Critical State Soil Mechanics for Clays and Beyond

Kenichi Soga
University of California, Berkeley
Fundamentals of Soil Behavior, James K. Mitchell

60 m tall
20 m in diameter
145 MN weight
5.5 degrees to the South,
6 m out of plumb

Constructed in three phases
– 1st stage = 1173 - 1178 Four floors
– 2nd stage = 1272 – 1278 Three floors
– 3rd stage = 1360 – 1370 Bell tower
Kansai International Airport

About the size of the city of Berkeley in length, or 2/3 length of Cambridge, MA
Temporary state - (sediment and geological history), $\phi_{\text{peak}}$

Disturbed state (Critical State) – Critical state friction angle, $\phi_{\text{critical}}$

Really disturbed state - Residual friction angle, $\phi_{\text{residual}}$

Really wet state – Liquid like

Fracture state – shear induced or tensile induced
After Roscoe et al., 1958
\[ LI = \frac{w - w_{PL}}{w_{LL} - w_{PL}} \]

Water content

- Sloppy
- Plastic
- Crumbly

Liquidity Index: LI = 1
Liquidity Index: LI = 0

vary water content until penetration is exactly 20mm

keep rolling it, so it slowly dries...
...until it just cracks

Fig. 5. Stresses within soil thread during plastic limit test: (a) total stress; (b) effective stress
Void ratio $e = G_s \times w = \text{Specific gravity} \times \text{water content}$

**Water content**

- Liquid Limit (LL)
- Plastic Limit (PL)

Critical state line:
- $p'_\text{LL}$
- $p'_\text{PL}$
- $\ln(p')$

**Deviator stress $q$**

- $q'_\text{LL}$
- $q'_\text{PL}$

- $3-4 \text{ kPa}$
- $300-400 \text{ kPa}$
Kaolin specimens

Sample A  Localised failure
Sample B  Buldging failure
Ductile behavior

Rupture/shear localization

Tensile fracture

Modified from Schofield Rankine Lecture
Shear localization leads to particle reorientation – Residual friction angle

Figure 11.74 Influence of clay fraction on the peak and residual friction angles for sand–bentonite mixtures as determined by ring shear tests (after Lupini et al., 1981).

Figure 11.76 Residual friction angle of volcanic clays and clays in relation to their location on the plasticity chart relative to the A-line (from Wesley, 2003).
Questions

Zone 1
• Why the critical state line in e-lnp’ plane becomes straight in clay?

• What are the microstructural changes occurring during critical state (constant volume shearing)?

Zone 2
• How far can we use the critical state soil mechanics in Zone 2? Limit L1_{eq}?

• What is the mechanism of shear localization in Zone 2? How clay particles start to orient themselves to slide easier (residual friction angle)?
Reconstituted ‘lab consolidated’ clay

Natural sedimented clay
Figure 8.17 Compression curves for freshwater glacial lake clay at pressures below and above yield (from Burland, 1990).
Figure 8.16  (a) Compression curves for several clays (from Skempton, 1970). (b) Normalized compression curves for clays in (a) showing the intrinsic compression line (ICL) and sedimentation compression line (SCL).
Normalized Compression curve
Burland 1990 Rankine lecture

Natural sedimented clay

‘lab consolidated’ clay
Questions

• Why slow sedimentation gives larger water content at a given confining stress?

• What post-depositional processes (pore fluid chemistry-clay mineral interactions) create metastable soil structure?
  • Leading to catastrophic failure upon undrained shear.
“Really wet” state

Source Area

Flow Extent

Water content

Liquid Limit (LL)

Plastic Limit (PL)

$\rho_{\text{LL}}$

$\rho_{\text{PL}}$

$\lambda$

Time $= 0$ s

[Graph showing liquid limit and plastic limit with water content]
Mechanism of Submarine Run-out

- Presence of ambient water (larger drag force & less gravity)
- Water entrainment - Hydroplanning
Soil surface cone

\[ \lambda^* = 0.125 \]

\[ \Gamma^* = 1.35 \]

Fall cone
Rheometer (Yield strength)
Equation (3-8)

\[ \ln v \]
\[ \ln p' \text{ (kPa)} \]
Depth-averaged material point (column)

Depth-averaged stress $\bar{\sigma}$ and strain $\bar{\varepsilon}$

Basal stress $\tilde{\sigma}$ and strain $\tilde{\varepsilon}$

Water entrainment

Mixing at the bottom interface
Questions “Really wet” state

• At what water content the critical state soil mechanics fails to work?
  • When clay becomes like a liquid like material?

• At this condition, what is the microscopic interaction of clay particle and fluid mixture under rapid shear?
Fracture state – shear induced or tensile induced
Normally consolidated clay

Water Injection

Laponite injection (Slightly more viscous than water)
Tensile induced mode

\[ P_f = 2\sigma_0 - u_0 + \sigma'_t \]

Shear induced mode

\[ P_f = \sigma_0 + ns_u \]

Figure 11.106 Fracture mechanisms of injection fluids into a cavity: (a) tensile fracture in undrained conditions and (b) shear failure in undrained conditions.
Tensile induced mode

\[ P_f = 2\sigma_0 - u_0 + \sigma'_t \]

Slope of 2:1

Shear induced mode

\[ P_f = \sigma_0 + ns_{tu} \]

Slope of 1:1
Tensile induced mode

\[ P_f = 2\sigma_0 - u_0 + \sigma'_t \]

Shear induced mode

\[ P_f = \sigma_0 + n s_u \]
Questions – Fracture state

- Two different fracture modes
  - Tensile induced fracture
  - Shear instability induced fracture

- Tensile induced fracture
  - Tensile strength - Effective stress or Total stress?

- Shear induced fracture
  - Instability in undrained conditions
  - Microscopic behavior of fracture development and propagation
Temporary state - (sediment and geological history), $\phi_{\text{peak}}$

Disturbed state (Critical State) – Critical state friction angle, $\phi_{\text{critical}}$

Really disturbed state - Residual friction angle, $\phi_{\text{residual}}$

Really wet state – Liquid like

Fracture state – shear induced or tensile induced

So lots of microscopic behavior questions to understand why our theories work sometimes and do not work some other times....

Thank you
Empirical relation by Jeong et al. (2007)

\[ \mu = \left( \frac{9.27}{I_L} \right)^{3.3} \]

(From Jeong et al, 2007)
Possible mechanisms

- Undrained failure of the Pancone clay
- Consolidation of the Pancone clay
- Shear failure of the sand layer
Kansai Int. Airport

Thickness (m) vs. Time (1987/9 to 2011/9)

- Landfill height

Settlement (m) vs. Time (1987/9 to 2011/9)

- Holocene layers, improved with sand drains
- Airport inauguration 1994/9

Settlement (m) vs. Layers

- Pleistocene layers, without soil improvement

Prediction of the long-term consolidation is very important

1st phase: \( \Delta p = 450 \text{ kPa} \)
S = 14.5 m

2nd phase: \( \Delta p = 600 \text{ kPa} \)
S = 18 m

Berkeley
UNIVERSITY OF CALIFORNIA
\[ q_{u,\text{res}} = M_{\text{comp}} p'_{\text{res}} \]

stable fractal compression

unstable 1D crushing - drained

sensitive

insensitive

\[ q_{u,\text{max}} \]

\[ q_{u,\text{res}} \]

\[ q_{u,\text{res}} \]

\[ q_{u,\text{res}} \]

unstable 1D crushing - drained

stable fractal compression
Soil Properties at High Water Content

Bingham Fluid

\[ \tau = \tau_y + \mu \dot{\gamma} \]

Constant

New Model

\[ \tau = \tau_y (\phi', \varepsilon_d^p, w) + \mu(w)\dot{\gamma} \]

Cam Clay Model  Empirical Relation

Shear Stress $\tau$

Shear Strain Rate $\dot{\gamma}$

Water Entrainment

Constant $\tau_y$
Specific volume $\nu$

$\nu_{\text{max}}$

$\nu_{\text{initial}}$

Rate of volume expansion $R_\nu$

Travel distance of material point
Figure 4.3 Section of Modified Consolidometer

Note: Not to Scale
Void ratio $e$

Sample is initially consolidated to $\sigma'_0$

Drained tests

Normally consolidated

Over consolidated

Overconsolidated

Critical

Shear strain or Axial strain

$e(\text{NC})$

$e(\text{OC})$

$e(\text{critical})$

Void ratio

Dilation

Contraction

Overconsolidated

$\ln \sigma'_0$, $\ln \sigma'$
Simulation of 2010 St. Jude Landslide, Canada