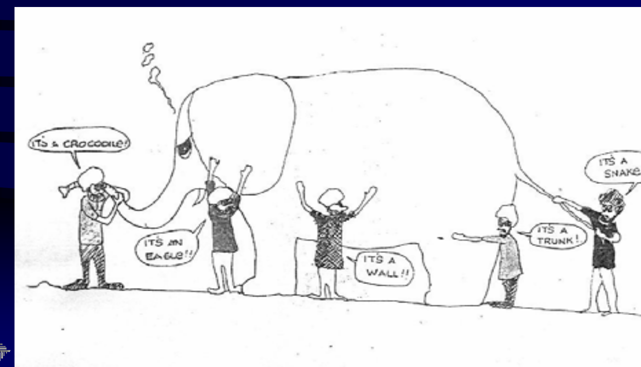
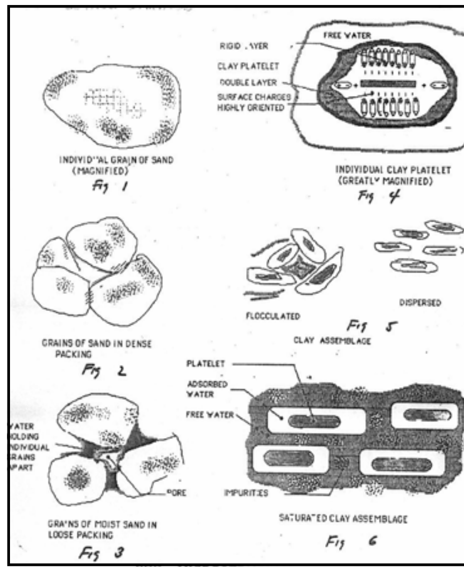
SOIL MECHANICS AND APPLIED FOUNDATION ENGINEERING

The Subsurface is Unknown to many and Blind Guesswork cannot be used to Determine the Character and Behavior of the Underlying Soil Conditions.



FUNDAMENTAL FACTS OF SOIL BEHAVIOR



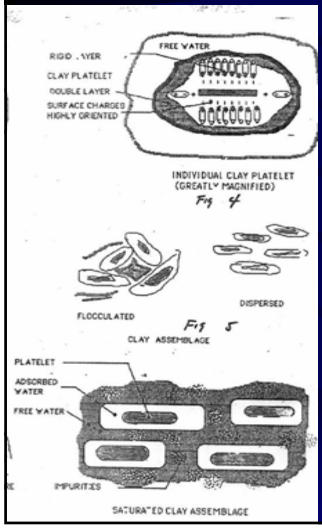
An intimate Understanding of Soil Microstructure is important in the solution of Problems in Soils and Foundations.

The Microstructure is governed by the Parent Material, Depositional and erosion patterns, Physico-chemical reactions and the amount of weathering in the soils.

At a Micro level the significance of the grain size becomes very important in predicting how the soil would behave under various environmental influences such as loading, presence of water and response to mechanical compaction.

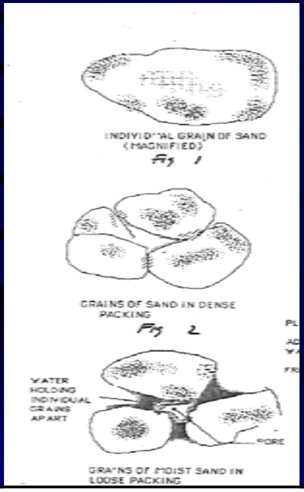
Soils can be classified thus as:
* Coarse Grained Granular soils (sands and Gravels).
*Fine Grained soils (Silts and Clays).
*Fibrous Materials (Peats and Highly Organic Soils) .

Fine Grained Soils

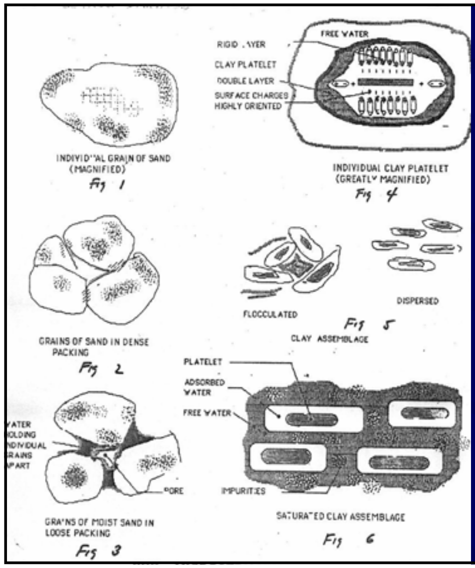


- **BASIC FUNDAMENTAL FACTS**
- Fine Grained soils such as Clays and silts are almost submicroscopic in Grain size and as such they have greater surface area to Volume ratio.
- The small grain size is significant because the behavior of the soil is dependent on Electrical and chemical forces of Attraction rather than contact friction Grain to Grain.
- Water thus plays a big role in the behavior of these soils because water is absorbed and adsorbed by the individual clay platelet. The degree attraction of the several layers of water between the clay platelets are determined by the subatomic distance from the core which is electrically charged. Strongly held water is bonded electrically and for all practical purposes cannot be removed except with tremendous application of pressure and/or temperature
- The outermost layer is weakly held because of the distance to the platelet and thus can be dislodged easily.
- The Characteristic shear strength of the soil, its "cohesion" is therefore dependent on the degree and magnitude of these attractive forces.
- The expulsion of porewater in fine grained soils is a time dependent process due to the Characteristic low permeability of these soils. Thus time is needed after load application before settlements occur.
- Saturated fine grained soils when subjected to rapid application of loads(Vibratory or Static) would result in rapid elevation of pore pressure which can be detrimental to the microstructure..

Coarse Grained Soils



- **BASIC FUNDAMENTAL FACTS**
- Coarse Grained soils normally can be seen by the naked eye.
- The size and shape of the grains determine the frictional resistance that could be mobilized by the soil through intimate grain to grain contact.
- As a granular material, the characteristic strength is dependent on the stress history and intimacy of the interparticle grain to grain contact brought about by the manner of deposition, parent material and confinement stress. The higher the confining stress, the higher is the resistance against shearing or sliding.
- The strength of the granular soil, its shear strength is determined by the amount and integrity of the grain to grain contact and the crushing resistance of the asperities in the grain In turn, these are governed by the confining stresses which are due to the overburden stress and previous stress history.
- There is no Unique Phi Angle (Φ) for any given type of granular soil. Again this is dependent on several factors.
- Water does not dramatically affect the performance of Coarse grained soils in the way it does with fine grained clays and silts. Except in very unique and special conditions such as in the presence of strong groundshaking and loose soil condition.
- Loose coarse grained soils can be easily dislodged by vibration or ground shaking..
- .



Side by side comparison of soil Microstructure

APPLICATIONS OF BASIC PRINCIPLES OF SOIL MICROSTRUCTURE TO EVERYDAY PROBLEMS

Bearing Capacity and Settlement

Soil Bearing Capacity

Classical Terzaghi Bearing Capacity Equation $\Phi>0; C>0$

- $q_{ult} = cN_c + q\,N_q + 0.5\,\gamma B N_\gamma$
  - Where:
  - $N_q =$ $a^2/(a\cos^2(45+\Phi/2)$
  - $a =$ $e^{(0.75\pi - \Phi/2)\tan\Phi}$
  - $N_c =$ $(N_q-1)\cot\Phi$
  - $N_\gamma =$ $[(\tan\Phi)/2]\,[(K_{p\gamma}/\cos^2\Phi)-1]$

Classical Terzaghi Bearing Capacity Equation When $\Phi = 0$ (Cohesive Soil)

- $q_{ult} = cN_c + q\,N_q + 0.5\,\gamma B N_\gamma$
  - Where:
  - $N_q =$ $a^2/(a\cos^2(45+\Phi/2)$
  - $a =$ $e^{(0.75\pi - \Phi/2)\tan\Phi}$
  - $N_c =$ $(N_q-1)\cot\Phi$
  - $N_\gamma =$ $[(\tan\Phi)/2]\,[(K_{p\gamma}/\cos^2\Phi)-1]$

$q_{ult} = cN_c + q\,N_q$

Classical Terzaghi Bearing Capacity Equation When c=0 (Cohesionless Soil)

- $q_{ult} = cN_c + q\,N_q + 0.5\,\gamma B N_\gamma$
  - Where:
  - $N_q =$ $a^2/(a\cos^2(45+\Phi/2))$
  - $a =$ $e^{(0.75\pi - \Phi/2)\tan\Phi}$
  - $N_c =$ $(N_q - 1)\cot\Phi$
  - $N_\gamma =$ $[(\tan\Phi)/2]\,[(K_{p\gamma}/\cos^2\Phi) - 1]$

$$q_{ult} = q\,N_q + 0.5\,\gamma B N_\gamma$$

Settlement

Immediate or Elastic Settlements

Consolidation Settlements

Immediate Settlements

$\Delta H = q_o B'[(1-\mu^2)/(E_s)]$

Where:

q_o=Intensity of Contact pressure

B'=Least dimension of Footing

μ, E_s =Elastic parameters

Consolidation Settlements

$\Delta H = C_c/[(1-e_o)]H[\log\{(p_o+\Delta p)/p_o\}]$

Where:

C_c =Coefficient of consolidation

p_o =overburden stress

Δp =Incremental pressure

e_o =initial voids ratio

Consolidation Settlements

$$\Delta H = m_v \Delta p H$$

Where:

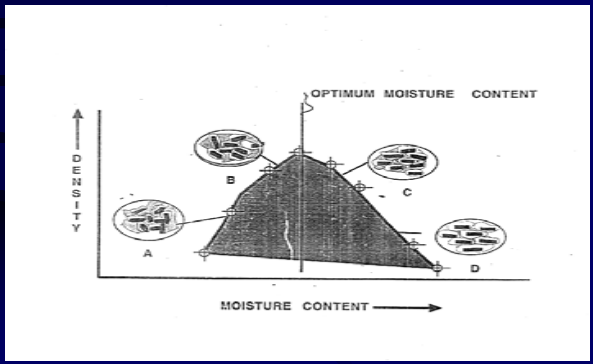
m_v =constrained modulus=2.3(1+e_o)/Cc

Δp =Incremental pressure

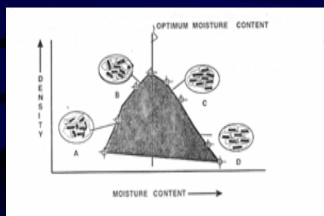
1) SOIL COMPACTION AND GROUND IMPROVEMENT

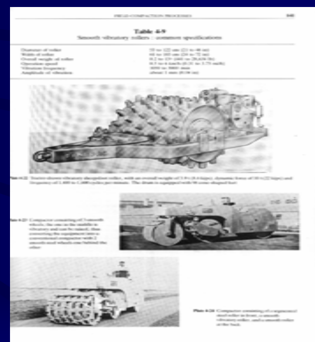
Mechanical Compaction was Probably Started by the Chinese in the ancient past

Fine Grained Soils Characteristic Behavior when subjected to compaction at varying Moisture Contents (MC)

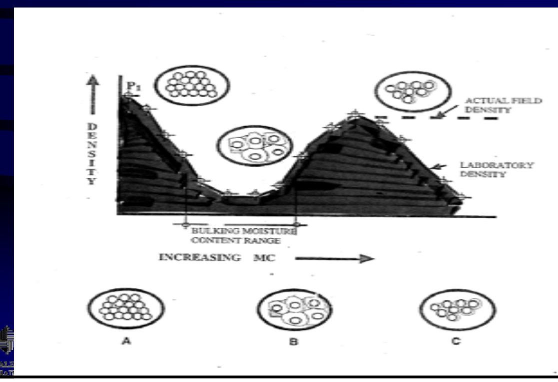


Characteristic Behavior when subjected to compaction at varying Moisture Contents (MC)





Coarse Grained Soils
Characteristic Behavior of Clean Granular Soils (Sands and Gravels) when subjected to compaction at varying Moisture Contents (MC).



Coarse Grained Soils
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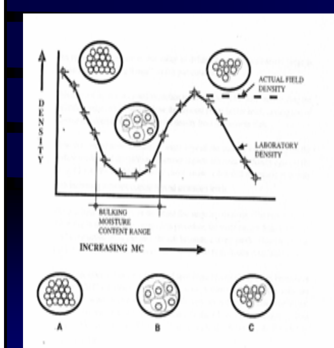
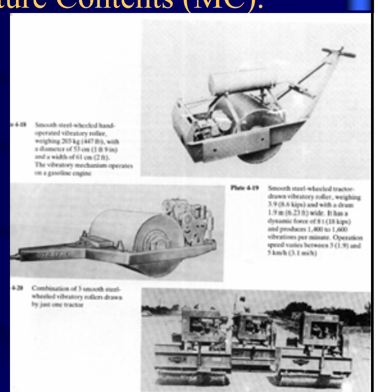
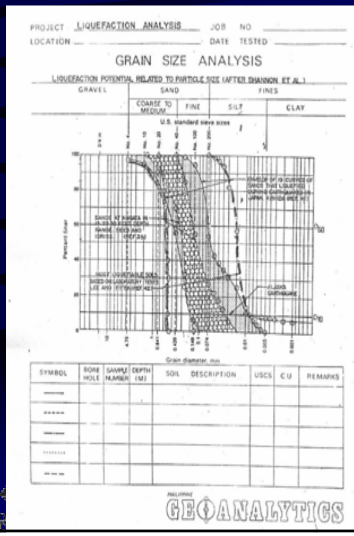
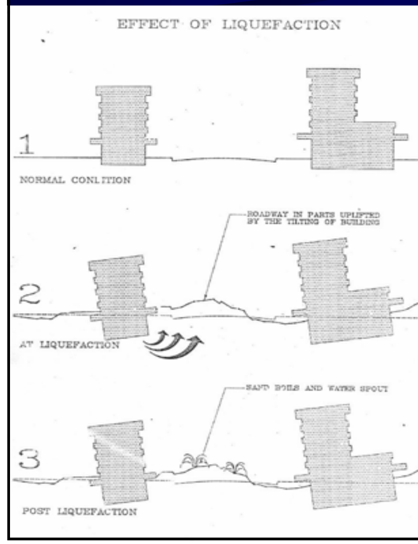




2) SOIL LIQUEFACTION PHENOMENON



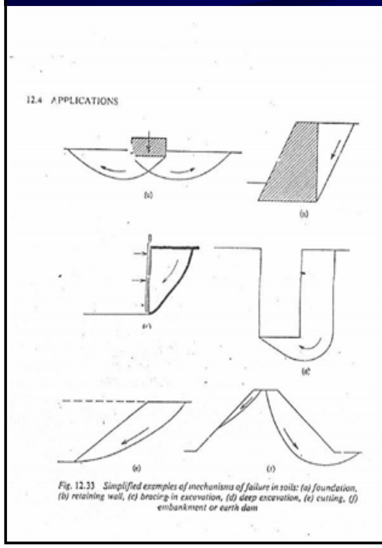
Certain Sandy soils under the right conditions of Looseness, Saturation and Strong Ground Motion due to Earthquakes Can be induced to " Liquefy "



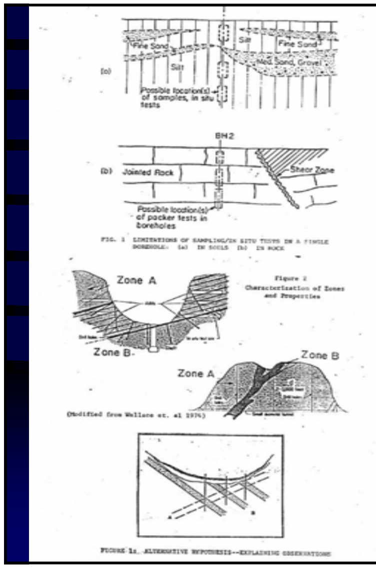
A site underlain by Potentially Liquefiable soils, when subjected to Strong Ground Motion Behaves like a Fluid which possess very little Shear Strength. Thus, it is incapable of sustaining loads from structures.

Ground Investigation and its Importance

Soils Exploration and why it is needed.



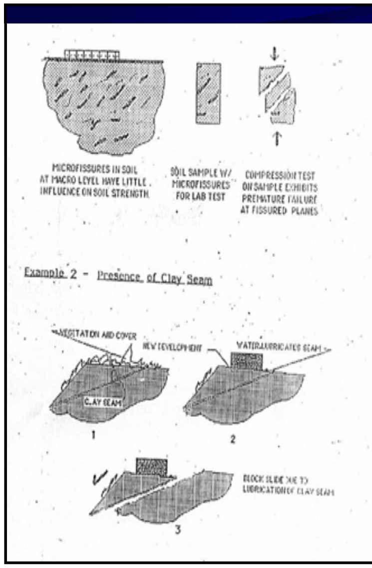
Soil behaves in different ways when subjected to various load influences and exposures. Geometry plays a critical role in stability problems involving soils



The results of Conventional Soil Exploration and sampling procedures often leave critical gaps in the Data gathering process.

Continuous Profiling of the subsoil is most desirable but is seldom done due to heavy costs involved.

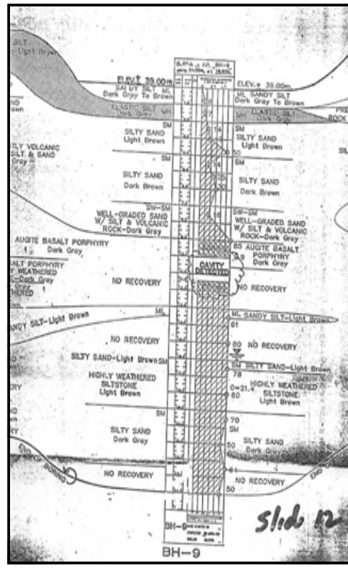
Thus, it would be desirable to augment this with other procedures which could fill in gaps in the data gathered or provide continuous profiles.



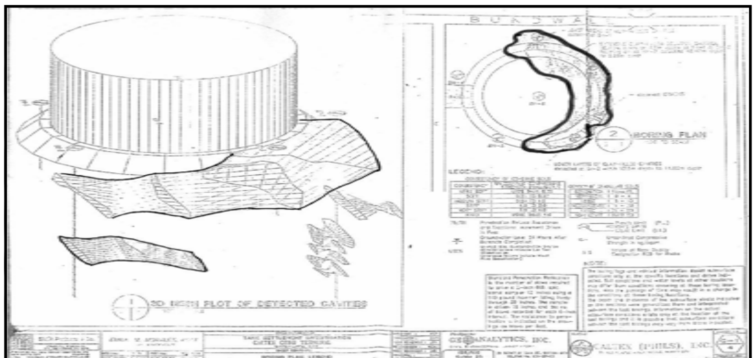
Microfissures in Soils may not be important at a Macro Level but could lead to erroneous test results and wrong interpretation of actual soil behavior during Loading.
Likewise, the presence of lines of weaknesses at the Macro level but undetected during Soil Investigation could lead to unforeseen Distress and costly problems.

GEOTECHNICAL AND GEOLOGIC ANOMALIES REVEALED BY SOIL EXPLORATION

CAVITIES AND VOIDS

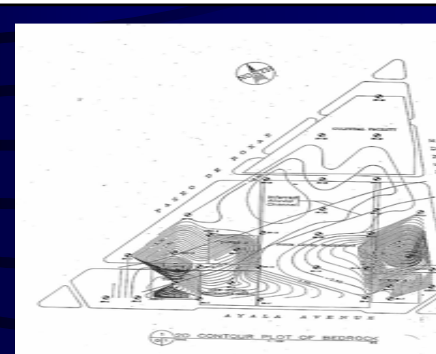


Large Cavities and other Geotechnical anomalies can be detected by careful attention to drilling procedures and telltale signs during the exploration program.

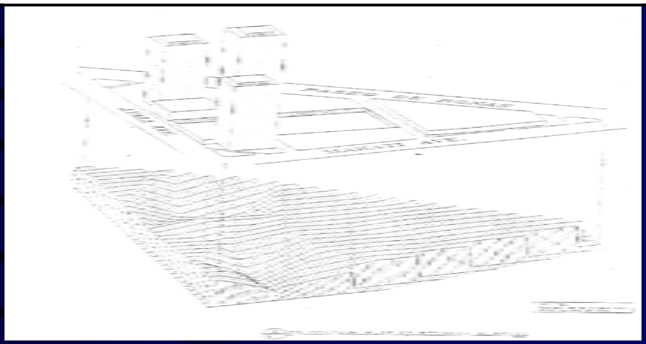


The major cavities and solution channels underneath the Tank were not detected by previous soils exploration. During Construction and even before the Tank could be commissioned, large settlements occurred, damaging the Tank Shell and Floor plates. Subsequent detailed investigation revealed the Cavities and solution channels which were plotted in 3D. The discovery resulted in relocation of the tank and abandonment of the existing site after major costs have been incurred in the construction of the Tank and its

Foundation.

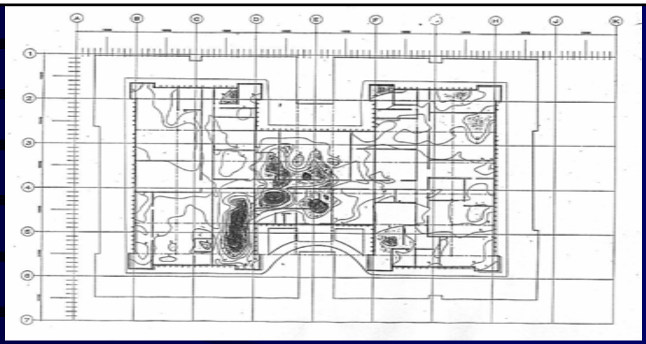


The presence of ancient buried streams in various Developed Land poses a problem more so if it remains undetected due to inadequate soils exploration.

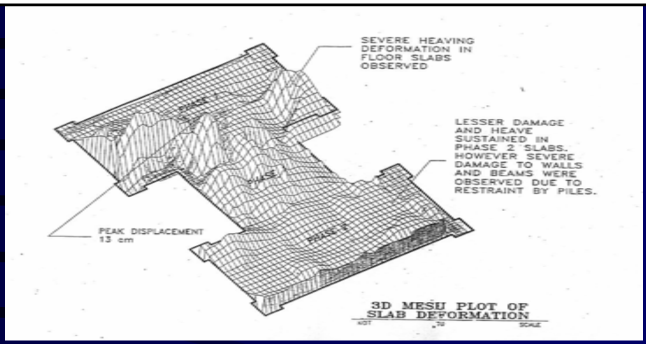


In this specific case, an ancient buried stream partly underlies the proposed site of a High Rise building. The initial soil borings indicated this possibility and this was verified by additional borings and Seismic Refraction methods in two directions. The 3D plot shows the depressions . As a result, the building was offset forward to avoid the Stream and the basement level was increased to fully seat the mat on bedrock .

PRESENCE OF SWELLING SOILS

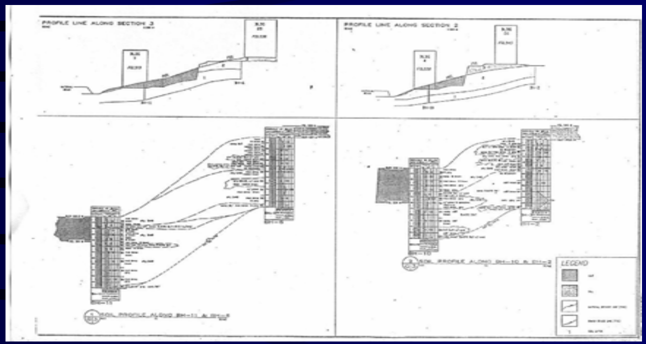


The Manifestations of swelling soils in some instances are misinterpreted as the reverse-"Settlement" and the corresponding response and corrective actions even aggravates the problem. Thus it is important to fully understand Soil Behavior in order that proper remedial measures can be implemented effectively.

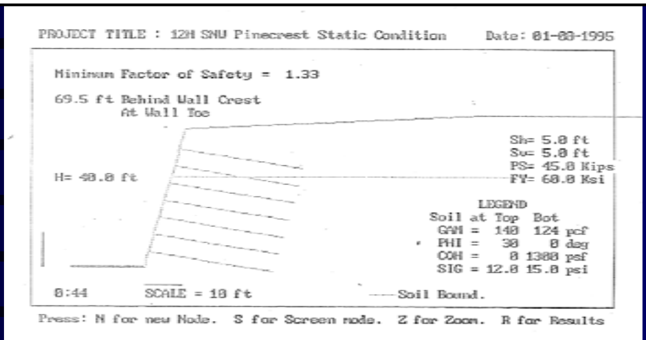


The 3D Plot of the floor Slab revealed the Heave Magnitudes and severity of swelling .

STABILITY OF SLOPES AND EARTHWORKS



The study and Utilization of Marginal Slopes in Land Development is becoming very important because of the scarcity of Land.



Critical unstable slopes can be stabilized by introducing Inclusions in the soil in a process known as "Soil Nailing."

Result of a Slope Erosion that triggered a Slide.

SOIL EXPLORATION
EQUIPMENT

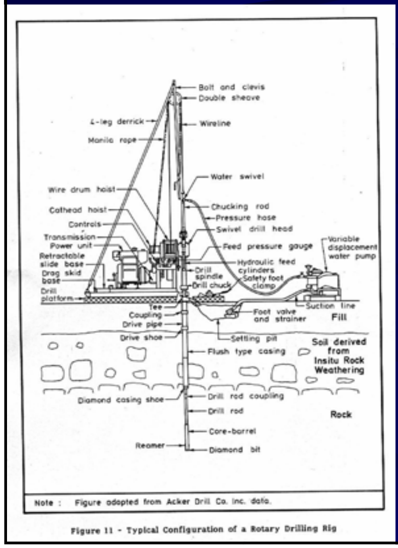
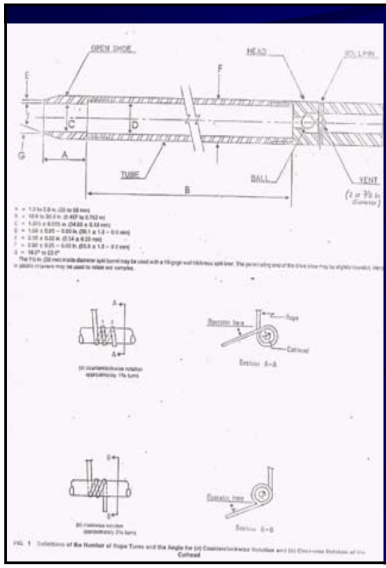
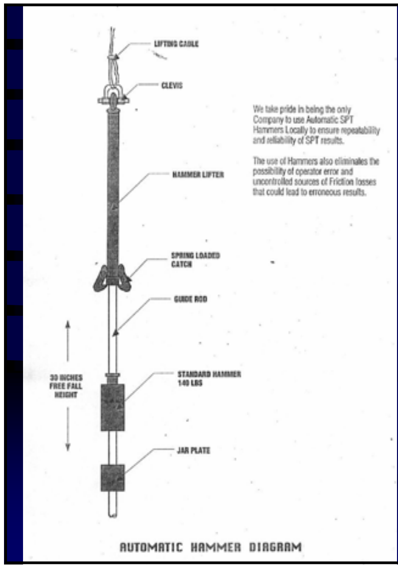


Figure 11 - Typical Configuration of a Rotary Drilling Rig

The Typical
Rotary Drilling
Rig Setup.



The Standard Penetration Test uses several turns of a Manila Rope on a Cathead to raise a 140 LB. Hammer 30" above. The coil is slackened to release and Drop the Hammer to deliver Blows to the SPT Sampler. It is readily apparent that the procedure is very much subject to the skill level of the operator and his dedication.



AUTOMATIC HAMMER DIAGRAM

Much of the operator error inherent in a Manual Procedure can be eliminated with the use of an Automatic Trip Hammer Device.

ALTERNATIVE METHODS OF SOIL EXPLORATION

The Electric Cone Penetrometer CPT/CPTU

The Electric Cone Penetrometer (CPT/CPTU) .

*The Electric cone penetrometer is widely used in Europe and the US for soil exploration particularly for Soils of low to medium consistency or Density.

*It has the advantage of speed and low cost while providing continuous profiling data of the subsurface.

*Empirical correlations with direct and derived CPT data are widely available for the prediction of:

* Strength
* Unit Weight
* Soil Classification
* Compressibility
* Liquefaction Potential
* Ko, Elastic parameters etc.

Thus dependence on extraction and testing of samples in the lab are reduced to the minimum

necessary only for correlations and verification

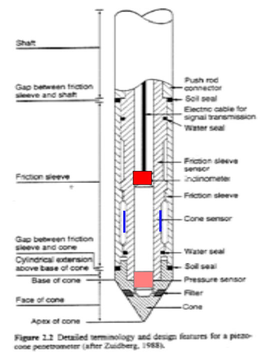


Figure 2.2 Detailed terminology and design features for a piezo-cone penetrometer (after Zuidberg, 1988).

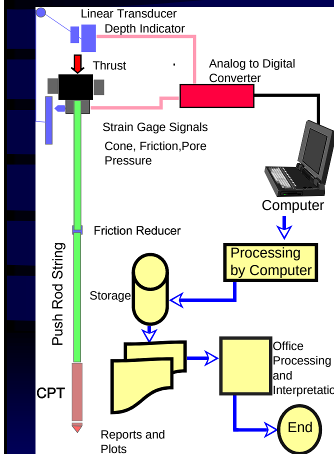
Figure 2.3 Two different positions of u_2 filter.

The Electric Cone penetrometer consists of the following basic components:

- Cone Tip and transducer.
- Friction Sleeve and transducer.
- Pore pressure port and transducer.
- Inclinometer.

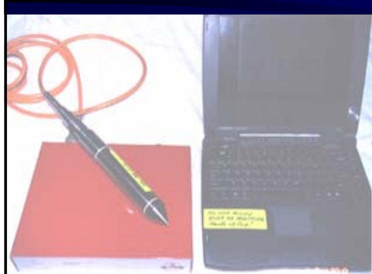
All these components are linked to the analog to digital converter by Electric cable.

CPT Rig with 6X6 LGP Chassis and Screw Anchors



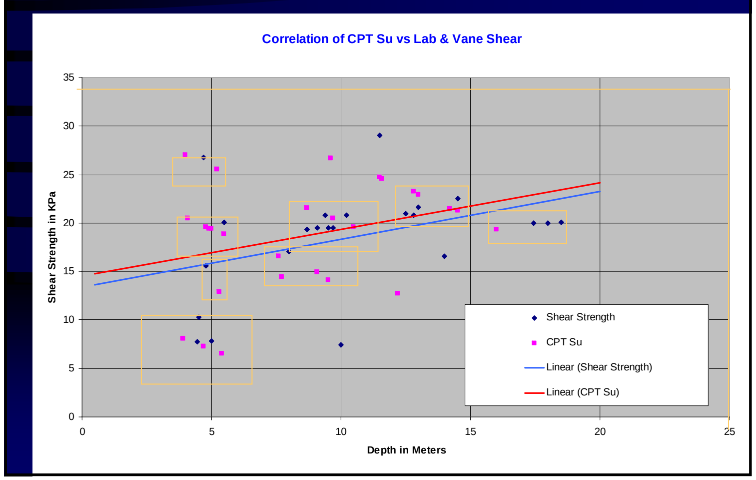
- **Schematic Diagram of Electric CPT/CPTU Testing**
  - The CPT Rod string with the electric CPT Cone is pushed into the soil by a hydraulic push equipment
  - A linear transducer Electrically registers the depth of penetration and stops when the drill string is to be mated with another rod.
  - Strain gages in the Penetrometer (Tip, sleeve and pore pressure)send signals to the Analog to Digital converter to record resistances.
  - The Inclinometer at the rear of the penetrometer registers the inclination of the tip to prevent damage to the instrument.
  - All the electric signals are sent in turn by the analog to digital converter to the computer , where software converts it into real time graphics display vs depth. The computer also stores the results as electronic files for postprocessing at the office or in the field.
  - Software converts the recorded data into soil design parameters by correlation.

The Basic Electronic Components of the System

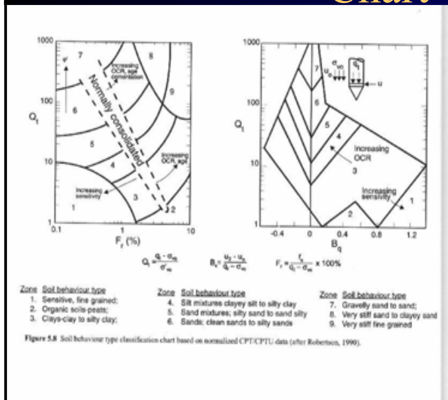


- Electric cone with cable attached .
- Analog to Digital Converter converts electric signals to digital values.
- Laptop computer for real time display of parameters and for postprocessing.

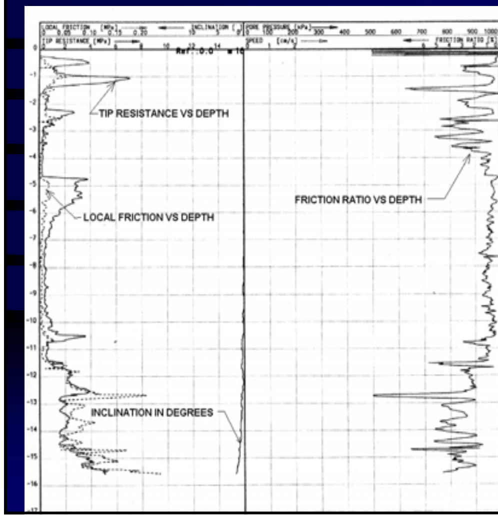
Strength Correlation Plot of CPT Su values vs Lab and Vane Test Shear Strength at Depth of Occurrence.



Soil Characterization Correlation Chart



- The CPT correlation chart by Robertson relates derived parameters from the CPT/CPTU to soil classification based on well established Empirical Correlations.
- The Chart on the left relates normalized cone resistance (Qt) with the Friction Ratio (Fr) to come out with the soil description or classification.
- The Chart on the right does the same by correlating Qt with the pore pressure parameter Bq. Normally this chart is used for very soft clays and very loose sands.



- An actual typical printout of test results of a correlation test done in the Manila Bay Reclamation site .

Correlation of Soil Shear Strength (S_u)

- Correlation Studies were done in Manila Bay Reclamation area to correlate strength properties obtained by the CPT with a number of Lab Unconfined Compression Test results and Field Insitu Vane Shear Tests.

Shear Strength S_u:

The Shear Strength Su is Obtained from the CPT By the formula:

$$Su = \frac{Q_t - \sigma_{vo}}{N_k}$$

Where:

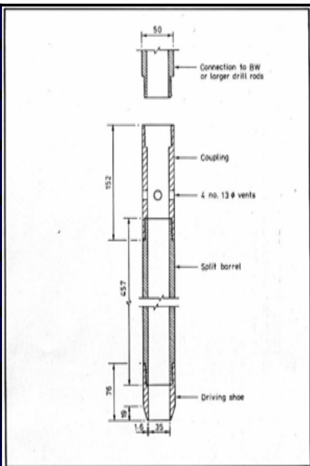
Su= Undrained Shear Strength KPa

Q_t = Normalized CPT Cone Resistance KPa

= (qt- σ_{vo})/ σ'_{vo}

σ_{vo} = Total overburden Stress KPa

N_k = Empirical Cone Factor



Notes : (1) Figure based on BS 1377 (BSI, 1975b).
(2) A slightly enlarged inner diameter of the split barrel is permitted, provided removable liners are always used which have an inside diameter of 35 mm.
(3) A ball valve in the base of the coupling as shown in ASTM (1985a) is also permitted.
(4) All dimensions are in millimetres.

Figure 25 - Split Barrel Sampler for Standard Penetration Test

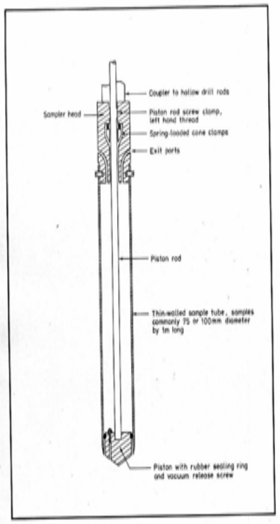
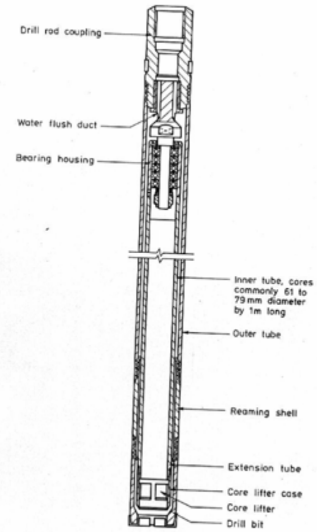
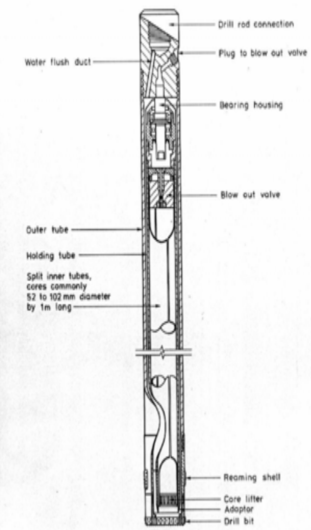


Figure 15 - Thin-walled Stationary Piston Sampler





The End