Class

Principles of Foundation Engineering
CEE430/530
General Information

Lecturer: Scott A. Barnhill, P.E.
Lecture Time: Thursday, 7:10 pm to 9:50 pm
Classroom: Kaufmann, Room 224
Office Hour: I have no office. Contact me to meet before class.
Email: sabarnhill@geronline.com (ODU email not working)
Cell Phone: 621-6783


**NOT the SI Version**
Class Web Site

http://www.geronline.com/odu.php

• Class Dates
• Information to Be Covered
• Homework Answers
• Any Handouts
• Other Reading Sources
Homework

Homework is to have an answer sheet as the first page.
• Problem Number
• Answers with any sub problems answers also shown.

Show data given at top of each problem.
Show every calculation step.
No Excel or MathCad – Period!!!
Grading

- Homework - 20%
- Mid Term Exam - 30%
- Final Exam - 30%
- Project - 20%

Class Presentation – 10%
Report – 10%
Your Personal Calculations – 20%
Peer Review – 50%
My Opinion – 10%

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<thead>
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<th>Range</th>
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Purpose of the Class

- Familiarize you with soil properties
- Learn how subsurface soils are tested
- Learn how to apply soil properties to foundation design
- Learn about analyzing various foundation alternatives
- Learn about retaining walls
- Learn about how to improve the ground
- Apply what you have learned to actual projects
Chapter 1: Geotechnical Properties of Soil
Your Knowledge

You are already suppose to know everything covered in this chapter.
Strength

- What is the strength of steel?
- What is the strength of concrete?
- What if their strengths varied wildly even with a single column or beam.
- How would you design?
Soil

- What is the strength of soil?
Strength of Soil

The graphs illustrate the relationship between undrained shear strength (tsf) and elevation (feet) or depth (feet) for different CPT tests (CPT-1 to CPT-6). The data points are color-coded for each test, and a line indicating the geomean is also shown. The graphs help in assessing the variability and trends in soil strength at different depths and elevations.
Lesner Bridge
## Classification Schemes

### Table 1.2 Soil-Separate Size Limits

<table>
<thead>
<tr>
<th>Classification system</th>
<th>Grain size (mm)</th>
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<tr>
<td>Unified</td>
<td>Gravel: 75 mm to 4.75 mm&lt;br&gt;Sand: 4.75 mm to 0.075 mm&lt;br&gt;Silt and clay (fines): &lt;0.075 mm</td>
</tr>
<tr>
<td>AASHTO</td>
<td>Gravel: 75 mm to 2 mm&lt;br&gt;Sand: 2 mm to 0.05 mm&lt;br&gt;Silt: 0.05 mm to 0.002 mm&lt;br&gt;Clay: &lt;0.002 mm</td>
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Weight-Volume

Figure 1.3 Weight–volume relationships
# Unit Weight Relationships

<table>
<thead>
<tr>
<th>Unit-weight relationship</th>
<th>Dry unit weight</th>
<th>Saturated unit weight</th>
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<tbody>
<tr>
<td>$\gamma = \frac{(1 + w)G_s \gamma_w}{1 + e}$</td>
<td>$\gamma_d = \frac{\gamma}{1 + w}$</td>
<td>$\gamma_{sat} = \frac{(G_s + e) \gamma_w}{1 + e}$</td>
</tr>
<tr>
<td>$\gamma = \frac{(G_s + Se) \gamma_w}{1 + e}$</td>
<td>$\gamma_d = \frac{G_s \gamma_w}{1 + e}$</td>
<td>$\gamma_{sat} = \left[ (1 - n)G_s + n \right] \gamma_w$</td>
</tr>
<tr>
<td>$\gamma = \frac{(1 + w)G_s \gamma_w}{1 + \frac{wG_s}{S}}$</td>
<td>$\gamma_d = G_s \gamma_w (1 - n)$</td>
<td>$\gamma_{sat} = \left( \frac{1 + w}{1 + wG_s/S} \right) G_s \gamma_w$</td>
</tr>
<tr>
<td>$\gamma = G_s \gamma_w (1 - n)(1 + w)$</td>
<td>$\gamma_d = \frac{G_s \gamma_w}{1 + \frac{wG_s}{S}}$</td>
<td>$\gamma_{sat} = \left( \frac{e}{w} \right) \left( \frac{1 + w}{1 + e} \right) \gamma_w$</td>
</tr>
<tr>
<td>$\gamma_{sat} = \gamma_d + n \gamma_w$</td>
<td>$\gamma_{sat} = \gamma_d + \left( \frac{e}{1 + e} \right) \gamma_w$</td>
<td>$\gamma_{sat} = \gamma_d + n \gamma_w$</td>
</tr>
</tbody>
</table>

### Table 1.3 Various Forms of Relationships for $\gamma$, $\gamma_d$, and $\gamma_{sat}$
Common Relationships

\[ W = W_s (1 + w) \]

\[ V = V_s (1 + e) \]

\[ \gamma_d = \frac{\gamma_m}{1 + w} \]

\[ \gamma' = \gamma_{sat} - \gamma_w \]

\[ \gamma_d = \frac{G_s \cdot \gamma_w}{1 + e} \]

\[ \gamma_{sat} = \gamma_d (1 + w_{sat}) \]

\[ e = \frac{n}{1 - n} \]

\[ n = \frac{e}{l + e} \]

\[ \frac{w \cdot G_s}{S} \]

\[ S = \frac{w_{sat}}{w} \]

\[ w_{sat} = \frac{\gamma_w}{\gamma_d} - \frac{1}{G_s} \]

W – Total Weight
V – Total Volume
w – water content
Ws – Weight of Solids
Vs – Volume of Solids
e – void ratio
\( \gamma_d \) – Dry Unit Weight
\( \gamma_m \) – Moist Unit Weight
\( \gamma_w \) – Unit Weight of Water
\( \gamma' \) – Bouyant Unit Weight
Gs – Specific Gravity
n – porosity
S – Saturation
w_{sat} – Saturated Moisture Content
Specific Gravity

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<th>Type of soil</th>
<th>$G_s$</th>
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<tr>
<td>Quartz sand</td>
<td>2.64–2.66</td>
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<tr>
<td>Silt</td>
<td>2.67–2.73</td>
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<tr>
<td>Clay</td>
<td>2.70–2.9</td>
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<tr>
<td>Chalk</td>
<td>2.60–2.75</td>
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<tr>
<td>Loess</td>
<td>2.65–2.73</td>
</tr>
<tr>
<td>Peat</td>
<td>1.30–1.9</td>
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</tbody>
</table>
Atterberg Limits

Liquid Limit (LL), Plastic Limit (PL) & Plasticity Index (PI) = LL - PL

Soils at the plastic limit are 100 times stronger than at the liquid limit.

Figure 1.4 Definition of Atterberg limits
Atterberg Limit Relationships

- Soils with moisture content near or at the liquid limit are usually normally consolidated.
- As the moisture content moves towards the plastic limit, preconsolidation increases.
- Soils with moisture contents exceeding the liquid limit, the soils can be underconsolidated. Must know site history.
- Cohesive strength increases as moisture content moves towards the plastic limit.
Atterberg Limits Chart

**Figure 1.5** Plasticity chart
Empirical Correlations

Atterberg Limits are used in numerous empirical correlations

- Preconsolidation
- Undrained Strength
- Constrained Modulus
- Permeability
- Moist Unit Weight
- Dry Unit Weight
- Submerged Unit Weight
- $\phi'$

- $C_c$ & $C_r$
- Swell Pressure
- $C_v$
- Void ratio
- Critical State Soil Mechanics Parameters – $\Gamma, \lambda, N, \Lambda_o$

Use correlations to compare to more sophisticated tests. Look for how consistent the soil.
## Unified Soils Classification

### Table 1.8 Unified Soil Classification Chart (after ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)

| Criteria for assigning group symbols and group names using laboratory tests* | Group symbol | Group name*
|---|---|---|
| **Coarse-grained soils**
More than 50% retained on No. 200 sieve | Gravels
More than 50% of coarse fraction retained on No. 4 sieve | Gravels
Clean Gravels
More than 5% fines
Gravels with Fines
More than 12% fines
Sands
50% or more of coarse fraction passes No. 4 sieve | Clean Gravels
$C_a > 4$ and $1 < C_e < 3$
Fines classify as ML or MH
Fines classify as CL or CH
Clean Sands
$C_a > 6$ and $1 < C_e < 3$
Fines classify as ML or MH
Fines classify as CL or CH
Less than 5% fines
Sand with Fines
More than 12% fines | GW
Well-graded gravel
GM
Silty gravel
GC
Clayey gravel
SW
Well-graded sand
SP
Poorly graded sand
SM
Silty sand
SC
Clayey sand
| **Fine-grained soils**
50% or more passes the No. 200 sieve | Silts and Clays
Liquid limit less than 50 | Inorganic
PI > 7 and plots on or above “A” line
PI < 4 or plots below “A” line
Liquid limit—oven dried
Liquid limit—not dried | CL
Lean clay
ML
Silt
| Organic
Liquid limit—oven dried
Liquid limit—not dried | OL
Organic clay
Organic silt
| Silts and Clays
Liquid limit 50 or more | Inorganic
PI plots on or above “A” line
PI plots below “A” line | CH
Fat clay
MH
Elastic silt
| Organic
Liquid limit—oven dried
Liquid limit—not dried | OH
Organic clay
Organic silt
| **Highly organic soils** | Primarily organic matter, dark in color, and organic odor | PT
Peat

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*Based on the material passing the 75-mm. (3-in.) sieve.
*If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.
*Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.
*Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.
*C$_a$ = $D_{60}/D_{10}$, $C_e = (D_{60})^2/D_{60} 	imes D_{10}$
*If field sample contains >15% sand, add “with sand” to group name.
*If fines classify as CL-ML, use dual symbol GC-GM or GC-SC.
*If fines are organic, add “with organic fines” to group name.
*If soil contains >15% gravel, add “with gravel” to group name.
*If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
*If soil contains 15 to 29% plus No. 200, add “with sand” or “with gravel,” whichever is predominant.
*If soil contains >30% plus No. 200, predominantly sand, add “sandy” to group name.
*If soil contains >30% plus No. 200, predominantly gravel, add “gravely” to group name.
*PI > 4 and plots on or above “A” line.
*PI < 4 or plots below “A” line.
*PI plots on or above “A” line.
*PI plots below “A” line.
Coarse Grained Soils

Figure 1.6 Flowchart for classifying coarse-grained soils (more than 50% retained on No. 200 Sieve) (After ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)
Fine Grained Soils

Figure 1.7 Flowchart for classifying fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2009)(ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)
Figure 1.8  Flowchart for classifying organic fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)
Effective Stress

If you cannot master the concept of effective stress and cannot calculate it accurately, you will not get a good grade in this class. All foundation design requires it.
You must understand the concept of effective stress. It is so fundamental to foundation engineering that you will simply not be able to complete almost every design problem.

Figure 1.13 Calculation of effective stress
Effective Stress – No Seepage

In Figure, pore pressure at Point A is

\[ u = h_2 \cdot \gamma_w \]

Where \( \gamma_w \) is the unit weight of water (62.4 pcf)

\[ \sigma' = \sigma - u = (h_1 \cdot \gamma_m + h_2 \cdot \gamma_{sat}) - h_2 \cdot \gamma_w \]

\[ = h_1 \cdot \gamma_m + h_2 (\gamma_{sat} - \gamma_w) = h_1 \cdot \gamma_m + h_2 \cdot \gamma' \]

Where \( \gamma' \) = effective, or submerged, unit weight of soil

\[ \gamma_{sat} = \frac{G_s \cdot \gamma_w + e \cdot \gamma_w}{1 + e} \]

\[ \gamma' = \gamma_{sat} - \gamma_w = \frac{G_s \cdot \gamma_w + e \cdot \gamma_w}{1 + e} - \gamma_w = \frac{\gamma_w (G_s - 1)}{1 + e} \]
Effective Stress – Seepage

In this problem, there is upward seepage of water. For this case, the effective stress at Point A is

\[ \sigma = h_1 \cdot \gamma_w + h_2 \cdot \gamma_{sat} \]
\[ u = (h_1 + h_2 + h) \cdot \gamma_w \]

\[ \sigma' = \sigma - u = (h_1 \cdot \gamma_w + h_2 \cdot \gamma_{sat}) - (h_1 + h_2 + h) \cdot \gamma_w \]
\[ = h_2(\gamma_{sat} - \gamma_w) - h \cdot \gamma_w = h_2 \cdot \gamma' - h \cdot \gamma_w \]

or

\[ \sigma' = h_2 \left( \gamma' - \frac{h}{h_2} \cdot \gamma_w \right) = h_2 \left( \gamma' - i \cdot \gamma_w \right) \]

"i" is the hydraulic gradient. If "i" is very high so that \( \gamma' - i \cdot \gamma_w = 0 \), the effective stress = 0. There will be no contact between soil particles. This is referred to as the quick condition (quick sand), or failure by heave.
Variation of effective stress in a soil profile

- **Dry Sand** $\gamma_d = 14.5 \text{ kN/m}^3$
- **Clay** $\gamma_{sat} = 17.2 \text{ kN/m}^3$

- Water Table at 4 m
- 5 m depth
Effective Stress Solution

<table>
<thead>
<tr>
<th>Point</th>
<th>Depth (m)</th>
<th>( \sigma ) (kN/m³)</th>
<th>( u ) (kN/m²)</th>
<th>( \sigma' ) (kN/m²)</th>
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<tbody>
<tr>
<td>A</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>4</td>
<td>((4)(\gamma_d) = (4)(14.5) = 58)</td>
<td>0</td>
<td>(58 - 0 = 58)</td>
</tr>
<tr>
<td>C</td>
<td>9</td>
<td>(58 + (5)(\gamma_{sat}) = (5)(17.2) = 144)</td>
<td>((5)(\gamma_w) = (5)(9.81) = 49.05)</td>
<td>(144 - 49.5 = 94.95)</td>
</tr>
</tbody>
</table>

- Dry Sand \( \gamma_d = 14.5 \text{ kN/m}^3 \)
- Clay \( \gamma_{sat} = 17.2 \text{ kN/m}^3 \)
- Water Table
Effective Stress #2

Dry Sand $\gamma_d = 14.5 \text{ kN/m}^3$  
Dry Sand $\gamma_{sat} = 15.2 \text{ kN/m}^3$

Clay $\gamma_{sat} = 17.2 \text{ kN/m}^3$
Effective Stress Solution #2

What happens of the groundwater changes in the future?
Preconsolidation Profile
Preconsolidation Profile - CPT
Consolidation

Consolidation is the movement of pore water out of the soil.

Initially the applied load is carried by the pore water. This creates an increase in pore water pressure.

As pore water moves out of the soil which is controlled by the permeability, pore pressure dissipates and the soil matrix begins to carry the load. The soil then compresses.
Principles of Consolidation

\[ \Delta h_i = \Delta \sigma \]

立即加载后:

时间 \( t = 0 \)

\[ \Delta h_i \]

图1.14 合并原理
Consolidation Testing

Figure 1.15 (a) Schematic diagram of consolidation test arrangement; (b) $e$–$\log \sigma'$ curve for a soft clay from East St. Louis, Illinois (Note: At the end of consolidation, $\sigma = \sigma'$)
Normally Consolidated Clay

Figure 1.16 Construction of virgin compression curve for normally consolidated clay
Overconsolidated Clay

Figure 1.17 Construction of field consolidation curve for overconsolidated clay
Real World

Note unload-reload cycle to remove disturbance
NAVFAC LI Versus P’c

LI = (w-PL)/PI

Note: $p_a =$ atmospheric pressure [~100 KN/m2 (1 U.S. ton/ft$^2$)]
One Dimensional Consolidation

Added pressure $= \Delta \sigma$

Groundwater table

Sand

Clay

Initial void ratio $= e_o$

Average effective pressure before load application $= \sigma'_o$

$H_e$

Sand

Figure 1.18 One-dimensional settlement calculation
Primary Consolidation Settlement

Equation 1.65

\[ S := \frac{(C_s \cdot H)}{1 + e} \cdot \log \left( \frac{\sigma'_c}{\sigma'_o} \right) + \frac{(C_c \cdot H)}{1 + e} \cdot \log \left( \frac{\sigma'_o + \Delta\sigma}{\sigma'_c} \right) \]

For normally consolidated soils \( \sigma_c = \sigma'_o \). Therefore \( \log(\sigma_c/\sigma'_o) = 0 \) and the first quantity goes to zero as well.
Pore Pressure Dissipation

Figure 1.19 (a) Derivation of Eq. (1.68); (b) nature of variation of $\Delta u$ with time
Drainage Conditions

$H = 0.5 \ H_c$
Field Data

PZ-2:A

No Longer Responding

PZ-2:B

PZ-2:C

PZ-2:D
Constant $C_v$ Modeling

**Pore Pressure Response Using Single Layer Theory**

Initial Instantaneous Loading

- Time (days):
  - 0.0
  - 1250.0
  - 2500.0
  - 5000.0
  - 10000.0
  - 20000.0
  - 40000.0
  - 75000.0

Multilayer Model Indicates Quicker Dissipation
Range of $C_v$
(after U.S. Dept. of Navy)

Provided by consolidation test and varies with pressure. Use pressure at effective stress.
Tv Versus %U

Time factor against average degree of consolidation ($\Delta u_0 = \text{constant}$)

$$Tv := \frac{\pi}{4} \left( \frac{U\%}{100} \right)^2$$  \text{For } \text{U}= 0 \text{ to } 60\%

$$Tv := 1.781 - 0.933 \log(100 - U\%)$$  \text{For } U>60\%

$$Tv := \left[ \frac{\pi}{4} \left( \frac{U\%}{100} \right)^2 \right] \left[ 1 - \left( \frac{U\%}{100} \right)^{5.6} \right]^{-0.357}$$  \text{For } U = 0 \text{ to } 100\%

\textbf{Figure 1.21} Plot of time factor against average degree of consolidation ($\Delta u_0 = \text{constant}$)
## Table 1.11 Variation of $T_v$ with $U$

<table>
<thead>
<tr>
<th>$U$ (%)</th>
<th>$T_v$</th>
<th>$U$ (%)</th>
<th>$T_v$</th>
<th>$U$ (%)</th>
<th>$T_v$</th>
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<td>0.188</td>
<td>75</td>
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<td>0.197</td>
<td>76</td>
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<td>51</td>
<td>0.204</td>
<td>77</td>
<td>0.511</td>
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Time for Compression

Typical client wants to know how long he has to wait before starting construction.

$$ Tv \ := \frac{(Cv \cdot t)}{H^2} $$

For $U\% = 90\%$ $Tv = 0.849$

With $H = 10$ feet & double drainage
- $H/2 = 5$ feet
- $Cv = 0.2$ ft$^2$/day

$$ t = \frac{0.849(5)^2}{0.2} = 106 \text{ days} $$

For single drainage $H=10$ feet
$$ t = \frac{0.849(10)^2}{0.2} = 424.5 \text{ days} $$
Ramp or Construction Loading

A new layer of structural fill or building structure cannot be loaded instantaneously on the ground.

For this reason, the increase in loading gradually rises to the maximum load.

This gives time for excess pore water pressure to begin dissipating.
One-dimensional consolidation due to single ramp loading

**Figure 1.22** One-dimensional consolidation due to single ramp loading
Ramp Loading Parameters

\[ T_c := C_v \cdot \frac{t_c}{H^2} \]

Where do we get \( t_c \)? Construction Schedule.

Remember double or single drainage.
Olson’s Ramp Loading

Figure 1.23 Olson’s ramp-loading solution: plot of $U$ versus $T_v$ (Eqs. 1.78 and 1.79)
Ramp Loading Example

Example 1.10

With $H = 10$ feet & double drainage $H/2 = 5$ feet
$C_v = 0.2$ ft$^2$/day
$t_c = 15$ days, what $%U$ at 50 days

$T_c = 0.2(15/(5)^2) = 0.12$
$T_v = 0.2(50/(5)^2) = 0.4$

From chart $U% = 70%$. 

\[
T_c := C_v \cdot \frac{t_c}{H^2}
\]
Shear Strength

\[ S = c' + \sigma' \cdot \tan(\phi') \]

Shear Strength Tests

- Unconfined Compression Tests
- Direct Shear Tests
- Direct Simple Shear
- Triaxial Shear Tests
  - Unconsolidated – Undrained (UU) \( \Delta u = 0 \)
  - Consolidated – Undrained w/PPM (CU)
  - Consolidated – Drained (CD)

Each test will yield a different value of Su
Drained Strength typically > Undrained Strength
Figure 1.25 Direct shear test in sand: (a) schematic diagram of test equipment; (b) plot of test results to obtain the friction angle $\phi'$
## Typical Values of $\phi$

**Table 1.12** Relationship between Relative Density and Angle of Friction of Cohesionless Soils

<table>
<thead>
<tr>
<th>State of packing</th>
<th>Relative density (%)</th>
<th>Angle of friction, $\phi'$ (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>&lt;20</td>
<td>&lt;30</td>
</tr>
<tr>
<td>Loose</td>
<td>20–40</td>
<td>30–35</td>
</tr>
<tr>
<td>Compact</td>
<td>40–60</td>
<td>35–40</td>
</tr>
<tr>
<td>Dense</td>
<td>60–80</td>
<td>40–45</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt;80</td>
<td>&gt;45</td>
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</tbody>
</table>
Triaxial Shear Test

Figure 1.26: Triaxial test
Sequence of Stress Application

Figure 1.27 Sequence of stress application in triaxial test
Unconfined Shear Test

(a) soil specimen;
(b) Mohr’s circle for the test;
(c) variation of $q_u$ with the degree of saturation

$$S_u = c_u = q_u/2$$

**Figure 1.28** Unconfined compression test: (a) soil specimen; (b) Mohr’s circle for the test; (c) variation of $q_u$ with the degree of saturation
Triaxial Test in Progress
\( \phi \) Versus Void Ratio

![Graph showing the variation of friction angle \( \phi' \) with void ratio for Chattahoochee River sand.](image)

*Figure 1.29* Variation of friction angle \( \phi' \) with void ratio for Chattahoochee River sand (After Vesic, 1963) (From Vesic, A. B. Bearing Capacity of Deep Foundations in Sand. In Highway Research Record 39, Highway Research Board, National Research Council, Washington, D.C., 1963, Figure 11, p. 123. Reproduced with permission of the Transportation Research Board.)
$\phi' \text{ Versus Plasticity Index}$

$\phi = 0.0011 \cdot \text{PI}^2 - 0.2603 \cdot \text{PI} + 35.975$

*Figure 1.30* Variation of sin $\phi'$ with plasticity index (PI) for several normally consolidated clays

Note scatter
Deviator stress vs. axial strain-drained triaxial test

$\sigma_3 = \sigma'_3 = \text{constant}$

Strain Softening

Figure 1.31 Plot of deviator stress versus axial strain–drained triaxial test
Peak & Residual Strength Envelopes for Clays

Figure 1.32 Peak- and residual-strength envelopes for clay
Variation of $\phi'_r$ with CF

![Graph showing variation of $\phi'_r$ with CF for different soil types: Sand, Kaolin, and Bentonite. The graph includes data points and curves for each soil type, with annotations for Skempton (1985) formula: $\frac{\sigma'}{p_a} \approx 1$, Plasticity index, PI: 0.5 to 0.9, and Clay fraction, CF: 0% to 100%.]

**Figure 1.33** Variation of $\phi'_r$ with CF (Note: $p_a$ = atmospheric pressure)
## Empirical Correlations

<table>
<thead>
<tr>
<th>Reference</th>
<th>Relationship</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton (1957)</td>
<td>( \frac{c_u}{\sigma_0'} = 0.11 + 0.00037 ) (PI)</td>
<td>For normally consolidated clay</td>
</tr>
<tr>
<td></td>
<td>( \text{PI} = \text{plasticity index (%)} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( c_u^{(VST)} = \text{undrained shear strength from vane shear test} )</td>
<td></td>
</tr>
<tr>
<td>Chandler (1988)</td>
<td>( \frac{c_u}{\sigma_c'} = 0.11 + 0.0037 ) (PI)</td>
<td>Can be used in overconsolidated soil; accuracy ±25%; not valid for sensitive and fissured clays</td>
</tr>
<tr>
<td></td>
<td>( \sigma_c' = \text{preconsolidation pressure} )</td>
<td></td>
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<tr>
<td>Jamiołkowski, et al. (1985)</td>
<td>( \frac{c_u}{\sigma_c'} = 0.23 \pm 0.04 )</td>
<td>For lightly overconsolidated clays</td>
</tr>
<tr>
<td>Mesri (1989)</td>
<td>( \frac{c_u}{\sigma_0} = 0.22 )</td>
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<tr>
<td>Bjerrum and Simons (1960)</td>
<td>( \frac{c_u}{\sigma_0} = 0.45 \left( \frac{\text{PI} %}{100} \right)^0.5 )</td>
<td>Normally consolidated clay</td>
</tr>
<tr>
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<td>for ( \text{PI} &gt; 50% )</td>
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<tr>
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<td>( \frac{c_u}{\sigma_0} = 0.118 (\text{LI})^{0.15} )</td>
<td>Normally consolidated clay</td>
</tr>
<tr>
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<td>for ( \text{LI} = \text{liquidity index} &gt; 0.5 )</td>
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<tr>
<td>Ladd, et al. (1977)</td>
<td>( \left( \frac{c_u}{\sigma_0} \right)_{\text{overconsolidated}} = \text{OCR}^{0.8} )</td>
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<tr>
<td></td>
<td>( \left( \frac{c_u}{\sigma_0} \right)_{\text{normally consolidated}} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OCR = \text{overconsolidation ratio} = \sigma_c'/\sigma_0'</td>
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</tr>
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</table>
Variation of $\phi'_r$ with liquid limit for some clays

(after Stark, 1995)
Variation With Depth - Clay Deposit

In normally consolidated clays, undrained shear strength increases almost linearly with effective overburden pressure.

\[ \frac{S_u}{\sigma'} \text{ ratio} \]
$S_u/\sigma'$ Relationships

Use consolidated-undrained triaxial tests at different levels of stress to determine $S_u/\sigma'$ ratio. Can be estimated by:

$$
\left(\frac{S_u}{\sigma'}\right)_{NC} = 0.11 + 0.0037\text{PI}
$$

$$
\left(\frac{S_u}{\sigma'}\right)_{OC} = \left(\frac{S_u}{\sigma'}\right)_{NC} \cdot \text{OCR}^{0.8}
$$

Critical State
Pore Pressure Parameter $\Lambda_0$
Varies with Soil

$$
\Lambda_0 := 1 - \frac{C_s}{C_c}
$$
Variation of Cu with LI

LI = (w-PL)/PI

(based on Bjerrum and Simons 1960)

\[
\frac{c_u}{\sigma'_o} = 0.0074\cdot LI^4 - 0.0706\cdot LI^3 + 0.2547\cdot LI^2 - 0.4258\cdot LI + 0.403
\]
Homework

From Chapter 1

CE 420
- 1.11
- 1.12
- 1.13
- 1.14

CE 520
- All of CE 420 plus
- 1.15
- 1.19

Read Chapter 2