Ground Improvement for Liquefaction Risk Mitigation
Methods, Verification, and Recent Research

Allen L. Sehn, Ph.D., P.E
Vice President, Engineering
Hayward Baker Inc

ALSohn@HaywardBaker.com
Outline

- Liquefaction Evaluation & Liquefaction Mitigation Methods
- Verification of the Mitigation Effectiveness
  - Densification
  - Reinforcement
- Research on Shear Reinforcement Effects
  - Discrete Columns
  - Soilcrete Shearwall Grid
Liquefaction Prerequisites

- Saturated soil
- Loose granular or other non-plastic soils.
- Strong ground motion.
  - Shear strains cause tendency for contraction.
  - Water cannot drain fast enough.
  - Pore water pressure increases and effective stress decreases (may approach zero).
  - After shaking stops pore water pressures dissipate and settlement occurs.
Liquefaction Evaluation

- Youd, et al. (2001)
  1998 NCEER/NSF Workshop

- California SP 117


- Idriss and Boulanger (2004 and 2008)
  EERI Monograph 12

- Baez and Martin (1993 and 1995)
SPT Based Approaches

Curves derived by:
1. Seed (1979)
2. Seed & Idriss (1982)
5. Recommended Curve

From Idriss & Boulanger (2004)
CPT Based Approaches

- Shibata & Teparaksa (1988)
- Robertson & Wride (1997)
- Moss (2003) - 5% Probability
- Idriss & Boulanger (2004)

Cyclic Resistance Ratio (CRR) vs. Normalized Corrected CPT Tip Resistance, $q_{c1n}$

- Clean Sands
- Liquefaction
- No Liquefaction
Liquefaction Mitigation Methods

1. Remove and replace with nonliquefiable soil
2. Densify loose granular soil
3. Modify cohesive properties of the soil
4. Provide shear reinforcement
5. Provide adequate drainage
6. Permanent lowering of the GWT
7. Deep Foundations piles or piers
8. Reinforced Shallow Foundations grade beams, combined footings, rigid raft foundations,
9. Design to accommodate settlement and loss of strength
Liquefaction Mitigation Methods

- Densification Methods
  - Deep Dynamic Compaction (DDC)
  - Vibro Compaction
  - Vibro Displacement (stone columns)
  - Compaction Grouting
Liquefaction Mitigation Methods

- Improvement of Cohesive Properties
  - Deep Mixing
  - Jet Grouting
  - Permeation Grouting
Verification of Liquefaction Mitigation

- Densification Verification
  - SPT
  - CPT
  - Shear Wave Velocity
  - Modulus/Plate Load Test?
  - Void Reduction vs. Volume Intake?

- Reinforcement Verification
Densification Verification

- SPT (ASTM D6066)
- CPT
CPT Comparison

PostVR-1A
- M = 6.8
- 8 by 8 grid
- Ar = 15.0%
- PGA = 0.37 g
- Fill = 0'
- GWT = 9'
- Without Thin Layer Correction

Post-treat Dynamic Settlement = 0.48"
Plate Load Test to Verify Stiffness?

- Can a plate load test (modulus test) be used to determine the modulus of the aggregate pier within the liquefiable soil layer?

- If the liquefiable layer is more than about 2B below the plate load test, do the test results reflect the properties of the pier within the liquefiable soil?
Densification Verification

- Soil Void Reduction ≠ Volume Intake

\[ \varepsilon_x \approx -\varepsilon_y \]
\[ \varepsilon_{vol} \approx 0 \]
Verification of Liquefaction Mitigation

- Densification Verification
  - SPT
  - CPT
  - Shear Wave Velocity
  - Modulus/Plate Load Test?
  - Void Reduction vs. Volume Intake?

- Reinforcement
Verification of Reinforcement Effect

- Discrete Columns
  - Aggregate/Sand Columns
  - Soil Mixing/Jet Grouting Columns
  - Auger Displacement Piles
  - Compaction Grouting Columns
  - Rigid Inclusion Columns

- Cellular Structures (grids)
  - Soil Mixing/Jet Grouting Panels
Liquefaction Mitigation by Cells or Blocks

SMW Treatment Pattern, Jackson Lake Dam Project, WY (after Pujol-Rius, et. al., 1989)

Improved Soil Foundation for Building, Kagoshima City, Japan (after Babasaki, 1991)
Failure Modes

- Aggregate does not have tensile strength
- Soilcrete is a brittle material
  - Failure strain <1%
  - Low residual strength
  - Low tensile strength
- Discrete columns may fail in bending
- Cellular configuration resists shear loading
Liquefaction Mitigation by Reinforcement

- Reduce cyclic shear stress applied to liquefiable soil by installing ‘stiffer’ elements within the soil matrix that will attract shear stress.
- Can be used in non-densifiable soils (silts, silty sands).
- Not easily verified by field testing
  - Post-installation CPT or SPT results will not differ from pre-installation.
  - Vertical load testing of elements is not applicable.
The basic assumption in evaluating the distribution of stresses according to the stiffness of the individual elements is that shear strains for both loose and stiff material are compatible (personal communication Byrne, 1992). The assumption is valid because there is no inertial loading from the superstructure directed to the stone columns which can cause displacements in directions other than that of the ground motion. Therefore,

\[ \gamma_S = \gamma_{SC} \]  

(1)

and,

\[ \frac{\tau_S}{G_S} = \frac{\tau_{SC}}{G_{SC}} \]  

(2)

where,

\[ \gamma_S \]  = shear strain in the soil
\[ \gamma_{SC} \]  = shear strain in the stone/concrete column
\[ \tau_S \]  = shear stress in the soil
\[ G_S \]  = shear modulus of the soil
\[ \tau_{SC} \]  = shear stress in the stone column
Liquefaction Mitigation - Reinforcement

- Design Methodology
  - Shear stress reduction factor \( K_G \) (Baez and Martin, 1993):

\[
K_G = \frac{1}{1 + ARR \left( \frac{G_{INC}}{G_{Soil}} - 1 \right)}
\]

  - \( G_{INC} \) = Inclusion shear modulus
  - \( G_{Soil} \) = Soil shear modulus
  - \( ARR = \frac{A_{inclusion}}{A_{total}} \)

- Strain compatibility and force equilibrium
  - \( CSR_{applied \ to \ soil} = K_G * CSR_{earthquake} \)
The variation of reduction factor $S_G = f(a_r, G_r)$ of Eq. 5. is given in Fig. 2. The $G_r$ range of 10-150 values are utilized in the Fig. 2. It could be seen that area replacement ratio of $a_r = 7-10\%$ will be effective to obtain reduction factor of nearly $S_G(\%)$ 10-60. Since, the factor of safety is inversely proportional with $S_G$, a great increase in factor of safety could be obtained based on the specific values of modulus, and area replacement ratios.
Stiffness Values

- Can a column be too stiff?
- Strain Compatibility?
- Failure mechanism of column
  - Bending
  - Shear
Shear Reinforcement for Liquefaction Mitigation
Research Team

- PI: Dr. Ross Boulanger, UC Davis
  - Thang V. Nguyen (Hayward Baker Inc)
- Dr. Ahmed Elgamal, UCSD
  - Dr. Jinchi Lu
- Dr. Scott A. Ashford, OSU
  - Deepak Rayamajhi
- Dr. Lisheng Shao, Hayward Baker Inc
Discrete Columns

3D analyses by Nguyen et al. (2012) explore a wide range of parameters and loadings to develop a design relationship.
Discrete Columns

b) Soil profile

1 m - dense sand
\( \rho = 1.92 \text{Mg/m}^3 \)
\( v = 0.4 \)
\( V_s = 300 \text{ m/s} \)

9 m - loose sand
\( \rho = 1.92 \text{Mg/m}^3 \)
\( v = 0.4 \)
\( V_s = 150 \text{ m/s} \)

2 m - dense sand
\( \rho = 1.92 \text{Mg/m}^3 \)
\( v = 0.4 \)
\( V_s = 300 \text{ m/s} \)

c) FE model elevation

d) FE model plan - section at A-A with selected points of analyses
Discrete Column

\[
CSR_U = \frac{\tau_{s,U}}{\sigma'_v} = 0.65 \left( \frac{a_{\max,U}}{g} \right) \left( \frac{\sigma_v}{\sigma'_v} \right) r_{d,U}
\]

\[
CSR_I = \frac{\tau_{s,I}}{\sigma'_v} = 0.65 \left( \frac{a_{\max,I}}{g} \right) \left( \frac{\sigma_v}{\sigma'_v} \right) r_{d,I}
\]

\[
R_{CSR} = \frac{CSR_I}{CSR_U} = \left( \frac{a_{\max,I}}{a_{\max,U}} \right) \left( \frac{r_{d,I}}{r_{d,U}} \right) = R_{a_{\max}} R_{rd}
\]

- \( R_{a_{\max}} \) - ratio of peak ground accelerations,
- \( R_{rd} \) - ratio of shear stress reduction coefficient for improved & unimproved case
- \( \gamma_r \) - ratio of shear strains in the column to shear strains in the surrounding soil
Pseudo-static loading

$A_r = 20\%$ and $G_r = 10$
Pseudo-static loading

\[ A_r = 20\% \quad \text{and} \quad G_r = 10 \]
Spatial distribution $R_{rd}$ and $Y_r$ from earthquake time history analysis with $A_r=20\%$ and $G_r=10$
$R_{rd}$ includes adjustment factors for the effects of discrete column flexure and shear strain incompatibility

\[
R_{rd} = \frac{1}{G_r \left[ A_r \gamma_r C_G + \frac{1}{G_r} (1 - A_r) \right]}
\]

- $C_G$ - equivalent shear factor of the discrete column
  $C_G = 1.0$ for circular discrete columns

- $\gamma_r$ - is dependent on $G_r$ and independent of $A_r$

- $K_G$ - from Baez (1995), is equivalent to $R_{CSR} = (R_{rd})(R_{amax})$
  pseudo-static analyses, $R_{amax} = 1$ and $R_{CSR} = R_{rd}$
Comparison of $R_{rd}$

(a) based on strain compatibility
(b) based on proposed relationships

Conclusions – Discrete Columns

- Current (former?) design practice assumes that
  - discrete columns deforming in pure shear
  - shear strains are compatible between columns & soil

- 3D FEM analyses
  - discrete columns deformed in both flexure & shear
  - flexural & rotational deformations greatly diminished their ability to reduce dynamic shear stresses in the surrounding soils.

- Current design methods overestimate the reduction in dynamic shear stresses in the soil

- Revised design equation
  - accounts for column flexure & difference in shear strains between column & surrounding soil
  - more reasonable estimates of the shear stress reduction provided by discrete circular columns.
Linear Elastic Analyses of Cemented Soil Grids using OpenSees Platform
Oriental Hotel in Kobe, 1995

- Loose fill to depths of about 12 m.
- Perimeter quay walls moved 1-2 m due to liquefaction.
- No damage to foundation or evidence of liquefaction inside DSM walls.
Linear Elastic FE Model - DSM

- $V_s = 300 \text{ m/s}$
  - $\nu = 0.3$
  - $\rho = 1.92 \text{ Mg/m}^3$

- $V_s = 150 \text{ m/s}$
  - $\nu = 0.3$
  - $\rho = 1.92 \text{ Mg/m}^3$

Half DSM Unit Cell Mesh in OpenSeesPL
Standard DSM Half Unit Cell Under Earthquake

\[ R_{rd} \text{ and } R_{y} \text{ profiles} \]

- Depth (m) vs. \( R_{rd} \) and \( R_{y} \) profiles
- Graphs showing depth vs. \( R_{rd} \) and \( R_{y} \) values for different points (Point 1, Point 2, Point 3, Point 4)
- Diagram indicating the standard DSM half unit cell under earthquake conditions.
Spatial Variation

Great similarity between Pseudo Static and Earthquake case was observed which lead to the following proposed design equation.
Proposed Design Relationships

**Proposed Equation**

**Strain Compatibility Equation**

Scheduled for publication:
Conclusion – Soilcrete Grid

- DSM grids affect both:
  - seismic site response (e.g., $a_{\text{max}}$)
  - seismic shear stress distributions (e.g. $R_{\text{rd}}$)

- Effect of DSM grids on seismic site response can be significant and may require site-specific FEM analyses.

- The reduction in seismic shear stresses by DSM grids can be over-estimated by current design methods that assume shear strain compatibility.

- A modified equation is available for estimating seismic shear stress reduction effects.

- The top 2m-3m of DSM wall could potentially be the critical wall section in term of tension development.
Primary Lessons Learned from Recent Research on Reinforcement for Liquefaction Risk Mitigation

- Discrete columns are significantly less effective than predicted by methods based on the shear strain compatibility assumption.

- Soilcrete elements installed to create a grid or cellular pattern of shearwalls can result in a significant reduction in the cyclic shear stresses experienced by the soil during an earthquake.


References and Additional Resources


Comments or Questions?

Al Sehn, Ph.D., P.E.
Hayward Baker Inc
ALSSehn@HaywardBaker.com