Seismic Design Recommendations
Liquefaction of Ponded Ash
Field and Laboratory Characterization
Recommendations to Evaluate Embankment Stability for Seismic Loads

William Wolfe – The Ohio State University
Robert Bachus – Geosyntec Consultants
Welcome….Lovers of Fly Ash

We are a Band of Brothers (and Sisters) who share a passion (or at least an interest) in fly ash or CCRs

http://jonvilma.com/band-of-brothers.html

However, for the last 10 years, we have also been jumping over hurdles and beating our heads against the wall as we try to understand how to best close CCR ponds

http://www.alamy.com/stock-photo

https://bashny.net/t/en/6016
Welcome….Lovers of Fly Ash

Why, the frustration? Could be tricky material or tricky regulations, as we try to separate the realities from perceptions of CCRs.

And through it all, we often feel what Hester Prynne must have felt … eternally branded with a Scarlett Letter.


Our goal over the next 50 minutes is to try in some measure to address the perception and demonstrate the reality of CCRs and seismic liquefaction.

www.wikipedia.org
Liquefaction of Ponded Ash

- Liquefaction occurs when stresses and deformation in the ground caused by earthquake shaking disturb the soil structure.
- Water in the pore spaces will resist the natural tendency of the soils to consolidate into a denser and more stable arrangement during shaking.
- Because the soil cannot change in volume until water is drained from the pore spaces during the earthquake, porewater pressure will rise, soil particles may lose contact with each other, and the soil mass may lose much of its strength.

Why Is Liquefaction Important when We Consider Ponded Ash?

Earthquake induced liquefaction is a leading cause of earthquake damage worldwide.

But before we discuss liquefaction and seismic design of CCR ponds, we need to understand some of the basic characteristics and realities of ponded ash.

State of the Art and Practice in the Assessment of Earthquake-induced Soil Liquefaction and Its Consequences (National Academies of Science, Engineering, Medicine, 2016)
Field Methods
• Standard Penetration Test (SPT)
• Cone Penetration Tests (CPTu)
• Geophysical Test for Shear Wave Velocity ($V_s$)

Laboratory Methods
• Cyclic Triaxial Shear (CyTXC)
• Cyclic Direct Simple Shear (CyDSS)

We will discuss some of these methods to characterize the properties of ponded fly ash
Presentation Outline

• Basic Material Characterization
• Laboratory Strength Characterization
  • Static and Dynamic
• Field Characterization
• Seismic Design Recommendations
This work would not be possible without the encouragement, collaborative, and financial support from a number of different organizations and individuals.

Ken Ladwig – Electric Power Research Institute (EPRI)
Mehdi Maibodi – Duke Energy
Pedro Amaya – American Electric Power
National Ash Management Advisory Board (NAMAB) – Duke Energy
Tarunjit Butalia – The Ohio State University
Mehedy Amin – The Ohio State University
Carlos Santamarina – Georgia Institute of Technology (now KAUST)
Greg Hebeler – Golder Associates
Kula Kulasingam – AECOM
Ken Daly – AMEC Foster Wheeler
Students at Ohio State and Georgia Tech who performed the tests cited in this presentation
Basic Material Characterization
**Material Characterization**

- Material chemistry
- Particle size
- Specific gravity
- Particle shape
- Plasticity
- Pore fluid-solid interaction

**Engineering Properties**

- Realizable density
- Sample preparation
- In-pond layering
- Compressibility
- Drained strength
- Undrained strength
- Static liquefaction
- Seismic liquefaction
- Flow potential
Material Characterization

- Material chemistry
- Particle size
- Specific gravity
- Particle shape
- Plasticity
- Pore fluid-solid interaction

Engineering Properties

- Realizable density
- Sample preparation
- In-pond layering
- Compressibility
- Drained strength
- Undrained strength
- Static liquefaction
- Seismic liquefaction
- Flow potential
Notes:
- Height of the box = 2 times standard deviation
- Length of black lines: from maximum to minimum value found in literature
Magnetically separated fraction:

Hematite Fe₂O₃ (weakly magnetic), Magnetite Fe₃O₄ and Maghemite Fe₂O₃ (both strongly magnetic).
Particle Size Distribution

Diameter $d$ [mm]

% Passing

- US
- Asia
- Europe

- Bin-shafique (2002)
- Palmer (2000)
- Das and Yudhbir (2006)
- Reyes (2007)
- Senol (2001)
- Tu (2007)
- Torrey (1978)
- EPRI (1993)
- EPRI (1995)

- Cousens (2003)
- Reyes (2007)
- Premchitt (1995)

Clay
Fine gravel
Medium sand
Fine sand
Coarse sand
Silt
Silt-sized

$R = 0.1 \mu$
$R = 1 \mu$
$R = 10 \mu$
Specific Gravity of Solids

Water based $G_s$ [-]

1.6 1.8 2.0 2.2 2.4 2.6 2.8

Benzyl Alcohol $G_s$ [-]

1.6 1.8 2.0 2.2 2.4 2.6 2.8

$G_s$ [-]

1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2

Frequency

1 2 3 4 5 6 7 8 9 10 11 12 13

Europe

Asia

US

this study


Note shape and size of “holes” (recall clay-sized = 2 μm)
How to fit 3,000μm drop in a 2μm hole??

Water drop = 1/8 inch = 3,000μm
Material Characterization

- Material chemistry
- Particle size
- Specific gravity
- Particle shape
- Plasticity
- Pore fluid-solid interaction

Engineering Properties

- Realizable density
- Sample preparation
- In-pond layering
- Compressibility
- Drained strength
- Undrained strength
- Static liquefaction
- Seismic liquefaction
- Flow potential
## Methods

<table>
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<th>Formation under shear</th>
<th>Dry Density</th>
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<td>880</td>
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<th>Dry Density</th>
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<th>Dry Density</th>
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<td>$G_S$: 2.26</td>
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<th>Dry Density</th>
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<th>Standard compaction test</th>
<th>Dry Density</th>
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<td>$G_S$: 2.26</td>
</tr>
<tr>
<td></td>
<td>1285</td>
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</table>
Application of Realizable Density

Realizable Density

0.00  0.20  0.40  0.60  0.80  1.00

contractive

dilative

circles: undrained
triangles: drained
Application of Realizable Density

realizable density

contractive

dilative

circles: undrained

triangles: drained

Target

Realizable Density

0.00  0.20  0.40  0.60  0.80  1.00
Laboratory Sedimentation - Photograph

Field Sedimentation - Radiograph
Evidence of Layering - Laboratory
this study

<table>
<thead>
<tr>
<th>Compressibility</th>
<th>Montmorillonite</th>
<th>Illite</th>
<th>Kaolinite</th>
<th>Sand</th>
</tr>
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<tbody>
<tr>
<td>Cc</td>
<td>0.00</td>
<td>0.01</td>
<td>0.10</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Kaniraj (2004)  
Pandian (2004)  
Premchitt (1995)  
Tu (2007)  
Pandian (2004)
Laboratory Strength Characterization Static
Critical state

$\phi_{\text{crit}} = 28.8^\circ$

Peak state

$\phi_{\text{peak}} = 31.0^\circ$

Table 4.1 Representative Values of $\phi_d$ for Sands and Silts

<table>
<thead>
<tr>
<th>Material</th>
<th>Degrees</th>
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<tbody>
<tr>
<td></td>
<td>Loose</td>
</tr>
<tr>
<td>Sand, round grains, uniform</td>
<td>27.5</td>
</tr>
<tr>
<td>Sand, angular grains, well</td>
<td>33</td>
</tr>
<tr>
<td>graded</td>
<td></td>
</tr>
<tr>
<td>Sandy gravel</td>
<td>35</td>
</tr>
<tr>
<td>Silty sand</td>
<td>27–33</td>
</tr>
<tr>
<td>Inorganic silt</td>
<td>27–30</td>
</tr>
</tbody>
</table>

(after Peck, Hanson, and Thornburn, 1953)
Undrained Triaxial Strength

Effective mean stress, $p = (\sigma_1' + \sigma_3')/2$ [kPa]

Deviatoric stress, $q = (\sigma_1 - \sigma_3)/2$ [kPa]

$e = 0.87$
$e = 0.80$
$e = 0.54$
$e = 0.48$
$e = 0.42$

$\phi = 30^\circ$
$\phi = 35^\circ$

Series 1
Series 2

Constant Unit Weight

Constant Initial Consolidation Pressure
Shear Stress vs. Effective Vertical Stress

- Site A, S-8, Depth = 32.75 ft
- Site B, AB-102, Depth = 11.15 ft
- Site C, DB-101, Depth = 37.10 ft
- Site D, SB-105A, Depth = 18.0 ft
- Site D, SB-134, Depth = 16.15 ft
- Site E, SB-214, Depth = 11.90 ft
- Site F, AP-9, Depth = 29.70 ft
- Site G, SD-4-OS, Depth = 25.35 ft
- Site G, SD-4-OS, Depth = 25.65 ft
- Site G, SD-4-OS, Depth = 25.8 ft
- Site G, EB-SPT01*, Depth = 9.2 ft
- Site G, EB-SPT01*, Depth = 9.05 ft

φ = 34 to 36° (after AECOM, 2016)
Laboratory Strength Characterization Dynamic
The Ohio State University has been actively involved for the past several years to develop a large database of dynamic triaxial test results on behalf of the American Electric Power Company (AEP).

**Objectives of Laboratory Study**

- Develop sample preparation techniques to mimic the field sedimentation process of a ponded fly ash.
- Conceptually, this implies that the specimens are formed by when saturated ash is deposited under water. The process is termed “wet pluviation.”
- Develop, refine, and document a specimen preparation process that produces repeatable specimen characteristics at target realizable density.
- Perform a series of undrained cyclic triaxial tests on a wide range of fly ashes from across the AEP portfolio.
- Compile and synthesize the test results.
Specimen Preparation Process

- Split mold (2.8" x 6")
- Latex membrane (0.012-in. thick)
- Vacuum applied to stretch membrane
- Mold filled with water

Premeasured amount of fly ash placed in flask

Flask then filled with water
Specimen Preparation Process

- Wet pluviation from flask to split mold
- Vibrate flask to prevent clogging
- Near the end of wet pluviation
Specimen Preparation Process

Specimen after removal of flask

Apply vacuum to specimen

Specimen after removal of split mold
Specimen in triaxial chamber for consolidation

Cyclic testing of consolidated specimen
## Results of Specimen Preparation

### Realizable Density

<table>
<thead>
<tr>
<th>Site ID</th>
<th>Realizable Density</th>
<th>Specimen Density, $\gamma d_{specimen}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma d_{min}$ (pcf)</td>
<td>$\gamma d_{max}$ (pcf)</td>
</tr>
<tr>
<td>GL</td>
<td>45.2</td>
<td>61.2</td>
</tr>
<tr>
<td>AS</td>
<td>70.1</td>
<td>86.0</td>
</tr>
<tr>
<td>SE</td>
<td>58.2</td>
<td>85.5</td>
</tr>
<tr>
<td>BY</td>
<td>69.1</td>
<td>96.2</td>
</tr>
</tbody>
</table>

**Realizable Density, $D_r (%) = \frac{\gamma d_{specimen} - \gamma d_{min}}{\gamma d_{max} - \gamma d_{min}} \times 100$**

Minimum realizable density after EPRI (2012)
Maximum realizable density after ASTM D698-12
Cyclic Liquefaction Test Results
Site A - S-8 (ST-5) - 30-32’ (95% Fines), $\sigma' = 2.16$ ksf (15 psi)

Site B - AB-102 (PT-1) - 10-12’ (53% Fines), $\sigma' = 2.16$ ksf (15 psi)

Site C - DB-101 (36'-38’) and DB-102 (21’-23’) (19% to 24% Fines), $\sigma' = 1.87$ ksf (13 psi)

Site D - SB-105A - ST-1 - 18’-20’ (49% Fines), $\sigma' = 2.02$ ksf (14 psi)

Site D - SB-134 - ST-2 - 15’-17’ (79% Fines), $\sigma' = 4.32$ ksf (30 psi)

Site E - SB-214 (ST-7) - 10-12’ (14% Fines), $\sigma' = 2.16$ ksf (15 psi)

Site F - AP-9 (PS-8) - 28’-30’ (99% Fines), $\sigma' = 2.16$ ksf (15 psi)

Site G - SD-6 (ST-5) - 39-41’ (92% Fines), $\sigma' = 2.16$ ksf (15 psi)

Site G - EB-SPT08 (ST-5) - 10-12’ (59% Fines), $\sigma' = 2.16$ ksf (15 psi)

Wolfe (Water Pluviated Samples), $\sigma' = 2.88$ ksf (20 psi)

Testing Terminated at 500 Cycles

(after AECOM, 2016)
Dynamic Undrained Results
Triaxial and DSS

Source:
Wolfe (2016) - fly ash
deAlba (1976) - sand
Field Characterization
Cone Penetration Test (CPTu)

www.lankelma.co.uk

http://www.geotechdata.info/geotest/cone-penetration-test.html
CPTu Test Results

(after AECOM, 2016)
CPTu Test Interpretation

(after AECOM, 2016)
Current SOP for soils
  - Based on Robertson (2010)
  - Converts $q_t$ to $Q_{tn,cs}$
  - Estimate strengths using case histories

Potentially too conservative for CCRs?

(after Golder, 2017)
Vane Shear Test - VST

Field Vane Shear Test (VST) deployed at depth

http://www.greggdrilling.com/canada/services/cFQUY/vane-shear-testing

Portable VST deployed at Shallow Depth

www.itlineels.com

(http://geonor.no/en/soil-testingmed-vingebor/

(after discussion with Golder, 2017)

(after discussion with CALM, 2016)
Field Vane Shear Test (VST) deployed at depth

Typical VST Results

http://english.geocpt.es/Vane_Test.htm
Stacked Ash
Generally achieves good strengths & is not prone to liquefaction

Sluiced Ash then dried
CRR that remains dry also exhibits significant peak strengths but maintains low residual & post liquefied strengths

Sluiced Ash but still wet
CCR typically exhibits residual strengths close to NC behavior and very small strength increases with depth (may form lightly cemented structure)

(after Golder, 2017)
Excavated and Dewatered Ash Stands in Vertical Sidewalls

After Transportation (i.e., Vibrations) Material Can Flow

(after Golder, 2017)
Flow/Slump Potential - Laboratory
No Free Water and Unsupported: Material will Slump

Vibration in Absence of Free Water: Material can Flow and Impact Workability
Importance of Eliminating Water

(after AECOM, 2016)
Draining of free water is “easy” and yields large quantities of water.
Eliminating free water is essential to reducing flow potential!
Dewatering the ash may lower the phreatic surface, but may yield small amounts of water (e.g. < 10% to ~30% of ash volume) depending on the particle size of the ponded materials, especially when compared to the drained volume.
Dewatering the ash will not significantly (e.g., >10%) alter the “moisture content” of the ponded ash, but will likely be sufficient to allow vertical cuts for construction and benefit liquefaction potential.
Dewatering may NOT “dry” most CCRs to a level sufficient for compaction.
Important to express the goal of dewatering to manage expectations.
Seismic Design Recommendations

State of the Art and Practice in the Assessment of Earthquake-induced Soil Liquefaction and Its Consequences (National Academies of Science, Engineering, Medicine, 2016)
East Coast Seismicity
Mineral, VA Earthquake (23 August 2011)

Magnitude ~5.8

Landslides noted within 150 miles of epicenter

Evidence of liquefaction?
Map showing areas where there is a 2% chance in 50 years that peak ground acceleration will attain or exceed 0.1g
(modified from Petersen et al., 2014)

Ground shaking is presumed to be intense enough to cause liquefaction in water-saturated granular materials.

(after https://www.usgs.gov)
Peak Ground Acceleration Map

Two-percent probability of exceedance in 50 years map of peak ground acceleration

(after https://www.usgs.gov)
Regional Seismicity Map

U.S. Seismic Hazard
2% in 50 years PGA Hazard (%g)

- 0-2
- 2-4
- 4-6
- 6-10
- 10-14
- 14-20
- 20-25
- 20-40
- 40-60
- >60

Plates
- Subduction
- Transform
- Divergent
- Others
- Counties

(after https://www.usgs.gov)
Seismic Design Recommendations

Input Earthquake Motions: To commence a seismic design, one must first select a series of input ground motions that are “appropriate” for their site.

How to make this selection for East Coast seismic events where records for large events are often not available. This has been an age-old recognition, but one that we have found ways of resolving.

We first identify a recurrence interval that we use for our design. This will define the peak ground acceleration (PGA) in bedrock that is used for design. We then find a suite of historic ground motions that exhibit the frequency content and scale these records to the design PGA.

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Profile</th>
<th>Input Motion</th>
<th>Peak Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>A-A'</td>
<td>1952 Taft</td>
<td>0.08</td>
</tr>
<tr>
<td>A2</td>
<td>A-A'</td>
<td>1952 Taft</td>
<td>0.15</td>
</tr>
<tr>
<td>A3</td>
<td>A-A'</td>
<td>1940 El Centro</td>
<td>0.08</td>
</tr>
<tr>
<td>A4</td>
<td>A-A'</td>
<td>1940 El Centro</td>
<td>0.15</td>
</tr>
</tbody>
</table>
Fly Ash Pond Example

Location of Site: Eastern U.S.

Depth of Ash in Pond: 158 ft

Clay Cover at Top of Profile: 10-ft thick

Drainage Layer below Clay Cover: 8-ft thick

Bedrock Layer at Base of Site: Sandstone Bedrock

Groundwater Table: 20 ft. below top of ash (z = 38 ft.)
Seismic Design Recommendations
Idealized Ash Pond Profile

Section A-A’
- Subdivide pond into discrete analysis layers (i.e., ten (10) layers in this example)
- Apply bedrock input motion at base of profile
- Propagate wave from the base through the ash column
- Calculate the response (i.e., shear stress) of the ash due to the propagation shear wave
## Seismic Design Recommendations

### Properties for Layers

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Material Description</th>
<th>Thickness (ft)</th>
<th>Unit Weight (pcf)</th>
<th>G$_{max}$ (ksf)</th>
<th>V$_s$ (ft/sec)</th>
<th>Modulus Reduction</th>
<th>Damping Curve</th>
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<tbody>
<tr>
<td>1</td>
<td>Recompacted Clay Liner</td>
<td>10</td>
<td>125</td>
<td>3,885</td>
<td>1,000</td>
<td>Clay (Seed and Sun 1989)</td>
<td>Clay (Idriss 1990)</td>
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<tr>
<td>2</td>
<td>Drainage layer</td>
<td>8</td>
<td>125</td>
<td>3,300</td>
<td>921</td>
<td>Sand (Seed &amp; Idriss) - Average</td>
<td>Sand (Seed &amp; Idriss) Average</td>
</tr>
<tr>
<td>3</td>
<td>Fly ash</td>
<td>10</td>
<td>100</td>
<td>1,000</td>
<td>567</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
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<tr>
<td>4</td>
<td>Fly ash</td>
<td>10</td>
<td>100</td>
<td>1,200</td>
<td>621</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
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<tr>
<td>5</td>
<td>Fly ash</td>
<td>18</td>
<td>100</td>
<td>1,400</td>
<td>671</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
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<tr>
<td>6</td>
<td>Fly ash</td>
<td>30</td>
<td>100</td>
<td>1,600</td>
<td>717</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
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<tr>
<td>7</td>
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<td>30</td>
<td>100</td>
<td>1,800</td>
<td>761</td>
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<tr>
<td>8</td>
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<td>30</td>
<td>100</td>
<td>1,900</td>
<td>781</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
</tr>
<tr>
<td>9</td>
<td>Fly ash</td>
<td>30</td>
<td>100</td>
<td>2,000</td>
<td>802</td>
<td>Sand (Seed and Idriss 1970)</td>
<td>Sand (Idriss 1990)</td>
</tr>
<tr>
<td>10</td>
<td>Sandstone</td>
<td>Infinite</td>
<td>140</td>
<td>135,360</td>
<td>5,577</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>
Propagation of shear wave from bedrock through the overlying saturated ash is calculated using established computer simulation programs. In this case, we will use the one-dimensional wave propagation program SHAKE. Other programs are now available as freeware (primarily developed by university faculty).


The following assumptions are implied in the analysis:
1. The soil system extends infinitely in the horizontal direction
2. Each layer in the system is completely defined by its value of shear modulus, critical damping ratio, density, and thickness. These values are independent of frequency.
3. The responses in the system are caused by the upward propagation of shear waves from the underlying rock formation.
4. The shear waves are given as acceleration values of equally spaced time intervals. Cyclic repetition of the acceleration time history is implied in the solution.
5. The strain dependence of modulus and damping is accounted for by an equivalent linear procedure based on an average effective strain level computed for each layer.
6. The program is able to handle systems with variation in both moduli and damping and takes into account the effect of the elastic base.
7. The motion used as a basis for the analysis, the object motion, can be given in any one layer in the system and new motions can be computed in any other layer.
Taft EQ Input at Top of Rock

Time History of Shear Stress

\( a_{\text{max}} = 0.08g \)

Shear Stress (psf)

Time (sec)

Layer: 3 - EQ No: 1 - Outcrop: No

Peak Shear Stress

Depth (ft)

Shear Stress (psf)

Layer: 3 - EQ No: 1 - Outcrop: No

Number  Description  Motion  Output
---  ---------  ------  -----  
1     Matured Clay Liner  Horizontal  
2     Sandstone Layer  Vertical  
3     Pipe A  Horizontal  
4     Pipe B  Vertical  
5     Pipe C  Horizontal  
6     Pipe D  Vertical  
7     Pipe E  Horizontal  
8     Pipe F  Vertical  
9     Block Sandstone  
10    Sandstone  

\( amax = 0.15g \)
El Centro EQ Input at Top of Rock

Time History of Shear Stress

$\alpha_{max} = 0.08\text{g}$

Shear Stress (psf) vs. Time (sec)

Layer: 3 - EQ No: 1 - Cut crop: N

Peak Shear Stress

$\alpha_{max} = 0.15\text{g}$

Shear Stress (psf) vs. Depth (ft)

Layer: 3 - EQ No: 1 - Cut crop: N

Descriptions:
- Compacted Clay Lint
- Overlying Layer
- Fine Sand
- Fine Sand
- Sandstone

Shear Stress (psf) vs. Depth (ft)

Layer: 3 - EQ No: 1 - Cut crop: N
Results of Ground Response Analysis

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Layer #</th>
<th>Depth (ft)</th>
<th>$\sigma_0' \text{ (psf)}$</th>
<th>$\tau_{\text{max}} \text{ (psf)}$</th>
<th>$\tau_{\text{cyc}} \text{ (psf)}$</th>
<th>$N_{\text{equ}}$</th>
<th>CSR field</th>
<th>CSR lab</th>
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Noted Modifications

1. $\tau_{\text{cyc}} = \tau_{\text{max}}$

2. $(\text{CSR})_{\text{field}} = 0.9 \times (\text{CSR})_{\text{lab}}$

3. $(\text{CSR})_{\text{DSS}} = 0.65 \times (\text{CSR})_{\text{TXC}}$
Dynamic Undrained Results
Triaxial and DSS

Source:
Wolfe (2016) - fly ash
deAlba (1976) - sand
Cyclic Triaxial Tests

No liquefaction after 500 cycles

- Light Compaction (85%) @ $\sigma_{\text{cell}} = 20$ psi
- Medium Compaction (95%) @ $\sigma_{\text{cell}} = 10, 20, 50$ psi
- Heavy Compaction (105%) @ $\sigma_{\text{cell}} = 20$ psi

- 85% Compacted with 20 psi effective confining
- 95% Compacted with 20 psi effective confining
- 105% Compacted with 20 psi effective confining
- 85% Compacted with 10 and 50 psi effective confining
SHAKE Analysis Results

The image shows a graph titled "Liquefaction Potential of F.A. Samples". The graph plots CSR (Cyclic Stress Ratio) on the y-axis against No. of Cycles to Liquefaction on the x-axis. The graph includes lines representing different compaction levels:

- Light compaction ($\sigma_c = 20$ psi)
- Medium compaction ($\sigma_c = 20$ psi)
- Heavy compaction ($\sigma_c = 20$ psi)
- Medium compaction ($\sigma_c = 10$ psi)

The text indicates that predicted and adjusted CSRs are shown from the SHAKE analysis results.
Overall Observations
• Fly ash under cyclic loads behaves similarly to natural granular fine grained soils.
  • For a given relative density, the number of loading cycles to produce liquefaction decreases with increasing shear stress ratio (CSR); and
  • For a given shear stress ratio (CSR), liquefaction resistance increases with increasing relative density

• Cyclic strength, expressed as CRR, falls within the range of strength values measured for fine sands and silts

• The factor of safety against liquefaction depends on the compactive effort, cyclic stress ratio, and initial confining pressure.

Specific Observations
• For the example problem, the measured cyclic strength (CRR) of the tested material was determined to be greater than the loading imposed by the two design earthquakes (CSR).
Presentation Outline

• Basic Material Characterization
• Laboratory Strength Characterization
  • Static and Dynamic
• Field Characterization
• Seismic Design Recommendations
Basic Material Characterization

- Ponded fly ash is characterized by a relatively consistent mineral chemistry that does not “react” chemically with water and unique particle shape.
- Fly ash generally exhibits low specific gravity of solids relative to mineral soils, thus relatively low unit weight, and is best characterized as a well-graded silt.
- Fly ash is typically low- to non-plastic and as such the properties are very sensitive to even subtle changes in moisture content.
- Small particle size leads to large surface tension that “holds onto water” when partially saturated and controls the permeability.
- Recognizing the concept of “realizable density” shows that wet pluviation (similar to the depositional conditions in an ash pond) does not results in an extremely loose structure, but does result in layering.
- Because there is grain-to-grain contact, the ponded fly ash is only moderately compressible, and the permeability is relatively high.
- Apparent “unusual” behavior of ponded fly ash is somewhat compounded by the fact that it is unusual to encounter thick strata of silt-sized particles in most geotechnical practices.
Laboratory Strength Characterization
• Sample preparation is a key and must model the depositional process in the field to be used in design
• Static strength of ponded fly ash is consistent with that anticipated (and measured) for granular materials and generally indicates dilative behavior
• Cyclic response of ponded fly ash is consistent with the cyclic response of sands

Field Characterization
• Dewatered fly ash is NOT dry fly ash, due to the high capillary tension potential of the well-graded silty fly ash
• The layering of coarse and fine CCRs contribute beneficially to dewatering, but can contribute free water that can lead to rapid destabilization
• Standard penetration test (SPT) blow counts are very low and there are not any reliable “correlations” to engineering properties
• Cone penetration test (CPTu) profiling seems to be best technique for characterizing CCR ponds. We are still learning how to best utilize the correlations developed for “conventional” mineral soils
• Vane shear test (VST) assessment, while not typically used in silts, seems to provide better assessment of residual strengths relative to correlations from CPTU soundings
Seismic Design Recommendations
- Seismic design example to assess liquefaction resistance was developed and presented
- Techniques developed to assess liquefaction resistance for sands is believed to be appropriate for ponded fly ash
- There may be some inconsistent observations/conclusions when compared to CPTu correlations for liquefaction potential
Conclusions

http://jonvilma.com/band-of-brothers.html

Hopefully, this presentation will allow our Band of Brothers and Sisters to remain strong, informed, and vigilant....

... such that we can proudly wear the Scarlett Letter

Thank you for your interest and participation

www.wikipedia.org