The Consolidation and Strength Behavior of Mechanically Compressed Fine-Grained Sediments

A Ph.D. Defense

by

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Outline

• Motivation and Objectives
• Resedimentation
• Permeability Results
• Triaxial Equipment and Procedures
• Principle of Effective Stress
• Shear Strength Behavior
• Summary and Conclusions
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Motivation

For soils and ‘soft’ rock, shear strength is complex a function of:

\[ \tau_{max} = f \]

- composition \((w_L)\)
- effective stress \((\sigma')\)
- stress history \((OCR)\)
- mode of shear \((b, \alpha)\)
- temperature \((T)\)
- strain rate \((\dot{\varepsilon})\)
- water saturation \((S_w)\)
- diagenesis
• Majority of previous studies have involved testing intact samples
  ➢ cannot isolate and quantify individual factors influencing behavior
  ➢ disturbance and cost, particularly for deep or offshore samples

• Resedimentation
  ➢ Technical necessity!
  ➢ Practical advantages
  ➢ Compares well with intact behavior

• Best data for resedimented clay behavior from Abdulhadi (2009)
  ➢ tested RBBC for stresses from 0.1→ 10 MPa in triaxial compression

• Very limited testing of resedimented soil over a wide stress range
  – Bishop et al. (1975); tested London Clay at Imperial College
  – Yassir (1989); tested mud volcano clay at UCL
  – Nüesch (1991); tested unsaturated Opalinus Shale
  – Berre (1992); tested a kaolinite – Moum clay mixture at NGI
  – William (2007); tested Bringelly Shale at University of Sydney
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• **Resedimentation**

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Resedimentation

1. Obtain core material
2. Breakdown into powder and blend
3. Mix dry powder and water into slurry
4. Vacuum the slurry
5. Pour slurry into a consolidometer

Comparisons of resedimented vs. intact behavior:
- Berman 1993 (BBC)
- Mazzei 2008 (RGoM Ursa)
- Casey 2011 (BBC)
- House 2012 (BBC)
- Betts 2014 (RGoM Eugene Is.)
Resedimentation

4. Load incrementally
   - Different consolidometers used depending on testing needs
   - Low stress triaxial: $\sigma'_p = 0.1$ MPa
   - Medium stress triaxial: $\sigma'_p = 2$ MPa
   - High stress triaxial: $\sigma'_p = 10$ MPa
   - Time required for resedimentation strongly dependent on soil type ($c_v$)

5. Swell to OCR = 5

6. Extrude and trim test specimen
What am I dealing with?

Contributing researchers:
- Grennan (2010)
- Abdulhadi (2009), Sheahan (1991)
- Jones (2010)
- Kontopoulos (2012)
- Betts (2014), Fahy (2014)
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**Permeability**

\[
\log(k) = \gamma (n - 0.5) + \log(k_{0.5})
\]

for \(0.20 < n < 0.75\)
Permeability Correlations

\[ \log(k) = \gamma \cdot (n - 0.5) + \log(k_{0.5}) \]

\[ \gamma = 0.067 \cdot w_L + 5.1 \quad r^2 = 0.75 \]

\[ \log(k_{0.5}) = -7.55 \log(w_L) - 3.4 \quad r^2 = 0.90 \]
Permeability Model: Error Analysis

![Graph showing measured vs. predicted permeability values for different models. The graph includes a legend with various line styles and markers for different models such as SS, RPC, RBBC, etc. There is a note indicating that if measured permeability $k = 1$, predicted permeability $k = 0.2 \rightarrow 5$. The graph has a linear scale on the x-axis for predicted permeability and a log scale on the y-axis for measured permeability.](image)
Permeability: Predicting In situ Behaviour

Boston Blue Clay

Liquid limit / Porosity

Permeability, k (m²)

Depth (ft)

- liquid limit
- porosity
Permeability: Predicting In situ Behaviour

- Measured Permeability, \( k \) (m\(^2\)) vs. Predicted Permeability, \( k \) (m\(^2\))
- Data points for different categories are represented with different symbols and colors.
- The graph includes a zoomed-in map of the area around New Orleans with a distance scale of 100 km or 50 miles.
- The data is shown with a range of +/- 5 for certain datasets.
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Typical Triaxial Test Procedure

1. Setup and back-pressure saturation (*1 day*)
2. $K_O$-consolidation of specimens (*3-10 days*)
   - Important to mimic field conditions
3. Secondary compression/creep (*1 day*)
4. $K_O$-swelling (*1 – 2 days*)
5. Undrained shear in triaxial compression (*1 day*)
low pressure triaxial
($\sigma'_p < 2 \text{ MPa}$)

medium pressure triaxial
($2 < \sigma'_p < 10 \text{ MPa}$)

high pressure triaxial
($10 < \sigma'_p < 100 \text{ MPa}$)
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Effective Stress

- **Effective Stress**: Partial stress which controls changes in deformation and shear resistance of porous materials.

- Conventional Terzaghi (1923) definition for saturated soil:
  \[ \sigma' = \sigma - u \]
  - assumes particles are: 1) incompressible, and 2) have a constant yield strength

- Some have proposed modified definitions, such as:
  - \[ \sigma' = (\sigma - u) + au + (R - A) \quad \text{‘Intergranular stress’} \]
  - \[ \sigma' = \sigma - \left(1 - a \frac{\tan \psi}{\tan \phi'}\right)u \quad (Skempton 1960) \]
    \( a \) = contact area between particles per unit area.

- At high stresses the contact area can become significant; can true effective stress deviate from Terzaghi definition? ...literature typically assumes no
Tests of Bishop and Skinner (1977)

- Most significant testing program to examine effective stress in relation to shear resistance
- Drained triaxial compression tests involving large changes in back-pressure but keeping \((\sigma_3 - u_b)\) constant during shearing
- Significance of interparticle contact area determined from discontinuities in shear stress-strain curve
- Tested sand, silt, crushed marble, lead shot for pore pressures up to 40 MPa
Tests of Bishop and Skinner (1977)

Results and conclusions:

• Terzaghi definition applicable for full range of stresses tested with no observable change in shear resistance
• Intergranular stress equation not valid
• Inconclusive re. Skempton’s (1960) equation

However….

• No clays were tested
• Nature of inter-particle contacts is potentially different for clays
Effective Stress Tests

$$\sigma'_p = 0.6 \text{ MPa}$$

<table>
<thead>
<tr>
<th>Line</th>
<th>Soil</th>
<th>$u_b$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RBBC</td>
<td>0.49</td>
<td></td>
</tr>
<tr>
<td>RBBC</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>RBBC</td>
<td>4.90</td>
<td></td>
</tr>
<tr>
<td>RBBC</td>
<td>9.80</td>
<td></td>
</tr>
<tr>
<td>RBBC</td>
<td>9.80</td>
<td></td>
</tr>
<tr>
<td>RGoM</td>
<td>varies</td>
<td></td>
</tr>
<tr>
<td>Ursa</td>
<td>varies</td>
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</tr>
</tbody>
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Stress-Strain Response during Shearing

Normalized Shear Stress, $q/\sigma'_{vc}$ vs. Axial Strain, $\varepsilon_a$ (%)

- $s_u/\sigma'_{vc}$
- $\sigma'_{vc} = 0.2$ MPa
- 1.2 MPa
- 9.8 MPa
- 105 MPa

R. Ugnu Clay @ OCR = 1
Undrained Strength @ OCR = 1

\[
\frac{s_u}{\sigma'_{vc}} = S_1 (1000\sigma'_p [\text{MPa}])^T
\]
Undrained Strength @ OCR = 1

- Skibbereen Silt
- R. Presumpscot Clay
- R. Boston Blue Clay
- R. GoM Ursa Clay
- R. Ugnu Clay
- R. S.F. Bay Mud
- R. London Clay
- R. GoM Eugene Is.

Preconsolidation Stress, $\sigma'_p$ (MPa) vs. Undrained Strength Ratio, $s_u/\sigma'_v$
Undrained Strength - Liquid Limit Correlations

\[ S_1 = 0.86 \log(w_L) - 1.04 \]
\[ r^2 = 0.97 \]

\[ T = -0.46 \log(w_L) + 0.73 \]
\[ r^2 = 0.95 \]
Overconsolidated Behavior

\[ \frac{\sigma_v'}{\sigma_v'} = \begin{cases} \text{OCR} = 8 \\ \text{OCR} = 4 \\ \text{OCR} = 2 \\ \text{OCR} = 1 \end{cases} \]

\[ \sigma_v' = 0.6 \text{ MPa} \]

\[ \sigma_v' = 40 \text{ MPa} \]

R. Boston Blue Clay

Normalized Shear Stress, \( \frac{q}{\sigma_v'} \) vs. Axial Strain, \( \varepsilon_a \) (\%)
Increase in Ductility with Stress

R. Boston Blue Clay

Axial Strain to Undrained Failure, $\varepsilon_f$ (%) vs. Overconsolidation Ratio Ratio, OCR

- $\sigma'_p = 40$ MPa
- $\sigma'_p = 10$ MPa
- $\sigma'_p = 0.2$ MPa
Undrained Strength: Overconsolidated Soil

R. Boston Blue Clay

OCR = 8

OCR = 4

OCR = 2

OCR = 1

Undrained Strength Ratio, $s_u/\sigma'_{vc}$

$T$ is independent of OCR

$s_u/\sigma'_{vc} = 1.701(1000\sigma'_p)^{-0.028}$

$s_u/\sigma'_{vc} = 1.083(1000\sigma'_p)^{-0.035}$

$s_u/\sigma'_{vc} = 0.593(1000\sigma'_p)^{-0.020}$

$s_u/\sigma'_{vc} = 0.366(1000\sigma'_p)^{-0.024}$

Preconsolidation Stress, $\sigma'_p$ (MPa)

Undrained Strength Ratio, $s_u/\sigma'_{vc}$

$s_{1(OC)}$
Undrained Strength: Overconsolidated Soil

\[ S_{1(OC)} = 0.368(OCR)^{0.73}, \quad r^2 = 0.9999 \]

approx. constant for fine-grained soils

R. Boston Blue Clay
Summary of Strength Equations

• Undrained triaxial compressive strength:

\[ \frac{s_u}{\sigma'_v} = S_1 (1000\sigma'_p \text{[MPa]})^T (OCR)^{0.73} \]

- \[ S_1 = 0.86\log(w_L) - 1.04 \]
- \[ T = -0.46\log(w_L) + 0.73 \]
Effect of $K_0$ on Undrained Strength @ OCR=1

$$\sigma'_V$$

$$K_0 = \frac{\sigma'_H}{\sigma'_V}$$

$$\frac{s_u}{\sigma'_{vc}} = 0.57 - 0.50K_{ONC}$$

$$+/- 0.02$$
Friction Angle

\[ \phi = A(0.001\sigma'_p [\text{MPa}])^B \]

Critical State Friction Angle, \( \phi'_{cs} \) (°)

Preconsolidation Stress, \( \sigma'_p \) (MPa)

R. Ugnu Clay

\( \Delta \) R. Ugnu Clay
Friction Angle

Critical State Friction Angle, $\phi'_{cs}$ (°)

Preconsolidation Stress, $\sigma'_p$ (MPa)

- Skibbereen Silt
- R. Presumpscot Clay
- R. Boston Blue Clay
- R. GoM Ursa
- R. Ugnu Clay
- R. S.F. Bay Mud
- R. S.F. Bay Mud
- R. London Clay
- R. GoM Eugene Is.
Friction Angle - Liquid Limit Correlations

\[ A = -75 \log(w_L) + 148 \]
\[ r^2 = 0.89 \]

\[ B = -0.39 \log(w_L) + 0.59 \]
\[ r^2 = 0.95 \]
Summary of Strength Equations

• Undrained triaxial compressive strength:

\[ s_u / \sigma'_{vc} = S_1 (1000 \sigma'_p [\text{MPa}])^T (OCR)^{0.73} \]

- \( S_1 = 0.86 \log(w_L) - 1.04 \)
- \( T = -0.46 \log(w_L) + 0.73 \)

• Drained triaxial compressive strength:

\[ \phi'_{cs} = A (0.001 \sigma'_p [\text{MPa}])^B \]

- \( A = -75 \log(w_L) + 148 \)
- \( B = -0.39 \log(w_L) + 0.59 \)
Effect of OCR on $\phi'_{cs}$

Critical State Friction Angle, $\phi'_{cs}$ (°)

Preconsolidation Stress, $\sigma'_p$ (MPa)

OCR = 1
OCR = 2
OCR = 4
OCR = 8

R. Boston Blue Clay
(assuming drained conditions and no surcharge)

– a change in friction angle from 40° to 35° reduces bearing capacity by 56 %

– a change in friction angle from 40° to 30° reduces bearing capacity by 80 %!
Particle Reorientation

… but failure in triaxial compression occurs at $\sim 50^\circ \rightarrow 65^\circ$

→ Particle reorientation with stress cannot explain strength behavior
At very high stresses...

- Porous materials will ultimately reach the friction angle of the solid material, referred to as the *intrinsic friction angle* $\psi$ (Skempton 1960)
- Tests on marble, metals, quartz and limestone

<table>
<thead>
<tr>
<th>Material</th>
<th>$\Psi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>8</td>
</tr>
<tr>
<td>Calcite</td>
<td>8</td>
</tr>
<tr>
<td>Quartz</td>
<td>16</td>
</tr>
<tr>
<td>Clay minerals</td>
<td>~ 5–10</td>
</tr>
</tbody>
</table>

*from Skempton (1960)*

\[ c' = 350 \]
\[ \Phi' = 34^\circ \]
\[ \tau_d \]
\[ \tau_f \]
\[ \sigma_x' \]
Yield Surface Evolution

R. Boston Blue Clay

Normalized Shear Stress, $q/\sigma'_{vc}$ vs. Normalized Effective Stress, $p'/\sigma'_{vc}$

- Green line: Low stress (< 1 MPa) yield surface
- Red line: High stress (> 10 MPa) yield surface
Conclusions

- Resedimentation is a technical necessity and practically advantageous to study the behavior of soils systematically.
- Correlations developed from resedimented soil using liquid limit can predict intact permeability, a robust indicator of composition.
- Conventional Terzaghi definition of effective stress is valid for fine-grained soils at high in situ pore pressures.
- Shear strength properties vary consistently with stress level and are closely linked to composition/plasticity.
- Variations in strength properties with stress reflect an evolving yield surface.
For soils and ‘soft’ rock, shear strength is complex a function of:

\[ \tau_{\text{max}} = f \]

- composition \( (w_L) \)
- effective stress \( (\sigma') \)
- stress history \( (OCR) \)
- mode of shear \( (b, \alpha) \)
- temperature \( (T) \)
- strain rate \( (\dot{\varepsilon}) \)
- water saturation \( (S_w) \)
- diagenesis

**This work**

**Future Work**
Publications


• Casey, B. & Germaine, J.T. (2014). “An Evaluation of Three Triaxial Systems with Results from 0.1 to 100 MPa” Geotechnical Testing Journal, in review