Soil Liquefaction
地盤の液状化

Damage Caused by Earthquakes

- Human damage was caused by tsunami, collapse of structures, and fire.
- Damage to infrastructures, such as roads, bridges, **harbor facilities** is often related to liquefaction.
  - In 1995 Hyougoken-nambu Earthquake infrastructures were severely damaged by liquefaction.

Liquefaction as a Disaster

- Liquefaction has been recorded as a natural phenomenon from ancient age.
- Liquefaction was widely recognized as a natural disaster after two earthquakes in 1964.

  - ***Niigata Earthquake*** (Japan)
  - ***Alaska Earthquake*** (US)

  modern structures were severely damaged.
- Research activities on liquefaction have been leaded by the US and Japan, since 1964.

Evidences of liquefactions in ancient age

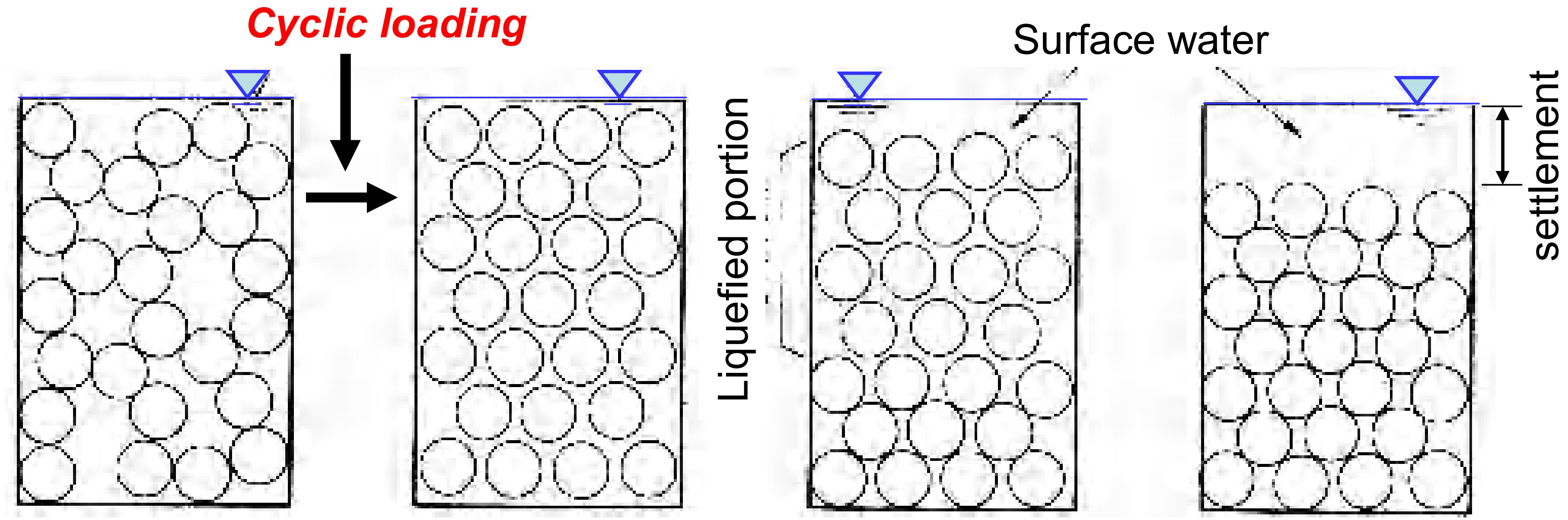
Archaeological excavations reveal the evidences of liquefaction (sand boiling) .

Research Activity on Liquefaction in Japan

- **Mechanisms of liquefaction**
  - Element test and model test
- **Classification of damage**
  - Site investigation of the damages
- **Evaluation of liquefaction potential**
  - Site exploration and laboratory test

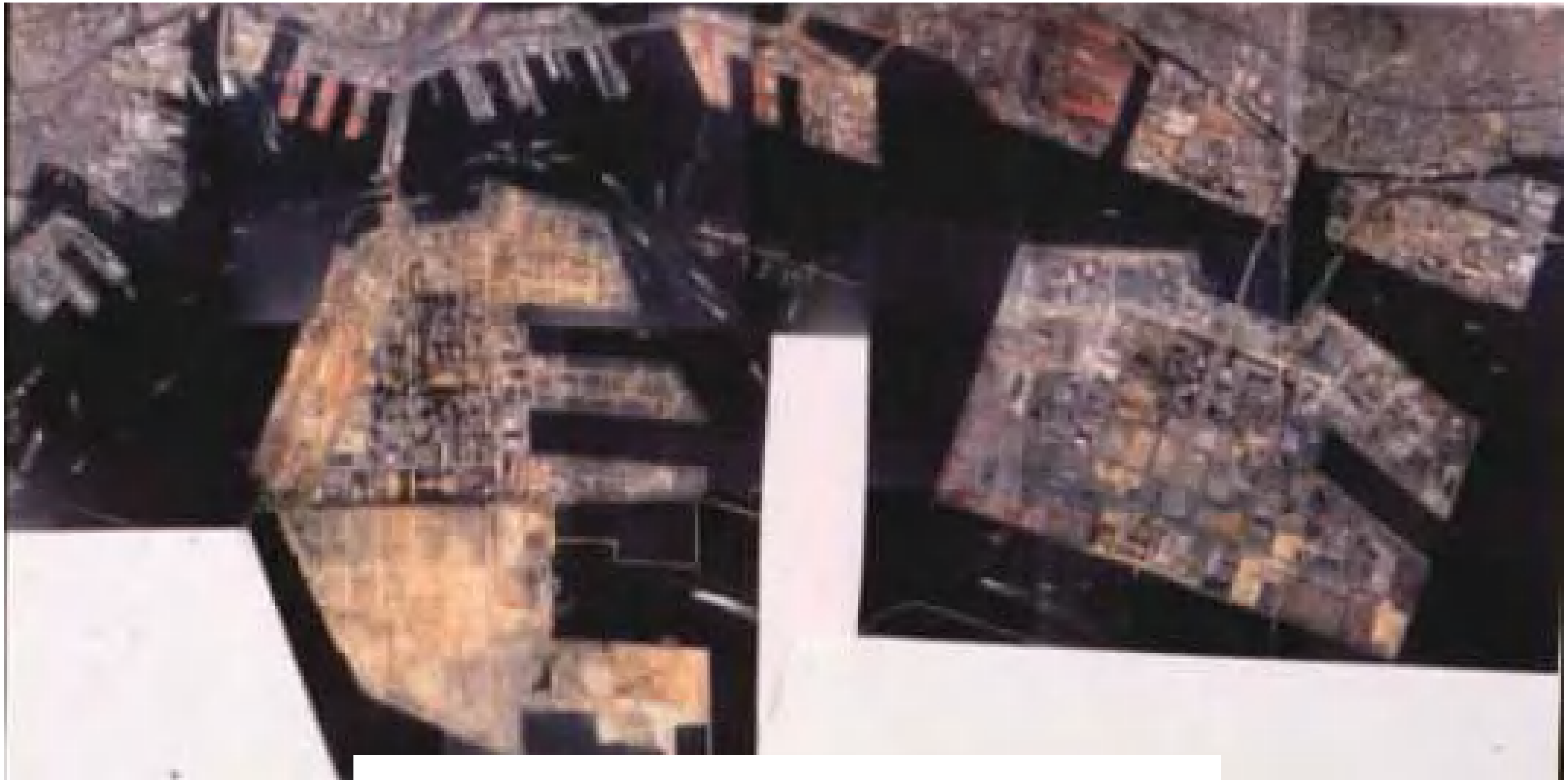
Earthquake resistance design + Counter measures

Process of liquefaction from onset to termination



(a)	(b)	(c)	(d)
Before liquefaction, **Loosely** saturated sand deposition	Just after liquefaction. All particles are suspended in water.	After a certain time At the bottom, liquefaction is terminated, but not at the top.	After long time No liquefied portion. Densification of sand causes settlement.
	liquefied		***volume decrease***

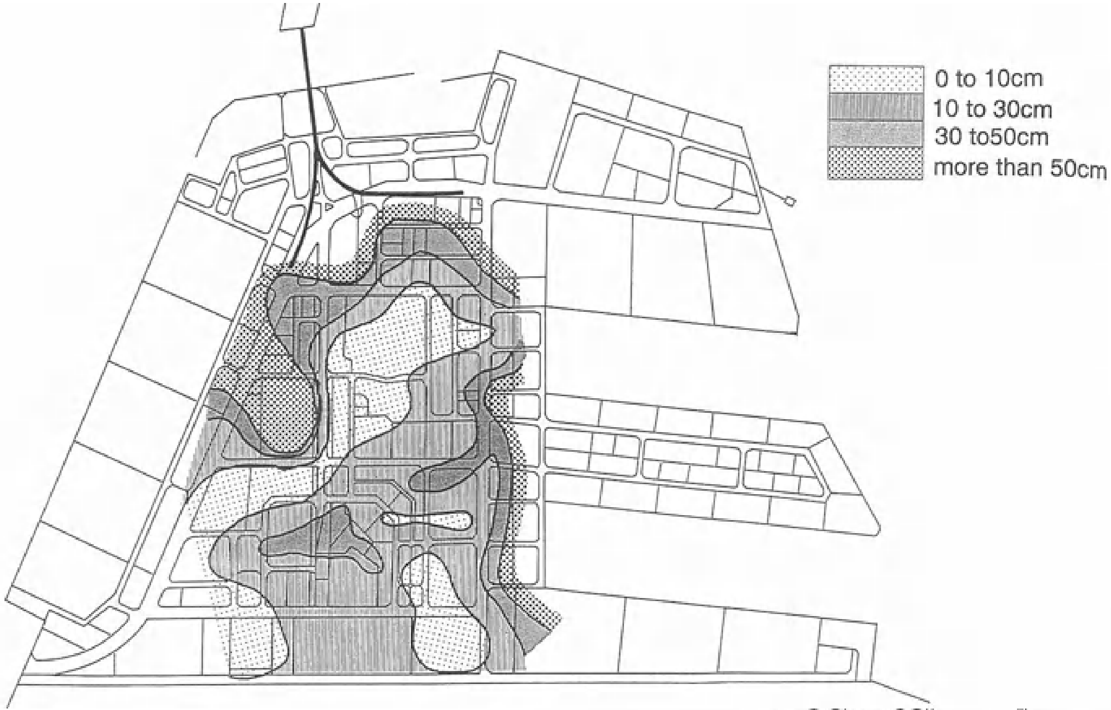
Water Front in Kobe City after 1995 Hyogoken Nambu Earthquake

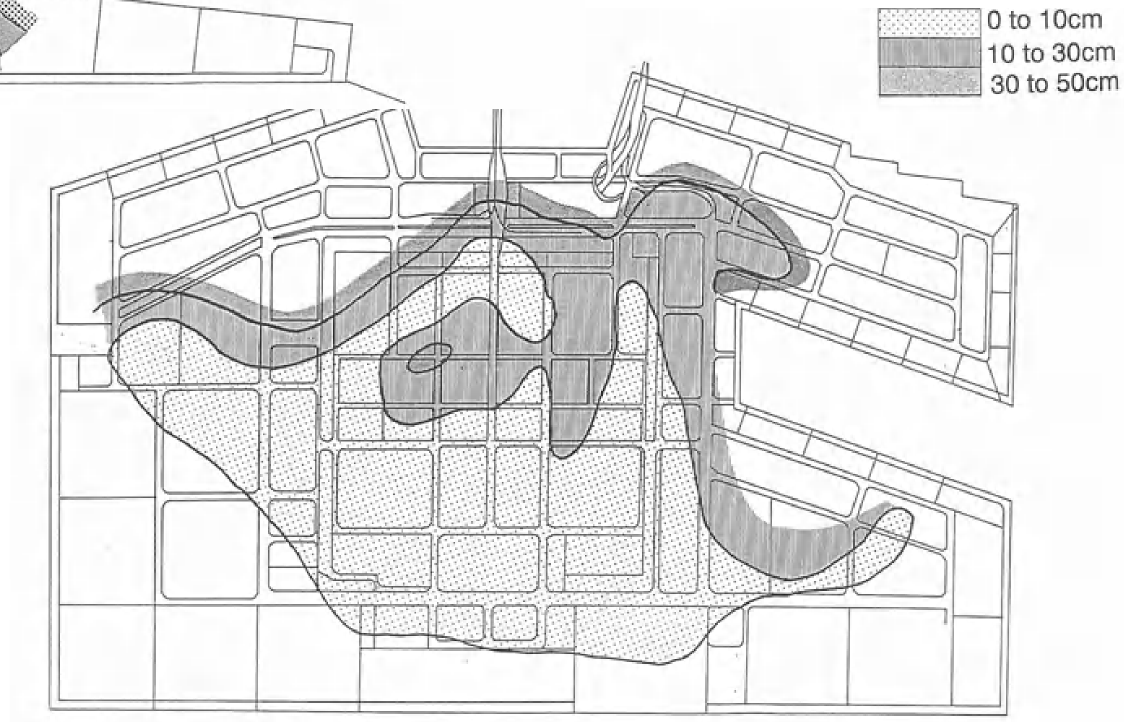


liquefaction occurred extensively

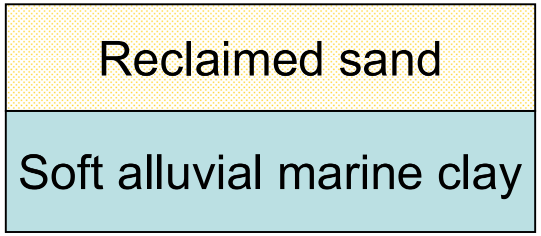
Settlements at reclaimed islands observed at 1995 Hogoken-Nanbu Eq.

S&F special issue (1996)

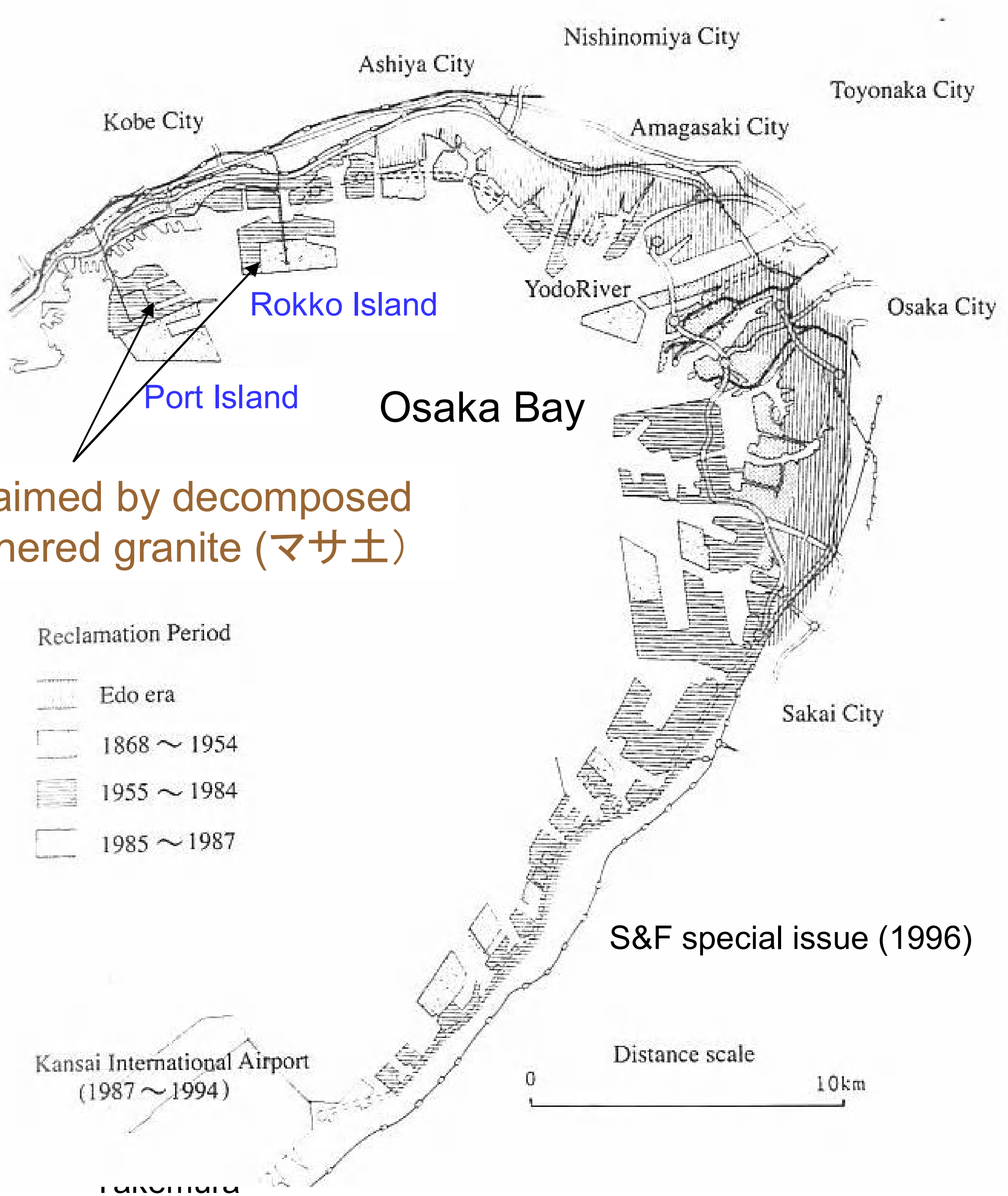




Reclaimed lands in Osaka Bay

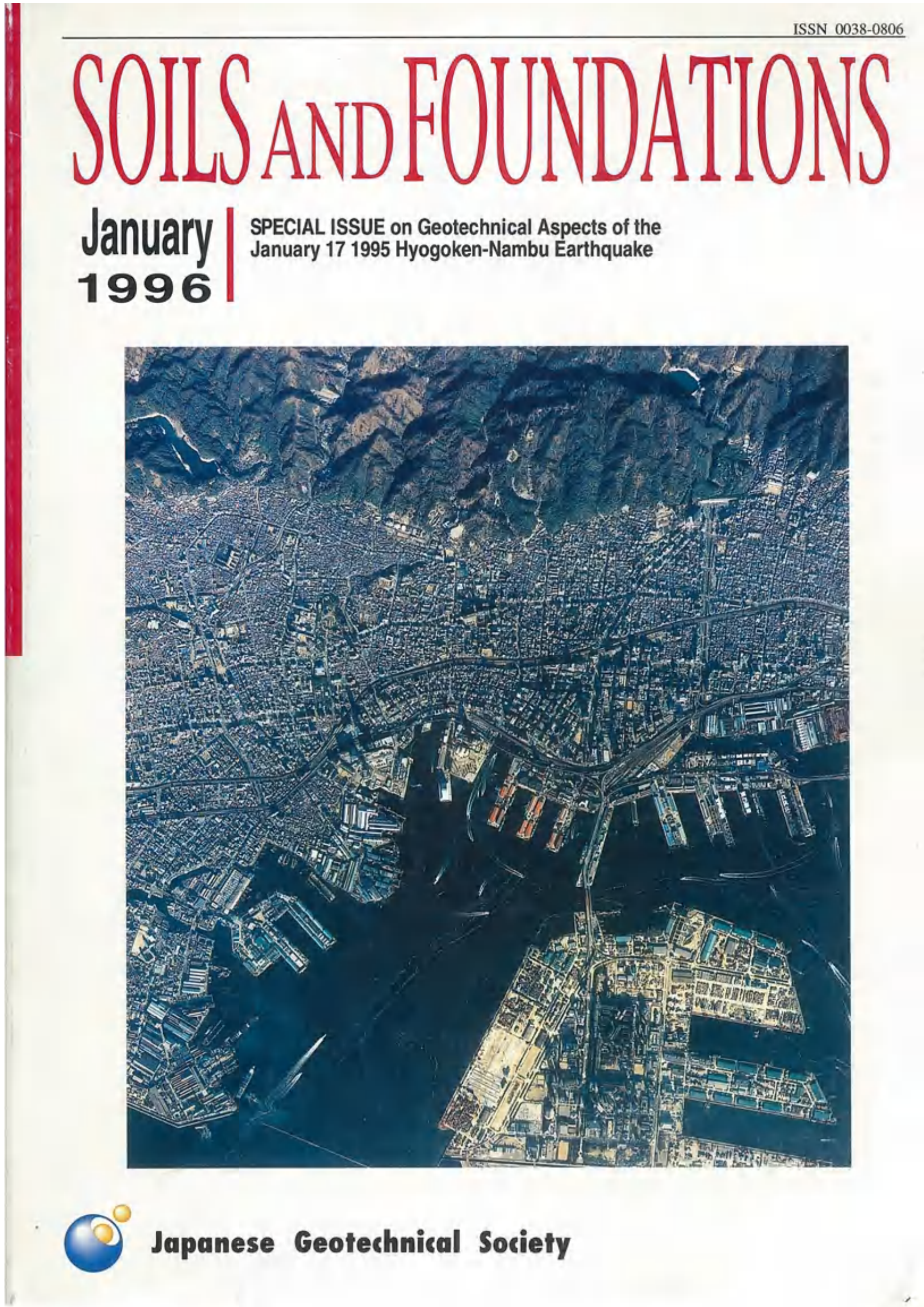


Typical soil profile of reclaimed land in Osaka Bay



S&F special issue (1996)

Soils & Foundations
Special Issue
January 17 1995
Hyogoken-Nanbu Earthquake



Mechanism of Liquefaction

- Liquefaction mechanism in sand element
  - –Rearrangement of sand particles
  - –Motion of sand particles under water
  - –**Volume change due to shear deformation**

[**Dilatancy**] in *Drained condition*

excess pore water ***Δu in Undrained condition.***

Relative density

decrease of effective stress

decrease of strength

decrease of stiffness

loss of strength and stiffness

The Principle of Effective Stress

-Basic principle in Soil Mechanics-
by Prof. Karl Terzaghi

Stress in soil: Total stress=effective stress + pore pressure

σ σ' $\mathbf{u}$

"All measurable effects of a change of stress, such as compression, distortion and change of shear resistance, are exclusively due to change in the effective stress."

Strength, stiffness of soils= f(σ')

ex) $\mathbf{s}$=c' + σ' tanϕ'

$\mathbf{G}$= C$\sigma'^{1/2}$

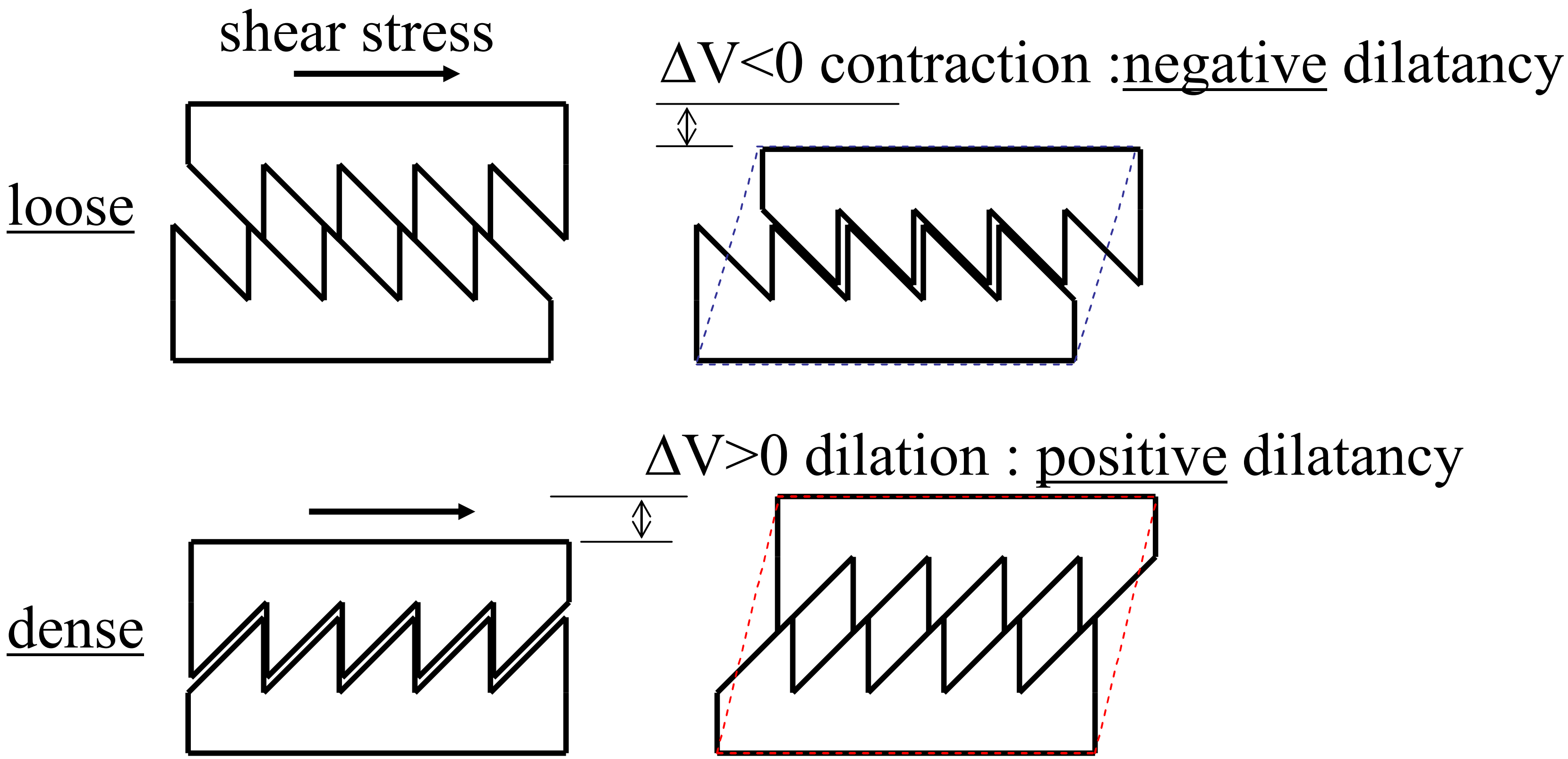
if c'=0 and σ' is 0, $\mathbf{s}$=0 and $\mathbf{G}$=0.

$\boxed{\mathbf{u} = \sigma}$

increasing u ⇨ decreasing **s** and **G**.

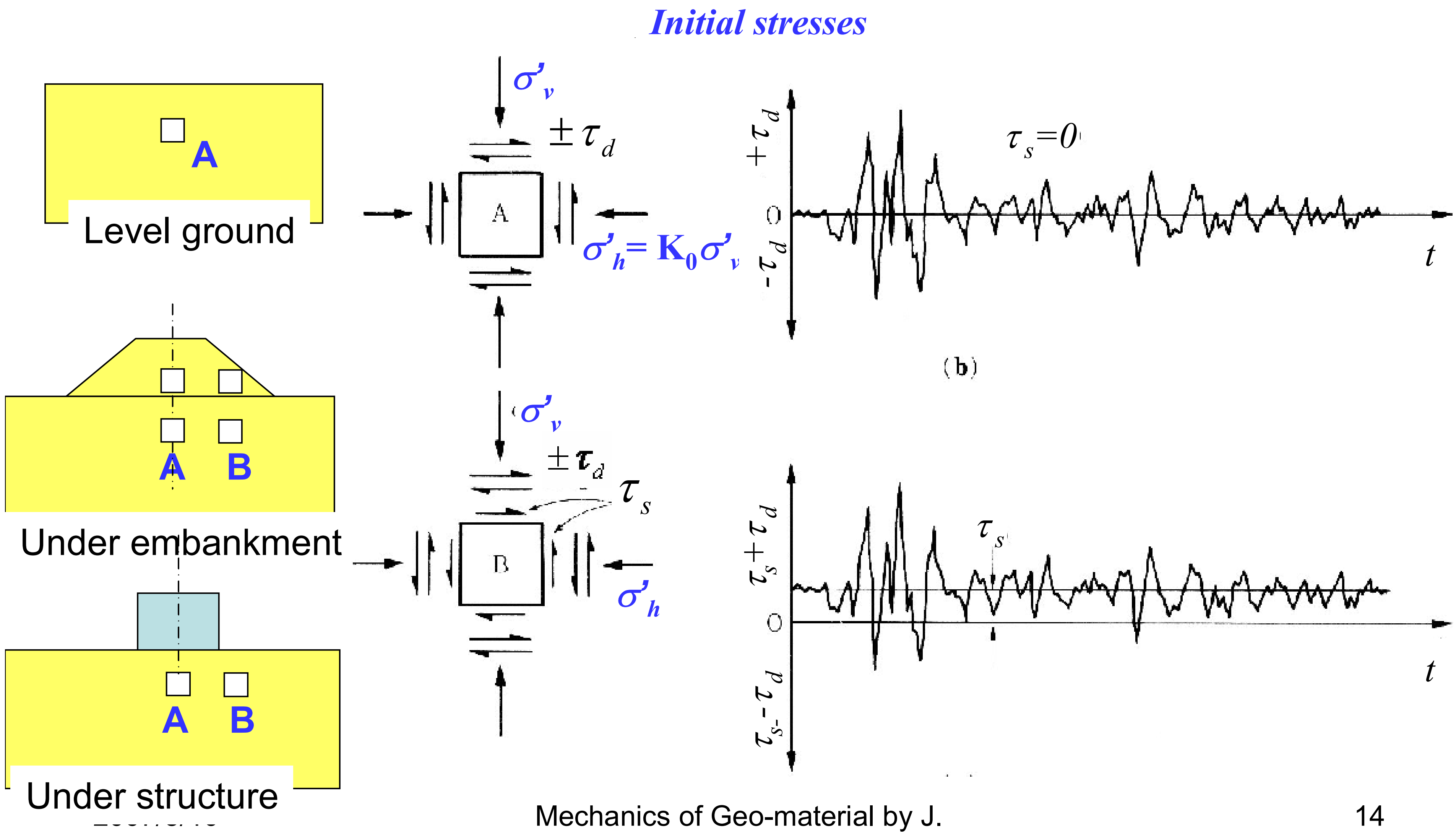
liquefaction

Analogy of dilatancy of granular material



dilatancy: volume change due to shear deformation

Static and dynamic shear stresses in soil elements during earthquake

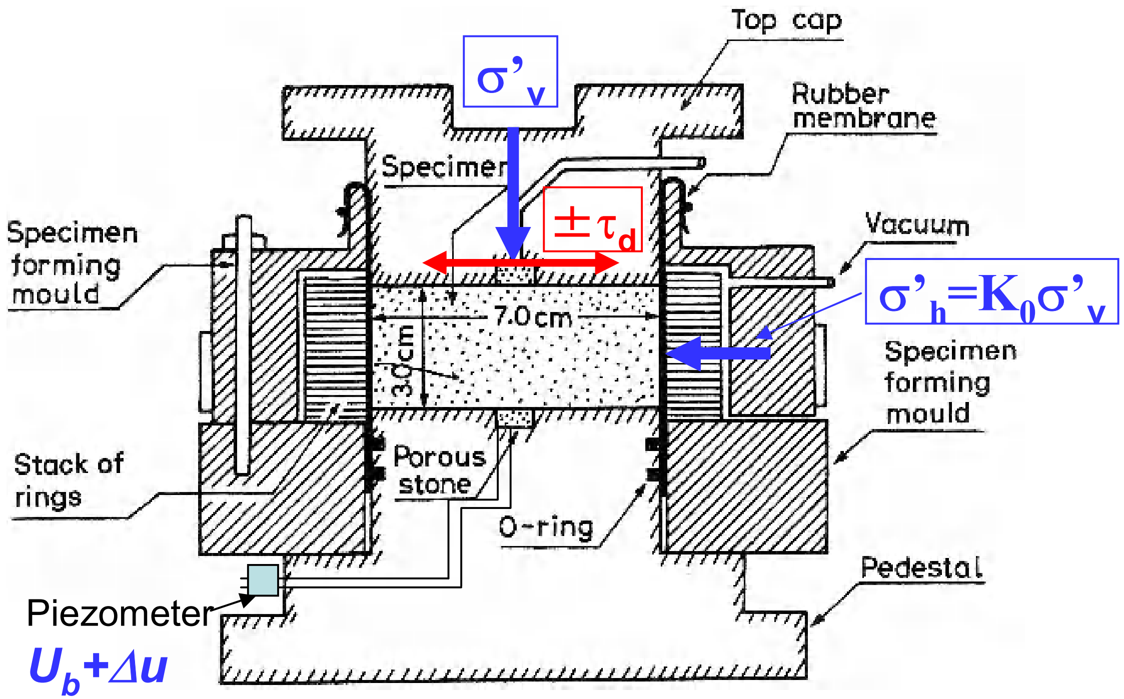


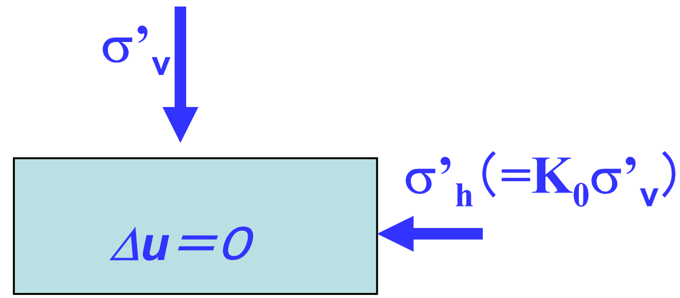
Element tests on liquefaction

Cyclic shearing tests:

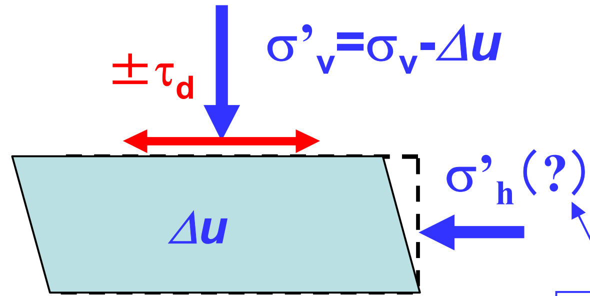
- **Triaxial test(三軸せん断）(commonly used for design);**
- **Direct simple shear test**（直接単純せん断）
  (mainly for research);
  simulating real stress conditions, (initial, cyclic shear by EQ)
  K_0 consolidated sample, simple shear
- **Hollow cylindrical torsion test(**中空円筒ねじりせん断**)**
  (mainly for research)
  simple shear, stress path by <u>stress invariants</u>
  (応力不変量）

Direct simple shear test (DSS)





Before cyclic test

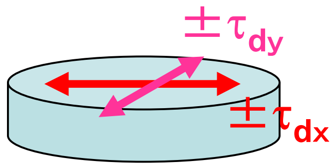


During cyclic test

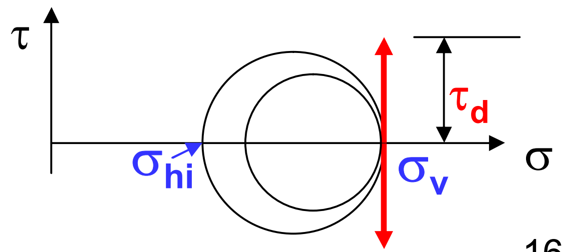
$\sigma_h(?)$

similar to real site:
because σ_h is indeterminate

Shear stress can be applied not only in one direction but also two direction.

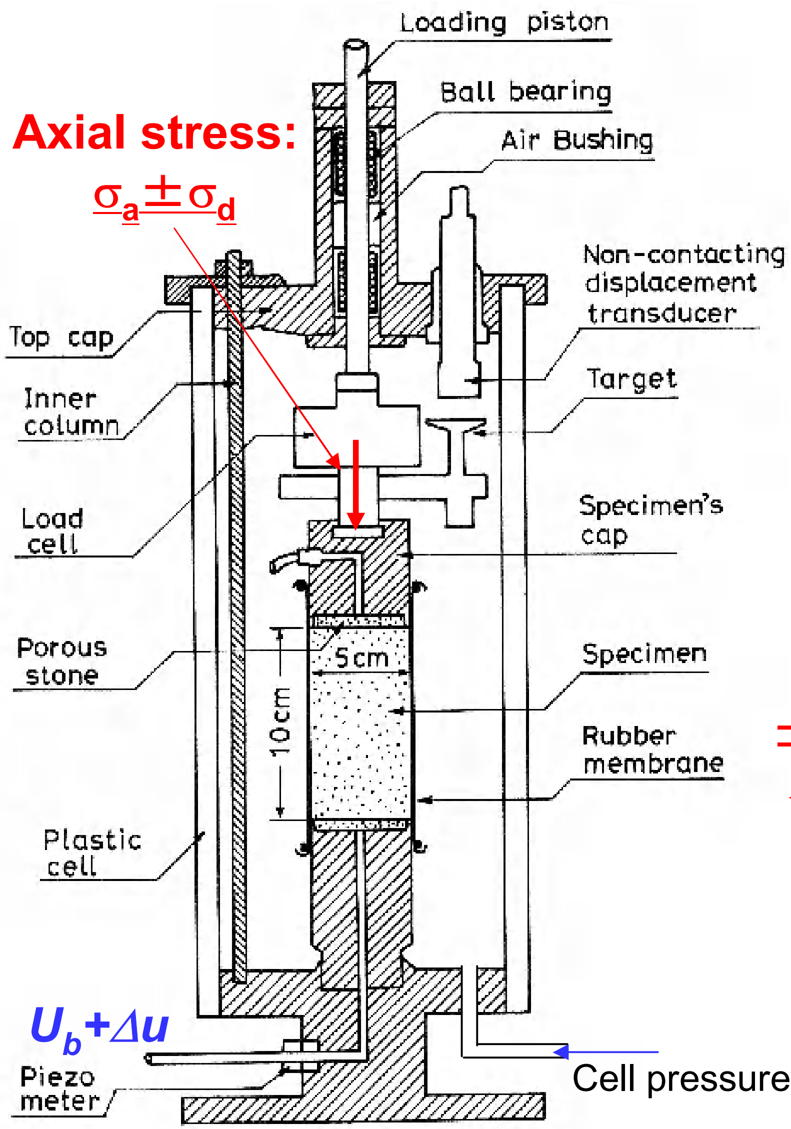


Total stress path on the horizontal plane.



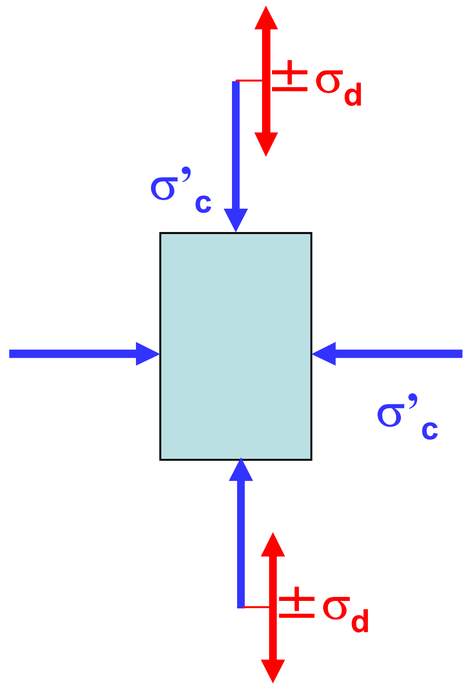
Triaxial test

Axial stress: $\sigma_a \pm \sigma_d$



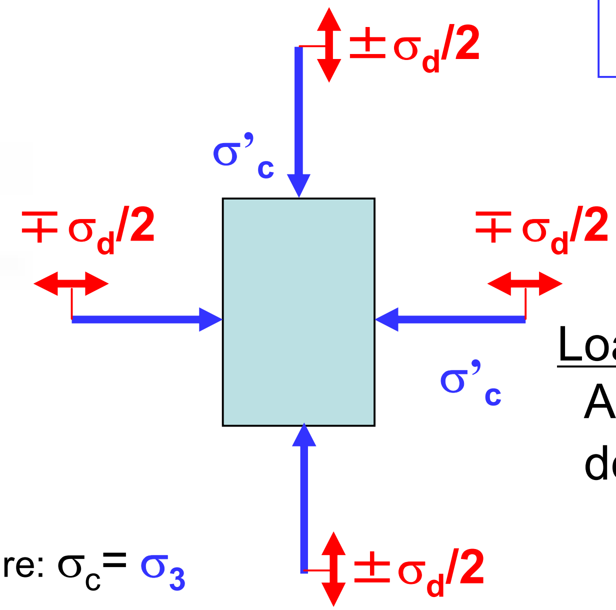
$U_b + \Delta u$

Cell pressure: $\sigma_c = \sigma_3$



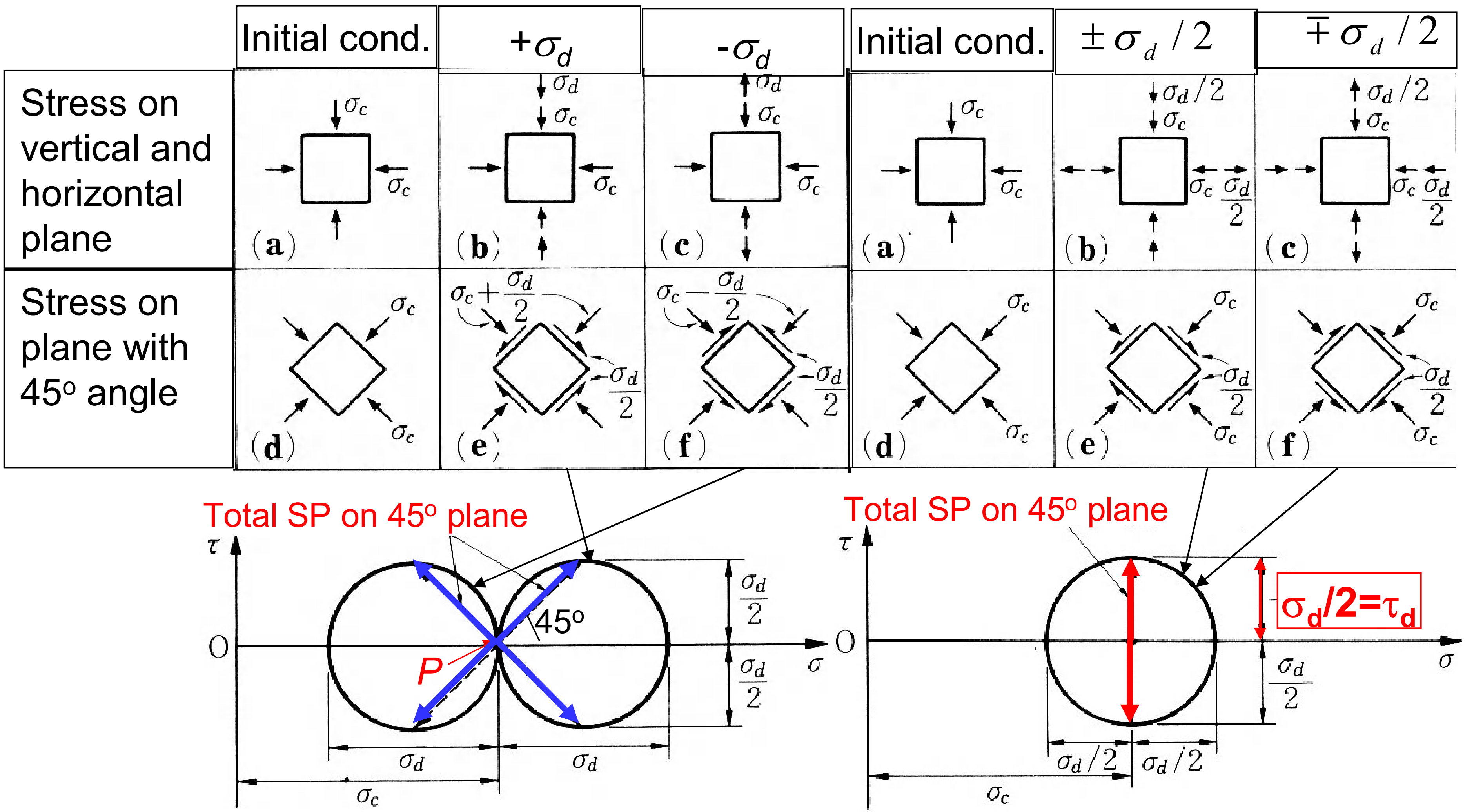
Loading pattern 1
(commonly used)
Amplitude of
deviator stress: $\pm\sigma_d$

Under undrained C. with the same initial C., the **effective stress path are the same** in pattern 1 &2.



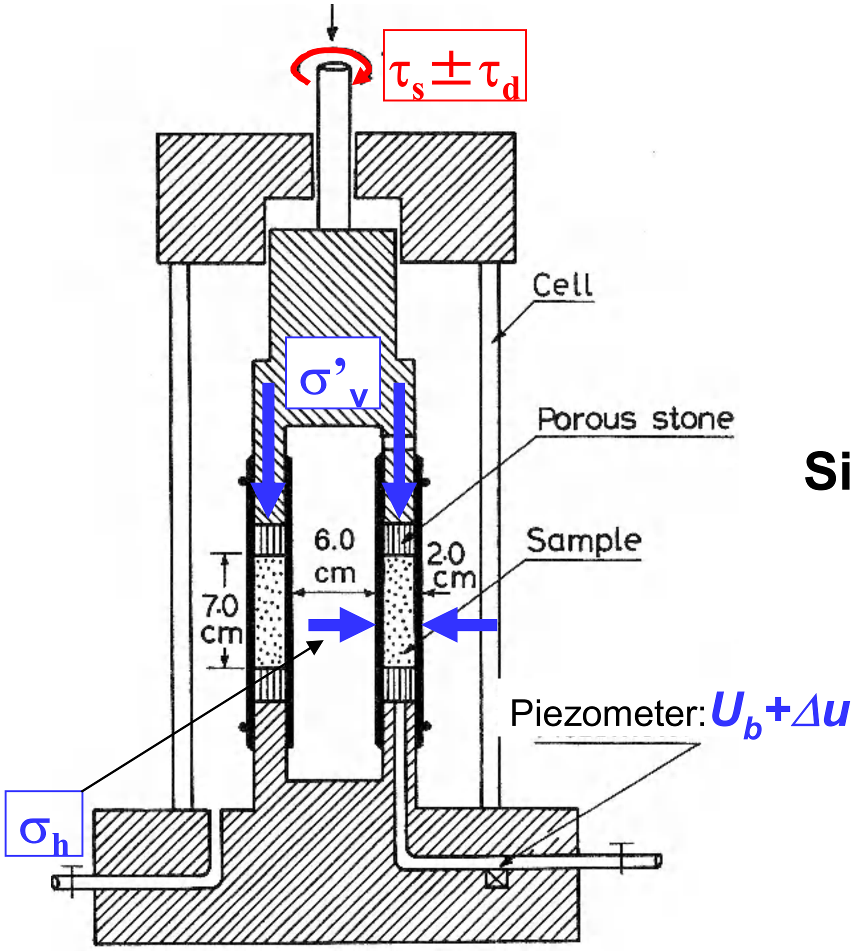
Loading pattern 2
Amplitude of
deviator stress: $\pm\sigma_d$

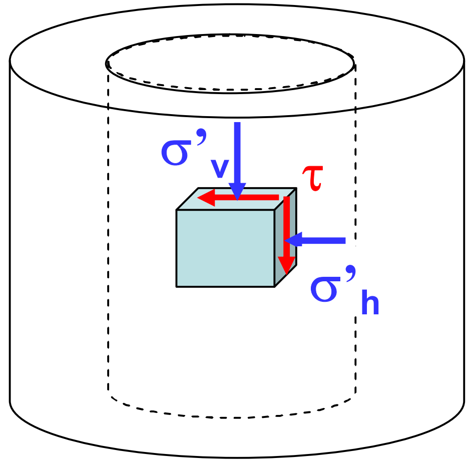
Stress path in compression and extension test



Same as TSP on horizontal plane for DSS

Hollow cylindrical torsional test





Oedometer

Simple shear test:

Two types of horizontal constraint

1) <u>Horizontal strain constraint</u>: $\varepsilon_h = 0$ (DSS)

level ground

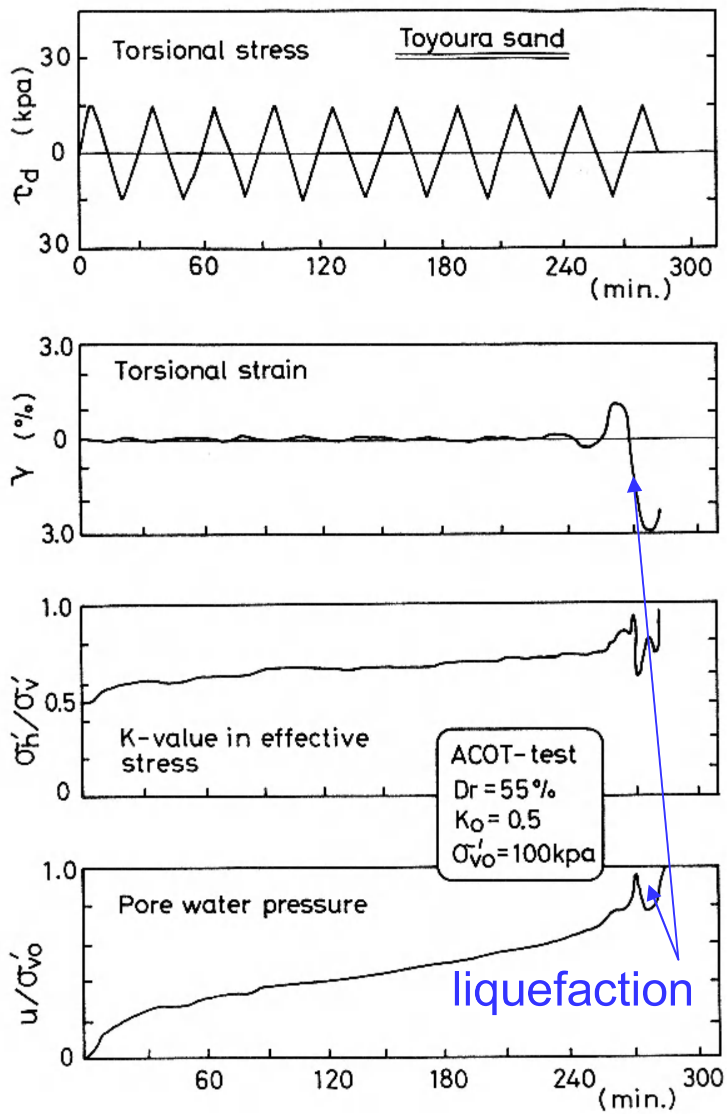
by measuring σ_v, and Δu, σ'_h/σ'_v can be observed in this test.

2) No horizontal constraint: e.g. σ_h=const

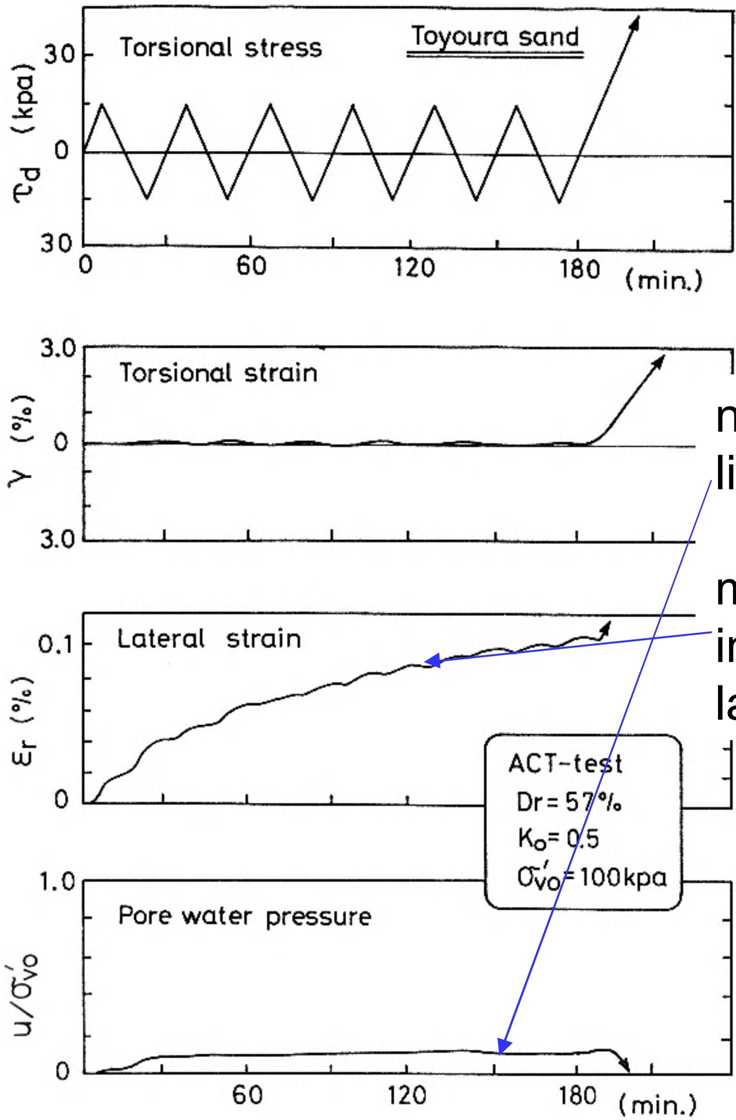
sloping ground (initial shear stress)

Change in lateral stress and pore pressure build-up in Hollow cylindrical torsion tests with and without lateral confinement

1)ACOT (Anisotropic Consolidation <u>Oedometer</u> Torsion)



2) ACT (Anisotropic Consolidation Torsion)

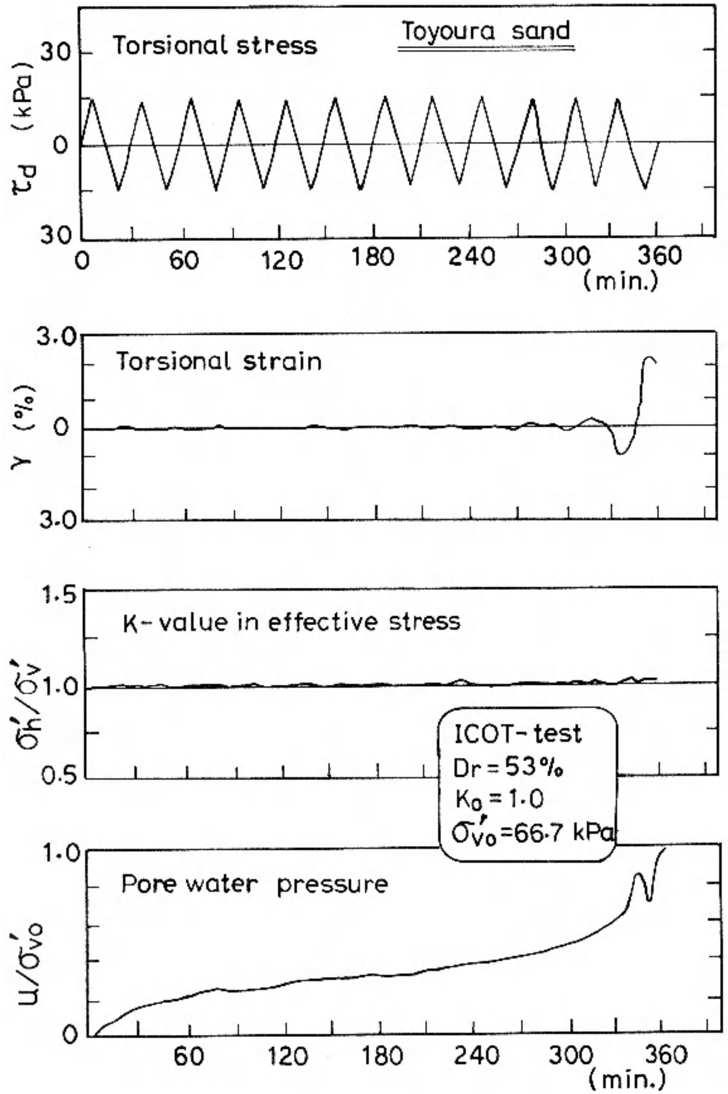


no clear liquefaction but monotonic increase of lateral strain

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.

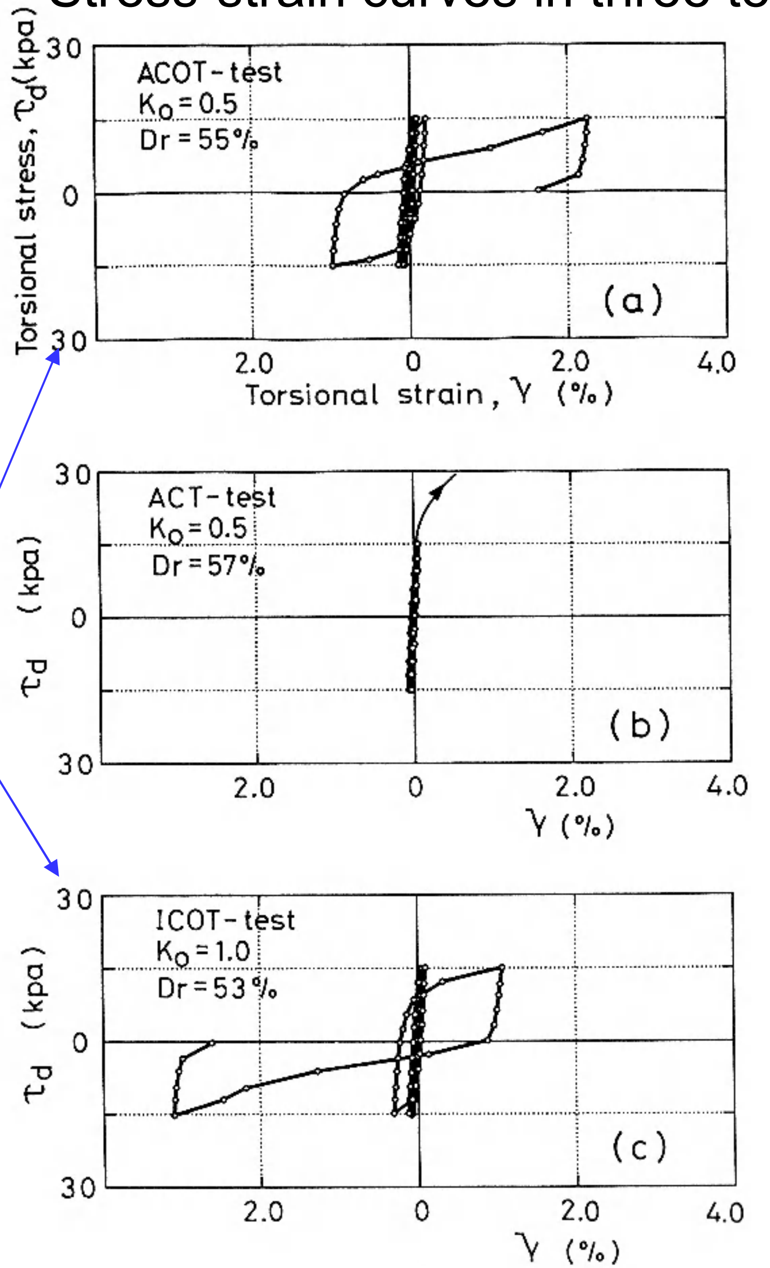
Change in lateral stress and pore pressure of IC sand in HCT

ICOT (Isotropic Consolidation Oedometer Torsion)

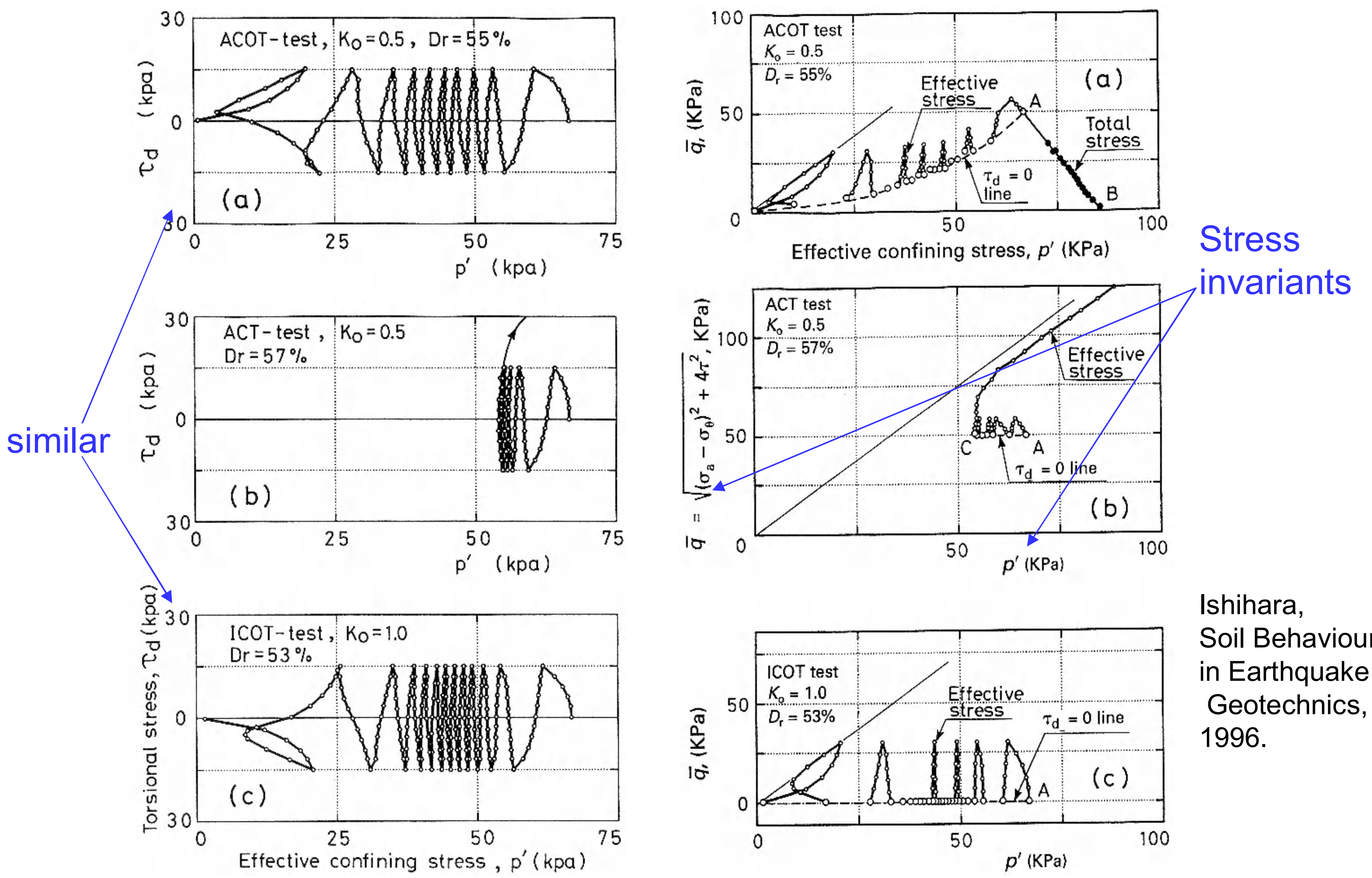


similar

Stress-strain curves in three tests

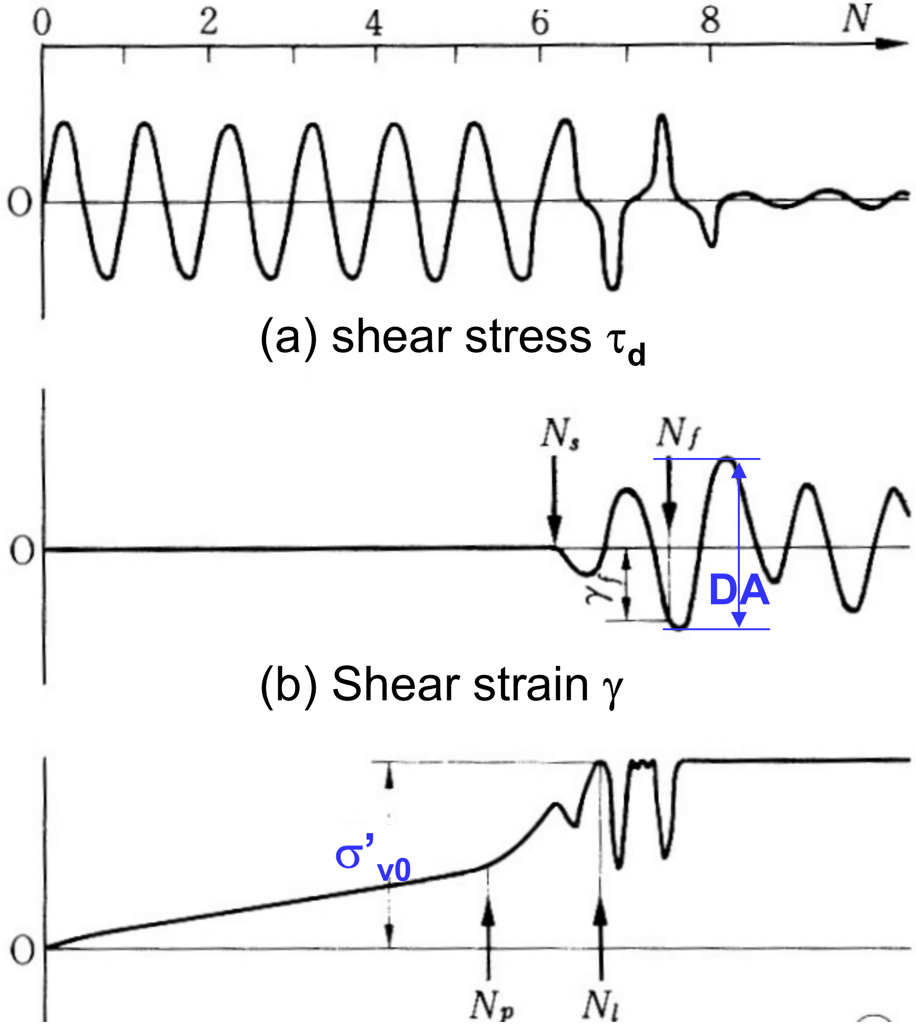


Effective stress paths in three types of torsion tests



Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.

Behavior of loose sand subjected to cyclic loading



(c) Excess pore water pressure Δu

Definition of liquefaction points

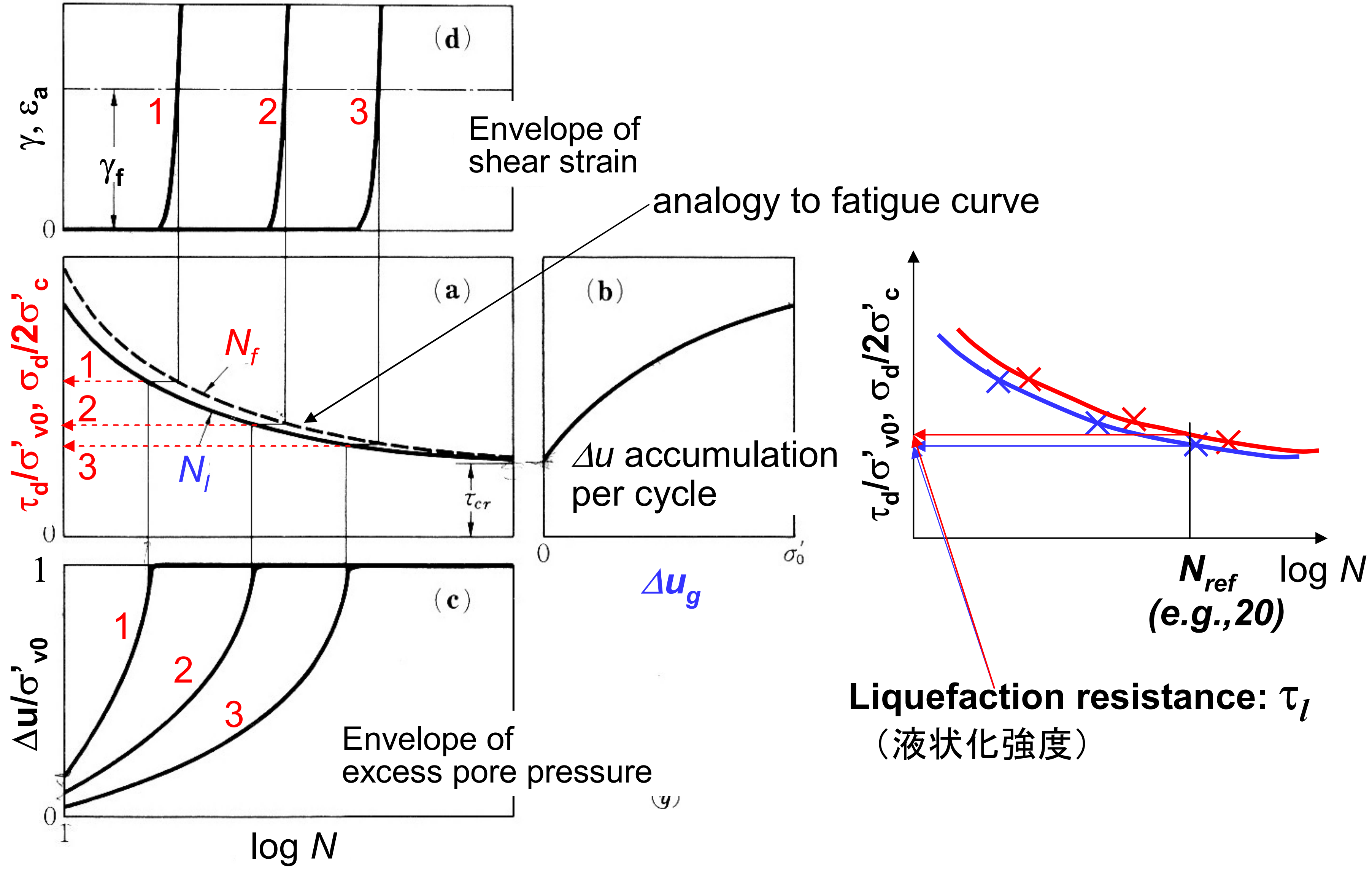
N_p: onset of marked increase of Δu ($\Delta u \sim 0.5\sigma'_{v0}$);

N_s: onset of visible shear strain;

***N_l*: Δu reaches σ'_{v0}** , (zero effective stress)
initial liquefaction(初期液状化);

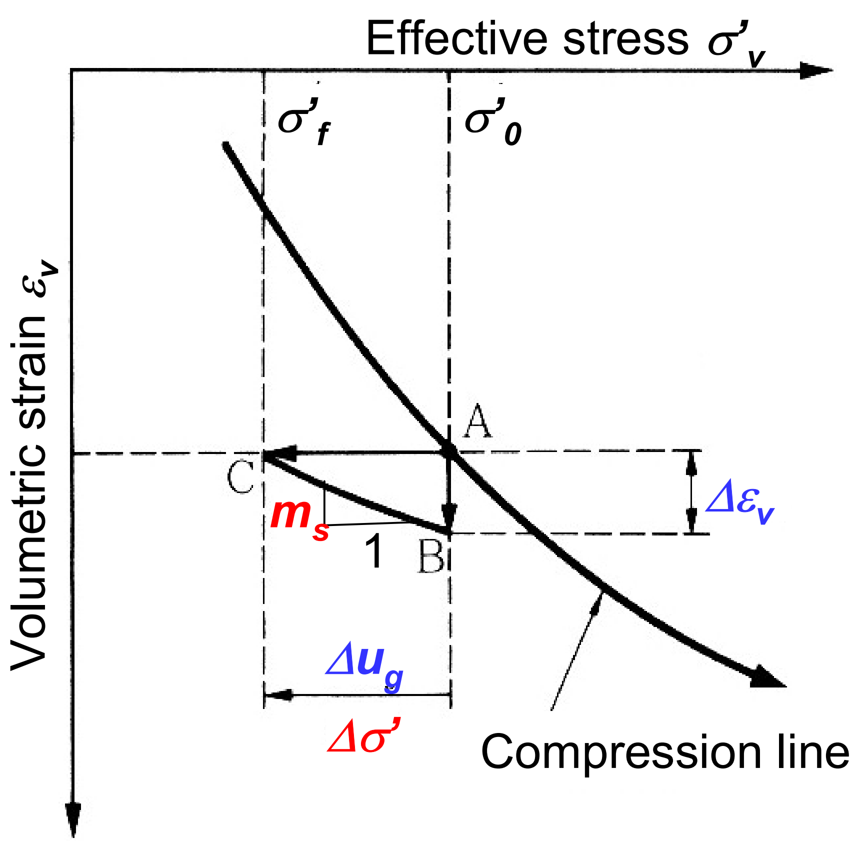
***N_f*: γ reaches a certain value**, e.g. DA (double amplitude) = 5%

Definition of liquefaction resistance
液状化強度の定義



Liquefaction resistance: τ_l
（液状化強度）

Accumulation of Δu in one cycle of loading - Dilatancy and swelling -



n: porosity;
m_w: volume compression coef. of water;
m_s: volume expansion coef. of soil;
$\Delta\varepsilon_v$: volumetric strain due to one cycle of loading under drained condition;
Δu_g: generation of Δu due to one cyclic under undrained condition;
$\Delta\sigma'$: decrease of effective stress.

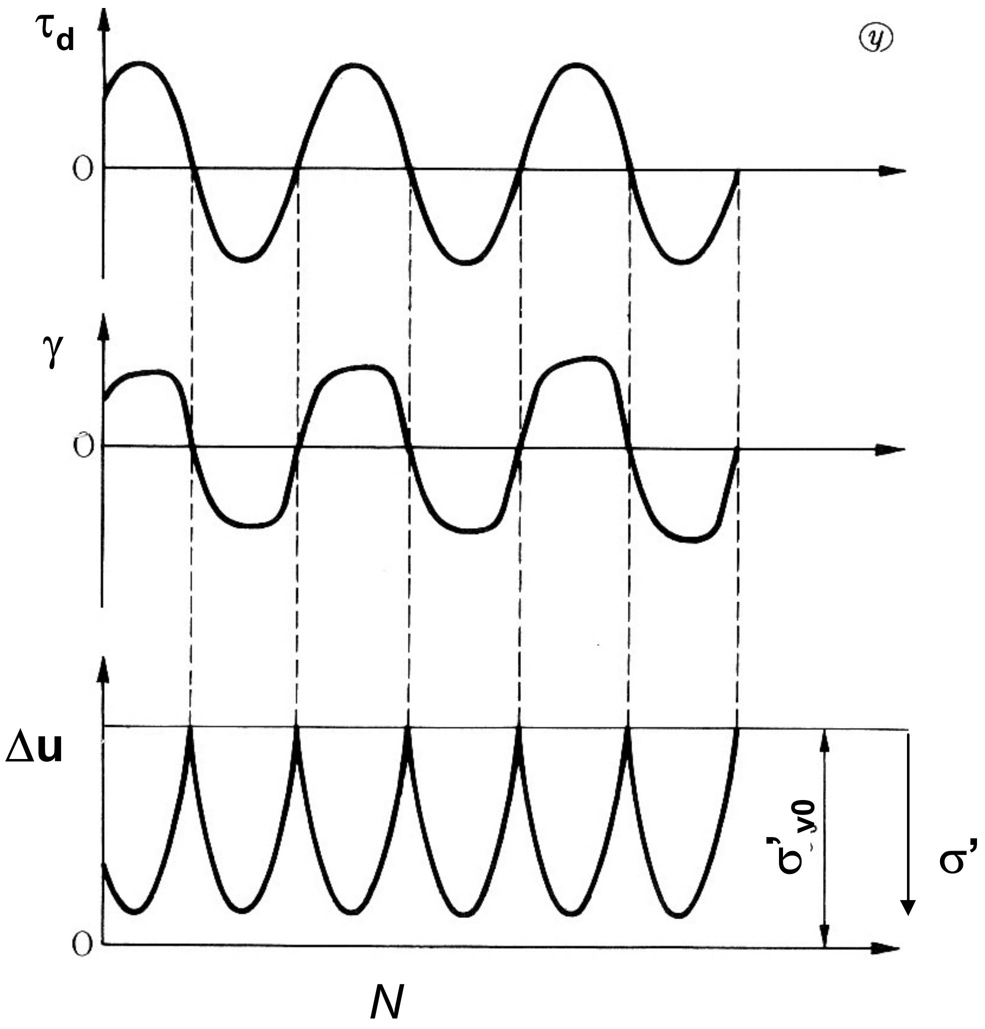
Considering unit volume of soil under undrained condition,

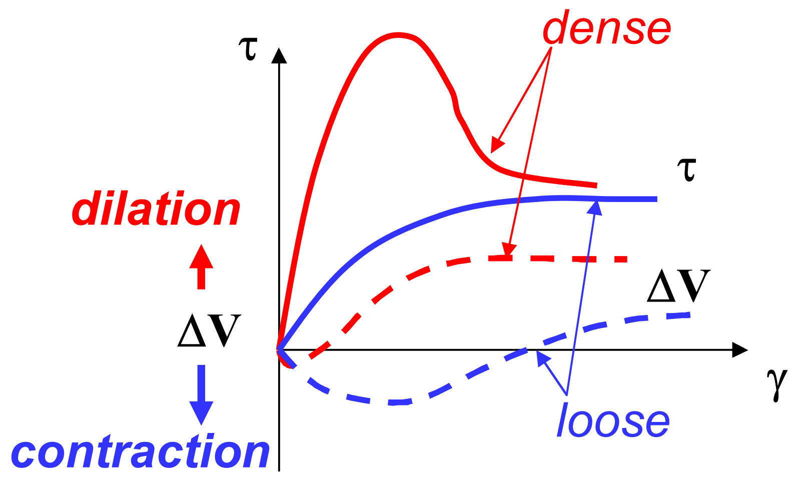
- change of volume of water: $\Delta u_g m_w n$ ①
- swelling volume of soil due to decrease of effective stress: $m_s\Delta\sigma'$ ②
- volume change of soil $\Delta\varepsilon_v$-②: $\Delta\varepsilon_v - m_s\Delta\sigma'$ ③ (neglecting volume change of soil grain)
- change of void = volume change of soil:

$$\Delta u_g m_w n = \Delta\varepsilon_v - m_s\Delta\sigma' \quad ④$$

$\Delta u_g=\Delta\sigma'$ + ④ $\Longrightarrow$ $$\Delta u_g = \frac{\Delta\varepsilon_v}{m_s + m_w n} \quad ⑤$$, $m_s >> m_w$ $$\boxed{\Delta u_g \approx \frac{\Delta\varepsilon_v}{m_s}}$$

Behavior of dense sand subjected to cyclic loading
- *Cyclic mobility* -





Stress and volume change – strain curve of sand in monotonic loading test

Even in dense sand,
contraction takes place at small strain,
=> initial liquefaction (σ'=0)
but dilation at large strain,
=> recovery of effective stress and stiffness, not losing strength.

Factors affecting liquefaction resistance (*LR*)

$\tau_{dl}/\sigma'_{v0},\ \sigma_{dl}/2\sigma'_c$

Properties of soil

- Density (**D_r**);
- Soil type: Grading, Fine contents (**F_c**), plasticity index (I_p);
- Stress history (OCR), aging effects, microscopic structure (sedimentation process, pre shearing history);

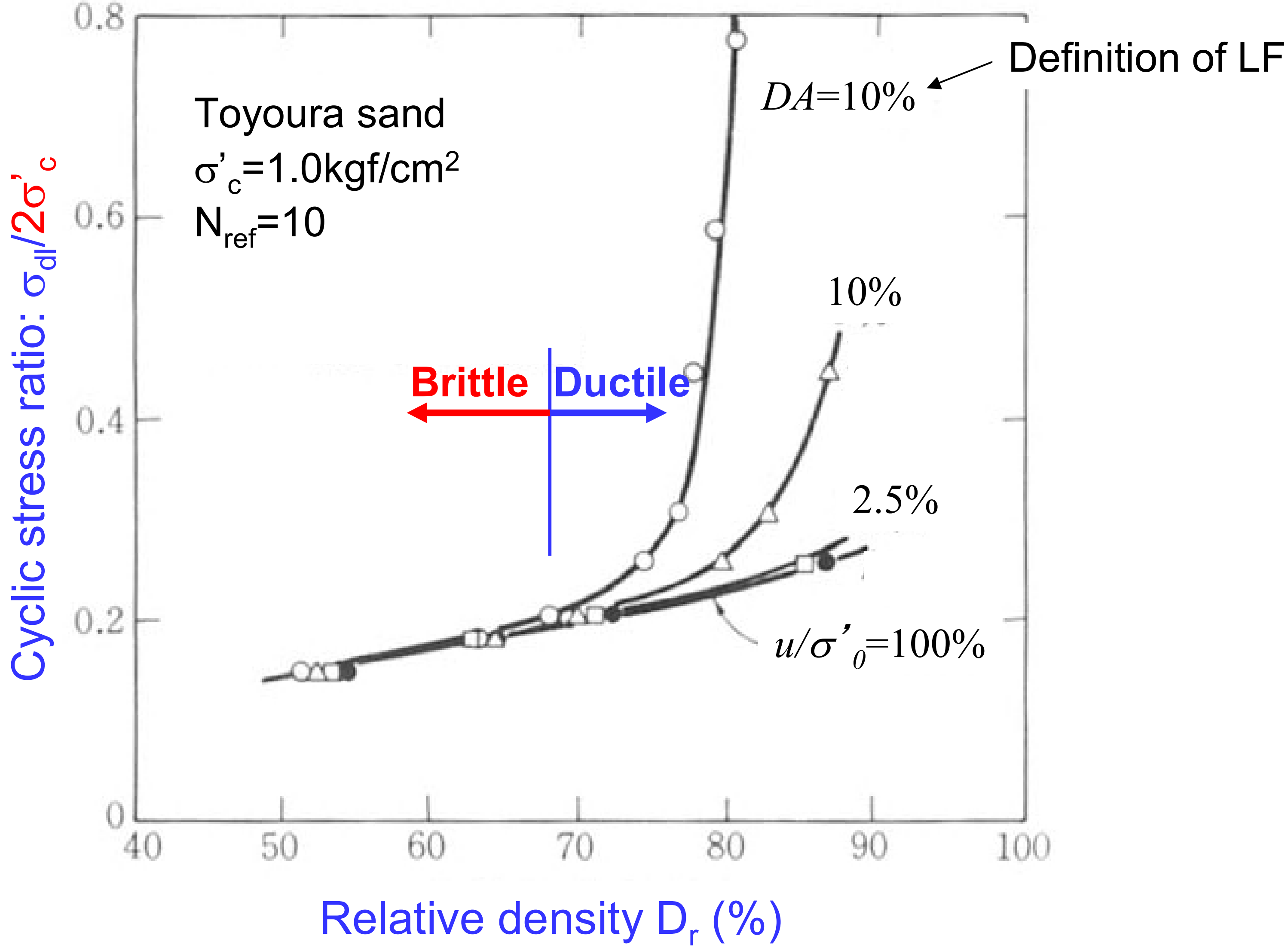
Loading condition, initial stress condition

- Confining pressure (σ'_{v0}, σ'_c);
- Stress Anisotropy under K_0 condition ($\sigma'_h = K_0\sigma'_v$, $K_0 \neq 1$);
- Initial shear stress;
- Frequency of cyclic stress;
- Irregularity of seismic loading;
- Multi-directional seismic motion;

Methods of evaluation

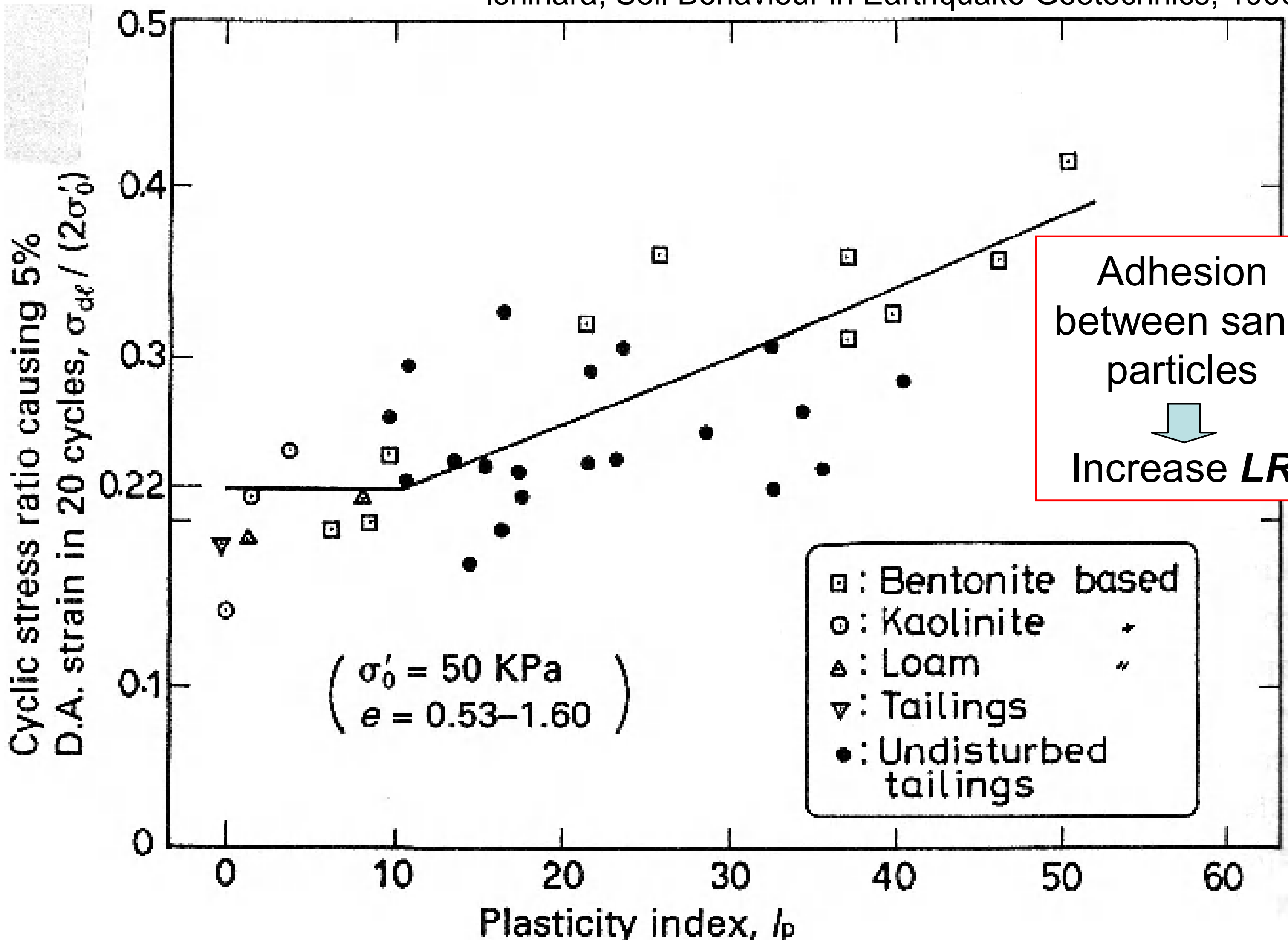
- Sampling disturbance;
- Testing methods (ICT, DSS, HCT);
- Empirical relation of field testing methods (**SPT**, CPT);

Effect of Density



Effect of fine content (F_c), I_p, cohesion

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



Adhesion between sand particles

Increase ***LR***

Fine particles

cohesion

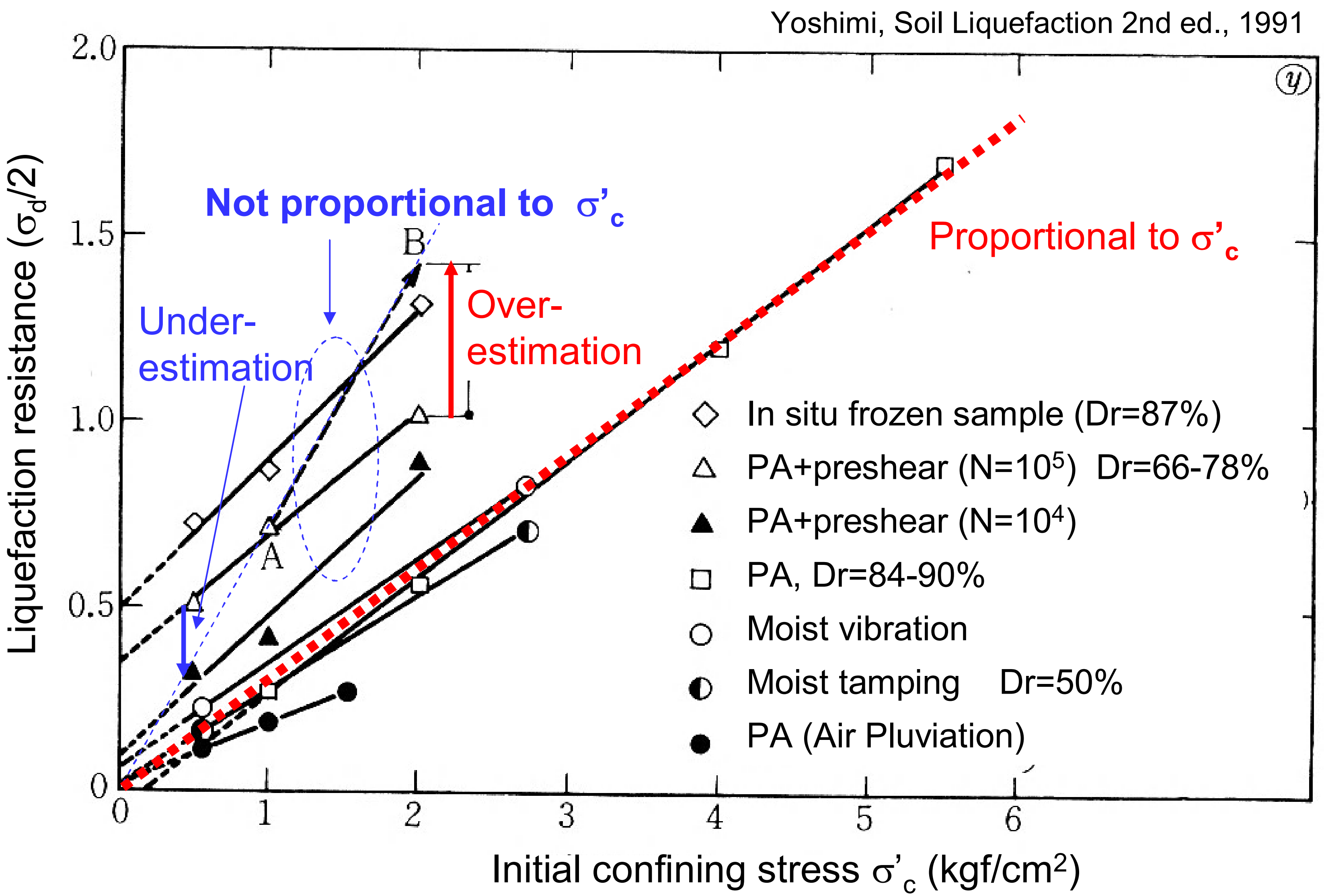
depending on type of fine

Parameters,

F_c, I_p

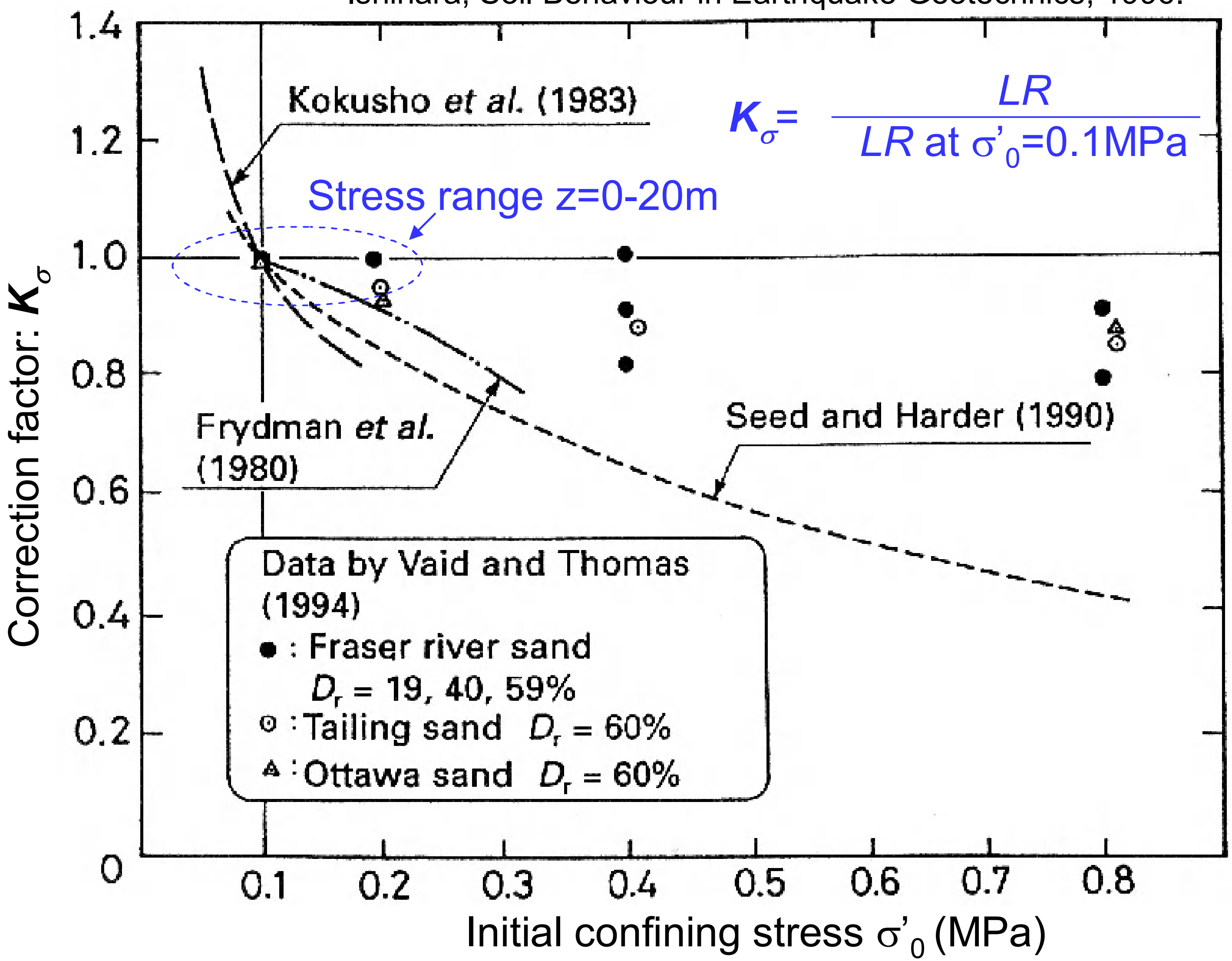
Effects of F_c & I_p higher In OC than NC

Effect of initial confining pressure:初期拘束圧



Effect of initial confining pressure

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



The higher σ'_c,
the more contractive.

Lower resistance.

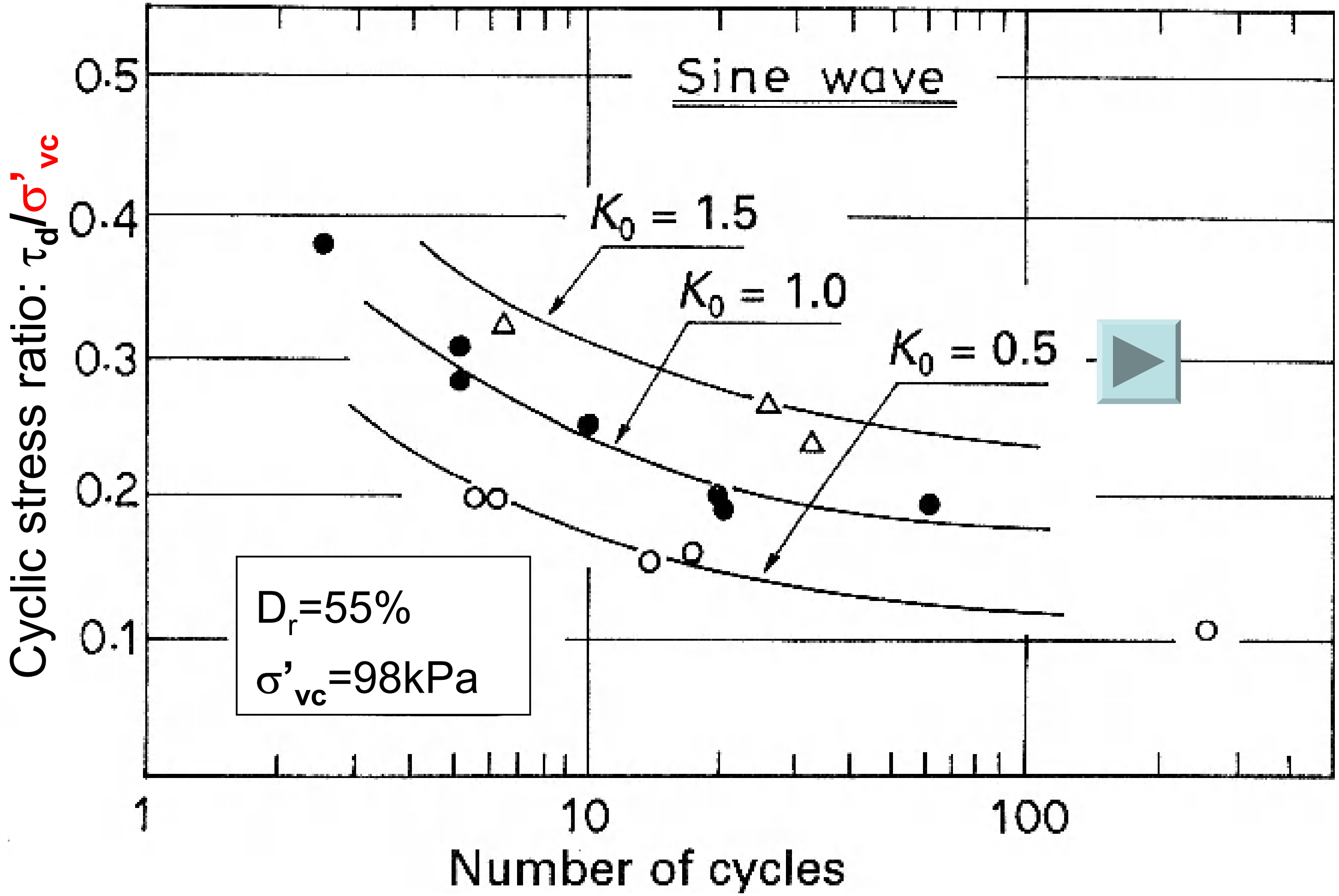
Over estimation for the case of high confining stress, e.g., Foundation of Dam.

But not for the level ground until 20m, i.e., Liquefiable layer.

Justification of normalized strength (Liquefaction Resist.)

Effect of K_0 consolidation

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



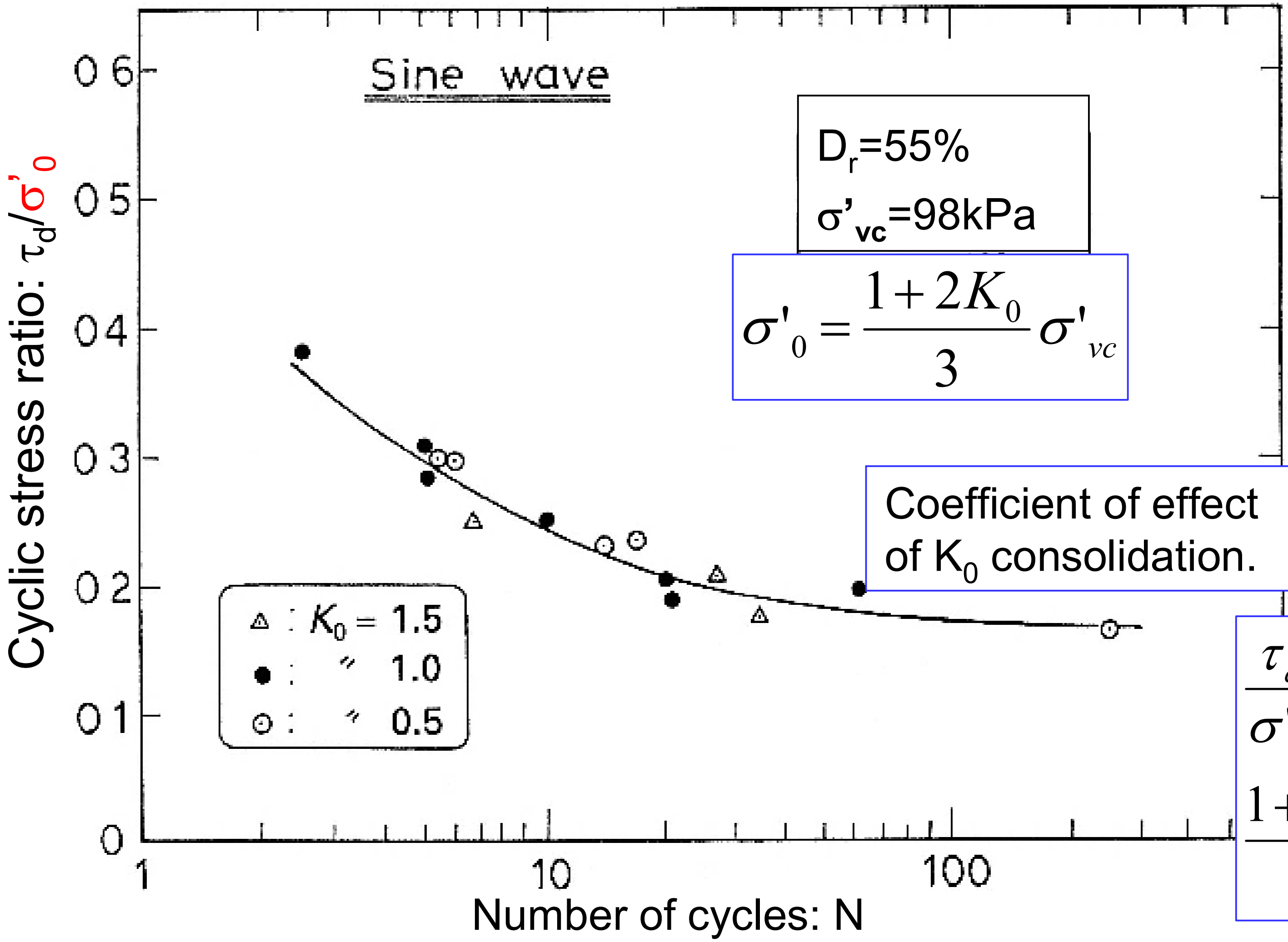
The larger K_0, the higher LR.

The larger K_0, the higher mean effective stress.

$$\sigma'_0 = \frac{1+2K_0}{3}\sigma'_{vc}$$

Effect of K_0 consolidation, mean initial effective confining stress

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



D_r=55%

σ'_{vc}=98kPa

$$\sigma'_0 = \frac{1+2K_0}{3}\sigma'_{vc}$$

Coefficient of effect of K_0 consolidation.

Difference betw. IC and K_0C

Difference of Mean initial effective confining stress

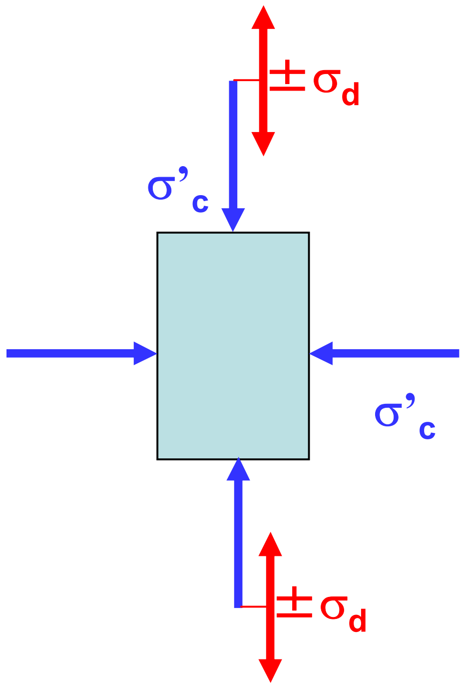
e.g. K_0=0.5 (typical value for *NC sand*)

$$\frac{\tau_{dl}}{\sigma'_{vc}} = \boxed{C_1}\frac{\sigma_{dl}}{2\sigma'_c}$$

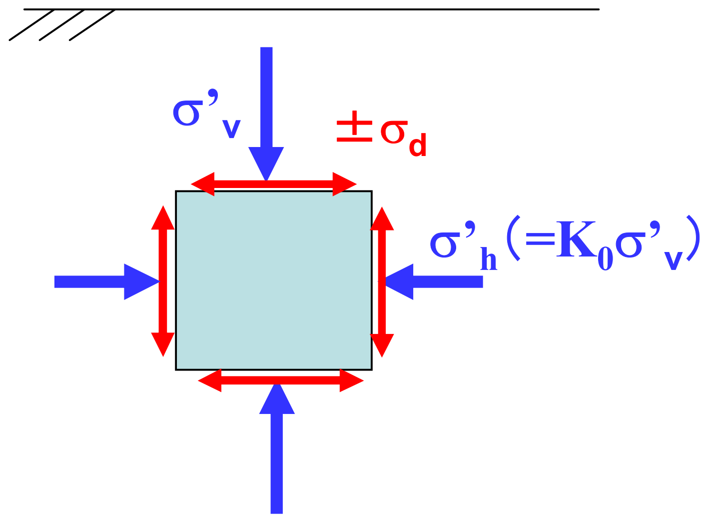
$$\frac{1+2K_0}{3}\frac{\sigma_{dl}}{2\sigma'_c} = \boxed{\frac{2}{3}}\frac{\sigma_{dl}}{2\sigma'_c}$$

Assignment (due 17 May)

Explain why the normal cyclic triaxial test on isotropically consolidated sample can be used for evaluating liquefaction behaviour of anisotropically consolidated soils in a level ground



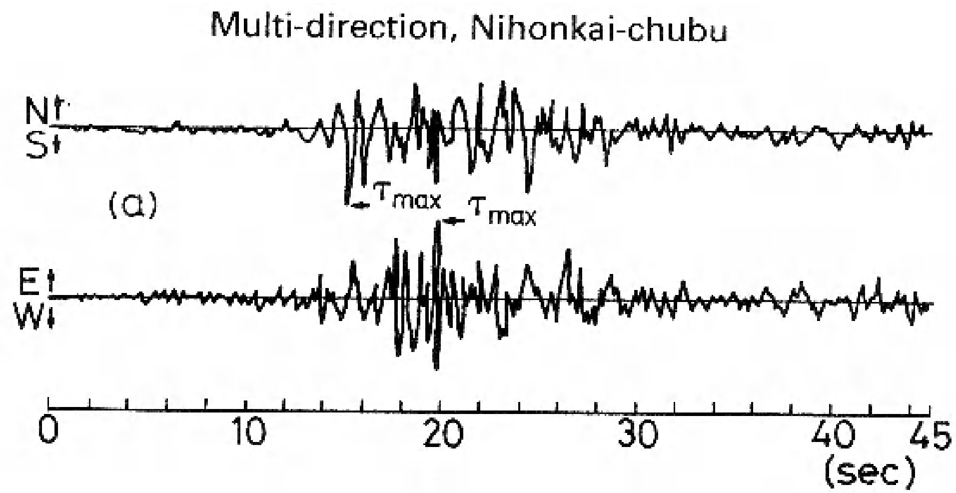
normal cyclic triaxial test on isotropically consolidated sample



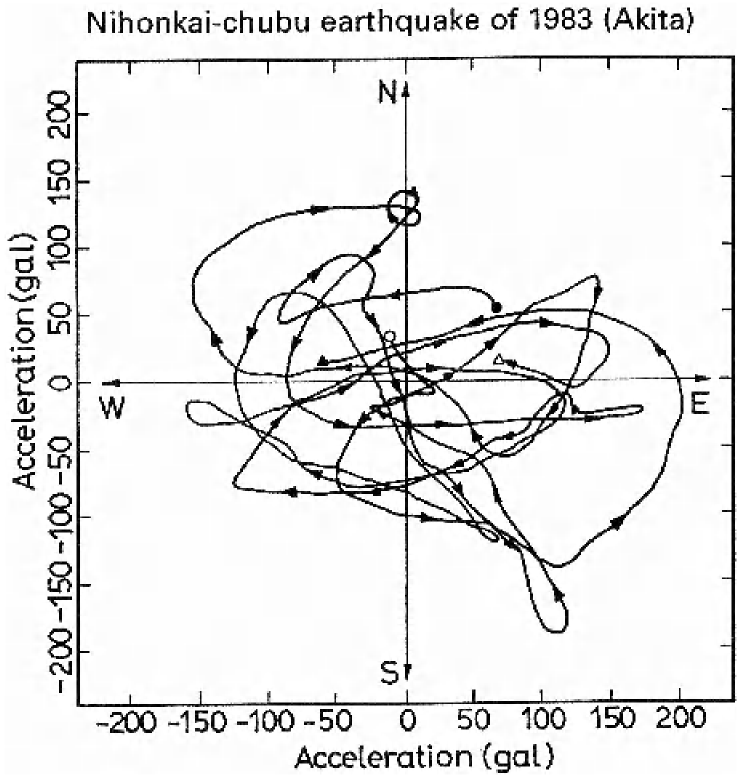
Stress condition of anisotropically consolidated soils in level ground

Effect of irregular seismic loading
irregular motion & multi-directionality

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



Irregular time history



Trajectory in plan of acceleration time history

Two types of seismic motion:

•Vibration type(振動型):

3 =< (number of τ_d >0.6τ_{max} before τ_{max})

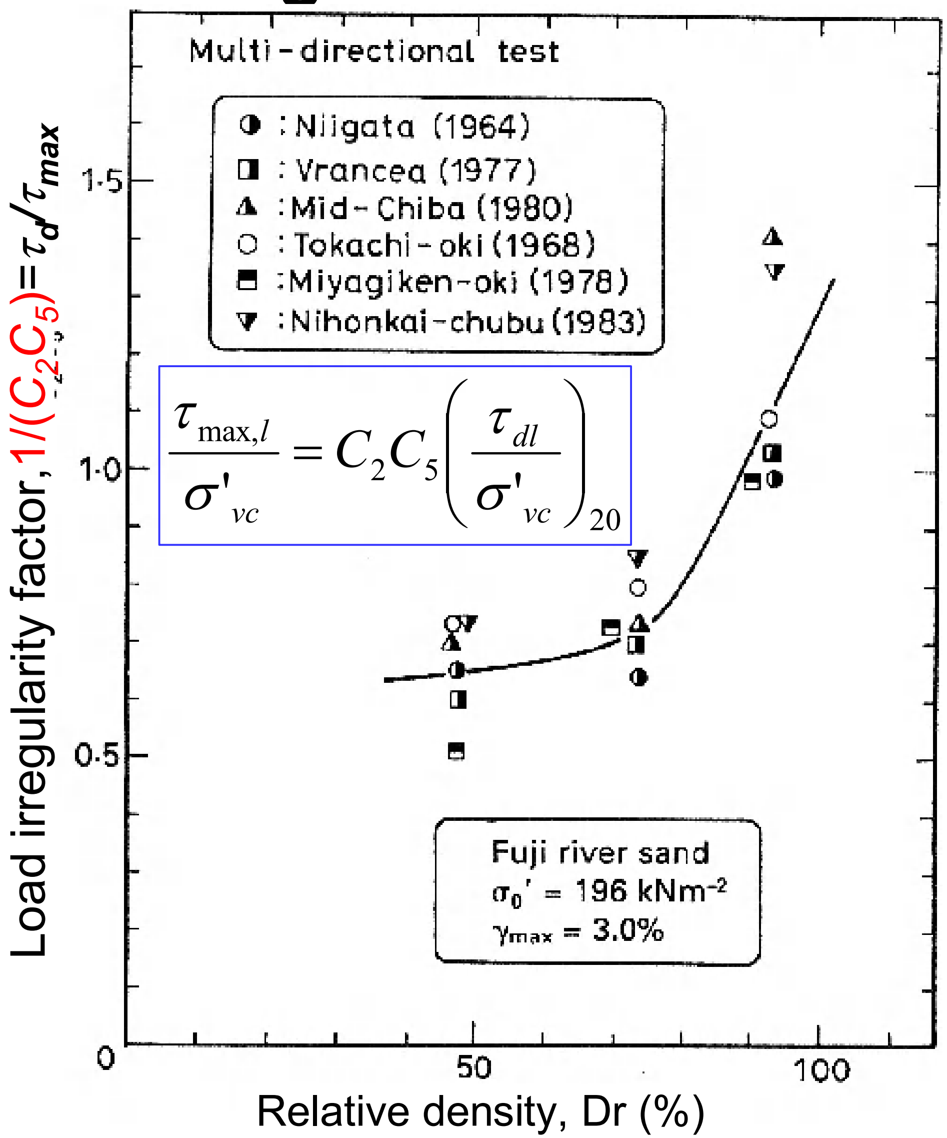
•Impact type(衝撃型):

2 >= (number of τ_d >0.6τ_{max} before τ_{max})

Random $(\tau_{max,l} > (\tau_{d,l})_{20})$ ***uniform***

1D $(\tau_{max,l} > \tau_{max,l})$ ***2 D***

Effect of irregular seismic loading
irregular motion & multi-directionality (cont.)



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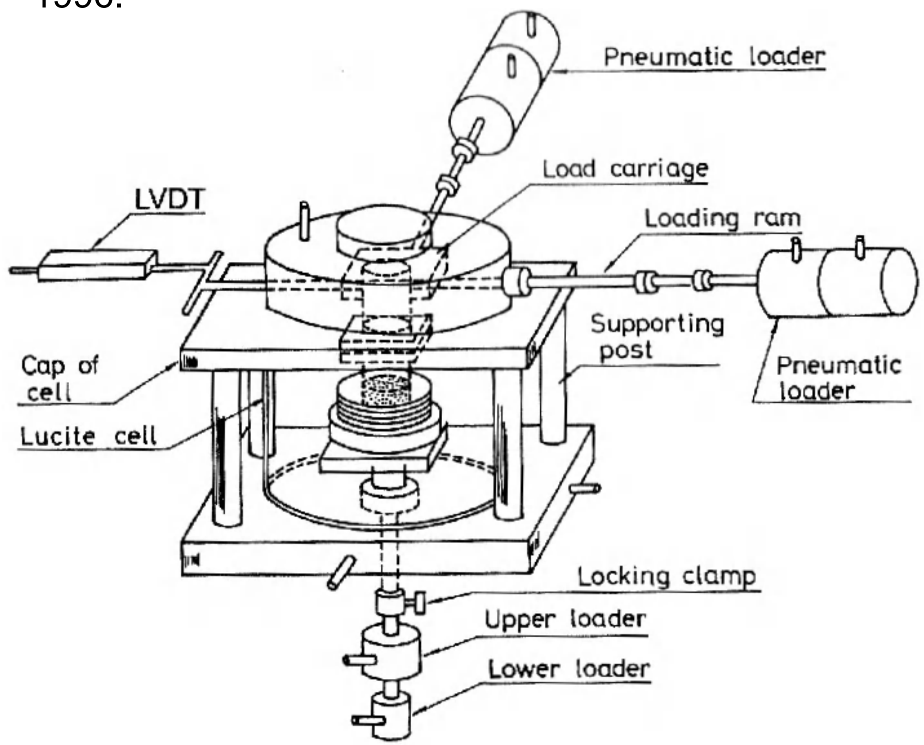
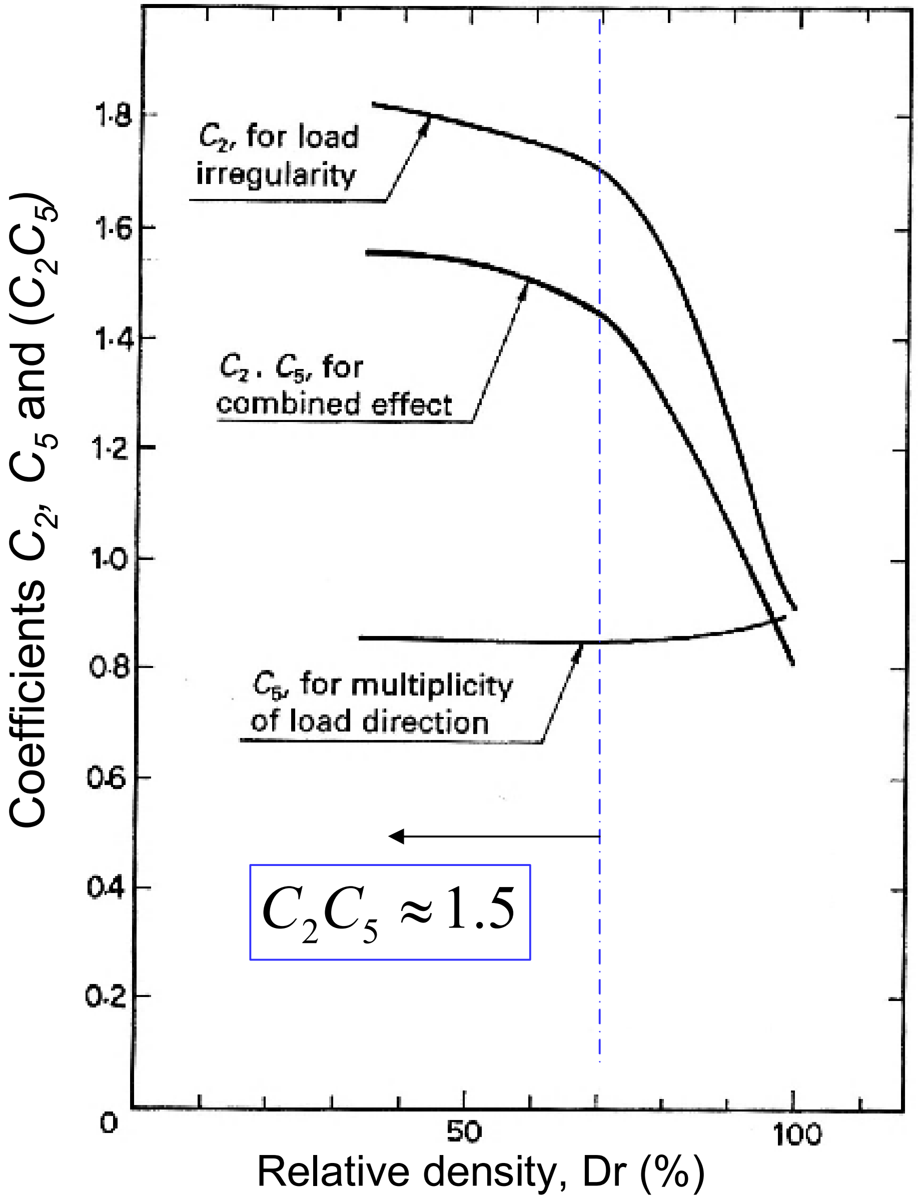


C_2: coefficient on load irregularity
C_5:coefficient on multi-directionality

Irregular motion & multi-directionality (cont.)



$$\frac{\tau_{\max,l}}{\sigma'_{vc}} = C_2 C_5 \left(\frac{\tau_{dl}}{\sigma'_{vc}}\right)_{20} = C_1 C_2 C_5 \left(\frac{\sigma_{dl}}{\sigma'_{c}}\right)_{20}$$

$$\frac{\tau_{dl}}{\sigma'_{vc}} = C_1 \frac{\sigma_{dl}}{2\sigma'_{c}}$$

$$C_1 \approx 2/3$$
$$C_2 C_5 \approx 1.5$$

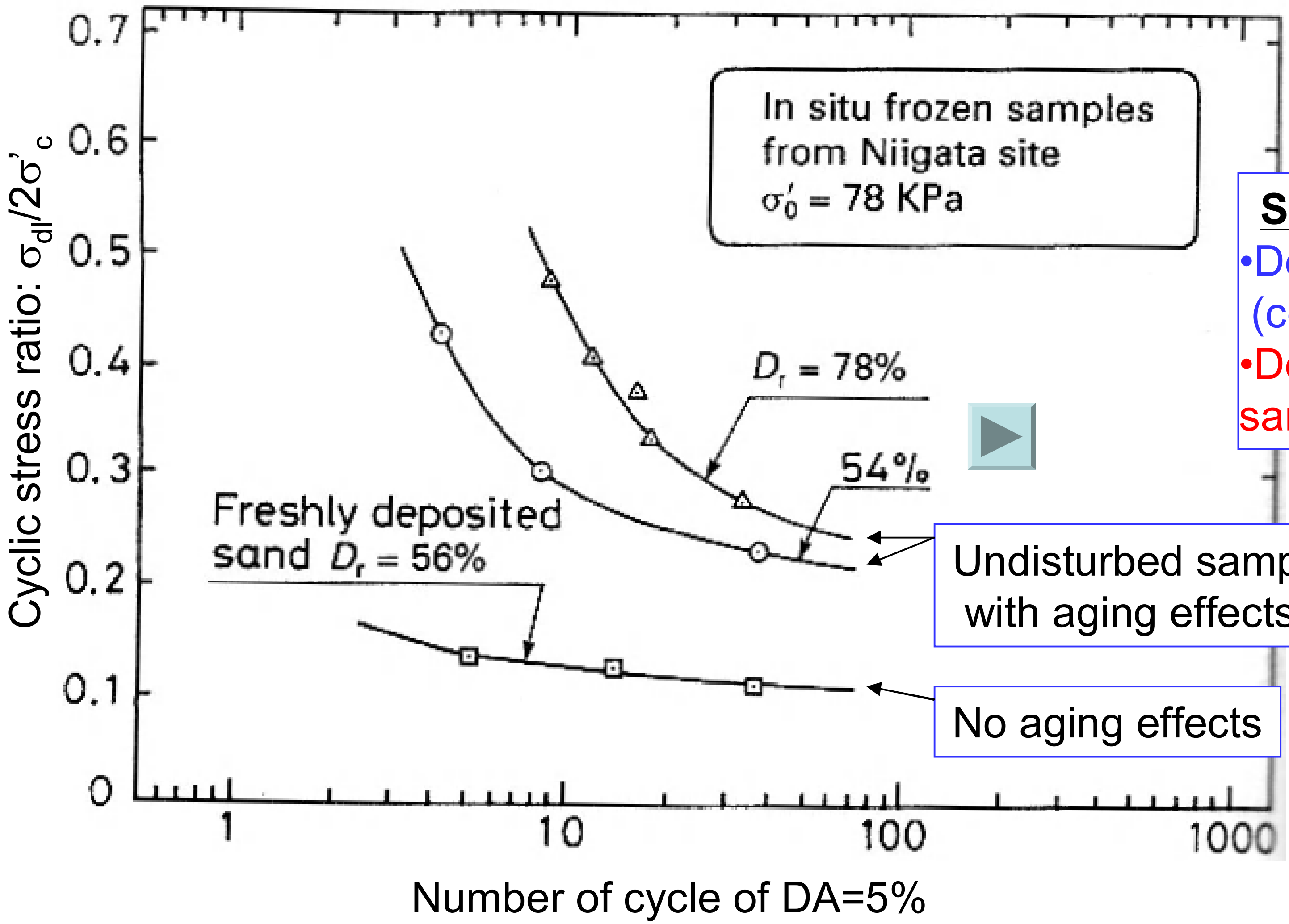
$$\frac{\tau_{\max,l}}{\sigma'_{vc}} \approx \left(\frac{\sigma_{dl}}{2\sigma'_{c}}\right)_{20}$$

Anisotropic C sample under random irregular loading (real condition)

Isotropic C sample under sinusoidal loading (cyclic triaxial test)

Sampling disturbance

Ishihara, Soil Behaviour in Earthquake Geotechnics, 1996.



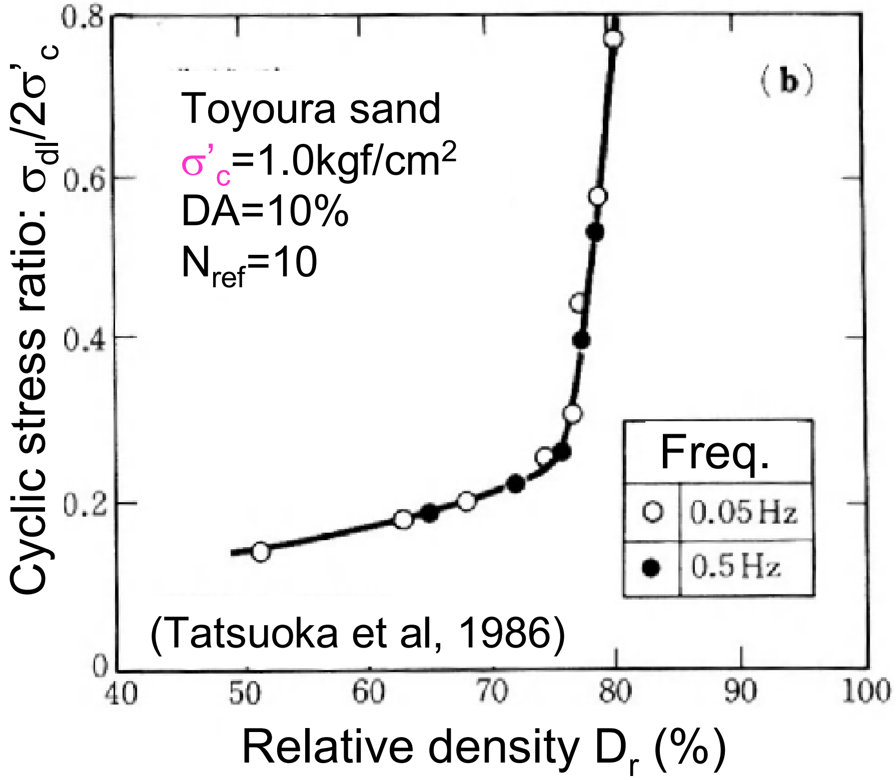
Insertion of sampler pre-shear

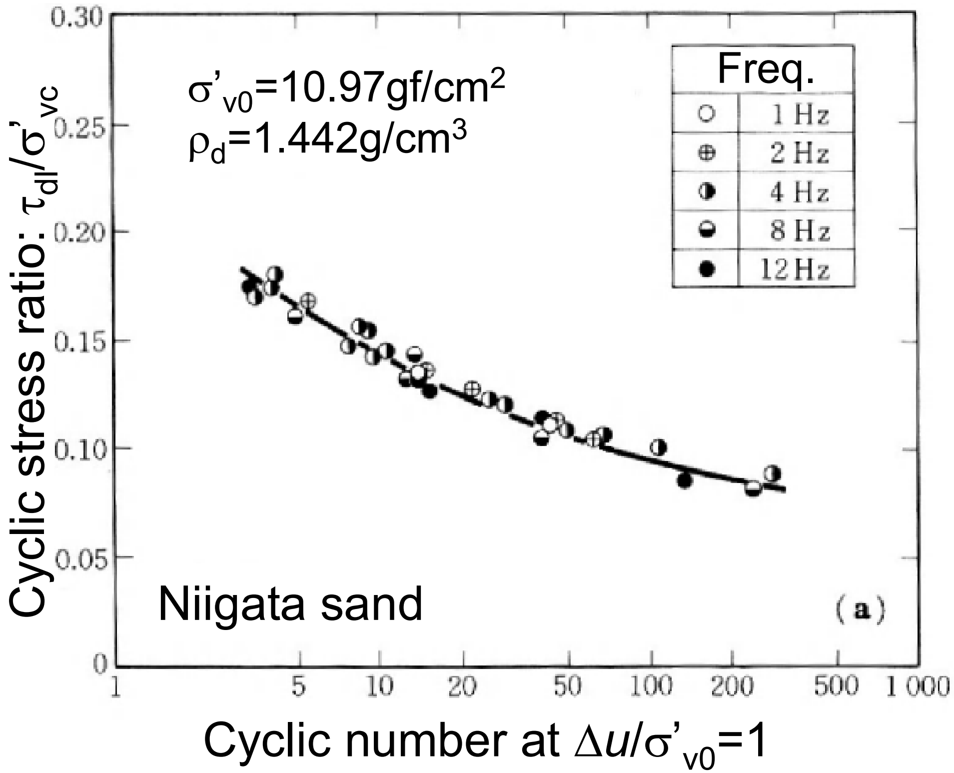
Sample disturbance

- Destroy aging effects (cementation)
- Densification for loose sand

C_3 & C_4 effects

Effect of Frequency





Evaluation of possibility of Liquefaction and its severeness

Liquefaction strength or resistance

\+

<u>Cyclic shear stress</u> and <u>cycles</u>

Eq. intensity

Eq. magnitude

Cyclic shear stress caused by earthquake

Equation of shear vibration

$$\frac{\gamma}{g}\frac{\partial^2 u}{\partial t^2} = \frac{\partial \tau}{\partial z}$$

$$\int_0^z \frac{\partial \tau}{\partial z} dz = \tau(z,t) = \int_0^z \frac{\gamma}{g}\frac{\partial^2 u}{\partial t^2} dz$$

where $\frac{\partial^2 u}{\partial t^2} = \alpha(z,t)$

If the ground is rigid, $u(z,t) \Rightarrow u(t)$

$$\tau(z,t) = \frac{\alpha(t)}{g}\int_0^z \gamma(z)dz = \frac{\alpha(t)}{g}\sigma_v$$

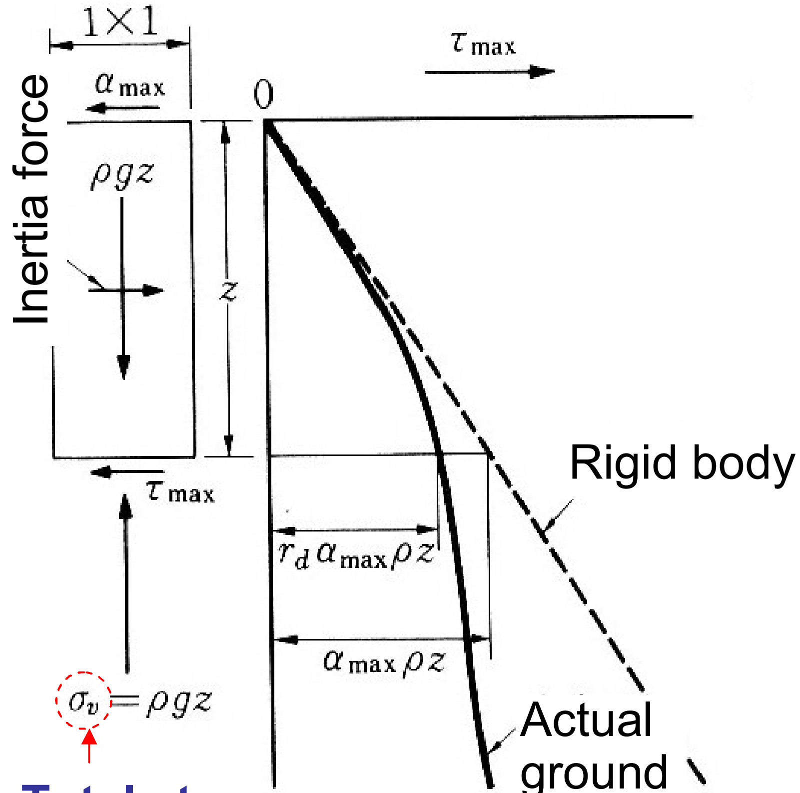
$$\tau^r_{\max} = Max\,\tau(z,t) = \frac{\alpha_{\max}}{g}\sigma_v$$

Total unit weight

$$\tau_{\max} = \frac{\alpha_{\max}}{g} r_d \gamma_t z$$

Shear stress ratio

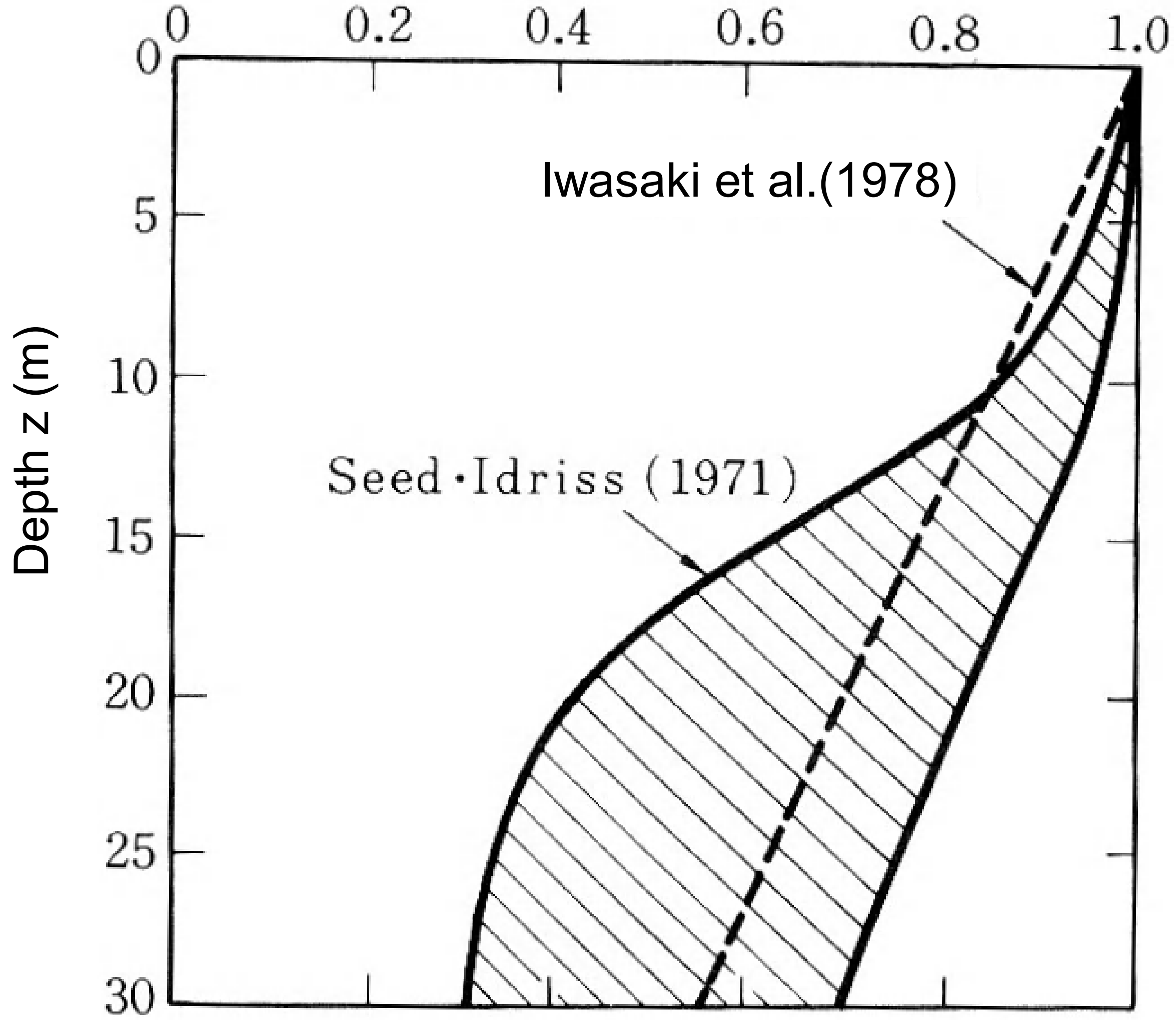
$$\frac{\tau_{\max}}{\sigma'_v} = \frac{\alpha_{\max}}{g} r_d \frac{\sigma_v}{\sigma'_v}$$



Total stress

Stress reduction factor

$$r_d = \frac{\tau_{\max}}{\left(\alpha_{\max}/g\right)\sigma_v}$$



Factor of safety: F_L

Liquefaction resistance $: R = \dfrac{\tau_{\max,l}}{\sigma'_v}$

Maximum stress ratio $: L = \dfrac{\tau_{\max}}{\sigma'_v} = \dfrac{\alpha_{\max}}{g} r_d \dfrac{\sigma_v}{\sigma'_v}$

$$\boxed{F_L = \frac{R}{L} = \frac{\tau_{\max,l} / \sigma'_v}{\tau_{\max} / \sigma'_v}}$$

- Reduction of mechanical properties of soils and design parameter

Liquefaction potential: P_L

$$P_L = \int_0^{20} (1 - F_L)\underline{(10 - 0.5z)}dz$$

Weighing coefficient for depth:
(*below 20m, no liquefaction*)

- Design of pile foundation
- Assessment of stability of oil tank

Assessment of F_L

Simplified methods

R: Field Test

(***N-value***, cone resistance)

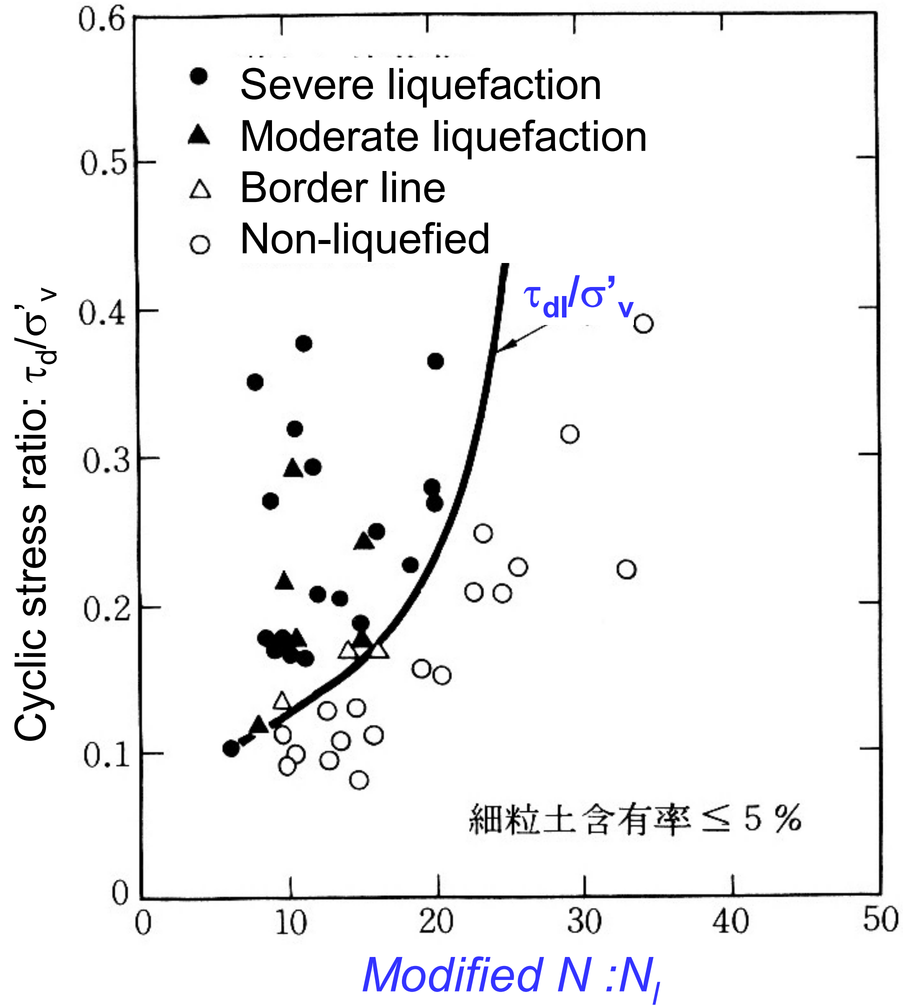
correction for various factors

- Seed' method:
- Specification of Highway Bridge
- Eq. Resist. Design of Port Facility (Critical N-value)

Precise method (not common)

R: sampling + lab test ***(?)***

L: dynamic response analysis



$$N_1 = \frac{1.7}{0.7+\sigma'_v}, \sigma'_v : kg/cm^2$$

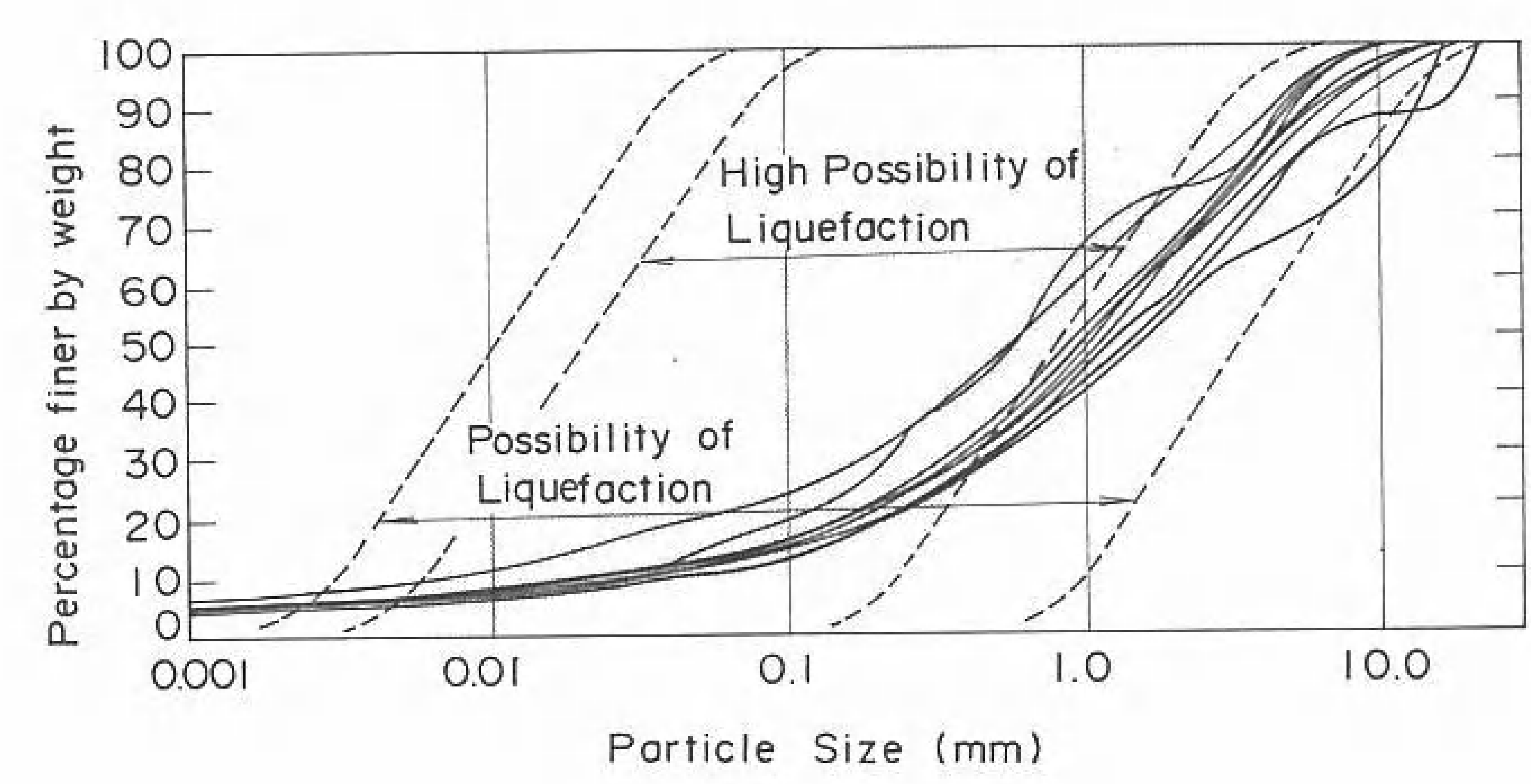
Conditions of Liquefaction

- **Cyclic shear deformation**
- Uniformly graded **sand ground??**
  - silt or clay exhibits no dilatancy behavior
  - liquefaction is not sustainable in gravel

    (high permeability prevents the accumulation of Δu)

    particle size and distribution
- **Loose ground**
  - loose grounds show large volume reduction
- High level of ground water table (**Saturation**)
  - liquefaction is not sustainable in dry condition
  - ground water keeps liquefaction

Grading curve of decomposed granite (Masado) used for landfill



Liquefiable Grounds and Soil Structures

- Loosely deposited sand at river side or sea side
- Reclaimed ground in port and harbor areas
- Embankments filled loosely
  - road, railway, river dike, housing lots, dam

Comparison of liquefaction assessment in Highway bridge specifications in 1990 and 1996

Liquefiable soil layer

1990 version	1996 version
Alluvial sand layers which satisfy 1. Water table less than 10m from the ground surface. 2. Depth less than 20m from GS 3. $0.02mm \leqq D_{50} \leqq 2.0mm$	Alluvial sand layers which satisfy 1. Water table less than 10m from the ground surface. 2. Depth less than 20m from GS 3. Either $F_c \leqq 35\%$ or $I_p \leqq 15$ 4. $D_{50} \leqq 10mm$ and $D_{10} \leqq 1mm$

Liquefaction occurrence

Liquefy when $F_L \leqq 1$, where $F_L = R/L$

Comparison of liquefaction assessment in Highway bridge specifications in 1990 and 1996 (cont.)

<u>Cyclic Shear resistance ratio, *R*</u> **1990 version**

$$R = C_1 \cdot C_2 \cdot C_3 \cdot C_4 \cdot C_5 \cdot R_L$$

$$R_L = R_1 + R_2 + R_3$$

R_L: liquefaction strength obtained by triaxial test

$$R_1 = 0.0882\sqrt{\frac{N}{\sigma'_v + 0.7}}$$

σ'$_v$ => unit: kgf/cm^2

C_1, C_2, C_3, C_4, C_5 :

correction factors

$$R_2 = \begin{cases} 0.19 & (0.02mm \le D_{50} \le 0.05mm) \\ 0.225\log_{10}\left(\dfrac{0.35}{D_{50}}\right) & (0.05mm \le D_{50} \le 0.6mm) \\ -0.05 & (0.6mm \le D_{50} \le 2.0mm) \end{cases}$$

$$R_3 = \begin{cases} 0.0 & (0\% \le F_c \le 40\%) \\ 0.004F_c - 0.16 & (40\% \le F_c \le 100\%) \end{cases}$$

Comparison of liquefaction assessment in Highway bridge specifications in 1990 and 1996 (cont.)

Cyclic Shear resistance ratio, *R* **1996 version**

$$R = C_1 \cdot C_2 \cdot C_3 \cdot C_4 \cdot C_5 \cdot R_L$$

R_L: liquefaction strength obtained by triaxial test

C_1, C_2, C_3, C_4, C_5 :
correction factors

$$R_L = \begin{cases} 0.0882\sqrt{\dfrac{N_a}{1.7}} & (N_a < 14) \\ 0.0882\sqrt{\dfrac{N_a}{1.7}} + 1.6\times10^{-6}\cdot(N_a - 14)^{4.5} & (14 \le N_a) \end{cases}$$

Sandy soils

N_1:Equivalent N-value at 1kgf/cm² of effective overburden stress (σ'_v)

$$N_a = aN_1 + b$$

N_a: Corrected N-value

$$N_1 = 1.7N/(\sigma'_v + 0.7)$$

$$a = \begin{cases} 1 & (0\% \le F_c < 10\%) \\ (F_c + 40)/50 & (10\% \le F_c < 60\%) \\ F_c/20 - 1 & (60\% \le F_c) \end{cases}$$

$$b = \begin{cases} 0 & (0\% \le F_c < 10\%) \\ (F_c - 10)/18 & (10\% \le F_c) \end{cases}$$

Gravelly soils

$$N_a = \left(1 - 0.36\log_{10}(D_{50}/2)\right)N_1$$

Comparison of liquefaction assessment in Highway bridge specifications in 1990 and 1996 (cont.)

Correction from the strength under triaxial conditions to that in-situ

1990 version

$$C_1 \cdot C_2 \cdot C_3 \cdot C_4 \cdot C_5 = 1.0$$

C_1: difference of confining stress between triaxial condition (isotropic) and in-situ(K_0)
C_2: Irregularity of earthquake wave
C_3: Disturbance from sampling to test
C_4: Densification from sampling to test
C_5: Multi-directional characteristics of ground shaking

1996 version

C_1, C_2, C_3, C_4, C_5: same as1990 version

$C_3 \cdot C_4 \cdot C_5 = 1.0$
$C_w = C_1 \cdot C_2$

<Type -1 earthquake motion> tectonic type

$C_w = 1.0$

<Type -2 earthquake motion> near-field

$$C_w = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 \leq R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases}$$

Comparison of liquefaction assessment in Highway bridge specifications in 1990 and 1996 (cont.)

<u>Shear stress ratio during an earthquake, *L*</u>

1990 version

$$L = r_d \cdot k_s \cdot \frac{\sigma_v}{\sigma'_v}$$

$$k_s = c_z \cdot c_G \cdot c_I \cdot k_{s0}$$

r_d: Reduction factor of the shear stress ratio with depth

k_s: Horizontal seismic coefficient at the ground surface

c_Z: Seismic zone factor

c_G: Ground condition factor

c_I: Factor on the importance of the structure

k_{s0}: Standard horizontal seismic coefficient (0.15)

1996 version

$$L = r_d \cdot k_{hc} \cdot \frac{\sigma_v}{\sigma'_v}$$

$$k_{hc} = c_z \cdot k_{s0}$$

k_{hc}: Horizontal seismic coefficient at the ground surface

c_Z: Seismic zone factor

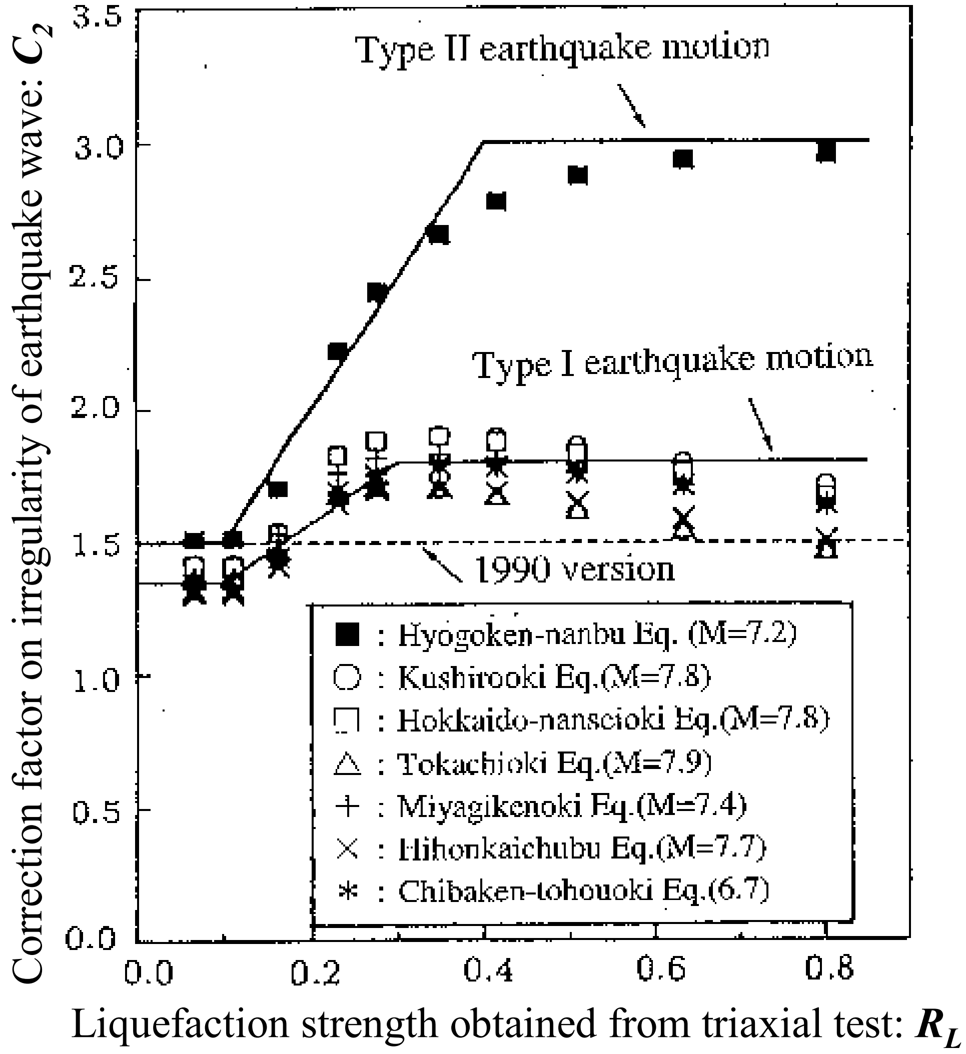
c_G: Ground condition factor

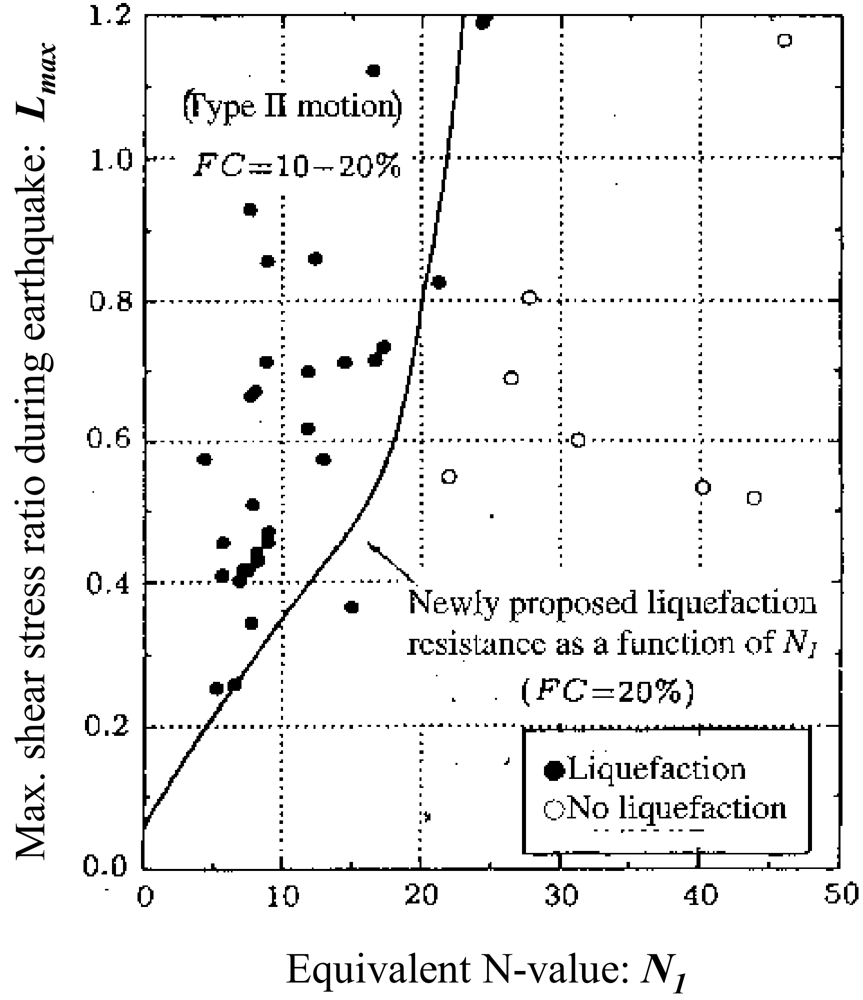
k_{s0}: Standard horizontal seismic coefficient shown in the following table

hard ⇔ ***soft***

Ground type	Type 1	Type 2	Type 3
Type 1 ground motion	0.30	0.35	0.40
Type 2 ground motion	0.80	0.70	0.60

Back data of 1996 version





Sand sampling

Method of sampler penetration：

Loose sand=> pushing

Hard, dense sand =>rotary sampling

Mechanical properties of the sand sampled by these methods are different from those of ideal ones.

Properties of sand evaluated from sounding, field tests (e.g., SPT=>N-value)

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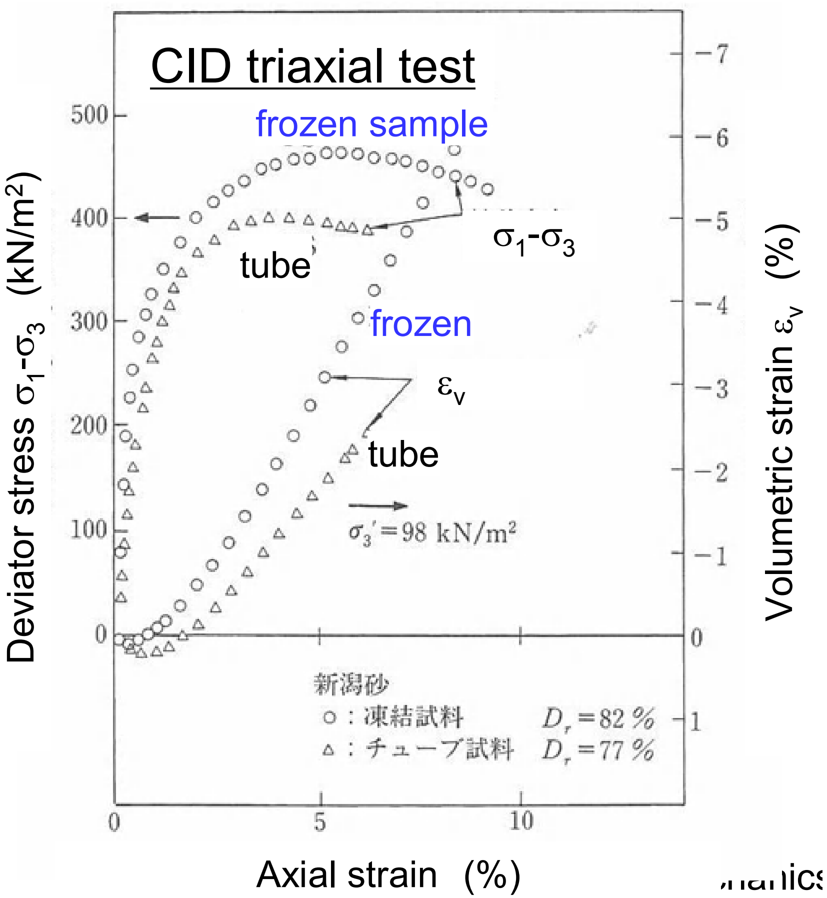

To obtain the reliable properties

Freezing sampling

(凍結サンプリング)

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