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Liquefaction Mitigation Synthesis Report

Prepared for: The Ground Improvement Committee of the Deep Foundations Institute

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PROLOGUE

This report presents the results of a synthesis on the design and analysis of ground improvement for liquefaction mitigation. The synthesis included an industry survey concerning the practice of ground improvement for liquefaction mitigation. Participation in the survey was solicited by advertisements in several trade magazines and by e-mail for the DFI membership. The survey participants numbered 150. Their professional roles include consulting engineers, specialty contractors, design engineers, government engineers, and academicians. They represent a variety of geographical areas including North/Central/South America, United Kingdom, Middle East, Caribbean, Hawaii, Japan, India, Egypt, France, Australia and New Zealand. Upon completion of the survey, several professionals in the field of liquefaction and ground improvement were interviewed for them to elaborate on the survey results. The interviews are included in the Appendix of this report. Financial support for the project was provided by DFI and Dan Brown and Associates PC.

The concept of the liquefaction mitigation synthesis was developed by DFI’s Ground Improvement Committee in recognition that:

(a) The results of recent research and post-earthquake reconnaissance have challenged previously long-held beliefs about liquefaction and associated mitigation techniques, and;

(b) The DFI membership and the engineering/construction industry are interested to know if and how engineers and designers are subsequently adjusting their practice in consideration of recent research and post-earthquake reconnaissance.

For more detailed information on recent research and post-earthquake reconnaissance, presentations are available from the State-of-the-Art Forum: Liquefaction Consequences and Mitigation that was held in St. Louis in 2012. A commentary of the state-of-practice in ground improvement for liquefaction mitigation (prepared by DFI’s Ground Improvement Committee) is included in this issue of the DFI Journal.

The author would like to thank the participants of the survey and especially Mr. Mike Jeffries, Dr. Les Youd, and Dr. Ikuo Towhata for their willingness to share their expertise in interviews. The author also acknowledges Mary Ellen Bruce of DFI, Billy Camp of S&ME, Inc., and Marty Taube of DGI Menard (and Chair of DFI’s Ground Improvement Committee) for their significant contributions.

INTRODUCTION

Cyclic liquefaction is a phenomenon where high excess pore-water pressure develops in saturated soil as a result of cyclic loading (Seed and Lee, 1965 and 1966). When the ratio of pore-water pressure to the total vertical stress is essentially 1 (i.e., the state of zero effective stress), the soil is considered “liquefied” and loses a large portion of its shear resistance. At lower relative densities (less than approximately 70%), soils may contract resulting in large ground settlements. Soils with higher relative densities (greater than approximately 70%) are dilative, preventing a substantial loss of shear strength and large ground settlements. Liquefaction and its impact on engineering structures came to the engineering forefront in the 1960s due, in part, to the widespread liquefaction-induced damage (primarily settlement, tilting and lateral displacement of buildings) that occurred as a result of the 1964 Niigata (Japan) and 1964 Good Friday Alaska earthquakes (Grantz, et al., 1964; Japan National Committee on Earthquake Engineering, 1965; Seed and Idriss, 1967). Liquefaction also caused severe damage in the 1959 Jaltipan (Mexico) earthquake (Marsal, 1961) and the 1960 Chilean earthquake (Duke
and Leeds, 1963). In 1971, the “simplified
procedure” (Seed and Idriss, 1971; Whitman,
1971) for anticipating liquefaction was developed
based on the soil conditions and the design
earthquake. Seed and Booker (1977) proposed
the installation of columnar gravel drains in soil
of high liquefaction potential to prevent the
development of excessively high pore-water
pressure. In this way, a routine framework
was established. First, the liquefaction potential
is evaluated using a rational method involving
either field or laboratory testing. Second, if a
potential for liquefaction is present and the
effects of liquefaction are determined to present
unacceptable risk to the performance of the
structure, then ground improvement can be
designed to mitigate the risk.

Over the past 40 years, the evaluation of
liquefaction and the methods for mitigation
design has continued to rapidly evolve. The
evolution process is not without difficulties.
Due, in part, to the use of the research
community as the primary technical source
serving practicing engineers and contractors,
new insights are continuously being delivered.
Practicing engineers and contractors are
challenged to implement the conclusions of the
most recent findings in a coherent and rational
manner. However, considerable controversy
and an absence of consensus exist regarding
several aspects related to liquefaction including
the subject of the efficacy of various mitigation
techniques. The resulting negative impacts
may include over-conservatism (and increased
cost) by the designers and consultants, conflicts
within the design team, and confusion among
owners and their representatives.

The dichotomy between research and practice
on the subject liquefaction is not new. In his
technical note from 1979, Ralph Peck advised
that engineers and those that depend on
engineers would be well served to distinguish
between research (or science) and practice:

In short, engineering science and engineering
practice are not identical. Advances
in science may temporarily appear to
run counter to good practice. When
this occurs, the implications should be
evaluated carefully, but it should by no
means be assumed that the latest scientific
advancement is always the right direction.
Science has its own ways of making
progress, as evidence accumulates it corrects
its errors and improves its predictions. In
the end, it is certain to improve practice as
well. But science may temporarily mislead
the unwary, and it should not intimidate
either the experienced engineer or the
overburdened regulatory agency.

Another way of saying this is that engineering
practice should be careful not to assume that
the most recent opinion on the subject of
liquefaction must be correct to the extent that it
automatically invalidates those that precede it.

**LIQUEFACTION MITIGATION
SYNTHESIS**

As primarily an effort to support the
Deep Foundations Institute (DFI) membership,
this synthesis attempts to help define the
current state-of-practice in liquefaction
mitigation by surveying practicing engineers
and specialty contractors involved in the
selection and implementation of ground
improvement techniques for liquefaction.
The survey was divided into four sections:
(1) general practice, (2) liquefaction analysis,
(3) mitigation design, and (4) verification.
This report summarizes the results of the
survey and presents the conclusions that may
be made from the survey results.

**General Practice**

These questions are intended to provide
a profile of the participant. Taken as a
summary, this information will characterize
the population involved in the survey. The
questions and the answers (in terms of
percentages) are:

1. What best describes your position in the
liquefaction mitigation industry?
   - Consulting Engineer (58%)
   - Design Engineer (16%)
   - Specialty Contractor (14%)
   - Other (12%)

**Commentary:** The ground improvement
industry, especially for liquefaction mitigation,
is different from most other geotechnical/
foundation designs. The design engineer for
liquefaction mitigation is typically employed
directly by the specialty contractor performing
the installation. Because the owner and the
owner's consultants often do not have the
technical expertise to prepare or review the
liquefaction mitigation design, they provide
the performance requirements (which can be very stringent). Such an environment has its problems ranging from performance requirements that are unrealistic to significant differences between competing designs.

2. What area of the US are most of your projects involving liquefaction (check all that apply)?
   - California (29%)
   - Western US (ID, UT, NE, MT, WY) (6%)
   - Pacific Northwest (OR, WA) (19%)
   - Midwest (New Madrid) (10%)
   - Southeast (Charleston) (10%)
   - Other (26%)

Commentary: Earthquake and liquefaction concerns have been an essential part of engineering design in California and other parts of the Western United States since the 1960s. Because of the adoption of the International Building Code and the associated increases in seismic demand since the mid to late 1990s, liquefaction analysis and mitigation have become part of engineering design in parts of the eastern United States.

3. How many projects involving liquefaction analysis, mitigation design and/or construction do you participate in over a one year period?
   - less than 5 (39%)
   - between 5 and 10 (38%)
   - between 10 and 40 (18%)
   - over 40 (5%)

4. How standardized do you believe the state-of-practice in liquefaction mitigation using ground improvement techniques is?
   - very consistent and uniform (3%)
   - somewhat consistent and uniform (33%)
   - somewhat non-uniform (46%)
   - highly non-uniform (18%)

Commentary: As recently as the last few years, research and post-earthquake reconnaissance have provided results that contradict previous held beliefs on liquefaction and the effectiveness of some mitigation efforts. This is one reason why it has been particularly difficult to establish a more uniform and standardized state-of-practice in ground improvement for liquefaction mitigation and that it is not unexpected that over half of the participants believe that state-of-practice is “somewhat” or “highly non-uniform”.

5. How significant have recent considerations of liquefaction of fine-grained soil (i.e., > 30% passing No. 200 sieve) been to your projects?
   - very significant (24%)
   - significant (39.5%)
   - marginally significant (24.5%)
   - not significant (12%)

Commentary: Results of recent research (Boulanger and Idriss, 2006; Bray and Sancio, 2006) and earthquake reconnaissance (Martin and Olgun, 2008) support that fine-grained soils can be susceptible to liquefaction. Engineering practice has, in many areas, incorporated fine-grained soils into the triggering analysis and design of mitigation by adapting the procedures developed for sands.

### Liquefaction Analysis

These questions relate to liquefaction analysis in engineering practice. The questions and the answers (in terms of percentages) are:

<table>
<thead>
<tr>
<th>Test</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard penetration test (SPT)</td>
<td>58%</td>
<td>27%</td>
<td>8%</td>
<td>3%</td>
<td>4%</td>
</tr>
<tr>
<td>Cone penetration test (CPT)</td>
<td>32%</td>
<td>43%</td>
<td>16%</td>
<td>5%</td>
<td>3%</td>
</tr>
<tr>
<td>Shear-wave velocity test</td>
<td>3%</td>
<td>15%</td>
<td>43%</td>
<td>30%</td>
<td>9%</td>
</tr>
<tr>
<td>Cyclic lab test</td>
<td>1%</td>
<td>5%</td>
<td>11%</td>
<td>35%</td>
<td>49%</td>
</tr>
<tr>
<td>Liquefaction maps</td>
<td>6%</td>
<td>10%</td>
<td>22%</td>
<td>27%</td>
<td>35%</td>
</tr>
</tbody>
</table>
6. How often are the following techniques used for site characterization associated with liquefaction analysis on your projects? 1 is most often, . . . 5 is least often.

**Commentary:** The standard penetration test (SPT) (Seed et al., 1983 and 1985) can involve significant error due to the variation in energy delivered by the hammer during the test. In this regard, the more controlled in situ tests – cone penetration test (Robertson and Wride, 1998) and shear-wave velocity test (Andrus and Stokoe, 2000) – provide more repeatable and reliable results. However, there can be limitations with any test. For example, Dr. Brady Cox of the University of Texas at Austin showed that calcareous sands experienced liquefaction during the 2010 Haiti earthquake even though the shear-wave velocity profile would have indicated otherwise (DFI Presentation, 2012).

7. What presumptive analytical maximum depth do you consider in your liquefaction analysis?
   - 30 ft (9 m) or less (8%)
   - between 30 ft and 40 ft (9 m and 12 m) (4%)
   - between 40 ft and 50 ft (12 m and 15 m) (25%)
   - between 50 ft and 75 ft (15 m and 23 m) (25%)
   - No presumptive analytical maximum depth (38%)

**Commentary:** The recently published FHWA reference manual entitled LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations (2011) recommends that liquefaction be evaluated over the greatest of the following depths: (a) at least 20 ft (6 m) below the lowest expected foundation level for deep foundations, or (b) 80 ft (24 m) below the existing ground surface or lowest proposed finished grade. It should be noted that the geologic and hydrogeologic setting of the site should also be part of the basis for determining the required depth of analysis.

8. What water level do you use in the liquefaction analysis for level sites?
   - water level observed during field exploration (43%)
   - an assumed elevated water level for earthquake (48%)
   - ground surface (9%)

**Commentary:** Selection of the water level determines the minimum depth of potential liquefaction and an emphasis on identification of the water level during site exploration is well placed. A significant amount of published research exists (e.g., Okamura and Soga, 2006; Hossain et al, 2013) that supports the conclusion that partially saturated soils (even those soils near saturation) have a significantly greater resistance to liquefaction than fully saturated soil. It is understood that the water table can fluctuate, but trapped air is typically present for short term high water table events. It may be overly conservative to select a high water table for liquefaction analysis, especially where the high water table is a temporary condition.

9. What water level do you use in the liquefaction analysis for slopes?
   - water level observed during field exploration (40%)
   - an assumed elevated water level for earthquake (53%)
   - ground surface (7%)

10. What is the minimum thickness of soil layer that you consider to be significant with respect to liquefaction potential?
    - no minimum thickness (31%)
    - 6 to 12 inches (150 to 300 mm) (20%)
    - 12 to 24 inches (300 to 600 mm) (23%)
    - greater than 24 inches (600 mm) (26%)

**Commentary:** Whether or not to exclude thin zones that may be categorized as liquefiable can have a dramatic effect on code-related design decisions. For example, the 2012 International Building Code requirements for steel reinforcement for cast-in-place deep foundations can be controlled by the location of "strata that are liquefiable". To illustrate an extreme but not unrealistic scenario, a long reinforcing cage length could be interpreted to be a code requirement based on a few data point(s) within a very detailed CPT sounding. Judgment should be applied as there are typically other important considerations, such as constructability and quality.
11. What screening criteria do you primarily use to differentiate between “sand-like” and “clay-like” cyclic behavior of soils?
   - Chinese criteria (9%)
   - Idriss and Boulanger (69%)
   - Bray and Sancio (13%)
   - Other (9%)

Commentary: The Chinese criteria are no longer considered to provide a suitable indication of “clay-like” cyclic behavior.

12. What reference in published literature do you primarily use for estimating liquefaction-induced settlement for sands?
   - Tokimatsu and Seed, 1987 (45%)
   - Ishihara and Yoshimine, 1990 (30%)
   - Zhang, Robertson and Brachman, 2002 (16%)
   - Other (9%)

Commentary: In his 2012 H. Bolton Seed lecture in Oakland, California, Dr. Geoffrey Martin presented laboratory test results supporting that sands may experience different degrees of liquefaction-induced compression depending on their gradation, shape, etc. Shamoto et al. (1996) showed that the liquefaction-induced compression can be uniquely related to the relative compression defined by \(\Delta e/(e_i - e_{\text{min}})\).

In his 2013 Ralph B. Peck lecture in San Diego, California, Dr. Jonathan Bray presented the results of post-earthquake reconnaissance and concluded that the procedures described above are not applicable for building settlements. While the procedures may be applicable to free-field conditions, they do not represent the conditions within the zone of influence of foundations. In general, these procedures are expected to under-predict building settlement, particularly for thinner liquefiable strata.

13. Do you estimate liquefaction-induced settlement of liquefiable fine-grained soils using the published charts for sands?
   - Yes (45%)
   - No (55%)

Commentary: Considering the recent developments that show that fine-grained soils are liquefiable, the absence of research concerning liquefaction-induced compression for these soils is understandable. Dr. Ed Kavazanjian at the Arizona State University stated that limited research suggests that published literature for estimating liquefaction-induced settlement for sands provides reasonable results for non-plastic silts (DFI Seminar, 2012).

14. What approach do you primarily use for estimating lateral spread in liquefiable sand?
   - Empirical correlations (53%)
   - Laboratory-based methods (13%)
   - Newmark sliding block analysis (15%)
   - Numerical modeling/analyses (10%)
   - Other (9%)

15. What approach do you primarily use for estimating lateral spread in liquefiable fine-grained soil?
   - Empirical correlations (47%)
   - Laboratory-based methods (13%)
   - Newmark sliding block analysis (18%)
   - Numerical modeling/analyses (11%)
   - Other (11%)

16. How much confidence do you put in the calculated lateral spread displacement?
   - 0 to 10% (12%)
   - 10 to 50% (64%)
   - 50 to 90% (21%)
   - greater than 90% (3%)

Commentary: Dr. Scott M. Olson of the University of Illinois presented the results of his research indicating that actual lateral spread displacements are within one-half to 2 times the predictions made based on the accepted estimation approaches (DFI Seminar, 2012)

Mitigation Design

These questions relate to liquefaction mitigation design in engineering practice. The questions and the answers (in terms of percentages) are:
17. How do you rank the following engineering tools for liquefaction mitigation designs?

<table>
<thead>
<tr>
<th>Engineering Tool</th>
<th>Very Important</th>
<th>Less Important</th>
<th>Least Important</th>
</tr>
</thead>
<tbody>
<tr>
<td>theory/analysis/modeling</td>
<td>49%</td>
<td>33%</td>
<td>19%</td>
</tr>
<tr>
<td>local precedence</td>
<td>25%</td>
<td>34%</td>
<td>41%</td>
</tr>
<tr>
<td>published reconnaissance of earthquake damage</td>
<td>26%</td>
<td>33%</td>
<td>40%</td>
</tr>
</tbody>
</table>

**Commentary:** While it is recognized that analysis should be validated by field performance, this is problematic in the practice of earthquake design where the opportunities for first-hand observations are rare. The implementation of liquefaction mitigation techniques solely on precedent does not explicitly consider the variation in soil and seismic conditions; however, numerical modeling without calibration and validation can provide misleading results.

18. How often do you use the following fundamental approaches to mitigate liquefaction on your projects?

<table>
<thead>
<tr>
<th>Fundamental Approach</th>
<th>Most often</th>
<th>Less often</th>
<th>Least often</th>
</tr>
</thead>
<tbody>
<tr>
<td>densification</td>
<td>51%</td>
<td>35%</td>
<td>14%</td>
</tr>
<tr>
<td>reinforcement</td>
<td>40%</td>
<td>46%</td>
<td>14%</td>
</tr>
<tr>
<td>drainage</td>
<td>9%</td>
<td>19%</td>
<td>72%</td>
</tr>
</tbody>
</table>

19. How often do you use the following densification methods to mitigate liquefaction on your projects?

<table>
<thead>
<tr>
<th>Densification method</th>
<th>Most often</th>
<th>Less often</th>
<th>Least often</th>
</tr>
</thead>
<tbody>
<tr>
<td>vibrocompaction</td>
<td>66%</td>
<td>27%</td>
<td>8%</td>
</tr>
<tr>
<td>dynamic compaction</td>
<td>18%</td>
<td>41%</td>
<td>41%</td>
</tr>
<tr>
<td>compaction grouting</td>
<td>16%</td>
<td>32%</td>
<td>51%</td>
</tr>
</tbody>
</table>

20. What technical resources (literature, software, etc.) do you primarily use in designing against liquefaction using densification?

**Commentary:** There was a wide variety of responses to this question. Specialty contractor's typically responded that their design approaches were proprietary. For the remaining respondents, the results to this question may be broadly categorized as follows:

- The criteria are established by the consultant but the design, implementation and verification are made the responsibilities of the specialty contractor;
- Spreadsheets and commercial software, and;
- Numerical models (e.g., Plaxis and FLAC)

21. How often do you use the following reinforcement methods to mitigate liquefaction on your projects?

<table>
<thead>
<tr>
<th>Reinforcement method</th>
<th>Most often</th>
<th>Often</th>
<th>Less often</th>
<th>Least often</th>
</tr>
</thead>
<tbody>
<tr>
<td>vibro-stone columns</td>
<td>55%</td>
<td>28%</td>
<td>11%</td>
<td>6.5%</td>
</tr>
<tr>
<td>rammed aggregate piers</td>
<td>19%</td>
<td>28%</td>
<td>17%</td>
<td>35%</td>
</tr>
<tr>
<td>grout columns</td>
<td>11%</td>
<td>20%</td>
<td>42%</td>
<td>26.5%</td>
</tr>
<tr>
<td>deep soil mixing cells</td>
<td>15%</td>
<td>24%</td>
<td>30%</td>
<td>32%</td>
</tr>
</tbody>
</table>

22. What technical resources (literature, software, etc.) do you primarily use in designing against liquefaction using reinforcement?

**Commentary:** The results to this question may be broadly categorized as follows:

- No analysis is typically performed but rely on precedent and judgment with the recognition that reinforcement may not fully mitigate the liquefaction;
- Use of methodology proposed by Baez and Martin (1993)
- Use of numerical models (e.g., Plaxis and FLAC)

23. How often do you use the following drainage methods to mitigate liquefaction on your projects?

<table>
<thead>
<tr>
<th>Drainage method</th>
<th>Most often</th>
<th>Less often</th>
<th>Least often</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ drains</td>
<td>22%</td>
<td>20%</td>
<td>57%</td>
</tr>
<tr>
<td>gravel drains</td>
<td>41%</td>
<td>44%</td>
<td>15%</td>
</tr>
<tr>
<td>pre-fabricated vertical drains</td>
<td>37%</td>
<td>36%</td>
<td>28%</td>
</tr>
</tbody>
</table>
Commentary: Vertical gravel drains were described by Seed and Booker (1972). However, they are not widely used in the United States. One reason is a concern about their effectiveness and reliability. It is recognized that vertical gravel drains need to reliably provide a high ratio of permeability between the drain material and the adjacent soil to prevent the buildup of high excess pore water pressure. Dr. Russell Green with Virginia Tech (DFI Seminar, 2012) presented research results showing that high degree of control during installation is required to maintain an effective gradation in order to achieve the target permeability.

From the comments provided by participants, it may be concluded that the use of drains is rarely relied upon as the primary or sole mechanism for mitigating liquefaction in the U.S. The use of EQ drains is focused on parts of the US, namely Charleston, SC.

24. What technical resources (literature, software, etc.) do you primarily use in designing against liquefaction using drainage?

Commentary: FEQDrain (Pestana et al., 1997) was recognized as a technical resource for the design of EQ drains. Gravel drains, which have been more popular in Japan (Towhata, 2008), can be designed by using the charts presented by Onoue (1988).

25. What is the typical liquefaction-induced settlement tolerance or design criteria used on your foundation projects?

- No liquefaction as determined by a required post-improvement SPT or CPT resistances (23%)
- 1 inch (25 mm) (27%)
- 3 inches (76 mm) (21%)
- Greater than 3 inches (76 mm) (6%)
- No maximum settlement so long as there is an adequate factor-of-safety against bearing capacity failure (23%)

Commentary: Participants commented that the type of structure and that whether the design is to be determined based on life safety or serviceability were important considerations regarding selection of the tolerable settlement. Assuming that most foundations can tolerate about one inch of settlement with only cosmetic damage, the results indicate that about half of the participants of the participants typically design for serviceability. Considering that we often use the one inch as the tolerable foundation settlement for non-seismic conditions as well, it is very conservative to use either one inch of liquefaction-induced settlement or no liquefaction even for serviceability.

26. Is the typical liquefaction-induced settlement tolerance or design criteria used on your foundation projects reasonable and achievable?

- Yes (82%)
- No (18%)

27. What is the typical lateral spread tolerance used on your foundation projects?

- less than 1 foot (0.3 m) (55%)
- 1 to 3 ft (0.3 to 0.9 m) (35%)
- greater than 3 ft (0.9 m) (10%)

28. Is the typical lateral spread tolerance or design criteria used on your foundation projects reasonable and achievable?

- Yes (87%)
- No (13%)

29. What primary reference do you use in estimating residual strength of liquefied soils?

- Seed and Harder, 1990 (23%)
- Olson and Stark, 2002 (17%)
- Idriss and Boulanger, 2008 (50%)
- Other (10%)

30. Would you be in favor of performance-based design where the tolerable ground movements were more closely related to the design of the structure?

- Yes (96%)
- No (4%)
Verification
These questions relate to verification of liquefaction mitigation efforts in engineering practice. The questions and the answers (in terms of percentages) are:

31. For mitigation dependent on densification, what approximate percentage of your projects includes post-improvement verification testing?

Commentary: The responses ranged from 0 to 100%. Approximately ½ of the participants responded that 100% of their projects included post-improvement verification testing and the majority of the remaining participants responded with values that were between 25% and 50%.

32. How often do you use the following techniques for post-improvement verification testing?

<table>
<thead>
<tr>
<th>Verification test technique</th>
<th>Most often</th>
<th>Less often</th>
<th>Least often</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Test</td>
<td>40%</td>
<td>42%</td>
<td>18%</td>
</tr>
<tr>
<td>Cone Penetration Test</td>
<td>55%</td>
<td>38%</td>
<td>6%</td>
</tr>
<tr>
<td>Shear-Wave Velocity Test</td>
<td>5%</td>
<td>20%</td>
<td>76%</td>
</tr>
</tbody>
</table>

33. When evaluating the densification by the CPT, do you use the fines content estimated from the pre-improvement or post-improvement?
   - Pre-Improvement (61%)
   - Post-Improvement (39%)

Commentary: The fines content interpreted from CPT data can change between the pre-improvement testing and post-improvement testing. This emphasizes that a good practice is to validate CPT data with the fines content determined from laboratory gradation testing performed on samples collected in the field.

34. For sites improved by densification, does your post-improvement liquefaction analysis consider ageing effects?
   - Yes (29%)
   - No (71%)

Commentary: Research supports that dynamic compaction, blasting and vibro-compaction can temporarily destroy inter-particle structure and bonds associated with aging. Therefore, the cone tip resistance is expected to increase with time after improvement using these techniques (Mitchell and Solymar, 1984; Schmertmann, 1986; Mesri et al., 1990; Charlie et al., 1992). Other densification methods such as compaction grouting, displacement piles, or compaction piles may also have the same effect although it has not been documented. Lunne et al. (1997) states the recommended procedure is to perform field trials at the start of the project by performing CPT at different time intervals after compaction to evaluate the significance of any time effect. There are financial drawbacks to such field trials including extending the construction schedule and requiring a greater amount of CPT services.

35. For sites improved by densification, does your post-improvement liquefaction analysis consider lateral stress relaxation?
   - Yes (27%)
   - No (73%)

Commentary: Mejia and Boulanger (1995) performed SPT and CPT to evaluate the effects of compaction grouting in silt and sand. The study observed a large increase in the penetration resistance one week after treatment. A loss of approximately 30% of the average increase was subsequently observed within the following 18 months.

36. For sites improved by densification, does your post-improvement liquefaction analysis consider lateral variation in the degree of densification?
   - Yes (48%)
   - No (52%)

Commentary: Degen (1998) reports that the practice of testing at the mid-point between three vibro-compaction improvement points (assuming an equilateral triangular spacing) introduces “a rather large additional factor of safety into the design”. Field data suggest that the CPT resistance is about 20% higher, only 500 mm (20 in) away from the midpoint.
37. How do you model the response of liquefied soil when evaluating lateral loading on a deep foundation?

- use a p-multiplier of 0.1 for loose sand and 0.25 for dense sand (16%)
- use the equivalent fluid pressure of the liquefied sand (19%)
- use a p-y curve for soft clay based on the residual strength (26%)
- use the Rollins et al. (2005) liquefied sand p-y curves (27%)
- other (12%)

**CONCLUSIONS**

On the basis of the subject survey, the following conclusions are presented:

1. The state-of-practice is perceived to be “somewhat” to “highly” non-uniform by a majority of the survey respondents. This illustrates the need for continued efforts to develop greater consensus within engineering practice for many of the issues included in this synthesis.

2. The SPT and CPT are the two primary tools for evaluating the site conditions for the design and verification for ground improvement for liquefaction mitigation.

3. A majority of the survey respondents use an elevated ground water level or a ground water level at the ground surface for their liquefaction analysis. Such a practice is expected to introduce conservatism because unsaturated soil (even soil near saturation) has a higher resistance to liquefaction than saturated soil.

4. Almost one-third of the survey respondents do not apply a minimum liquefied thickness when performing liquefaction analysis. Such a practice may introduce conservatism, especially where liquefaction is isolated to one or a few thin zones within the subsurface profile.

5. A performance-based design where the design criteria are determined based on the tolerance(s) of the proposed structure, is overwhelmingly preferred. While two levels of design performance were recognized (serviceability and life safety), it was not clear which level provides the basis of most designs.

6. Densification is the most implemented primary mechanism for liquefaction mitigation and is followed by reinforcement. Post-improvement testing for densification projects may involve significant judgment to consider the effects of cementation associated with aging, stress relaxation and lateral variation in improvement.

7. While the owner’s consulting engineers typically define the densification requirements, it is the specialty contractor (and/or their subconsultant) that is given the responsibilities of design, implementation and verification of the means and methods.

8. The application of reinforcement for liquefaction mitigation relies on precedence and judgment, as well as, the results of numerical modeling. Research is in progress to better define the efficacy of reinforcement and to develop simplified design methods (Nguyen et al., 2012; Rayamajhi et al., 2012.)

9. Drainage as the primary mechanism does not appear to be widely implemented in the U.S. Toshilata (2008) reports that the installation of drains for liquefaction mitigation has experienced a decrease in Japan.

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482.
(1) What are your reservations regarding the use of numerical modeling in the design of ground improvement for liquefaction mitigation? Please elaborate.

The biggest reservation is the generic classification “numerical modeling”, which would seem to allow anybody to pick up a standard package (Plaxis, Flac) with some standard properties lifted from the user manual to generate a result… “Garage In, Garbage Out”.

In principle I am a great fan of numerical modeling and think it will become the way forward. But, when specifying “modeling” it is also essential to add the following:

a) an appropriate stress strain model. you can do a lot with standard Mohr Coulomb provided that an appropriate dilation angle is chosen and $G = G_{\text{max}} / 3$, but for many ground improvement projects we will need to go further and adopt a “good” stress strain model that predicts how the proposed improvement changes the soil stiffness and strength. And the problem that you then run into is that none of the standard numerical codes used by engineers in practice have such models as one of their “menu” choices. So, you wind up with a decent numerical model that actually requires a user defined model where you code one of the “good” models for yourself – predictably, rarely done! Things will improve in the future as the software developers pre-pack good models into commercial codes, but even then users will need to be aware of how their, seemingly innocent, choice of model impacts their results.

b) Relevant and reliable soil properties. Too many times I have seen people (both clients and fellow engineers) ask for sophisticated finite element analysis using SPT blowcounts as the basis of soil properties; this is complete nonsense and simply produces delusions about adequacy. The very minimum is that anything using numerical analysis must have measured $G_{\text{max}}$ data as the starting point for the analysis (at least we get the

Appendix: Interviews

elastic response right, and Gmax/3 is not a bad approximation for the secant modulus to mobilized strength for many soils at usual FS). This is actually not a big $$$ requirement, as geophysical methods to assess Gmax are cheap and easy to do in situ and ‘bender elements’ are becoming readily available in commercial testing labs. It is a matter of appropriate understanding and attitude. How often do you see the analyst reporting calibration of their model to the soil behavior they are trying to capture?

c) Validation studies have been done. Although one might like to think (and hope) that numerical modeling would be formally correct, in reality there are ways of setting up the problems, sorting out initial conditions, and dealing with the loading conditions that all affect the results. As well as modeling in 2D when the works are 3D... All of which means that the procedures used need to be validated against ‘case histories’. This validation of modeling procedures is actually a requirement in the European standard EN 1997 (Eurocode 7), but not often done by the consulting firms I know.

If you put (a) – (c) together, you wind up with my fear that simply giving numerical methods as an allowable/desirable design approach will allow all sort of bogus work to be put in front of clients as good engineering. Done well, numerical methods are a brilliant technology to help us in what we do. But, they are done badly 95% of the time (in my view) and it would be better to not do them at all in this situation.

(2) What future improvements do you consider most important regarding the different technical aspects related to ground improvement for liquefaction mitigation - liquefaction analysis, design, verification? Please elaborate.

“None of the above”; the biggest problem I see is intellectual dishonesty/incompetence in academia where we have largely outsourced all development – possibly a surprising view, but it is detailed in my book! In reality, liquefaction is not difficult to analyze, but what is acceptable has been hijacked by one faction (lets us be charitable and call them the ‘engineering geology view’) and they have the support of the regulators. As engineers, we can do much better than present practice but it simply is not allowed.

On the bright side, I think the survey results show a super-majority consensus on post-treatment validation, and it would be easy to pull an ASTM standard together on this issue. The only real issue seems to be the degree of post-treatment aging we include and how that is assessed.

(3) What practices (either technical or non-technical) by those involved - consultant, designer, specialty contractor, owner - do you consider problematic to the consistency within the practice? Please elaborate.

a) Continued use of the SPT. The test, and its correction factors, are so variable that even a single organization is challenged in producing a uniform standard between projects if they base their work on the SPT.

b) Uncertainty and/or lack of understanding of how “finer content” affect liquefaction susceptibility, leading to wildly inconsistent engineering in anything other than clean sand. And it is not the contractors or owners who are the problem; academia is at a loss and consultants seemingly take little interest in challenging them.

c) Lack of best-practice guides. I’ve just been part of a consensus guide to compaction grouting, and really similar guides are needed for dynamic compaction and vibro-densification (in its various forms).

(4) Are you satisfied with the current approach for considering liquefaction-induced settlement and lateral spread? Do you consider these estimates as relative measures of liquefaction severity or actual/accurate values? Please elaborate.

No! None of the approaches to settlement are properly based on soil behavior (post
liquefaction settlement is a consolidation problem requiring Cc as a basic input parameter; any method without Cc as a soil property is bogus mechanics). Similar comments follow about lateral spreads with the exception of Newmark’s method.

(5) **Do you have any other comments that haven’t been covered by the survey or this interview?**

Could I suggest the compaction grout guide as a prototype of what is needed across other areas of the ground improvement industry?

**Interview with Dr. Les Youd, Professor Emeritus of Civil Engineering at Brigham Young University.**

(1) **Do you have any reservations regarding the use of numerical modeling in the design of ground improvement for liquefaction mitigation? Please elaborate.**

I have major reservations on this issue. Numerical modeling is only one of several tools that should be applied by designers. Over reliance on numerical modeling can lead to nonsensical results because of imperfect or inadequate models. With the present state of practice, it is generally impossible to construct accurate models of (1) subsurface soil stratigraphy, (2) lateral and vertical variances in stratigraphy, (3) soil properties, (4) variances in soil properties in space and time, and (5) imposed seismic loads in both space and time. Numerical analyses are generally useful to gain a rough perspective of expected results and to perform parametric analyses to estimate how the results might change with variations in stratigraphy, soil properties, loading assumptions, etc., but not as the sole basis for design.

A more important tool, in my opinion, is assessment of case histories of past performance compiled from post-earthquake investigations and successful or unsuccessful similar projects. Empirical procedures are generally applied in present practice; these procedures were primarily developed from analysis of case histories, and thus are grounded on observed actual performance. Although past performance and empirical techniques may not allow exact duplication of site conditions and constraints for each project, they usually provide realistic estimates as a guide to the design engineer.

The third tool that should be applied is engineering judgment. Expert guidance from those with past earthquake experience and from analysis and design should be sought after to assure that sound engineering judgment is applied on all critical ground modification projects. Conversely, design of critical ground modification projects should not be entrusted to inexperienced engineers, although they may have attained expert computer skills but with little practical experience.

(2) **What future improvements do you consider most important regarding the different technical aspects related to ground improvement for liquefaction mitigation - liquefaction analysis, design, verification? Please elaborate.**

I believe that much money has been wasted in the past on mitigation to prevent liquefaction from occurring, when the occurrence of liquefaction would not lead to significant damage. To avoid such waste, education of the profession is needed to increase understanding of the following key points (extracted from a short paper I prepared for the ASCE publication Geo-Strata (Youd, September-October 2011, p. 53-54)). This paper was written to increase this needed understanding. “In analyzing liquefaction the following fundamental questions should be asked and appropriate answers and actions determined: 1. Will liquefaction occur? If the answer to this question is “no,” mitigation is obviously not required. If the answer is “yes,” the analysis proceeds to the second question: 2. Will liquefaction lead to potentially damaging ground deformations, displacements or ground failure? If the answer is “no,” liquefaction will not cause significant damage and the hazard can be safely accepted without mitigation. If the answer is “yes,” the analysis proceeds to the third question: 3. What mitigative measures are
required to reduce the hazard to an acceptable risk?" Only when this level of understanding has been gained should the analysis proceed to design of mitigation measures.

Verification of the effectiveness of ground modifications has been a major issue for several projects I have encountered. Additional research, discussion and consensus building is required to improve verification procedures for use in engineering practice.

(3) What practices (either technical or non-technical) by those involved - consultant, designer, specialty contractor, owner - do you consider problematic to the consistency within the practice? Please elaborate.

Procedures for liquefaction hazard evaluation and mitigation relies heavily upon empirical procedures, which are generally based on analyses of collected case histories and performance assessments. Development of empirical procedures usually occurs through research and analyses by individual investigators or teams of investigators; these investigators or teams do not always (or seldom) agree; thus development of empirical procedures tends to be a messy and often chaotic process; disagreements and disputes are common and to be expected. Most of this chaos occurs at the researcher and consultant levels. Because of this chaos, practitioners and designers often are confused or uncertain as to which expert they should rely on or which procedure they should follow. With time the chaos usually calms as procedures are vetted or tested and consensus builds. Sometimes professional societies or other professional groups can speed the process through workshops or expert panels to develop consensus guidelines for engineering practice. Such was the case with the NCEER/NSF workshop I chaired in 1996 on evaluation of liquefaction resistance, which developed consensus guidelines that calmed the atmosphere for triggering evaluations for about 10 years. Groups such as DFI may assist by organizing or supporting workshops or panels of this type to build consensus and reduce chaos and develop improved and more consistent usage within the profession.

(4) Are you satisfied with the current approach for considering liquefaction-induced settlement and lateral spread? Do you consider these estimates as relative measures of liquefaction severity or actual/accurate values? Please elaborate.

a) Lateral Spread: As an author of the empirical MLR procedure, one of the more widely used procedures for evaluating lateral spread displacement, I feel that the MLR procedure is a valid procedure if applied within the limitations specified by the authors (Youd et al 2002 from Journ of Geotech and Geoenviron Engr, v. 128, no 12, p. 1007-1017). This procedure provides mean predicted values that are demonstrated to be accurate within a factor of plus or minus two if applied within the specified limits. Extension beyond the stated limits leads to greater uncertainty of results. Because the MLR procedure is empirical, it is not valid for all conditions that may be encountered. In some instances extrapolation using numerical procedures may allow reasonable, but still uncertain results for a wider range of site conditions. For example, inclusion of a deep foundation for a bridge in a sediment cross-section to be analyzed creates a condition beyond that in the empirical database. The additional influence of this bridge foundation could be analyzed through numerical procedures. An accuracy of plus or minus 2 may seem too uncertain for engineering applications, however, such uncertainties are common in other geotechnical engineering calculations. For example, similar uncertainty is inherent in calculations of bearing capacity and foundations settlement under static conditions.

One of the greater sources of erroneous results that I have encountered reviewing the work of others using the MLR procedure is insufficient geotechnical information. Often analyses are made on the basis of one or a few boreholes or soundings exacerbated by an improper assumption that penetrated soil layers are laterally homogeneous.
and continuous across and beyond site boundaries. If critical layers thin or pinch out or important facies changes occur within the layer, inaccurate to nonsensical predicted displacements may be calculated. Thus, I feel that adequately accurate tools are available for calculation of lateral spread displacements for many applications. However, MRL and other empirical procedures need to be verified and updated as additional earthquakes occur and new case histories are developed.

b) Ground Settlement: The same general limitations apply to empirical procedures for calculation of liquefaction-induced ground settlement as for lateral spread. However, the limitations for ground settlement do not seem to be as well defined as for lateral spread. For example, most empirical settlement procedures appear to be based on relatively clean sand conditions. Limits on the procedure with respect to silt and gravel contents do not seem to be clearly defined. Thus, the user must evaluate soil conditions at a site in question, compare those conditions against those implicit in the development of the empirical procedure, and then make an unspecified adjustment for incompatible soil conditions. Such adjustments increase the uncertainty of the calculated settlements.

(5) **Do you have any other comments that haven't been covered by the survey or this interview?**

From reading the survey text and these questions, difficulties faced by design engineers appear to stem from one or more of the following issues:

(1) lack of adequate communication between researchers, expert consultants, analysts, and designers;

(2) confusion within these same groups with respect to which procedures should be recommended for application in practice, and;

(3) lack of adequate research and verification of procedures to answer many fundamental questions.

DFI and other professional organizations could play a major role in fostering communication, supporting studies to develop and verify procedures, education of professionals at all levels, assisting profession to identify unresolved issues, and assisting in the development of support for research, workshops and other means to resolve important issues.

**Interview with Ikuo Tawhata, Professor of Geotechnical Engineering at the University of Tokyo and author of the “Geotechnical Earthquake Engineering” from Springer Series.**

(1) *Do you have any reservations regarding the use of numerical modeling in the design of ground improvement for liquefaction mitigation? Please elaborate.*

Everybody points out the shortcomings of the use of numerical analysis in design. Those shortcomings are caused by the complex stress-strain-dilative behavior of soils, heterogeneous subsoil conditions that cannot be fully recorded by current soil investigations and many others. Although those critical attitudes are understandable, I feel that some critics use those shortcomings as an excuse for not challenging advanced (numerical) studies. Recent desires towards seismic performance design require approaches that can calculate residual deformation of structures in place of the conventional factor of safety. I would ask a question whether or not the traditional non-numerical approaches are more reliable and more useful than the numerical approaches. The traditional approaches often rely on empirical knowledge and their use is certainly limited within the range of available knowledge. It is risky to apply them to totally new soil and load conditions. Moreover, the traditional methods cannot be applied to the behavior of complicated underground structures that are subject to liquefaction of soils around. In this regard, we should not discriminate numerical methods. They should be considered to be tools which
give us indices that help us assess the seismic performance of structures to be designed. Do not misunderstand that I am trying to favor numerical methods. Good numerical methods have to be associated with elaborate but costly field / laboratory investigations. I feel that many current projects do not allocate reasonable budget to investigations, resulting in unexpected difficulty during the later construction stages. A small investigation budget results in a great loss of money and time during construction.

The attitudes of numerical people are also a problem. They do not go to the site. They prefer to stay in a comfortable office and 100% trust documented data. They do not imagine that the reality is more complicated than information in paper.

Because my most interested field of study is the assessment of liquefaction-induced large displacement, I should make one more point about numerical approaches. To my knowledge, the constitutive models that are employed in major computer codes were developed in 1970s and 80s when nobody cared that the liquefaction-induced large deformation of subsoil. Also, even today laboratory devices cannot reproduce such large shear deformation as 30%, 50%, or more. Laboratory tests after the onset of liquefaction are not possible because of segregation of water and sand grains within a tested specimen. Therefore, those constitutive models are not fully supported by laboratory test data after onset of liquefaction and development of large shear deformation.

In summary, I would propose to use both simple traditional approaches and numerical approaches and compare them prior to drawing the final conclusion.

(2) What future improvements do you consider most important regarding the different technical aspects related to ground improvement for liquefaction mitigation - liquefaction analysis, design, verification? Please elaborate.

After the M=9 gigantic earthquake in 2011, I encountered a very difficult problem of soil improvement. It was how to improve soil (reduction of liquefaction vulnerability) under existing houses with relatively low financial burden to house owners.

The current solution is two-fold. For a frequent design earthquake (return period being about 50 years), public and private funds are combined. Liquefaction vulnerability is mitigated by either constructing underground rigid walls under streets and house-lot borders to constrain cyclic shear deformation of soil, or pumping ground water to lower the ground water level and to create an unliquefiable soil crust. The former has a limitation that the spacing of walls cannot be very small because of existing houses at the surface. The latter has a problem of possibly triggering consolidation settlement in the underlying thick soft clay.

Note further that house owners have to spend their own money on soil improvement, if they wish to do it, against a stronger design earthquake with a return period of hundreds of years. This is difficult and costly because the ground surface is occupied by a house and compaction or other traditional soil improvement is not possible.

Soil improvement under existing structures is further important for big industries as well. This is because the intensity of design earthquake tends to be increased and existing old structures cannot satisfy the safety requirement under newer regulations.

I suppose that the following kinds of soil improvement are feasible under existing structures; installation of drainage, compaction grouting by good technicians, injection of colloidal silica, and construction of underground walls around the foundation of houses. Noteworthy is that some improvement methods cannot prevent the onset of liquefaction but reduce the residual deformation. Thus, methodology is necessary to assess the residual deformation of subsoil and surface structures (possibly by a numerical method) and also to determine the allowable extent of deformation.
Gravel drains became less popular after the 1995 Kobe earthquake. This is because the intensity of design earthquake was increased and design calculation could not prove that gravel drains under the stronger design earthquake can still maintain the development of excess pore water pressure less than 50% of the full liquefaction. However, it is possible that the columns of gravel drain maintain some rigidity during a strong earthquake and reinforce the stability and integrity of subsoil. Further study is needed in this direction.

(3) **What practices (either technical or non-technical) by those involved - consultant, designer, specialty contractor, owner - do you consider problematic to the consistency within the practice? Please elaborate.**

Owners try not to spend sufficient money on subsoil investigation. Hence all the input data for analysis have to be determined by SPT-N only. Although liquefaction of fine-grained soil is important, plasticity index is hardly measured. Owners should understand that they should allocate more money on soil investigation so that they can avoid unnecessary big expenditures on construction and unnecessary liquefaction damage during future earthquakes.

Some consultants do not want to visit sites. They prefer to stay in the office and analyze the supplied borhole data. For them, the data on paper is the reality and they do not want to experience the reality on site. One reason for this situation is found in the owners who do not pay sufficient money for field activities. To accurately interpret borehole data, it is important for consultants to have good knowledge of local soils and local geological history as well as history of human action on soils such as land reclamation and consolidation settlement. Hence, it is possible that a local experienced consultant is better than an international famous consultant.

(4) **Are you satisfied with the current approach for considering liquefaction-induced settlement and lateral spread? Do you consider these estimates as relative measures of liquefaction severity or actual/accurate values? Please elaborate.**

My attitude towards numerical methods was described already in (1). I do not think that numerical methods are worse than simplified and traditional factor-of-safety approach. All the methods give us an index to assess the performance of a designed structure.

Numerical methods can assess the lateral displacement of liquefied subsoil with an error of ±50%. This is not too bad. The error of non-numerical methods is most probably similar.

Some people state that the Newmark rigid block analogy is better than other methods to assess the liquefaction-induced lateral displacement. I would say that the use of the Newmark method in liquefaction problem is beyond the applicability of this method, because Newmark intended to calculate the movement of a "rigid block" subjected to seismic action. Liquefied soil is never a rigid block.

(5) **Do you have any other comments that haven't been covered by the survey or this interview?**

There are several methods of subsoil investigation. In publications, I often encounter such an article in which the author insists that his device is able to determine all the required soil data accurately. In reality, this is difficult. Every method has good points and bad points. I suggest that we should combine different methods and get the best subsoil data. It is good to interpolate a big spacing among SPT by means of less expensive CPT or other device.

It is often stated that engineering judgment is extremely important and that less experienced engineers should not be trusted. Then it becomes important how to produce the next generation of experienced engineers. If a young engineer is not trusted, he will never become an experienced one. Moreover, the engineering judgment is a kind of empiricism. During the medieval time, technology relied totally on empiricism. There was no systematic education. Hence, the power of technology was
poor. This situation was changed drastically during the time of modern technology and education because the problems were analyzed, interpreted, understood, and solved. Long patience behind a “meister” is not necessary any more. It should be borne in mind that too much emphasis on empiricism will push things back to the medieval time.