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IGT Professur für Geotechnik

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Introduction to Soil Liquefaction

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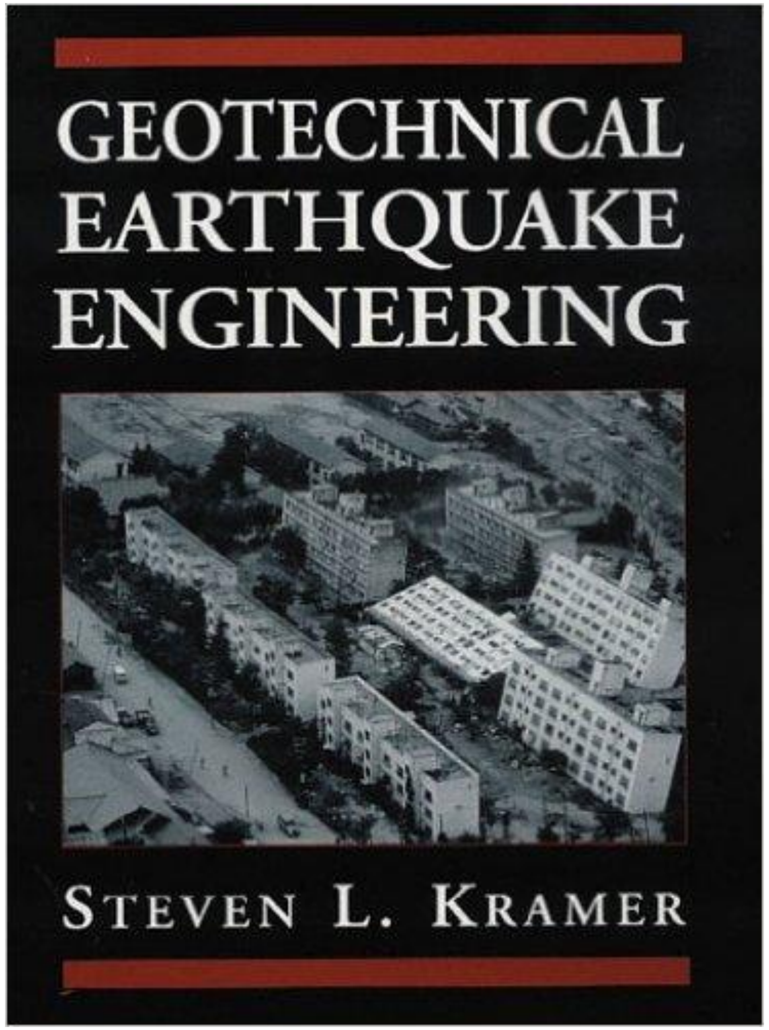
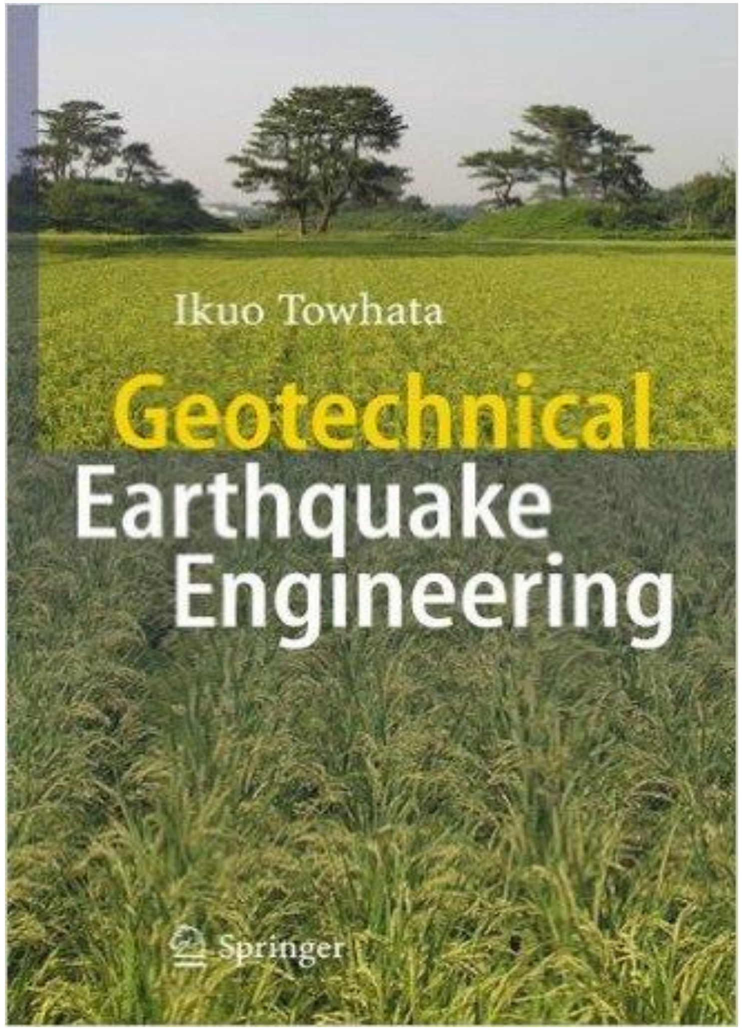
Contents

Suggested Books

Geotechnical Earthquake Engineering





Introduction

What is “Soil Liquefaction”?

The phenomenon where **a fully or partially saturated soil looses strength and stiffness in response to an applied shear stress** (e.g. seismic shaking or other sudden load), causing it to **behave like a liquid**, being unable to support structures or remain stable.

- The term is used to describe a variety of different, yet related phenomena, observed in ***loose, saturated sandy soils***.
- The common factor is the *generation of **excess pore water** pressures under undrained loading conditions*.

Japan 3.11.2011
Youtube Video

What is “Soil Liquefaction”?

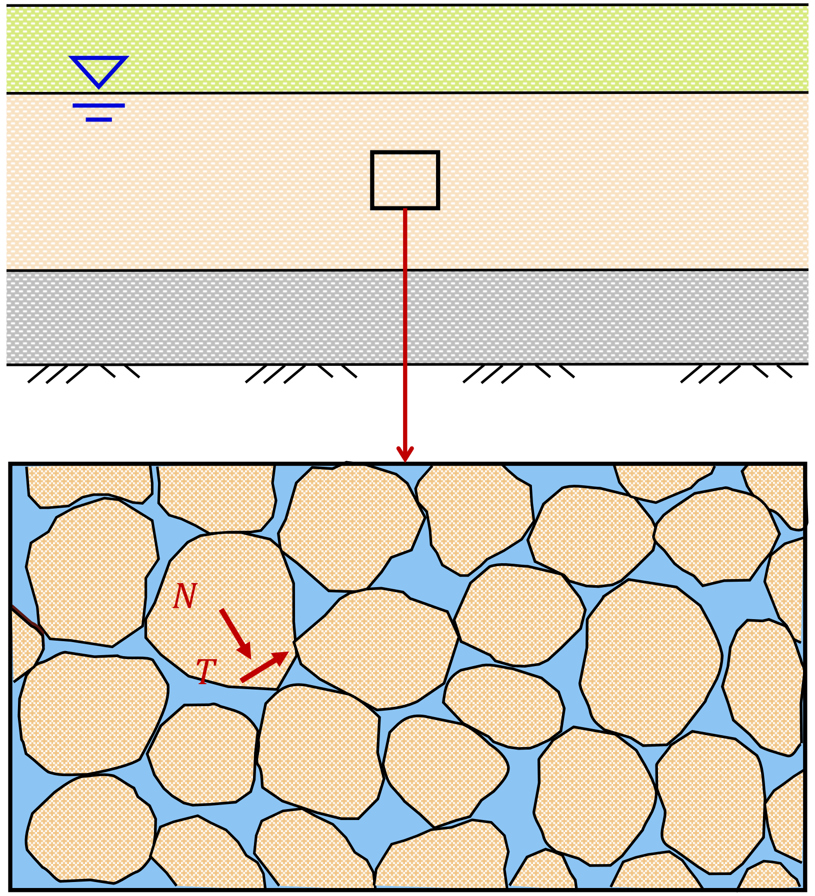
Effective Stresses

Terzaghi (1925) was the first to recognize and explain the difference between total and effective stresses.

- The total stresses $\boldsymbol{\sigma}$ are divided into effective stresses $\boldsymbol{\sigma'}$ acting on the soil skeleton (i.e., the soil particles) and the pore water pressures $\boldsymbol{u}$ acting on the pore water.

$$\boxed{\boldsymbol{\sigma} = \boldsymbol{\sigma'} + \boldsymbol{u}}$$

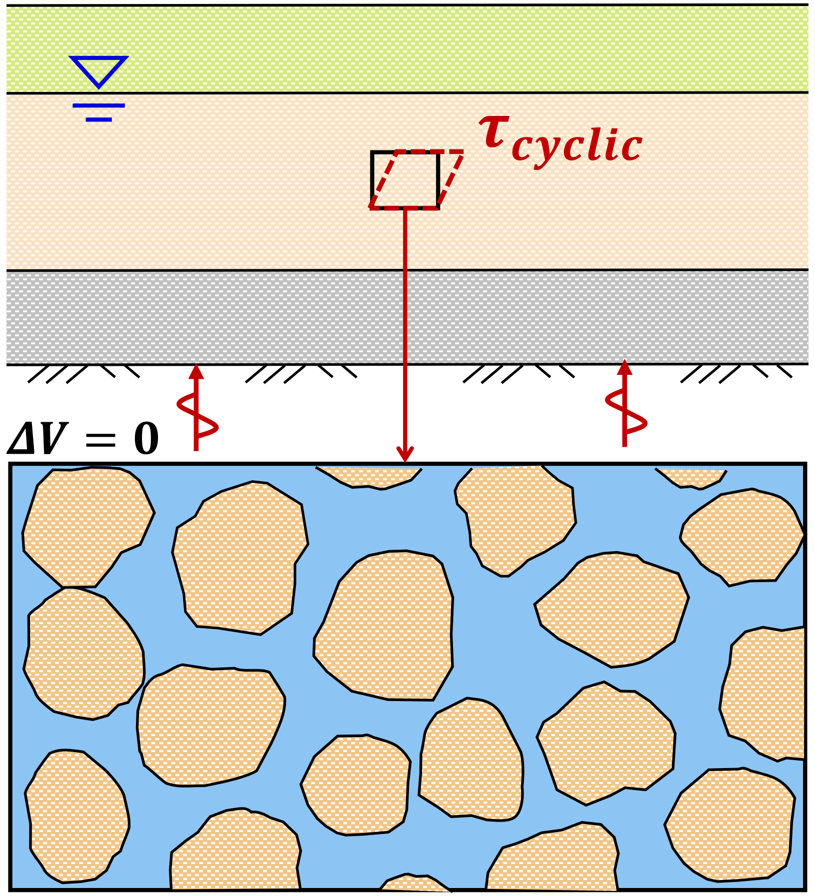
- The ***shear strength*** of soil is exclusively related to the effective stress $\boldsymbol{\sigma'}$.



What is “Soil Liquefaction”?

Cyclic Loading

- The tendency of dry (especially loose) cohesionless soils to densify when subjected to cyclic loading $\boldsymbol{\tau_{cyclic}}$ is well known.
- However, when such loose cohesionless soils are saturated, rapid loading will take place under ***undrained conditions***.
- Since volume change is impossible under undrained conditions, $\boldsymbol{\Delta V = 0}$, this inherent tendency for densification will unavoidably lead to the development of ***excess pore water pressures $\boldsymbol{\Delta u}$***.



What is “Soil Liquefaction”?

Cyclic Loading: excess pore water pressures

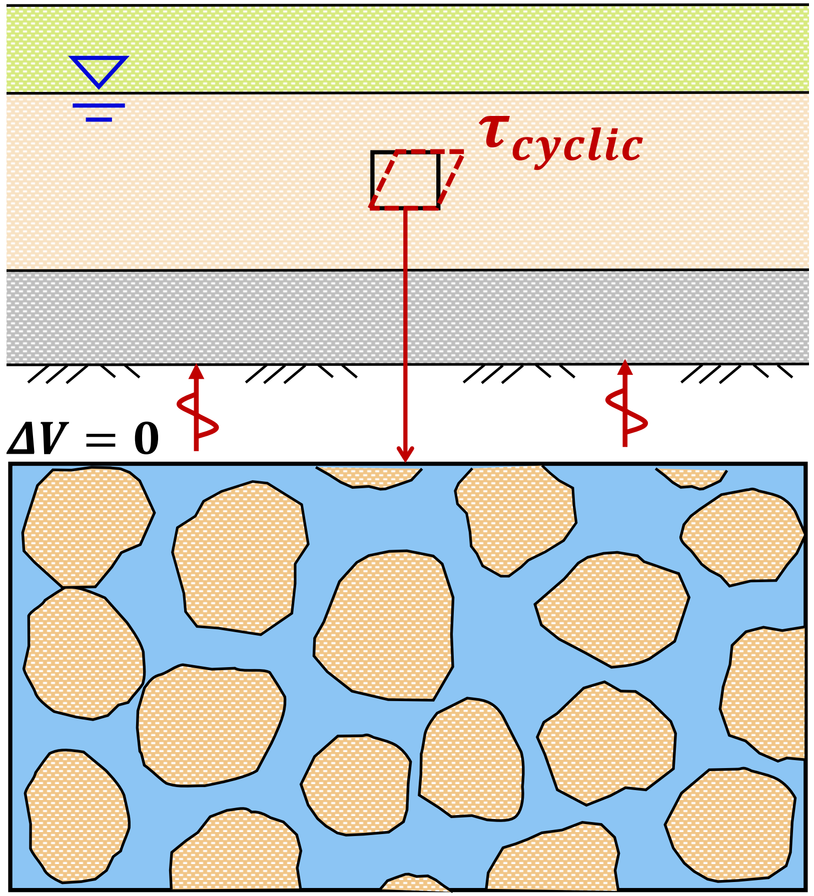
- The development of such excess pore water pressures $\boldsymbol{\Delta u}$ leads to a corresponding decrease of the effective stress $\boldsymbol{\sigma'}$.

- Given that the effective stress is:

$$\boldsymbol{\sigma' = \sigma - u}$$

  if the excess pore water pressure together with the initial $\boldsymbol{u}$ becomes equal to $\boldsymbol{\sigma}$, the effective stress will reduce to zero: $\boldsymbol{\sigma' = 0}$.

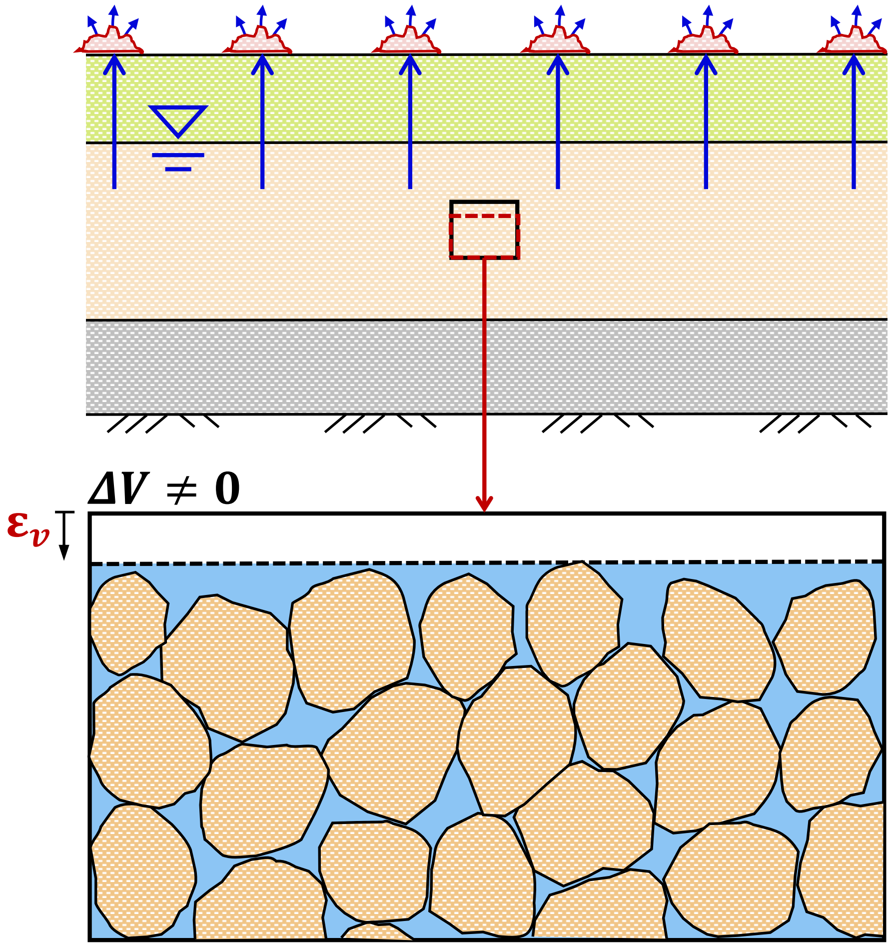
- Contact between soil grains is lost and the soil behaves as a liquid “slurry” of (almost) no shear strength.



What is "Soil Liquefaction"?

Dissipation of excess pore water pressures

- After a period of time, excess pore water pressures will dissipate, as water will flow out from the voids and soil particles will eventually regain contact.
- Since the soil is liquefied, a mixture of water and sand will be ejected towards the ground surface, leading to development of sand boils.
- The volume will decrease ($\Delta V \neq 0, \varepsilon_v$) and the liquefied soil deposit will unavoidably experience permanent soil deformations (settlements etc.).



Flow Liquefaction

2018 Palu Earthquake
[Youtube Video]

Flow liquefaction produces the most dramatic effects of all liquefaction-related phenomena. It may lead to spectacular instabilities, known as ***"flow failures"***.

- It occurs when the ***static shear stress*** (required for static equilibrium) is greater than the ***shear strength*** of the liquefied soil.
- Once triggered, excessively large deformations are produced, ***driven by static shear stresses***.
- Such failures are sudden and develop quickly, with the liquefied material moving fast over large distances.

Flow liquefaction failure of the Lower San Fernando Dam (California) after the 1971 San Fernando Earthquake.

Cyclic Mobility

In contrast to flow liquefaction, cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil. Permanent deformations can still be large.

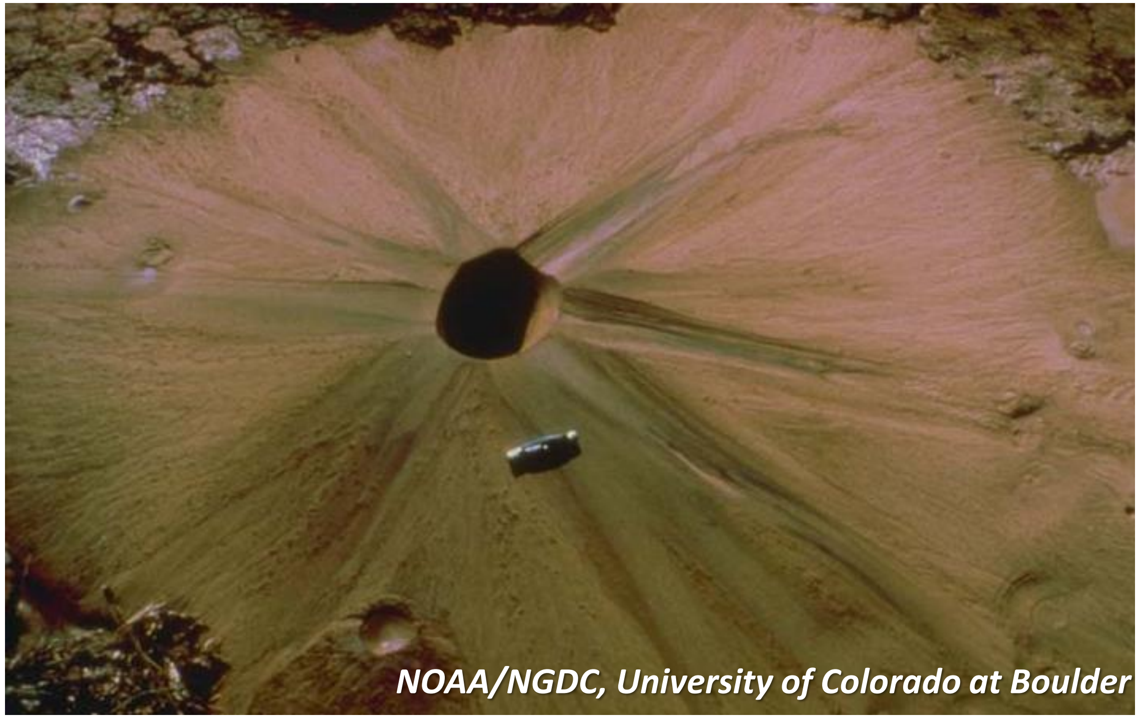
- Such deformation, known as ***"lateral spreading"*** develops incrementally during shaking.
- In contrast to cyclic mobility, the deformation is ***driven by both cyclic and static shear stresses***.
- It can occur in gently sloping ground or adjacent to bodies of water.
- In the presence of structures, it may lead to dramatic failures.

Failure due to lateral spreading of the Kawagishi-cho apartment buildings in Japan, following the 1964 Niigata Earthquake.

Cyclic Mobility

A special case of cyclic mobility is level–ground liquefaction, as the horizontal static shear stresses that could drive lateral deformations do not exist.

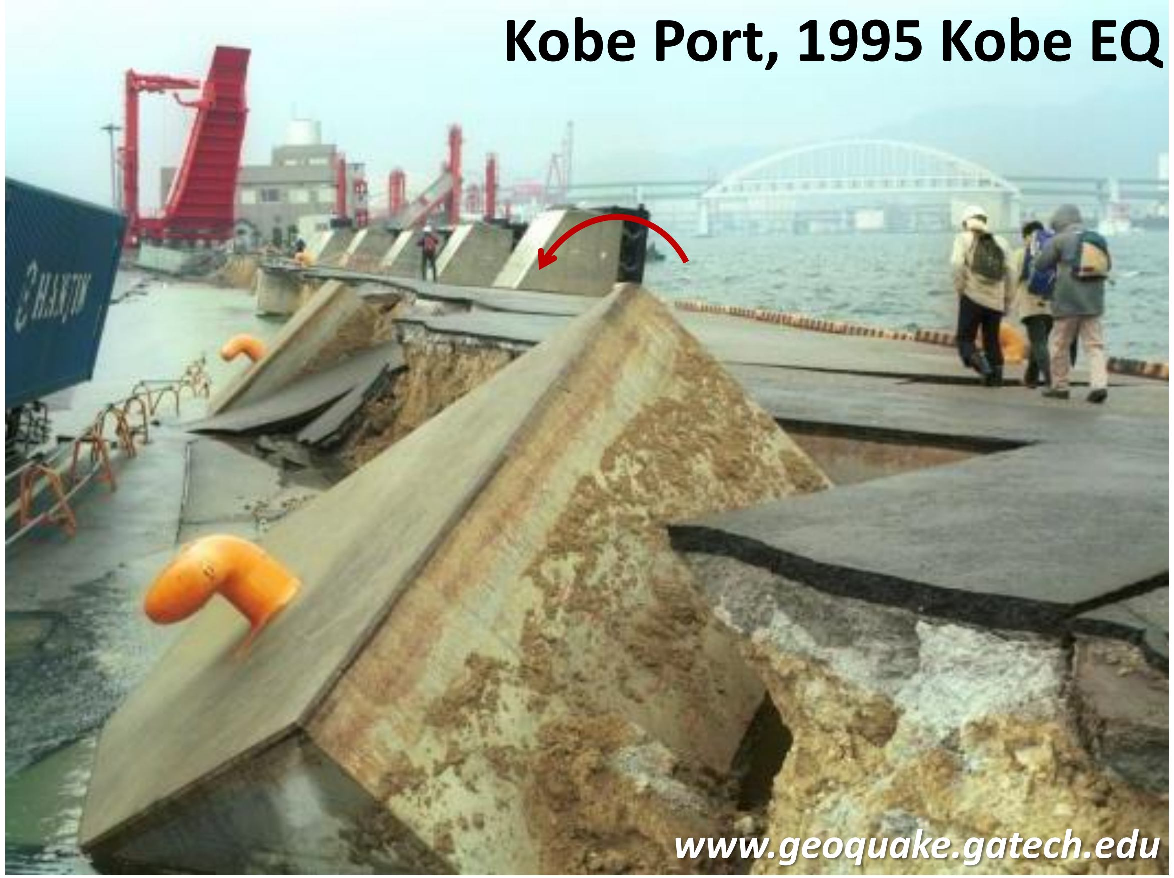
- Permanent lateral deformations are limited, but large chaotic movement can take place during seismic shaking, known as ***"ground oscillation".***
- Level-ground failures are caused by the upward flow of water during dissipation of excess pore water pressures: ***excessive settlement and flooding***
- Depending on the time required to reach hydraulic equilibrium, failure may occur well after the end of shaking.



Liquefaction-induced sand boil, after the 1979 El Centro earthquake.

Liquefaction–induced Failures

Harbour Quay Walls



Liquefaction–induced Failures

Buildings

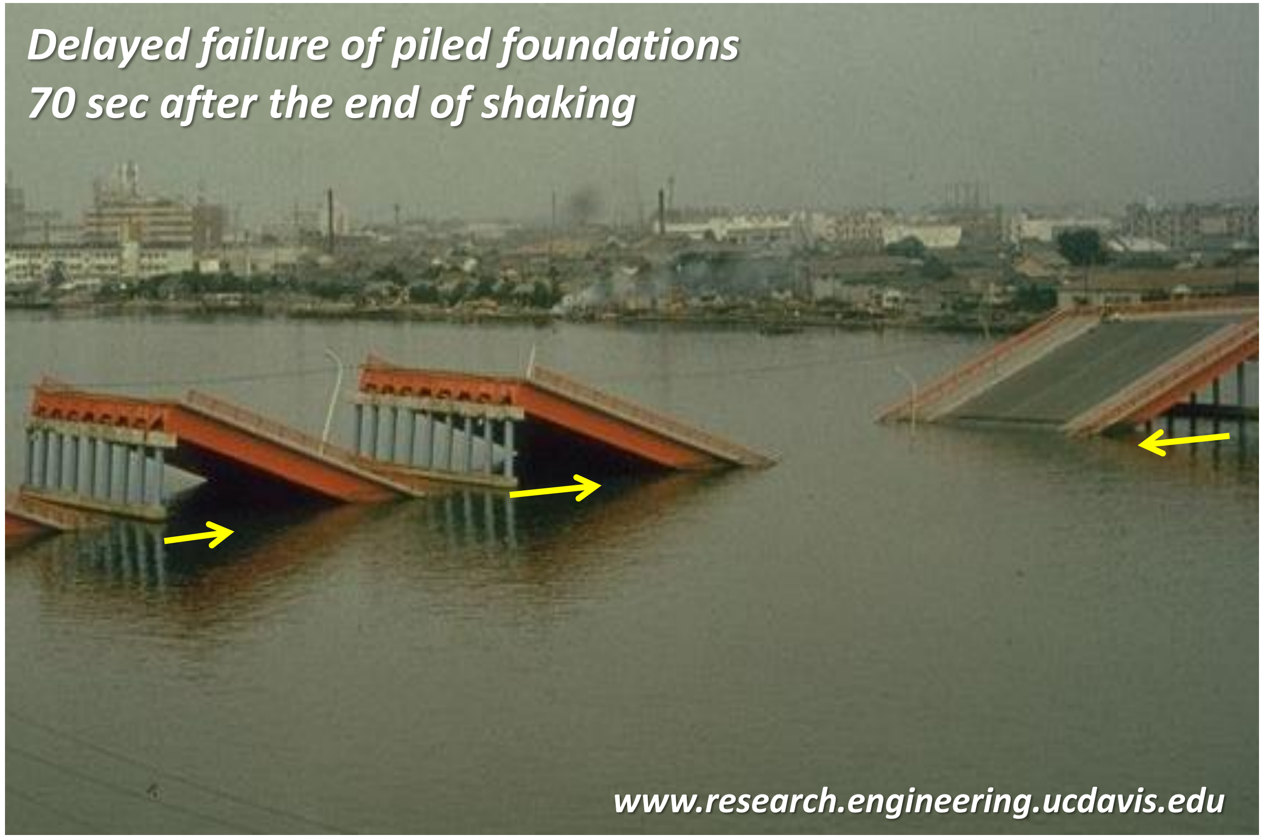
Liquefaction–induced Failures

Pavements

Liquefaction–induced Failures

Bridges

Showa brdige, 1964 Niigata (Japan) EQ



Nishinomiya-ko Ohashi, 1995 Kobe EQ

Evaluation of Liquefaction Hazard

Potential liquefaction hazards can be evaluated by addressing 3 questions:

1) Is the soil susceptible to liquefaction?

2) Is the disturbance strong enough to trigger liquefaction?

3) If liquefaction is triggered, will damage occur (and to what extent)?

- If the answer to ***Question 1*** is **NO** → the site is ***free of liquefaction hazard***
- If the answer to ***Question 1*** is **YES** → Questions 2 and 3 should be answered
- If the answer to ***All questions*** is **YES** **AND** the anticipated damage is unacceptable → the site should be abandoned or remediated

Liquefaction Susceptibility

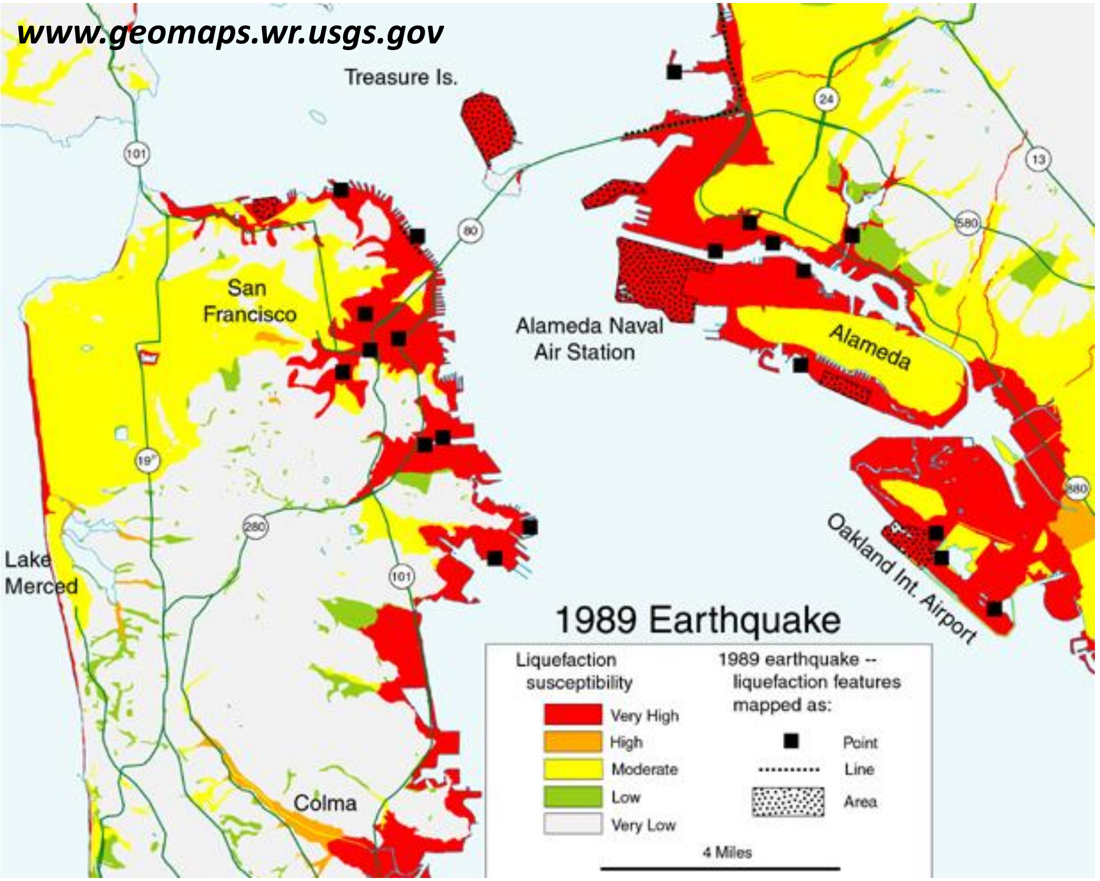
Introduction

- Evaluation of liquefaction susceptibility is the first step in liquefaction hazard assessment.

  → If the soil is <u>not liquefiable</u>, the evaluation can be ended.

- If it is <u>susceptible to liquefaction</u>, liquefaction initiation and its effects have to be addressed.

- Liquefaction susceptibility can be judged on the basis of: ***historical, geological, compositional, and state-related criteria***



San Francisco Bay Area Liquefaction Susceptibility map

Historical Criteria

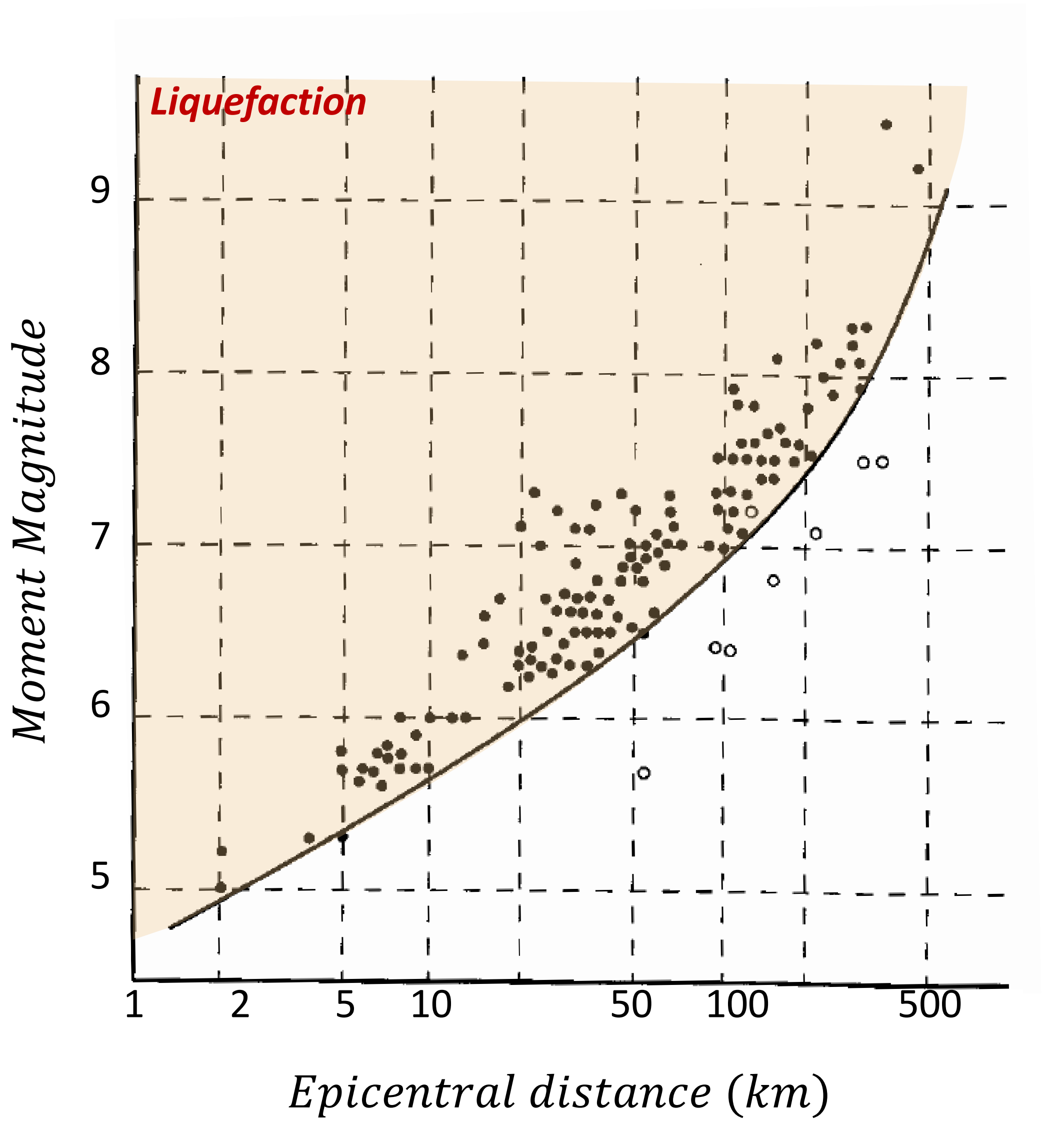
(Kramer, after Ambraseys, 1988)

- Liquefaction often recurs at the same location, when site (soil and groundwater) conditions remain the same.

  → ***liquefaction case histories*** can be used to identify susceptible soil sites.

- Strong correlation between M_w and epicentral distance of sites where liquefaction has been observed (Ambraseys, 1988).

  → The distance at which liquefaction is possible increases dramatically with M_w



Geological Criteria

Soils that are highly susceptible to liquefaction:

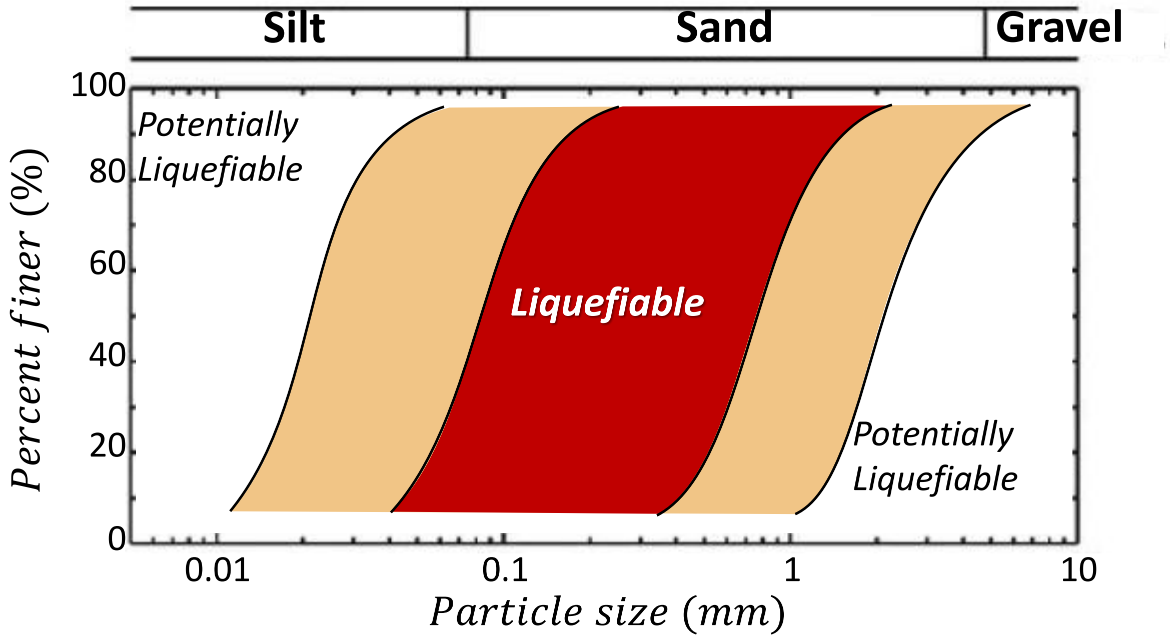
- **Loosely deposited cohesionless soils** with uniform gain size distribution (e.g., fluvial, alluvial, colluvial, and aeolian deposits).
- **Newer soil deposits:** soils of Holocene are more susceptible than those of Pleistocene.
- **Saturated soils** with groundwater table close to the ground surface.
- **Human–made deposits**, such as loose fills placed without compaction (hydraulic-fill dams, reclaimed land, etc.).

Alluvial deposit

Compositional Criteria

- Susceptibility to liquefaction is a function of ***particle size, shape, and gradation.***
- Liquefaction is mainly related to sands; finer-grained soils were believed not to be liquefiable → recently the gradation boundaries have been broadened
- Liquefaction of non-plastic silts has been observed (lab and field) → ***plasticity*** is of importance.
- **Coarse silts** with bulky particle shape (nonplastic and cohesionless) are fully susceptible to liquefaction.
- **Gravelly soils** can also be susceptible to liquefaction.

Example ranges of grain size distribution for soils susceptible to liquefaction (after Tsuchida, 1970)



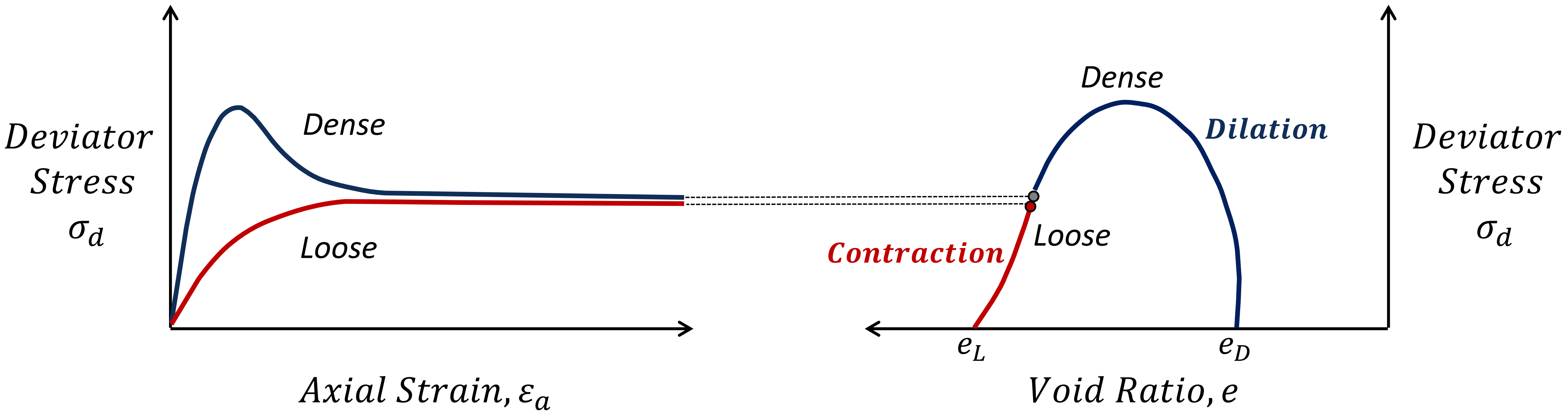
State Criteria

- Even if all previous criteria are satisfied, a soil may still not be susceptible to liquefaction → the ***initial state*** at the beginning of the earthquake plays a role.
- The tendency for contraction of a specific soil, and hence excess pore water pressure generation, is a function of its ***density and initial stress conditions***.
- In contrast to the previously discussed criteria, state–related liquefaction susceptibility criteria are ***different for flow liquefaction and cyclic mobility***.

State Criteria

Critical Void Ratio e_c

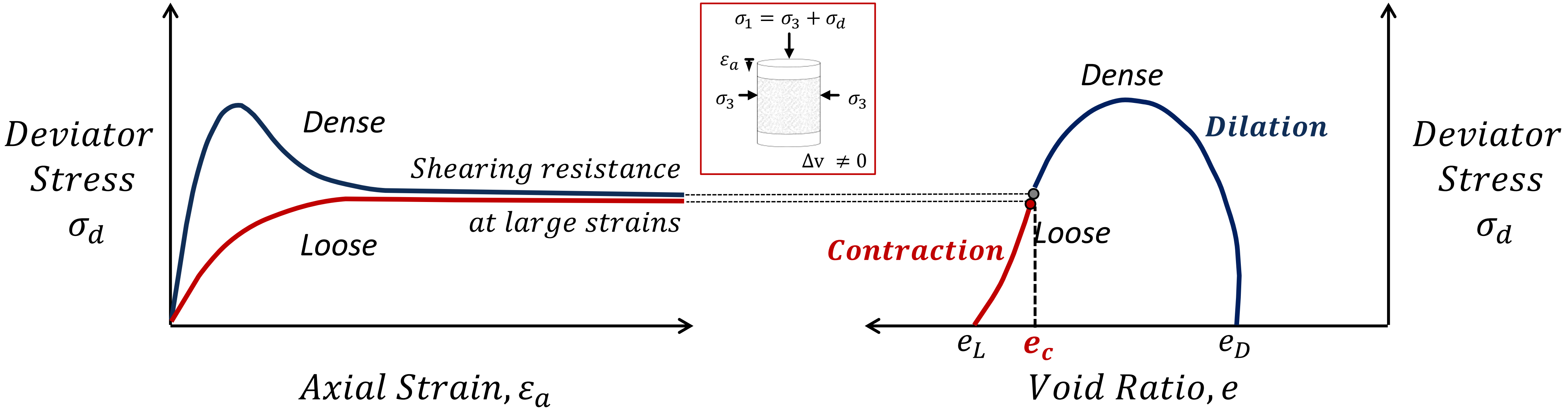
- Casagrande (1936) conducted drained ***strain-controlled*** triaxial tests on initially loose and dense specimens.
- The results showed that all specimens tested at the same effective confining pressure σ'_{3c} approached the same density when sheared to large strains.



State Criteria

Critical Void Ratio e_c

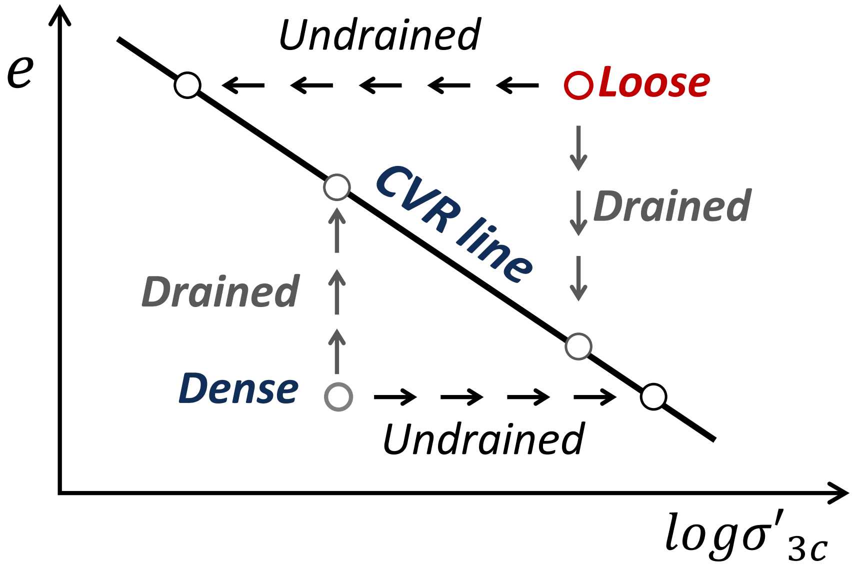
- During shearing, initially loose specimens contracted (or densified).
- Initially dense specimens first contracted (slightly), but then quickly began to dilate.
- At large strains, dense and loose specimens reach the **same critical void ratio e_c** , and continue to shear with the ***same shearing resistance***.



State Criteria

Critical Void Ratio e_c

- Conducting tests at different effective confining pressures σ'_{3c}, Casagrande concluded that the critical void ratio $\boldsymbol{e_c}$ is uniquely related to $\boldsymbol{\sigma'_{3c}}$.
- He named the locus ***"Critical Void Ratio" (CVR) line,*** which can be used to define the boundary between loose (contractive) and dense (dilative) states.
- **The CVR line** describes the state toward which any soil specimen will migrate at *large shear strains*, by:
  - ✓ *volume change* under *drained conditions*, or
  - ✓ changes in effective confining pressure σ'_{3c} under *undrained conditions*, or
  - ✓ partially drained conditions: some combination

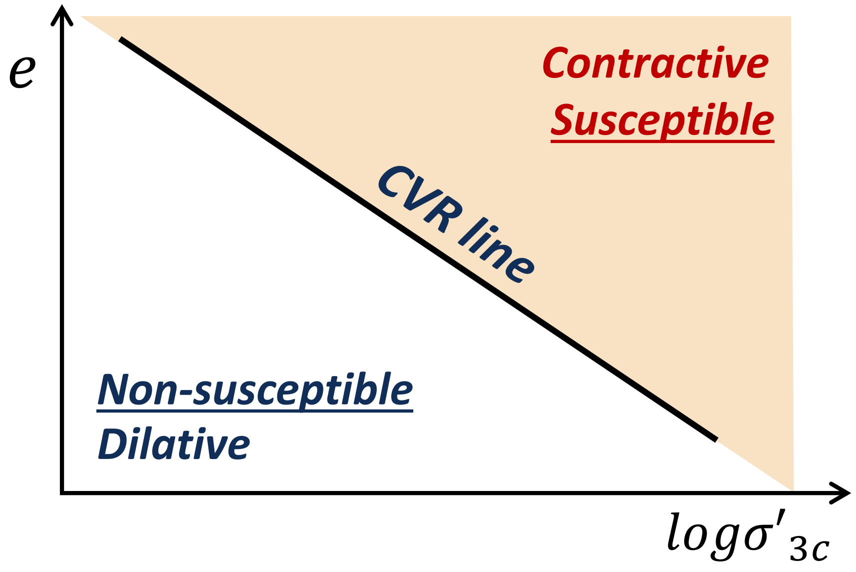


State Criteria

Critical Void Ratio e_c

- The CVR line represents the boundary between contractive and dilative behavior.
- So, it can also be considered as the boundary between soils susceptible and non-susceptible to ***flow liquefaction***.
- Saturated soils with ***initial void ratio e high enough*** to plot above the CVR line are considered susceptible to flow liquefaction.
- This was somehow challenged after failure of the ***Fort Peck Dam in Montana (1938),*** which was due to static flow liquefaction during construction

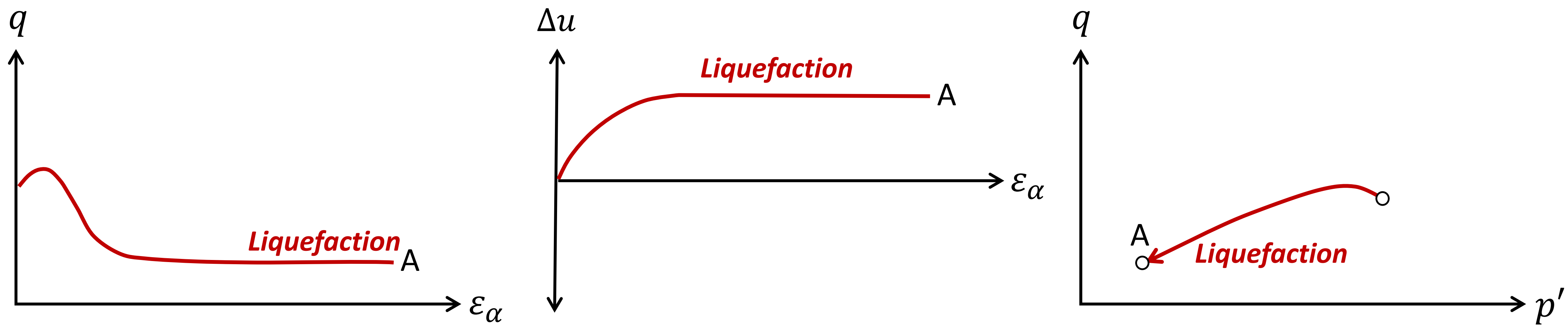
→ the initial e was a bit lower than the CVR line…



State Criteria

Steady state of Deformation

- Castro (1969), who was a student of Casagrande, condcuted a series of *stress-controlled* tests on <u>anisotropically consolidated</u> sand specimens.
- **Very loose sand (A)** specimens showed a peak undrained strength (at small shear strains), and then "collapsed" flowing rapidly to large strains at low effective confining pressure and low strength, a behavior termed *"flow liquefaction"*.

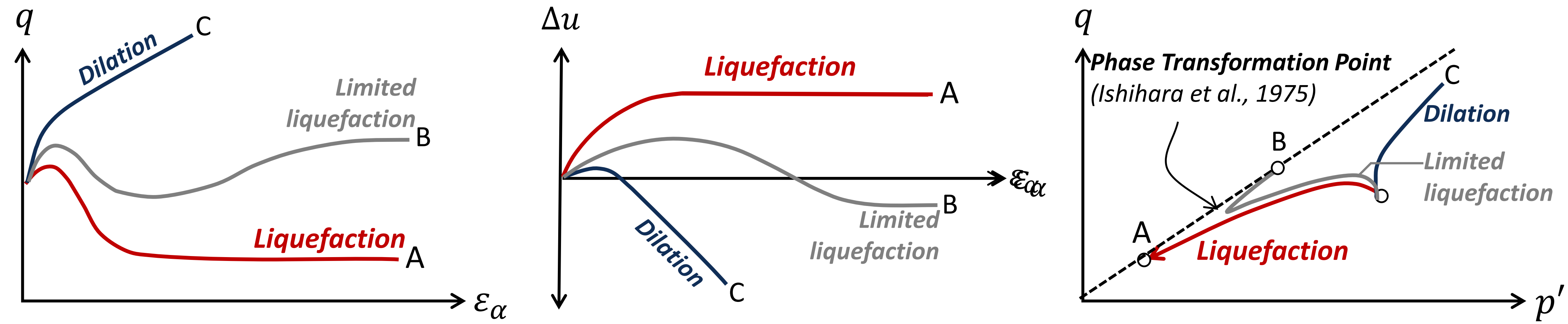


A = **Very loose sand** B = **Intermediate sand** C = **Dense sand**

State Criteria

Steady state of Deformation

- **Dense sand (C)** specimens initially contracted, but then dilated until reaching a relatively high constant effective confining pressure, accompanied by high large-strain shear strength.
- **Intermediate density (B)** specimens reached a peak undrained shear strength, which was followed by strain-softening response, ending up with onset of dilation at intermediate strains.
- This change from contractive to dilative behavior occurs at the ***phase transformation point***

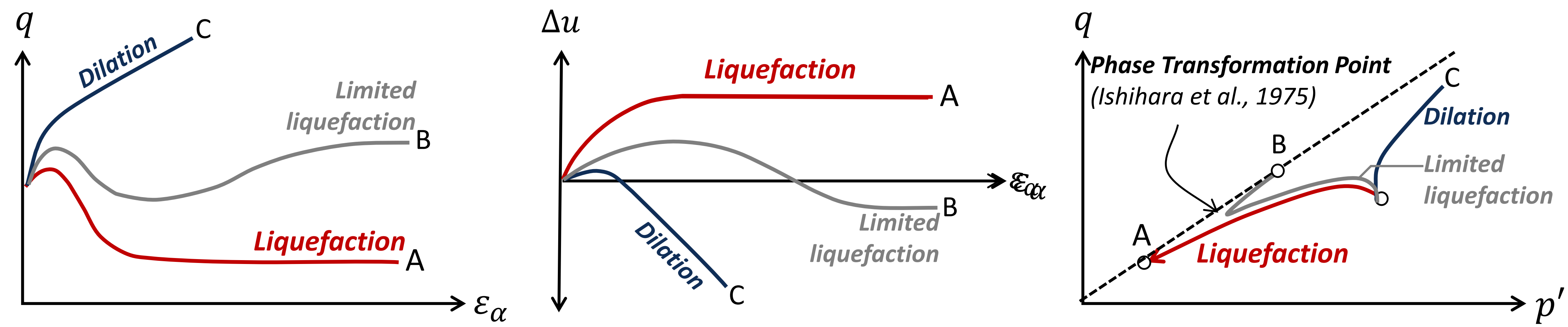


A = **Very loose sand** B = **Intermediate sand** C = **Dense sand**

State Criteria

Steady state of Deformation

- Further loading led to dilative behavior to higher effective confining pressures at large strains, a behavior termed ***"limited liquefaction".***
- The tests showed that there is a unique relationship between the void ratio and effective confining pressures at large strains, which plots ***parallel to the CVR line, but a little lower***.

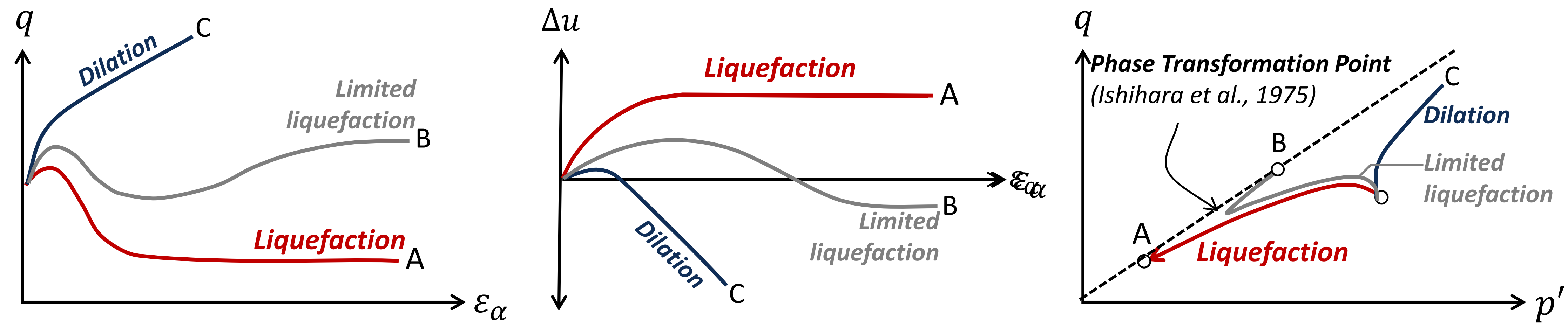


A = **Very loose sand** B = **Intermediate sand** C = **Dense sand**

State Criteria

Steady state of Deformation

- The difference was attributed to the development of the flow structure under ***stress-controlled*** conditions (Casagrande's tests were *strain-controlled*).
- The state in which the soil flows continuously under constant shear stress, constant effective confining pressure at constant volume and constant velocity is defined (Poulos, 1977) as the ***steady state of deformation***.

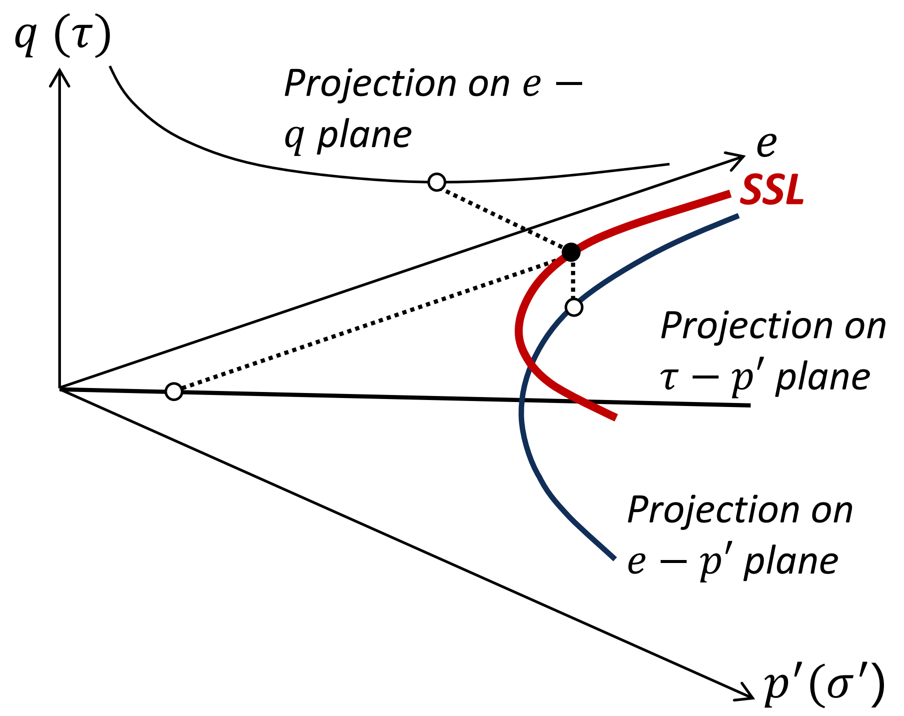


A = **Very loose sand** B = **Intermediate sand** C = **Dense sand**

State Criteria

Steady state of Deformation

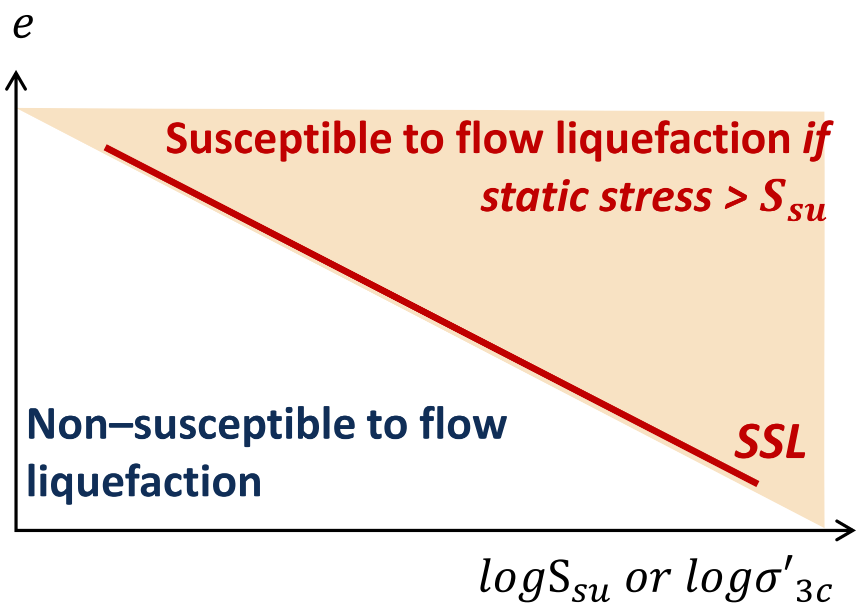
- The *steady state of deformation* was believed to be a function of ***density only*** → it was later proven that deposition, stress, and loading conditions play a role.
- The unique relationship between effective confining stress $\boldsymbol{p'}$ **(or** $\boldsymbol{\sigma'}$**)** and void ratio $\boldsymbol{e}$ at the steady state of deformation is described graphically by the ***Steady State Line (SSL).***
- The **SSL** is a 3D curve in the $(e - p' - q)$ or $(e - \sigma' - \tau)$ space. It can be projected in:
  - ✓ $e - q$ *plane*
  - ✓ $\tau - p'$ *plane*
  - ✓ $e - p'$ *plane*



State Criteria

Steady state of Deformation

- The SSL can also be expressed in terms of steady state shear strength S_{su}.
- Since the shearing resistance in the steady state of deformation is proportional to the effective confining pressure, the strength-based SSL is parallel to the effective confining pressure-based SSL, when both are plotted in *logarithmic scale*.
- Soils with initial condition ***above the SSL*** are prone to *flow liquefaction,* only if the static shear stress exceeds the steady state shear strength S_{su}



Cyclic mobility does not follow this criterion. It may occur in both dense & loose soils.

Initiation of Liquefaction

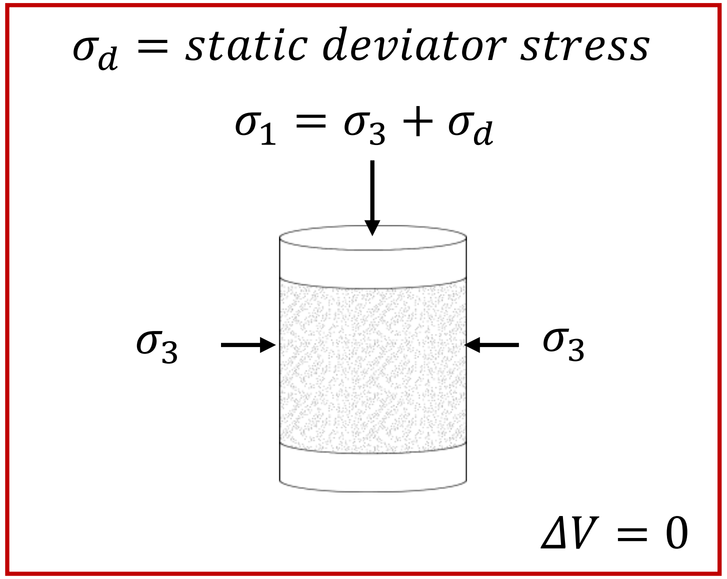
Introduction

- Even if a soil deposit is susceptible to liquefaction, ***a strong enough disturbance*** *(i.e., change in stress) is required for its initiation*.
- Evaluation of such disturbance is one of the most critical parts of liquefaction hazard evaluation.
- ***Cyclic mobility*** is an earthquake–related phenomenon, but flow liquefaction can be initiated in various ways.
- Flow slides triggered by monotonic loading, ***"static liquefaction"***, have been observed in natural soil deposits and human-made fills.
- Flow liquefaction has also been triggered by ***non-seismic sources***, such as blasting, train traffic, pile driving…
- Understanding the ***initiation of liquefaction*** requires identification of the state of the soil when liquefaction is triggered.

Flow Liquefaction Surface (FLS)

- The effective stress conditions at the initiation of flow liquefaction can be described in the stress-path $(e - p' - q)$ or $(e - \sigma' - \tau)$ space by a 3D surface, which is termed as the ***Flow Liquefaction Surface (FLS).***
- The conditions at initiation of flow liquefaction can be explained easier when the soil is subjected to ***monotonically increasing stresses***.
- Let's see how the ***FLS*** can be constructed in the $(q - p')$ plane
- For this purpose, let's consider the response of isotropically consolidated specimens of very loose, saturated sand, subjected to ***triaxial compression under undrained conditions.***

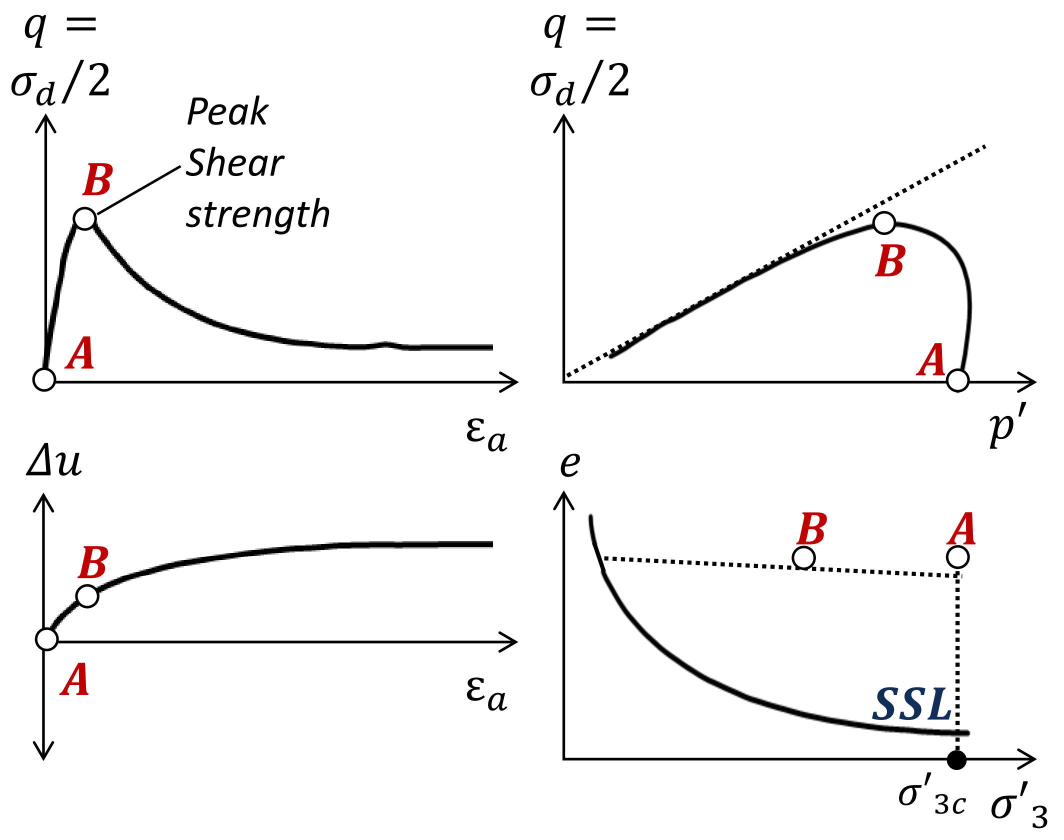
Undrained Triaxial Compression Test



Flow Liquefaction Surface

Monotonic Loading

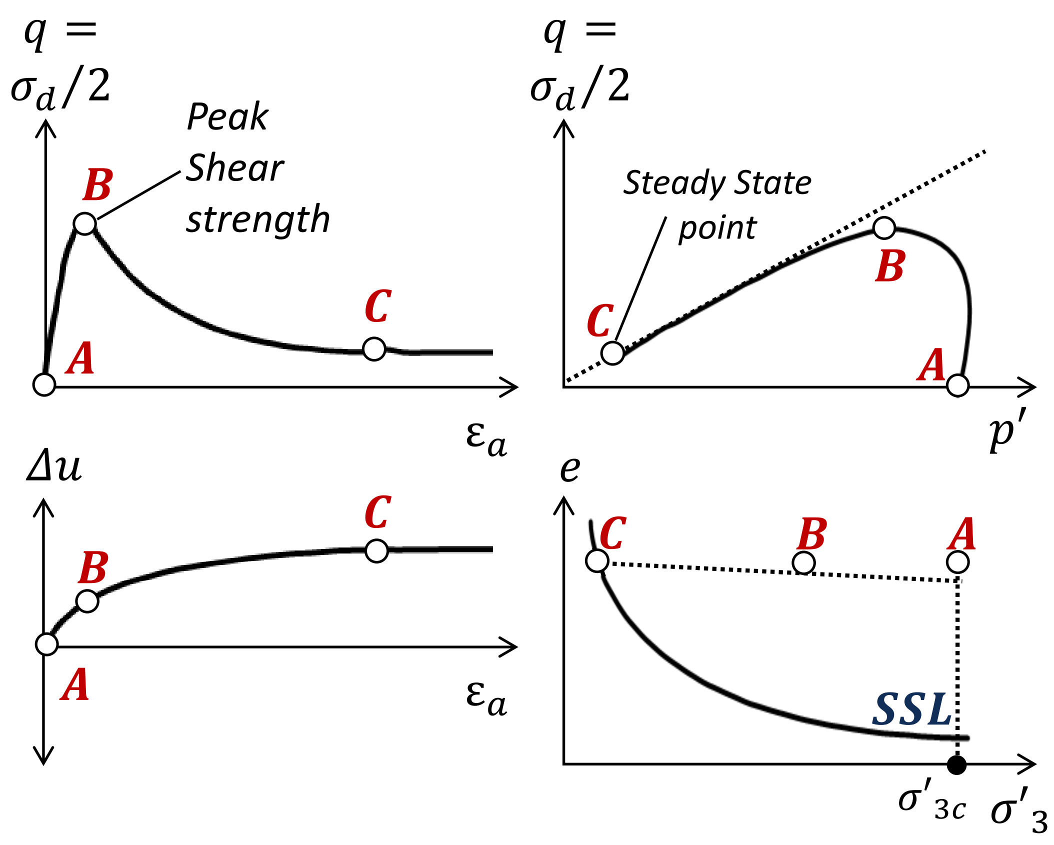
- Prior to undrained shearing (***A***), the specimen is in drained equilibrium under initial σ'_{3c}, shear stress $q = 0$, and excess pore pressure $\Delta u = 0$.
- Since the initial state is well above the ***Steady State Line (SSL)***, the specimen will exhibit contractive behaviour.
- With the initiation of undrained shearing, the specimen will generate $\boldsymbol{\Delta u}$ while mobilizing its shear resistance up to a peak value (***B***).



Flow Liquefaction Surface

Monotonic Loading

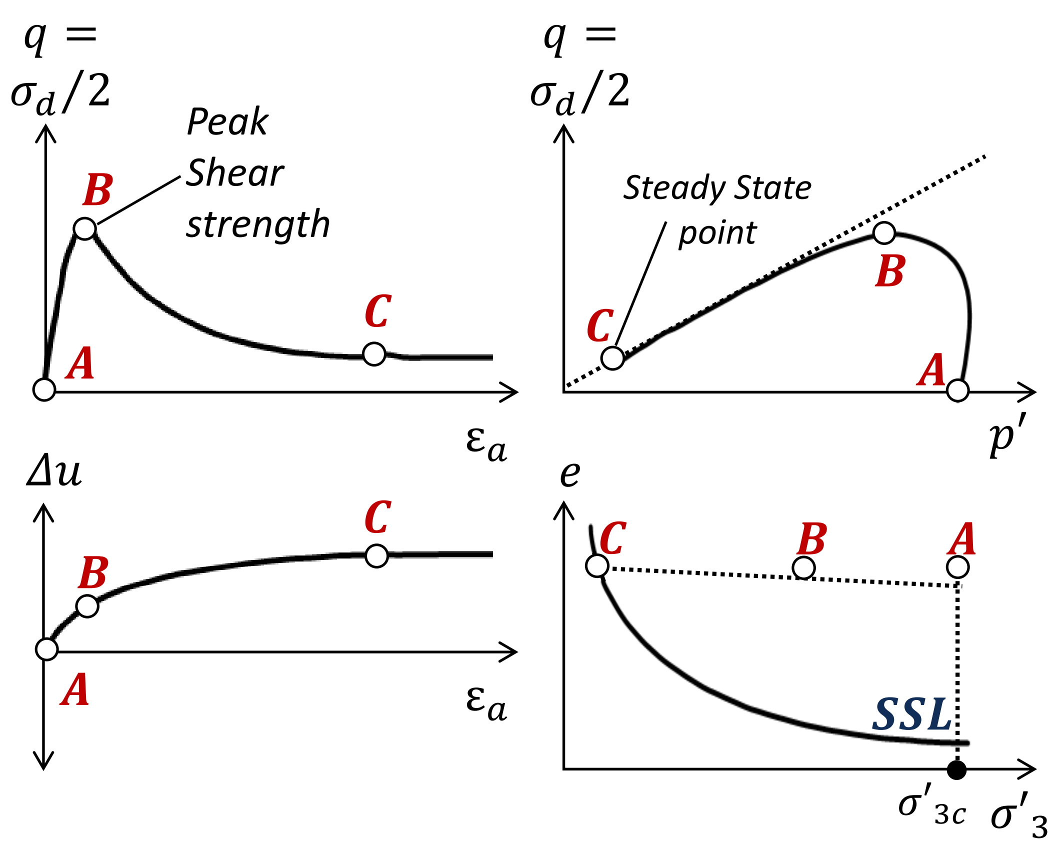
- At ($\boldsymbol{B}$) Δu is relatively small, with the ***pore pressure ratio*** $\boldsymbol{r_u = \Delta u/\sigma'_{3c}}$ well below 1.0.
- Upon reaching ($\boldsymbol{B}$), however, the specimen becomes unstable and collapses: $\boldsymbol{\varepsilon_a}$ increases abruptly.
- As the specimen strains from ($\boldsymbol{B}$) to ($\boldsymbol{C}$), the excess pore pressure $\boldsymbol{\Delta u}$ increases substantially.
- At and beyond ($\boldsymbol{C}$), the specimen is in the ***steady state of response***.



Flow Liquefaction Surface

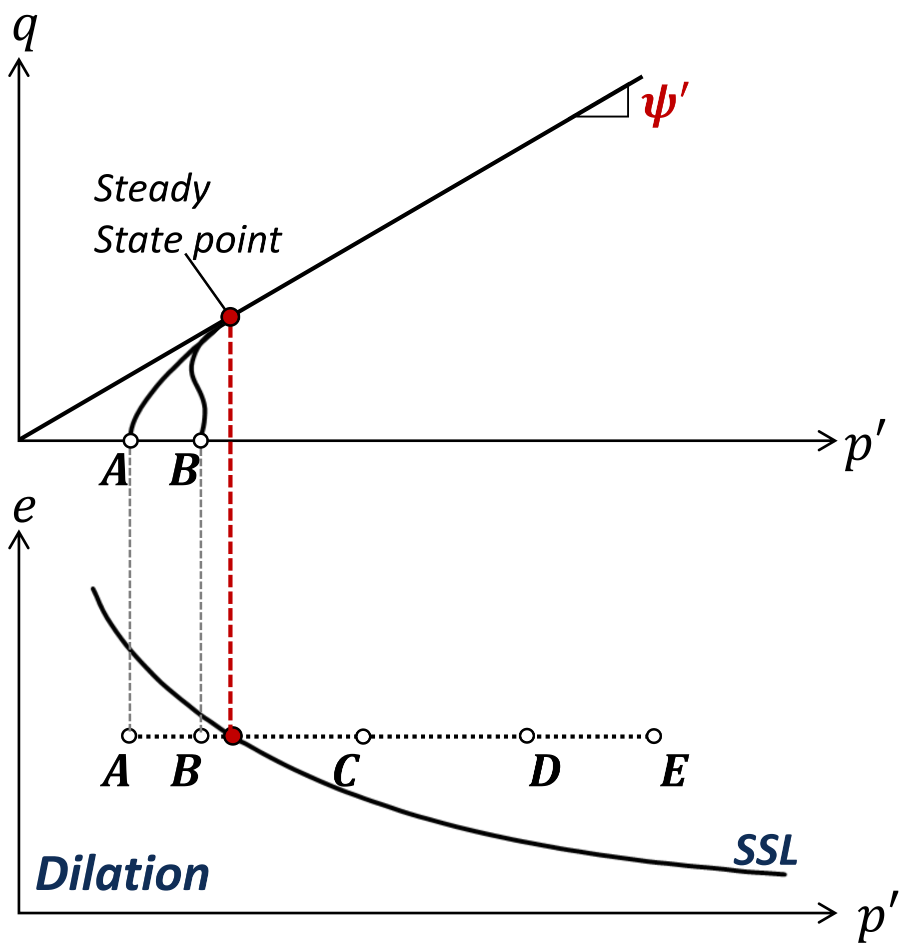
Monotonic Loading

- The effective confining pressure has decreased dramatically, and the ***pore pressure ratio*** $\boldsymbol{r_u = 1}$
- The specimen has exhibited ***flow liquefaction behavior,*** because the static shear stress required for equilibrium at ($\boldsymbol{B}$) was greater than the available shear strength of the liquefied soil ($\boldsymbol{C}$).
- Flow liquefaction was initiated at the instant it became irreversibly unstable ($\boldsymbol{B}$).



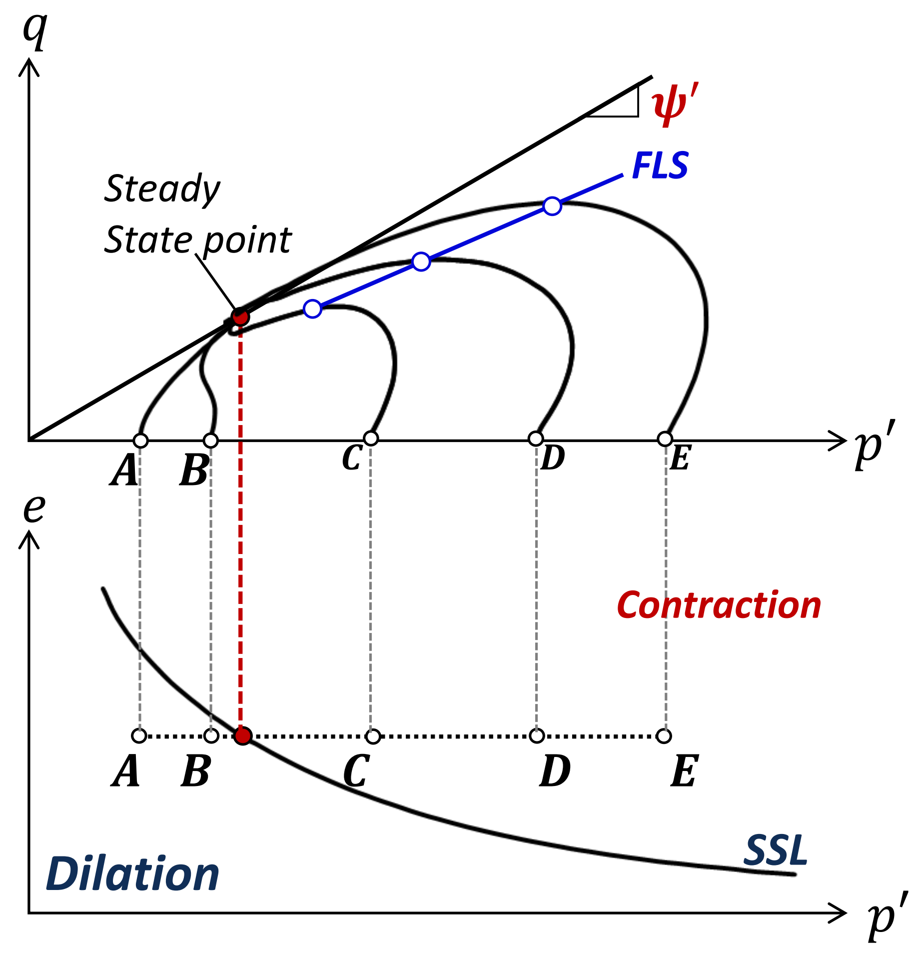
Flow Liquefaction Surface

- Let's now consider 5 specimens ($\boldsymbol{A}$ to $\boldsymbol{E}$), isotropically consolidated to the same initial void ratio $\boldsymbol{e}$, but at different initial effective confining pressures $\boldsymbol{p}'$.
- Since all specimens have the same $\boldsymbol{e}$, they will all reach the same effective stress conditions at steady state, but they will get there following different stress paths.
- The initial states of $\boldsymbol{A}$ and $\boldsymbol{B}$ are bellow the ***steady state line (SSL)*** and therefore they will exhibit dilative behavior upon shearing.



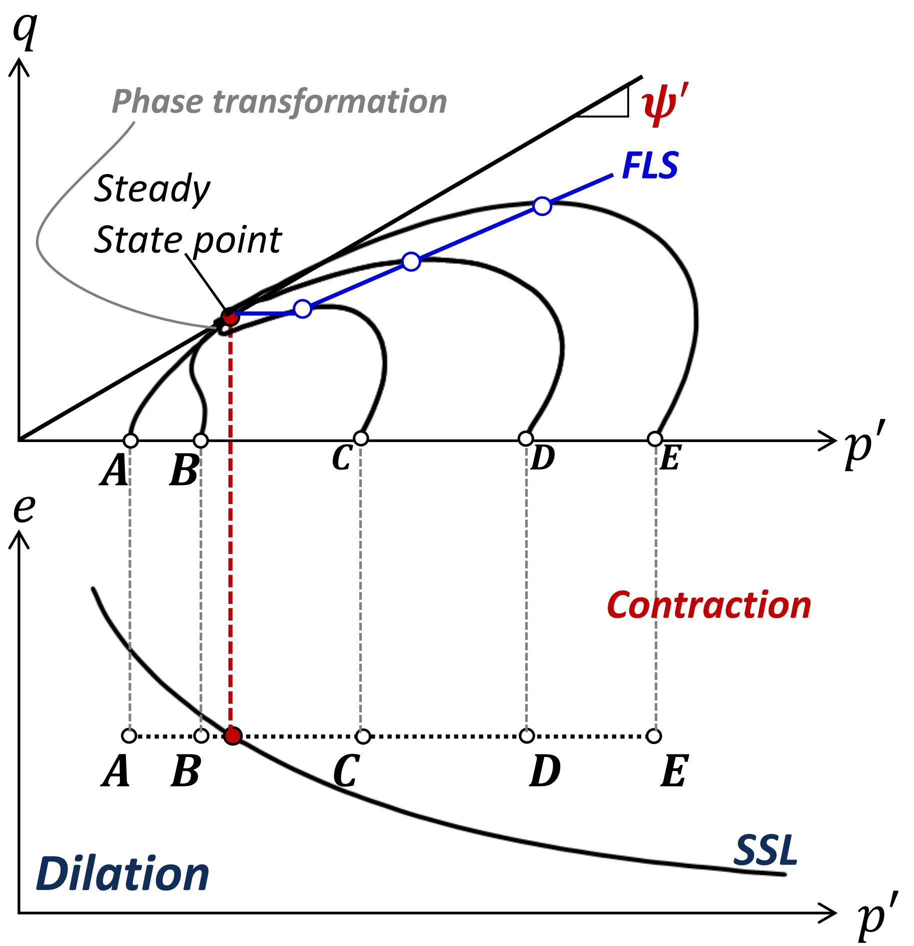
Flow Liquefaction Surface

- The initial states of $\boldsymbol{C}, \boldsymbol{D}, \boldsymbol{E}$ are above the ***SSL*** and therefore they will exhibit ***contractive*** behavior upon shearing.
- Each specimen reaches a peak strength, after which it strains rapidly towards the steady state.
- Flow liquefaction is initiated at the peak of each stress path: ***blue dots***.
- The locus of these points, describing the effective stresses at liquefaction initiation define the ***Flow Liquefaction Surface (FLS).***



Flow Liquefaction Surface

- Since flow liquefaction cannot occur if the stress path is below the ***Steady State point***, the ***FLS*** is truncated → It defines the boundary between *stable and unstable states*.
- If the ***FLS*** is reached under undrained conditions, flow liquefaction is triggered and the shearing resistance is reduced to the steady state shear strength.
- Specimen $\boldsymbol{C}$ exhibits ***limited liquefaction*** → After flow liquefaction is triggered, it first reaches a quasi–steady state of lower shear resistance (***phase transformation point***) and then the Steady State.

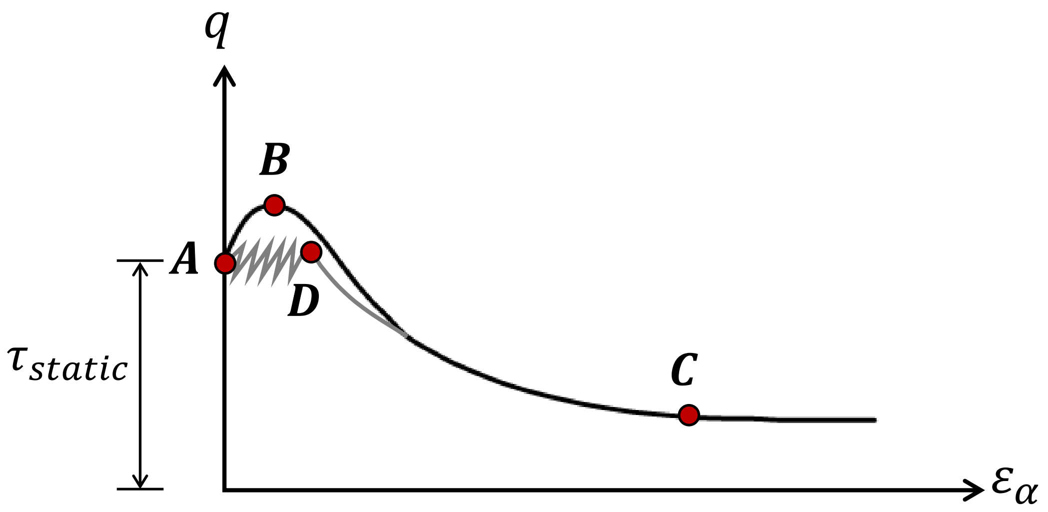


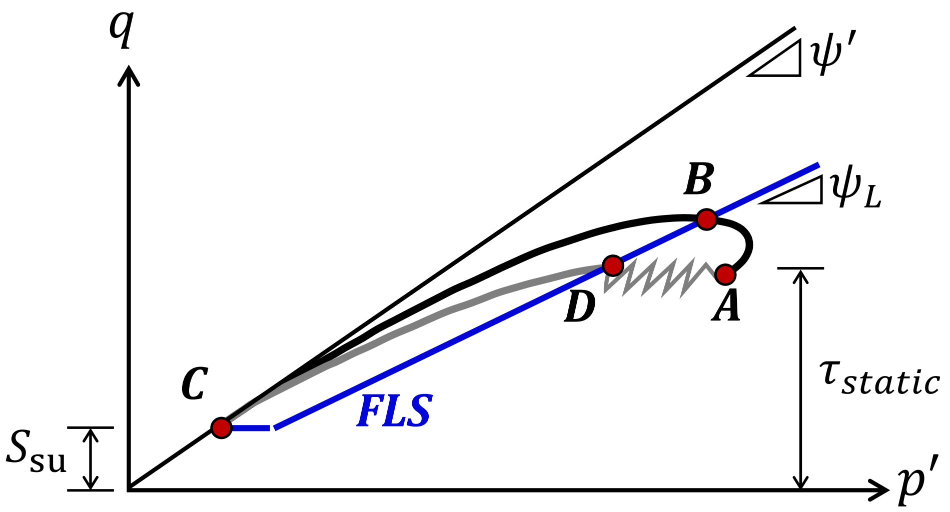
Flow Liquefaction Surface

Monotonic vs. Cyclic Undrained Loading

- ***FLS*** applies to both monotonic and cyclic loading, but *stress conditions are different at the point of flow liquefaction initiation.*
- Let's take a look at the stress paths of 2 identical, anisotropically consolidated, triaxial specimens of loose sand loaded in ***monotonic*** and ***cyclic loading.***

  → Flow liquefaction starts at ($\boldsymbol{B}$) for monotonic loading, but at ($\boldsymbol{D}$) for cyclic loading

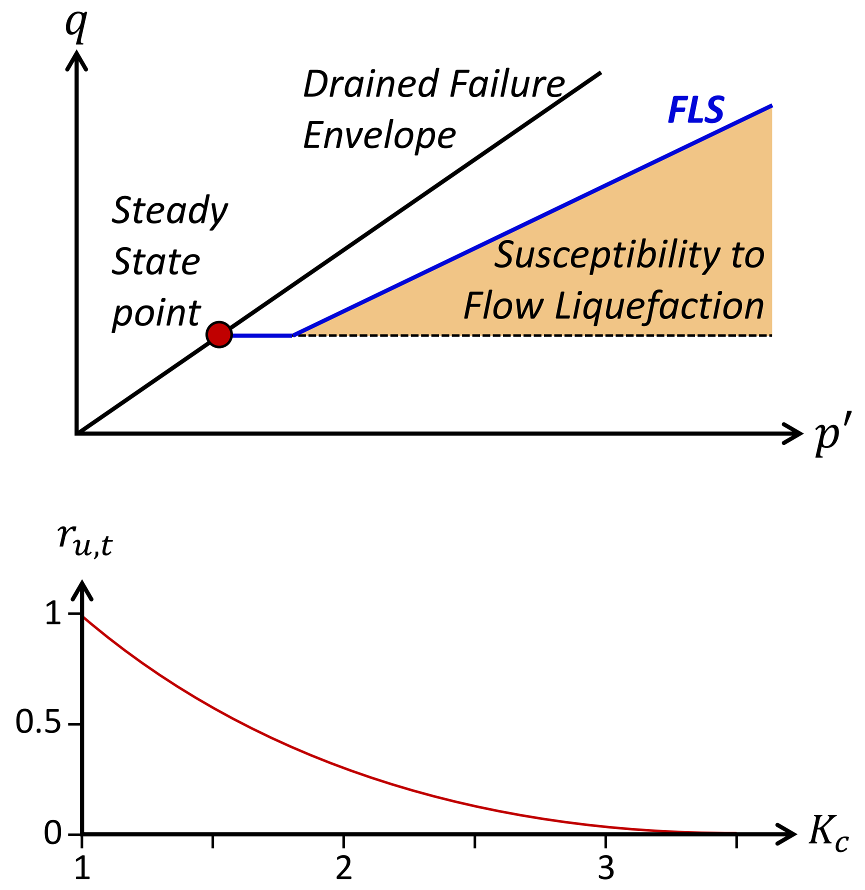




Influence of excess pore pressures

Flow Liquefaction

- Flow liquefaction is triggered when the static shear stresses required for equilibrium are greater than the steady–state strength.
- Initial stresses that plot within the *shaded area* are *susceptible to flow liquefaction.*
- The amplitude of undrained disturbance required to trigger flow liquefaction, i.e. the pore pressure ratio $r_{u,t}$, decreases with the **principal effective ratio $K_c = q/p'$.**
- At high initial K_c, liquefaction may be triggered by very small disturbances $r_{u,t}$.

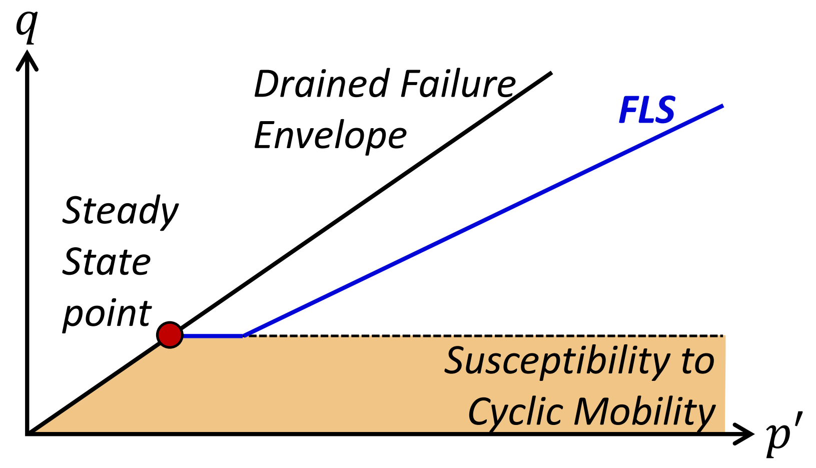


Influence of excess pore pressures

Cyclic Mobility

- Although flow liquefaction cannot be triggered, cyclic mobility can develop when the static shear stress is smaller than the steady-state shear strength.

  → initial stresses that plot within the *shaded area* are *susceptible to cyclic mobility.*



- ***Cyclic mobility can occur for both loose and desne specimens***

  → The shaded area extends from very low to very large confining pressures p', and would plot above and below the SSL

- The definition of failure for cyclic mobility is not precise. Contrary to flow liquefaction, *there is no clear point at which cyclic mobility is initiated.*

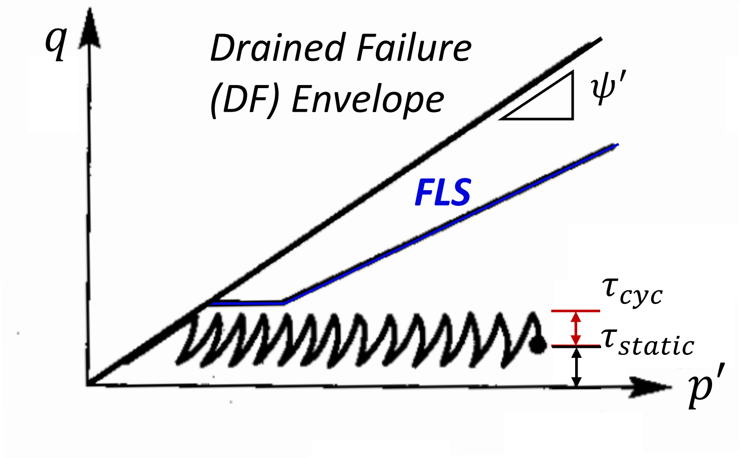
Influence of excess pore pressures

Cyclic Mobility

Three combinations of initial and cyclic loading conditions produce cyclic mobility:

1) No shear stress reversal, no exceedance of S_{su} : $\tau_{static} - \tau_{cyc} > 0, \tau_{static} + \tau_{cyc} < S_{\text{su}}$

- The effective stress path moves to the left, until reaching the *DF Envelope.*
- Additional loading cycles simply lead to up and down movement along the Envelope.
- Flow liquefaction cannot occur, but the reduction of p' allows large permanent strains.

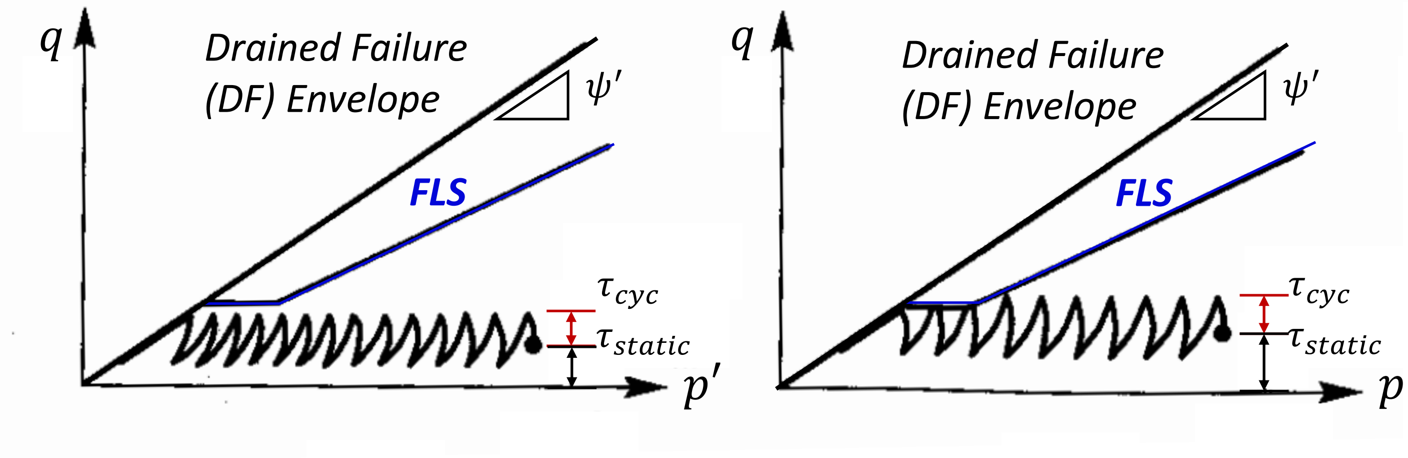


Influence of excess pore pressures

Cyclic Mobility

2) No shear stress reversal, but exceedance of S_{su} : $\tau_{static} - \tau_{cyc} > 0, \tau_{static} + \tau_{cyc} > S_{su}$

- The effective stress path moves to the left, until reaching the ***FLS***.
- When it touches the ***FLS,*** momentary periods of instability will occur.
- Large permanent strains develop, but stability will be reached.

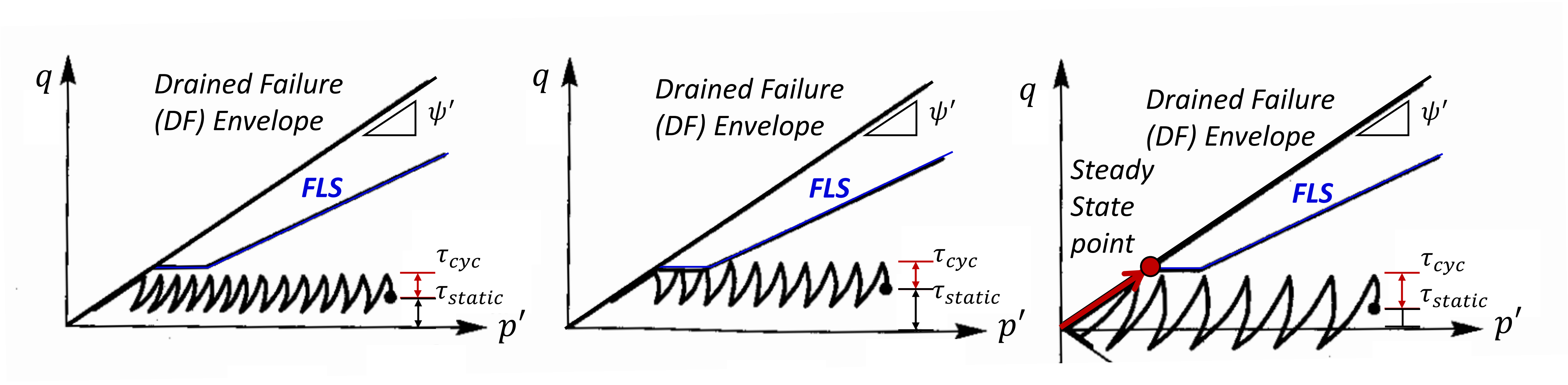


Influence of excess pore pressures

Cyclic Mobility

3) Shear stress reversal, and exceedance of S_{su} : $\tau_{static} - \tau_{cyc} < 0, \tau_{static} + \tau_{cyc} > S_{su}$

- Each time the stress path passes through the origin, $\boldsymbol{r_u = 100\%}$ – ***initial liquefaction***.
- This does not mean that the soil has no shear strength. If loaded ***monotonically***, the soil will dilate until mobilizing the steady state shear strength $\boldsymbol{S_{su}}$.
- Signifficant permanent strains may accumulate, but flow liquefaction cannot occur.



Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential

Cyclic Stress Approach

The fundamental assumption is that pore pressures are generated due to cyclic shear stresses. The method is conceptually simple and requires the following 3 steps:

1) **Characterization of Earthquake Loading:** Calculation of earthquake–induced loading, expressed in terms of *cyclic shear stresses* (amplitude and number of cycles).

2) **Characterization of Liquefaction Resistance:** Calculation of soil liquefaction resistance, also expressed in terms of *cyclic shear stresses*.

3) **Evaluation of Liquefaction potential**, by comparing earthquake loading with liquefaction resistance for the soil deposit of interest.

Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Earthquake Loading

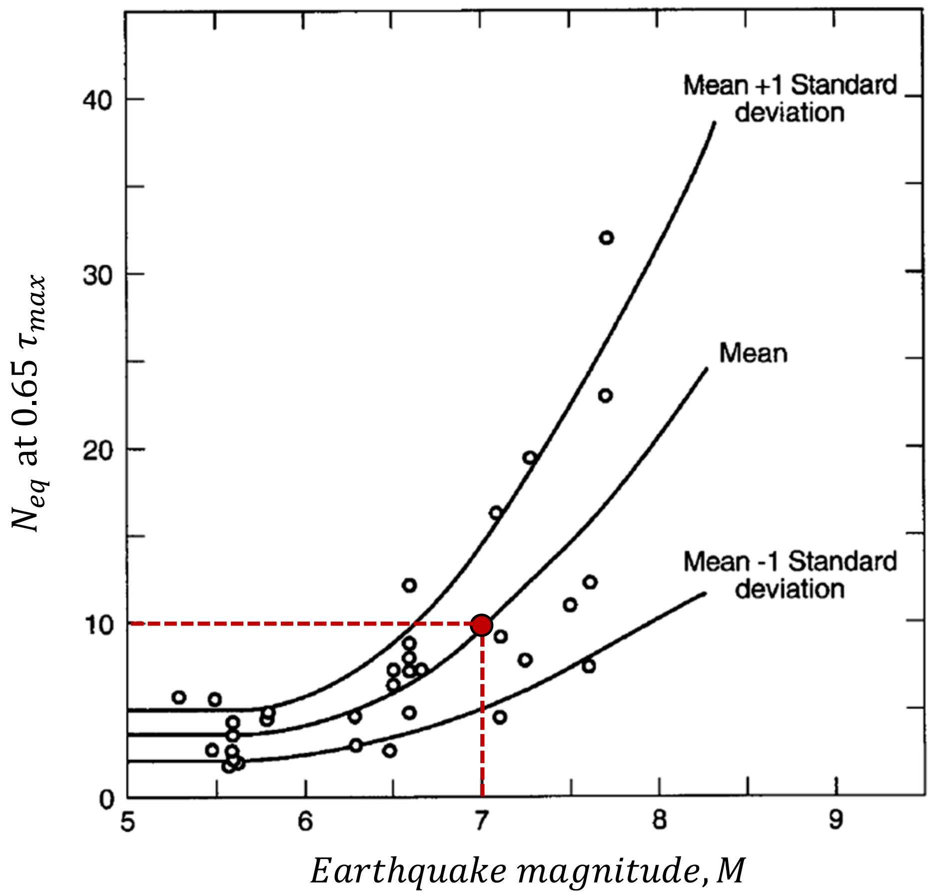
- The level of excess pore water pressure to initiate liquefaction is a function of amplitude and duration of EQ-induced cyclic loading.

- The transient shear stress $\boldsymbol{\tau(t)}$ time history *(computed by ground response analysis),* is converted to an equivalent number of cycles N_{eq}, of uniform cyclic shear stress τ_{cyc} :

$$\tau_{cyc} = 0.65\,\tau_{max}$$

where τ_{max} = maximum EQ shear stress

(Kramer 1996, after Seed et al., 1975)



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Earthquake Loading

- For level (or gently sloping) sites, the simple formula of Seed & Idriss (1971) can be used:

$$\tau_{cyc} = 0.65\,\tau_{max} = 0.65\,\frac{a_{max}}{g}\,\sigma_v r_d$$

where:

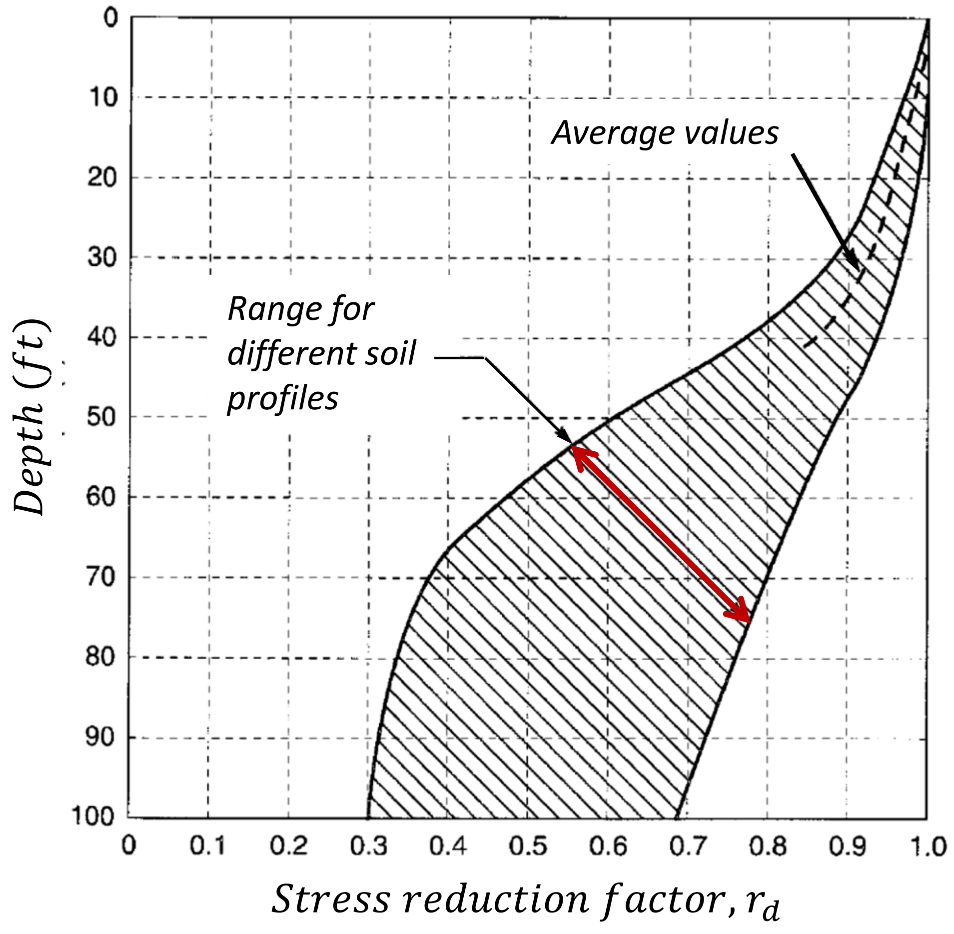
a_{max} = surface peak ground acceleration

g = gravitational acceleration

σ_v = total vertical stress

r_d = stress reduction factor for the depth of interest

(Kramer 1996, after Seed & Idriss, 1971)



Evaluation of Liquefaction Potential

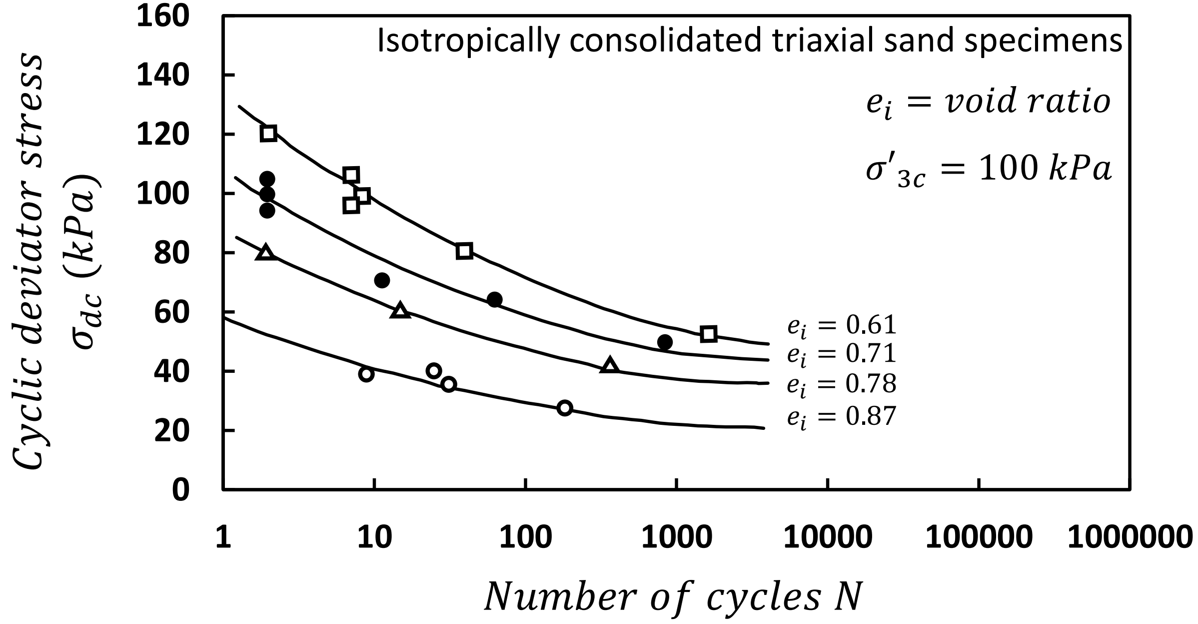
Cyclic Stress Approach

Characterization of Liquefaction Resistance *based on Laboratory Tests*

- The relationship between density (expressed through $\boldsymbol{e_i}$), cyclic stress amplitude $\boldsymbol{\sigma_{dc}}$ and number of cycles $\boldsymbol{N}$ to liquefaction failure is expressed by laboratory-measured ***Cyclic Strength Curves***.

- Cyclic strength curves are usually normalized to the initial effective overburden pressure σ'_{vo} to produce a ***Cyclic Stress Ratio (CSR)***.

(Seed & Lee, 1965)



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Liquefaction Resistance *based on Laboratory Tests*

- The *Cyclic Stress Ratio* ***(CSR)*** is defined differently for different types of tests.
- For *cyclic simple shear* tests as the ratio of cyclic shear strees τ_{cyc} to the initial vertical effective stress σ'_{vo}:

$$(CSR)_{ss} = \frac{\tau_{cyc}}{\sigma'_{vo}}$$

- For *cyclic triaxial tests*, as the ratio of maximum cyclic shear stress $\tau_{cyc} = \sigma_{dc}/2$ to the initial effective confining pressure σ'_{3c}:

$$(CSR)_{tx} = \frac{\tau_{cyc}}{\sigma'_{3c}} = \frac{\sigma_{dc}}{2\sigma'_{3c}}$$

(see Appendix 2 for their relation)

Evaluation of Liquefaction Potential

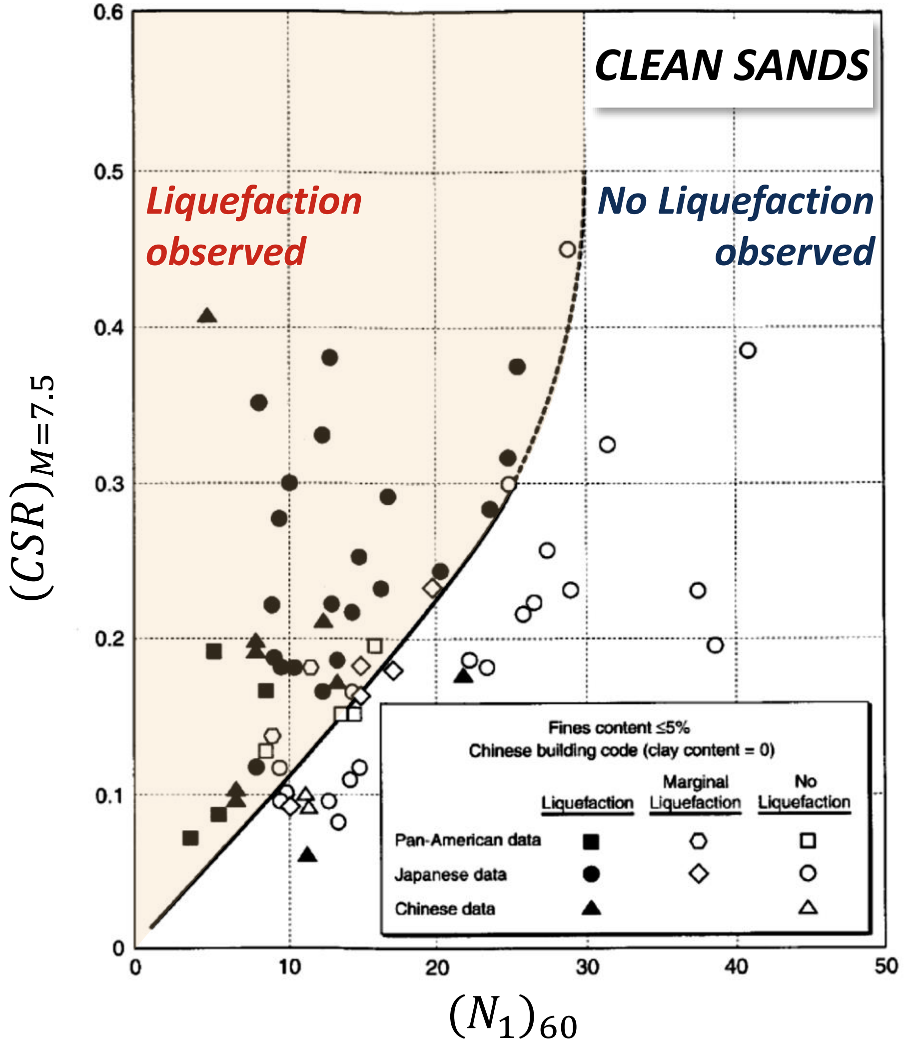
Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Standard Penetration Resistance

- The *Standard Penetration Test (SPT)* is the most commonly used in-situ test for characterization of liquefaction resistance.
- For earthquakes of $M = 7.5$, the $\boldsymbol{CSR}$ is related to the *corrected SPT resistance* $(N_1)_{60}$, to determine the *minimum value of* CSR *for which liquefaction could be expected.*
- The corresponding graph is for ***CLEAN SANDS***.

(Kramer 1996, after Seed et al. 1975)



Evaluation of Liquefaction Potential

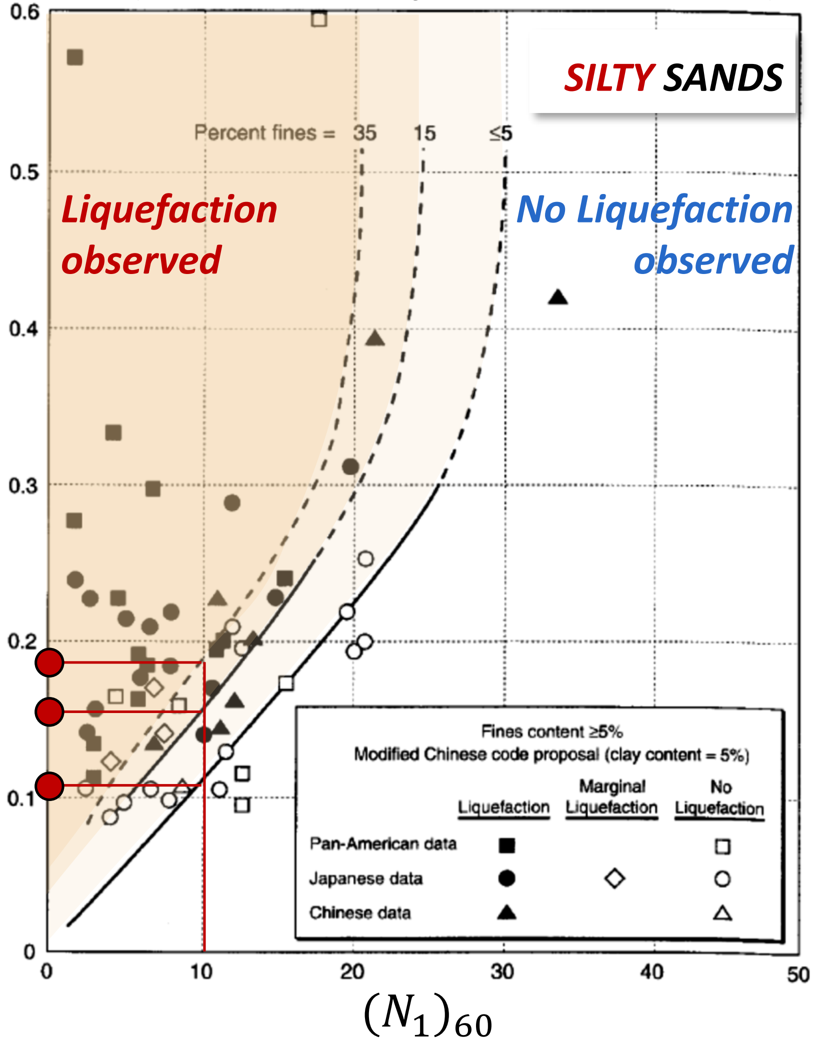
Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Standard Penetration Resistance

- In silty sands, the presence of fines affects CSR only if they comprise at least 5% of the soil.
- For soils of the same $(\boldsymbol{N_1})_{\boldsymbol{60}}$, the increase of fines content leads to increase of $\boldsymbol{CSR}$, which means that the soil becomes less susceptible to liquefaction.
- The plasticity of fines can also affect liquefaction resistance (Appendix 2)

(Kramer 1996, after Seed et al. 1975)



Evaluation of Liquefaction Potential

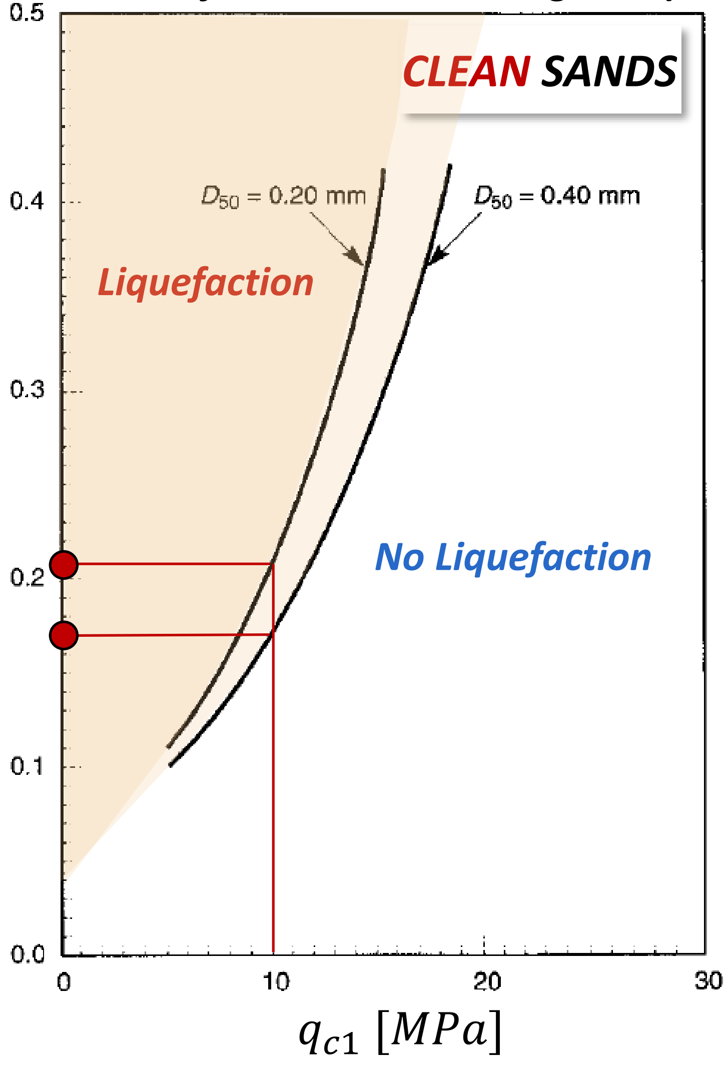
Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Cone Penetration Resistance

- Liquefaction resistance can be assessed based on results from *Cone Penetration Tests (CPT)*.
- More recent liquefaction curves (e.g., Idriss & Boulanger, 2008) rely on more extensive field data where occurrence of liquefaction has been documented.
- The effect of fines can be estimated by increasing the tip resistance up to an equivalent value for clean sand (Ishihara, 1993).

(Kramer 1996, after Mitchell & Tseng 1990)



Evaluation of Liquefaction Potential

(Andrus et al., 2004)

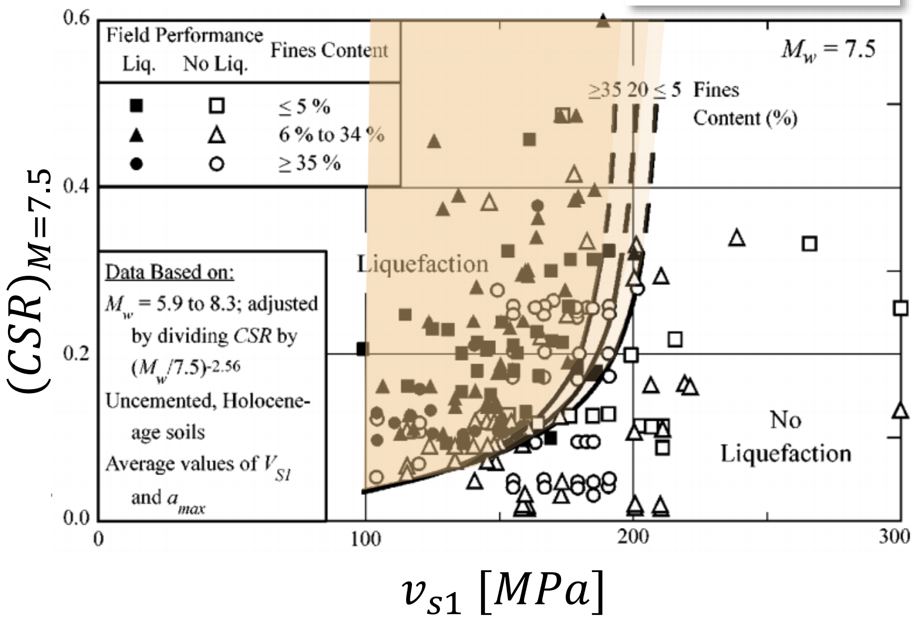
Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Low-strain geophysical tests

- Shear-wave velocity measurements v_s (using crosshole, downhole, SASW) can be used to estimate liquefaction resistance.
- Localized testing of high quality and resolution is required, along with sufficient borings (typical averaged v_s measurements for soil class determination are not adequte)
- The critical layer must be Holocene in age and contain little or no carbonate

SILTY SANDS



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Standard Penetration Resistance

- At sites with sloping ground, or sites that support heavy structures, the presence of ***initial static shear stress*** influences liquefaction resistance.
- If the initial static shear stress exceeds the steady–state shear strength, the initial conditions are closer to the ***FLS*** and liquefaction resistance is reduced.
- On the other hand, liquefaction resistance increases with ***effective confining pressure***.
- To account for these 2 effects, Seed (1983) proposed a modification of $\boldsymbol{CSR}$:

$$(\mathrm{CSR}_{field})_{\alpha\,,\,\sigma} = (\mathrm{CSR}_{field})_{0\,,\,\sigma\,<1\,ton/ft^2}\, K_\alpha\, K_\sigma \quad , \quad a = \tau_{h,static}/\sigma'_{vo}$$

where: K_a, K_σ = correction for initial shear stress & overburden pressure (Appendix 2)

Evaluation of Liquefaction Potential

Cyclic Stress Approach

Evaluation of Liquefaction Initiation

- As soon as the cyclic loading imposed by the earthquake $\boldsymbol{\tau_{cyc}}$ and the liquefaction resistance $\boldsymbol{\tau_{cyc,L}}$ are evaluated, the liquefaction potential can be evaluated:

1) Earthquake–induced loading is expressed as: $\boldsymbol{\tau_{cyc} = 0.65\,\tau_{max}}$

2) Soil Liquefaction resistance is expressed in terms of CSR_L: $\boldsymbol{\tau_{cyc,L} = (CSR_L)\sigma'_{vo}}$

→ $\tau_{cyc,L}$ corresponds to the same Magnitude and equivalent cycles as τ_{cyc}

- We can then define the ***Factor of Safety against liquefaction*** $\boldsymbol{FS_L}$:

$$\boxed{\boldsymbol{FS_L = \frac{CSR_L}{CSR} = \frac{\tau_{cyc.L}}{\tau_{cyc}}}}$$

→ *Liquefaction is triggered when* $\boldsymbol{FS_L \leq 1}$

Evaluation of Liquefaction Potential

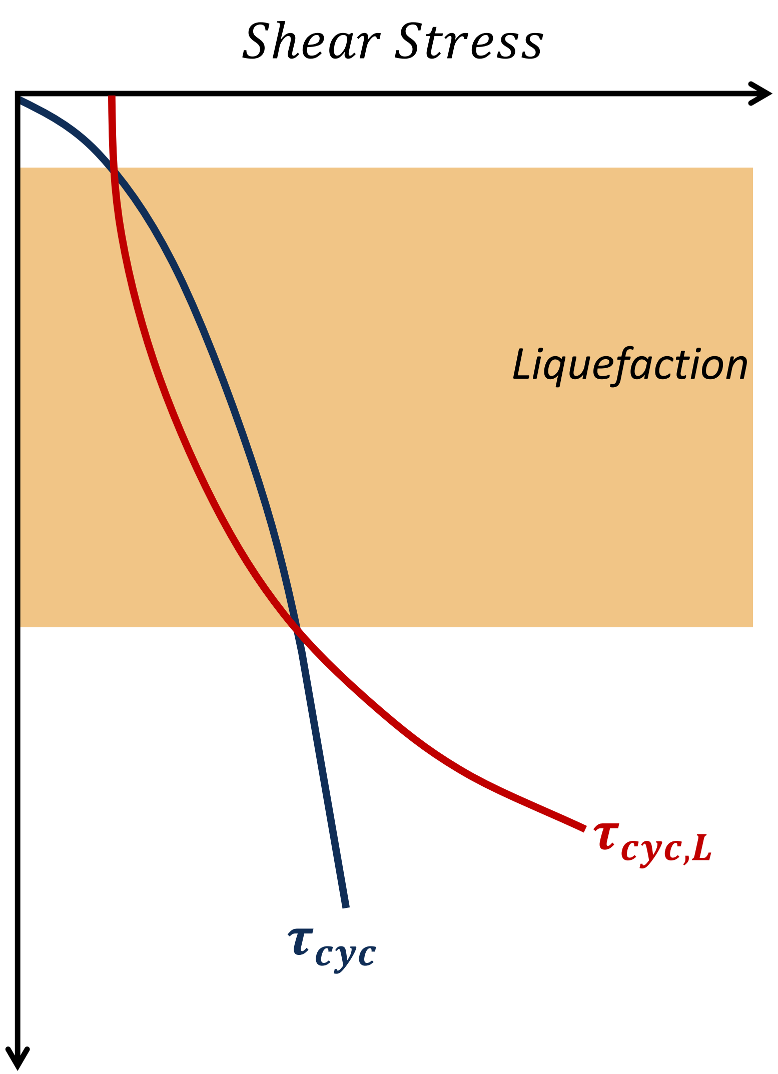
Cyclic Stress Approach

Evaluation of Liquefaction Initiation

- The evaluation can easily be performed graphically.
- Earthquake loading τ_{cyc} and soil liquefaction resistance $\tau_{cyc,L}$ are plotted with depth.

Note: The values need to correspond to the same earthquake magnitude or equivalent number of cycles.

- Liquefaction can be expected in the zone where $\tau_{cyc} \geq \tau_{cyc,L}$



Evaluation of Liquefaction Potential

Other Approaches (see Appendix 2)

- Cyclic Strain Approach (Kramer 9.5.3.2)
- Energy Dissipation Approach (Kramer 9.5.3.3)
- Effective Stress–Based Response Analysis Approach (Kramer 9.5.3.3)
- Probabilistic Approach (Kramer 9.5.3.3)

Liquefaction Effects

Liquefaction Effects

Liquefaction may affect almost any type of infrastructure (buildings, bridges, port facilities, pipelines, embankments, slopes) in many different ways:

- Alteration of ground motion
- Development of sand boils
- Settlement
- Instability
- Flow failures

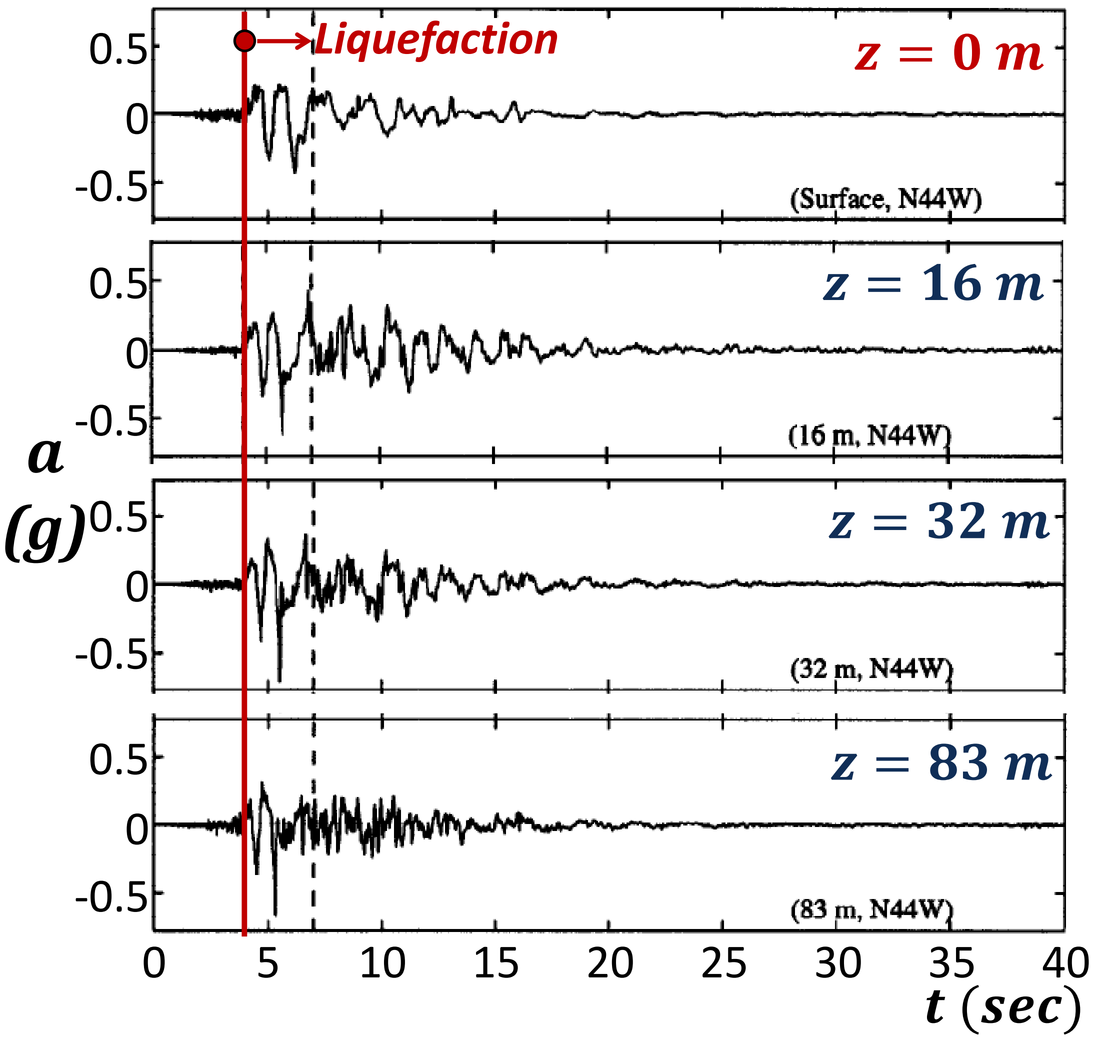
Alteration of Ground Motion

- Generation of excess pore pressures decreases soil stiffness and strength.
- The ***amplitude and frequency*** of the seismic motion are affected (substantial decrease).
- High–frequency components of bedrock motion may not be transmitted through the liquefied layer to reach the surface.

Example 1:

Port Island, Kobe (Japan) → measured accelerations at various depths (1995 EQ).

(Elgamal et al., 1996)

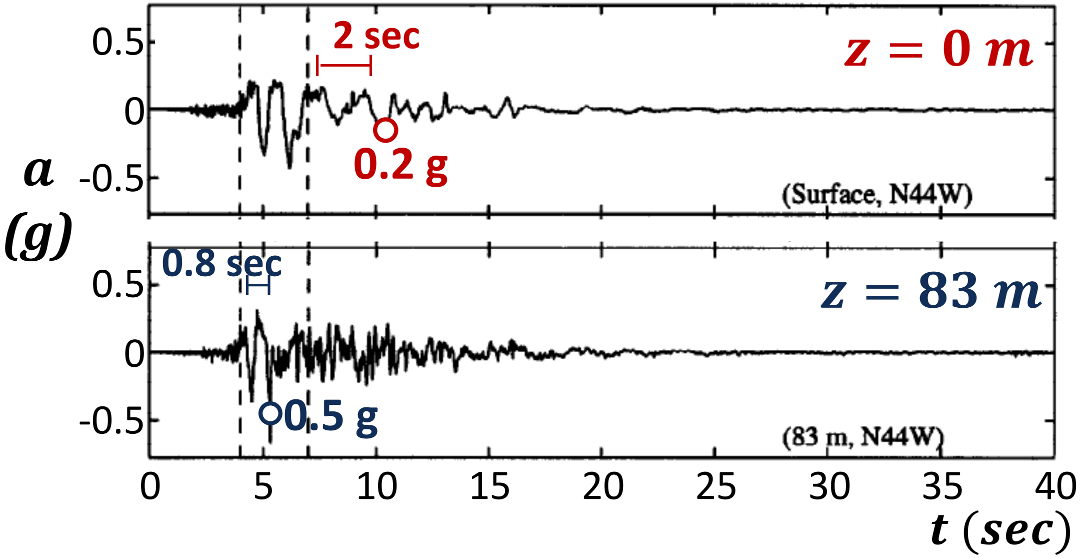


Alteration of Ground Motion

(Elgamal et al., 1996)

- The reduction of acceleration amplitude is accompanied by a substantial decrease of frequency of the ground motion.
- For a *harmonic motion*, the displacement amplitude is a function of frequency $\boldsymbol{f}$:

$$u_{max} = \frac{a_{max}}{2\pi f^2}$$



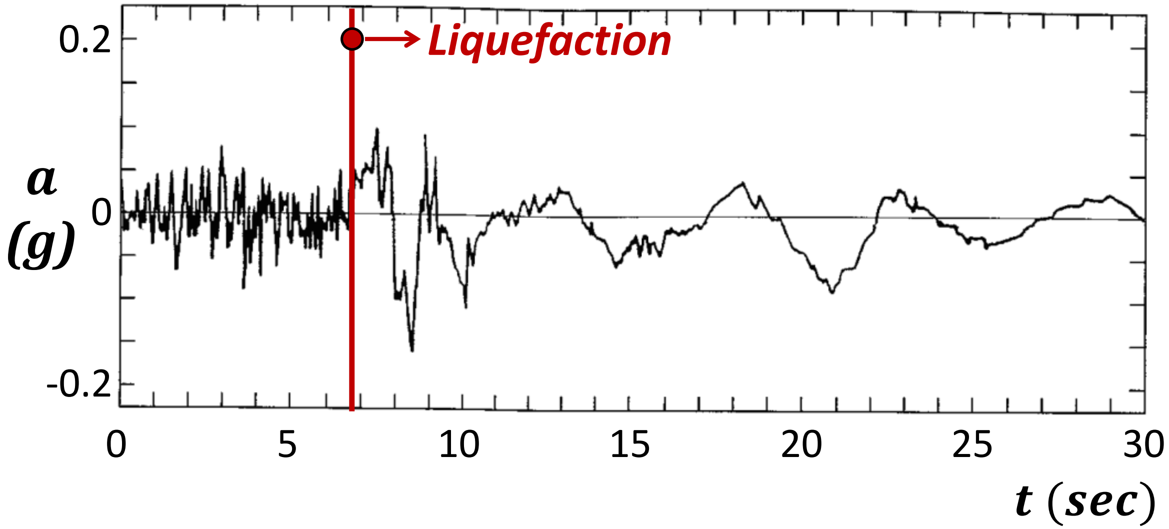
- At $\boldsymbol{z = 83\ m}$, below the liquefied layer, $a_{max} \approx$ 0.5 g = 5 m/sec^2 (ignoring the high-frequency spike) and the dominant frequency $f \approx$ 1.25 Hz ≈ 1/0.8 sec.
  → With these very crude assumptions $\boldsymbol{u_{max}} \approx$ **0.5 m**
- At the ***ground surface*** $\boldsymbol{z = 0\ m}$, above the liquefied layer, $a_{max} \approx$ 0.2 g = 2 m/sec^2 (after liquefaction), but the dominant frequency $f \approx$ 0.5 Hz ≈ 1/2 sec.
  → With these very crude assumptions $\boldsymbol{u_{max}} \approx$ **1.3 m**

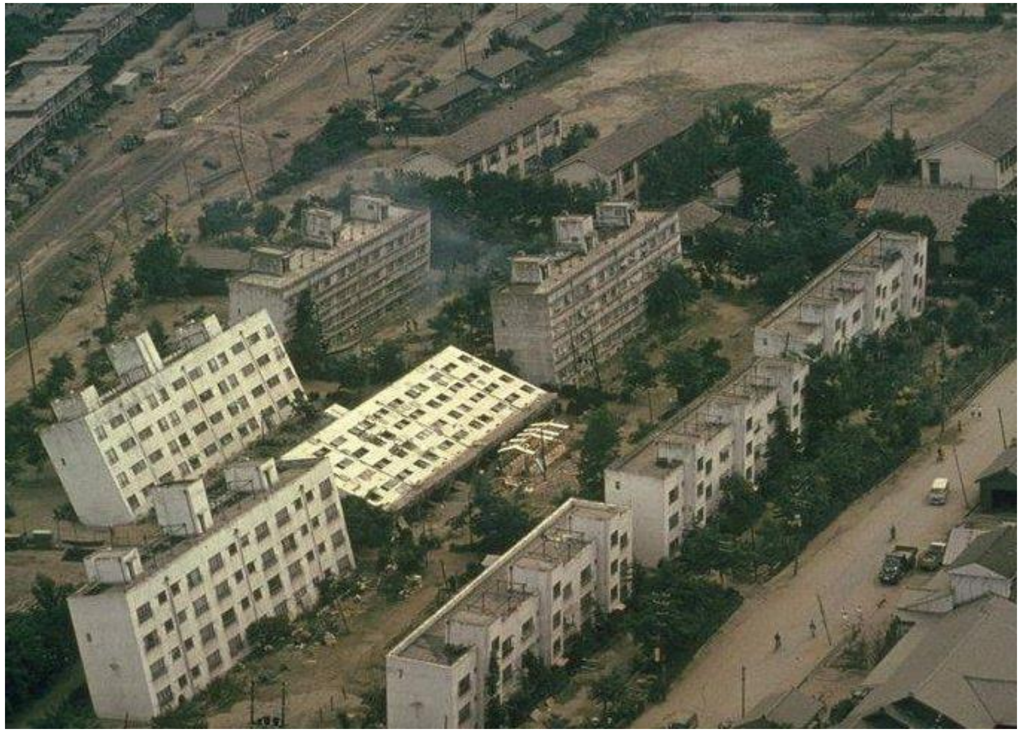
Alteration of Ground Motion

Example 2:

1964 Niigata (Japan) earthquate → measured accelerations near overturned buildings

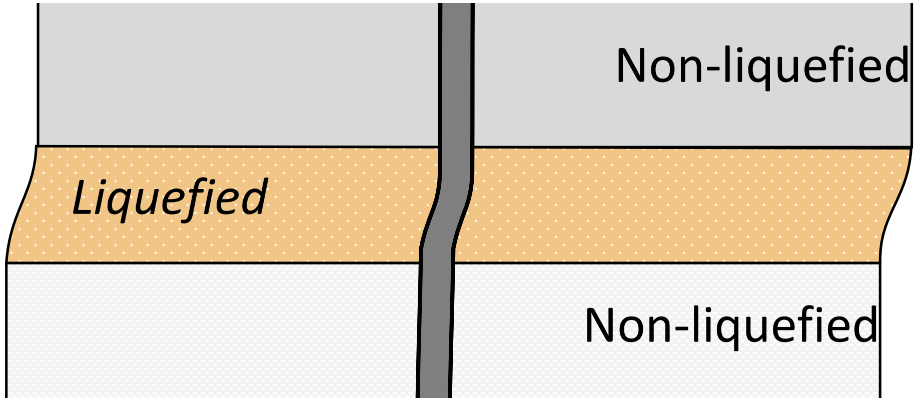
- The decrease of acceleration amplitude does not necessarily lead to reduction of damage potential.
- Notice the elongation of period for ***t* > 7 s**.

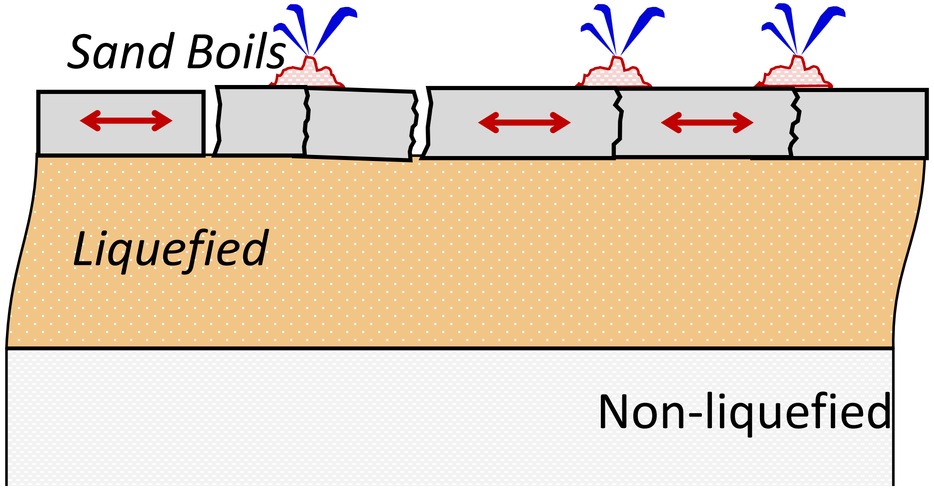




Alteration of Ground Motion

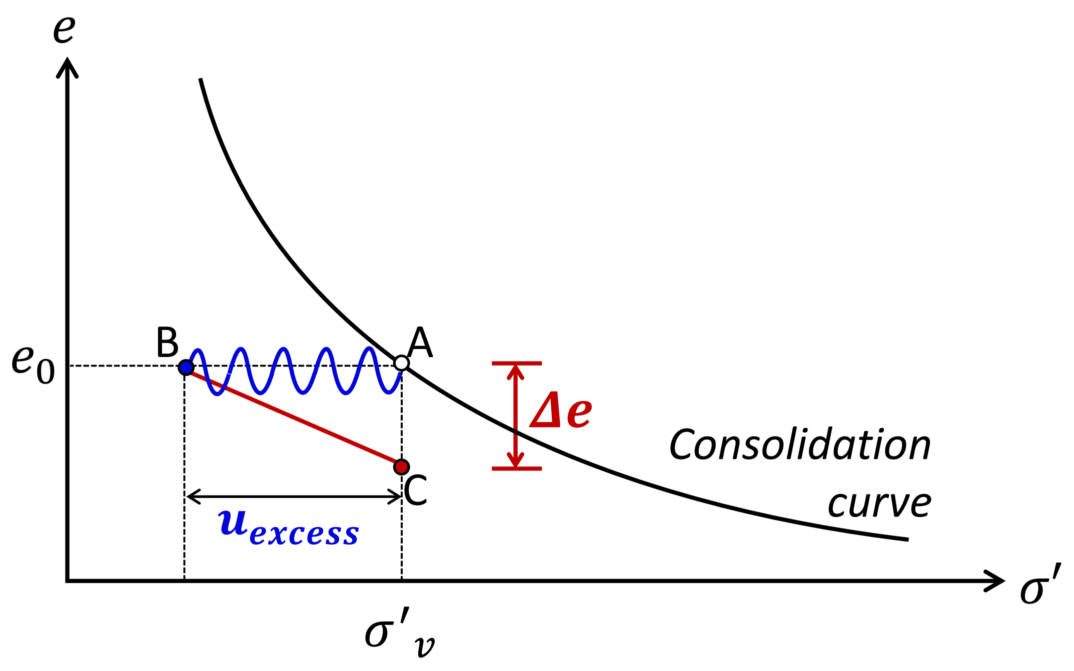
- The increase of displacement amplitude u_{max} of the liquefied layer may lead to ***extensive damage*** of structures above or within the liquefied layer, such as piles and pipelines.
- Liquefaction beneath a flat surface may cause detachment of the liquefied soil from the surficial soils, leading to excessive ***ground oscillations***.
- The surficial soil may ***break into blocks*** with fissures that open & close during the earthquake, but limited permanent displacement.





Settlement

- Loose sand has a tendency to densify when subjected to seismic loading. Such subsurface densification is manifested as settlement at the ground surface.
- Large liquefaction–induced settlements can cause ***substantial damage*** to *shallow foundations, piles, and buried structures*.
- EQ cyclic loading leads to development of excess pore water pressures $\boldsymbol{u_{excess}}$ and reduction of effective stress $\boldsymbol{\sigma'_v}$ (AB).
- After the end of shaking, $\boldsymbol{u_{excess}}$ dissipate (BC), leading to an increase of volumetric strain $\boldsymbol{\Delta e}$, resulting to *ground surface settlements.*

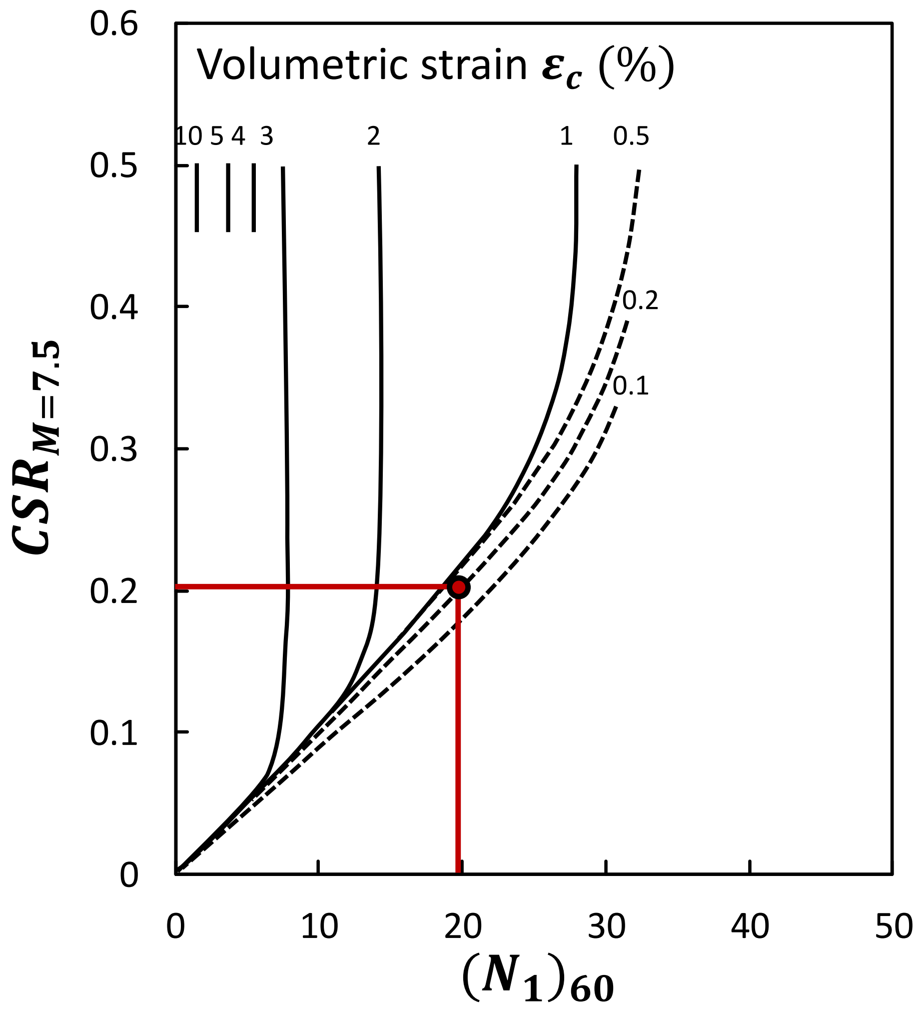


Settlement

(Tokimatsu & Seed, 1987)

Tokimatsu & Seed Method

- Post–earthquake settlement can be estimated on the basis of volumetric strain ε_c.
- The latter can be estimated from an empirical diagram on the basis of the corrected $(N_1)_{60}$ and the $CSR_{M=7.5}$ (for *M = 7.5 earthquakes*).
- CSR is computed as previously discussed, applying correction factors for fines content and earthquake magnitude ***M***.



Example:

$CSR_{M=7.5}$ = 0.2 , $(N_1)_{60}$ = 20 → ε_c **= 0.2%**

Instability

- Liquefaction–induced instabilities can be particularly damaging to infrastructure.
- They have been observed in numerous earthquakes and may be in the form of:
  - ✓ Flow slides
  - ✓ Lateral spreads
  - ✓ Retaining wall failures
  - ✓ Foundation failures
- Instability is triggered when:

  Shear stress required for equilibrium **>** *Soil shear strength*

- In such a case, the soil will deform until reaching a configuration for which equilibrium can be attained.

Instability

- The amount of deformation is a function of the ***difference*** between the shear stress required for equilibrium and the soil shear strength

  → If there is only a slight difference, permanent deformations are likely to be small.

- Realistic evaluation of the effects of liquefaction-induced instability requires accurate estimation of the ***shear strength of the liquefied soil***.

- Three approaches are available for this purpose:

  1) Laboratory testing approach
  2) In-situ testing approach
  3) Normalized strength approach

→ All three methods have considerable uncertainties: at least 2 should be used

Instability

Laboratory testing approach

- The method uses the previously discussed steady-state shear strength S_{su}. The latter is extremely sensitive to density, and therefore very careful sampling is necessary.
- Unfortunately perfect sampling (transportation, and handling) is impossible and the lab measured S_{su} has to be corrected to match the in-situ state, something which can be very tricky.

In–situ testing approach (Appendix 3)

- Developed by Seed (1986), the method correlates the residual undrained shear strength to an equivalent clean-sand SPT.

Instability

Flow failures

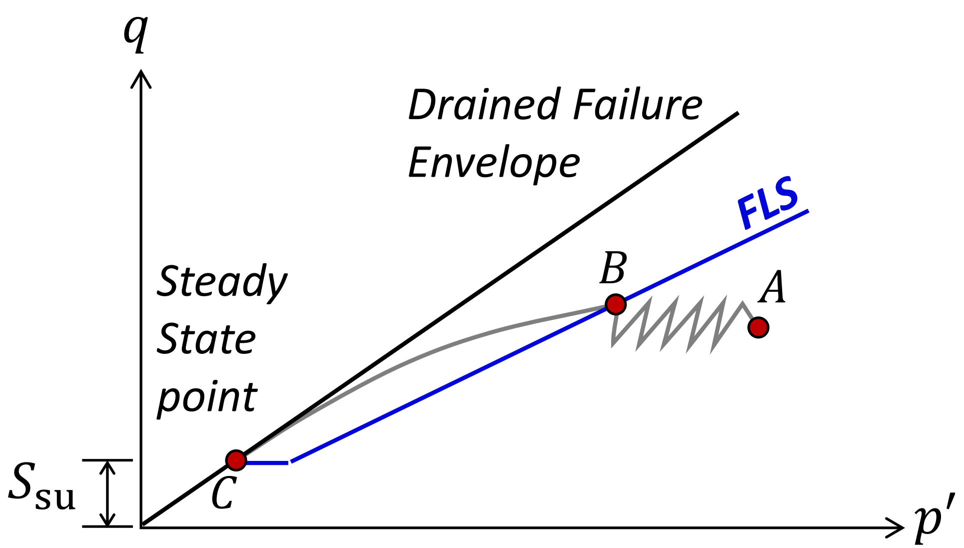
Occur when the shear stresses required for static equilibrium exceed the shear strength of the liquefied soil.

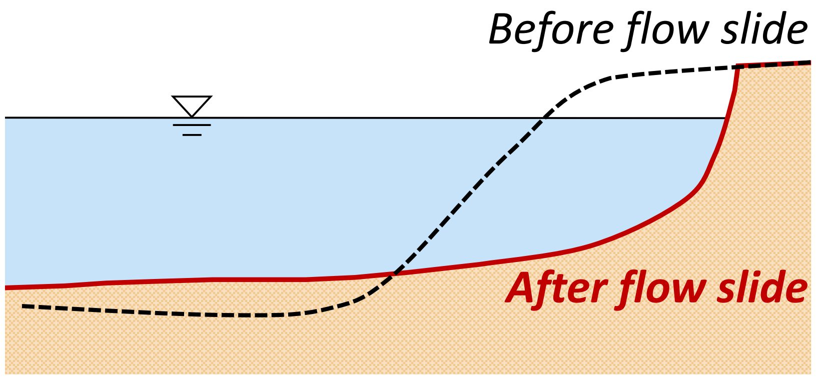
- ***Four different mechanisms*** can be identified (National Research Council, 1985):
  1) Flow Liquefaction Failures – NRC Mechanism A
  2) Local Loosening Flow Failure – NRC Mechanism B
  3) Global Loosening Flow Failure – NRC Mechanism C (Appendix 3)
  4) Interface Flow Failure – NRC Mechanism D (Appendix 3)

Instability

Flow Liquefaction Failures (NRC Mechanism A)

- Occur under *totally undrained conditions* (no ε_v).
- Due to excess pore pressure generation, a soil element "moves" from initial static equilibrium position (**A**) to the ***FLS*** (**B**).
- The soil element becomes unstable and flow liquefaction is triggered.
- The shearing resistance drops to steady–state strength $\boldsymbol{S_{su}}$ (**C**).
- Flow liquefaction failures occur very quickly and produce ***large soil movements***.





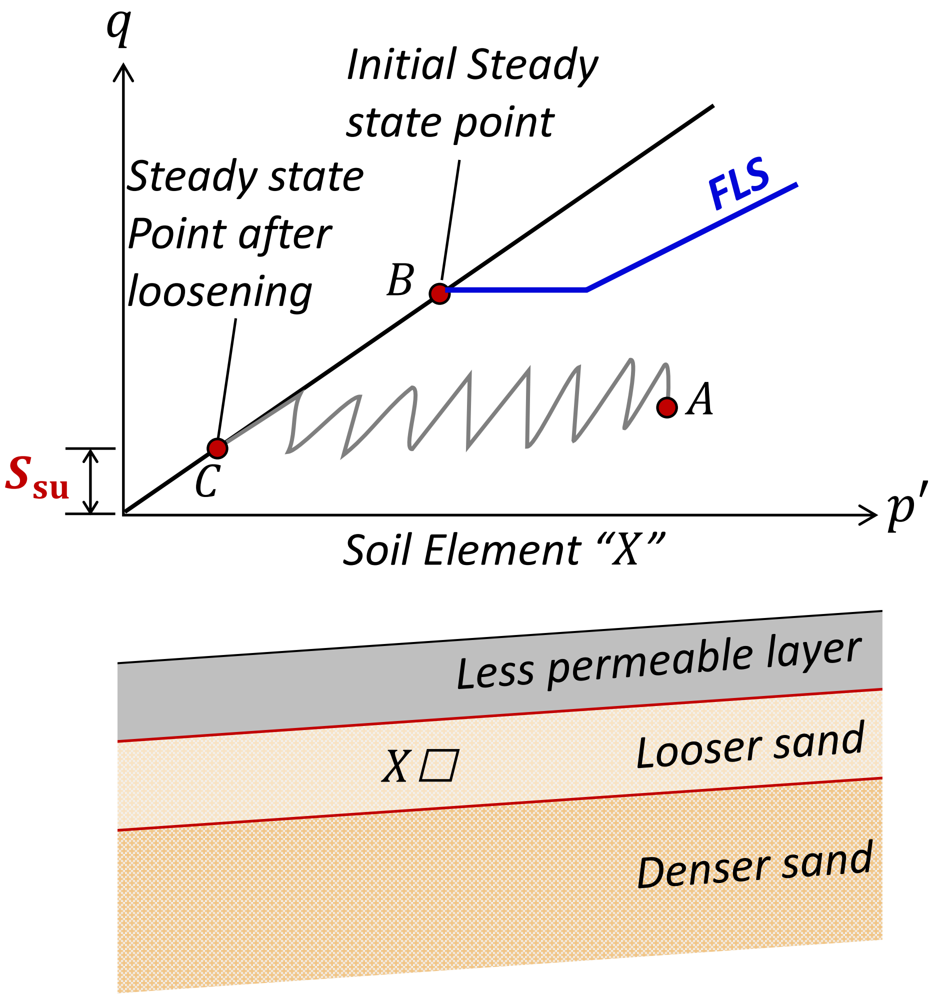
Instability

Local Loosening Flow Failure (NRC Mechanism B)

- If a sand layer is overlain by a less permeable stratum, the total volume of sand will remain constant during the earthquake (no drainage).
- If initial liquefaction is reached, there may be *a rearrangement of grains due to gravity*.
- The sand layer will become looser at the top and denser at the bottom

  → Due to the reduction of density (top), the *steady–state strength* S_{su} is reduced

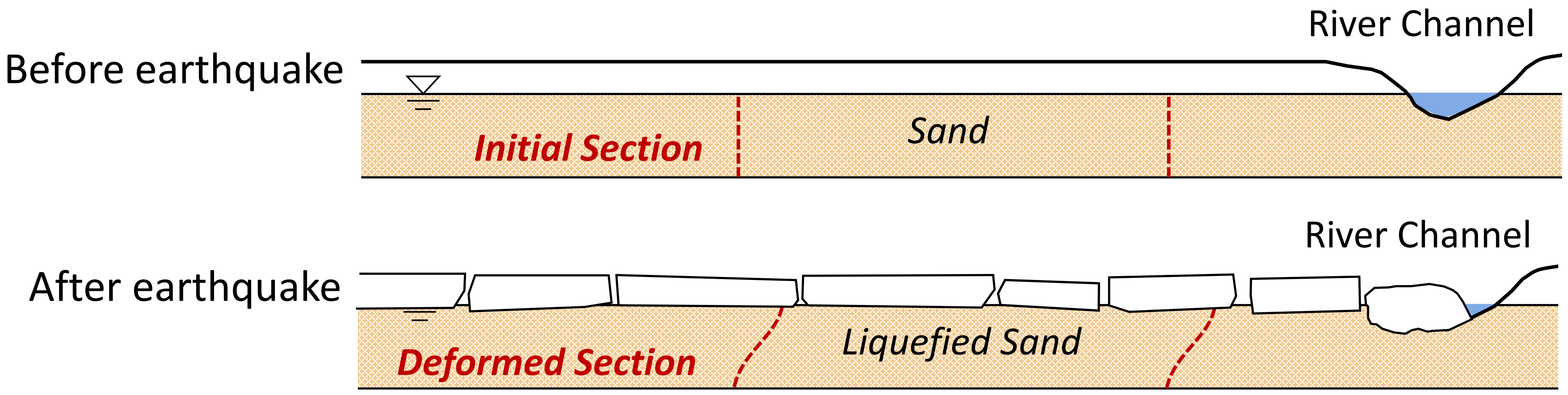
  → If S_{su} ***after loosening*** **< static shear stress**, *flow failure will be initiated.*



Instability

Deformation failures

- Cyclic mobility can produce small, incremental, permanent deformation, that by the end of the earthquake may be sufficient to lead to substantial structural damage.
- ***Lateral spreading*** is a typical example of deformation failure.
- Lateral deformations may range from few centimeters to 2 m, but they may be even larger is the seismic motion is very long and/or strong (e.g., Kobe Port).



Thank you for your attention!

ETH zürich

IGT Professur für Geotechnik

Cyprus Association of Civil Engineers

Thank you for your attention!

Prof. Dr. Ioannis Anastasopoulos

Appendix 1
Liquefaction susceptibility

Compositional Criteria

- ***Non-uniformly graded*** soils are less sensitive to liquefaction.
- Soils with rounded particles are considered more liquefiable than with angular ones.
- Clays are not susceptible to liquefaction (but sensitive clays may exhibit substantial strain softening).
- **Fine-grained soils**, that satisfy the following **4 Chinese criteria** (Wang, 1979) may be considered susceptible to signifficant strength loss:

1) Fraction finer than 0.005 mm ≤ 15%

2) LL (Liquid Limit) ≤ 35%

3) Natural Water content ≥ 0.9 LL

4) Liquidity index ≤ 0.75

State Criteria

State Parameter ψ

- A soil of specific void ratio e, may or may not be liquefiable, depending on σ'_{3c}.
- To avoid using absolute measures (such as density or e), the **state parameter ψ** (Been & Jeffries, 1985) is introduced:

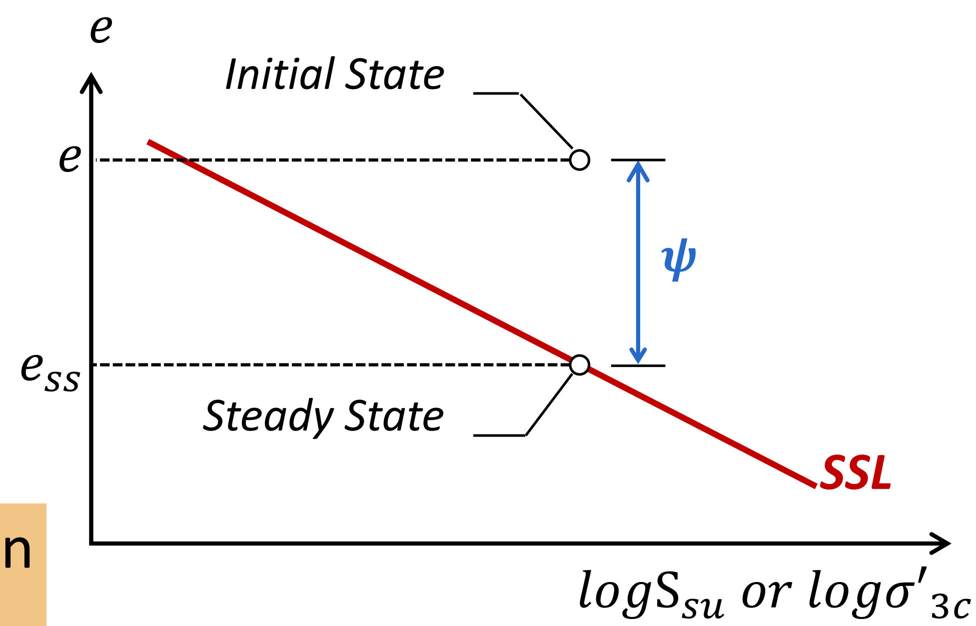
$$\boxed{\psi = e - e_{ss}}$$

where:

e = void ratio at initial state of interest

e_{ss} = void ratio of the steady state line, at the same σ'_{3c}

If $\psi > 0$: Soil susceptible to flow liquefaction
If $\psi < 0$: Soil non–susceptible



Appendix 2
Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Liquefaction Resistance *based on Laboratory Tests*

- The two *CSR*s are usually related through a correction factor $\boldsymbol{c_r}$:

$$\boxed{(CSR)_{ss} = c_r (CSR)_{tx}}$$

- The correction factor $\boldsymbol{c_r}$ can be estimated on the basis of the following Table:

		$\boldsymbol{c_r}$ **for:**	
Reference	**Equation**	$\boldsymbol{K_o = 0.4}$	$\boldsymbol{K_o = 1.0}$
Finn et al. (1971)	$c_r = (1 + K_o)/2$	0.7	1.0
Seed and Peacock (1971)	Varies	0.55 – 0.72	1.0
Castro (1975)	$c_r = 2(1 + 2K_o)/3\sqrt{3}$	0.69	1.15

Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Liquefaction Resistance *based on Laboratory Tests*

- The laboratory-derived *CSR* is based on unidirectional loading.
- But real earthquakes actually produce ***multi-directional*** shear loading, which has been proven to lead to faster build up of pore pressures, and hence easier triggering of liquefaction.
- Seed et al (1975) showed that the CSR required in the field to produce initial liquefaction is roughly 10% less than the one for unidirectional simple shear testing.
- The liquefaction resistance of a soil element in the field is thus (Seed et al., 1975b):

$$\boxed{(\mathrm{CSR})_{field} = 0.90\,(CSR)_{ss} = 0.90\,c_r(CSR)_{tx}}$$

Evaluation of Liquefaction Potential

Cyclic Stress Approach

Characterization of Liquefaction Resistance *based on Laboratory Tests*

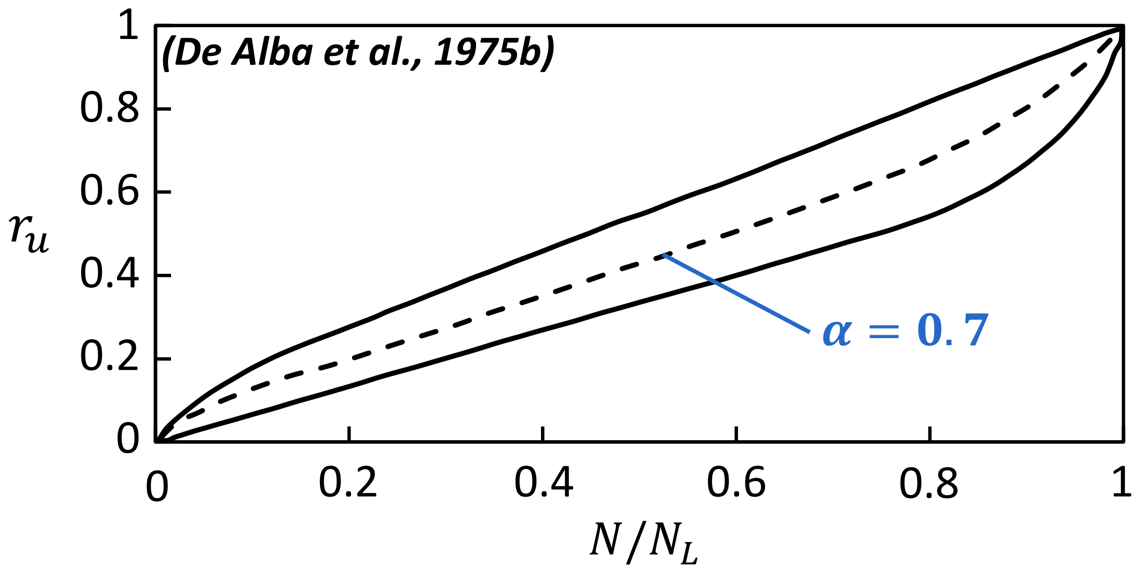
- Laboratory tests can also offer information on the development of excess pore water pressures. According to Lee & Albaisa (1974) and DeAlba et al. (1975b), the ratio of excess pore pressures $\boldsymbol{r_u}$ is related to the number of loading cycles $\boldsymbol{N}$ as follows:

$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_L}\right)^{1/\alpha} - 1\right]$$

where:

N_L = number of cycles required for initial liquefaction

a = parameter related to soil properties and test conditions



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Liquefaction Resistance based on *in-situ Tests*

Standard Penetration Resistance

- The plasticity of fines can affect liquefaction resistance.
- Adhesion tends to reduce relative movement of soil particles, thus reducing the generation of excess pore water pressures.
- The effect of fines' plasticity in silty sands can be accounted for by multiplying $\boldsymbol{CSR}$ with the $\boldsymbol{F}$ factor (Ishihara, 1993):

$$F = \begin{cases} 1.0 & PI \leq 10 \\ 1.0 + 0.022(PI - 10) & PI > 10 \end{cases}$$

- F is multiplied by a correction factor to account for $\boldsymbol{M}$.

Magnitude Correction Factors

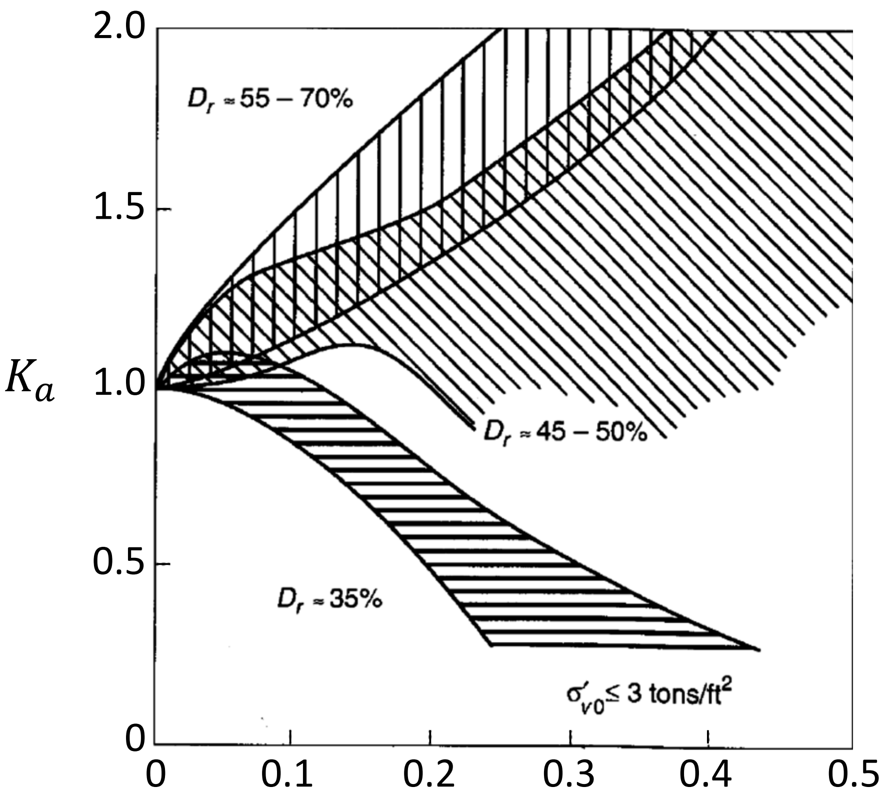
$\boldsymbol{Magnitude}$ $\boldsymbol{M}$	$\dfrac{\boldsymbol{CSR_M}}{\boldsymbol{CSR_{M=7.5}}}$
$5\frac{1}{4}$	1.50
6	1.32
$6\frac{3}{4}$	1.13
7.5	1.00
8.5	0.89

Evaluation of Liquefaction Potential

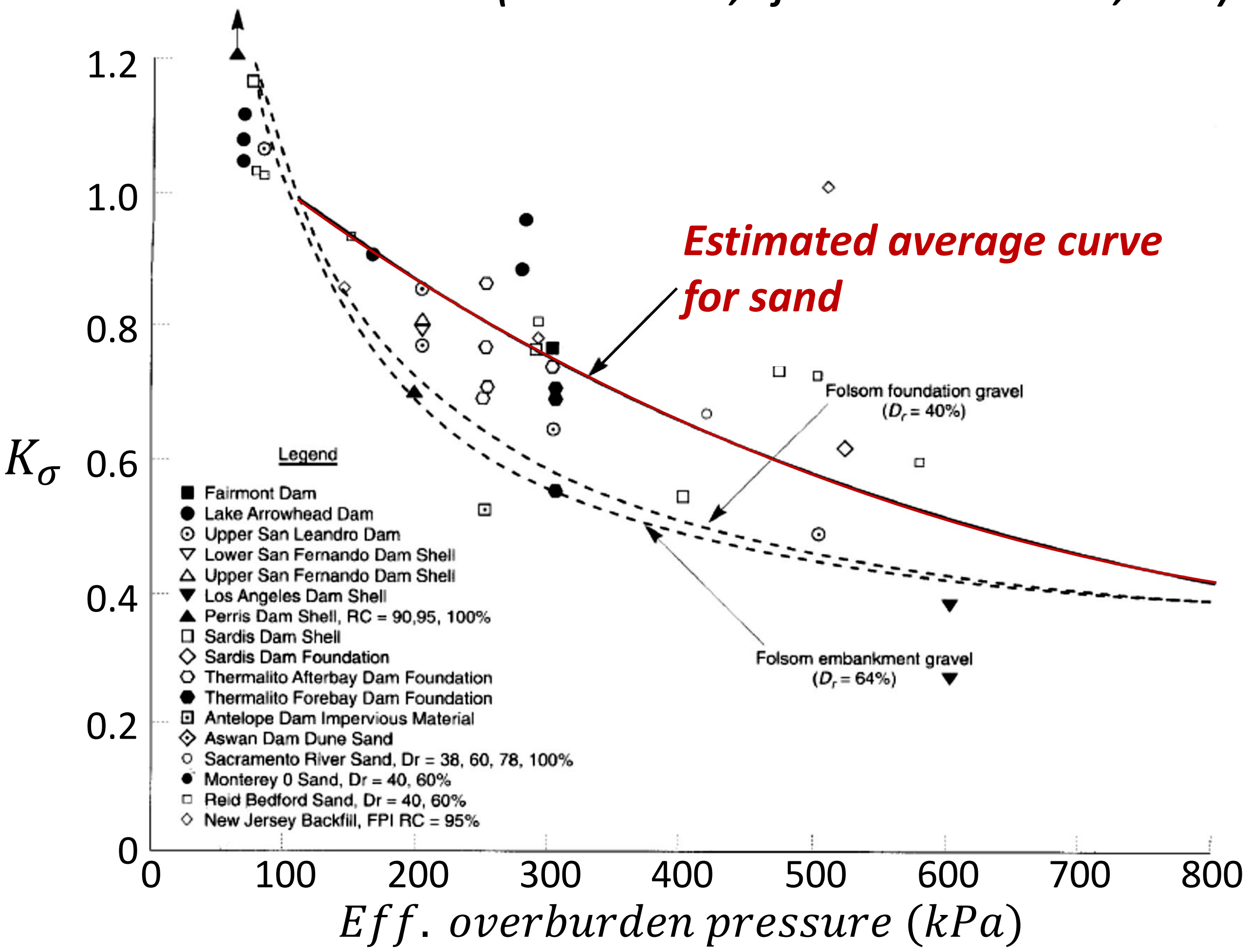
Cyclic Stress Approach

Liquefaction Resistance, *in-situ Tests*

(Kramer 1996, after Seed & Harder, 1990)



(Kramer 1996, after Marcuson et al., 1990)



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Liquefaction Resistance, *in-situ Tests*

Cone Penetration Resistance

- The Cone Penetration Test (***CPT***) provides continuous measurements of soil resistance and can detect thin layers of potentially liquefiable soil – *a clear advantage over SPT*.

- In CPT-based liquefaction evaluations, the tip resistance $\boldsymbol{q_c}$ is normalized to a standard effective overburden pressure of $p_a = 1\ ton/ft^2$ (96 kPa), by:

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

where: σ'_{vo} = vertical effective pressure at the depth of interest ($tons/ft^2$)

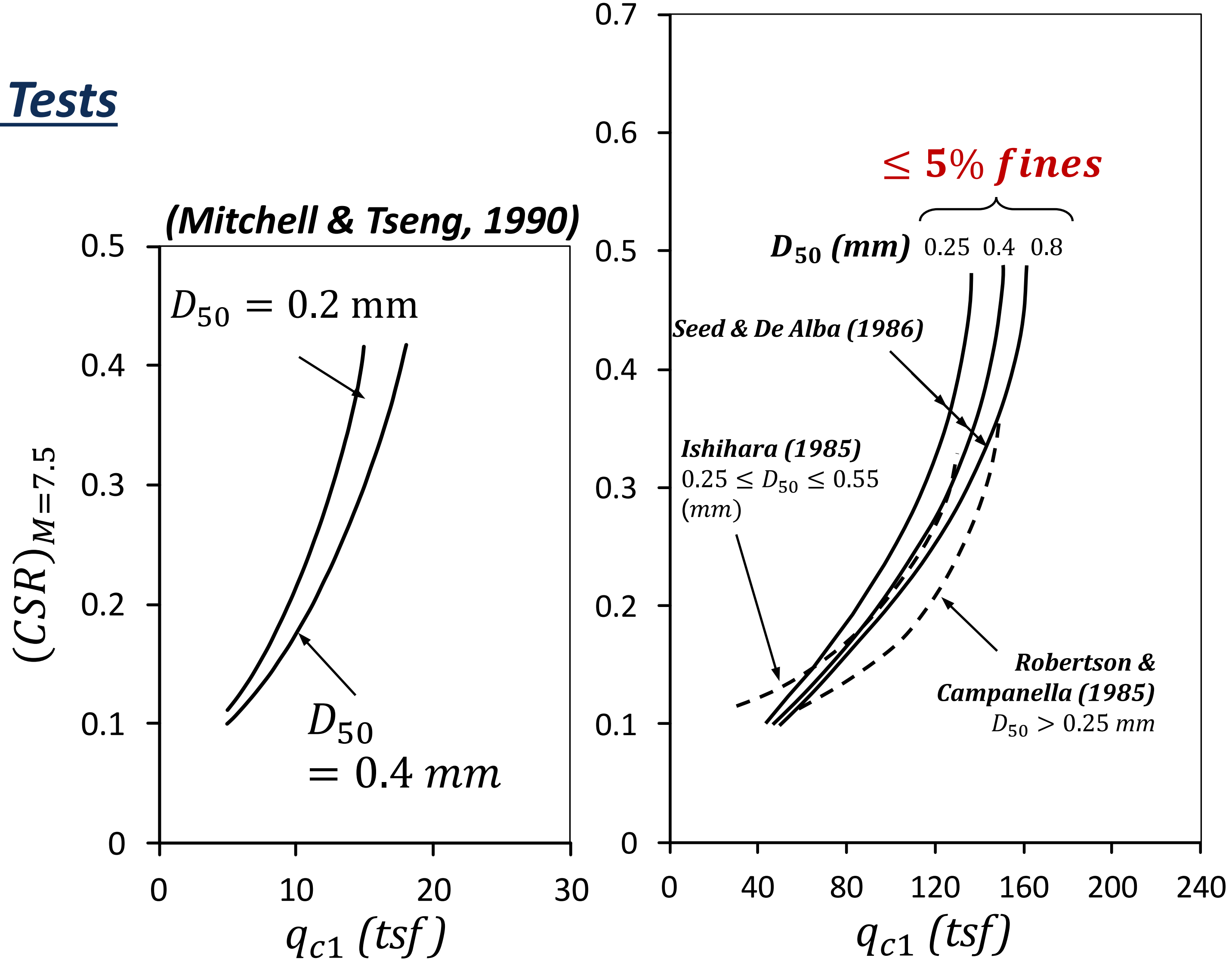
Evaluation of Liquefaction Potential

Cyclic Stress Approach

Liquefaction Resistance, *in-situ Tests*

Cone Penetration Resistance

- CPT–based liquefaction resistance curves have been derived for ***clean sands*** of *various grain sizes*.
- Note that the normalized cone resistance $\boldsymbol{q_{c1}}$ is expressed in tsf (= tons per square foot) on both. graphs.



Evaluation of Liquefaction Potential

Cyclic Stress Approach

Liquefaction Resistance, *in-situ Tests*

Cone Penetration Resistance

- For silty sands with fines content $\geq 5\%$, the effects of fines can be estimated by adding the tip resistance increments of the Table to the measured tip resistance $\boldsymbol{q_c}$, to obtain an equivalent clean sand tip resistance.
- For earthquakes of magnitude $\boldsymbol{M \neq 7.5}$, the previously discussed Magnitude correction factors can be used to derive $(\boldsymbol{CSR})_{M\neq 7.5}$.

(Ishihara, 1993)

$\boldsymbol{Fines\ Content}$ (%)	$\boldsymbol{Tip\ Resistance\ Increment\ (tons/ft^2)}$
≤ 5	0
~ 10	12
~ 15	22
~ 35	40

Evaluation of Liquefaction Potential

Other Approaches

Other approaches have been proposed and are briefly mentioned here.

Cyclic Strain Approach (Kramer 9.5.3.2)

- The generation of excess pore pressures is directly related to cyclic strain amplitude.
- Since generation of excess pore pressures is the main characteristic of liquefaction, *cyclic strains* instead of *stresses* are used to evaluate liquefaction potential.
- The method is not being used as much as the Cyclic Stresses Approach, because accurate prediction of cyclic strains is not such an easy task in reality.

Energy Dissipation Approach (Kramer 9.5.3.3)

- Dissipated energy is used as a measure of liquefaction resistance.
- Energy content of a seismic ground motion is used as the loading to compare.

Evaluation of Liquefaction Potential

Other Approaches

Effective Stress–Based Response Analysis Approach (Kramer 9.5.3.3)

- Cyclic, nonlinear stress–strain models, pore pressure models, or advanced constitutive models are used.
- Such models can be incorporated into nonlinear dynamic ground response analyses to predict the development of excess pore pressures and their redistribution before and after seismic loading.

Probabilistic Approach (Kramer 9.5.3.3)

- Uses measurements of laboratory tests or field observations, to assess uncertainty of parameters that influence liquefaction.
- Statistical classifications and regression analyses are used to evaluate liquefaction probability under specific loading conditions.

Appendix 3
Liquefaction Effects

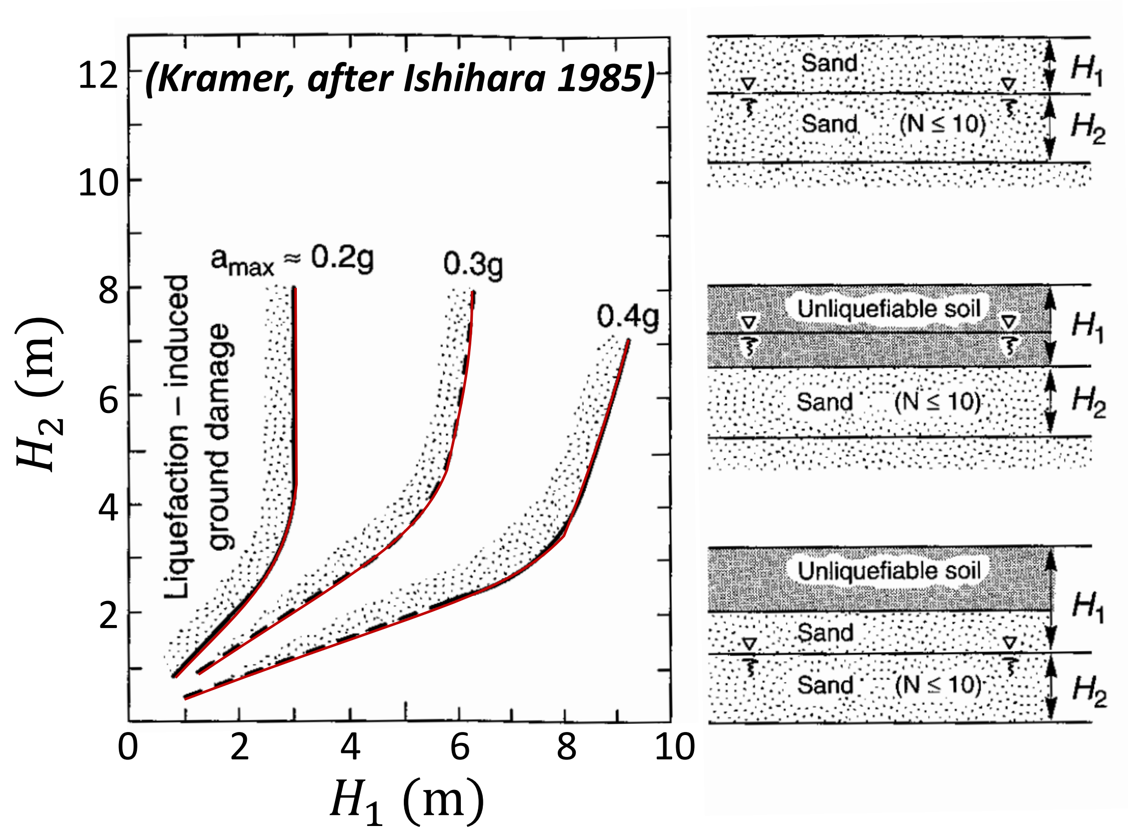
Development of Sand Boils

- Seismically induced excess pore water pressures are dissipated via upward water flow. Water, silt & sand erupts upwards under pressure through cracks and channels of soil layers forming ***sand boils*.**

- Their development depends on the amplitude of $\boldsymbol{u_{excess}}$, and the characteristics of soil layers.

- Sand boils are of little engineering significance, but their development is an indication of the developed $\boldsymbol{u_{excess}}$.

- When water is trapped beneath an impermeable layer, a ***water interlayer*** is formed → large *flow deformations*.



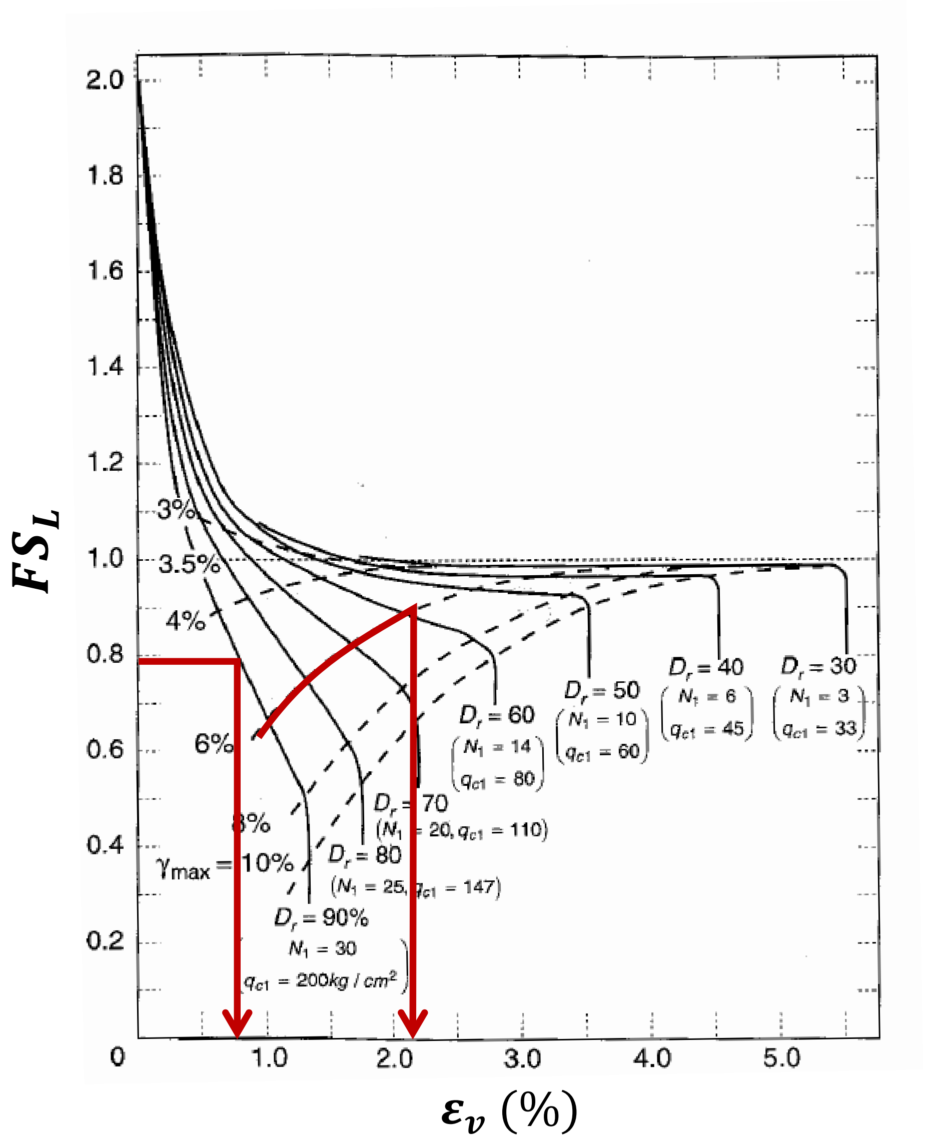
Settlement

Ishihara & Yoshimine Method

- This approach uses γ_{max} or the previously discussed safety factor against liquefaction:

$$FS_L = \frac{CSR_L}{CSR} = \frac{\tau_{cyc.L}}{\tau_{cyc}}$$

- The graph provides an estimate of the volumetric strain ε_v as a function of relative density D_r, *or* corrected $(N_1)_{60}$, *or* CPT tip resistance (q_{c1}).



Example 1:

FSL = 0.8 , $(N_1)_{60}$ = 30 → ε_v **= 0.8%**

Example 2:

γ_{max} = 6% , D_r = 60% → ε_v **= 2.1%**

Settlement

(Tokimatsu & Seed, 1987)

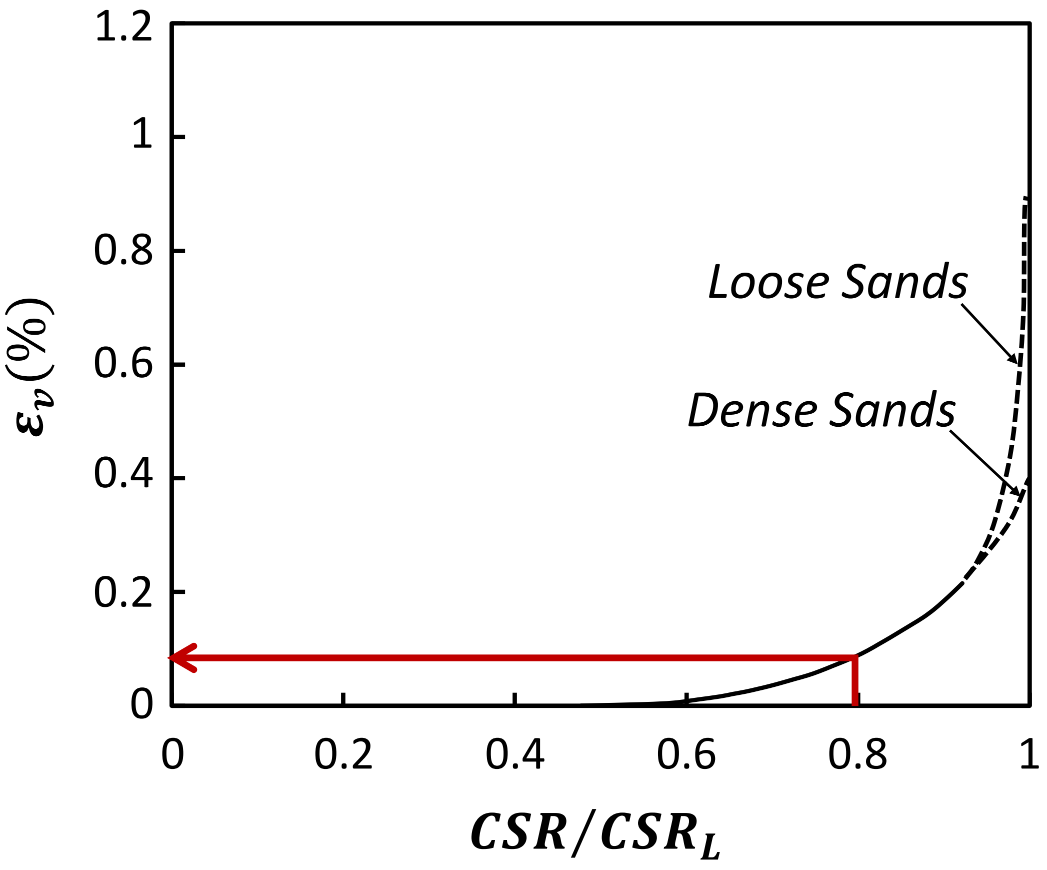
Estimation of ε_v for $r_u < 1$

- Even if initial liquefaction *is not triggered* ($r_u < 1$), the dissipation of excess pore water pressures u_{excess} will unavoidably induce an amount of volume change.

- The relationship shown in the graph is valid for $r_u < 1$, offering a correlation of the expected volumetric strain ε_v in function of the normalized stress ratio CSR/CSR_L

Example:

CSR/CSR_L = 0.8 → ε_v **= 0.08%**



Instability

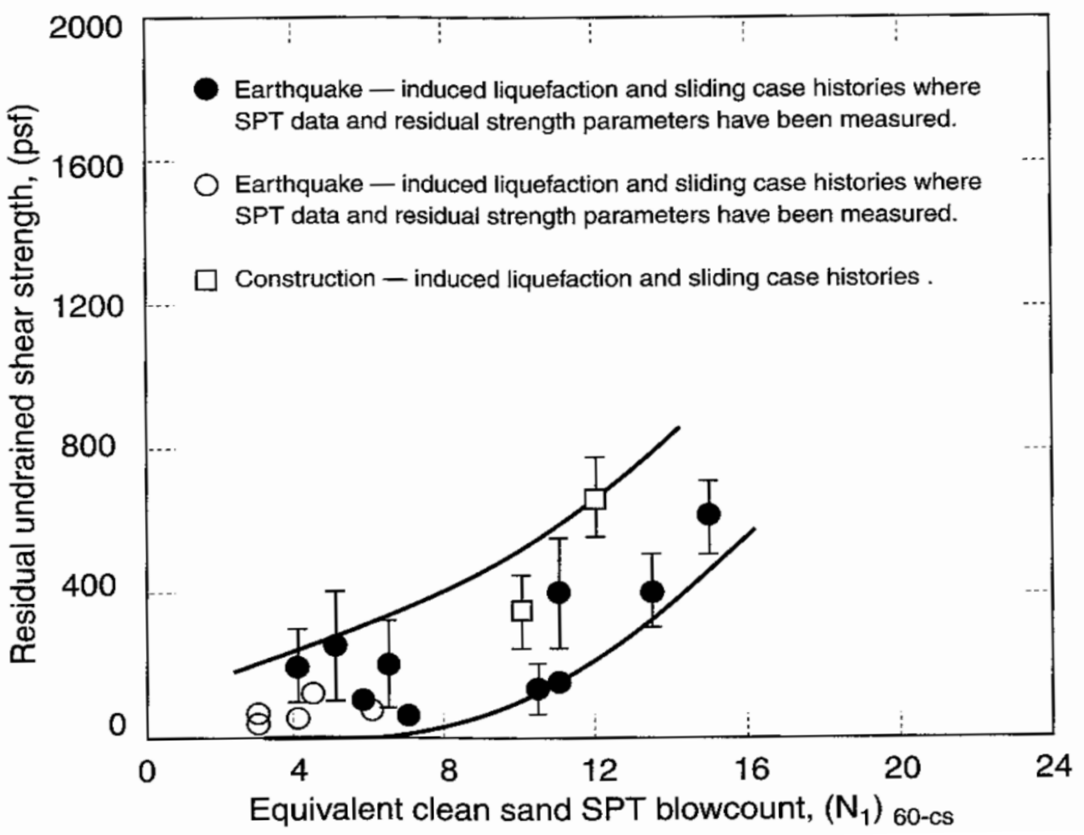
Seed (1986)

In–situ testing approach

- Developed by Seed (1986), the method correlates the residual undrained shear strength to an equivalent clean-sand SPT:

$$(N_1)_{60-cs} = (N_1)_{60} + N_{corr}$$

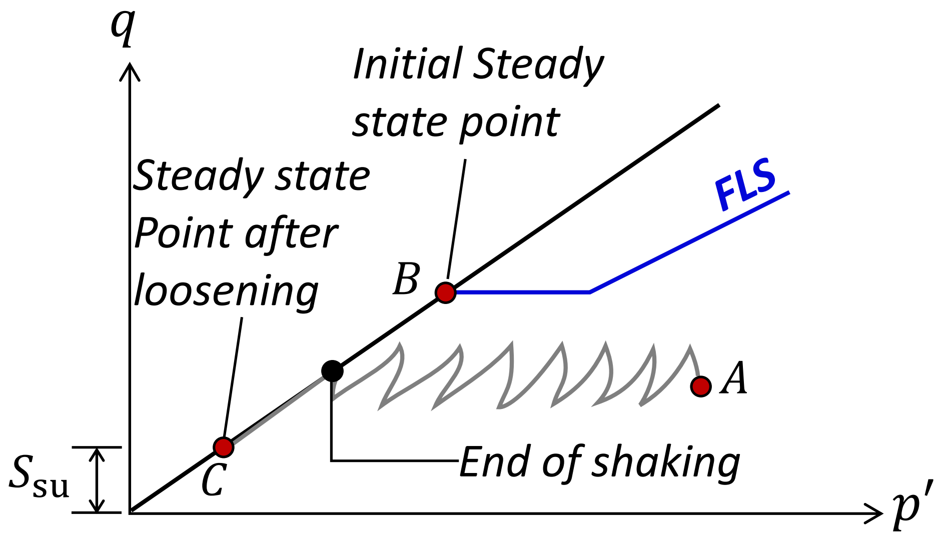
	N_{corr} (blows/ft)	
Percent Fines	Seed–Harder	Stark–Mesri
0	0	0
10	1	2.5
15	—	4
20	—	5
25	2	6
30	—	6.5
35	—	7
50	4	7
75	5	7

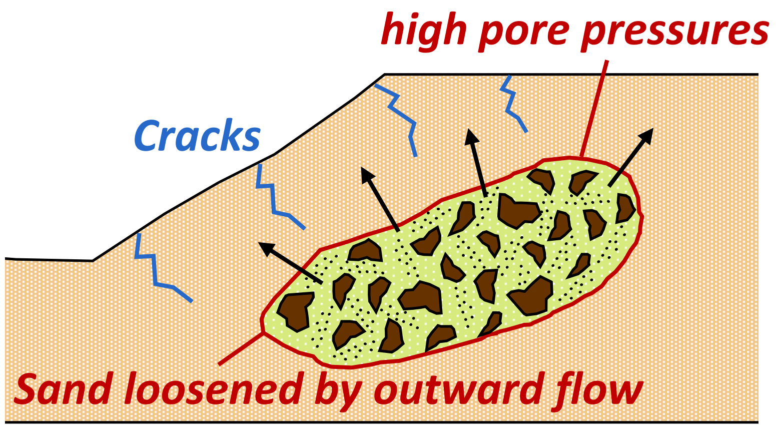


Instability

Global Loosening Flow Failure (NRC Mechanism C)

- When high pore pressures are developed at depth, porewater will ***flow through drainage boundaries*** of shallower layers towards the surface.
- Such flow may cause loosening of the shallow layers, leading to a decrease of steady–state strength. Cracks may also contribute.
- Such loosening may take place ***well after the end of shaking***, since time is needed for the water to flow towards the shallow layers.
- If $\boldsymbol{S_{su}}$ ***after loosening*** becomes smaller than the static shear stress, *flow failure will be initiated.*





Instability

Interface Flow Failure (NRC Mechanism D)

- Occurs when the shear strength of an interface between a liquefiable soil layer and a structure is less than required to maintain equilibrium.
- If the structure’s surface is smooth (e.g., the surface of a steel pile),mechanism D may be triggered without soil volume change.
- This means that NRC Mechanism D can also appear in dilative soils.

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Thank you for your attention!

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