Ground Improvement and Liquefaction Mitigation using Driven Timber Piles

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Presentation Outline

- Introduction and motivation
- Research program
- Experimental field test program
  - Selection and characterization of test site
  - Ground improvement test program in-situ tests
  - Controlled blasting program
- Numerical and Analytical Investigation
- Summary and Conclusions
Introduction and Motivation

- Liquefaction-susceptible soils: saturated, loose to medium dense, granular and slightly plastic soils
- Earthquake-induced ground motions, if strong enough or if providing sufficient number of shear stress cycles, can produce liquefaction
- Definition (with excess pore pressure): $r_u = \frac{u_e}{\sigma'_{vo}} = 1.0$
  
  Note: this definition not quite correct...
- Consequences of liquefaction:
  - Flotation of underground structures
  - Excessive settlement and tilting of structures
  - Ground failure: lateral spreading, flow failure
Introduction and Motivation

Ground Improvement Methods

- **Densification**
  - vibro-compaction and vibro-replacement (stone columns)
  - dynamic compaction
  - compaction grouting
  - blasting
  - displacement piles

- **Reinforcement**
  - vibro-replacement (stone columns)
  - deep soil mixing / jet grouting
  - driven piles or drilled shafts

- **Drainage**
  - earthquake drains
  - stone columns (?)
Outstanding Questions: Densification?

- Plantema and Nolet (1957), Meyerhof (1959), Broms (1966):
  - Showed that displacement piles effectively densified granular soils
  - Loose sand densified 3.5 to 5 pile diameters away from the pile
  - Cone tip penetration resistance increased up to $2x$ near the pile following installation

- Some Dutch recommendations exist w/r/t densification, but for settlement of adjacent buildings, not liquefaction

- Questions include:
  - Effect of pile spacing on magnitude of densification?
  - Effect of time?
  - Magnitude of excess pore pressure reduction?
Outstanding Questions: Reinforcement (?)

- Reinforcement effect – two modes
  - Vertical support and shear reinforcement: global stability
  - Stiffened elements divert the cyclic stresses away from soils, reduce $u_e$
- Baez (1995):
  - Introduced a theory of seismic shear stress redistribution for stone columns
  - Shear strain compatibility (SSC) assumption
- SHRP2: use SSC for CFA piles, deep soil mixing, jet grouting, vibro-concrete columns
- Olgun & Martin (2008); Rayamajhi et al. (2014):
  - Performed finite element modeling on discrete columns
  - Showed that the shear strain compatibility assumption may not be valid...
- Does the reinforcement effect result in a reduction of excess pore pressures?
Full Scale Field Test Program and Modeling

- Compare densification and reinforcement effects of drained and conventional piles with respect to pile spacing, drainage, and time elapsed since installation;

- Evaluate the generation and dissipation of excess pore pressures and subsequent post-liquefaction settlements from controlled blasting program;

- Calibrate a finite element model to the response of an unimproved control zone; make true predictions of the excess pore pressure response treated ground; and,

- Assess the efficacy of the reinforcement effect w/r/t shear strain compatibility (SSC) assumption.
[ Experimental Setup and In Situ Tests ]
Test Site Characterization

Location: Hollywood, SC – Pile Drivers, Inc.
Test Site Characterization

- Baseline in-situ testing in each of five treated zones
  - CPTu’s in each treatment zone at Piles 1, 6, 7, 8, and 9
  - Shear wave velocity tests in the center of each zone (Pile 1)
  - SPT between Piles 3 and 7
- Baseline in-situ testing in control zone
  - One CPTu (P-1); and
  - One SPT in the center
Test Site Characterization
Subsurface Profile and Identification of Liquefiable Layer

Liquefiable Soils
Initial relative density: 40 – 50%
Test Site Characterization
Fines Content correlation for Coastal Plain Beach Sands of South Carolina

\[ FC = 54 \cdot I_c - 101 \]

\[ FC = 80 \cdot I_c - 137 \]

\[ I_c = \left[ \left( 3.47 - \log(Q) \right)^2 + \left( \log(F) + 1.22 \right)^2 \right]^{0.5} \]

\[ Q = \left( \frac{q_c - \sigma_{vo}}{P_a} \right) \left( \frac{P_a}{\sigma'_{vo}} \right)^n \]

\[ F = \left( \frac{f_s}{q_c - \sigma_{vo}} \right) \cdot 100 \]
Full Scale Field Test Program: Installation

Pre-drilling and spudding through fill

<table>
<thead>
<tr>
<th>Pile length (m) [feet]</th>
<th>Head Diameter (m) [inches]</th>
<th>Toe Diameter (m) [inches]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>12.3 [40.3]</td>
<td>0.31 [12.2]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.21 [8.3]</td>
</tr>
</tbody>
</table>

Average 12.3 [40.3] 0.31 [12.2] 0.21 [8.3]
Drained Timber Pile Prototype

- Holtz and Boman (1974): PVDs fixed to timber piles reduced driving-induced positive excess pore pressures generated within soft clay

- Rollins et al. (2006; 2009): PVDs between stone columns improved densification in silty sands

- Millport Slough Replacement Bridge, US 101; PVDs between driven displacement piles improved $q_t$ substantially

- Driving-induced contractive excess pore pressures should be reduced if drainage can be provided, improving densification in silty sands
Drained Timber Pile Prototype

Note damage to pile
Investigation of Densification: In-situ Tests

- Shear wave velocity test was performed at sounding A in cell C3
- SPT between Piles 1 and 4
Investigation of Densification: Cone Tip Resistance

Zone 1: 5DPVD
Zone 2: 3DPVD
Zone 3: 5D
Zone 4: 3D
Zone 5A: 2D
Zone 5B: 4D

Corrected Cone Tip Resistance, $q_t$ (MPa)

Depth (m)

Fines Content, FC (%)

--- Fines Content
--- Baseline
Investigation of Densification: Cone Tip Resistance

Zone 1: 5DPVD
Zone 2: 3DPVD
Zone 3: 5D
Zone 4: 3D
Zone 5A: 2D
Zone 5B: 4D

Corrected Cone Tip Resistance, $q_c$ (MPa)

Depth (m)

Fines Content, FC (%)

- Fines Content
- Baseline
- 10-days Post-installation
- 255-days Post-installation

SM and SC [FILL]
SP
SP with lenses of SM
CH
Investigation of Densification: Cone Tip Resistance

Quantitative Summary of the Liquefiable Layer

$q_t$ averaged over “average toe depth of inner piles”

<table>
<thead>
<tr>
<th>Pile Spacing</th>
<th>Treatment Zone #</th>
<th>Average Toe Depth, Inner Piles (m)</th>
<th>Pre-treatment Geometric Average of $q_t$ (MPa)</th>
<th>10 Days Post-Installation</th>
<th>255 Days Post-Installation</th>
<th>Change in $q_t$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5D PVD</td>
<td>1</td>
<td>12.1</td>
<td>5.23</td>
<td>7.55</td>
<td>6.14</td>
<td>44</td>
</tr>
<tr>
<td>5D</td>
<td>3</td>
<td>11.7</td>
<td>5.35</td>
<td>10.07</td>
<td>6.81</td>
<td>88</td>
</tr>
<tr>
<td>4D</td>
<td>5B</td>
<td>10.6</td>
<td>5.89</td>
<td>11.02</td>
<td>6.95</td>
<td>87</td>
</tr>
<tr>
<td>3D PVD</td>
<td>2</td>
<td>9.3</td>
<td>5.43</td>
<td>17.65</td>
<td>14.34</td>
<td>225</td>
</tr>
<tr>
<td>3D</td>
<td>4</td>
<td>11.1</td>
<td>5.22</td>
<td>12.21</td>
<td>10.52</td>
<td>134</td>
</tr>
<tr>
<td>2D</td>
<td>5A</td>
<td>10.6</td>
<td>5.60</td>
<td>19.76</td>
<td>13.23</td>
<td>253</td>
</tr>
</tbody>
</table>
Investigation of Densification: SPT N Blow Count

Zone 1: 5DPVD  Zone 2: 3DPVD  Zone 3: 5D  Zone 4: 3D  Zone 5A: 2D  Zone 5B: 4D

Energy-corrected SPT Penetration Resistance, $N_{60}$ (bpf, blow/0.3 m)

Depth (m)

- Fines content (%)
- Baseline
- 292-days Post-installation

SM and SC [FILL]

SP with lenses of SM

CH
Investigation of Densification: Shear Wave Velocity

Shear Wave Velocity, $V_s$ (m/sec)

Zone 1: 5DPVD
Zone 2: 3DPVD
Zone 3: 5D
Zone 4: 3D
Zone 5: 2D
Zone 5: 4D

Depth (m)

Baseline
255-days Post-Installation

SM and SC [FILL]
SP
SP with lenses of SM
CH
Summary: Average Improvement in CPT $q_t$

- **Conventional Piles**: $\Delta q_t (%) = 11.0 a_r$
- **Drained Piles**: $\Delta q_t (%) = 24.6 a_r$

The graph shows the relationship between the average area replacement ratio ($a_r$) and the long-term change in $q_t$ (%) for both conventional and drained piles. The data points are marked with circles for conventional piles and filled circles for drained piles.
Application to Liquefaction Mitigation

- Conduct triggering analysis for liquefiable layer(s)
- Select spacing (area replacement ratio) and estimate densification (i.e., $\Delta q_{c1Ncs}$)
- Re-evaluate triggering analysis as needed to select final design spacing
- Conduct post-densification \textit{in situ} tests to confirm design assumptions

\[ \varepsilon_{vol} = \text{acceptable?} \]
[ Controlled Blasting ]
Liquefaction Assessment and Mitigation

**Controlled Blasting Program:**
- Install pore pressure transducers to observe blast-induced excess pore pressures, perform baseline survey
- Evaluate explosive charge weight and blast sequence req’d to induce liquefaction in unimproved control zone
- Apply same charge weight and sequence to timber pile treated zones
- Compare excess pore pressures generated from blast program
- Compare ground settlements resulting from reconsolidation and dissipation of excess pore pressures
Controlled Blasting Program for the Control Zone

LEGEND

■ EXPLOSIVES
■ #789 CRUSHED STONE
□ EXPLOSIVE NUMBER
B-#E# EXPLOSIVES BORING
Controlled Blasting Program for the Control Zone
Controlled Blasting Program for the Control Zone

Contractive Soil Response

Offset from Control Zone Center (m)

Settlement (mm)

-9 -6 -3 0 3 6 9

0 75 150 225 300

Blast Area

Settlement in mm [1” = 25 mm]

Time (min)

A-line, B-line, C-line

Diameter = 7.82 m
Mid-1970’s: Assessment of Post-liquefaction Volumetric Strain

- From cyclic TX tests, we expect significant reductions in post-shaking settlements as $D_r$ increases.
- For an increase in $D_r$ from 45 to 80%, we expect a 3-fold reduction in 1-D settlement.
Controlled Blasting Program for the Treated Zones
Controlled Blasting Program for the Treated Zones
Controlled Blasting Program for the Treated Zones
Effect of Densification on Excess PWP Response

Phase Transformation: Dilative Response

Contraction

$\sigma$  $p'$

$\text{Time (s)}$

$\text{r}_u$ ($\%$)

Zone 3: 4.83 m

Control Zone
Controlled Blasting Program for the Treated Zones

Contraction

Dilation

$z = 4.57 \text{ m}$

$z = 6.10 \text{ m}$

$z = 7.62 \text{ m}$

$z = 9.14 \text{ m}$
Controlled Blasting Program for the Treated Zones

Settlements = 1/6 to 1/3 that of control zone

*These observations confirm the post-liquefaction $\varepsilon_v$ measurements from the mid-70's*

Median settlement of piles tipped in Dense Sand: 20 mm (3/4”)

![Settlement map](image-url)
And now for something completely different…
Blast Event #3 (BE3)

Blast Event #4 (BE4)

(a) Graph showing pressure profiles over time for different distances from the blast.

(b) Graph showing settlement over time for different blast events.

(c) Graph comparing settlement profiles for Blast Event BE3 and Blast Event BE4.

(d) Graph showing cumulative settlement with markers.
Pre- and Post-Blast $V_s$ Profiles

Depth (m) vs. Shear Wave Velocity, $V_s$ (m/s)
Time Variation of Normalized $V_s$ (Layers 4 – 6)
[ Assessment of Reinforcement ]

Baez (1995) shear strain compatibility (SSC) approach: assuming the “simplified” method for liquefaction triggering

\[
CSR = \frac{\tau}{\sigma'_{v0}} = 0.65 \cdot \frac{a_{\text{max}}}{g} \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d \cdot MSF
\]

substitute \( \tau = \gamma G \) and rearrange for shear strain:

\[
\gamma_{SSC} = 0.65 \cdot \frac{a_{\text{max}}}{g} \frac{\sigma_{v0}}{G_{\text{comp}}} \cdot r_d
\]

\[G_{\text{comp}} = G_{\text{soil}}(1 - A_{rr}) + G_{\text{pile}}A_{rr}\]

since \( G_{\text{pile}} >> G_{\text{soil}} \), small \( A_{rr} \) still provides high \( G_{\text{comp}} \), and theoretically small strains \( \gamma_{SSC} \)... If SSC assumption is appropriate….

Note that MSF disappears for assessments of blast-induced shaking.

\( G_{\text{comp}} \) = shear modulus of composite ground
\( A_{rr} \) = area replacement ratio
Reinforcement Effect – Estimation of Shear Strains

If we can estimate shear strains...we can make some observations on the reinforcement effect and the shear strain compatibility (SSC) assumption for reinforcement-type ground improvement.

\[ G_1 = \text{Shear modulus at exceedance of threshold shear strain } \gamma_{th} \approx 0.01\% \]

\[ \sigma'_c = 96 \text{ kPa} \]

Curve based on Data by Dobry et al. (1982)

MRCs from Zhang et al. (2005) for SC soils

\[ G/G_1 \]

\[ G/G_{max} \]

\[ 1 - r_u = 60\% \]

\[ r_u = 60\% \]
Reinforcement effect – Results of Assessment

Shear Strain

Depth (m)

5D Spacing

3D Spacing

Shear Strain

Depth (m)

Excess Pore Pressure Ratio, \( r_u \)

\( a_{\text{max}} = 0.17g \)

\( = 0.30g \)

- Estimated Strains
- \( r_u \) with Estimated Strain
- Range in SSC Strains
- Range in \( r_u \) (Dobry 1985)
Summary / Conclusions

Field Test Program

- Cone tip resistance increased 45 to 250%, immediately following installation of timber piles depending on the spacing (this corresponds to relative densities of 60 to 95% from 40 to 50%).
- Long-term observations suggested that relaxation of horizontal stresses occurred following installation of driven timber piles.
- Blasting performed in the control zone produced complete liquefaction for the deeper soils, resulting in maximum settlements of about 200 mm in the center of the control zone.
- Peak residual $r_u$ values in the treated zone were all less than those of the unimproved ground, and produced dilative responses.
- The average settlements observed in the improved zones were approximately one sixth to one third of the settlement observed for the same charge sequence applied to the unimproved control zone.
- Timber piles embedded in the dense sand layer had a median settlement of 20 mm compared to piles that were not tipped in the dense; these exhibited settlements similar to the reinforced soil.
Summary / Conclusions

Analytical Investigations

- The finite element (FE) model prediction of generation and dissipation of excess pore pressures for conventional timber piles in Zones 3 and 4 were generally in good agreement.
- The FE model over-predicted the pore pressure reductions in the drained timber pile zones – suggesting discharge capacity insufficient for dynamic use.
- The shear strain compatibility approach was found to under-predict the estimated shear strains experienced by the soil compared to those estimated based on measured excess pore pressure ratios in the field.
- Use of the shear strain compatibility approach is not recommended for use with discrete elements.
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Master’s Student:
Tygh Gianella, Staff Engineer
GeoEngineers, Inc., Portland, OR


Analytical Investigations and Comparison to Controlled Blasting

- Finite Element Analysis: FEQDrain
  - Developed by Pestana et al. (1997)
  - Models earthquake-induced generation and dissipation of pore water pressure in layered sand deposits

- Input parameters
  - Earthquake loading parameters
    \( N_{eq}, t_d \)
  - Soil input parameters
    \( k_h, k_v, \gamma, m_v, N_L, D_r \)
Calibrated Model: Generation and Dissipation of Excess Pore Pressure in the Control Zone

- $z = 5.06$ m
- $z = 6.32$ m
- $z = 8.02$ m
- $z = 8.58$ m
Treated Zone Response – Conventional Piles

NOTE: Only Relative Density and #Cycles to Liquefaction altered

Zone 3: $z = 4.83$ m

Zone 3: $z = 6.08$ m

Zone 4: $z = 7.44$ m

Zone 4: $z = 8.98$ m
Treated Zone Response – Drained Piles: Comparison of Measured and Computed Excess Pore Pressure

Zone 2: $z = 4.83$ m

Zone 2: $z = 6.08$ m

Zone 2: $z = 7.44$ m

Zone 2: $z = 8.98$ m