WEEK 8

ACTIVITY

Lecture (3 hours)
LEARNING OUTCOMES

Week 8 : (3H) Coverage : Field tests, Correlations of field data.

Learning outcomes:

At the end of this lecture/week the students would be able to:

- discuss different field tests to determine bearing capacity and shear strength of soils
- Discuss the geophysical methods of ground investigation
4.1 Introduction to *in situ* testing

4.2 Methods of *in situ* testing and analysis

4.3 Geophysical Methods of Ground Investigation
4.1 Introduction to *in situ* testing

Refers to the procedure of determining soil properties or other subsurface conditions at the actual surface or subsurface location.

*In situ* soil testing includes determination of shear strength, permeability, *in situ* density, plate bearing and settlement, lateral movement, and pore pressure measurement.

In situ shear strengths are determined where undisturbed samples for laboratory testing cannot be obtained or where it is desired to eliminate the need to obtain samples.
Over the years several *in situ* testing devices have emerged to characterize the soil and to measure strength and deformation properties. The most popular devices are:

- Vane shear test (VST)
- Standard penetration test (SPT)
- Cone penetrometer test (CPT)
- Pressuremeter test (PMT)
- Flat Plate dilatometer (DMT)
- JKR/Mackintosh Probe
Boreholes and Probe points

Site Survey using Subsurface Boring Kit

Use of the boring kit can often provide cost effective information without resorting to expensive boring/drilling operations.

BH1, BH2, BH3 = Control Borehole
P1, P2, P3, P4, P5, P6, P7 = Probe Site

5 Kilometres
4.2 Methods of *in situ* Testing

4.2.1 Vane Shear Test (VST)

VST is simple, inexpensive, quick to perform. The vane is pushed, usually from the bottom of a borehole to the desired depth. A torque is applied at a rate of 6° per minute by a torque head device located above the soil surface and attached to the shear vane rod.

Errors in the measurement of the torque include excessive friction, variable rotation and calibration. The VST cannot be used for coarse grained soils and very stiff clays.
Shear vane Apparatus

\[ T = c_u \left\{ \pi h d \times \frac{d}{2} + 2\pi \frac{d^2}{4} \times \frac{d}{3} \right\} \]
4.2.2 Standard Penetration Test (SPT)

The SPT is performed by driving a standard spoon sampler into the ground by blows from a drop hammer of mass 63.5 kg, free falling from 760 mm.
Note how the SPT value and recovery ratios are recorded!!
# Typical Borelog

<table>
<thead>
<tr>
<th>Depth</th>
<th>Description of Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>21.00</td>
<td>Hard pale brown mottled with pale grey clayey SILT with a little of gravels and sand</td>
</tr>
<tr>
<td>23.00</td>
<td>Hard dark grey clayey SILT with a little of gravels</td>
</tr>
<tr>
<td>25.00</td>
<td>Hard pale brown mottled with medium grey clayey SILT with traces of gravels</td>
</tr>
<tr>
<td>26.00</td>
<td>Hard reddish pale brown mottled with medium grey clayey SILT with traces of gravels</td>
</tr>
<tr>
<td>27.00</td>
<td>Hard dark brown mottled with medium grey and red clayey SILT with some fine sand and gravels</td>
</tr>
<tr>
<td>29.00</td>
<td>Hard reddish medium brown clayey SILT with some fine sand and traces of gravels</td>
</tr>
<tr>
<td>30.00</td>
<td>No recovery</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>P = Standard Penetration Test (SPT)</td>
</tr>
<tr>
<td>D = Densified sample</td>
</tr>
<tr>
<td>N = SPT Result</td>
</tr>
<tr>
<td>R = Recovery ratio</td>
</tr>
<tr>
<td>V2 = Vane Shear Test</td>
</tr>
<tr>
<td>US = Unconfined Shear Strength</td>
</tr>
<tr>
<td>SS = Shear Strength</td>
</tr>
<tr>
<td>UD = 30 mm disc undisturbed sample</td>
</tr>
<tr>
<td>P = 5 mm disc undisturbed piston sample</td>
</tr>
<tr>
<td>M = Mass sample</td>
</tr>
<tr>
<td>W = Water sample</td>
</tr>
<tr>
<td>C = Core sample (Rock)</td>
</tr>
<tr>
<td>BOC = Bentonite Overnight (percentage)</td>
</tr>
<tr>
<td>WL = Water level</td>
</tr>
<tr>
<td>BSIS = British Standard Classification System</td>
</tr>
</tbody>
</table>

## Consistency / Relative Density

<table>
<thead>
<tr>
<th>Cohesive Soil (N)</th>
<th>Non-Cohesive Soil (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2 Very Soft</td>
<td>0-4 Very Loose</td>
</tr>
<tr>
<td>2-4 Soft</td>
<td>4-10 Loose</td>
</tr>
<tr>
<td>4-10 Medium Soft</td>
<td>10-20 Medium Dense</td>
</tr>
<tr>
<td>15-30 Very Soft</td>
<td>30-50 Dense</td>
</tr>
<tr>
<td>30-50 Hard</td>
<td>50-60 Very Dense</td>
</tr>
</tbody>
</table>

**Legends:**
- CLAY
- SILT
- SAND
- GRAVEL
- PEAT

**Remarks:**
- Lagged by: 
- (Surveyor) 
- Checked by: 
- (Geologist/Engineer)
Standard Penetration Test

A dynamic test carried out in boreholes during site investigations. Terzaghi and Peck, 1967 provided a chart to determine the allowable bearing capacity. However, the N-values obtained from SI need to be corrected.

Correction due to the presence of water table (Terzaghi and Peck, 1948):

\[
N_{corr.} = 15 + \frac{1}{2}(N - 15)
\]
Standard Penetration Test ..... cont.

Correction with respect to effective overburden stress:

\[ N_{corr.} = C_N N \]

where \( C_N \) = a correction factor

Amongst a number of correction proposals is the chart given by Peck, Hanson and Thornburn (1974) in which:

\[ C_N = 0.77 \log \left( \frac{2000}{\sigma'_o} \right) \]
Correlation of SPT N-values for overburden stress (Peck, Hanson and Thornburn, 1974)
The effect of water table may be taken into account by applying the following correction:

\[
C_w = \frac{1}{2} \left\{ 1 + \frac{D_w}{D + B} \right\}
\]

where

- \(D_w\) = depth of water table below surface
- \(D\) = founding depth below surface
- \(B\) = footing breadth

Thus

\[q_a = C_w q_{TP}\]

Yields conservative values with settlement < 25 mm.

For wide footings and rafts the limiting values may be raised to 50 mm.
Meyerhof (1965) suggested that the $q_{TP}$ values could be increased by 50% and that no correction should be made for the water table since the effect would be incorporated in the measured N-values. He proposed the following simple relationships:

For $B < 1.25$ m:

$$q_a = \frac{s_L N}{1.9}$$

For $B > 1.25$ m:

$$q_a = \frac{s_L N}{2.84} \left[ \frac{B + 0.33}{B} \right]^2$$

For rafts:

$$q_a = \frac{s_L N}{2.84}$$

where $s_L = \text{permitted settlement limit}$

$N = \text{average N-value between } z = D \text{ and } z = D + B^*$

$B = \text{breadth of footing}$
Peck et al. proposed approximate relationships between the N-value and the peak angle of shearing resistance and also the bearing capacity factors.

Notes: 1. $\phi'/N$-value relationship after Peck, et al. (1974)
2. $N_d/\phi'$ and $N_r/\phi'$ relationship from Table 11.2
Allowable bearing capacity of sands

BEARING CAPACITY OF SOILS

Relationship between
N-value and allowable bearing pressure (after Terzaghi and Peck, 1967)

The breadth of the footing and the N-value are used as entry data and the allowable bearing capacity ($q_{TP}$) is read off the left vertical axis.
<table>
<thead>
<tr>
<th>Pile type</th>
<th>Soil type</th>
<th>Ultimate base resistance $q_b$ (kPa)</th>
<th>Ultimate shaft resistance $f_s$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven</td>
<td>Gravelly sand and sand</td>
<td>$40(L/B)N \leq 400 , N$</td>
<td>$2N_{av}$</td>
</tr>
<tr>
<td>Driven</td>
<td>Sand silt and silt (ML)</td>
<td>$30(L/B)N \leq 300 , N$</td>
<td>$2N_{av}$</td>
</tr>
<tr>
<td>Bored</td>
<td>Gravels and sands</td>
<td>$13(L/B)N \leq 130 , N$</td>
<td>$N_{av}$</td>
</tr>
<tr>
<td>Bored</td>
<td>Sandy silt and silt (ML)</td>
<td>$10(L/B)N \leq 100 , N$</td>
<td>$N_{av}$</td>
</tr>
</tbody>
</table>
4.2.3 Cone Penetration test
Cone Penetrometer
CONE PENETRATION TEST
ACCESSORIES & TEST RESULTS

![Diagram showing cone penetration test results with depth below surface and cone resistance values.](image-url)
Cone Penetration Test

The cone is pushed into the soil at a rate of 20 mm/s and the cone resistance ($q_c$) measured as the maximum force recorded during penetration divided by the end area. Although originally developed for the design of piles, the cone penetrometer has been used to estimate the bearing capacity and settlement of foundations.

A compressibility coefficient was suggested by de Beer and Martens (1951):

$$C = \frac{1.5 q_c}{\sigma_o}$$

where

- $q_c =$ cone resistance (MPa)
- $\sigma_o =$ effective overburden pressure (MPa)
Cone Penetration Test … cont.

The settlement $s_i$ at the centre of a layer of thickness $H$ is given by:

$$s_i = \frac{H}{C} \log \left( \frac{\sigma'_o + \Delta q}{\sigma'_o} \right)$$

where $\Delta q = \text{Increase in stress at the centre of the layer due to a foundation pressure } q$

$C = \text{the expression in the previous slide.}$
However the above method is considered to overestimate the value of $s_i$. A rapid method was suggested by Meyerhof (1974).

\[ s_i = \frac{q_n B}{2q_c} \]

where

\[ q_n = \text{net applied loading} = q - \sigma'_o \]

\[ q_c = \text{average cone resistance over a depth below the footing equal to the breadth } B \]
Cone Penetration Test … cont.

- Schmertmann’s method (1970)
- Schmertmann et al.’s method (1978)

Probably the most thorough and reliable method for computing immediate settlement from CPT results.

\[ s_i = C_1 C_2 q_{net} \sum \frac{I_z \Delta z}{E} \]

where:
- \( C_1 = 1 - 0.5(\sigma_o'/q_n) \); for \( q_n < \sigma_o' \), \( C_1 = 0.5 \)
- \( C_2 = 1 + 0.2 \log(10t) \); \( t \) is time in years
  - \( = 1.0 \) for immediately after construction case
- \( I_z \) = vertical strain influence factor
- \( \Delta z \) = thickness of sub layers
- \( E \) = stiffness modulus = 2.5\( q_c \) for square foundation (L/B=1.0)
  - \( = 3.5q_c \) for long foundation (L/B>10)
General conclusion on CPT

CPT is quick to perform with fewer performance error compared with SPT. It can provide continuous records of soil conditions. However it cannot be used in dense, coarse-grained soil and mixed soils containing boulders, cobbles, clays and silt. The cone tip is prone to damage from contact with dense objects.
Conclusion

General conclusion … cont.

Other CPT variants are:

- **Piezocone (uCPT or CPTu)** is a cone penetrometer that has porous elements inserted into the cone or sleeve to allow for pore water measurements.

- **Seismic cone (SCPT)** – geophones are installed inside the cone to record seismic waves resulting from hammer blows on the soil surface which produce surface disturbance. The recorded data are analysed to give damping characteristics and soil strength parameters.
General conclusion … cont.

- **Vision cone (VisCPT or VisCPTu)** have miniature cameras installed in the CPT probe that provide continuous images of the soil adjacent to the cone. Through image processing, the soil texture can be inferred. The VisCPTu can also be used to detect liquefiable soils.
CPT/SPT relationship

Amongst the proposed relationships are:

\[ q_c = 0.4N \ (MPa) \]  
Meyerhof (1956)

\[ q_c = 0.25N \ (MPa) \]  
Meigh and Nixon (1961)  
for silty fine sand

\[ q_c = 1.2N \ (MPa) \]  
Meigh and Nixon (1961)  
for coarse gravel

\[ q_c/N \ vs \ Av. \ grain \ size \ D_{50} \ (MPa) \]  
Burland and Burbidge (1985) – see NEXT SLIDE
Relationship between CPT and SPT (after Burland and Burbidge, 1985)
4.2.4 Pressuremeter test

Read-out device, hydraulic probe and lines conforming with the ASTM Standard Method for Pressuremeter Testing in soils D 4719-87.

A pressuremeter test is an in-situ stress controlled loading test performed on the wall of a borehole using a cylindrical probe which can expand radially. From the test readings a stress-strain curve can be obtained which yields:
- the pressuremeter modulus,
- the creep pressure,
- the limit pressure.

Once the volume and pressure calibrations have been performed.
To assess bearing capacity and settlement of foundations
Menard Pressuremeter

Figure 13-16 Menard-type pressuremeter: Components and Operating Principles.
4.2.5 Flat Plate Dilatometer (DMT)

Consists of tapered blade 95 mm wide and 15 mm thick and 240 mm long. On the flat face the dilatometer is a flexible steel membrane 60 mm in diameter that when inflated pushes the soil laterally. Tests are normally conducted every 200 mm. Results from the test have been related to undrained shear strength, lateral earth pressures, overconsolidation ratios and elastic modulus.

Simple and quick to conduct. Provides reasonable estimates horizontal stress and is less costly than the pressuremeter test.
4.2.6 JKR/Mackintosh Probe

- Can be used to determine the thickness of unsuitable material to be removed and also for preliminary design of embankments
- Limited to about 15 m
- Record no. of blows/ft. then correlate to established chart to determine bearing capacity of soil
## IN SITU TESTING & ANALYSIS

### Comparison between JKR Probe, Mackintosh Probe & SPT

<table>
<thead>
<tr>
<th>Type of Penetrometer</th>
<th>Con</th>
<th>Weight of Hammer (kg)</th>
<th>Height of Fall (mm)</th>
<th>Energy per Unit Area N·m/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>JKR Probe</td>
<td>25</td>
<td>491</td>
<td>5</td>
<td>27972</td>
</tr>
<tr>
<td>Mackintosh Probe</td>
<td>27.9</td>
<td>611</td>
<td>4.5</td>
<td>21675</td>
</tr>
<tr>
<td>SPT</td>
<td>50</td>
<td>1963</td>
<td>65</td>
<td>246874</td>
</tr>
</tbody>
</table>
Comparison of Energy between JKR Probe, Mackintosh Probe & SPT

Ratio of Energy of SPT to JKR Probe = \[\frac{246874}{27972} = 8.8\]

Ratio of Energy of SPT to Mackintosh Probe = \[\frac{246874}{21675} = 11.4\]

If the cone base diameter of Mackintosh probe is 25 mm, then

Ratio of Energy of SPT to Mackintosh Probe = \[\frac{246874}{26979} = 9.2\]
Correlation between JKR Probe, Mackintosh Probe & SPT

(i) JKR Probe’s correlation with SPT-N value

\[ SPT - N \text{ value} = \frac{\text{JKR value}}{8.8} \]

(ii) Mackintosh Probe’s correlation with SPT-N value

\[ SPT - N \text{ value} = \frac{\text{Mackintosh value}}{11.4} \]

(iii) JKR Probe’s correlation with Mackintosh Probe value

\[ 26 \text{ JKR value} = 20 \text{ Mackintosh Probe value} \]
### SPT/JKR or Mackintosh Probe relationship

<table>
<thead>
<tr>
<th>N (Blows/ft)</th>
<th>Consistency</th>
<th>Unconfined Compressive strength (Ton/Sq Ft)</th>
<th>Unconfined Compressive strength (kPa)</th>
<th>JKR or Mackintosh Probe (Blows/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Very soft</td>
<td>0.00 – 0.25</td>
<td>0.0 – 25</td>
<td>0 – 10</td>
</tr>
<tr>
<td>2 – 4</td>
<td>Soft</td>
<td>0.25 – 0.50</td>
<td>25 – 50</td>
<td>10 – 20</td>
</tr>
<tr>
<td>4 - 8</td>
<td>Medium (firm)</td>
<td>0.50 – 1.00</td>
<td>50 – 100</td>
<td>20 – 40</td>
</tr>
<tr>
<td>8 - 15-</td>
<td>Stiff</td>
<td>1.00 – 2.00</td>
<td>100 – 200</td>
<td>40 – 70</td>
</tr>
<tr>
<td>15 - 30</td>
<td>Very stiff</td>
<td>2.00 – 4.00</td>
<td>200 – 400</td>
<td>70 – 100</td>
</tr>
<tr>
<td>Over 30</td>
<td>Hard</td>
<td>4.00</td>
<td>400</td>
<td>100</td>
</tr>
</tbody>
</table>

Relationship between SPT, Mackintosh/JKR probe and unconfined compressive strength of clay.
### SPT/JKR or Mackintosh Probe relationship

<table>
<thead>
<tr>
<th>N (Blows/ft)</th>
<th>Relative density</th>
<th>Allowable soil pressure (Ton/Sq Ft)</th>
<th>Allowable soil pressure (kPa)</th>
<th>JKR or Mackintosh Probe (Blows/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 4</td>
<td>Very loose</td>
<td>Not suitable</td>
<td>Not suitable</td>
<td>0 - 10</td>
</tr>
<tr>
<td>4 - 10</td>
<td>Loose</td>
<td>0.0 – 0.8</td>
<td>0 – 80</td>
<td>10 - 30</td>
</tr>
<tr>
<td>10 - 30</td>
<td>Medium</td>
<td>0.8 – 2.8</td>
<td>80 – 280</td>
<td>30 - 80</td>
</tr>
<tr>
<td>30 - 50</td>
<td>Dense</td>
<td>2.8 – 4.7</td>
<td>280 – 470</td>
<td>80 - 110</td>
</tr>
<tr>
<td>Over 50</td>
<td>Very dense</td>
<td>4.7</td>
<td>470</td>
<td>110</td>
</tr>
</tbody>
</table>

**Note:** 1 Ton/sq ft = 100 kN/m²

**Relationship between SPT, Mackintosh/JKR probe and allowable soil pressure of sand.**
4.3 Geophysical Methods for Ground Investigation

Involve the techniques of determining underground materials by measuring some physical property of the material and, through some correlations, using the obtained values for identifications. Most methods determine conditions over a sizable distance. The methods do not actually measure engineering properties.

Several types can be utilised, namely:

- Seismic refraction method
- Electrical resistivity method
- Ground-penetrating radar
4.3.1 Seismic Refraction Method

Based on the seismic waves travelling through the surrounding soil and rock at speeds relating to the density and bonding characteristics of the material.

The velocity of the seismic waves passing through subsurface soil or rock materials is determined, and the magnitude of the velocity is then utilised to identify the material.
• The mechanical properties of the rocks through which the seismic waves travel quickly organize the waves into two types.

• **Compressional waves**, also known as primary or P-waves, travel fastest, at speeds between 1.5 and 8 kilometers per second in the Earth's crust.

• **Shear waves**, also known as secondary or S-waves, travel more slowly, usually at 60% to 70% of the speed of P-waves.
Compressional or P Wave

Particle Motion

Rarefaction

Compression

Travel Direction

Shear or S Wave

Particle Motion
### Representative Seismic values

<table>
<thead>
<tr>
<th>Material Description</th>
<th>m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most unconsolidated materials</td>
<td>Below 900</td>
</tr>
<tr>
<td>Soil – normal</td>
<td>250 – 450</td>
</tr>
<tr>
<td>- hard-packed</td>
<td>450 – 600</td>
</tr>
<tr>
<td>Water</td>
<td>1500</td>
</tr>
<tr>
<td>Loose sand – above water table</td>
<td>250 – 600</td>
</tr>
<tr>
<td>- below water table</td>
<td>450 – 1200</td>
</tr>
<tr>
<td>Loose mixed sand and gravel, wet</td>
<td>450 – 1100</td>
</tr>
<tr>
<td>Loose gravel, wet</td>
<td>450 – 900</td>
</tr>
<tr>
<td>Hard clay</td>
<td>600 - 1200</td>
</tr>
</tbody>
</table>
### Representative Seismic values

<table>
<thead>
<tr>
<th>Rock – consolidated material</th>
<th>m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most hard rocks</td>
<td>Above 2400</td>
</tr>
<tr>
<td>Shale – soft</td>
<td>1200 - 2100</td>
</tr>
<tr>
<td>- hard</td>
<td>1800 - 3000</td>
</tr>
<tr>
<td>Sandstone – soft</td>
<td>1500 – 2100</td>
</tr>
<tr>
<td>- hard</td>
<td>1800 - 3000</td>
</tr>
<tr>
<td>Limestone – weathered</td>
<td>1200?</td>
</tr>
<tr>
<td>- hard</td>
<td>2400 - 5500</td>
</tr>
<tr>
<td>Basalt</td>
<td>2400 - 4000</td>
</tr>
<tr>
<td>Granite and unweathered gneiss</td>
<td>3000 - 6000</td>
</tr>
<tr>
<td>Compacted glacial tills, hardpan, cemented gravels</td>
<td>1200 - 2100</td>
</tr>
<tr>
<td>Frozen soil</td>
<td>1200 - 2100</td>
</tr>
<tr>
<td>Pure ice</td>
<td>3000 - 3700</td>
</tr>
</tbody>
</table>
4.3.2 Electrical Resistivity Method

Resistivity is a property possessed by all materials.

The method for determining subsurface conditions utilizes the knowledge that in soil and rock materials, the resistance values differ sufficiently to permit that property to be used for identification purposes.
Two different field procedures are used:

*Electrical profiling* used for establishing boundaries between different materials and has practical application in prospecting for sand and gravel deposits or ore deposits.

*Electrical sounding* used to provide information on the variation of subsurface conditions with depth and has practical application in indicating layered conditions and approximate thicknesses.
Concept on electrical resistivity

- Ohm’s Law, $V=IR$, $R=V/I$
- The geometrically independent quantity is called *resistivity*.

*Resistivity is a fundamental parameter of a material and describes how easily a wire or the material can transmit an electrical current.*

*Resistance is a characteristic of a particular path of an electrical current whereas resistivity is a physical property of a material.*
- Electrical Resistivity Method
- Significant in investigating subsurface profile
- Image need to be interpreted
- Geo-material index required
FIELD MEASUREMENT

Electrical Resistivity on Marine Clay Deposit

- Filled Material: 0, 0.9, 1.2, 2.3
- Black soil: 0, 0.9, 1.2, 2.3
- Grey soil

Depth vs. Iteration 3 RMS error = 19.9%

Inverse Model Resistivity Section

<table>
<thead>
<tr>
<th>Resistivity in ohm.m</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.925</td>
</tr>
</tbody>
</table>

Unit electrode spacing 1.00 m
JPS ground water survey for projek tanaman cili Diraja Kelantan

LRAPAM_1

- **Sandstone**
- **Interbeded Sandstone/siltstone**
- **Weathered sandstone**
- **Weathered sandstone/quartzite**
- **Kaolin clay/china clay**
- **Shale**

Inverse Model Resistivity Section

Unit electrode spacing 5.0 m.
<table>
<thead>
<tr>
<th>Types of Materials</th>
<th>Resistivity (ohm-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet-to-moist clayey soils</td>
<td>5 - 10</td>
</tr>
<tr>
<td>Wet-to-moist silty clay and silty soils</td>
<td>10 - 50</td>
</tr>
<tr>
<td>Wet-to-moist silty and sandy soils</td>
<td>50 - 500</td>
</tr>
<tr>
<td>Well-fractured to slightly fractured bedrock with moist soil filled cracks</td>
<td>500 - 1000</td>
</tr>
<tr>
<td>Sand and gravel with silt</td>
<td>1000</td>
</tr>
<tr>
<td>Slightly fractured bedrock with dry soil-filled cracks; sand and gravel with layers of silt</td>
<td>1000 - 8000</td>
</tr>
<tr>
<td>Massive bedded and hard bedrock; coarse dry sand and gravel deposits</td>
<td>8000 +</td>
</tr>
</tbody>
</table>
4.3.3 Ground-penetrating Radar

Also identified as ground-probing radar. Capable of defining the shallow zones of soil and rock materials that underlie an area.

The method relies on the penetration and reflection of high frequency radio waves.
4.1.5 Plate Bearing Test
Plate Bearing Test

Estimated immediate settlement of a foundation (Terzaghi and Peck, 1967) is given by the expression:

\[ s_B = s_b \left( \frac{2B}{B + b} \right)^2 \]

where

- \( s_b \) = settlement of a test plate of side dimension \( b \)
- \( s_B \) = settlement of a foundation of side dimension \( B \) at the same intensity of loading.
Pressure bulbs indicating depth to which soil is significantly stressed – be mindful about misleading prediction based on PBT !!!