Evaluation of Flow
Liquefaction in tailings and mine waste using the CPT

P.K. Robertson
Edmonton, 2018
Definitions of Liquefaction

• **Cyclic (seismic) Liquefaction**
  – Zero effective stress (during cyclic loading)

• **Flow (static) Liquefaction**
  – Strain softening response
Tailings & mine waste

**Flow Liquefaction** is one of the main design issues for most tailings and mine waste structures

- High static shear (“steeply” sloping ground)
- Various trigger mechanisms
  - *Cyclic loading only one form of trigger*
- High risk if significant strength loss possible
Flow liquefaction - Case histories

• **Common soil features:**
  - Very young age
  - Non-plastic or low-plastic
  - Little or no stress history ($K_o \sim 0.5$)
  - Very loose (*contractive*)
  - Low effective stress ($\sigma'_{vo} < 3$atm)

• **Common instability features:**
  - Some triggered by very minor disturbance
  - Failures tend to occur without warning
  - Failures tend to be progressive & rapid
  - Observation approach not valid

Stava, Italy, 1985; 268 deaths, 190,000m³

Fundao, Brazil, 2015 19 deaths
Flow liquefaction – Evaluation Steps

Evaluation Sequence:
1. Evaluate **susceptibility** for strength loss
2. Evaluate **stability** using post-liquefaction (large strain) shear strengths
3. Evaluate **trigger** for strength loss

*If soils are susceptible, and instability possible (FS<1), it is often prudent to assume trigger will occur*
Outline

• **Trigger** events
  – Small events can trigger strength loss
  – Critical stress paths

• Classify materials *susceptible* to strength loss
  – Application of SCPTu

• **Stability**
  – Influence of high stresses
  – Sand-like and clay-like
  – Progressive failure
Triggers - Flow Liquefaction

After Olson & Stark, 2003

Undrained loading e.g. rapid construction A-B-C
Undrained cyclic e.g. earthquake A’-E-C
Unloading e.g. increasing GWL A-D-C
Example - Fundao

Unloading stress paths are the most critical - can be drained or undrained.

Higher static shear stress ratio - smaller the trigger.

Lab. testing
Example - Fundão

Unloading stress paths are the most critical - can be drained or undrained

Higher static shear stress ratio - smaller the trigger


Lab. testing

Figure 4-1 Stress paths for undrained loading and drained unloading of sand, Fundão test data
Classify **susceptibility to strength loss**

- Geo-materials must be *strain softening* in undrained shear
- Strain softening geo-materials are *contractive* at large strains

**CPT can be used to identify contractive soil** for young, uncemented soils - *Robertson(2010)*
Case Histories of Flow Liquefaction

Case Histories
- Nerlerk (sand) – 19,20,21
- Jamuna (sand) - 34
- Fraser River (silty sand) - 27
- Sullivan mines (silty tailings) - 35
- Northern Canada (silty clay) – 36
- L. San Fernando Dam (silt) – 15

CPT data in critical layers +/- 1 sd.
(Average stress level < 200 kPa)

All flow liq. case histories plot in ‘contractive’ portion of CPT SBT chart
Good theoretical support via State Parameter
Case Histories of Flow Liquefaction

$\frac{S_u}{\sigma'_{vo}} = \text{liq. undrained strength ratio (sand-like)}$
Case Histories of Flow Liquefaction

\[ s_{u(\text{liq})}/\sigma'_{\text{vo}} = \text{liq. undrained strength ratio (sand-like)} \]
Some natural soils and mine tailings/waste have some form of ‘structure’ that make their behavior different from ‘ideal’ soil

- **Macrostructure** (layering, fissuring, etc.)
- **Microstructure** (particle scale – aging, bonding, thixotropic hardening, etc.)

Canadian Geotechnical Journal, 2016

“CPT-based Soil Behavior Type (SBT) Classification System – an update” P.K. Robertson
Seismic CPTu

After Mayne, 2014

Seismic Cone Penetration Test (SCPT)
ASTM D 5778 and ASTM STP 1213

SCPTu
6 -7 measurements!

\( q_t \)
\( f_s \)
\( u_2 \)
\( V_s (V_p) \)
\( t_{50} \)
\( u_o \)
\( i \)

Diss.test
Identification of microstructure

- CPT penetration resistance, $q_t$ – controlled by peak strength
- Shear wave velocity, $V_s$ – controlled by small strain stiffness
- Potential to identify ‘structured’ soils from SCPT by measuring both peak strength and small strain stiffness
New $G_0/q_n$ Chart

Small strain rigidity index:

$$I_G = G_0/(q_t - \sigma_{vo})$$

Normalized rigidity index:

$$K^*_G = I_G \ (Q_{tn})^{0.75}$$
New $G_o/q_n$ Chart

Average normalized rigidity index for young, uncememented silica-based soils:

$$K^*_G = 215$$

Robertson, 2016
Updated CPT-based SBT Charts

Normalized SCPTu parameters: $Q_{tn}$, $F_r$, $U_2$ and $I_G$

Canadian Geotechnical Journal, 2016
“CPT-based Soil Behavior Type (SBT) Classification System – an update” P.K. Robertson
Updated CPT-based SBT Charts

Normalised SCPTu parameters: $Q_{tn}$, $F_r$, $U_2$ and $I_G$

Most Canadian Oil Sands tailings have little or no microstructure

Canadian Geotechnical Journal, 2016
“CPT-based Soil Behavior Type (SBT) Classification System – an update” P.K. Robertson
Suitable for most tailings that have little or no microstructure

Soils with no microstructure

Updated SBTn Charts

Behavior Descriptions

Soil Behaviour Type

1: CCS Clay-like - Contractive - Sensitive
2: CC Clay-like - Contractive
3: CD Clay-like - Dilative
4: TC Transitional - Contractive
5: TD Transitional - Dilative
6: SC Sand-like - Contractive
7: SD Sand-like - Dilative

$CD = (Q_{tn} - 11)(1 + 0.06F_r)^{1/7}$

$I_B = 100(Q_{tn} + 10)/(70 + Q_{tn}F_r)$

Soils with no microstructure

Robertson, 2016

Suitable for most tailings that have little or no microstructure
**Updated SBTn Charts**

**Behavior Descriptions**

**Soil Behaviour Type**

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\[ CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17} \]

\[ I_B = 100(Q_{tn} + 10)/(70 + Q_{tn}F_r) \]

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*Soils with no microstructure*
Updated SBTn Charts

Behavior Descriptions

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1: CCS Clay-like - Contractive - Sensitive
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Soils with no microstructure

\[ CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17} \]

\[ I_B = 100(Q_{tn} + 10)/(70 + Q_{tn}F_r) \]
Updated SBTn chart

Flow Liquefaction Case Histories

Robertson, 2010

DILATIVE

Fundao

OCR > 4

Soils with no microstructure

Robertson, 2016

SD

CD

CCS

TC

TD

SC

CD

I_B = 32

I_B = 22

CD

Q_mn

Q_mn = 70

1

10

100

1000

F_r

F_r (%)
Contours of $Q_{tn,cs}$ on updated SBTn chart

Based on case histories of cyclic liquefaction

Robertson & Wride, 1998

$K_c = -0.403I_1^4 + 5.981I_1^2 - 21.63I_1 - 33.751 - 17.88$

$(q_{co},c) = K_c q_{soil}$

$\left( Q_{tn} \right)_{cs} = K_c Q_{tn}$
Contours of $s_{u(\text{liq})}/\sigma'_\text{vo}$ based on $Q_{tn,cs}$

$s_{u(\text{liq})}/\sigma'_\text{vo} = \text{liq. undrained strength ratio (sand-like)}$
Contours of $s_{u(\text{liq})} / \sigma'_v$ based on $Q_{tn,cs}$

Based on case histories where $\sigma'_v < 2$ atm

Modified Robertson (Jeffries & Boen, 2016)

Robertson (2010)

Dilative
Strain hardening

$s_{u(\text{liq})} / \sigma'_v = 0.0055 \exp(0.05Q_{tn,cs})$

Based on $Q_{tn,cs}$ (extended to $I_c > 2.6$)

Only applies to Sand-like (SC) drained CPT
$I_c < 2.6$ or $I_B > 32$

$s_{u(\text{liq})} / \sigma'_v = \text{liq. undrained strength ratio (sand-like)}$
Clay-like soils (undrained CPT) remolded strength & sensitivity ($S_t$)

Robertson (2016)

Modified for clay-like soils (when $I_c > 2.6$)

$s_{u(R)} / \sigma'_{vo} = \text{remolded undrained strength ratio for clay-like soils}$

$s_{u(R)} / \sigma'_{vo} = f_s / \sigma'_{vo} = (F_r Q_{tn})/100$
Contours of $s_{u(liq)}/\sigma'_{vo}$ & $s_{u(R)}/\sigma'_{vo}$

Modified for clay-like soils (when $I_C > 2.6$)

$s_{u(liq)}/\sigma'_{vo} =$ liq. undrained strength ratio for sand-like soils
$s_{u(R)}/\sigma'_{vo} =$ remolded undrained strength ratio for clay-like soils
Contours of $s_{u(\text{liq})} / \sigma'_v$ based on $Q_{tn,cs}$

$s_{u(\text{liq})} / \sigma'_v = \text{liq. undrained strength ratio (sand-like)}$
Contours of

\[
\frac{S_{u(\text{liq})}}{\sigma'_\text{vo}} \quad \frac{S_{u(R)}}{\sigma'_\text{vo}}
\]

Based on case histories:
- Flow liquefaction for sand-like soils
- Remolded shear strength for clay-like soils

Both large strain undrained strengths

\[
S_{u(\text{liq})}/\sigma'_\text{vo} = \text{liq. undrained strength ratio for sand-like soils}
\]
\[
S_{u(R)}/\sigma'_\text{vo} = \text{remolded undrained strength ratio for clay-like soils}
\]
Stability

• Tailings dams are becoming increasingly higher (>100m) with much higher overburden stresses (> 8 atm).
  – How do we extrapolate to higher stresses?
• Many tailings are fine grained (i.e. clay-like)
  – How do we evaluate strength loss using same framework?
Questions?

Are all contractive soils strain softening?
Are all strain softening soils brittle?
How does stress level affect these?

PBDIII Earthquake Geot. Eng. , Vancouver - 2017
“Evaluation of Flow Liquefaction: Influence of high stresses” P.K. Robertson
Critical State Soil Mechanics - Framework

Clay-like soils

Sand-like soils

Sands become more contractive with increasing stress

$Spacing \quad r = p'_{NC}/p'_{CS} \sim 2 \text{ to } 4$

Modified from Jefferies and Been, 2016
Clay-like soils

Sand-like soils

Spacing

$r = \frac{p'_{NC}}{p'_{CS}} \sim 2$ to $4$

Critical State Soil Mechanics - Framework

Modified from Jefferies and Been, 2016
CSSM - Summary

• Clay-like soils
  – Unique normal consolidation line (NCL)
  – NCL parallel to CSL with $p'_{nc} / p'_{cs} \sim 4$
  – State defined by OCR (relative to NCL)
    • OCR $> 4$ mostly dilative at large strains

• Sand-like soils
  – Non-unique normal consolidation lines
  – State defined relative to CSL
    • either $\psi$ or $p'_o / p'_cs$
  – At high stress LCC parallel to CSL:
    • $p'_{LCC} / p'_{cs} \sim 4$
State parameter in sand-like soil

\[ p'_o / p'_cs = 10^{(\psi/\lambda)} \]

\[ \psi = e_o - e_{cs} \]

(+) Loose
CONTRACTIVE

(-) Dense
DILATIVE

Critical State Line
(CSL)

State Parameter after Been and Jefferies, 1985
**Brittleness (strength loss index)**

Bishop (1967) defined brittleness - *strength loss index*:

\[ I_B = \frac{(\tau_p - \tau_l)}{\tau_p} \]

\( \tau_p = \) peak strength

\( \tau_l = \) large strain (or liquefied) strength at same effective normal stress

\( I_B = 1.0 \) (100% strength loss)

\( I_B = 0 \) (no strength loss)
Critical State Lines (CSL)

Contractive
Dilative

Jefferies and Been, 2016

Limited stress range

Linear approximation for CSL

CSL for range of sands
Critical State Lines (CSL)

Schnaid et al., 2013

Verdugo & Ishihara, 1996

CSL over wide stress range

Contractive

Dilative

CSL non-linear over wide stress range

Silty tailings

Tovoura sand

Schnaid et al., 2013

Verdugo & Ishihara, 1996
Bolton (1986) Relative Dilatancy Index

Increasing compressibility/crushability

Boulanger 2003

Schnaid et al, 2013

Verdugo & Ishihara, 1996
This format is very helpful to estimate CSL in sand-like soils

Bolton’s empirical relationship is helpful guide
Example – Erksak Sand

\[ \Psi = 0.07 \]
\[ \frac{p'_o}{p'_{cs}} = 10^{(\Psi/\lambda)} \]

\[ \Psi = 0.20 \]
\[ \frac{p'_o}{p'_{cs}} = 4.5 \]

\[ \Psi = 0.25 \]
\[ \frac{p'_o}{p'_{cs}} = 3.5 \]

\[ M_{tc} = 1.2 \]

Path A: Contractive

Path B: Dilative

Path C: Loosest state

Erksak Sand

(data from Jeffries and Been, 2016)
Strength loss index \( (I_B) \) vs \( (p'_o/p'_cs) \)

\[ I_B = 0.9 - 0.84(p'_o/p'_cs) \]

Regardless of fabric and direction of loading

Increasing strength loss

\( \sim \) NC Clay

\[ p'_o/p'_cs \]

Modified from Sadrekarimi & Olson, 2011
Strength loss index \( (I_B) \) vs \( (p'_o/p'_cs) \)

Can use \( \psi \) but requires slope of CSL \( \lambda \) \( (\psi/\lambda) \) which is changing

\[
I_B = 0.9 - 0.84(p'_o/p'_cs)
\]

Increasing strength loss

\( \sim \) NC Clay

Modified from Sadrekarimi & Olson, 2011
Example in-situ consolidation curves

- Medium dense sand will become contractive at high stress but with little potential strength loss.
- Very loose sand at low stress has highest potential for strength loss.
Soil-like behavior

Flows like a liquid at large strains

\[ S_{u(l)} \approx 2 \text{kPa} \]

Clay-like (silt) tailings
(PI ~ 10%)

\[ S_{u(p)} > 5 \text{kPa} \quad (S_{u(p)} = 0.25 \sigma'_v) \]

Clay-like behavior

\[ S_{u(p)} > 5 \text{kPa} \]

Slurry consolidation
Clay-like (silt) tailings (PI ~ 10%)

- Flows like a liquid
- Slurry consolidation
- Vertical Effective stress, $\sigma_v'$ (kPa)
  - $S_{u(p)} > 5$ kPa ($S_{u(p)} = 0.25 \sigma'_{vo}$)
  - Liquid Limit
  - $p'_{o}/p'_{cs} \sim 5$

Soil-like behavior
- $S_{u(l)} \sim 2$ kPa

Significant strength
- Loss $I_B > 0.4$

Small strength
- Loss $I_B < 0.4$

Water content

CLAY-LIKE

CSL

NCL
Strength loss index ($I_B$) vs $s_{u,cs}/\sigma'_{vo}$

Modified from Sadrekarimi & Olson, 2011
Strength loss index \((I_B)\) vs \(s_{u,cs}/\sigma'_v\)

Modified from Sadrekarimi & Olson, 2011

Case histories

Low Brittleness

High Brittleness

A

B

C
Strength loss index ($I_B$) vs $s_{u,cs}/\sigma'_vo$

$s_{u,cs}/\sigma'_vo < 0.15$ have higher strength loss $I_B > 0.4$

Case histories

Modified from Sadrekarimi & Olson, 2011
\[ Y = -0.05 \]

\[ s_{u(\text{liq})} / \sigma'_{vo} = \text{liq. undrained strength ratio for sand-like soils} \]

\[ s_{u(R)} / \sigma'_{vo} = \text{remolded undrained strength ratio for clay-like soils} \]

Based on case histories:
- Flow liquefaction for sand-like soils
- Remolded shear strength for clay-like soils

Both large strain undrained strengths

Contours of

\[ S_{u(\text{liq})}/\sigma'_{vo} \]

\[ S_{u(R)}/\sigma'_{vo} \]
Sand-like and Dilative – SD

- Potential for cyclic liquefaction – depends on size and duration of cyclic loading
- CRR will increase with increasing static shear
- Sampling difficult & in-situ testing preferred
- Estimate CRR based on $Q_{tn,cs}$
- No strength loss expected unless state changes and/or some microstructure (e.g. cementation)

$s_{u(\text{liq})} / \sigma_{\text{vo}}' = \text{liq. undrained strength ratio for sand-like soils}$

$s_{u(R)} / \sigma_{\text{vo}}' = \text{remolded undrained strength ratio for clay-like soils}$
Liquefaction Summary

Contours in SC/TC region based on $\sigma_{vo}' < 2$ atm

Sand-like and Contractive – SC
- Potential for cyclic & flow liquefaction
- CRR will decrease with increasing static shear
- Sampling difficult & in-situ testing preferred
- Strength loss possible
- Estimate state ($\psi$) and $s_{u(\text{liq})}/\sigma_{vo}'$ based on $Q_{in,cs}$
- Strain to trigger strength loss can be small

$s_{u(\text{liq})}/\sigma_{vo}' = \text{liq. undrained strength ratio for sand-like soils}$
$s_{u(R)}/\sigma_{vo}' = \text{remolded undrained strength ratio for clay-like soils}$
**Liquefaction Summary**

- **Clay-like and Contractive – CC/CCS**
  - Potential for cyclic softening & flow liquefaction
  - CRR can decrease with increasing static shear
  - Sampling possible and recommended
  - Strength loss possible
  - Estimate state (OCR) and $s_u(R)/\sigma'_{vo}$ based on CPT & FVT
  - Strain to trigger strength loss can be large – depends on plasticity
  - Evaluate sensitivity using CPT $f_s$ and FVT
  - Check drainage CPTu dissipation tests
  - Measure $w_n$ and Atterberg

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$S_u(\text{liq})/\sigma'_{vo} = \text{liq. undrained strength ratio for sand-like soils}$

$s_u(R)/\sigma'_{vo} = \text{remolded undrained strength ratio for clay-like soils}$
Clay-like and Dilative – CD

- Liquefaction unlikely
- Sampling possible, if not too stiff
- No strength loss expected unless state changes and/or some microstructure
- Measure $w_n$ and Atterberg
- Drained strength valid

\[ \frac{s_u(\text{liq})}{\sigma'_{vo}} = \text{liq. undrained strength ratio for sand-like soils} \]

\[ \frac{s_u(R)}{\sigma'_{vo}} = \text{remolded undrained strength ratio for clay-like soils} \]
Soils with no microstructure

Contours of $s_u(\text{liq})/\sigma'_v = \text{liq. undrained strength ratio for sand-like soils}$

$s_u(\text{R})/\sigma'_v = \text{remolded undrained strength ratio for clay-like soils}$

**Liquefaction Summary**

**Transitional – TD/TC**

- Evaluate plasticity and drainage (Atterberg Limits & CPTu dissipation tests)
- Sampling possible depending on plasticity and fines
- Evaluate assuming both sand-like and clay-like and compare
  - typically drained strengths are more conservative in SD, SC, TD, TC and CD
Conclusion (1)

- Flow liquefaction (strength loss) can be triggered by relatively minor conditions and ‘unloading’ stress paths are the most critical, e.g.
  - Earthquakes (even small earthquakes)
  - Rising piezometric surface
  - Unloading (movements in foundations or tailings)
Conclusion (2)

- Not all CONTRACTIVE soils are strain softening in undrained shear
- Not all strain softening soils are have significant strength
  - strength loss (and ductility) varies with state, stress level and PI
- Although soils tend to become more contractive with increasing stress they also tend to become more ductile (i.e. larger strain before any strength loss)
- Low stress regions maybe most critical
Conclusion (3)

• SCPTu an excellent tool to evaluate potential risk of flow liquefaction
  – 5 to 6 measurements
  – Ability to identify microstructure
  – Classify soil behavior
  – $V_s$ and $V_p$ powerful additional measurements

• Lab. testing to helpful to support evaluation via CSL
Importance of in-situ water content

- In-situ water content in saturated clay-like soils a useful measure of in-situ void ratio
- Soil-moisture probe is a potentially useful device added to CPTu to measure water content profile with depth based on dielectric measurements
- Likely needs site specific correlation to modify basis empirical correlations to improve accuracy
Thank You

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