Liquefaction Phenomena
Field and Experimental Observations

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Acknowledgements

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Mechanics of Liquefaction

• Propagation of seismic waves through soil layers generates shear deformations within the layer

• Shear deformations cause collapse of loose granular soil structure

• Collapse of granular structure transfers stresses from particle contacts to the pore water

Courtesy of Professor T. L. Youd
Packing changes of particulate group during cyclic loading (after Youd 1977)

A. PARTICULATE GROUP CONTAINING LARGE HOLE
B. SMALL SHEAR STRAIN COLAPSES HOLE
C. LARGE STRAIN CREATES SMALL HOLES (DILATION)
D. STRAIN REVERSAL COLAPSES HOLES
E. LARGE STRAIN CREATES SMALL HOLES (DILATION)
F. AFTER A STRAIN CYCLE, VOLUME DECREASES

Mechanics of Liquefaction

• An increase of pore water pressure reduces intergranular or effective stress
• When the pore water pressure reaches a critical level, liquefaction occurs

Liquefaction

The transformation of a granular material from a solid state to a liquefied state as a consequence of increased pore water pressure and reduced effective stress.

» ASCE Comm. On Soil Dynamics, 1978
Courtesy of Professor T. L. Youd
Consequences of Liquefaction

- Sand boils
- Flow failure
- Lateral spread
- Ground oscillation
- Loss of bearing strength
- Ground settlement

Courtesy of Professor T. L. Youd

Sand boil generated by 1979 Imperial Valley, Calif. earthquake
Courtesy of Professor T. L. Youd

Cross section of sand boil generated by 1981 Westmorland, Calif. earthquake
Courtesy of Professor T. L. Youd
Sand Boil Erupted within House in Caucete during the 1977 San Juan, Argentina Earthquake

• Total volume of sand and water = 11.99 m³
• Volume of sand and silt particles = 2.77 m³
• Volume of water = 9.22 m³
• Ratio of water to solids = 3.33:1

Courtesy of Professor T. L. Youd
Consequences of Liquefaction

- Sand boils
- **Flow failure**
- Lateral spread
- Ground oscillation
- Loss of bearing strength
- Ground settlement

![Flow Failure Diagram](image)

Before earthquake

After earthquake

Courtesy of Professor T. L. Youd
Flow landslide, Half Moon Bay, Calif., 1906 San Francisco Earthquake
Courtesy of Professor T. L. Youd

Crest and Upstream Embankment of Lower San Fernando Dam Slipped Upstream and into Reservoir Due to Liquefaction-Induced Flow Failure During 1971 San Fernando, California Earthquake

Courtesy of Professor T. L. Youd
Crest and Upstream Embankment of Lower San Fernando Dam Slipped Upstream and into Reservoir Due to Liquefaction-Induced Flow Failure During 1971 San Fernando, California Earthquake

Courtesy of Professor T. L. Youd

View of Failed Lower San Fernando Dam after Draining of Reservoir

Courtesy of Professor T. L. Youd
Lower San Fernando Dam Before and After Failure (Harry Seed)

Courtesy of Professor T. L. Youd
Consequences of Liquefaction

- Sand boils
- Flow failure
- **Lateral spread**
- Ground oscillation
- Loss of bearing strength
- Ground settlement

Courtesy of Professor T. L. Youd
Aerial View of San Fernando Juvenile Hall Lateral Spread Area after 1971 San Fernando, Calif. Earthquake

Courtesy of Professor T. L. Youd

Fissures and Ground Displacements Generated by the San Fernando Juvenile Hall Lateral Spread; 1971 San Fernando, Calif. Earthquake

Courtesy of Professor T. L. Youd
San Fernando Valley Juvenile Hall Damaged by Lateral Spread During 1971
San Fernando, Calif. Earthquake

Courtesy of Professor T. L. Youd

San Fernando Valley Juvenile Hall Damaged by Lateral Spread During the 1971 San Fernando, Calif. Earthquake

Courtesy of Professor T. L. Youd

Diagrammatic View of Building Damage Caused by San Fernando Valley Juvenile Hall Lateral Spread

Courtesy of Professor T. L. Youd
Wall around San Fernando Valley Juvenile Hall Was Pulled Apart by Lateral Spread during 1971 Earthquake

Craters and Flooding Due to Pipeline Breaks, San Fernando Juvenile Hall Lateral Spread

Courtesy of Professor T. L. Youd

Water Pipeline Break Caused by Displ. of San Fernando Valley Juvenile Hall Lateral Spread

Courtesy of Professor T. L. Youd

Vectors of Lateral Spread Displacement, 1964 Niigata, Japan Earthquake

Courtesy of Professor T. L. Youd
Bridge Pier Displaced Toward Shinano River During 1964 Niigata, Japan Earthquake

Courtesy of Professor T. L. Youd

Buckled Railroad Bridge Caused by Lateral Spread During the 1964 Alaska Earthquake

Courtesy of Professor T. L. Youd
Measured Lateral Spread Displacement around N Building following the 1964 Niigata, Japan Earthquake

Fractured Piles Beneath Building Caused by Lateral Spread (1964 Niigata, Japan Earthquake)
Diagrams of Post Earthquake Pile Configuration and Standard Penetration Resistance versus Depth in Sandy Soil

Lateral Spread Pervasively Displaced Quay Walls Seaward Around perimeters of Port and Rokko Islands Decimating Port Facilities

Courtesy of Professor T. L. Youd
Consequences of Liquefaction

- Sand boils
- Flow failure
- Lateral spread
- **Ground oscillation**
- Loss of bearing strength
- Ground settlement
GROUND OSCILLATION

Before earthquake

After earthquake

Courtesy of Professor T. L. Youd

Walk and Curb Damage Caused by Ground Oscillation (1989 Loma Prieta, Calif. Earthquake)

Courtesy of Professor T. L. Youd
Consequences of Liquefaction

- Sand boils
- Flow failure
- Lateral spread
- Ground oscillation
- Loss of bearing strength
- Ground settlement

Pavement and Curb Damage Caused by Ground Oscillation during 1989 Loma Prieta, Calif. Earthquake

Courtesy of Professor T. L. Youd
LOSS OF BEARING STRENGTH

Before earthquake

After earthquake

Courtesy of Professor T. L. Youd

Tipped Buildings Caused by Liquefaction-Induced Loss of Bearing Strength, 1964 Niigata, Japan Earthquake

Courtesy of Professor T. L. Youd
Tipped Buildings Caused by Liquefaction-Induced Loss of Bearing Strength, 1964 Niigata, Japan Earthquake
Courtesy of Professor T. L. Youd

Tipped Building in Adapazari, Turkey Caused by Liquefaction-Induced Loss of Bearing Strength during 1999 Koaceli, Turkey Earthquake

Courtesy of Professor T. L. Youd

Building Settlement in Adapazari, Turkey Caused by Liquefaction-Induced Loss of Bearing Strength during 1999 Koaceli, Turkey Earthquake

Courtesy of Professor T. L. Youd
Oil Tank that Buoyantly Floated to Ground Surface through Liquefied Soil during 1983 Hokkaido-Nansei-Oki, Japan Earthquake

Consequences of Liquefaction

• Sand boils
• Flow failure
• Lateral spread
• Ground oscillation
• Loss of bearing strength
• Ground settlement
Water and Sand (brown areas) from Sand Boils that Erupted on Rokko Island during 1995 Kobe, Japan Earthquake; the Ground Surface also Subsided 0.5 m to 0.7m

Courtesy of Professor T. L. Youd

Differential Settlement Between Column on Piles and Surrounding Ground on Rokko Island; Settlement due to Liquefaction and Compaction of 12 m of Artificial Fill during 1995 Kobe, Japan Earthquake

Courtesy of Professor T. L. Youd
Cross Section Showing Soil Profile and Typical Pile Foundation Configuration for Buildings on Port and Rokko Islands that Were Shaken by the 1995 Kobe, Japan Earthquake

Courtesy of Professor T. L. Youd

Differential Settlement Between Building on Piles and Natural Ground Caused by Liquefaction and Compaction of Artificial Fill during the 1995 Kobe, Japan Earthquake

Courtesy of Professor T. L. Youd
Localities in Idaho Where Liquefaction Occurred During the 1983 Borah Peak Earthquake

Courtesy of Professor T. L. Youd

Whiskey Springs Lateral Spread Generated by the 1983 Borah Peak Earthquake Displaced Highway 93 1 m toward Camera

Courtesy of Professor T. L. Youd
Segment of Highway 93 Displaced 1 m to Left by Whiskey Springs Lateral Spread; 1983 Borah Peak, Idaho Earthquake
Courtesy of Professor T. L. Youd

Buckled Sod at Toe of Whiskey Springs Lateral Spread
Courtesy of Professor T. L. Youd
Buckled Sod at Toe of Whiskey Springs Lateral Spread

Courtesy of Professor T. L. Youd

Sand Boil Deposit and House on Pence Ranch Affected by Liquefaction During 1983 Borah Peak, Idaho Earthquake

Courtesy of Professor T. L. Youd
View of Back of House on Pence Ranch Showing Wall Pulled Away from Foundation because of Lateral Spread; 1983 Borah Peak, Idaho Earthquake

Courtesy of Professor T. L. Youd

Water Tank on Right Buoyantly Rose Due Liquefaction of Subsurface Soils at Pence Ranch; 1983 Borah Peak Earthquake

Courtesy of Professor T. L. Youd
Fence on Pence Ranch Pulled Apart by 0.45 m Due to Lateral Spread During 1983 Borah Peak, Idaho Earthquake

Gravel Sample Taken from Layer that Liquefied beneath Pence Ranch

Courtesy of Professor T. L. Youd
Sand Boil That Erupted In Pahsimeroi Valley During 1983 Borah Peak Earthquake

Courtesy of Professor T. L. Youd
Site Liquefaction

Stress-Strain Response
Stress-Strain Models
Site Response
Lateral Deformation

Nonlinear soil response
(Shear stress $\tau$ and shear strain $\gamma$)
The above nonlinear shear stress-strain relationship is sometimes called the *backbone* shear stress-strain relationship.

Among the typical equations used to represent this backbone behavior is the *Hyperbolic relationship*

\[ \tau = G \gamma / (1 + \gamma / \gamma_r) \]

where \( G \) = Low strain shear modulus \((G_{max})\)

and \( \gamma_r \) is a constant that is used to match the observed level of nonlinear response.

This relationship reaches a maximum shear stress \( \tau_{max} \) of \( G \gamma_r \) at infinite shear strain. As such, it is common to cap this relationship at a value of \( R \tau_{max} \) where \( R \) is generally in the range of 0.8 or higher.

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Cyclic Stress-strain response

*Hysteresis response is commonly observed, with *Masing-type* behavior often adopted to reproduce hysteretic damping. This damping mechanism is strain-level dependent and frequency-independent, both being desirable features that mimic data from experimentation.*
Confinement dependence

Shear stiffness and strength may or may not be significantly dependent on confinement.

If behavior is not confinement dependent, a cohesion intercept ($c$) describes shear strength, and

$$R_{\tau_{\text{max}}}$$ is then equated to this value of cohesion ($c$)

If behavior is confinement dependent, then the shear strength

$$R_{\tau_{\text{max}}} = c + p' \sin \phi \text{ where } \phi \text{ is the friction angle and } p' \text{ is confinement described by (for level ground scenarios)}$$

$$p' = (\sigma'_v + \sigma'_h)/2$$

where $\sigma'_v$ is vertical effective stress and $\sigma'_h$ is horizontal effective stress ($\sigma'_v = \sigma_v - u$, with $u =$ hydrostatic water pressure)

Shear-volume strain coupling (for confinement dependent soils with $c=0$)

During small-strain cyclic loading, upon shearing a loose (high void ratio) soil, volume gradually decreases.

If the soil is saturated with water, and the rate of loading is rapid (i.e., preventing water from exiting the soil skeleton), this tendency for volume decrease translates into the soil particles partially floating in the water, which then carries this additional soil granules weight (in the limit, this is termed “undrained behavior”).

The fraction of soil self weight carried by the water becomes “excess pore-water pressure” known as $u_e$ gradually reducing the effective confinement $p'$ (and therefore the shear strength). If $u_e$ reaches $\sigma'_v$, no effective confinement remains and the soil is “Liquefied” or reaches “Liquefaction” (at which point, $r_u = 1$, where $r_u$ is known as the excess pore-pressure ratio $= u_e / \sigma'_v$).
Shear-volume strain coupling (for confinement dependent soils with c=0), .. continued

If the applied cyclic strains are large enough, then the cyclic volume decrease at lower strains turns into a volume increase at larger strains. For the undrained scenario, this becomes a tendency for volume decrease ($u_e$ buildup) followed by a tendency for volume increase (which momentarily reduces $u_e$). Examples of this behavior are presented below.

Note: If the rate of loading is slow enough to permit some level of water exiting the soil skeleton, this allows excess pore-pressure to dissipate and the pore-pressure goes back towards the original hydrostatic value, thus allowing confinement to stay close to its original value. This will tend to occur for higher permeability sands/gravels that are capable of fast/very-fast excess pore-pressure dissipation.
Cyclic Torsional Tests (after Ishihara 1985)
Medium Fuji river sand (Dr=47%)

Cyclic Torsional Tests (after Ishihara 1985)
Dense Fuji river sand (Dr=75%)
Back-calculated soil response at Wildlife site during 1987 Superstition Hills earthquake (Zeghal and Elgamal 1994)

Shear stress - shear strain

Effective stress path

Mildly sloping ground and accumulation of lateral deformations
Elements of liquefaction stress-strain model

Parra 1996, Yang 2000

Stage:
0 - 1: Contractive phase
1 - 2: Perfectly plastic phase
2 - 3: Dilative phase
3 - 4: Unloading phase
4 - 5: Contractive phase (opposite)
5 - 6: Perfectly plastic phase (opposite)
6 - 9: Logic of 0 - 3

Shear Stress-Strain

Simulation of a stress-controlled cyclic simple shear test

Parra 1996
Simulation of a strain-controlled cyclic simple shear test

Parra 1996

Simulation of a stress-controlled cyclic triaxial test with an imposed static driving shear stress

Parra 1996
Model response under cyclic loading for different soil types

![Graphs showing model response under cyclic loading for different soil types: Medium, Medium Dense, Dense, Yang (2000)]

Model response under cyclic loading with a driving shear stress imposed for different soil types

![Graphs showing model response under cyclic loading with a driving shear stress imposed for different soil types: Dense, Medium-dense, Medium, Yang (2000)]
Suggested Cyclic Strain Levels During Liquefaction

The above soil constitutive model is incorporated in a solid-fluid fully coupled Finite Element program: CYCLIC

http://cyclic.ucsd.edu
Fluid Saturated Porous Media
Simplified u-p Formulation (Chan 1988)

**Assumptions**

- Soil is fully saturated.
- Constant fluid density with respect to space.
- Constant porosity with respect to time.
- Fluid is compressible and solid grains are incompressible.
- Fluid velocity gradient is small and all convective terms are negligible.
- Fluid acceleration relative to solid phase is negligible.
- Soil is considered a continuum.
- Isothermal process.


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**Notation**

- $u_i$ = Displacement of solid phase
- $p$ = Pore fluid pressure
- $w_i$ = Displacement of the fluid relative to solid phase
- $\rho$ = Mass density of the mixture
- $\rho_f$ = Mass density of the fluid
- $g_i$ = Acceleration of gravity
- $Q$ = Bulk modulus of the mixture
- $R_i$ = Viscous drag force exerted on the fluid by the solid
- $\kappa_{ij}$ = Permeability tensor
Fluid equilibrium and mass conservation

\[
\frac{\dot{p}}{Q} + \dot{\varepsilon}_{ii} - k_{ij}(p_i + \rho_f\ddot{u}_i - \rho_f g_i)_j = 0
\]

Mixture equilibrium

\[
\sigma_{ij,j} - \rho(\ddot{u}_i - g_i) = 0
\]

Chan (1988)

Above two equations constitute a strong form of simplified u-p formulation.

Finite element implementation

\[
M \ddot{u} + \int_{\Omega} B^T \sigma' d\Omega - Qp - f^m = 0
\]

\[
Q^T \ddot{u} + Hp + Sp - f^p = 0
\]

where

- \( u \) = displacement vector
- \( p \) = pore pressure vector
- \( M \) = mass matrix
- \( B \) = strain-displacement matrix
- \( \sigma' \) = effective stress vector
- \( Q \) = discrete gradient operator
- \( H \) = permeability matrix
- \( S \) = compressibility matrix
- \( f^m \) = force vector for the mixture
- \( f^p \) = force vector for the fluid phase

Typical 9-4-node element employed in

![Diagram of 9-4-node element with solid and fluid nodes]


CYCLIC simulation: effect of permeability gradient

- **Without clay cap**: Yang (2000)
  - Dense sand
  - Medium sand

- **With clay cap**
  - Clay
  - Medium sand
  - Dense sand
Hydraulic fill liquefaction (Adalier and Elgamal 1992)

Example Cyclic 1D Simulations

- 10 m soil profile height.
- 10 elements.
- Water table at ground surface.
- Rigid base.
- Inclination and material definition see the table:

<table>
<thead>
<tr>
<th>box</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
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<tr>
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<td>Material</td>
<td>Cohesionless medium</td>
<td>Cohesionless medium</td>
<td>Cohesionless medium, with clay cap</td>
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<tr>
<td></td>
<td>Permeability</td>
<td>sand</td>
<td>sand</td>
<td>gravel</td>
</tr>
<tr>
<td></td>
<td>Inclination</td>
<td>level</td>
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<td>4°</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td>3</td>
<td>4°</td>
</tr>
</tbody>
</table>
Example Cyclic 1D Simulations

Input motion is composed of 10 cycles of sinusoidal motion at a frequency of 1 Hz and amplitude of 0.2 g.

Cohesionless (Dr = medium, sand permeability, level)

- Horizontal Displacement (Relative to the base, m)
- Horizontal Acceleration (m/s/s)
- Excess Pore Pressure (kPa)

Cohesionless medium, sand permeability, 4° inclination
Cohesionless medium, sand permeability, level

Shear strain

Shear stress (kPa)

Effective confinement (kPa)

Cohesionless medium, sand permeability, 4° inclination

Surface Horizontal Displacement (Relative to the base, m)

Base Excess Pore Pressure (kPa)

Cohesionless medium, sand permeability, 4° inclination
Cohesionless medium, sand permeability, level

Shear stress-strain near base

Effective stress path near base

Cohesionless medium, sand permeability, 4° inclination

Surface horizontal acceleration (m/s/s)

Acceleration response spectrum (5% damping)

Acceleration Fourier transform amplitude

Cohesionless medium, gravel permeability, 4° inclination, with a clay cap

Shear strain

Shear stress (kPa)

Effective confinement (kPa)
Cohesionless medium, gravel permeability, 4° inclination

Surface horizontal acceleration (m/s²)

Acceleration response spectrum (5% damping)

Acceleration Fourier transform amplitude

Cohesionless medium, gravel permeability, 4° inclination, with a clay cap

Insightful simulation scenarios using Cyclic1D

Using the computer code Cyclic1D http://cyclic.ucsd.edu, or http://www.soilquake.net:

a) Run the default case (10 m saturated cohesionless medium, sand permeability soil, and 0.2g 1Hz base sinusoidal acceleration for 10 cycles of loading). Inspect the results and on this basis, discuss the observed liquefaction mechanisms (generation of excess pore pressure, stress-strain histories at different depths, changes in effective vertical stress versus shear stress, and the resulting form of acceleration at and near ground surface).

b) Repeat the above upon changing to soil to the cohesionless medium, gravel permeability soil. Pay particular attention to the main changes that occurred on account of the now higher soil permeability (gravel permeability versus sand permeability). Note also the changes that occur after the end of base excitation (computations continue for 10 more seconds after the base shaking ends).

c) Repeat the above upon changing to soil to the cohesionless dense, sand permeability soil. Pay particular attention to the main changes that occurred on account of the now dense soil characteristics of stress-strain response (e.g., lower tendency for excess pore-pressure $u_e$ buildup and so forth).
Insightful simulation scenarios using Cyclic1D (continued)

d) Repeat case a) above, upon changing the site inclination angle to 1.5 degrees (i.e., mild site inclination, imposing a small driving shear stress). Pay particular attention to the main changes that occurred on account of the now imposed driving shear stress. Discuss change in relative ground surface displacement and the displacement profile, compared to the corresponding zero inclination scenario of case a). Note and discuss the changes in shear stress-strain, excess pore-pressure histories, and shear stress versus effective confinement.

Important note: If you get the message below, it might be on account of selecting and inclination angle that results in excessive lateral deformations (upon liquefaction), precluding/hampering the possibility of convergence of the analysis (at some particular time step during the computations). Simply, the available shear strength is inadequate to sustain the inclination-imposed driving shear force (upon liquefaction and degradation of soil strength). A similar outcome would also result from imposing a high inclination when a very weak soil layer (low inadequate shear strength) is specified (in this case convergence would not be possible right from the start).

For Liquefaction-induced lateral-spreading countermeasures, see: [http://cyclic.ucsd.edu/openseespl](http://cyclic.ucsd.edu/openseespl)

References


Additional References


Liquefaction Evaluation

Ahmed Elgamal & Zhaohui Yang

Acknowledgements
The Liquefaction Evaluation section is prepared mainly following:

• Martin, G. R., and Lew, M., eds. (1999). Recommended procedures for implementation of DMG Special Publication 117: Guidelines for analyzing and mitigating liquefaction hazards in California, Southern, California Earthquake Center, University of Southern California, Los Angeles, California.
Updates and new additional information can be found in:


Other Main References:


Types of liquefaction

1. Flow liquefaction

- Occurs when shear stress required for equilibrium of a soil mass (the static shear stress) is greater than the shear strength (residual strength) of the soil in its liquefied state.

- Potentially very large post-liquefaction lateral deformations are driven by the static shear stress.
Types of liquefaction (cont'd)

2. Cyclic mobility

• Occurs when the static shear stress is less than the shear strength of the liquefied soil.

• Deformations are driven by both cyclic and static shear stresses.

• Deformations develop incrementally during earthquake shaking.

When is the soil liquefied ....

At a given site, typically manifestations include sand boils, large lateral deformation, and significant settlement.

For technical assessments, the “liquefaction” state is reached when the effective confining stress goes down to zero (i.e., the original effective confining stress has gradually decreased and has been become “excess pore-water pressure” known as $u_e$).

At this state, the value of the “excess pore pressure ratio” $r_u$ is 1.0 where $r_u = \frac{u_e}{\sigma'_v}$ and $\sigma'_v$ is the initial effective vertical stress.

Also, technically liquefaction may be described by a soil sample building up pore-pressure and reaching a shear strain of 3%-5% or more in a laboratory shear test.
Why does liquefaction occur
If the soil is loose and is being shaken, the particles will settle due to gravity. When the soil is saturated, the pore-water is unable to move of the way quickly enough (because the soil permeability is relatively low), and more and more particles start to partially float in the water (this leads to the excess pore-pressure buildup). Eventually as shaking continues, the particles float in the water temporarily as they settle downwards and reach a new densified and consolidated state.

Soils Susceptible to liquefaction
Most susceptible would be very loose cohesionless soils. The low permeability of non-plastic silts and sands is a disadvantage.

Higher permeability, higher relative density, and higher cohesion (plasticity) reduce the susceptibility.

Notes:

1) Objectionable deformations might still occur if $r_u$ values are high, even if liquefaction does not occur. Looser soils are more vulnerable.

2) As pore pressure builds-up, stratified soil profiles (particularly with permeability contrasts) may cause water to be temporarily trapped under a relatively impervious layer or seam (e.g., a due to alluvial or hydraulic fill construction, or presence of an upper clay stratum), generating a low friction interface and possibly leading to major lateral deformations. This mechanism actually is a driver of what we commonly observe as sand boils where this water escapes upwards through any available high permeability locale (e.g., taking advantage of a crack in the ground, or similar imperfections, …).
Evaluation of Liquefaction Potential and Consequences

I. Is the soil susceptible to liquefaction?

II. If the soil is susceptible, will liquefaction be triggered?

1) Cyclic stress approach (Discussed in notes)

2) Other methods (Refs. on page 2): effective-stress response analysis approach, cyclic strain approach, energy dissipation approach, probabilistic approach.

III. If liquefaction is triggered, how much damage would occur?

- Settlements
- Lateral deformations due to cyclic mobility: a) empirical approach, and b) effective-stress response analysis approach
- Flow Failure (see Kramer 1996).

From Kramer (1996)

I. Is the soil susceptible to liquefaction?

1. Historical criteria

The epicentral distance to which liquefaction can be expected, increases with increasing earthquake magnitude.

From Kramer (1996)
I. Is the soil susceptible to liquefaction? (cont’d)

2. Geologic criteria

- Depositional environment - Saturated loose fluvial, colluvial, and aeolian deposits are more susceptible to liquefaction.
- Age - Newer soils are more susceptible to liquefaction than older soils.
- Water table - Liquefaction susceptibility decreases with increasing groundwater depth.
- Human-made soils - Uncompacted soils (e.g., hydraulic fill) are more susceptible to liquefaction than compacted soils.

3. Compositional criteria

- Grain size and plasticity characteristics - Sands, nonplastic silts, and gravelly soils when surrounded by impermeable soils, are susceptible to liquefaction.
- Gradation - Well graded soils are less susceptible to liquefaction than poorly graded soils.
- Particle shape - Soils with rounded particles are more susceptible to liquefaction than soils with angular particles.
I. Is the soil susceptible to liquefaction? (cont’d)

4. Initial stress state criteria (for flow liquefaction)

- A loose soil will be susceptible to flow liquefaction only if the static shear stress exceeds its steady state (or residual) strength.

- Residual strength can be estimated as shown in Figure 2.

In Fig. 2 above, \((N_1)_{60-cs} = (N_1)_{60} + N_{corr}\)

where \(N_{corr}\) may be obtained from the table below. \((N_1)_{60}\) is the number of SPT blow count normalized to an overburden pressure of 1 ton/ft\(^2\) (96 kPa) and corrected to an energy ratio of 60%.

Note: \((N_1)_{60} = C_N \cdot N_{60}\) (see below)

\(N_{60} = N \cdot C_{60}\) (see next page)

Comment: All recommendations related to “fines” continue to be likely to change in the near future..
II. If the soil is susceptible, will liquefaction be triggered?  
(by cyclic stress approach)

Step 1. Calculate equivalent cyclic shear stress induced by a given earthquake (i.e., the “Demand”). Herein, this is dictated by an expected peak acceleration at the site scaled by a factor of 0.65 based on engineering judgment.

\[ \tau_{\text{cyc}} = 0.65 \frac{a_{\text{max}}}{g} \sigma_v r_d = CSR \sigma'_{v0} \]  

(1)

where \( a_{\text{max}} \) is the peak ground surface acceleration, \( g \) the acceleration of gravity, \( \sigma_v \) the total vertical stress, and \( r_d \) the value of a stress reduction factor at the depth of interest. \( r_d \) may be obtained from Figure 3 below. This equation also defines \( CSR \), the cyclic stress ratio, with \( \sigma'_{v0} \) being the initial vertical effective stress.
II. If the soil is susceptible, will liquefaction be triggered? (Cont’d)

by cyclic stress approach

Step 2. Calculate the cyclic shear stress required to cause liquefaction (i.e., the “capacity”):

\[ \tau_{cyc,L} = CSR_L \sigma'_v \]  \( (2) \)

where \( \sigma'_v \) is the initial vertical effective stress, \( CSR_L \) is the cyclic stress ratio, and may be obtained based on:

- SPT resistance (Fig. 4 for clean sands, Fig. 5 for silty sands).
- CPT resistance (Fig. 8).
- See also references for Shear wave velocity (Andrus and Stokoe 2000) and Arias Intensity (Kayen and Mitchell 1997) based techniques.
Note:

1. Use the following table for earthquake magnitudes other than $M=7.5$

<table>
<thead>
<tr>
<th>Magnitude, $M$</th>
<th>$CSR_M/CSR_{M=7.5}$</th>
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<tbody>
<tr>
<td>5$\frac{1}{2}$</td>
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<tr>
<td>6</td>
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<td>7$\frac{1}{2}$</td>
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<tr>
<td>8$\frac{1}{2}$</td>
<td>0.89</td>
</tr>
</tbody>
</table>

2. The influence of plasticity could be accounted for by multiplying the $CSR_L$ by the factor (Ishihara 1993):

$$ F = \begin{cases} 
1.0 & \text{PI} \leq 10 \\
1.0 + 0.022(\text{PI} - 10) & \text{PI} > 10 
\end{cases} $$
3. Figs. 4 and 5 are mainly for level-ground sites, and shallow liquefaction. To account for site slope (initial shear stress) and deep liquefaction, modify the $CSR_L$ by:

$$CSR_{\alpha,\sigma} = CSR_L K_{\alpha} K_{\sigma}$$

(3)

where $\alpha = \tau_{h,\text{static}} / \sigma_{v0}'$ and $K_{\alpha}$ and $K_{\sigma}$ are correction factors that may be obtained from Figs. 6 and 7 below.

(Figure from Kramer 1996)

Figure 6. Variation of correction factor, $K_m$, with initial shear/normal stress ratio. (After Seed and Harder, 1990. H. Bolton Seed Memorial Symposium Proceedings, Vol. 2, p. 364. Used by permission of BiTech Publishers, Ltd.)

The data in this figure is not accepted fully by all experts
**Figure 7.a** Variation of correction factor, $K_{ct}$, with effective overburden pressure. (After Marcussen et al., 1990. Used by permission of EERL.)

$K_{ct}$ vs. Effective overburden pressure (kPa)

- **Legend**
  - Fairmont Dam
  - Lake Arrowhead Dam
  - Upper San Leandro Dam
  - Lower San Fernando Dam Shell
  - Upper San Fernando Dam Shell
  - Los Angeles Dam Shell
  - Perris Dam Shell, RC = 90, 95, 100%
  - Sardis Dam Shell
  - Sardis Dam Foundation
  - Thermalito Afton Dam Foundation
  - Thermalito Forebay Dam Foundation
  - Antelope Dam Impervious Material
  - Aswan Dam Dune Sand
  - Sacramento River Sand, Dr = 38, 60, 78, 100%
  - Monterey 0 Sand, Dr = 40, 60%
  - Reid Bedford Sand, Dr = 40, 60%
  - New Jersey Batchfill, FPI, RC = 95%

$K_{ct}$ vs. Effective overburden pressure (kPa)

- **Folsom foundation gravel** ($D_t = 40\%$)
- **Folsom embankment gravel** ($D_t = 60\%$)

---

**Fig. 7b**: Recommended for practice by Youd et al. 2001
Note:

1. In Fig. 8, $q_{c1}$ is the tip resistance $q_c$ normalized to a standard effective overburden pressure $p_a$ of 1 ton/ft$^2$ (96 kPa) by:

$$ q_{c1} = q_c \left( \frac{p_a}{\sigma'_{v0}} \right)^{0.5} \quad \text{or} \quad q_{c1} = \frac{1.8}{0.8 + \sigma'_{v0}} q_c $$

Where $\sigma'_{v0}$ is the initial effective overburden pressure.

2. The effects of fines can be accounted for by adding tip resistance increments to the measured tip resistance $q_c$ (Ishihara 1993):

<table>
<thead>
<tr>
<th>Fines Content (%)</th>
<th>Tip Resistance Increment (tons/ft$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 5$</td>
<td>0</td>
</tr>
<tr>
<td>$\sim 10$</td>
<td>12</td>
</tr>
<tr>
<td>$\sim 15$</td>
<td>22</td>
</tr>
<tr>
<td>$\sim 35$</td>
<td>40</td>
</tr>
</tbody>
</table>

3. Use Table 2 for earthquake magnitudes other than $M=7.5$
II. If the soil is susceptible, will liquefaction be triggered? (cont’d)

by cyclic stress approach

Step 3. Calculate the safety factor against liquefaction.

\[ FS_L = \frac{\tau_{cyc,L}}{\tau_{cyc}} = \frac{CSR_L}{CSR} = \frac{CRR}{CSR} \]  \hspace{1cm} (4)

Liquefaction may be triggered if \( FS_L < 1 \).

Note: CRR above is Cyclic Resistance Ratio

Note:
To more accurately represent the earthquake shaking energy, Youd et al. (2001) suggested including a Magnitude Scaling Factor of the form

\[ MSF = \left(\frac{7.5}{M_w}\right)^n \]

where \( M_w \) is Moment magnitude, and \( n = 2.56 \) for \( M_w = 7.5 \) or greater, and up to 3.3 for \( M_w \) less than 7.5

As such, \( a_{maxM7.5} = a_{max} / MSF \)

and

\[ \tau_{cycM7.5} = 0.65 \frac{a_{maxM7.5}}{g} \sigma v r_d = CSR_{M7.5} \sigma' \]

With this adjustment, both \( CSR \) and \( CSR_L \) can be compared directly for \( M=7.5 \)
If liquefaction is triggered, how much Settlement will occur?

III. If liquefaction is triggered, how much damage would occur?

Settlement by Tokimatsu-Seed method

To use Fig. 9.53, the CSR can be calculated from Equation (1). For earthquake magnitudes other than 7.5, the CSR should be modified according to the Table above.
III. If liquefaction is triggered, how much damage would occur?

*Settlement by Ishihara-Yoshimine method*

To use Fig. 9.54, the $FS_L$ can be calculated using Equation (4).

Note $N_1 = 0.833(N_1)_{60}$

![Figure 9.54 Chart for estimating postliquefaction volumetric strain of clean sand as function of factor of safety against liquefaction or maximum shear strain. (After Ishihara and Yoshimine, 1992; used by permission of JSSMFE.)](image)

Residual Strength (see Fig. 2). In addition, for the residual shear strength $S_r$, Olson and Stark (2002) proposed:

$$S_r/\sigma'_v = 0.03 + 0.0075 (N_1)_{60} \quad \text{plus or minus 0.03}$$

for $(N_1)_{60}$ less or equal to 12

and

$$S_r/\sigma'_v = 0.03 + 0.0143 (q_{cl}) \quad \text{plus or minus 0.03}$$

for $q_{cl}$ less than or equal to 6.5 Mpa

Earlier, Baziar and Dobry (1995) proposed for loose silty sands:

$$S_r = 0.12 - 0.19 (\sigma'_v)$$
Summary of SPT-Based Empirical Method
NCEER/NSF Proceedings (Youd et al., 2001)

Step 1 – Discretize boring log into a series of soil layers;
Step 2 – For each soil layer, compute the vertical total stress ($\sigma_v$) and vertical effective stresses ($\sigma'_v$);
Step 3 – Determine Moment Magnitude and Peak Ground Acceleration ($a_{\text{max}}$) for project site;
Step 4 – Compute the Stress reduction coefficient, $r_d$;
Step 5 – Compute the Cyclic Stress Ratio, CSR;
Step 6 – Compute $(N_1)_{60}$ the SPT blow count normalized to overburden pressure of 100 kPa (1 ton/sq ft) and hammer energy ratio or hammer efficiency of 60%;
Step 7 – Adjust $(N_1)_{60}$ to account for fines content (FC) by calculating the equivalent clean sand value, $(N_1)_{60\text{CS}}$;
Step 8 – Calculate the Cyclic Resistance Ratio for Magnitude 7.5 earthquake, $\text{CRR}_{7.5}$;
Step 9 – Calculate the Magnitude Scaling Factor, MSF;
Step 10 – Calculate the Factor of Safety (FS) against liquefaction; and
Step 11 – Calculate the volumetric strain / settlement within each liquefied layer.

See Idriss and Boulanger (2008) EERI Monograph for Additional details

SPT-Based Empirical Method – Idriss & Boulanger, 2008

Step 1 – Discretize boring log into a series of soil layers;
Step 2 – For each soil layer, compute the vertical total stress ($\sigma_v$) and vertical effective stresses ($\sigma'_v$);
Step 3 – Determine Moment Magnitude and Peak Ground Acceleration ($a_{\text{max}}$) for project site;
Step 4 – Determine the shear stress reduction coefficient, $r_d$;
Step 5 – Compute the Cyclic Stress Ratio, CSR;
Step 6 – Compute $(N_1)_{60}$ the SPT blow count normalized to overburden pressure of 100 kPa (1 ton/sq ft) and hammer energy ratio or hammer efficiency of 60%;
Step 7 – Adjust $(N_1)_{60}$ to account for fines content (FC) by calculating the equivalent clean sand value, $(N_1)_{60\text{CS}}$;
Step 8 – Calculate the Cyclic Resistance Ratio for Magnitude 7.5 earthquake, $\text{CRR}_{7.5}$;
Step 9 – Calculate the Magnitude Scaling Factor, MSF;
Step 10 – Adjust the Cyclic Resistance Ratio for actual earthquake magnitude and overburden stress ($CRR_{M,\sigma_{\text{uc}}}$);
Step 11 – Calculate the Factor of Safety (FS) against liquefaction; and
Step 12 – Calculate the volumetric strain / settlement within each liquefied layer.
Soil Dynamics Short Course

This presentation consists of two parts:
Section 1
Liquefaction of fine grained soils and cyclic softening in silts and clays

Section 2
Empirical relationship for prediction of Lateral Spreading

Liquefaction of fine grained soils and cyclic softening in silts and clays

Main References


**Notation**

- $w_c$ = Water content = (weight of water / weight of soil) %
- $LL$ = Liquid Limit = $w_c$ at which soil starts acting like a liquid
- $PL$ = Plastic Limit = $w_c$ at which the soil starts to exhibit plastic behavior
- $PI$ = Plasticity Index = $LL - PL$ = range of $w_c$ when soil exhibits plasticity
- $e$ = void ratio = volume of voids / volume of solids
- $S_u$ = Undrained shear strength
- $OCR$ = Overconsolidation Ratio

**Notes:**

1. Low $PI$ implies low or lack of significant cohesion
2. High $PI$ implies presence of high cohesion
3. Higher $e$ implies looser soil samples with lower shear resistance, more susceptibility to liquefaction, and higher potential for post-liquefaction settlement (permanent volumetric strain). For a given soil, these effects are judged more precisely by the Relative Density $D_r = (e_{max} - e) / (e_{max} - e_{min})$ %

**Highlights**

Based on post-earthquake reconnaissance and related soil-testing and analysis:

The “Chinese Criteria” about liquefaction resistance of fine grained soils is not correct. It is based on % clay content with no regard to its plasticity (PI) which makes all the difference.

If relatively non-plastic, saturated fine grained soils can build-up significant excess pore water pressure and liquefy.

Cyclic loading of soft clays degrades strength and softens the shear resistance potentially leading to large objectionable deformations.

Sand-type excess pore-pressure build-up likely for scenarios of $w_c / LL > 0.85$ and $PI <$ or equal 12; being relatively non-plastic soils (some suggest $PI <$ or equal 7) … These soils exhibit a cyclic mobility-type response …

Clay-type softening behavior likely for soil with $w_c / LL > 0.8$ and $18 > PI > 12$ (some suggest $PI > 7$) … gradual reduction in shear stiffness and strength …

For $PI > 18$ soils tested at low confining pressure, potential for loss of shear resistance was minimal, but significant deformation is possible under strong shaking conditions.

Bray and Sancio suggest $PI$ rather than % fines to account for higher Liquefaction resistance

A procedure similar to the Liquefaction Cyclic Stress Approach (described earlier) has been developed for cyclic clay softening scenarios (Boulanger + Idriss).
Fig. 2. Data presented by Wang (1979) which led to the development of the Chinese Criteria. Figure from Bray and Sancio (2006) showing Chinese data left of the A-line indicating relatively high plasticity (a key issue that was overlooked when the Chinese Criteria was formulated). Note: CL = Clays of Low Plasticity, CH = Clays of High Plasticity, ML = Silts of Low Plasticity, CH = Silts of High Plasticity.

Seed and Idriss (1982) stated that clayey soils could be susceptible to liquefaction only if all three of the following conditions are met: 1) percent of particles less than 0.005 mm < 15%, 2) LL < 35, and 3) \( \frac{w_c}{LL} > 0.9 \). Due to its origin, this standard is known as the “Chinese criteria.”

Observed cyclic mobility

Fig. 5. Results of a slow cyclic triaxial test (loading frequency of 0.005 Hz) on Specimen F7-P3A (ML, PI=0, \( \varepsilon=0.76 \)): (a) deviator stress versus number of load cycles; (b) excess pore water pressure versus number of load cycles; (c) axial strain versus number of load cycles; (d) deviator stress versus axial strain; and (e) deviator stress versus mean effective confining stress.

Ref.: Bray and Sancio (2006)
Influence of PI on observed deformation

**Fig. 6.** Stress-strain relationship for the first cycle of loading (fine line) and the cycle at which ~3% axial strain is reached (thick line) for four specimens of increasing plasticity. The tests were performed on soils initially isotropically consolidated to an effective stress of 50 kPa.

Ref.: Bray and Sancio (2006)


Observed cyclic mobility response in fine grained soils

**Observed cyclic mobility response in fine grained soils**

**Fig. 9.** Results of a cyclic simple shear test (f = 1 Hz) on Specimen C4-P3 (ML, PI=0, e=0.83): (a) shear stress versus number of load cycles; (b) lateral effective stress versus number of load cycles; (c) shear strain versus number of load cycles; (d) shear stress versus mean effective stress; and (e) shear stress versus shear strain.

Ref.: Bray and Sancio (2006)

Cyclic reduction of shear stiffness and strength in Saturated clay

Fig. 4. Stress-strain response and effective stress paths for Cloverdale clay during undrained slow cyclic loading (adapted from Zergoun and Vaid 1994, used with permission)

Ref.: Boulanger and Idriss (2007)
Slope shear stress impacts NC clay (OCR = 1) as acting stress nears shear strength (minimal impact on highly OC clays).

See Boulanger and Idriss (2007) for full details.
**Additional Related References (Fine Grained Soils)**


Wang, W. (1979). Some findings in soil liquefaction, Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China. (Chinese Criteria was derived based on the data in this ref.)


Lateral Spreading: Empirical Approach

Primary References


Additional Reference


Empirical MLR Procedure

1) Large Case History Data Set

2) Multi-Linear Regression Analysis (MLR)

New predictive equation is based on additional new data sets from US and Japan, and some corrections and modifications
Displacement Versus Distance From Free Face for Lateral Spread Displacements Generated in Niigata, Japan During 1994 Earthquake

Courtesy of Professor T. L. Youd
DH = horizontal displacement (m),
M = moment magnitude,
R = distance from seismic energy source (km),
W = free face ratio = (H/L)(100) in percent (see figure above),
S = ground slope = (Y/X)(100) in percent (see figure above),
T_{15} = thickness of layer with (N_{1})_{60} < 15 (m),
F_{15} = fines content in T_{15} layer (%),
D_{50,15} = average mean grain size in T_{15} layer (mm).

**Recommended MLR Equations**


Free face conditions:

\[
\log DH = -18.084 + 1.581 M - 1.518 \log R^* - 0.011 R + 0.551 \log W + 0.547 \log T_{15} + 3.976 \log (100-F_{15}) - 0.923 \log (D_{50,15}+0.1)
\]

Ground slope conditions:

\[
\log DH = -17.614 + 1.581 M - 1.518 \log R^* - 0.011 R + 0.343 \log S + 0.547 \log T_{15} + 3.976 \log (100-F_{15}) - 0.923 \log (D_{50,15}+0.1)
\]

where \( R^* = R + R_0 \) and \( R_0 = 10^{0.89M-5.64} \)

**Note:**

- This model is valid for coarse-grained sites (\( D_{50,15} \) up to 10mm for silty sediments)
- Predicted displacements greater than 6 m are poorly constrained by observational data and are highly uncertain
Measured versus predicted displacements for Port and Rokko Islands, Japan showing that Bartlett and Youd Equations greatly under-predict displacements at coarse-grained sites (Youd et al. 1999).

Liquefaction Countermeasures

Ahmed Elgamal

Source: Hayward Baker
http://www.haywardbaker.com/

Compaction Grouting
When low-slump compaction grout is injected into granular soils, grout bulbs are formed that displace and densify the surrounding loose soils. The technique is ideal for remediating or preventing structural settlements, and for site improvement of loose soil strata.

Chemical Grouting
The permeation of very low-viscosity chemical grout into granular soil improves the strength and rigidity of the soil to limit ground movement during construction. Chemical grouting is used extensively to aid soft ground tunneling and to control groundwater intrusion. As a remedial tool, chemical grouting is effective in waterproofing leaking subterranean structures.
**Cement Grouting** Primarily used for water control in fissured rock, Portland and microfine cement grouts play an important role in dam rehabilitation, not only sealing water passages but also strengthening the rock mass. Fast-set additives allow cement grouting in moving water and other hard-to-control conditions.

**Soilfrac Grouting** Soilfrac<sup>sm</sup> grouting is used where a precise degree of settlement control is required in conjunction with soft soil stabilization. Cementitious or chemical grouts are injected in a strictly controlled and monitored sequence to fracture the soil matrix and form a supporting web beneath at-risk structures.

**Jet Grouting** Jet grouting is an erosion/replacement system that creates an engineered, in situ soil/cement product known as Soilcrete<sup>sm</sup>. Effective across the widest range of soil types, and capable of being performed around subsurface obstructions and in confined spaces, jet grouting is a versatile and valuable tool for soft soil stabilization, underpinning, excavation support and groundwater control.

**Vibro-Compaction** A site improvement technique for granular material, Vibro-Compaction uses company-designed probe-type vibrators to densify soils to depths of up to 120 feet. Vibro-Compaction increases bearing capacity for shallow-footing construction, reduces settlements and also mitigates liquefaction potential in seismic areas.
**Vibro-Replacement** Related to Vibro-Compaction, Vibro-Replacement is used in clays, silts, and mixed or stratified soils. Stone backfill is compacted in lifts to construct columns that improve and reinforce the soil strata and aid in the dissipation of excess pore water pressures. Vibro-Replacement is well suited for stabilization of bridge approach soils, for shallow footing construction, and for liquefaction mitigation.

**Vibro Concrete Columns** Very weak, cohesive and organic soils that are not suitable for standard Vibro techniques can be improved by the installation of Vibro Concrete Columns. Beneath large area loads, Vibro Concrete Columns reduce settlement, increase bearing capacity, and increase slope stability.

**Dynamic Deep Compaction** Dynamic Deep Compaction™ is an economic site improvement technique used to treat a range of porous soil types and permit shallow, spread footing construction. Soils are densified at depth by the controlled impact of a crane-hoisted, heavy weight (15-35 tons) on the ground surface in a pre-determined grid pattern. Dynamic Deep Compaction is also successful in densifying landfill material for highway construction or recreational landscaping.

**Soil Mixing** Typically used in soft soils, the soil mixing technique relies on the introduction of an engineered grout material to either create a soil-cement matrix for soil stabilization, or to form subsurface structural elements to support earth or building loads. Soil mixing can be accomplished by many methods, with a wide range of mixing tools and tool configurations available.
**Minipiles** Underpinning of settling or deteriorating foundations, and support of footings for increased capacity are prime candidates for minipile installation, particularly where headroom is limited or access restricted. These small diameter, friction and/or end bearing elements can transfer ultimate loads of up to 350 tons to a competent stratum.

Extensive Literature is available at the Hayward Baker Web-site:
http://www.haywardbaker.com/

Nicholson Construction Company
http://www.nicholsonconstruction.com/

**Grouting Applications**  
**Pin Piles**