INTERPRETATION OF LABORATORY AND FIELD TEST RESULTS FOR DESIGN

by Ir. Dr. Gue See Sew & Ir. Chow Chee Meng

http://www.gnpgroup.com.my
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1. INTRODUCTION
2. OBJECTIVES
3. SCOPE
4. INTERPRETATION
   - JKR PROBE
   - SPT
5. DESIGN PARAMETERS
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INTRODUCTION

NEED

- Neglected topic; only briefly covered in universities
- Danger of using results directly without interpretation
- Decision on choice of values for soil parameters

SCOPE

- Common tests only

PROCESSSES

- Specifications, Supervision, Presentation & Interpretation
<table>
<thead>
<tr>
<th>ت</th>
<th>Ta</th>
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</thead>
<tbody>
<tr>
<td>ت</td>
<td>Tsa</td>
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<tr>
<td>ح</td>
<td>Jim</td>
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<td>ح</td>
<td>Hha</td>
</tr>
</tbody>
</table>
OBJECTIVES

1) Illustrate the importance of interpretation

2) Show methods of compiling results and recognising errors
SCOPE
Common field and laboratory tests

FIELD TESTS
- JKR/ Mackintosh probe
- SPT (Standard Penetration Test)
- Piezocone
- Field Vane Shear
  - Geonor vane

LABORATORY TESTS
- Unconfined compression
- Triaxial Test (UU, CIU with pore pressure measurement & CD)
- Consolidation
FIELD TESTS
JKR Probes

- Primitive tool
- Limited use
  - Shallow bedrock profile (limestone with slump zone)
  - Weak zone at shallow depth
  - Shallow foundation
    - No recent fill and future settlement
    - Structure of low risk
    - If in doubt – use borehole
• **Apparatus**

Cased hardened steel pointer of 25mm dia. and 60° cone.

22mm outer dia. coupling Prevent buckling during driving

12mm dia. HY 55C steel rod

5kg drop hammer
## Cone Penetrometer Comparison

<table>
<thead>
<tr>
<th>Type of Penetrometer</th>
<th>Cone</th>
<th>DIA (mm)</th>
<th>CROSS SECTION (sq cm)</th>
<th>Angle (degree)</th>
<th>DIA of Rod (mm)</th>
<th>DIA of Coupling (mm)</th>
<th>Weight of Hammer (kg)</th>
<th>Height of Fall (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.K.R Probe</td>
<td>CONE</td>
<td>25</td>
<td>5</td>
<td>60</td>
<td>12</td>
<td>22</td>
<td>5</td>
<td>28</td>
</tr>
<tr>
<td>Mackintosh Probe</td>
<td>CONE</td>
<td>25</td>
<td>5</td>
<td>30</td>
<td>13</td>
<td>24</td>
<td>4.5</td>
<td>30</td>
</tr>
</tbody>
</table>

### Diagrams
- **J.K.R Probe:**
  - Cone angle: 60°
  - DIA of rod: 12 mm
  - DIA of coupling: 22 mm
  - Weight of hammer: 5 kg
  - Height of fall: 28 cm

- **Mackintosh Probe:**
  - Cone angle: 30°
  - DIA of rod: 13 mm
  - DIA of coupling: 24 mm
  - Weight of hammer: 4.5 kg
  - Height of fall: 30 cm
For practical application:

- Results of JKR Probe and Mackintosh Probe can be taken as equivalent

- JKR Probe created as equivalent to Mackintosh Probe as Mackintosh Probe is patented in the early days
• **Termination criteria**
  ✓ Blows/300mm  
    (maximum 400 blows/300mm)
  ✓ Max 15m depth

• **Precautionary measures**
  ✓ Free fall and consistent drop height
  ✓ Components and apparatus properly washed and oiled
## Typical Test Results

### Mackintosh Sounding Record Sheet

<table>
<thead>
<tr>
<th>Position No</th>
<th>MP - 1</th>
<th>MP - 2</th>
<th>MP - 3</th>
<th>MP - 4</th>
<th>MP - 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chainage</td>
<td>22.340M</td>
<td>22.320M</td>
<td>22.322M</td>
<td>22.172M</td>
<td>21.128M</td>
</tr>
<tr>
<td>R. Level</td>
<td>45.489M</td>
<td>46.529M</td>
<td>47.779M</td>
<td>63.618M</td>
<td>64.823M</td>
</tr>
<tr>
<td>Depth (M)</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0 - 0.3</td>
<td>13</td>
<td>30</td>
<td>31</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>0.3 - 0.6</td>
<td>10</td>
<td>71</td>
<td>39</td>
<td>19</td>
<td>16</td>
</tr>
<tr>
<td>0.6 - 0.9</td>
<td>9</td>
<td>52</td>
<td>64</td>
<td>15</td>
<td>21</td>
</tr>
<tr>
<td>0.9 - 1.2</td>
<td>18</td>
<td>26</td>
<td>48</td>
<td>16</td>
<td>26</td>
</tr>
<tr>
<td>1.2 - 1.5</td>
<td>12</td>
<td>*53</td>
<td>*72</td>
<td>21</td>
<td>35</td>
</tr>
<tr>
<td>1.5 - 1.8</td>
<td>14</td>
<td>64</td>
<td>96</td>
<td>33</td>
<td>26</td>
</tr>
<tr>
<td>1.8 - 2.1</td>
<td>13</td>
<td>105</td>
<td>60</td>
<td>27</td>
<td>32</td>
</tr>
<tr>
<td>2.1 - 2.4</td>
<td>92</td>
<td>81</td>
<td>73</td>
<td>21</td>
<td>30</td>
</tr>
<tr>
<td>2.4 - 2.7</td>
<td>95</td>
<td>77</td>
<td>103</td>
<td>28</td>
<td>35</td>
</tr>
<tr>
<td>2.7 - 3.0</td>
<td>90</td>
<td>33</td>
<td>110</td>
<td>19</td>
<td>25</td>
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<tr>
<td>3.0 - 3.3</td>
<td>52</td>
<td>220</td>
<td>200</td>
<td>20</td>
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<td>3.3 - 3.6</td>
<td>50</td>
<td></td>
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<td>3.6 - 3.9</td>
<td>59</td>
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<td>3.9 - 4.2</td>
<td>150</td>
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<td>4.2 - 4.5</td>
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<td>4.5 - 4.8</td>
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<td>4.8 - 5.1</td>
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<td>5.1 - 5.4</td>
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<td>5.4 - 5.7</td>
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<td>5.7 - 6.0</td>
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<tr>
<td>6.0 - 6.3</td>
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<td>6.3 - 6.6</td>
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<td>6.6 - 6.9</td>
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<tr>
<td>6.9 - 7.2</td>
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<td>7.2 - 7.5</td>
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<td>7.5 - 7.8</td>
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<td>7.8 - 8.1</td>
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<td>8.1 - 8.4</td>
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<td>8.4 - 8.7</td>
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<td>8.7 - 9.0</td>
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<td>9.0 - 9.3</td>
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<td>9.3 - 9.6</td>
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<td>9.6 - 9.9</td>
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<td>9.9 - 10.2</td>
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<tr>
<td>10.2 - 10.5</td>
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<td>10.5 - 10.8</td>
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<tr>
<td>10.8 - 11.1</td>
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<td>11.1 - 11.4</td>
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<td>11.4 - 11.7</td>
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<tr>
<td>11.7 - 12.0</td>
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<tr>
<td>12.0 - 12.3</td>
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<td>12.3 - 12.6</td>
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<tr>
<td>12.6 - 12.9</td>
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</tbody>
</table>

**Notes:**
- Depth of water = 0.15m
- Platform = 0.20m
- Rebound
- Depth of water = 0.40m
- Platform = 0.15m
- Depth of water = 0.40m
- Platform = 0.20m
Identifying localised soft/weak or slip plane.
• **Applications**

Identifying localised soft/weak or slip plane.
Identifying non-compliance fill.

\[ T = \text{compaction lift} \]
Allowable Bearing Capacity V.S. J.K.R. Dynamic Cone Penetration Resistance (After Ooi & Ting, 1975) **Conditions applied**
### Comparison between JKR probe and SPT

<table>
<thead>
<tr>
<th>Type of Penetrometer</th>
<th>Cone</th>
<th>Weight of hammer (kg)</th>
<th>Height of fall (mm)</th>
<th>Energy per unit area Nm/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dia (mm)</td>
<td>Area (mm²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>JKR probe</td>
<td>25</td>
<td>491</td>
<td>5</td>
<td>27970</td>
</tr>
<tr>
<td>SPT</td>
<td>50</td>
<td>1963</td>
<td>65</td>
<td>246736</td>
</tr>
</tbody>
</table>

**Ratio of Energy of SPT to JKR Probe**

\[
\frac{246,736}{27970} = 8.8
\]
Number of Blows per 300 mm

Depth From Ground Surface in Meter (m)

JP1A + JP1B

JP2A & JP2B

JP3

0.5m

0.25m
• **Limitations**

- Shallow depth
- Not for gravelly ground
- Human errors (e.g. wrong counting, non-consistent drop height, exerting force to the drop hammer)
- Misleading results at greater depth
Standard Penetration Test (SPT)

A popular test

- useful for pile foundation design

Common errors

Factors affecting SPT (N) values

- Inadequate cleaning of borehole
- Casing driven bottom of borehole
- Damage tip of sampling spoons
- Loose joints on connecting rods
- Not using guide rod
- Water level in borehole below ground water level
- Free fall not attainable

- N, sludge trapped in sampler
- N in sand
- N in clay
- N
- N, eccentric blows
- N esp sand at bottom of borehole, pipiing effect
63.5kg Hammer

760mm Free Fall

450mm

Split-Spoon Sampler

AW Rod
Split-Spoon Sampler

- **Driving Shoe**
- **Split Barrel**

- **OD = 50mm**
- **ID = 35mm**
- **Length ~ 650mm**
### SPT-N Value

<table>
<thead>
<tr>
<th>Penetration (mm)</th>
<th>75</th>
<th>75</th>
<th>75</th>
<th>75</th>
<th>75</th>
<th>75</th>
<th>N-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blow counts</td>
<td>1</td>
<td>3</td>
<td>5</td>
<td>7</td>
<td>7</td>
<td>9</td>
<td>28</td>
</tr>
</tbody>
</table>

- Seating drive
- Test drive
SPT-N = \( \frac{(30 + 20)}{(75 + 30)} \times 300 = 143 \)
### Maximum blows to be applied

<table>
<thead>
<tr>
<th></th>
<th>In seating drive</th>
<th>In test drive</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>‘Soft rock’</td>
<td>25</td>
<td>100</td>
</tr>
</tbody>
</table>

**BS1377: Part 9**
### Deep Boring Log

**Borehole No:** DB/1  
**Reduced Level:** 11.02 H (m)  
**Type of Unit:** After 1  
**Date:** 19.02.35  
**Sheet No.:** 1 of 2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Soil</th>
<th>Sample</th>
<th>Field Test</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>Dark grey graded medium to fine gravelly sand with organic matter.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.31</td>
<td>Light grey graded medium to fine gravelly sand.</td>
<td>15m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.35</td>
<td>Light grey graded medium to fine gravelly sand. &amp; organic matter.</td>
<td>19m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.50</td>
<td>Greenish grey graded medium to fine gravelly sand.</td>
<td>30m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.50</td>
<td>Greenish grey graded medium to fine gravelly sand.</td>
<td>50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.50</td>
<td>Greenish grey graded medium to fine gravelly sand. &amp; organic matter.</td>
<td>47m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00</td>
<td>Greenish grey graded coarse to fine gravelly sand. with silt clay and mull.</td>
<td>60m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.00</td>
<td>Greenish grey graded coarse to fine gravelly sand. with silt clay &amp; mull.</td>
<td>64m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.00</td>
<td>Greenish grey very soft sandy mud extended to coarse sandy mud.</td>
<td>75m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.00</td>
<td>Recovered core (m)</td>
<td>50</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- R = Standard Penetration Test (SPT)  
- V = Water level (W.L.)  
- P = SPT sample  
- D = Davy sample  
- Y' = Yield stress & Test  
- W = Water sample  
- C = Core sample  
- M = Rock Quality Designation  

**G&P Geotechnics Sdn Bhd**
SPT N (blows/ft)

DEPTH (m)
New Technology...automatic
Piezocone (CPTu)

1) To obtain soil profile and stiffness (strength) profile of the subsoil

2) To determine coefficient of consolidation of soil

3) Results can also be used directly for design (e.g. pile design)
Correlation with Undrained Shear

Shear ($s_u$)

\[ s_u = \frac{q_c - \sigma_{vo}}{N_k} \]

\[ N_k = 11-19 \]

Lunne & Kleven (1981)

or

\[ s_u = \frac{q_T - \sigma_{vo}}{N_{kt}} \]

\[ N_{kt} = 15 \]

Gue & Tan (2000)
SUNGAI DUA

- upper clay
- lower clay

JURU
- upper clay

ALOR PONGSU

- upper clay
- lower clay

S_{uv} - VANE SHEAR STRENGTH (kPa)

Relationship of corrected cone resistance to vane shear strength for clays in the data base
(After Dobie & Wong, 1990)
Cone factors related to plasticity index of the clays (After Dobie & Wong, 1990)
1. Sensitive, fine grained
2. Organic soils – peats
3. Clays – clay to silty clay
4. Silt mixtures – clayey silt to silty clay
5. Sand mixtures – silty sand to sandy silt
6. Sands – clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand (heavily overconsolidated or cemented)
9. Very stiff fine grained (heavily overconsolidated or cemented)
Dissipation Test
(C.I. Teh & G. T. Houlsby, 1991, Geotechnique No.41)

\[ T^* = \frac{C_h t}{a^2 \sqrt{l_r}} \quad \text{where } l_r = \frac{G}{S_u} \]

\[ a = \text{radius of cone} \]

\[ T^* = \text{modified time factors} \]

**Modified Time Factors, \( T^* \)**

<table>
<thead>
<tr>
<th>Degree of Consolidation</th>
<th>Cone Shoulder</th>
</tr>
</thead>
<tbody>
<tr>
<td>20%</td>
<td>0.038</td>
</tr>
<tr>
<td>30%</td>
<td>0.078</td>
</tr>
<tr>
<td>40%</td>
<td>0.142</td>
</tr>
<tr>
<td>50%</td>
<td>0.245</td>
</tr>
<tr>
<td>60%</td>
<td>0.439</td>
</tr>
<tr>
<td>70%</td>
<td>0.804</td>
</tr>
<tr>
<td>80%</td>
<td>1.60</td>
</tr>
</tbody>
</table>
Normalised dissipation curves plotted against $T^*$

$I_v$ values: 25 to 500
**Vane Shear Test**

1) Vane test in borehole

2) Geonor vane

3) Lab vane

*Use*  
- To determine in-situ undrained shear strength \((S_{uv})\) of soft clayey soils
Representative results of field vane tests using a 130mm x 65mm Geonor vane

Legend:

- + Embankment built to failure
- Trial embankments
- Peak Remoulded

FIELD VANE SHEAR STRENGTH (kPa)

DEPTH (mRL)
Most common errors
- Computation – spring factor
- Clay with organic materials

Recognise errors

Summarise results with $S_u$ from unconfined compression, UU and lab vane superimposed

Plot $S_{uv}$ against PI

Or $S_{uv}$ against $P_o'$ then find $S_{uv}/P_o'$
Relationship between $S_u/p'$ and Plasticity index, after Bjerrum and Simons (1960)
Wₙ - Natural Water Content
Wₗ - Liquid Limit
Wₚ - Plastic Limit

Field Vane Strength (kN/m²)
Sensitivity
OCR
C₀ / (k+e₀)
Cᵣ / (k+e₀)

Description of Soil

Weathered Clay

Very Soft Silty Clay

Soft Silty Clay

Organic Clay

Medium dense to dense Clayey Silty Sand

G&P Geotechnics Sdn Bhd
Design Parameters

Foundation Design
- Stability / Bearing Capacity
- Settlement Prediction

Bearing Capacity
- $S_u$
- $C'$ and $\phi'$

Settlement Prediction
- $e$ vs $\log_{10} p'$ ($m_v$, $C_c$)
- $c_v$ ($k$)
LABORATORY TESTS
LABORATORY TESTS

- Why?
- Types of Tests!
- How?
- Specifications?
  (Load, Pressure, Time)
SPECIFICATIONS

A) Consolidation Test

1) Which samples are appropriate and suitable for the test?
2) For consolidation test
   - Load increment \( \{ 0.5P_o \prime - 8P_o \prime \} \)
   - Pressure \( \{ e \approx 0.42e_o \} \)

B) Triaxial test

1) For triaxial tests
   - Strain rate
   - Back pressure


G&P Geotechnics Sdn Bhd
Special Attention

Triaxial Compression Test
- No/Minimum Trimming
- No Side Drains
- No Multistage
LABORATORY TEST SCHEDULE

<table>
<thead>
<tr>
<th>BOREHOLE SAMPLE NO.</th>
<th>DEPTH m</th>
<th>M/C</th>
<th>A.L.</th>
<th>B.D.</th>
<th>S.G.</th>
<th>Direct Shear Box</th>
<th>SIEVE ANALYSIS Mech. Hydro. Std. Rapid S.S.</th>
<th>CONSOLIDATION CIU UU UCT</th>
<th>TRIAXIAL ORGANIC CONTENT PH SULPHATE CONTENT CHLORIDE CONTENT</th>
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</table>

TOTAL Requested

Performed

Note:
1) CIU - Isotropic Consolidated Undrained Triaxial Test with pore pressure measurements
   - Use 70mm diameter sample (i.e. untrimmed Mazier sample)
   - Sample should not have side filter during consolidation
   - Shearing strain should be calculated using C_v values calculated during consolidation stage.
   - Multi-stage testing not allowed
   - P-Q Stress Path Plotting shall be submitted.
2) For CIU Tests, stress path and other relevant data shall be submitted in Hard Copy (Plots and Tabulated Data) and Soft Copy (Computer files data). Cell confining pressure of 0.5 \( \sigma_v \), 1.0 \( \sigma_v \), 2.0 \( \sigma_v \) shall be adopted for the shear box test, where \( \sigma_v \) is the total vertical in-situ stress.
3) UU - Unconsolidated Undrained Test (at total overburden pressure of the sample)
4) UCT - Unconfined Compression Test (untrimmed sample)
5) To determine \( C_v \) from Consolidation Tests -
   - Use Square-Root Time Method to determine \( d_0 \).
   - Then use Log-Time Method to determine \( d_{100} \).
6) Direct shear box test - Three (3) reconstituted specimens (60mm x 60mm x 20mm thick) shall be used.
   - Applied normal stress pressure of 0.5 \( \sigma_v \), 1.0 \( \sigma_v \), 2.0 \( \sigma_v \) shall be adopted for the shear box test, where \( \sigma_v \) is the total vertical in-situ stress.
7) All specimens for triaxial or consolidation tests shall be obtained from center of the recovered samples in UD sampler.
8) 2 moisture content tests shall be carried out on soil immediately besides the specimens retained for triaxial or consolidation tests.
9) Bulk density, particle size distribution and Atterberg Limit tests shall be carried out on every specimen after the triaxial or consolidation tests.
Test to obtain Consolidation Parameters

(Direct)

Oedometer Test

e vs log_{10} P' (Mv, c_c)

c_v (k)
Consolidation Settlement

![Consolidation Settlement Diagram]

- $\sigma'_{vo}$
- $\sigma'_{vc}$
- $\sigma'_{vc}$ or $\sigma'_{vm}$ (Casagrande)
- Recompression Index, $Cr$
- Minimum Radius
- Compression Index, $Cc$
- Swelling Curve
- Swelling Index, $Cs$
- $\sigma'_{vf}$

Effective Pressure (log scale), $\sigma'_v$

Void Ratio, $e$

$e_0$, $e_c$, $e_f$
CONSOLIDATION TEST RESULTS

- Void Ratio
- Cv m²/year
- Coefficient of Volume Change
- $M_v \times 10^{-3} \text{ m}^2 / \text{KN}$

---

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---

**Table:**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>A81 - A1</th>
<th>Liquid Limit</th>
<th>106</th>
<th>Sample Height</th>
<th>cm</th>
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<tbody>
<tr>
<td>Depth:</td>
<td>9.80 m</td>
<td>Plastic Index</td>
<td>62</td>
<td>Water Content</td>
<td>%</td>
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<tr>
<td>Test No:</td>
<td>PS 2</td>
<td>Specific Gravity</td>
<td>2.580 (Assumed)</td>
<td>Dry Unit Weight</td>
<td>Mg/m³</td>
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<tr>
<td>Soil Type:</td>
<td>0, 0</td>
<td>Precons. Pressure</td>
<td>80 KN/m³</td>
<td>Void Ratio</td>
<td>3,003</td>
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<tr>
<td>Diameter:</td>
<td>5,000 cm</td>
<td>$C_v = 2.113$</td>
<td></td>
<td>Saturation</td>
<td>%</td>
</tr>
</tbody>
</table>

Initial | Final
---|---
2,000 | 1,439
110 | 73
0.64 | 0.89
94 | 100
Compression Index
Coefficient of Consolidation, Ch m²/yr

Depth (m)

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Coefficient of consolidation versus mean consolidation pressure (After Raman et al., 1990)
**COEFFICIENT OF CONSOLIDATION VS LIQUID LIMIT**

**UNDISTURBED SAMPLES:**
- $C_v$ in range of virgin compression
- $C_v$ in range of recompression lies above this lower limit

**COMPLETELY REMOLDED SAMPLES:**
- $C_v$ lies below this upper limit

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NAVFA C DM7.1
Log time method

Fig. 7.20 The log time method.

Root time method

Fig. 7.21 The root time method.
Compression index, $C_c$ and Recompression index, $C_r$

a) $C_c = 0.009 \ (LL - 10\%)$  
   For inorganic soils, with sensitivity less than 4

b) $C_c = 0.007 \ (LL - 10\%)$  
   For normally consolidated clay

c) $C_c = 0.0115 \ W_n$  
   For organic soils, peat

d) $C_c = 1.15 \ (e_o - 0.35)$  
   For all clays

e) $C_c = (1 + e_o) \ [0.1 + (W_n - 25)0.006]$  
   For varved clays

f) $C_c = 0.5*\pi*G_s$  
   For OC clays
Compression index, $C_c$ and Recompression index, $C_r$

- For inorganic normally consolidated Klang Clay (Tan et al., 2004):
  - $C_c = 0.02LL - 0.87$
  - $C_c = 0.61e_o - 0.17$
  - $C_c = 0.02 W_n - 0.37$

- $C_r \approx (0.1 \text{ to } 0.2) * C_c$
Coefficient of secondary compression, $C_\alpha$

- $C_\alpha / C_c = 0.04 \pm 0.01$  \textit{For inorganic soft clays}

- $C_\alpha / C_c = 0.02 \pm 0.01$  \textit{For granular soils including rockfill}

- $C_\alpha / C_c = 0.03 \pm 0.01$  \textit{For shale and mudstone}

- $C_\alpha / C_c = 0.05 \pm 0.01$  \textit{For organic clays and silts}

- $C_\alpha / C_c = 0.06 \pm 0.01$  \textit{For peat and muskeg}
Interpretation of Laboratory Tests

TWO Major Categories:

(1) **Strength Parameters**:
   - Stability Analyses of Slopes & Embankment.
   - Bearing Capacity Analyses for Foundation.

(2) **Stiffness & Deformation Parameters**:
    Prediction & evaluation of:
    - Settlement, Heave, Lateral deformation,
    - Volume Change.
Low Rise Buildings (Link Houses)

Conventional Pile System

Pile Strip/Raft System

25m to 30m
Very Soft to Soft Compressible Clay
($S_u = 15$ to $45$ kPa)

Medium Stiff to Stiff Clayey SILT with Sand
($SPT'N = 5$ to $35$ Blows/ft)

Hard Stratum
($SPT'N \geq 60$ Blows/ft)
Conventional Foundation for Low Rise Buildings

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Conventional Foundation for Low Rise Buildings (Soil Settlement)

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Exposed Pile Settlement

Settling Platform Detached from Building
Strength Parameters

TWO Conditions:

(A) Total Stress:
- For Short Term Conditions in Cohesive Soils.
- Little of no drainage.

(B) Effective Stress:
- For Long Term & Permanent Conditions.
- Fully “Drained” Conditions.
Simple Check

\[ q_{\text{allow}} = \left( \frac{N_c \cdot S_u}{\text{FOS}} \right) \]

- \( q_{\text{allow}} = \) allowable bearing pressure
- \( = (\gamma_{\text{fill}} \cdot H + 10) \) (in kPa)

- \( N_c = 5 \)

\[ H_{\text{failure}} = \frac{(5 \times S_u)}{\gamma_{\text{fill}}} \]

e.g.:

When

- \( S_u = 10 \text{ kPa} \)
- \( \gamma_{\text{fill}} = 18 \text{ kN/m}^3 \)

\[ H_{\text{failure}} = \frac{(5 \times 10)}{18} = 2.8 \text{ m} \]
Excavation: Check Depth of Excavation

\[ \begin{align*}
\text{H/B} > 1: & \\
FOS_{\text{base}} & \text{by Bjerrum & Eide (1956)} \\
& = \frac{1}{\gamma} \frac{c_u N_c}{H} \\
N_c & = f(H/B) \\
\text{H/B} < 1: & \\
FOS_{\text{base}} & \text{by Terzaghi (1943)} \\
D & < (\sqrt{2}/2) B \\
& = \frac{1}{\gamma} \frac{N_c c_{u,2}}{H} \\
& - c_{u,1}/D \\
D & > (\sqrt{2}/2) B \\
FOS_{\text{base}} & = \frac{1}{\gamma} \frac{N_c c_{u,2}}{H} \\
& - \frac{2 c_{u,1}}{\sqrt{2} B} \\
\end{align*} \]
Total Stress Strength, $s_u$

Undrained Shear Strength, $s_u$ from:

(i) Unconfined Compression Test, UCT

(ii) Unconsolidated Undrained Triaxial Test, UU

(iii) Laboratory Vane Shear Test
Typical Set-up of Triaxial Test

a) Base
b) Removable cylinder and top cap
c) Loading ram
d) Rubber membrane
Effective Stress Strength

Parameters $c'$ & $\phi'$ → Interpretation from

(i) Isotropic Consolidated Undrained Triaxial Test, CIU + $\Delta U$

(ii) Isotropic Consolidated Drained Triaxial Test, CID

(iii) Laboratory Shear Box Test (at v. slow rate)

Note: Advantage to use Stress Path
Mohr-Coulomb
Stress Path Interpretation

Two types of Plot

(i) MIT Stress Path Plot (T.W. Lambe of MIT, 1967)

The vertical axis:
\[ t = (\sigma_1 - \sigma_3)/2 = (\sigma'_1 - \sigma'_3)/2 \]

The horizontal axis:
\[ s = (\sigma_1 + \sigma_3)/2 \quad \& \quad s' = (\sigma'_1 + \sigma'_3)/2 \]

(ii) Cambridge Stress Path Plot

(Roscoe, Schofield and Wroth (1958) at the Cambridge, England)

The vertical axis:
\[ q = \sigma_1 - \sigma_3 = \sigma'_1 - \sigma'_3 \]

The horizontal axis:
\[ p = (\sigma_1 + \sigma_2 + \sigma_3)/3 \quad \& \quad p' = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3 \]

Terminology & Interpretation
MIT & Cambridge Stress Path Plot
\[
\begin{align*}
\tan \theta &= \frac{t'}{s} \\
\tan \theta &= \sin \phi' \\
K &= c' \cos \phi' \\
C' &= \frac{K}{\cos \phi'} \\
\tan \eta &= \frac{q}{p'} \\
\sin \phi' &= \frac{3\eta}{6 + \eta} \\
r &= c' \frac{6 \cos \phi'}{3 - \sin \phi'} \\
C' &= \frac{r(3 - \sin \phi')}{6 \cos \phi'}
\end{align*}
\]
For Slopes & Walls Analyses
Parameter $c'$ and $\phi'$ shall be interpreted from

i) Isotropically Consolidated Undrained Triaxial Test, CIU + $\Delta u$

ii) Isotropically Consolidated Drained Triaxial Test, CID

iii) Laboratory Shear Box Test (at very slow rate)

Note: Advantage to use Stress Path
Large Strain
10% to 30%

Mohr-Coulomb indicate higher strength than actual

Stress path from Point A (consolidation pressure) to Point B (Peak Strength) to Point C (Critical State).
$s' = \frac{\sigma_1' + \sigma_3'}{2}$

$\phi' = \sin^{-1} m$

$c' = \frac{a}{\cos \phi'}$

**Upper Bound**
- $c' = 5 \text{ kPa}$,
- $\phi' = 39^\circ$

**Lower Bound**
- $c' = 0 \text{ kPa}$,
- $\phi' = 29^\circ$

**Proposed Design Line**
- $c' = 3.5 \text{ kPa}$,
- $\phi' = 32^\circ$
Correlations for Preliminary Assessment of $\phi'$
φ' Values vs Plasticity Index (after Terzaghi)

Typical PI = 30% to 70%
Φ' Values vs Clay Content (Skempton, 1964)
Figure 3: Peak versus Percentage of Fines in Residual Soils
Figure 4: $c'$ versus Percentage of Fines in Residual Soils
Total Stress:
- $C = 25$ kN/m$^2$
- $\phi = 17$ deg

Effective Stress:
- $C' = 6$ kN/m$^2$
- $\phi' = 33$ deg

LEGEND:
- = Total Stress
- = Effective Stress
$C$ = Apparent Cohesion
$\phi$ = Angle of shear Resistance
$C'$ = Cohesion Intercept Based On Effective Syress
$\phi'$ = Angle of Shear Resistance Based On Effective Stress
Correct Interpretation

\[ \tau \]

\[ \phi, \ c \ - \ incorrect \ interpretation \]

\[ S_u \ 1 \ - \ correct \ interpretation \]

\[ S_u \ 2 \ - \ correct \ interpretation \]

\[ S_u \ 3 \ - \ correct \ interpretation \]
Undrained Shear Strength

- **Limitations of UU Tests:**
  - Sample disturbance
  - Negative pore pressures generated during removal of sample from tube

- Undrained shear strength is best obtained from in-situ testing such as field vane, piezocone, etc.
YOU PAY FOR SOIL INVESTIGATION WHETHER YOU CARRY OUT OR NOT
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