Liquefaction

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- What is Liquefaction?
- Why does the Liquefaction occur?
- When has Liquefaction occurred in the past?
- Where does Liquefaction commonly occur?
- How can Liquefaction hazards be reduced?
What is Liquefaction?

- Liquefaction occurs in saturated soils. All pores are completely filled with water; Soil is no more thirsty!!

- The water in the pores exerts the pressure on soil particles called Pore Pressure that influences how tightly the soil particles are pressed together.

Why does the Liquefaction occur?

- Prior to an earthquake: the pore pressure is low.

- Earthquake shaking causes pore pressure to Increase: Undrained Condition!!

- Pore pressure = Overburden press/Confining stress. Effective stress is Zero; NO Shear Strength!!

- Soil particles start readily move with respect to each other due to zero shear strength. LIQUEFACTION …occurs!!
Mechanism: Why does the Liquefaction occur?

Soil grains in a soil deposit

The small contact forces are because of the high water pressure.

In an extreme case, the porewater pressure may become so high that many of the soil particles lose contact with each other. In such cases, the soil has very little strength, and behaves like a liquid; called “Liquefaction”.

When has Liquefaction occurred in the past?

- Alaska Earthquake, USA, 1964
- Niigata Earthquake, Japan, 1964
- Loma-Prieta Earthquake, USA, 1989
- Kobe Earthquake, Japan, 1995
- Chi-Chi Earthquake, Taiwan, 1999
- Bhuj Earthquake, India, 2001
- Many More…!!
Liquefaction: Lateral Displacement

Alaska Earthquake, USA, 1964: Displacement of Houses & cracked highway

Liquefaction: Tilting of Buildings

Niigata Earthquake, Japan, 1964: Tilting of apartment buildings at Kawagishi-Cho
Liquefaction: Collapse of Expressways

Kobe Earthquake, Japan, 1995: Collapse of Hanshin Expressway & collapse of Nishinomiya bridge

Bhuj Earthquake, India, 2001: Damage to rail-road and highway; sand boiling due to liquefaction
Failed river dike, California, 1989

Collapsed State highway, California, 1989

Sand boils in a house at Shetou, Taiwan, 1999

Sand boils in a room in Lunya-Li in Yuanlin Town, Taiwan, 1999
Sand boils in Loma Prieta Earthquake, 1989, California

Liquefaction-induced settlement at Taichung Port, Taiwan, 1999
1964 Niigata EQ
Multi span Bridge
Movement of Bridge Piers due to Lateral Spreading

1964 Niigata EQ
Tilting of Building
Loss of Bearing Capacity
Where does Liquefaction commonly occur?

Liquefaction occurs in saturated soil only, its effects are most commonly observed in low-lying areas near bodies of water such as rivers, lakes, bays, and oceans. Port and wharf facilities are often located in areas susceptible to liquefaction, and many have been damaged by liquefaction in past earthquakes.

Most ports and wharves have major retaining structures, or quay walls, to allow large ships to moor adjacent to flat cargo handling areas. When the soil behind and/or beneath such a wall liquefies, the pressure it exerts on the wall can increase greatly - enough to cause the wall to slide and/or tilt toward the water.

Liquefaction also frequently causes damage to bridges that cross rivers and other bodies of water. Such damage can have drastic consequences, impeding emergency response and rescue operations in the short term and causing significant economic loss from business disruption in the longer term.
Liquefaction Susceptible Soil

- Soil
  - Cohesionless
  - Loose (sometimes dense too)
  - Saturated
  - Why not in Dense Soil ??
    (negative pore pressure generation during shearing…!!)

Liquefaction: Principal Stress

*Increase in principal stress*
Shear Stresses in Soils under Earthquakes

- Under EQ ground motion
  - Shear stresses generated at bed-rocks
    - Transmitted upwards
  - Loose cohesionless soils tend to densify and settle
  - Pore water is forced out from the voids
  - Shaking duration too short for pore water to drain
    - Pore water pressure increases
  - Effective stress reduces
  - In critical state, soil loses stiffness & shear strength

Liquefaction: Flow Liquefaction

- Flow Liquefaction
  - Only in loose to moderately dense granular soils (silty sands, sands and gravels) with poor drainage
  - Soil stratum softens causing large cyclic shear deformations
  - In loose materials, associated loss of shear strength may cause flow failure
  - For sharp slopes (>~5 %), landslides on a large scale
  - Hundreds of meters of motion of soil mass
Liquefaction: Cyclic Mobility

Cyclic mobility
- In both loose and dense soils
- Gentle slopes (<~3 %)
- Movement of large blocks of mass towards free end
- Sliding and disintegration of large soil mass
- Tilting and subsidence of over-lying structures
- Lateral spreading

Cyclic mobility...
- Level-ground liquefaction in layered soils
Large Ground Oscillations

- Liquefaction at depth
- Separation of layers
- Ground soil layers jostle back and forth
- Ground waves
- Lateral spreading due to flow liquefaction and/or cyclic mobility

Loss of Bearing Capacity

- Liquefied underlying soil supporting the structure
- Subsidence of heavier structures
- Uplifting of lighter structures
Liquefaction Hazard

- Seismic History Criterion
  - Confined to a zone within a distance from EQ source
  - Magnitude
  - Distance from hypocenter
  - Local sub-surface conditions

Earthquake Magnitude
OR
Earthquake Intensity
is used for Designing the Structures??

Liquefaction Hazard

- Seismic history of the site
  - Densifying
  - Better inter-locking between particles
  - Increased shear resistance

- Earthquake ground shaking characteristics
  - Intensity
  - Duration

Structure should be
Earthquake Resistant
OR
Earthquake Proof.......??
Liquefaction Potential: Research

- Referred as “Simplified Procedure” to evaluate the Liquefaction potential of Soils (Seed & Idriss, 1971; ASCE)
- Largely empirical evolved over last 30 yrs
- Research reviewed and recommended in 2 workshops, NCEER 1996 and NCEER/NSF 1998
- Summary report in ASCE journal of Geotechnical and Geo-environmental Engg., October 2001
  Title: “Liquefaction Resistance of Soils”

Liquefaction Potential Evaluation using Cyclic Stress Approach (Youd et al., Oct 2001, ASCE)

- Earthquake loading is characterized by the amplitude of an equivalent uniform cyclic stress
- Liquefaction resistance is characterized by an equivalent uniform cyclic stress required to produce liquefaction in the same number of cycles
- For Liquefaction evaluation: compare loading and resistance.
- Loading: Equivalent stress induced by Earthquake
- Resistance: Stress required for Liquefaction

Liquefaction Resistance
OR
Liquefaction Susceptible
OR
Liquefaction Prone /Liquefiable Soils
SAME??
Initiation of Liquefaction

Cyclic Shear Stress

Depth

Zone of Liquefaction

Stress required for Liquefaction

Equivalent stress induced by earthquake

Evaluation of Liquefaction potential

\[
CSR = \frac{\tau_{av}}{\sigma_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}} r_d
\]

\[
CRR = CRR_{1.5} \times k_m \times k_\alpha \times k_\sigma
\]

\[
F_s = \frac{CRR}{CSR^2}; \text{If } F_s < 1.0; \text{ Liquefaction}
\]

CSR = Cyclic Stress Ratio

Seismic demand on the Soil due to Earthquake

CRR = Cyclic Resistance Ratio

Capacity of Soil to resist Liquefaction
Characterizing Earthquake Load

Irregular time history of shear stress is converted to an equivalent series of uniform stress cycles at an amplitude of 65% of peak shear stress (Seed et al. 1975).

\[ \tau_{av} = 0.65 \frac{a_{\text{max}} \sigma}{g} \]

\( \sigma \) = stress

\( \tau \) = shear stress

\( a \) = ground acceleration

\( \tau_{av} \) = average shear stress

\( \sigma \) = normal stress

\( g \) = acceleration due to gravity

Stress Reduction Coefficient, \( r_d \)

\[ r_d = 1.0 - 0.00765z \quad \text{for} \quad z \leq 9.15 \text{m} \] (2a)

\[ r_d = 1.174 - 0.0267z \quad \text{for} \quad 9.15 \text{m} < z \leq 23 \text{m} \] (2b)
Cyclic Stress Ratio (CSR)

- Seismic demand on the soil
- Seed & Idriss (1971)

\[
CSR = \frac{\tau_{av}}{\sigma_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}} r_d
\]

\[\sigma_{vo} = \text{Total vertical overburden stresses}\]
\[\sigma_{vo}^* = \text{Effective vertical overburden stresses}\]

Cyclic Resistance Ratio (CRR)

- Capacity of soil to resist liquefaction
- Liquefaction case histories used to characterize CRR in terms of measured in-situ test parameters (Whitman 1971)
- Parameter generally used
  - SPT
  - CPT
  - Shear Wave Velocity (SWV)

\[
CRR = CRR_{7,5} \times k_m \times k_{\alpha} \times k_{\sigma}
\]
### SPT Correlations

- **CRR\textsubscript{7.5}** for SPT data
  - CRR for M7.5 eqk for Corrected Blow Count (N\textsubscript{1})\textsubscript{60} normalized for 100 kPa overburden pressure and hammer efficiency of 60%
  - Correction factors for non-standard SPT values

![SPT Correlations Diagram](image)

*(Youd et al. 2001)*

### CPT Correlations

- **CRR\textsubscript{7.5}** for CPT data
  - CRR for M7.5 eqk for Normalized cone tip resistance (q\textsubscript{c(N)})\textsubscript{cs}
  - Correction factors for grain characteristics, and thin layer effect

![CPT Correlations Diagram](image)

*(Youd et al. 2001)*
SWV Correlations

- $CRR_{7.5}$ for Shear wave velocity data
- CRR for M7.5 Earthquake for normalized shear wave velocity
- Correction factors

$CRR = CRR_{7.5} \times k_m \times k_\alpha \times k_\sigma$

Earthquake Magnitude

- Correction to $CRR_{7.5}$ for Earthquake Magnitude other than 7.5 ($k_m$)

(Youd et al. 2001)
Overburden Pressure

- **Correction to CRR\textsubscript{7.5} for high overburden stresses, \( k_\sigma \)**
  - Correction factor for CRR\textsubscript{7.5} to extrapolate for soil layers with overburden pressures greater than 100 kPa
  - Simplified procedure is valid for depths less than 15 m.

\[
CRR = CRR_{7.5} \times k_m \times k_\alpha \times k_\sigma
\]

---

Initial Static Shear

- **Correction to CRR\textsubscript{7.5} for initial static shear,**
  - Correction factor for CRR\textsubscript{7.5} to account for initial static shear stress conditions such as due to presence of embankment, heavy structures, etc.

\[
CRR = CRR_{7.5} \times k_m \times k_\alpha \times k_\sigma
\]
Factor of safety against liquefaction

\[ F_s = \frac{CRR}{CSR} \]

**Example: Liquefaction potential Evaluation**

**Problem:**
- SPT data and sieve analysis
- Water table at 6 m below GL
- Expected earthquake M7.5 & Seismic Zone V

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>N₆₀</th>
<th>Soil Classification</th>
<th>Percentage fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>9</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>11</td>
</tr>
<tr>
<td>3.75</td>
<td>17</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>16</td>
</tr>
<tr>
<td>6.75</td>
<td>13</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>12</td>
</tr>
<tr>
<td>9.75</td>
<td>18</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>8</td>
</tr>
<tr>
<td>12.75</td>
<td>17</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>8</td>
</tr>
<tr>
<td>15.75</td>
<td>15</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>7</td>
</tr>
<tr>
<td>18.75</td>
<td>26</td>
<td>Poorly Graded Sand and Silty Sand (SP-SM)</td>
<td>6</td>
</tr>
</tbody>
</table>
Example: Liquefaction potential Evaluation

Liquefaction potential at depth 12.75 m

\[ \frac{a_{\text{max}}}{g} = 0.24, \ M_w = 7.5 \]

\[ \gamma_{\text{sat}} = 18.5 \text{ kN/m}^3, \ \gamma_w = 9.8 \text{ kN/m}^3 \]

\[ \sigma_v = 12.75 \times 18.5 = 235.9 \text{ kPa} \]
\[ u_0 = (12.75 - 6.00) \times 9.8 = 66.2 \text{ kPa} \]
\[ \sigma'_v = (\sigma_v - u_0) = 235.9 - 66.2 \]
\[ = 169.7 \text{ kPa} \]

\[ \text{CSR} = \frac{\tau_{sv}}{\sigma'_{sv}} = 0.65 \frac{a_{\text{max}}}{g} \frac{\sigma_{sv}}{\sigma'_{sv}} r_d \]

Example: Liquefaction potential Evaluation

Liquefaction potential at depth 12.75 m

- Compute stress reduction factor

\[ r_d = 1.174 - 0.0267z = 0.834 \]

- Compute CSR

\[ \text{CSR} = 0.65 \times (a_{\text{max}} / g) \times r_d \times (\sigma_v / \sigma'_v) \]
\[ \text{CSR} = 0.65 \times (0.24) \times 0.834 \times (235.9/169.7) = 0.18 \]
Example: Liquefaction potential
Evaluation

- Liquefaction potential at depth 12.75 m ...
  - Compute CRR

\[
CRR = CRR_{7.5} \times k_m \times k_\alpha \times k_\sigma
\]

Read value corresponding to \((N_1)_{so} = 14\) (after overburden correction) and fines content of 8%

\[
CRR_{7.5} = 0.143
\]

\[
k_m = 1.0 \quad k_\alpha = 1.0
\]

For vertical effective stress of 169.7 kPa = 1.7 atm \(k_\sigma = 0.87\)

Example: Liquefaction potential
Evaluation

- Liquefaction potential at depth 12.75 m ...
  - Compute CRR \((N = 17, N' = 14)\) (fines = 8%)

\[
CRR = CRR_{7.5} \times k_m \times k_\alpha \times k_\sigma
\]

\[
CRR = 0.143 \times 1 \times 1 \times 0.87 = 0.12
\]

- Compute \(F_s\)

\[
F_s = \frac{CRR}{CSR} = \frac{0.12}{0.18} = 0.67
\]
Example: Liquefaction potential Evaluation

- Repeat for all depths ...

<table>
<thead>
<tr>
<th>Depth</th>
<th>% fines</th>
<th>$\sigma_v$ (kPa)</th>
<th>$\sigma_v'$ (kPa)</th>
<th>$N_{60}$</th>
<th>$C_N$</th>
<th>$(N_i)_60$</th>
<th>$r_d$</th>
<th>CSR</th>
<th>CRR</th>
<th>$F_v$</th>
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</thead>
<tbody>
<tr>
<td>0.75</td>
<td>11.0</td>
<td>13.9</td>
<td>13.9</td>
<td>9</td>
<td>2.00</td>
<td>18</td>
<td>0.99</td>
<td>0.15</td>
<td>0.22</td>
<td>0.22</td>
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<tr>
<td>3.75</td>
<td>16.0</td>
<td>69.4</td>
<td>69.4</td>
<td>17</td>
<td>1.13</td>
<td>19</td>
<td>0.94</td>
<td>0.15</td>
<td>0.29</td>
<td>0.29</td>
</tr>
<tr>
<td>6.75</td>
<td>12.0</td>
<td>124.9</td>
<td>117.5</td>
<td>13</td>
<td>0.98</td>
<td>13</td>
<td>0.95</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>9.75</td>
<td>8.0</td>
<td>180.4</td>
<td>143.6</td>
<td>18</td>
<td>0.90</td>
<td>16</td>
<td>0.91</td>
<td>0.18</td>
<td>0.19</td>
<td>0.17</td>
</tr>
<tr>
<td>12.75</td>
<td>8.0</td>
<td>235.9</td>
<td>169.7</td>
<td>17</td>
<td>0.83</td>
<td>14</td>
<td>0.83</td>
<td>0.18</td>
<td>0.14</td>
<td>0.12</td>
</tr>
<tr>
<td>15.75</td>
<td>7.0</td>
<td>291.4</td>
<td>195.8</td>
<td>15</td>
<td>0.80</td>
<td>12</td>
<td>0.75</td>
<td>0.17</td>
<td>0.13</td>
<td>0.11</td>
</tr>
<tr>
<td>18.75</td>
<td>6.0</td>
<td>346.9</td>
<td>221.9</td>
<td>16</td>
<td>0.77</td>
<td>20</td>
<td>0.67</td>
<td>0.16</td>
<td>0.23</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Liquefaction Potential for different soils using SPT

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$\sigma_{min}$</th>
<th>$\sigma_{max}$</th>
<th>$(N_i)<em>{60}$ blows/ft; $\sigma</em>{max}$ MPa; $V_{s15}$ m/s</th>
<th>Cyclic stress ratio</th>
<th>Cyclic resistance ratio</th>
<th>FS = CRR/CSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean sand</td>
<td>0.40</td>
<td>7/8</td>
<td>7.7 blows/ft</td>
<td>0.34</td>
<td>0.09</td>
<td>0.26</td>
</tr>
<tr>
<td>Sand—15% fines</td>
<td>0.10</td>
<td>7/8</td>
<td>7.7 blows/ft</td>
<td>0.08</td>
<td>0.14</td>
<td>1.47</td>
</tr>
<tr>
<td>Clean sand</td>
<td>0.20</td>
<td>7/8</td>
<td>7.7 blows/ft</td>
<td>0.17</td>
<td>0.14</td>
<td>0.82</td>
</tr>
<tr>
<td>Clean sand</td>
<td>0.40</td>
<td>7/8</td>
<td>5.8 MPa</td>
<td>0.34</td>
<td>0.70</td>
<td>0.26</td>
</tr>
<tr>
<td>Clean sand</td>
<td>0.40</td>
<td>7/8</td>
<td>185 m/s</td>
<td>0.34</td>
<td>0.16</td>
<td>0.47</td>
</tr>
<tr>
<td>Crushed limestone</td>
<td>0.40</td>
<td>7/8</td>
<td>5.0 MPa</td>
<td>0.34</td>
<td>0.11</td>
<td>0.53</td>
</tr>
<tr>
<td>Silty gravel</td>
<td>0.40</td>
<td>7/8</td>
<td>7.5 MPa</td>
<td>0.24</td>
<td>0.27</td>
<td>0.79</td>
</tr>
<tr>
<td>Clean gravelly sand</td>
<td>0.40</td>
<td>7/8</td>
<td>14 MPa</td>
<td>0.34</td>
<td>0.44</td>
<td>1.29</td>
</tr>
<tr>
<td>Silt sand</td>
<td>0.40</td>
<td>7/8</td>
<td>7.7 blows/ft</td>
<td>0.34</td>
<td>0.09</td>
<td>0.26</td>
</tr>
<tr>
<td>Loess</td>
<td>0.40</td>
<td>7/8</td>
<td>7.7 blows/ft</td>
<td>0.34</td>
<td>0.18</td>
<td>0.53</td>
</tr>
</tbody>
</table>
Ground Improvement Techniques to overcome Liquefaction

Measures to Overcome Liquefaction

1) Vertical Stress Increase (Surcharge)
2) Stone Columns (De-watering)
3) Compaction (Densification of soil)
4) Removal of Liquefiable soil
5) Anchored Piles
6) Liquefaction Resistant Structures
1) Vertical Stress Increase

- Increasing surcharge

Factor of Safety against Liquefaction

\[
CSR = \frac{\tau_{av}}{\sigma_{yo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma'_{vo}}{\sigma_{vo}} r_d
\]

Before Surcharge

\[
\frac{\sigma_{yo}}{\sigma_{yo}} = \frac{50}{40} = 1.25
\]

After Surcharge

\[
\frac{\sigma_{yo}}{\sigma_{yo}} = \frac{50 + 20}{40 + 20} = 1.17
\]

\[
F_s = \frac{CRR}{CSR}
\]
Vertical Stress Increase

- Surcharge

![Graph showing Cyclic Shear Stress vs Depth with areas representing expected zone of liquefaction and stress required for liquefaction.]

2) Stone Columns

- Drainage or De-watering

![Diagram of stone columns with flow arrows indicating drainage or de-watering.]

- Excess Pore Water
- Pressure release

![Stone Columns flow diagram indicating seepage flow.]

- Equivalent stress induced by EQ with surcharge
- Equivalent stress induced by EQ without surcharge
- Stress required for liquefaction
Stone Columns

- Liquefiable Sand
- Drains
- Liquefied

Cyclic Shear Stress

Depth

- Expected Zone of Liquefaction
- Stress required for Liquefaction with stone columns
- Stress required for Liquefaction without stone columns
- Equivalent stress induced by EQ
3) Compaction (Densification of soil)

a. Roller Compaction
b. Dynamic Compaction
c. Vibro-compaction (vibro-floatation)
d. Compaction Grouting
e. Compaction by Pile Driving

3a) Roller Compaction

- Rubber Tyre Roller
- Smooth wheel vibratory roller
3b) Dynamic Compaction

Big weights dropped from 25 m, compacting the ground.

Craters formed in compaction

3c) Vibro-floatation

- Used for cohesionless deposits of sand and gravel with not more than 20% silt or 10% clay
- Craters are formed during vibratory motion. Sand / gravel is added to the crater formed.
Vibro-floatation

Compaction

Cyclic Shear Stress

Expected Zone of Liquefaction

Stress required for Liquefaction with densification

Stress required for Liquefaction without densification

Equivalent stress induced by EQ

Depth

3d) Compaction Grouting

- Stiff, low mobility grout is slowly injected into loose soils under high pressure
- Grout does not enter the soil pores, but forms a bulb that compacts and densifies the soil by forcing it to occupy less space

3e) Compaction by Pile Driving

- When pile is driven in loose sand deposit, compacts the sand within area covered by 8 times around it
- Increase in the stiffness of soil stratum due to pile driving

4) Removal of Liquefiable soil

- Removal of soil layers that can liquefy
5) Anchored Piles

- Piles are anchored in bed rock

6) Liquefaction Resistant Structures

A structure that possesses ductility, has the ability to accommodate large deformations, adjustable supports for correction of differential settlements, and having foundation design that can span soft spots; which can decrease the amount of damage a structure may suffer due to liquefaction.

To achieve these features in a building, following aspects need to be considered:

a. Shallow Foundation Aspects
b. Deep Foundation Aspects
6a) Shallow Foundation Aspects

All foundation elements in a shallow foundation should be tied together to make the foundation move or settle uniformly, thus decreasing the amount of shear forces induced in the structural elements resting upon the foundation. (For example: The well-reinforced perimeter and interior wall footings should be tied together to enable them to bridge over areas of local settlement and provide better resistance against soil movements.)

A stiff foundation mat is a good type of shallow foundation, which can transfer loads from locally liquefied zones to adjacent stronger ground.

Buried utilities, such as sewage and water pipes, should have ductile connections to the structure to accommodate the large movements and settlements that can occur due to liquefaction.

6b) Deep Foundation Aspects

Piles driven through a potentially liquefiable soil layer to a stronger layer not only have to carry vertical loads from the superstructure, but also resist the horizontal loads and bending moments induced lateral movements if that layer liquefies. Sufficient resistance should be achieved by piles of larger dimensions and/or more reinforcement.

Piles should be connected to the cap in a ductile manner which could allow some rotation to occur without a failure of the connection. If the pile connections fail, the cap cannot resist overturning moments from the superstructure by developed vertical loads in piles.
Liquefaction Induced Lateral Spreading

Mechanism (Zhang et al. 2004)
- Lateral spreads occurs due to residual shear strains in liquefied layers.
- Residual shear strains depends on Maximum cyclic shear strains.
- Thicker Liquefied layer $\rightarrow$ Greater Displacement
- Increase in cyclic shear strain $\rightarrow$ Greater Displacement

Soil Properties and Earthquake Characteristics

Cyclic Shear strain and Thickness of Liquefied soil layers


Liquefaction Induced Lateral Spreading

- Mechanism (Zhang et al. 2004) Continued...
  - At a given relative density of soil; A limited amount of shear strain could be developed in soil; regardless of the number of stress cycles applied.
  
  - Maximum Cyclic shear strain can be obtained using:
    - Factor of Safety obtained during Liquefaction Potential Evaluation
    - Relative density of soil for given earthquake loading conditions

Liquefaction Induced Lateral Spreading using SWV

Maximum Cyclic shear strain and Factor of Safety against Liquefaction for different relative densities for Clean Sands

(Based on data from Ishihara and Yoshimine 1992; Seed 1979)
Liquefaction Induced Lateral Spreading using SWV

Relative Density Calculations:

\[ D_r = 14\sqrt{(N_i)_{se}}; (N_i)_{se} \leq 42 \]

\[ D_r = -85 + 76\log(q_{uy}); q_{uy} \leq 200kPa \]

The above equations are for clean sands. *Assumption: It is assumed that the effect of grain characteristics or fines content on lateral spreading is similar to its effect on liquefaction triggering.* (Zhang et al. 2004)

Lateral Displacement Index (LDI):

\[ LDI = \int_0^{z_{\text{max}}} \gamma_{\text{max}} dz \]

\[ z_{\text{max}} = \text{Maximum depth below all the potential liquefiable layers with a } F_s < 2.0 \]

\[ \frac{LD}{LDI} = S + 0.2 \]

\( S = \text{Ground slope in percentage} \)

\( LD = \text{Lateral Displacement} \)

Evaluation of liquefaction at depths greater than 23m is beyond the range where NCEER CPT&SPT methods have been verified and where routine applications should be applied (Youd et al. 2001)
Liquefaction Induced Lateral Spreading using SWV

Lateral displacements for the available case histories for (Zhang et al. 2004):
(a) Gently sloping ground without a free face
(b) Level ground with a free face
(c) Gently sloping ground with a free face

Liquefaction Induced Lateral Spreading using SWV

Shear Wave Velocity (SWV) Method (Fred, 2010):

\[ LDI = \int_{0}^{z_{\text{max}}} \gamma_{\text{max}} \, dz \]

\[ D_{h} = C_{h} (LDI) = 0.23 (LDI) \]

\[ D_{h} = \text{Lateral Displacement} \]
Thank You