Assessment of Residual Strength During Blast Liquefaction Experiments with Deep Foundations

Kyle M. Rollins and Travis M. Gerber

Civil & Environmental Engineering Department
Brigham Young University
Provo, Utah
Residual Strength Development

Force (or Strength) vs. Displacement (or Strain)
Lateral Load Analysis for Piles with $p$-$y$ Curves

Non-linear springs

Interval

$y_1, y_2, y_3, y_4, y_5$
Piles in Level Ground

Liquefied Sand

Stiff Clay
Piles in Sloping Ground

- Non-Liquefied Sand
- Liquefied Sand
- Stiff Clay
Pile Damage in Earthquakes
One good test is worth a thousand expert opinions.

--Werner Von Braun
Designer of Saturn V Moon Rocket
Sponsors

- Seven State DOTs
- FHWA
- NSF
- PDCA
- ADSC
Treasure Island Naval Station
Site Characterization

- **Field Testing**
  - Cone Penetration Testing (CPT, Visioncone)
  - Standard Penetration Testing (SPT)
  - Dilatometer Testing (DMT)
  - Pressuremeter Testing (PMT)
  - Shear Wave Velocity Testing
  - Radar Tomography

- **Lab Testing**
  - Atterberg Limits
  - Grain Size Distribution
  - Undrained Strength Testing
Interpreted Soil Profile

- Fine Sand w/ Shells (SP)
- Interbedded Fine Sand and Silty Sand (SP-SM)
- Fine Silty Sand (SM)
- Gray Silty Clay (CL)
- Sand (SP)

CPT Cone Resistance, $q_{c1}$ (MPa)

SPT Blow Count, $N_{1(60)}$ (Blows/300 mm)

Relative Density, $D_r$ (%)

- Mean
- Mean-SD
- Mean+SD

From CPT

From SPT
Single Pile Test
Instrumentation

- 25 Pore pressure transducers.
- Strain gages on piles to determine bending moment versus depth.
- Load cells to determine pile head load.
- String potentiometers for pile head displacements and rotations.
- Survey markers to monitor settlement.
Load vs Deflection Curves for Single Pipe Pile

Displacement (mm) vs Load (kN) for non-liquefied and liquefied conditions.

- Non-Liquefied
- Liquefied
Comparison with Lab Tests

Sacramento river sand
ACU cyclic triaxial
$D_R=57\%, \sigma_3'=200 \text{ kPa}$
\[ \text{Load (kN)} = (R_u - u) \tan \theta \]
Moment Before & After Liquefaction

-2 -1 0 100 200 300 400 500

-2 0

0 2

2 4

4 6

6 8

8 10

10 12

-100 0 100 200 300 400 500

0 2

2 4

4 6

6 8

8 10

10 12

Depth Below Excavated Ground (m)

Moment (kN-m)

Before Liquefaction

After Liquefaction
Development of $p-y$ Curves

- Strain
- Curvature
- EI
- Moment
- Shear
- Slope
- Integrate
- Differentiate
- Deflection, $Y$
- Distributed load or "pressure", $P$
- P-Y curve
Generalized p-y Curves

<table>
<thead>
<tr>
<th>Curve</th>
<th>Depth (m)</th>
<th>Avg. Ru</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.00</td>
<td>94%</td>
</tr>
<tr>
<td>D</td>
<td>2.29</td>
<td>98%</td>
</tr>
<tr>
<td>E</td>
<td>3.05</td>
<td>100%</td>
</tr>
<tr>
<td>F</td>
<td>3.81</td>
<td>100%</td>
</tr>
<tr>
<td>G</td>
<td>4.57</td>
<td>98%</td>
</tr>
</tbody>
</table>

(B & C omitted, Avg. Ru < 90%)
Equation for $p$-$y$ Curves in Liquefied Sand

$p = P_d A(By)^C$ (for $D_r \approx 50\%$)

where:

$A = 3 \times 10^{-7} (z + 1)^{6.05}$

$B = 2.80 (z + 1)^{0.11}$

$C = 2.85 (z + 1)^{-0.41}$

$z = \text{depth in m}$

$P_d = \text{adjustment factor for pile diameter}$

$= 3.81 \ln(\text{pile dia.}) + 5.6$

Note: $p$ in kN/m and $y$ in mm.
Computed vs Measured Response
Undrained Strength Approach for Liquefied Sand

Horizontal Resistance/Length, P vs. Horizontal Displacement, y

- Ultimate Strength based on Residual Strength
- Soft clay curve shape
Residual Strength for Liquefied Sand

Seed and Harder (1990)

Fig. 11: Relationship Between Corrected "Clean Sand" Blowcount ($N_1^{60-CS}$) and Undrained Residual Strength ($S_r$) from Case Studies
Residual Strength Ratio for Liquefied Sand
Olson and Stark (1990)
P-multiplier Approach for Liquefied Sand

Horizontal Deflection, y

Horizontal Force/Length, p

Non-Liquefied Sand Curve

Liquefied Sand Curve using P-multiplier of 0.1 to 0.3
Comparison of p-y Curves for Liquefied Sand

- Calculated
- Stress Ratio
- No Soil Resistance

Markers = Measured Values Relative to Zero Pile Head Load
Bending Moment Comparisons
Undrained Strength Approach (S&H)

Depth Below Ground Surface (m)

Bending Moment (kN-m)
(A) Pile Load = 15.0 kN
(B) Pile Load = 30.5 kN
(C) Pile Load = 60.0 kN

- Measured
- Developed p-y Curves
- Pile Only (no soil resistance)
- Residual Undrained Shear Strength
  Approach
  Average
  Lower-bound
Bending Moment Comparisons
Undrained Strength Approach (O&S)

15 kN Pile Load

Moment (kN-m)

-50 0 50 100 150

Depth Below Load Pt (m)

0 2 4 6 8 10 12 14

Avg. $S_r$  lower bound $S_r$  Measured  Stress Ratio  No Strength

30 kN Pile Load

Moment (kN-m)

-100 0 100 200 300

Depth Below Load Pt (m)

0 2 4 6 8 10 12 14

60 kN Pile Load

Moment (kN-m)

-200 0 200 400 600

Depth Below Load Pt (m)

0 2 4 6 8 10 12 14
Bending Moment Comparisons
P-multiplier Approach

Bending Moment (kN-m)
(A) Pile Load = 15.0 kN
(B) Pile Load = 30.5 kN
(C) Pile Load = 60.0 kN

- Measured
- Developed p-y Curves
- Pile Only (no soil resistance)
- Sand P-Y Curve with P-multiplier Approach
  - P-mul = 0.3
  - P-mul = 0.1
Cooper River Bridge
Charleston, South Carolina

Longest Cable-stayed bridge in North and South America

New Bridge-Completed July 2005
CPT Profile & Relative Density

Interpreted Soil Type

- Sand (SP) to Silty Sand (SM)
- Clay (CH)
- Sand (SP)
- Silty Sand (SM) and Clayey Sand (SC)
- Sand (SP)
- Cooper Marl (CH)

Relative Density $D_r$ (%)

Friction Angle $\phi$ (degrees)

Depth (m)
Pile Description

Location of Load and LVDT (0.53 m)

Ground Surface Elevation (0.0 m)

SECTION A

Ø2.59 m

Steel Casing (25.4 mm)

36 #18 @ D = 2.14 m

SECTION B

Ø2.54 m

36 #18 @ D = 2.14 m

North Gages

South Gages

A

-Station 1 (1.00 m)

-Station 2 (3.45 m)

-Station 3 (5.88 m)

-Station 4 (8.32 m)

-Station 5 (10.14 m)

-Station 6 (13.20 m)

-Station 7 (15.94 m)

-Station 8 (18.08 m)

-Station 9 (20.52 m)

-Station 10 (23.26 m)

-Station 11 (25.70 m)

-Station 12 (27.84 m)

Bottom of Shaft (47.00 m)
Blasting and Piezometer Layout

Inner Blast Ring
3.96 m R

Outer Blast Ring
4.57 m R

Load Direction

Piezometer

Blast Holes

1st Ring
1.83 m R

2nd Ring
7.32 m R

3rd Ring
10.36 m R

4th Ring
14.63 m R

5th Ring
17.68 m R
Test Set-Up

Reference Beam

2.6 m Test Shaft

2 - 2200 kN Hydraulic Actuators
Load-Displacement Curves

Load (kN) vs. Deflection (cm)
Moment versus Depth Curves

Cycle 1 from all three tests

-5
0
5
10
15
20
25
30
35

Depth (m)

-5000 0 5000 10000 15000

Moment (kN-m)

-1285 kN, pre-blast, cycle1
-1294 kN, first blast, cycle1
-1293 kN, second blast, cycle1
Adjustments to $p-y$ Curve for Diameter

$P_d = 3.81 \ln (d) + 5.6$

- Charleston
- Treasure Island

Tests
Equation
Comparison of measured moments and deflections using p-y curves from Rollins et al (2005)

Applied Load = 3950 kN
Test Layout at Tokachi, Hokkaido Test
(Ashford et al, 2006)

Fig. 1. Site layout for the first lateral spreading test
Soil Profile at Tokachi Test Site
(Ashford et al, 2006)
Back-Calculated p-y Curves Tokachi Test
(Ashford et al, 2006)

- P-y curves for loose sand \((D_r = 15-25\%, (N_1)_{60} = 2 \text{ to } 6)\) were essentially flat
- Suggests no residual shear strength following liquefaction
- Generally consistent with predicted residual strength
Comparison with Centrifuge Test Results (Wilson, 1998)

$D_r \approx 55\%$

$D_r \approx 35-40\%$
Vancouver BC Test Site

Vancouver

Massey Tunnel

Downdrag Test Site (Canlex)
Geotechnical Soil Profile

Interpreted Soil Profile

- Fine Sand (SP)
- Sandy Silt/Silt (SM/ML)
- Fine Sand (SP)/Silty Sand (SM)
- Sand (SP)

CPT Cone Resistance, $q_c$ (MPa)

CPT Friction Ratio, $R_f$ (%)

Nominal Relative Density, $D_r$ (%)

Equivalent Blow Count, $(N)_{60}$

Depth (m)

May Site

July Site
Pore Pressure Generation

Excess Pore pressure Ratio, \( Ru \)

Time (sec)
Pore Pressure Dissipation

![Graph showing pore pressure dissipation over time for different depths.](image-url)
Shear Stress and $R_u$ vs time

(a) May Test

(b) July Test
Comparison of Measured vs. Predicted Strength

- In-Situ Vane Strength (1 to 10 kPa)
- Canlex Shear Tests on Frozen Samples
- Direct Simple Shear (9 to 20 kPa)
- Triaxial Extension (4 to 9 kPa)
- Predicted by Harder and Seed (5 to 25 kPa)
- Predicted by Olson and Stark (6 to 11 kPa)

Shear Strength (kPa)

0 5 10 15 20 25 30 35 40

Predicted

Measured

Seed & Harder
Olson and Stark
In-Situ Vane
Canlex Triax. Ext.
Canlex DSS
Conclusions from Single Pile Tests

- Blast liquefaction testing can provide useful information on the behavior of liquefied sand.
  - For sand with $D_r > 50\%$:
    - Resistance develops due to negative pore pressure at large deflections.
    - P-y curves shape stiffens with depth (confining pressure).
    - At large deflections, undrained residual strength approach appears to be reasonable, but it overestimates resistance at small deflections.
    - P-multiplier approach appears to overestimate lateral resistance.
  - For sand with $D_r < 35$ to $40\%$:
    - Lateral resistance is approximately zero
    - Strength is consistent with undrained strength approach
    - P-multiplier approach could overestimate resistance.
- In-Situ vane shear approach shows promise for evaluating residual shear strength in liquefied sand.