International Symposium
Qualification of dynamic analyses of dams and their equipments
and of probabilistic assessment seismic hazard in Europe
31th August – 2nd September 2016 – Saint-Malo

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Tokyo University of Science, Japan

Session : 3

Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability

Saint-Malo © Yannick LE GAL

SUMMARY-1

Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

●Practical simplified seismic stability analysis needs appropriate balance among the methods chosen in the following items:

1) Criterion to evaluate of the stability:
   Global safety factor relative to a specified required minimum vs.
   Residual deformation relative to a specified allowable largest.
2) Design seismic load at a given site:
   Conventional design load vs. Likely largest load in the future
3) Stress – strain properties of soil:
   Actual complicated behavior vs. Simplified model
4) Relevant consideration of the effects of other engineering factors:
   - compacted dry density; soil type; etc.

SUMMARY-2

Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

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2) Design seismic load at a given site:
   Conventional design load vs. Likely largest load in the future

3) Stress – strain properties of soil:
   Actual complicated behavior vs. Simplified model

Main topic in this presentation

4) Relevant consideration of the effects of other engineering factors:
 - compacted dry density; soil type; etc.

Stress – strain properties of soil: (A) actual complicated behaviour vs. (B) simplified model

(A) actual complicated behaviour

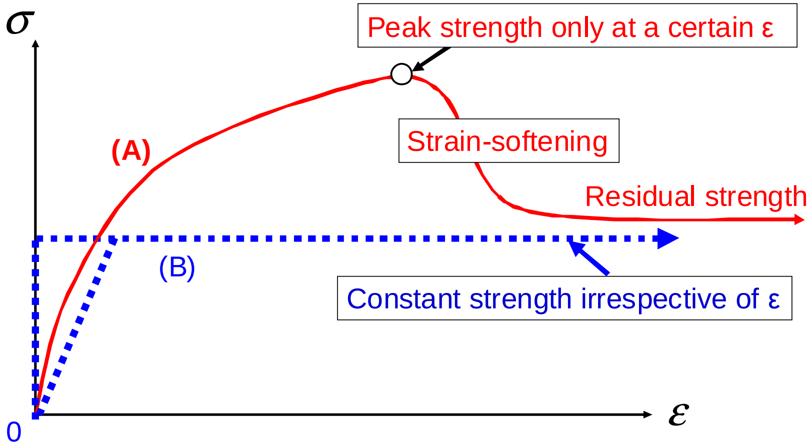
a) Peak strength corresponding to actual compacted dry density

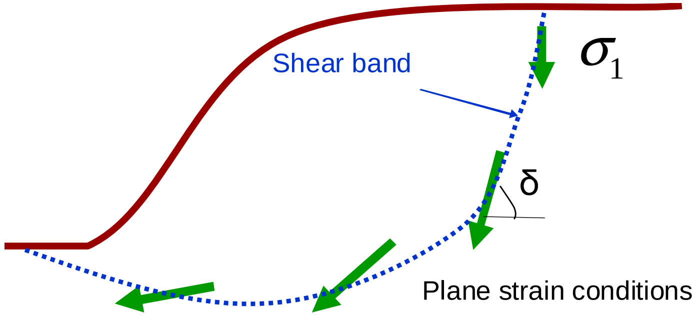
b) Anisotropic stress – strain properties as a function of δ

c) Plane strain condition in many cases

d) Strain-softening associated with shear banding with the thickness increasing with D_{50}

e) Progressive failure as a result of d) among others.





Stress – strain properties of soil: (A) actual complicated behaviour vs. (B) simplified model

(B) simplified model (explained in this presentation)

a) Design strength corresponding to conservatively (but not excessively) determined compacted dry density

b) Isotropic stress – strain properties

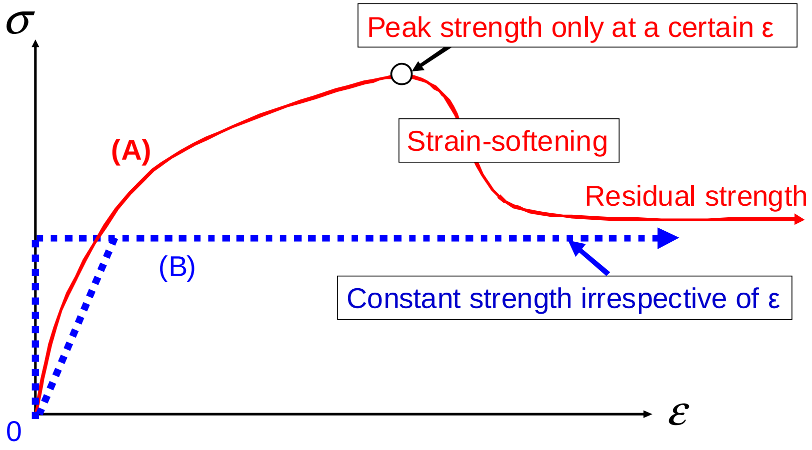
c) Strength by triaxial compression test at δ= 90°

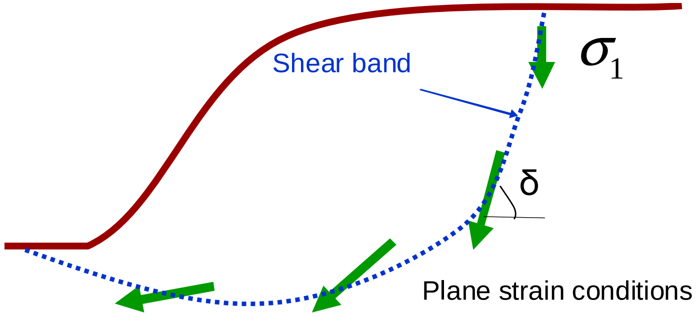
d) Strain-softening associated with shear banding with the thickness increasing D_{50} to account for the effects of compaction & particle size

e) No progressive failure in the limit equilibrium-based stability analysis

Good balance is required among simplifications a), b), c) and e)

d) is to encourage good compaction





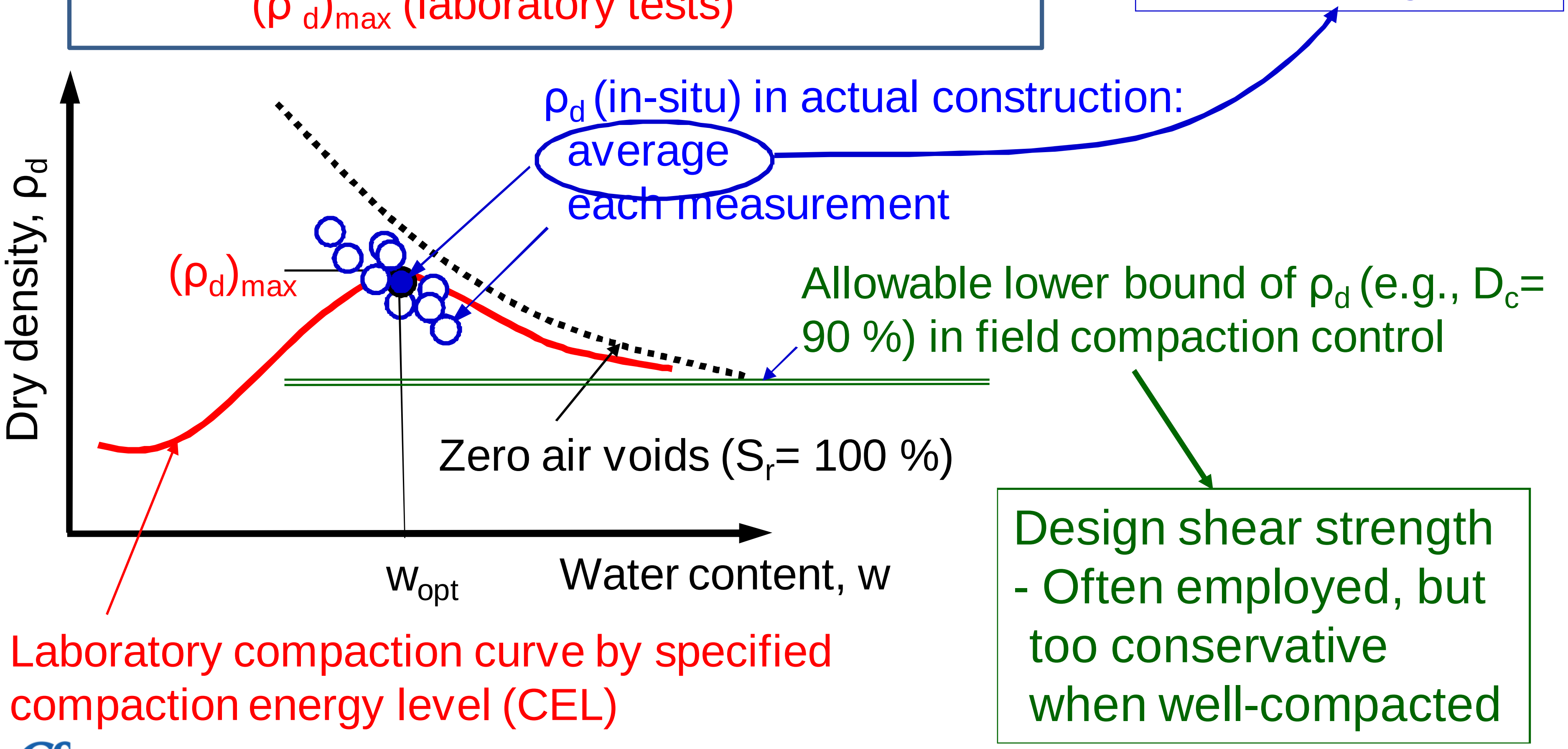
Discussions on these topics a) - e)

Conservative determination of design soil shear strength under drained conditions- 1

The degree of compaction

$$D_c = \frac{\rho_d \text{ (in-situ)}}{(\rho_d)_{max} \text{ (laboratory tests)}} \times 100\ (\%)$$

Average of actual shear strength



ρ_d (in-situ) in actual construction: average each measurement

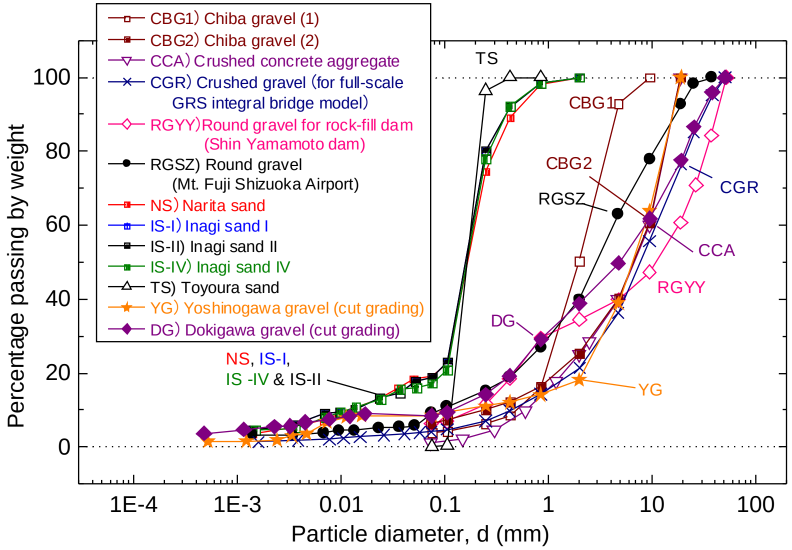
Zero air voids (S_r= 100 %)

Allowable lower bound of ρ_d (e.g., D_c= 90 %) in field compaction control

Laboratory compaction curve by specified compaction energy level (CEL)

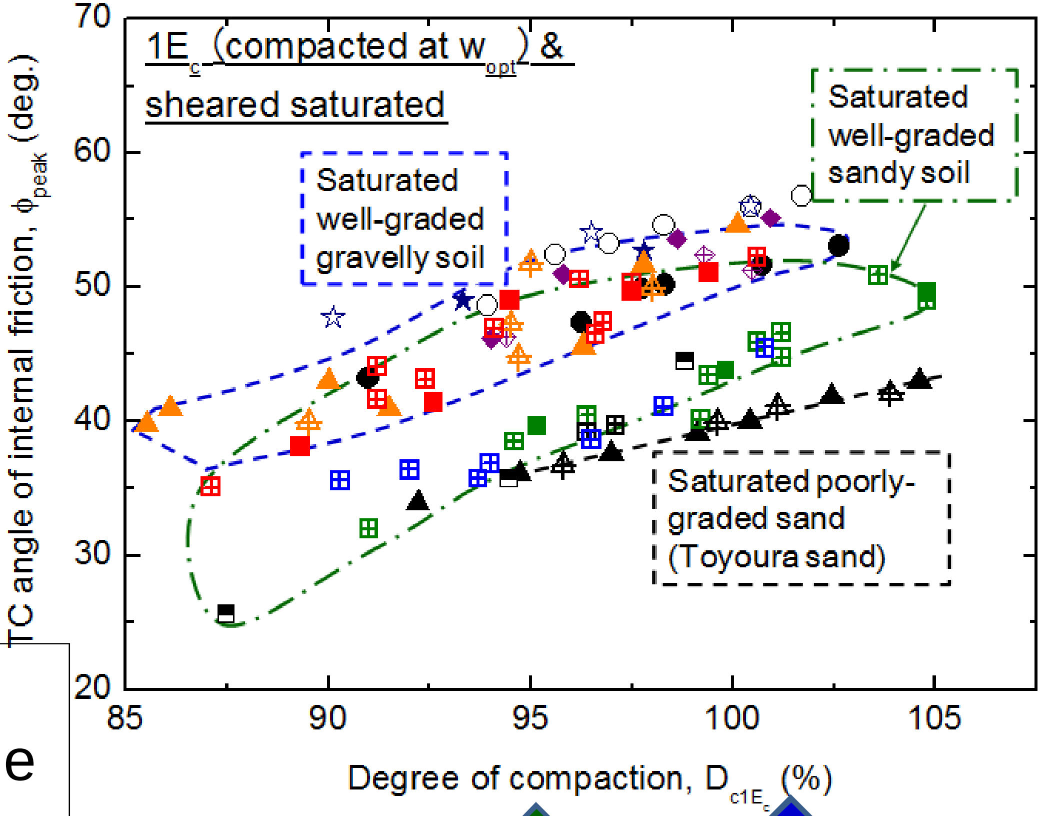
Design shear strength
- Often employed, but too conservative when well-compacted

Conservative determination of design soil shear strength under drained conditions- 2



Drained TC at σ'_3= 50 kPa

ϕ_{peak}= arcsin$[(\sigma'_1 - \sigma'_3)/[(\sigma'_1+\sigma'_3)]_{max}$



Design shear strength is often determined to correspond to the allowable lower bound of D_c used in field compaction control

⇒ conservative with better compacted soil

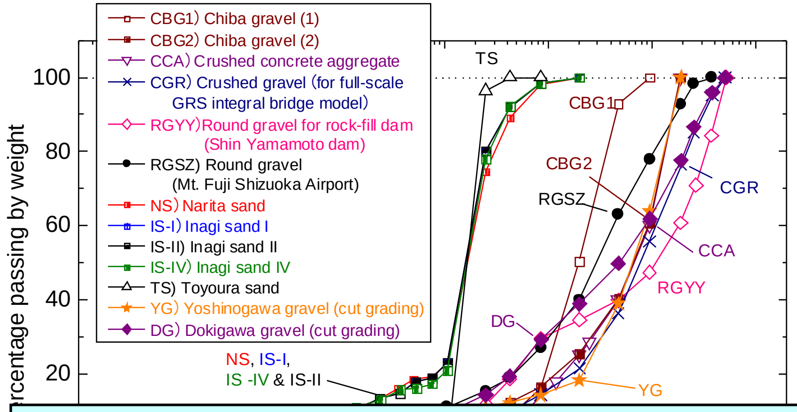
Typical allowable lower bound

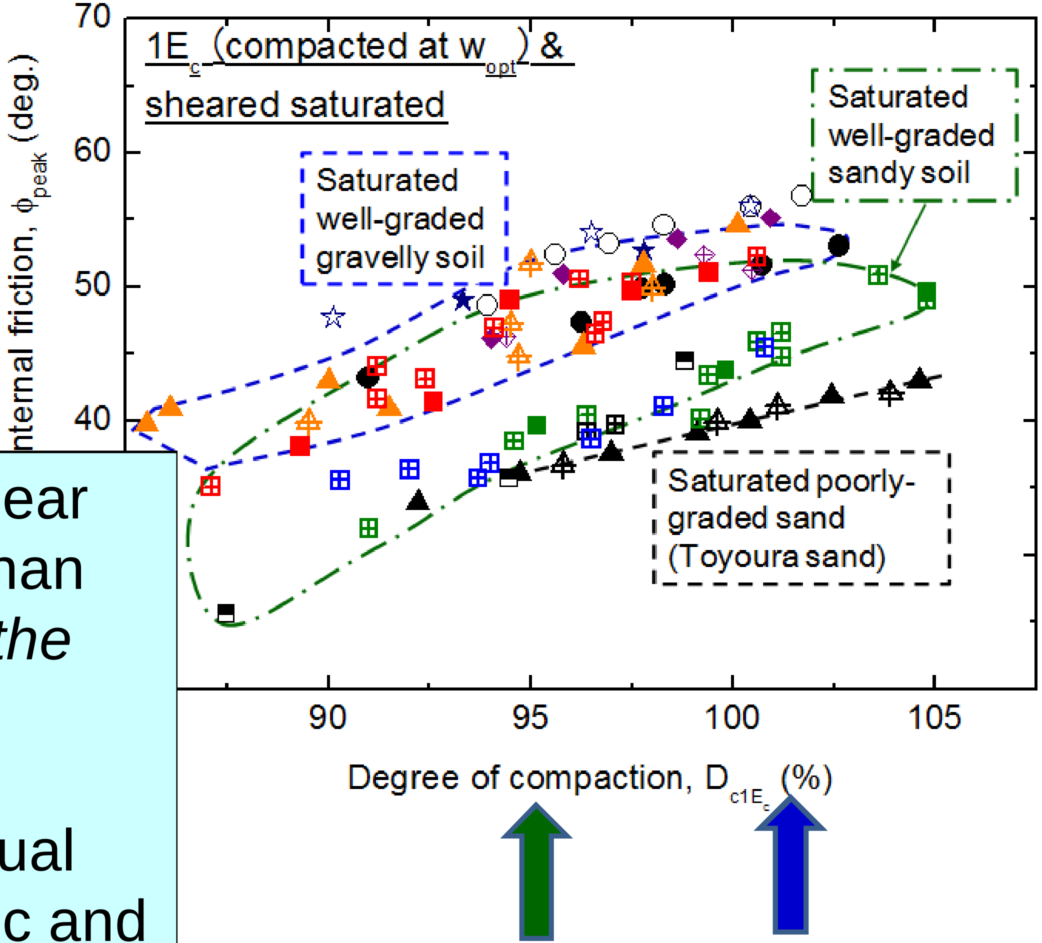
Average of actual values

Conservative determination of design soil shear strength under drained conditions- 2

Drained TC at $\sigma'_3 = 50$ kPa

$\phi_{peak} = \arcsin[(\sigma'_1 - \sigma'_3)/[(\sigma'_1 + \sigma'_3)]_{max}$





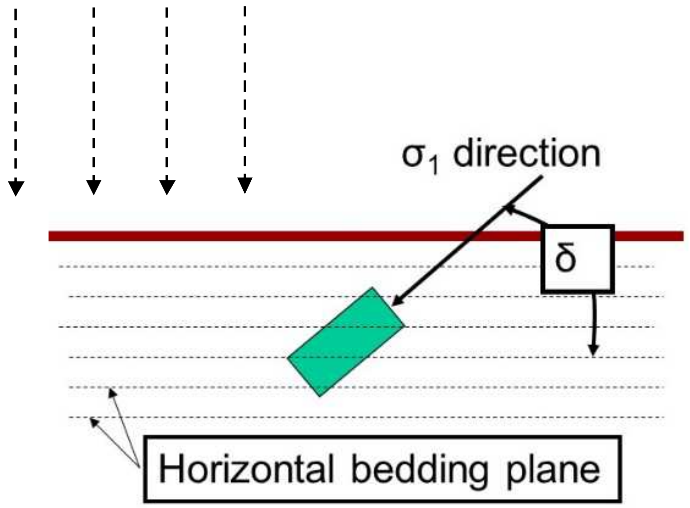
Typical allowable lower bound

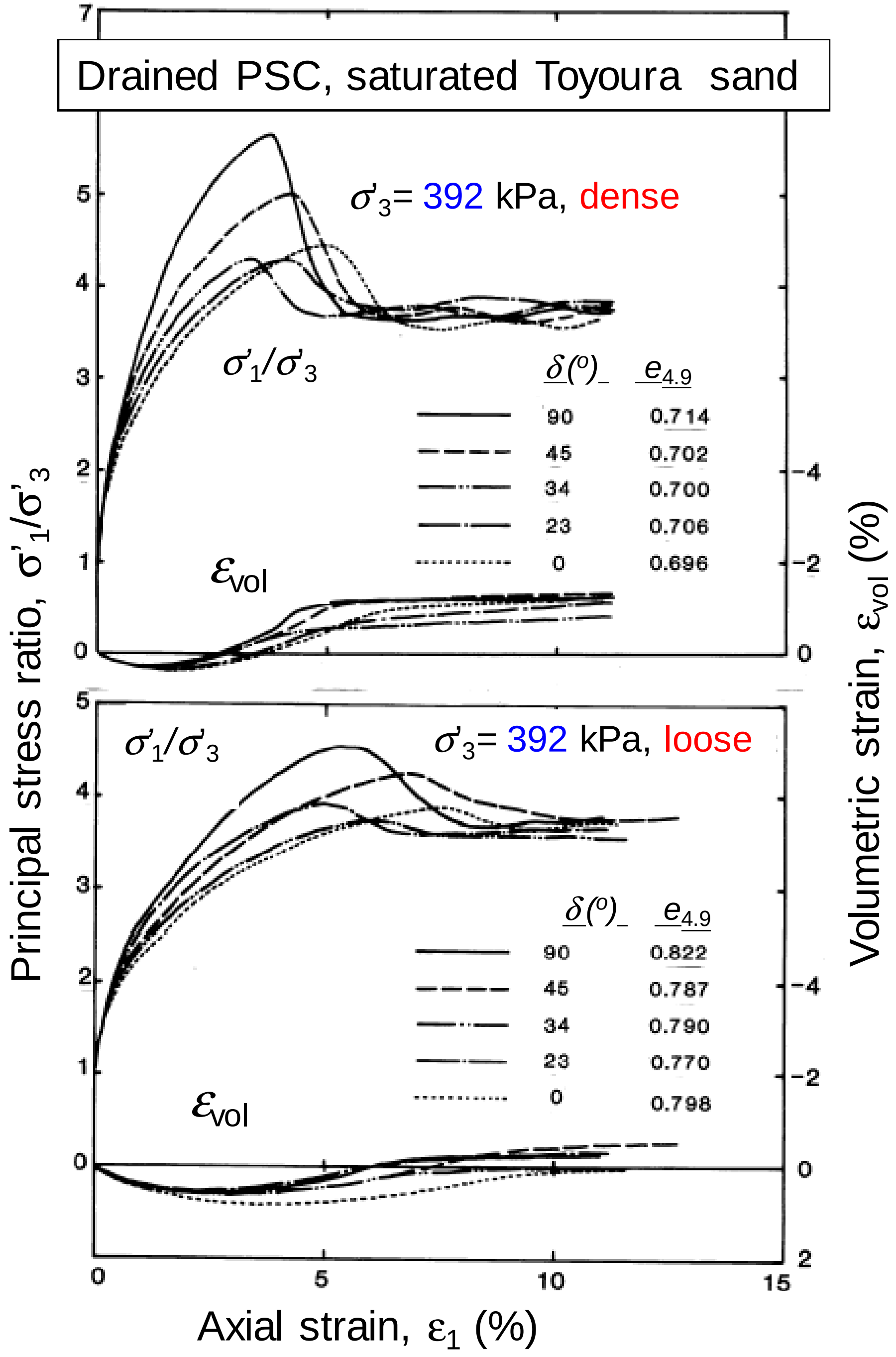
Average of actual values

The use of the design peak shear strength that is slightly lower than *the value that corresponds to the target of D_c set equal to the anticipated average of actual values*, together with the residual shear strength, is more realistic and can encourage better compaction (explained later)

Inherently anisotropic stress – strain behaviour under drained conditions- 1

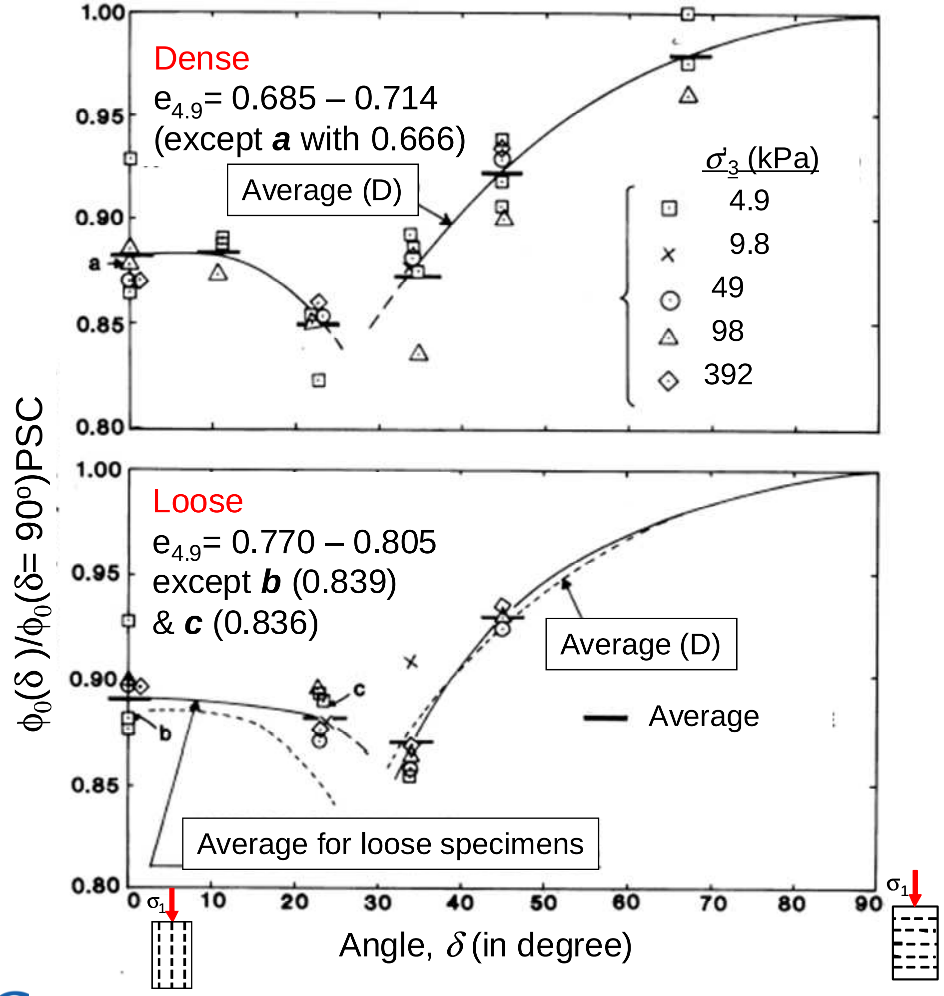
Pluviation through air

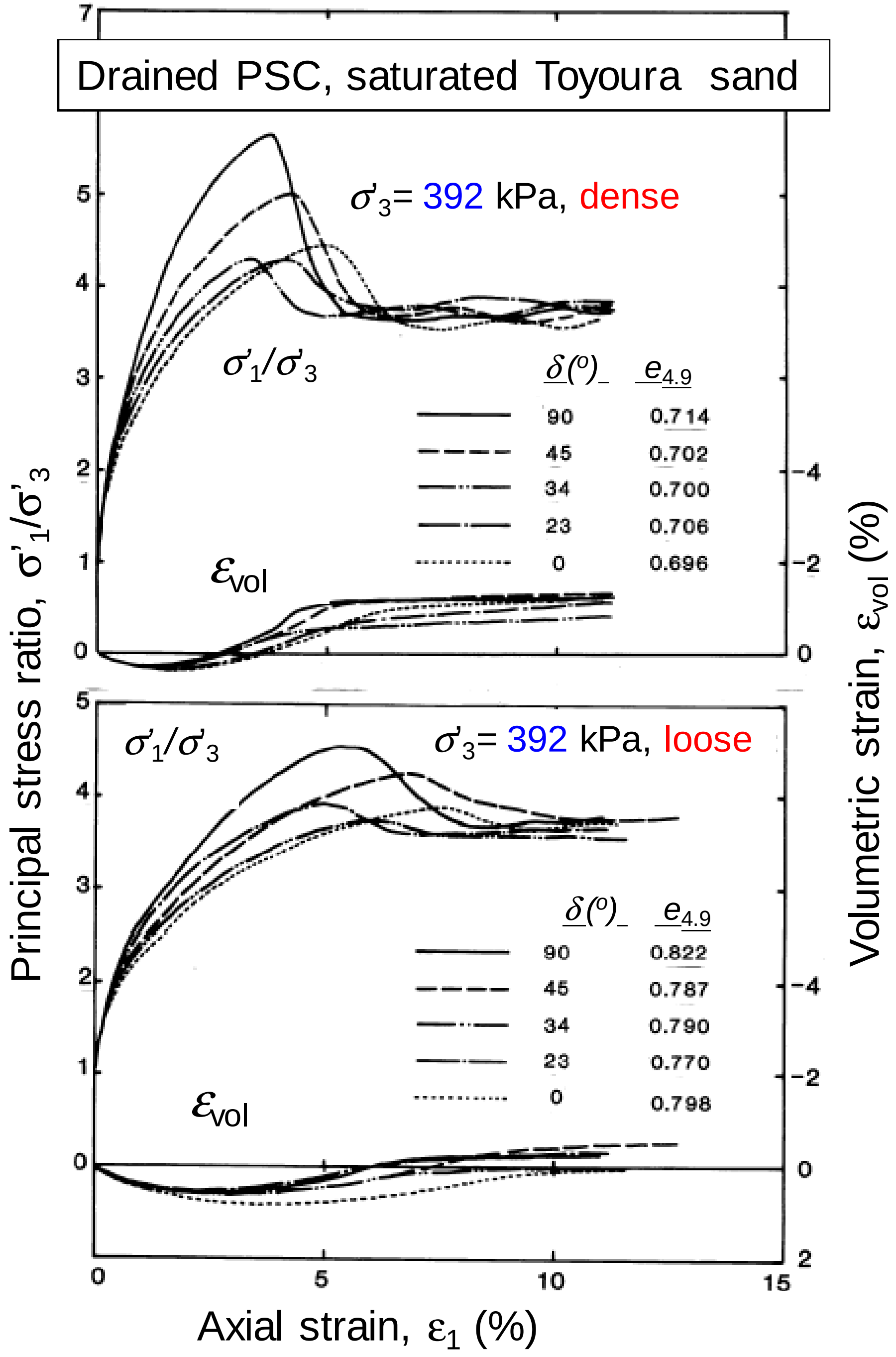




Inherently anisotropic stress – strain behaviour under drained conditions- 2

Summary

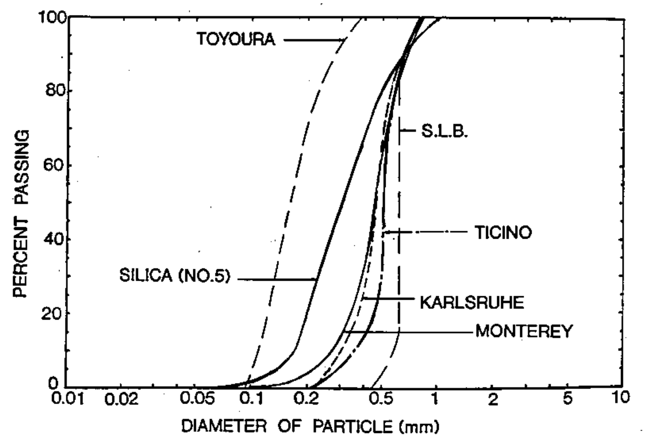


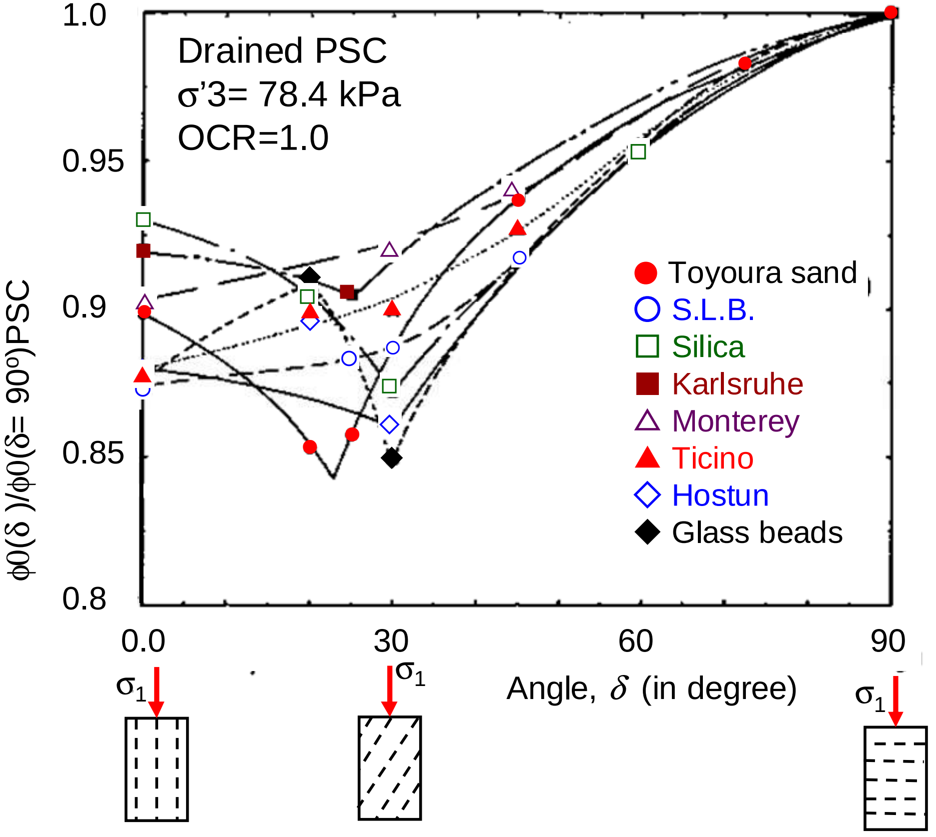


Inherently anisotropic stress – strain behaviour under drained conditions- 3

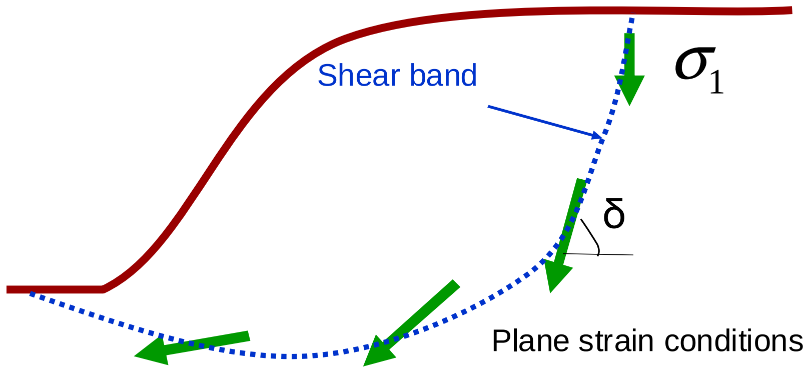
A similar trend among different poorly-graded sands collected from different countries,

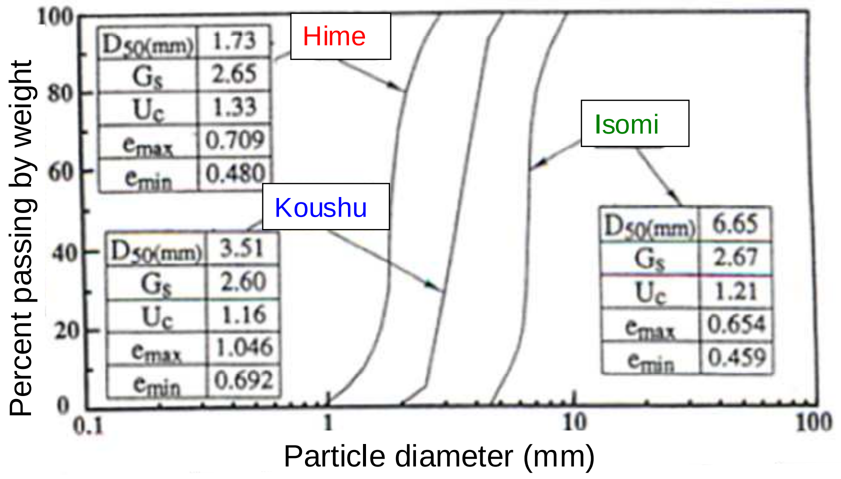
with and without a minimum at δ= 20^o – 30^o, where the shear band direction coincides with the bedding plane direction.

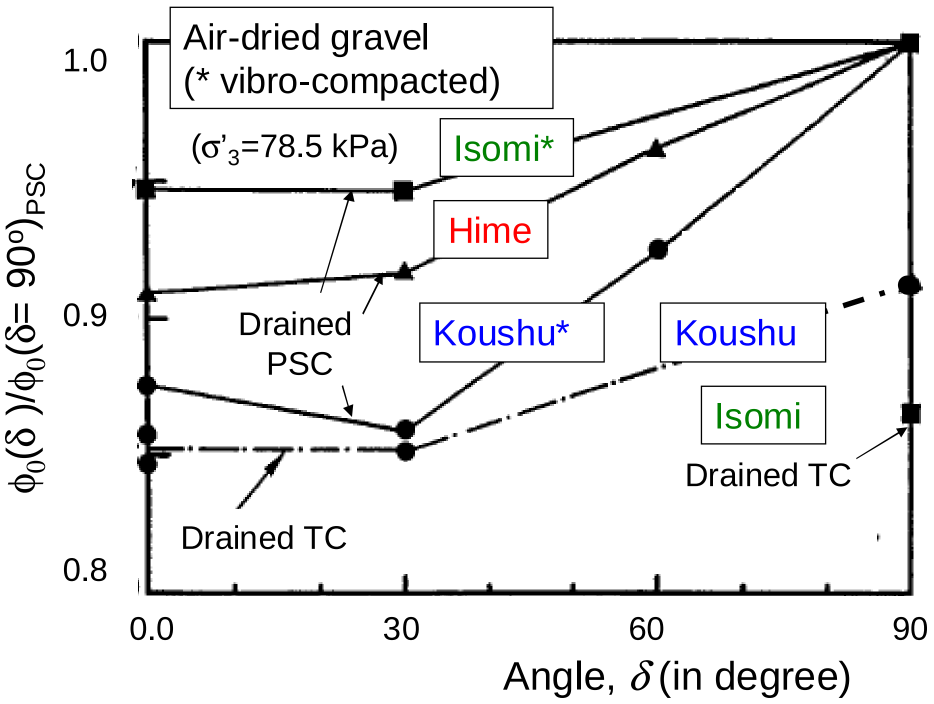




Inherently anisotropic stress – strain behaviour under drained conditions- 4

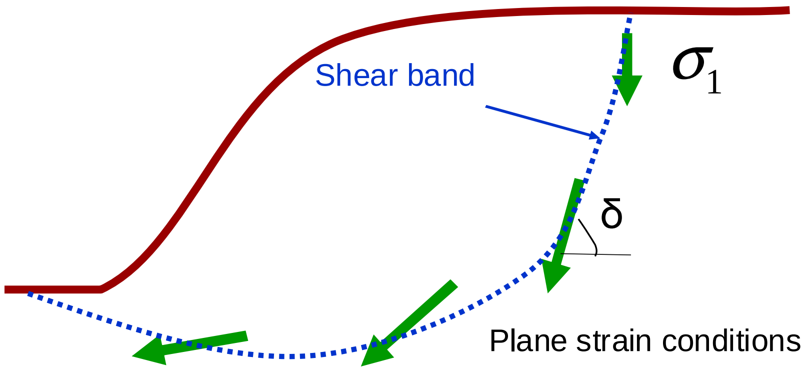


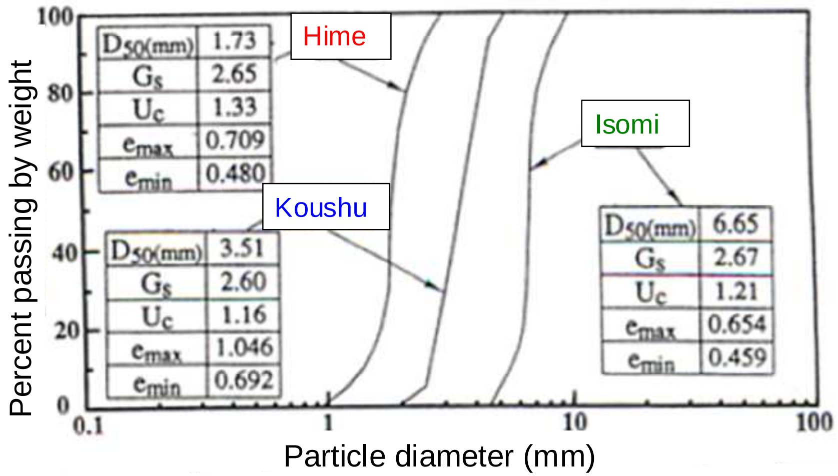


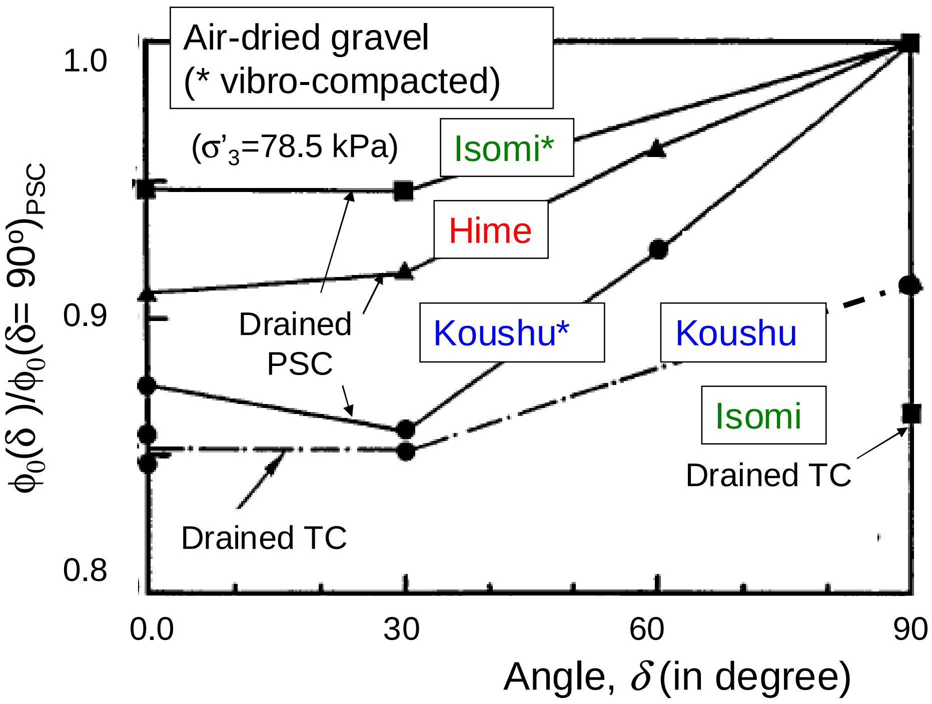


A similar trend among different poorly-graded gravelly soils produced by vertical vibratory compaction

Inherently anisotropic stress – strain behaviour under drained conditions- 5

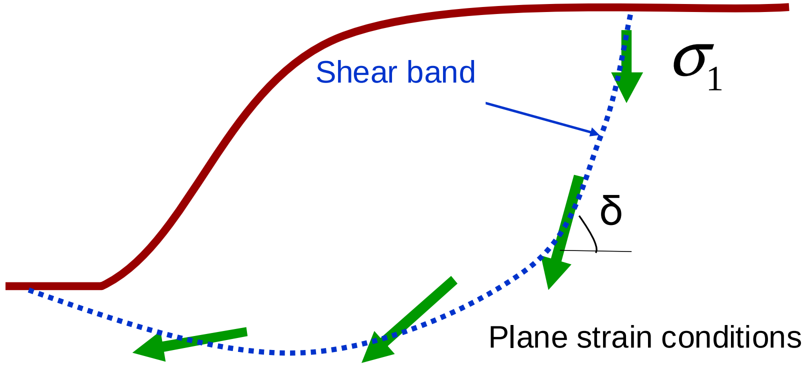


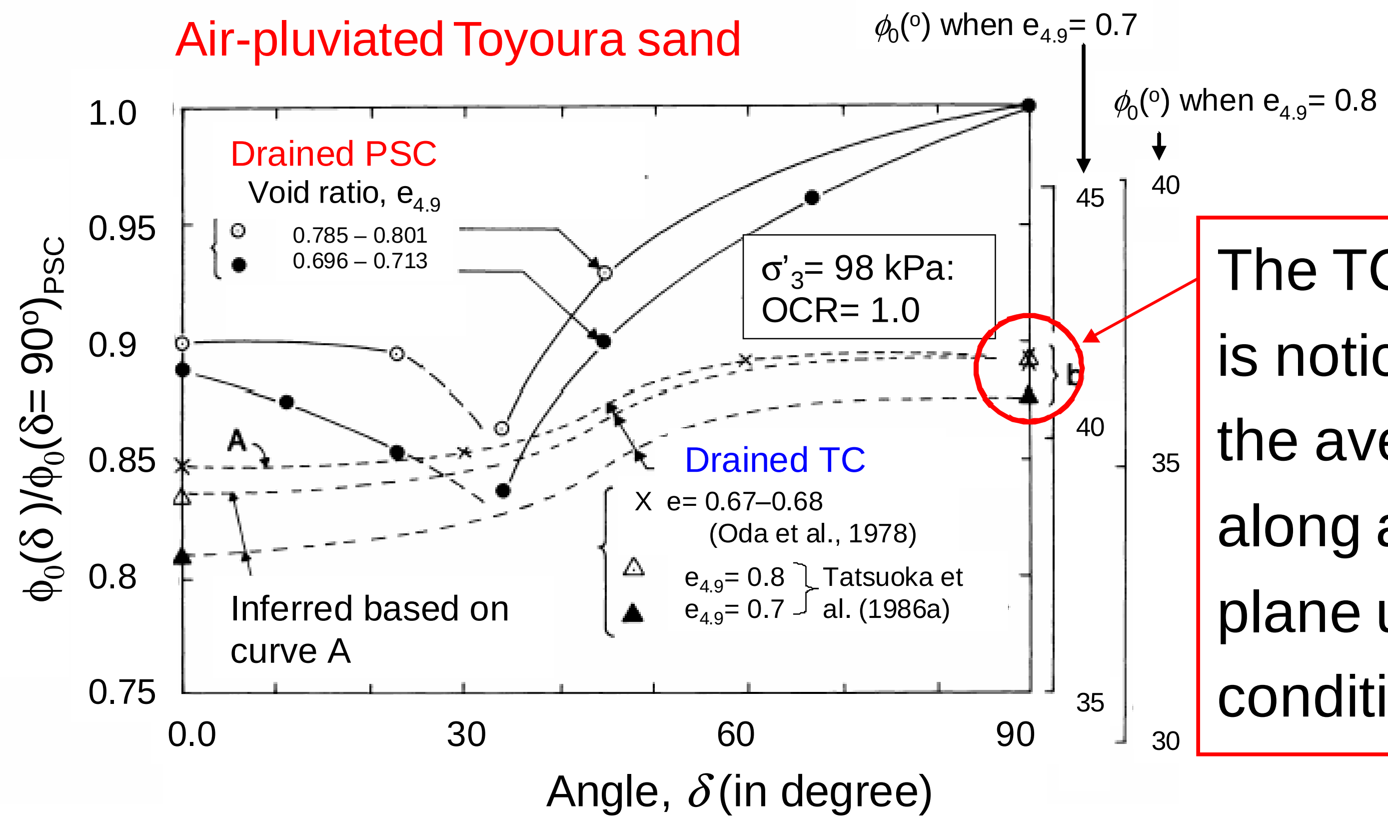




The TC strength (δ= 90º) is similar to, or smaller than, the average strength along a circular failure plane under plane strain conditions.

Inherently anisotropic stress – strain behaviour under drained conditions- 6



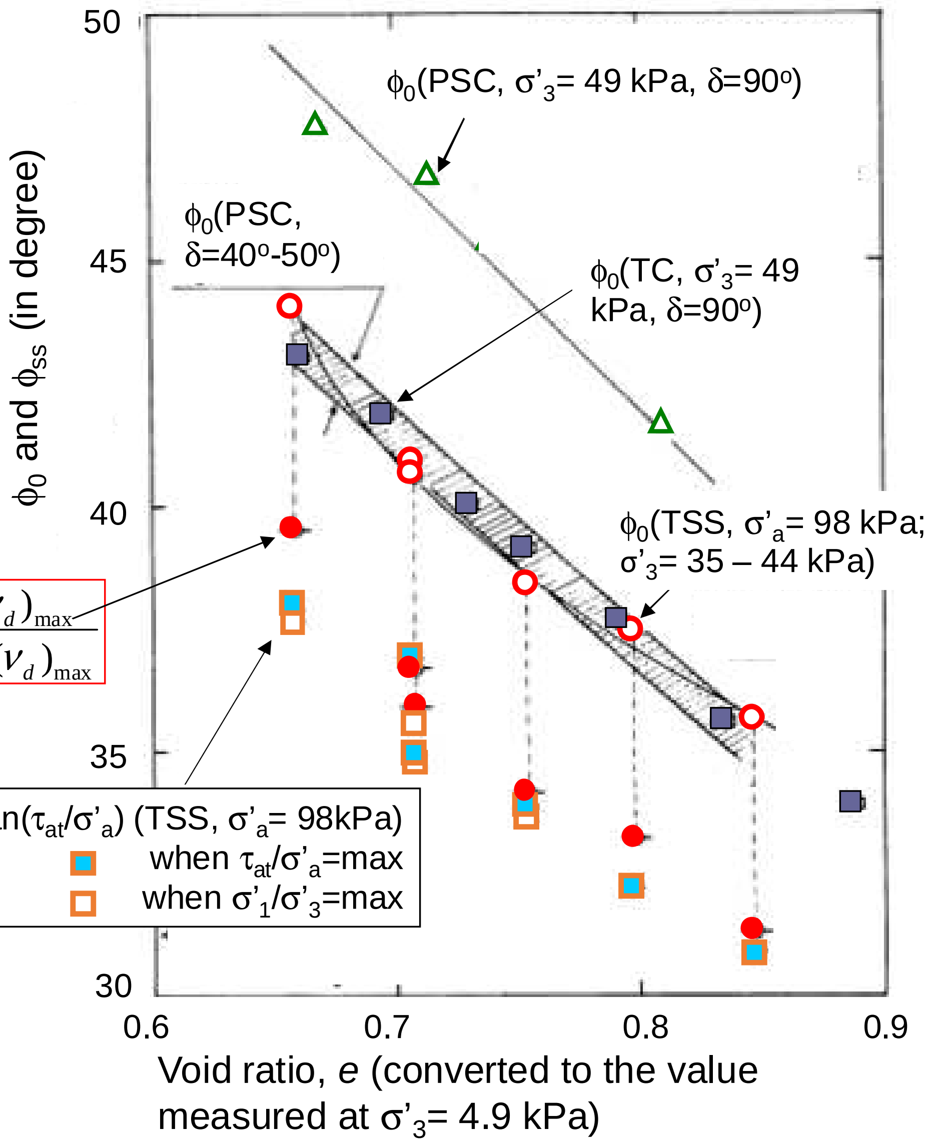


The TC strength (δ= 90°) is noticeably smaller than the average strength along a circular failure plane under plane strain conditions.

Inherently anisotropic stress – strain behaviour under drained conditions- 7

$\phi_0 = \arcsin[(\sigma'_1 - \sigma'_3)/[(\sigma'_1+\sigma'_3)]_{max}$ values in the direct shear test and the PSC test (δ= 40º-50º) are nearly the same, because both tests are plane strain tests with similar anisotropy effects.

Air-pluviated Toyoura sand



Theoretical value

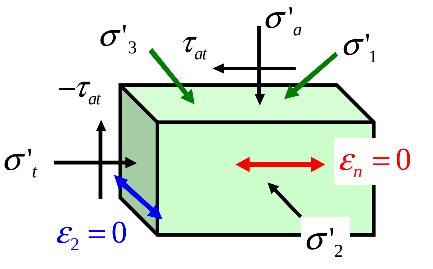
$$\phi_{ss} = \arctan \frac{\sin\phi_0 \cdot \cos(\nu_d)_{\max}}{1-\sin\phi_0 \cdot \sin(\nu_d)_{\max}}$$

Measured $\phi_{ss}=\arctan(\tau_{at}/\sigma'_a)$ (TSS, σ'_a= 98kPa)

▣ when τ_{at}/σ'_a=max

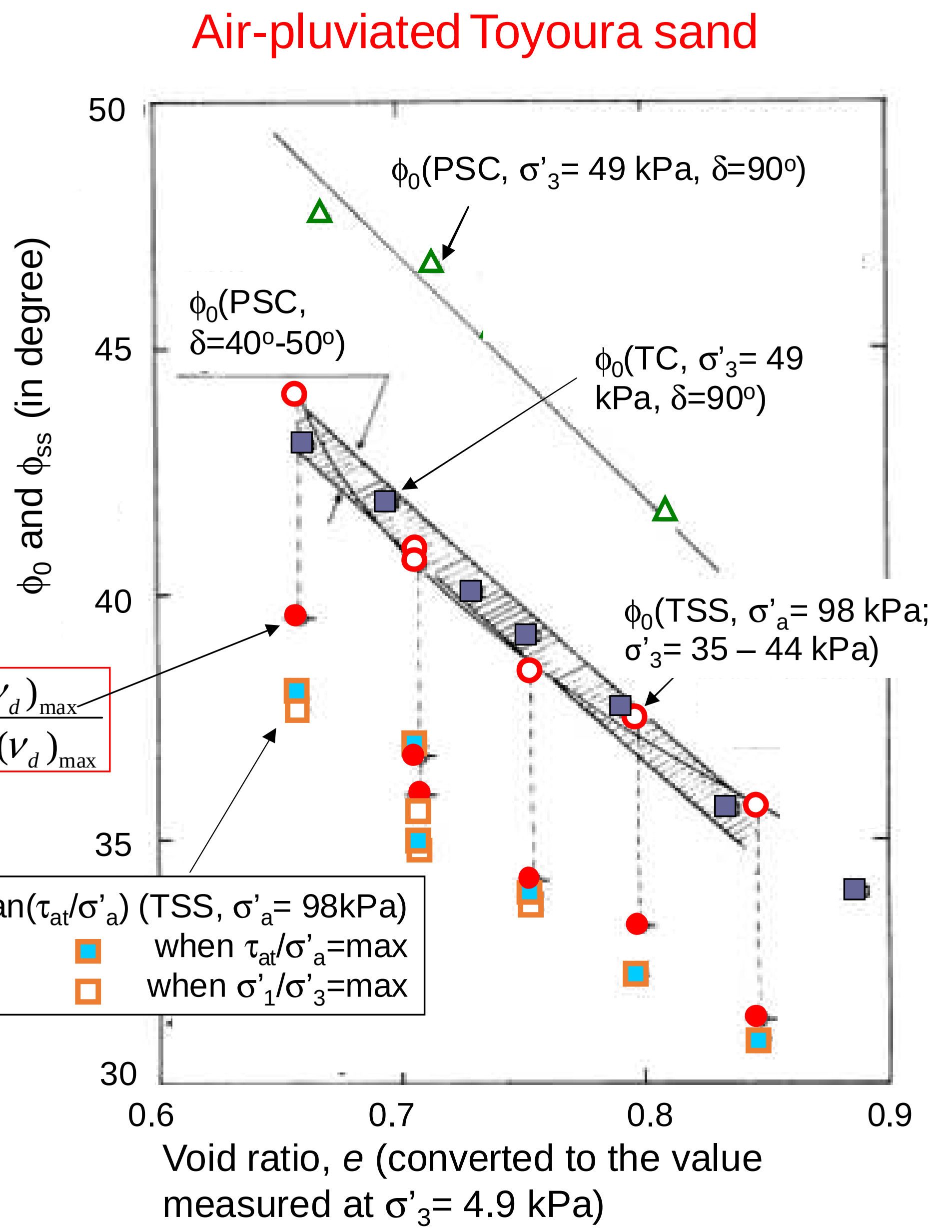
☐ when σ'_1/σ'_3=max

Stress condition in TSS test



Inherently anisotropic stress – strain behaviour under drained conditions- 8

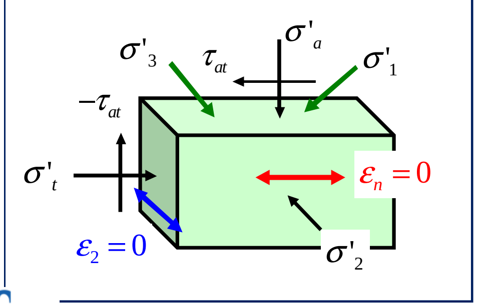
ϕ_0= arcsin[(σ'_1- σ'_3)/[(σ'_1+σ'_3)]$_{max}$ values in the direct shear test and the TC test (δ= 90º) happen to be nearly the same due to cancelling out of the effects of anisotropy and (σ'_2 - σ'_3)/(σ'_1 - σ'_3).



$$\phi_{ss} = \arctan \frac{\sin\phi_0 \cdot \cos(\nu_d)_{\max}}{1-\sin\phi_0 \cdot \sin(\nu_d)_{\max}}$$

Measured ϕ_{ss}=arctan(τ_{at}/σ'_a) (TSS, σ'_a= 98kPa)
▣ when τ_{at}/σ'_a=max
☐ when σ'_1/σ'_3=max

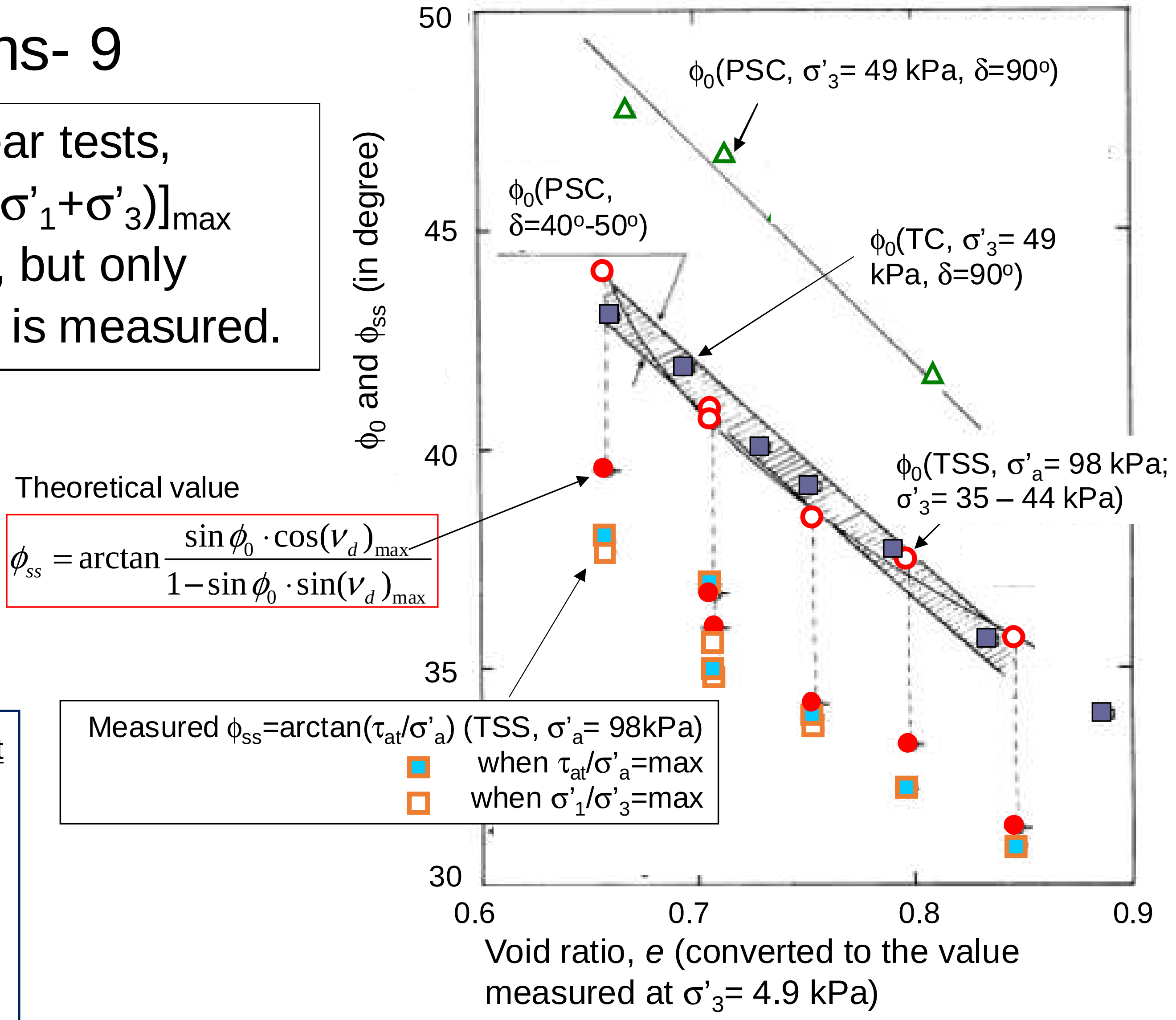
Stress condition in TSS test



Inherently anisotropic stress – strain behaviour under drained conditions- 9

In ordinary direct shear tests, $\phi_0 = \arcsin[(\sigma'_1 - \sigma'_3)/[(\sigma'_1 + \sigma'_3)]_{max}$ cannot be measured, but only $\phi_{ss} = \arctan(\tau_{at}/\sigma'_a)_{max}$ is measured.

Air-pluviated Toyoura sand



Theoretical value

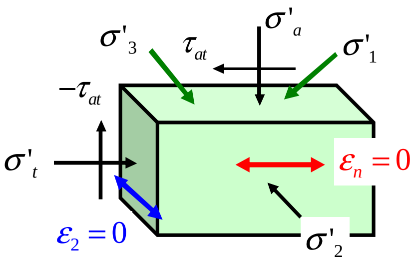
$$\phi_{ss} = \arctan \frac{\sin\phi_0 \cdot \cos(\nu_d)_{\max}}{1 - \sin\phi_0 \cdot \sin(\nu_d)_{\max}}$$

Measured $\phi_{ss}=\arctan(\tau_{at}/\sigma'_a)$ (TSS, σ'_a= 98kPa)

▣ when τ_{at}/σ'_a=max

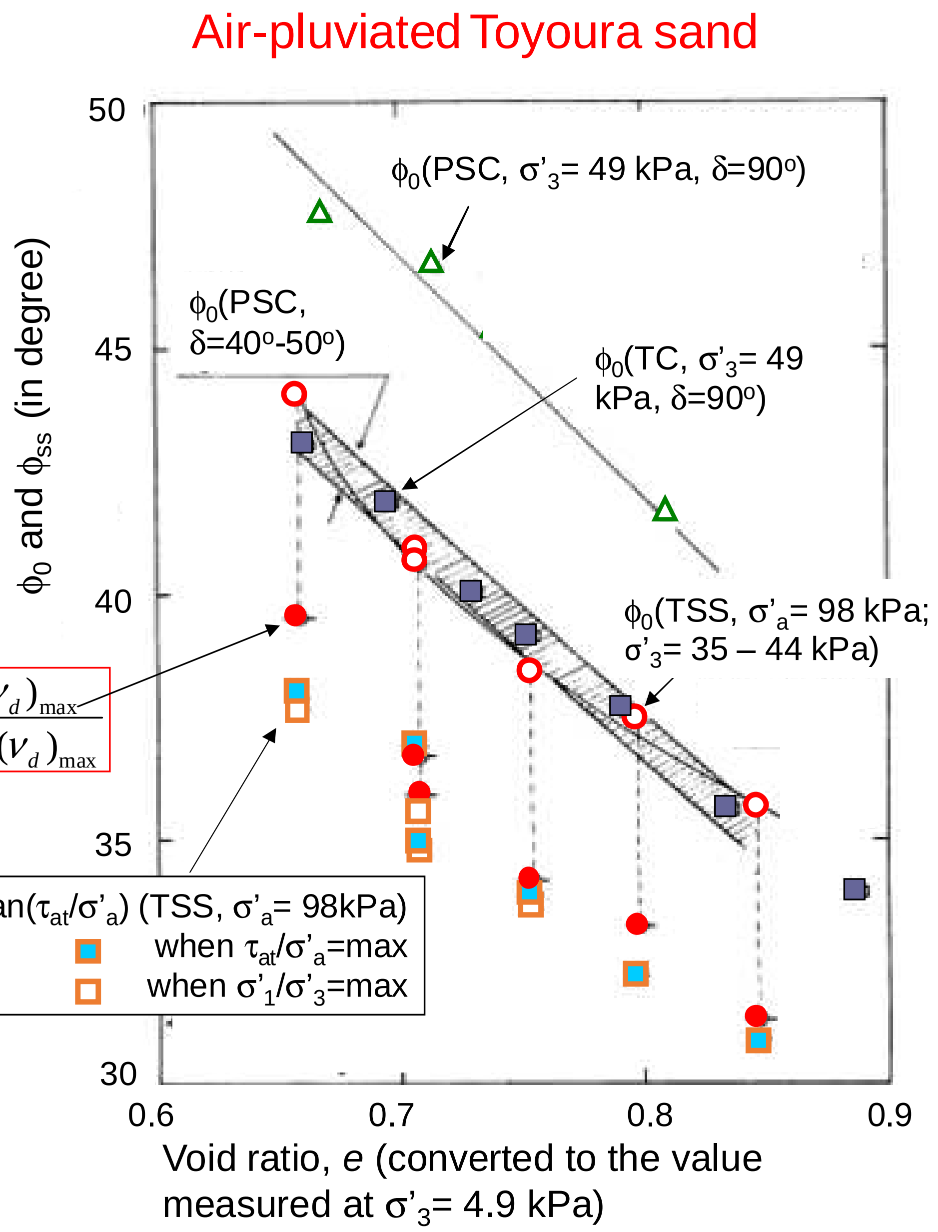
☐ when σ'_1/σ'_3=max

Stress condition in TSS test



Inherently anisotropic stress – strain behaviour under drained conditions- 10

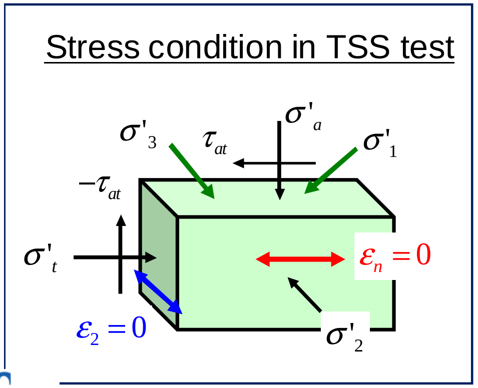
ϕ_{ss}= arctan$(\tau_{at}/\sigma'_a)_{max}$ from the direct shear test is significantly lower than ϕ_0 from TC test (δ= 90º).
The use of ϕ_{ss} in the slope stability analysis is usually too conservative.



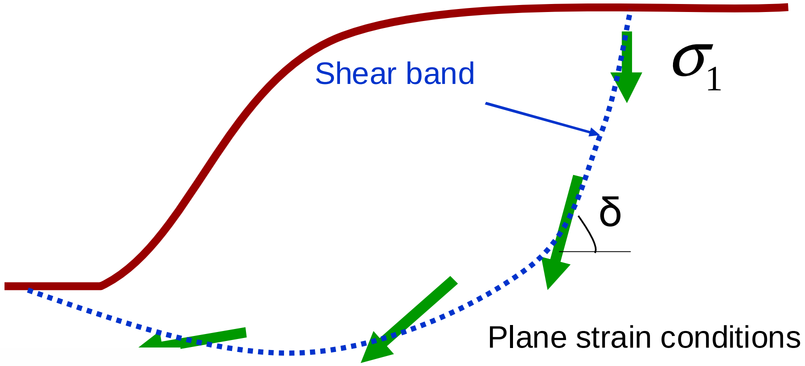
Theoretical value

$$\phi_{ss} = \arctan\frac{\sin\phi_0\cdot\cos(\nu_d)_{\max}}{1-\sin\phi_0\cdot\sin(\nu_d)_{\max}}$$

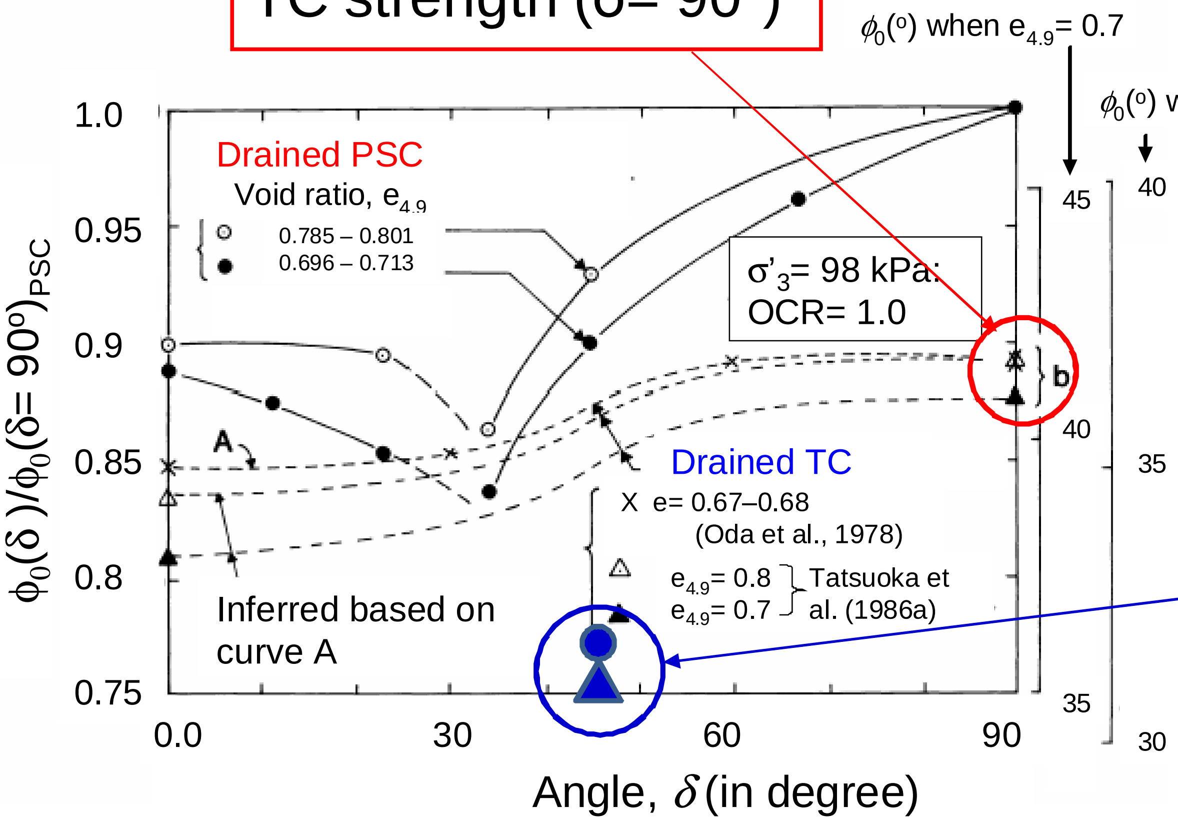
Measured ϕ_{ss}=arctan(τ_{at}/σ'_a) (TSS, σ'_a= 98kPa)
▣ when τ_{at}/σ'_a=max
□ when σ'_1/σ'_3=max



Inherently anisotropic stress – strain behaviour under drained conditions- 10



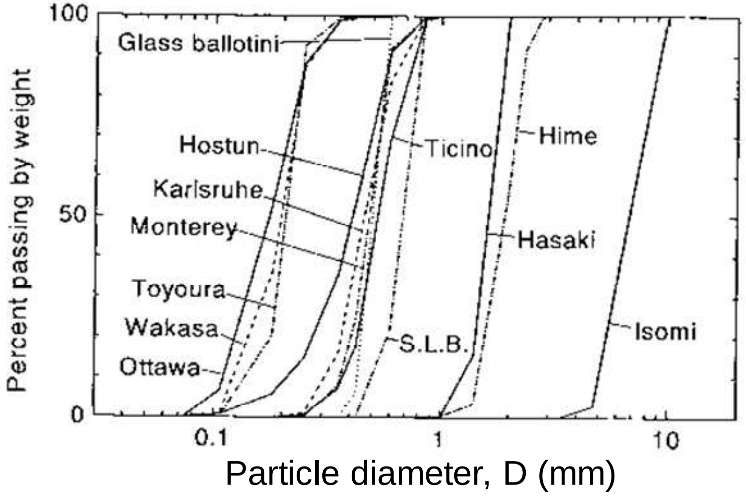
TC strength (δ= 90º)



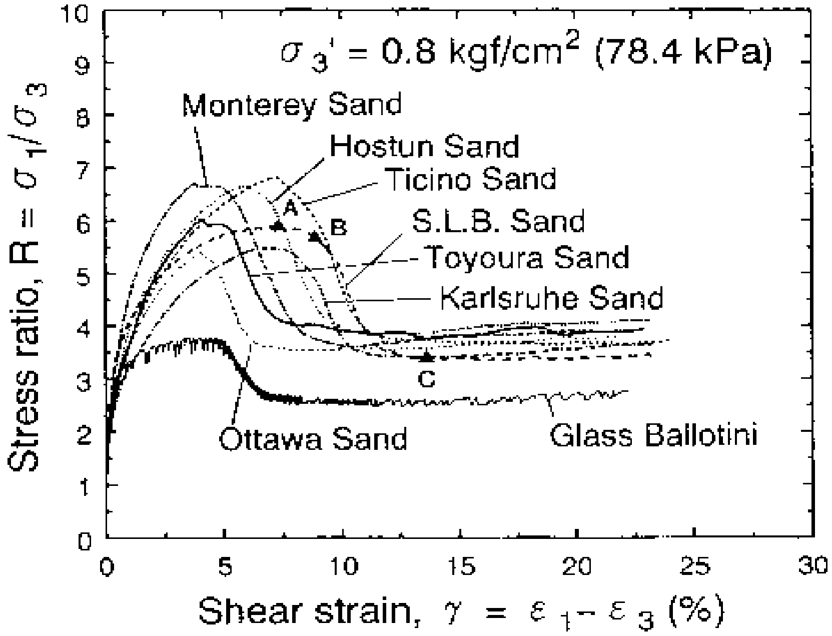
ϕ_{ss}= arctan$(\tau_{at}/\sigma'_{a})_{max}$ from the direct shear test, much lower than the average strength along a circular failure plane under plane strain conditions.

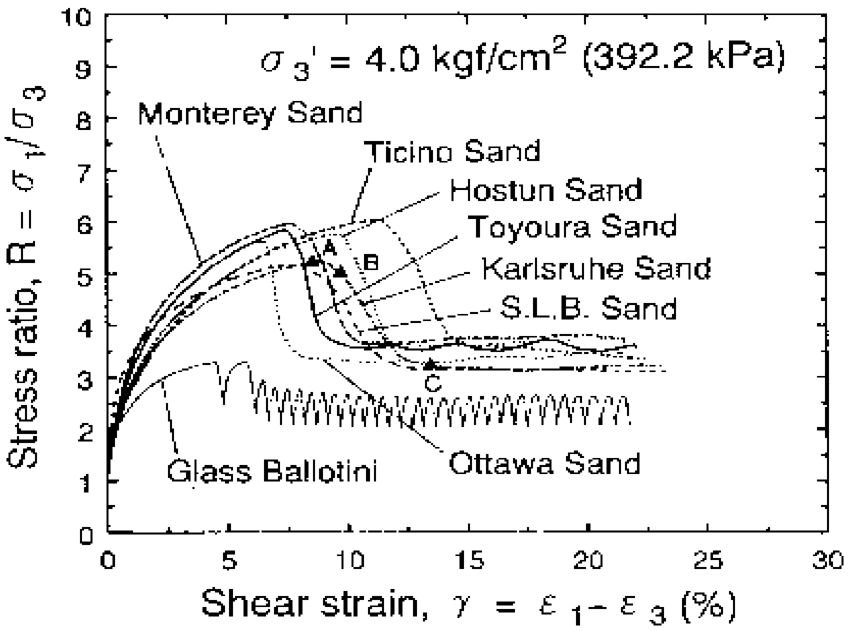
Strain-softening associated with shear banding under drained plane strain conditions- 1

Uniform granular materials



The stress is plotted against "strain averaged for the whole specimen", not representative of the strain in the shear band.



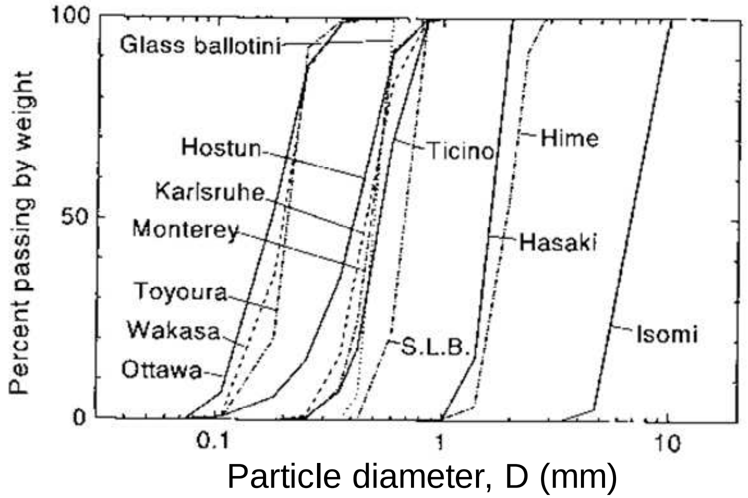


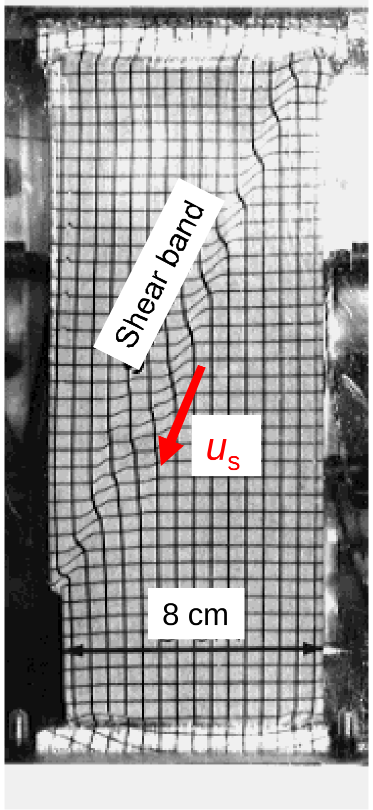
Strain-softening associated with shear banding under drained plane strain conditions- 2

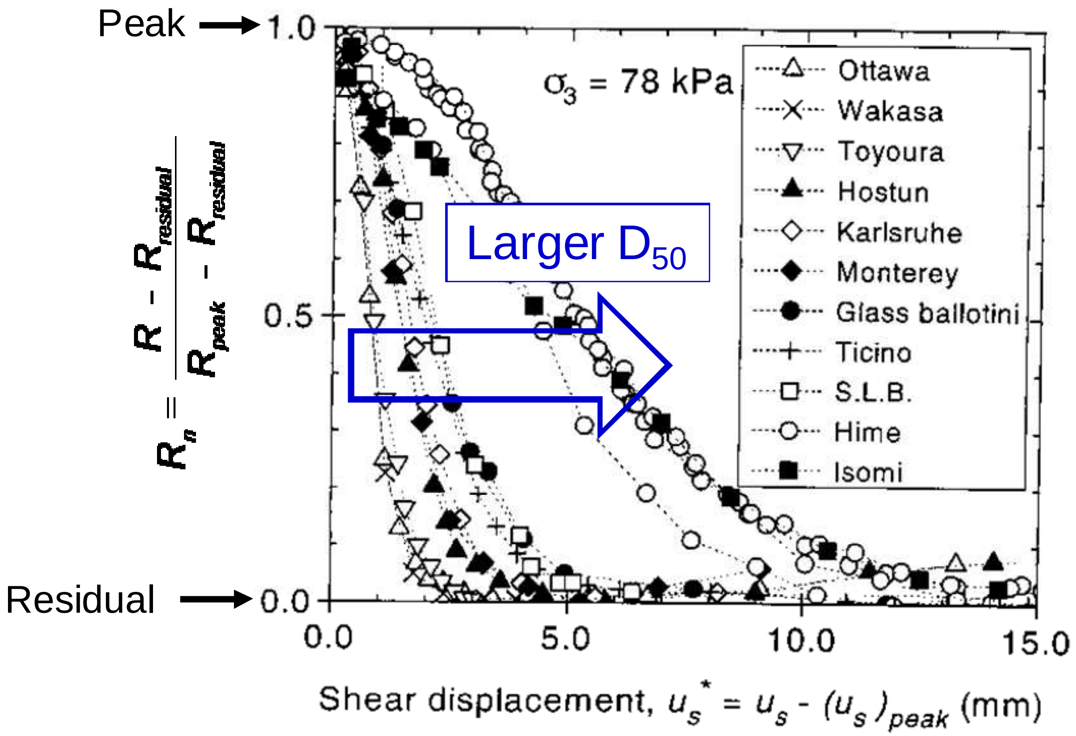
u_s: shear displacement along a shear band

$(u_s)_{peak}$: values of u_s at the peak stress (very small)

Uniform granular materials

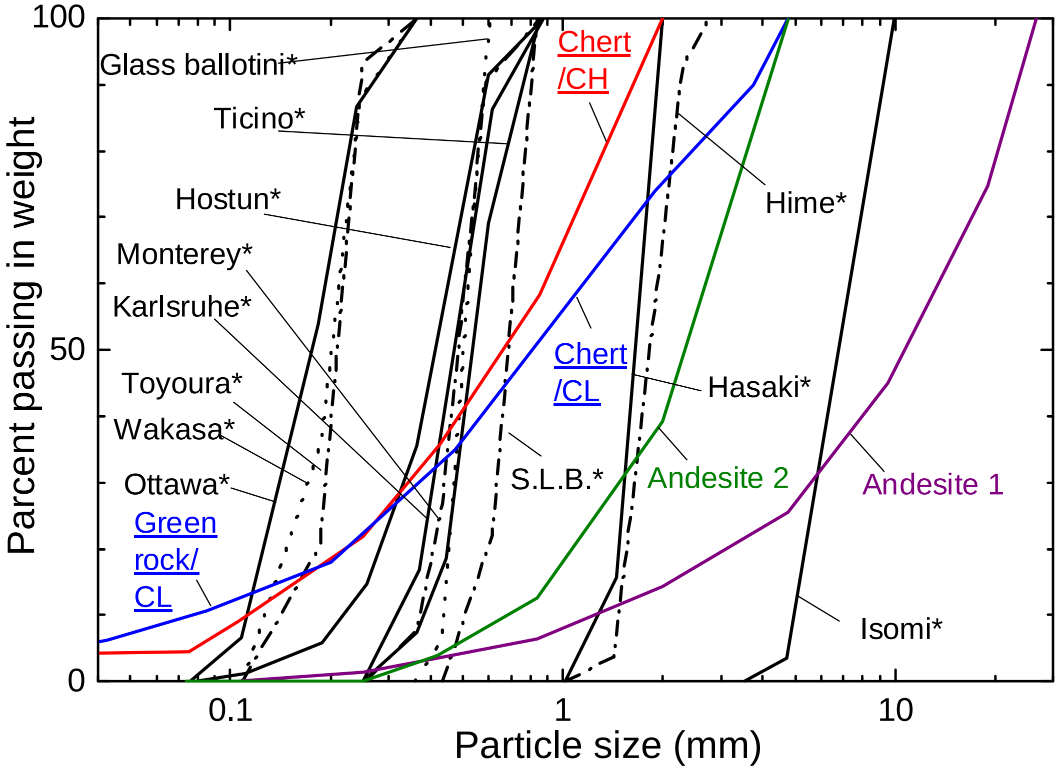




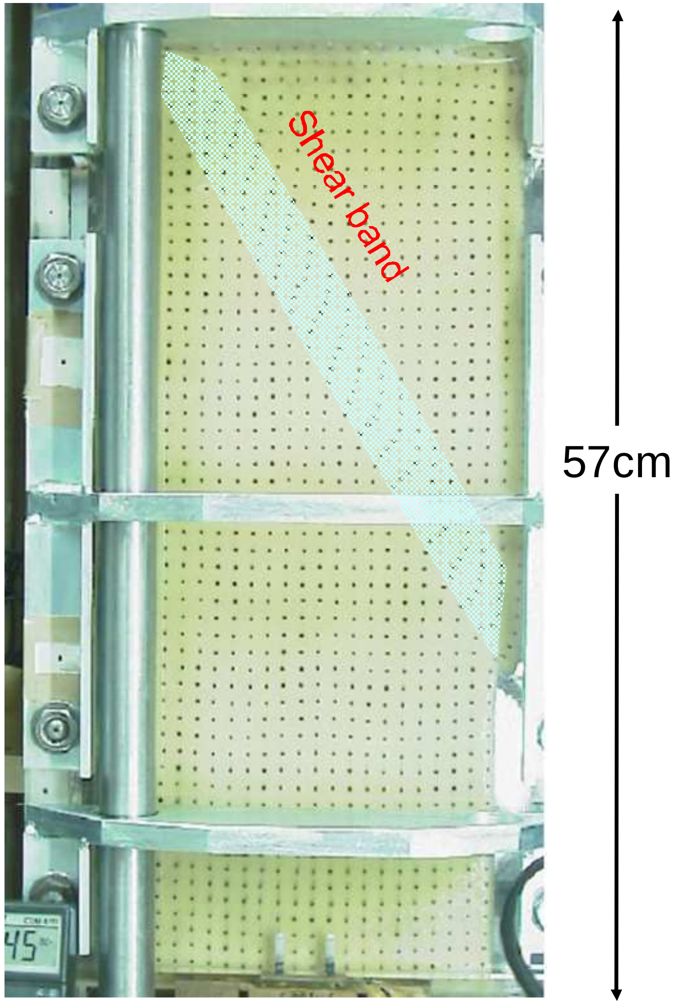


Strain-softening associated with shear banding under drained plane strain conditions- 3

PSC tests on many poorly- & well-graded gravelly soils



Large PSC test



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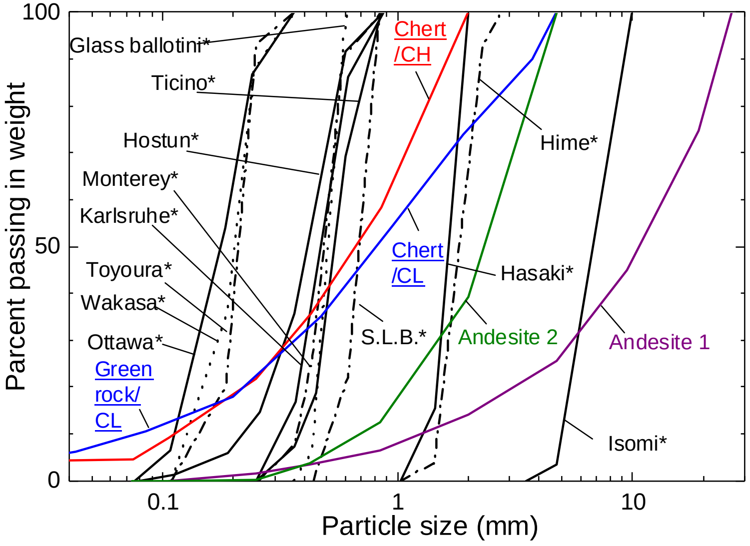
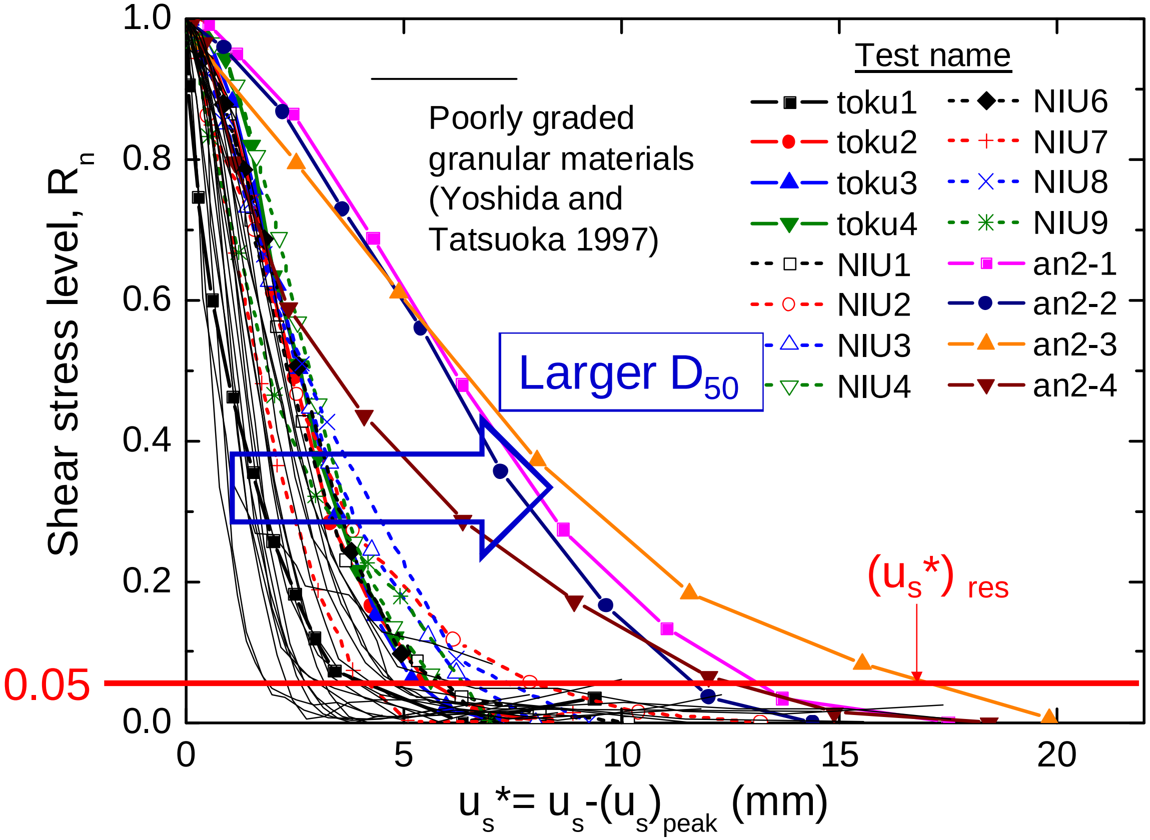
(D_{50}= 2.49 mm & U_c= 4.1),
at ε_1 = 4.25 %, σ'_3= 314 kPa

Strain-softening associated with shear banding under drained plane strain conditions- 3

PSC tests on many poorly- & well-graded gravelly soils

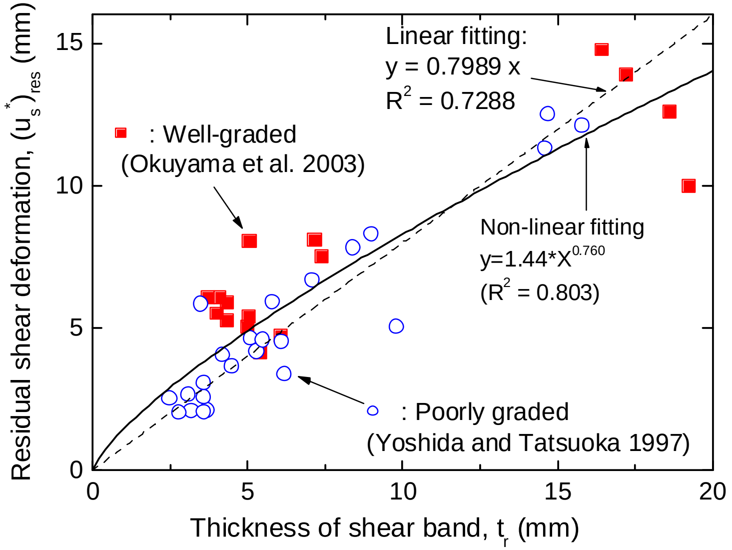


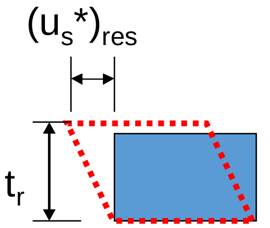


Larger shear deformation of shear band for larger D_{50} why ?

Strain-softening associated with shear banding under drained plane strain conditions- 5

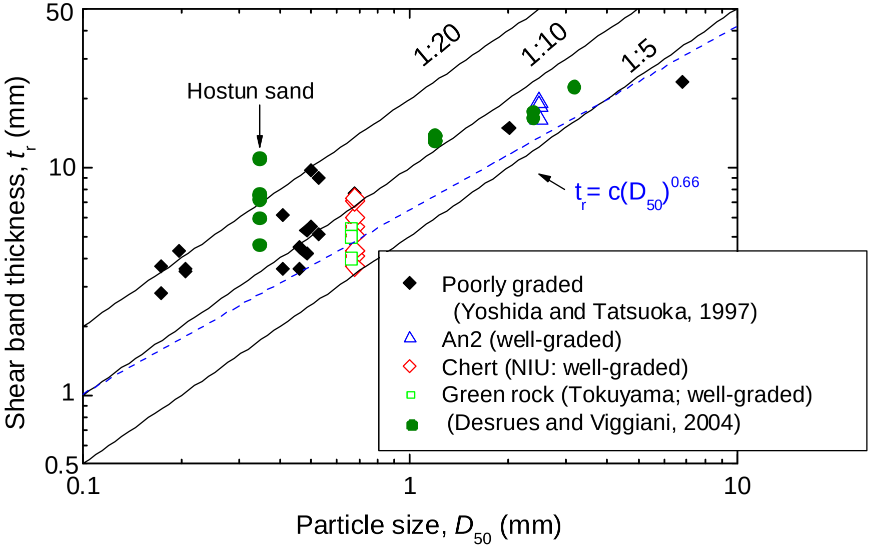
$(u_s^*)_{res}$ increases with shear band thickness, t_r, for a similar shear strain in the shear band



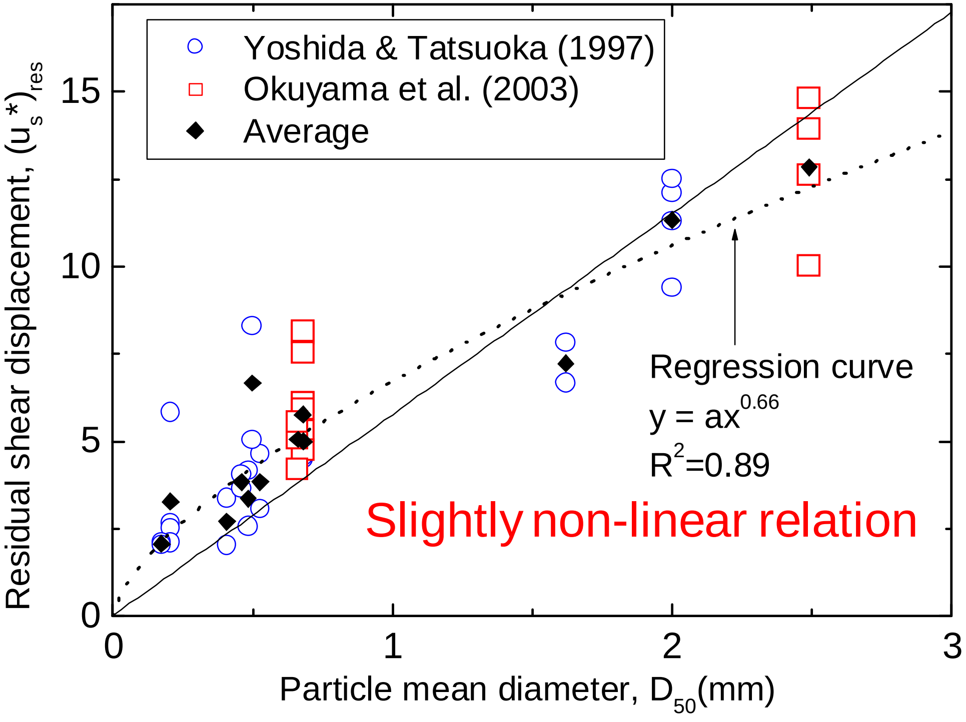


At the start of residual state

t_r increases with D_{50}



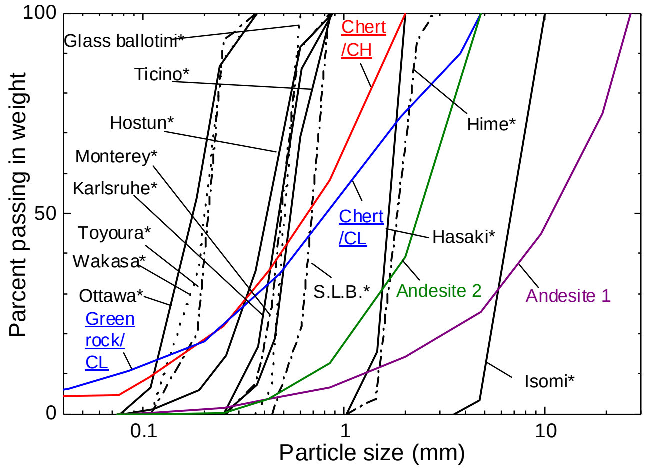
So, $(u_s^*)_{res}$ increases with D_{50}

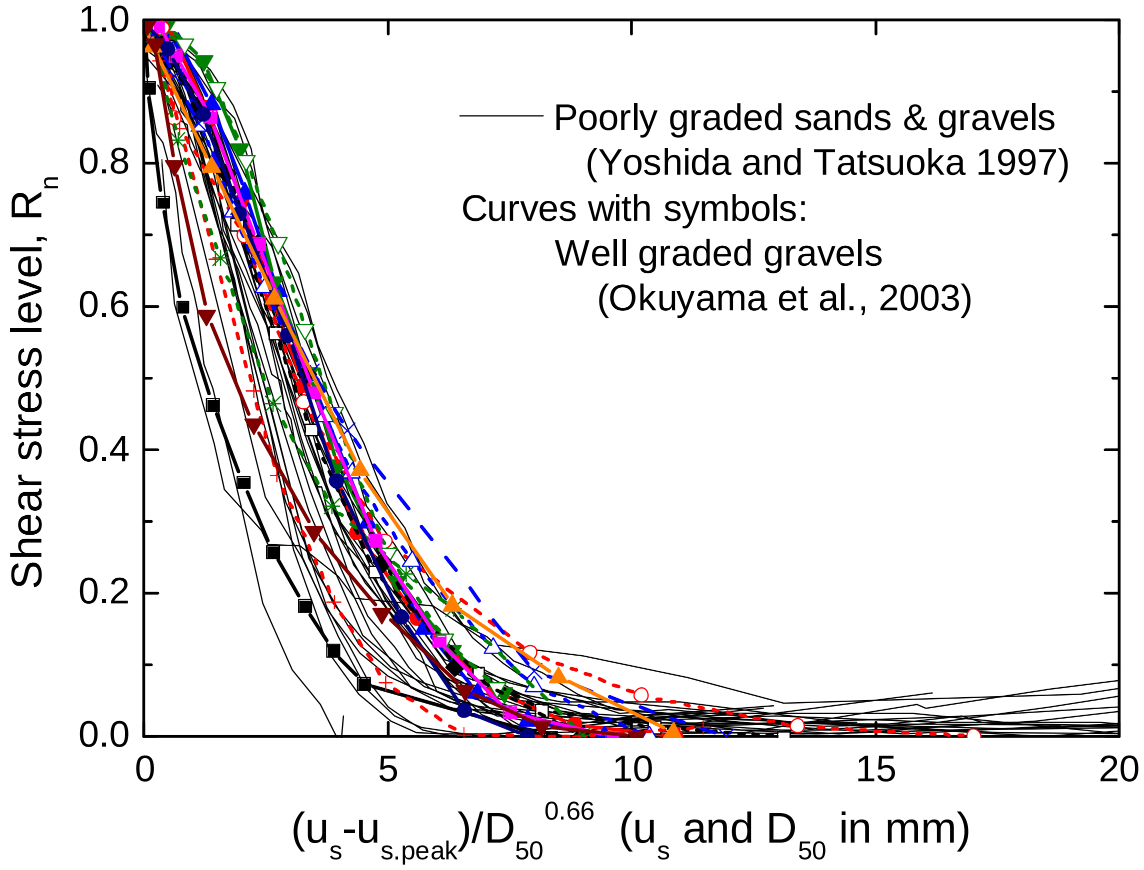


Slightly non-linear relation

Strain-softening associated with shear banding under drained plane strain conditions- 6

Poorly- & well-graded gravelly soil





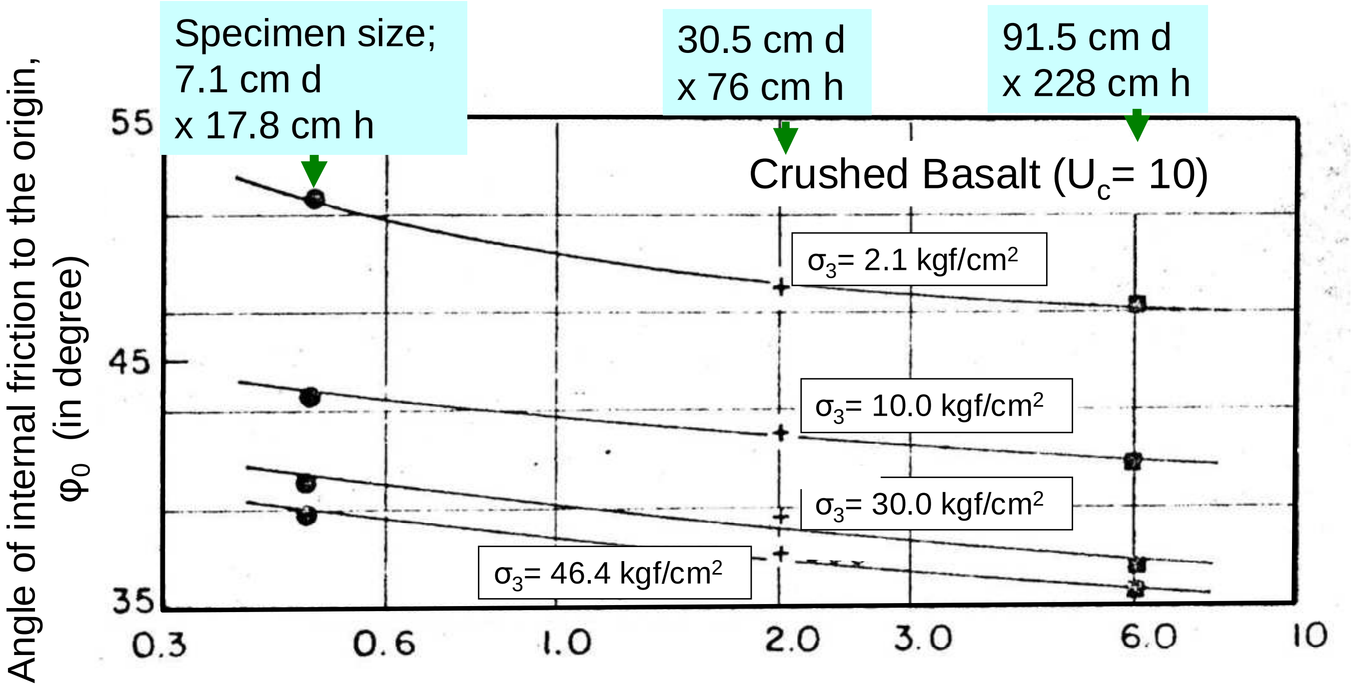
Normalization as $u_s/(D_{50})^{0.66}$: still a noticeable scatter, but no systematic effects of U_c, σ_3, density, strain rate, …..

Useful to infer the $R_n - u_s$ relation for a given D_{50} to be used in the slip displacement analysis by the Newmark method.

Strain-softening associated with shear banding under drained plane strain conditions- 7

On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the D_{max}/specimen size constant!

This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.



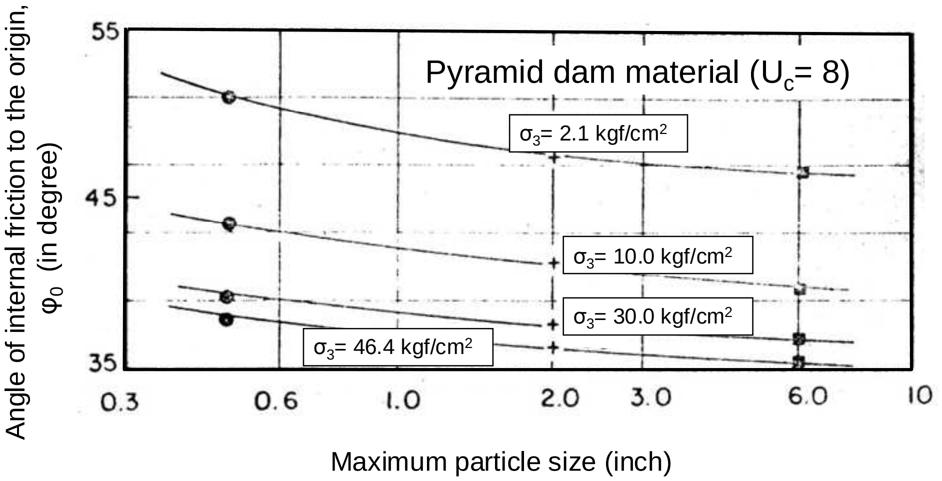
Maximum particle size (inch)

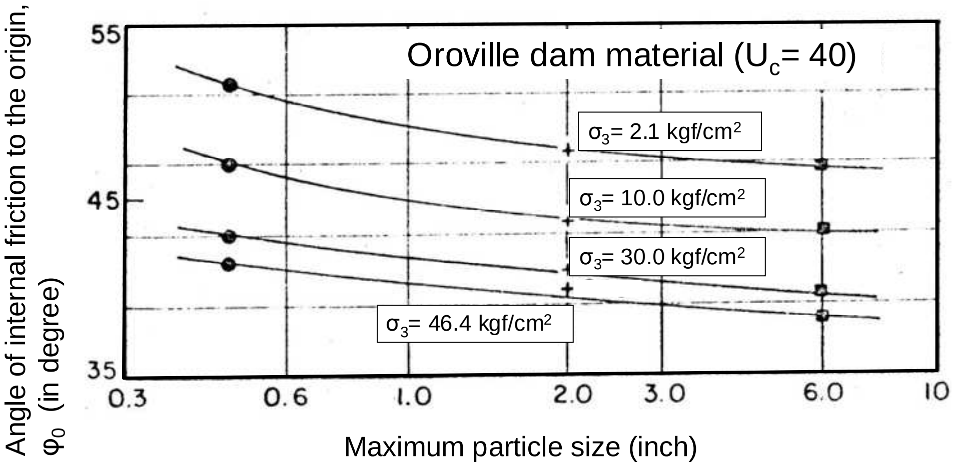
University of California, Berkeley (Marachi et al., 1969)

Strain-softening associated with shear banding under drained plane strain conditions- 8

On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the D_{max}/specimen size constant!

This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.





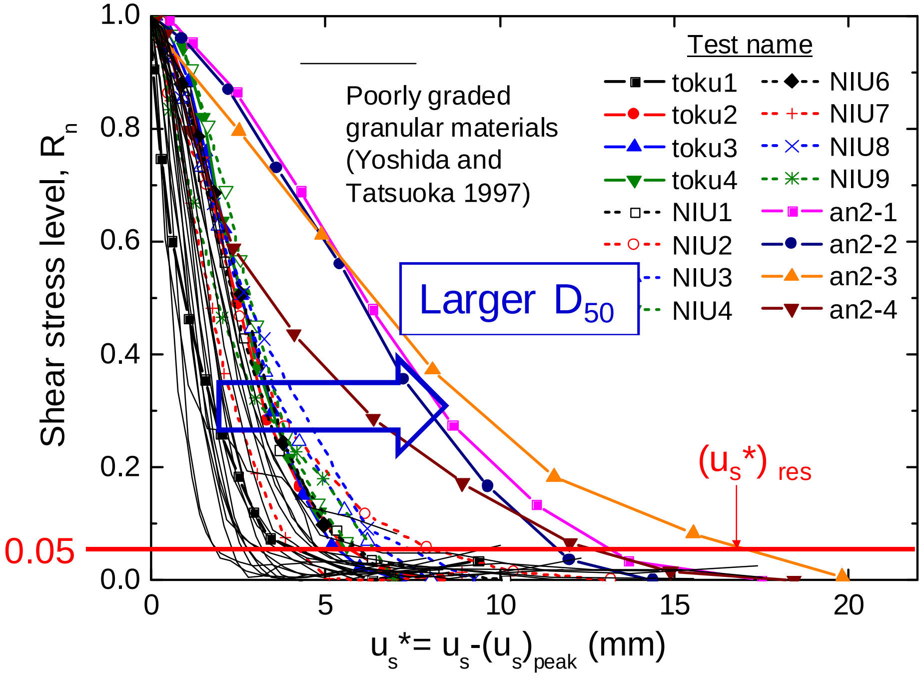
University of California, Berkeley
(Maranchi et al., 1969)

Strain-softening associated with shear banding under drained plane strain conditions- 9

On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the D_{max}/specimen size constant!

This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.

One method to alleviate this contradiction in the seismic design, at least partly, is the evaluation of slip displacement by the Newmark method taking into account the effects of D_{50} on the $R_n – u_s$ relation.



Poorly graded granular materials (Yoshida and Tatsuoka 1997)

Larger D_{50}

$(u_s^*)_{res}$

Undrained stress- strain behaviour of saturated soil

1. Effects of dry density:
   - much larger than those on drained strength; and
   - become more significant by effects of preceding cyclic undrained loading.

2. Degradation of the undrained stress-strain properties and strength in the course of cyclic undrained loading:
   - more when more cyclically sheared undrained; and
   - how to model this trend for numerical analysis ?

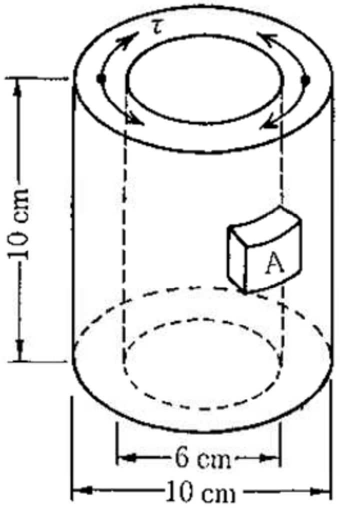
     *Simplified model for simplified numerical analysis** vs.

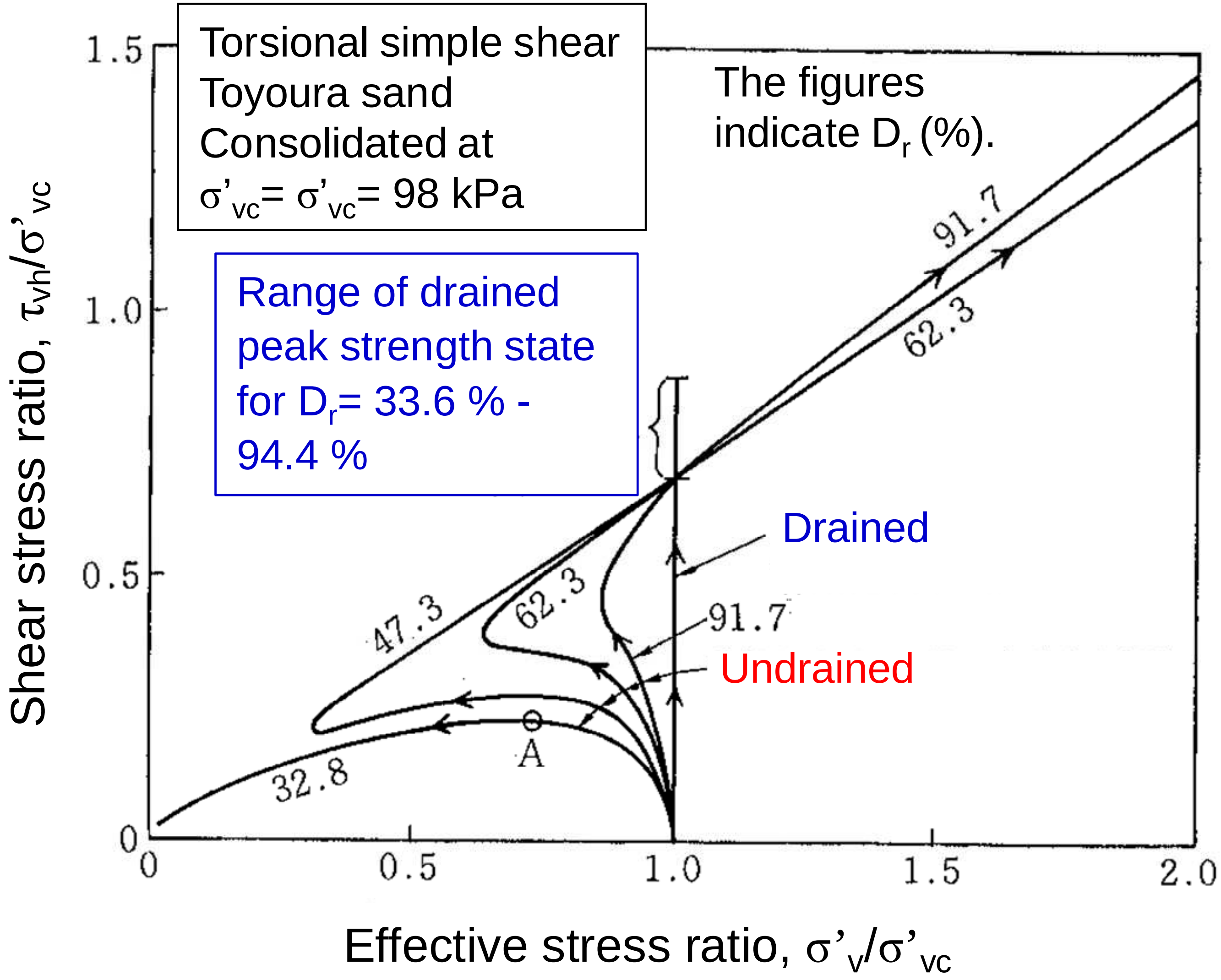
     Full model for rigorous numerical analysis

     (* explained in this presentation)

Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 1





Torsional simple shear
Toyoura sand
Consolidated at
$\sigma'_{vc} = \sigma'_{vc} = 98$ kPa

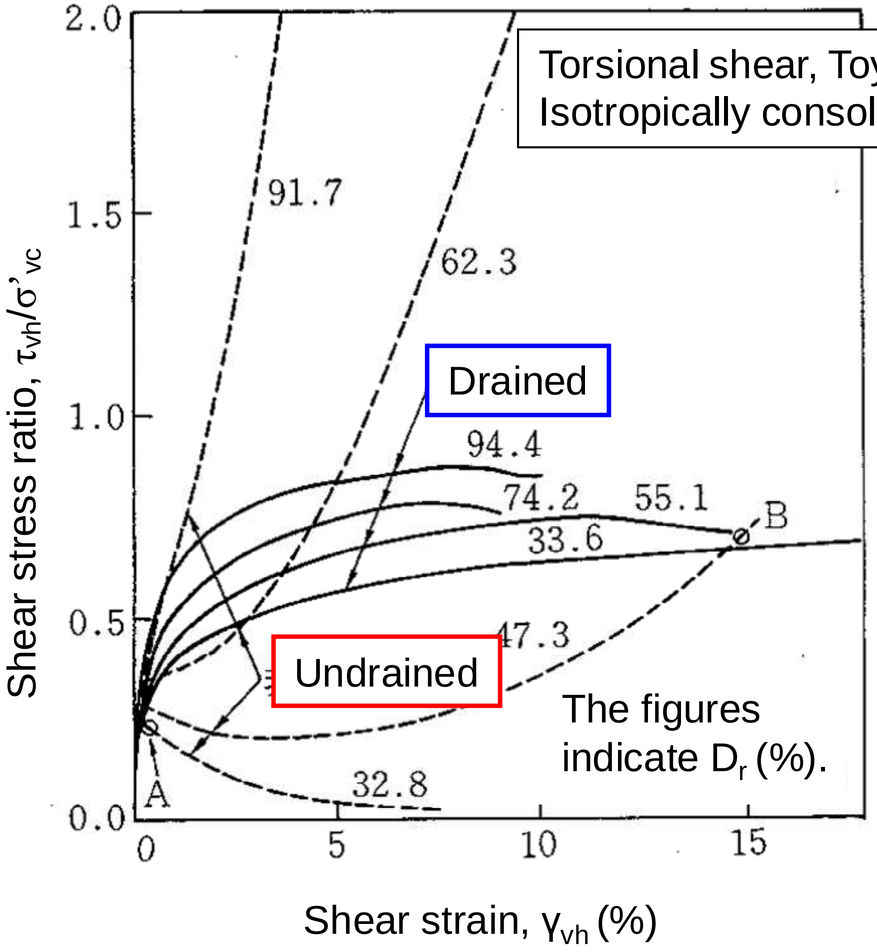
The figures indicate D_r (%).

Range of drained peak strength state for $D_r = 33.6$ % - 94.4 %

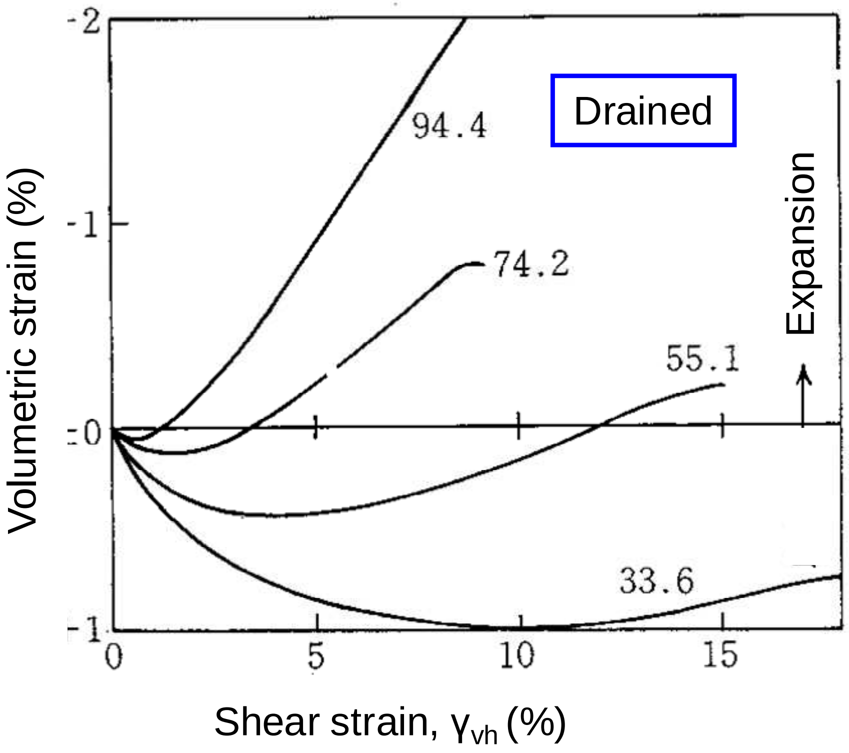
Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 2

Torsional shear, Toyoura sand
Isotropically consolidated to σ'$_{vc}$= σ'$_{vc}$= 98 kPa



The figures indicate D_r (%).

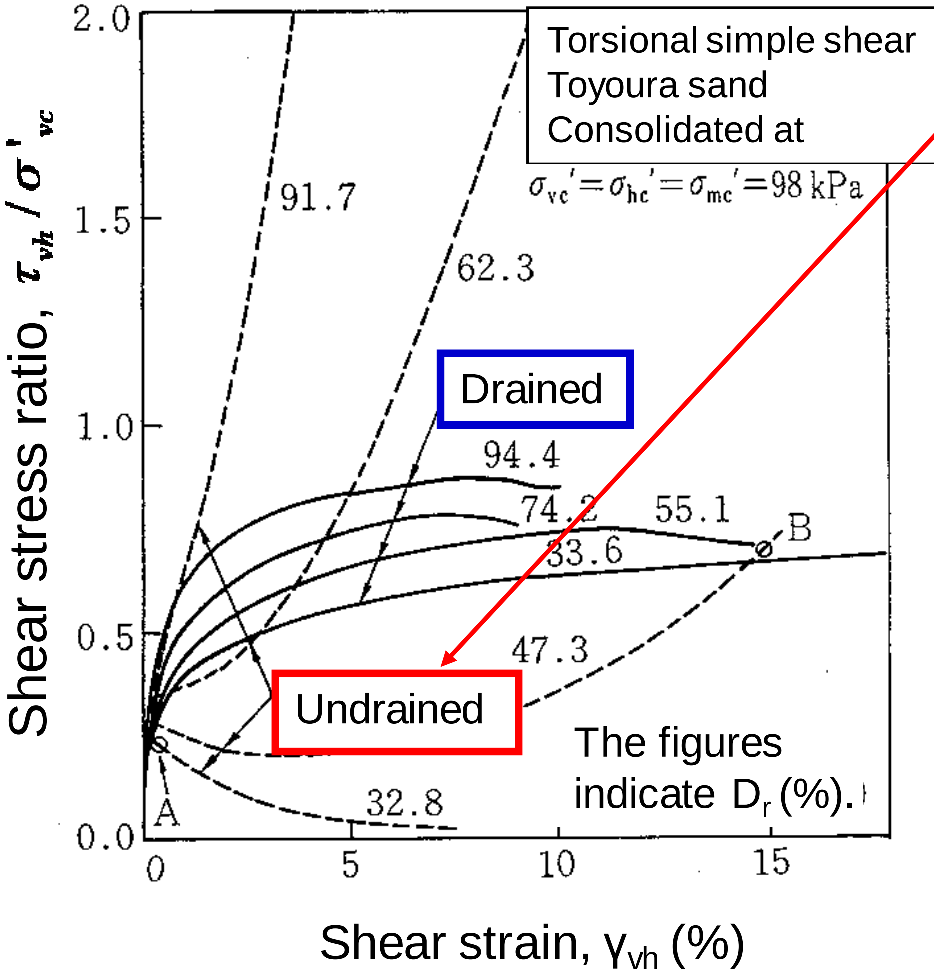


In drained tests, the peak strength is noticeably different with largely different volume changes for different dry densities (or different D_r values) !

Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 3

σ_v' changes at a constant volume !



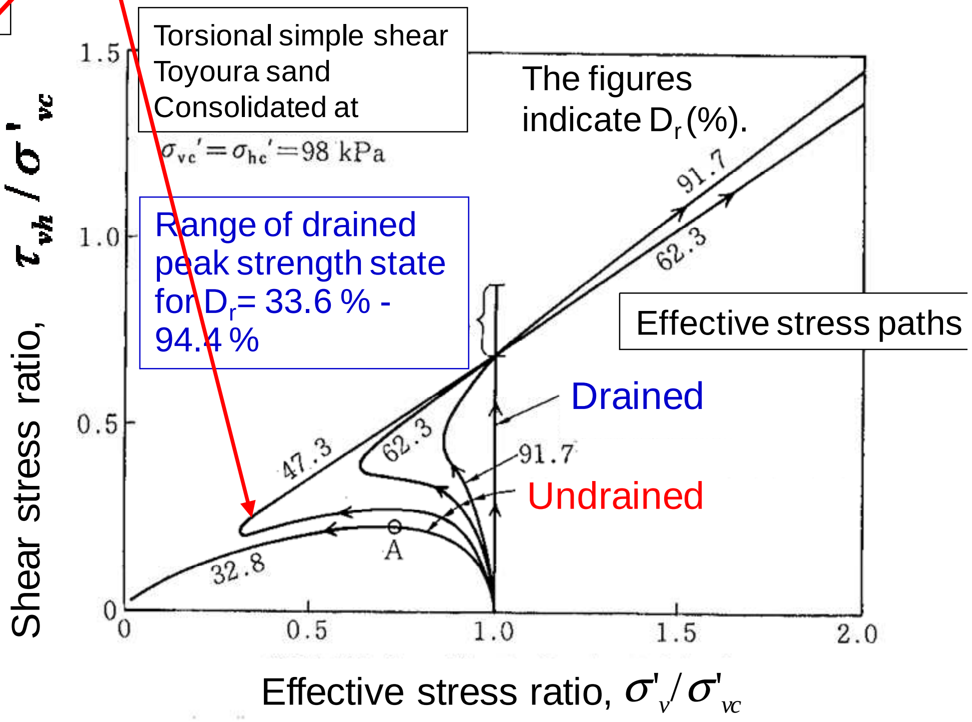
Torsional simple shear
Toyoura sand
Consolidated at

Drained

Undrained

The figures indicate D_r (%).

Shear strain, γ_{vh} (%)



Torsional simple shear
Toyoura sand
Consolidated at

The figures indicate D_r (%).

Range of drained peak strength state for D_r= 33.6 % - 94.4 %

Effective stress paths

Drained

Undrained

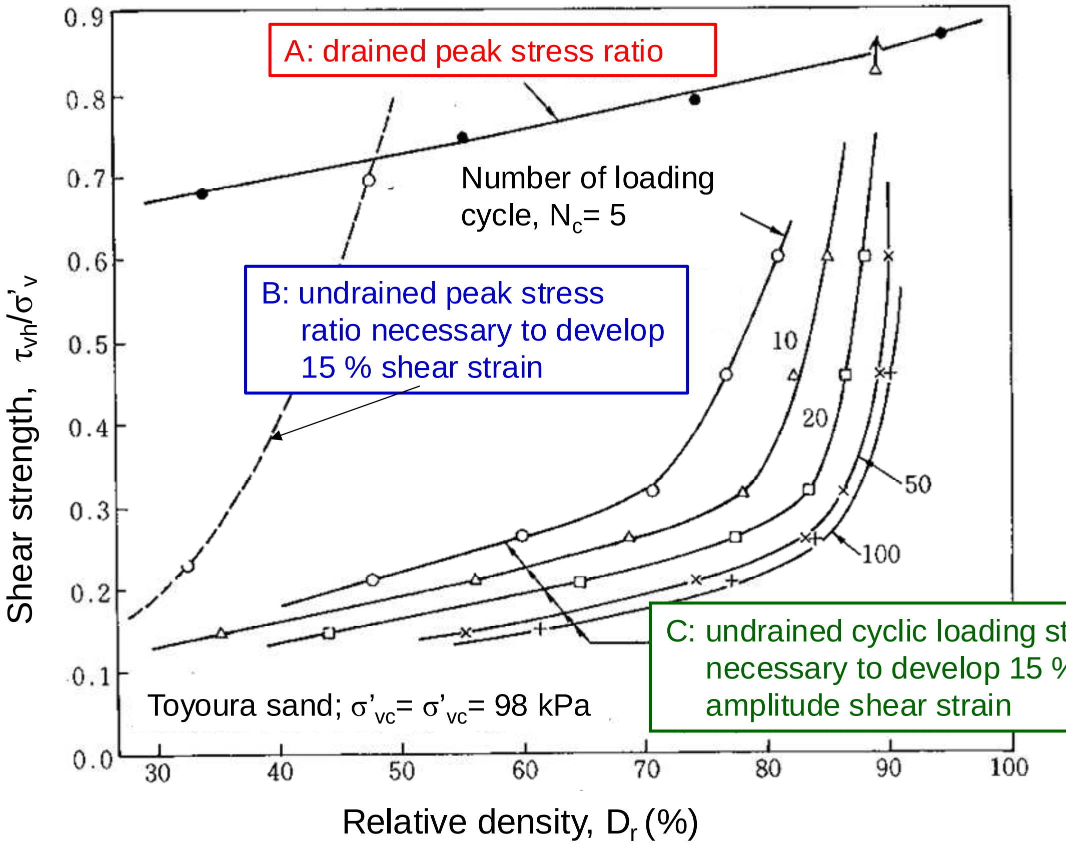
Effective stress ratio, σ'_v/σ'_{vc}

In undrained tests, the effective stress path is largely different with largely different peak strengths for different densities !

Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 4

Comparison among
(A)drained strength in ML,
(B) undrained strength in ML and
(C) undrained CL strength of Toyoura sand

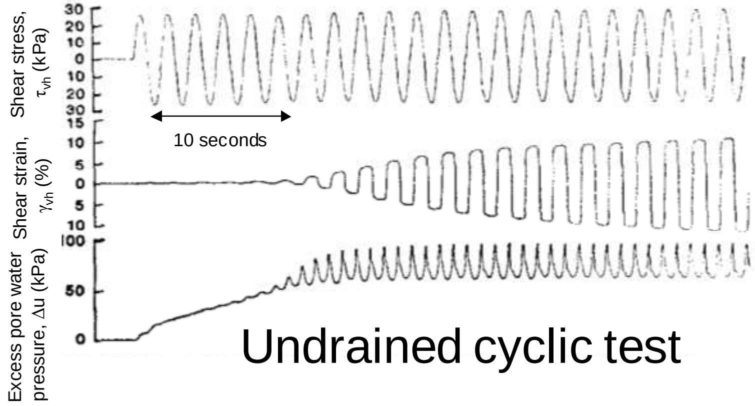


A: drained peak stress ratio

B: undrained peak stress ratio necessary to develop 15 % shear strain

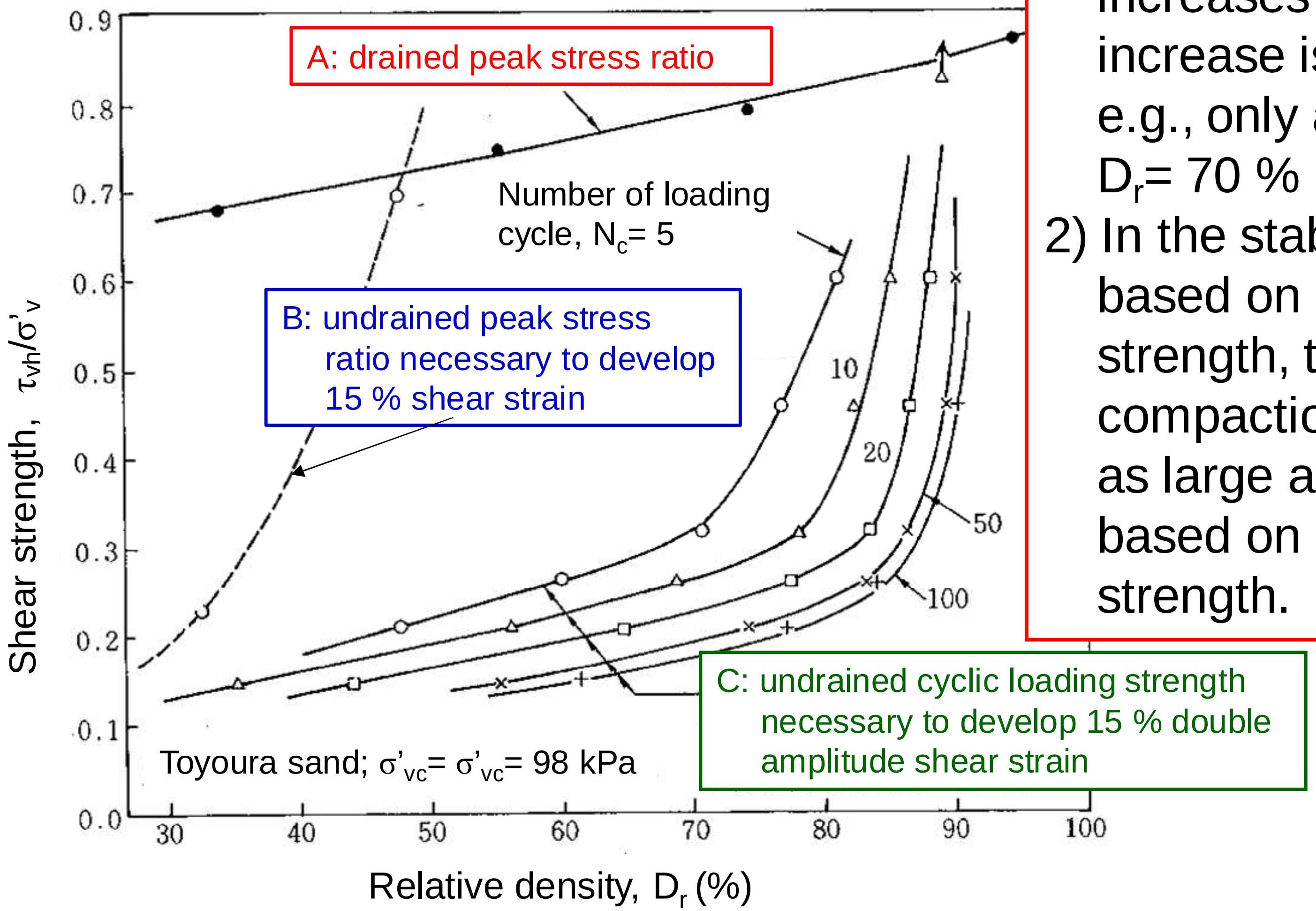
C: undrained cyclic loading strength necessary to develop 15 % double amplitude shear strain

Toyoura sand; $\sigma'_{vc} = \sigma'_{vc}$= 98 kPa



Undrained cyclic test

Undrained stress- strain behaviour of saturated soil:
1. Effects of dry density - 5



A: drained peak stress ratio

Number of loading cycle, N_c= 5

B: undrained peak stress ratio necessary to develop 15 % shear strain

C: undrained cyclic loading strength necessary to develop 15 % double amplitude shear strain

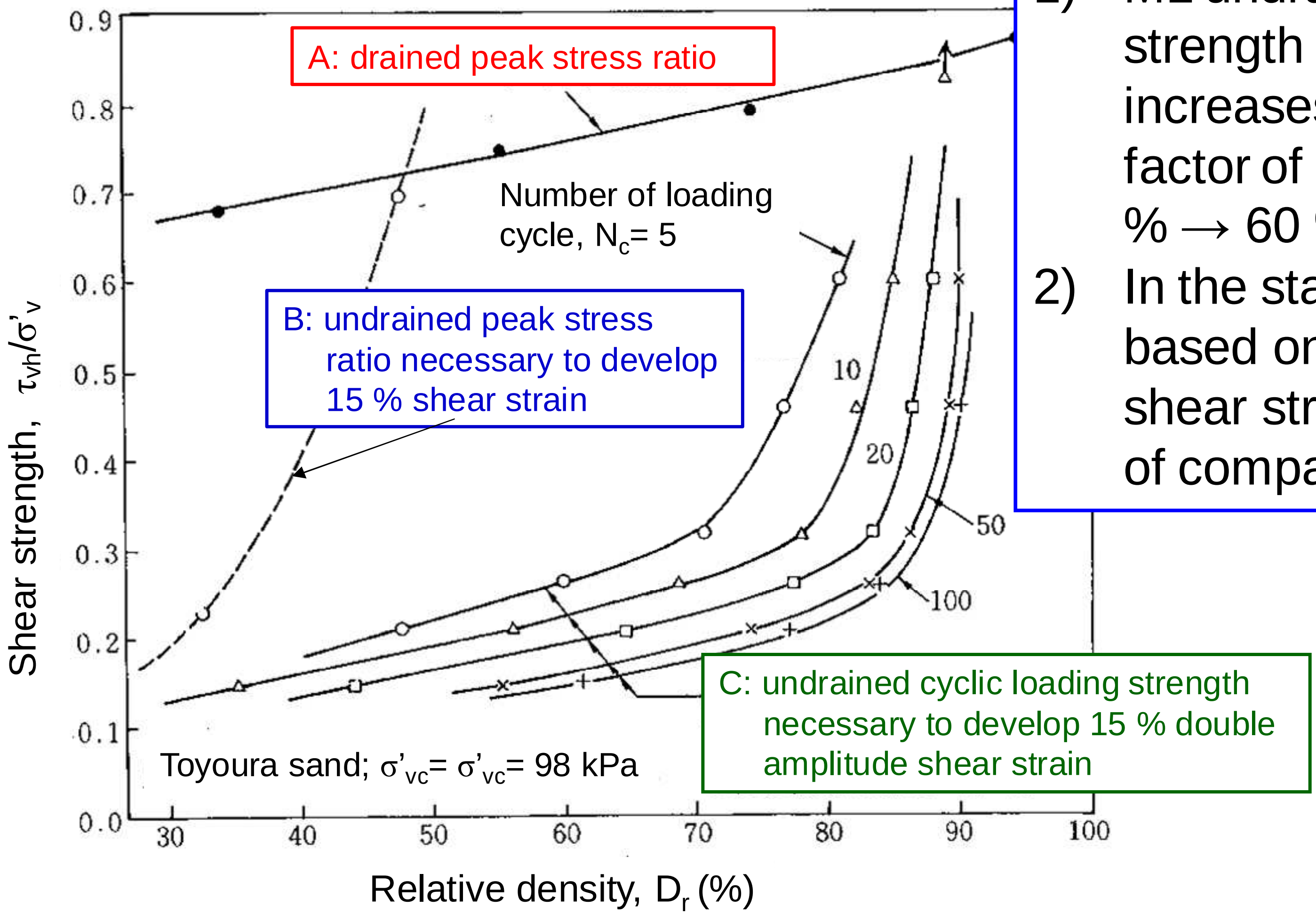
Toyoura sand; σ'_{vc}= σ'_{vc}= 98 kPa

1) ML drained shear strength increases with D_r, but the increase is not very large: e.g., only about 10 % when D_r= 70 % → 90 %.

2) In the stability analysis based on the drained shear strength, the benefit of compaction is large, but not as large as the one when based on undrained shear strength.

Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 6



A: drained peak stress ratio

Number of loading cycle, N_c= 5

B: undrained peak stress ratio necessary to develop 15 % shear strain

C: undrained cyclic loading strength necessary to develop 15 % double amplitude shear strain

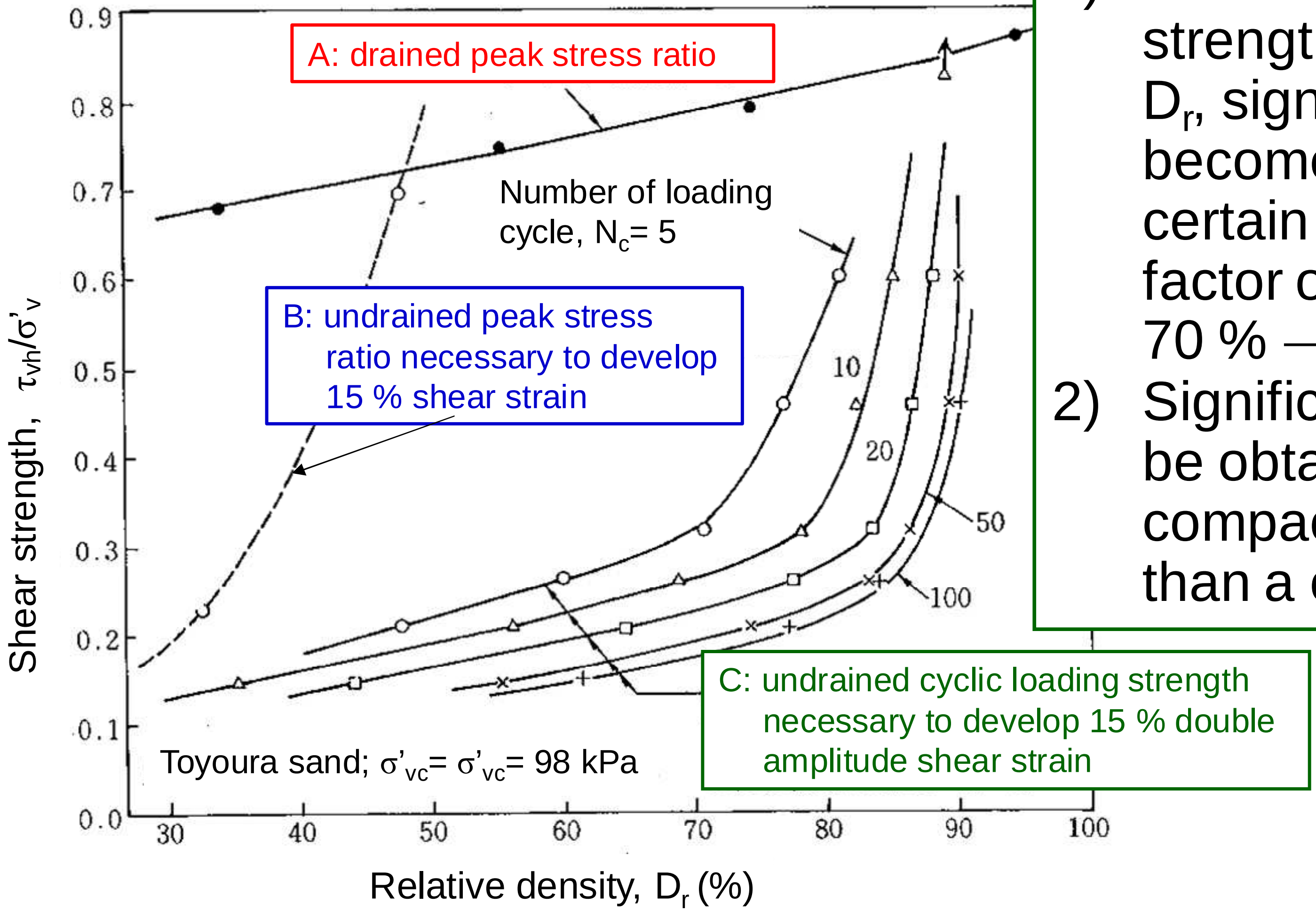
Toyoura sand; $\sigma'_{vc}= \sigma'_{vc}$= 98 kPa

1) ML undrained shear strength significantly increases with D_r: e.g., by a factor of three when D_r= 40 % → 60 %.

2) In the stability analysis based on the undrained shear strength, the benefit of compaction is significant.

Undrained stress- strain behaviour of saturated soil:

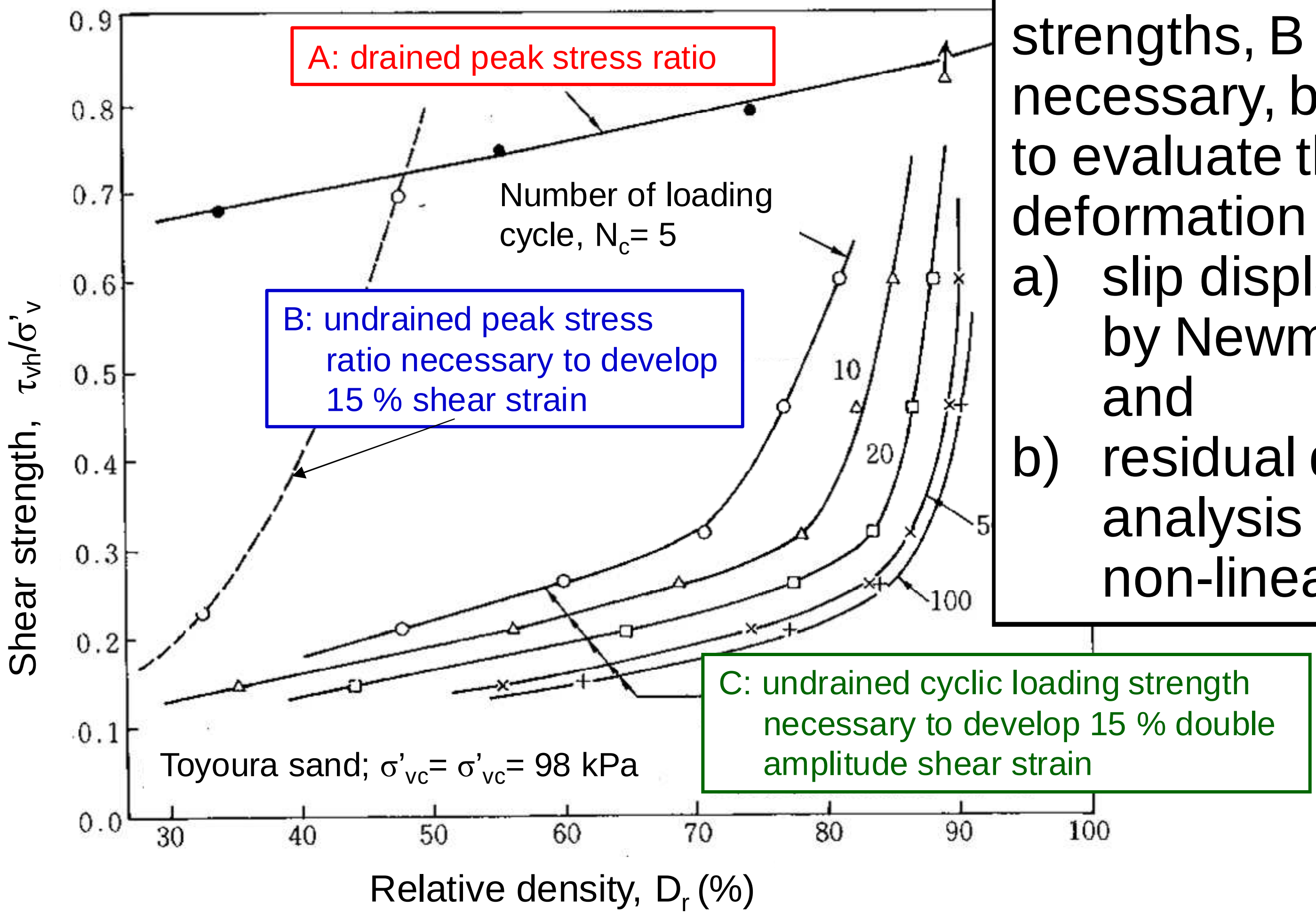
1. Effects of dry density - 7



1) Undrained cyclic shear strength increases with D_r, significantly when D_r becomes larger than a certain value: e.g., by a factor of three when D_r= 70 % → 90 %.
2) Significant benefits can be obtained by compaction to D_r higher than a certain value.

Undrained stress- strain behaviour of saturated soil:

1. Effects of dry density - 8



A: drained peak stress ratio

Number of loading cycle, N_c= 5

B: undrained peak stress ratio necessary to develop 15 % shear strain

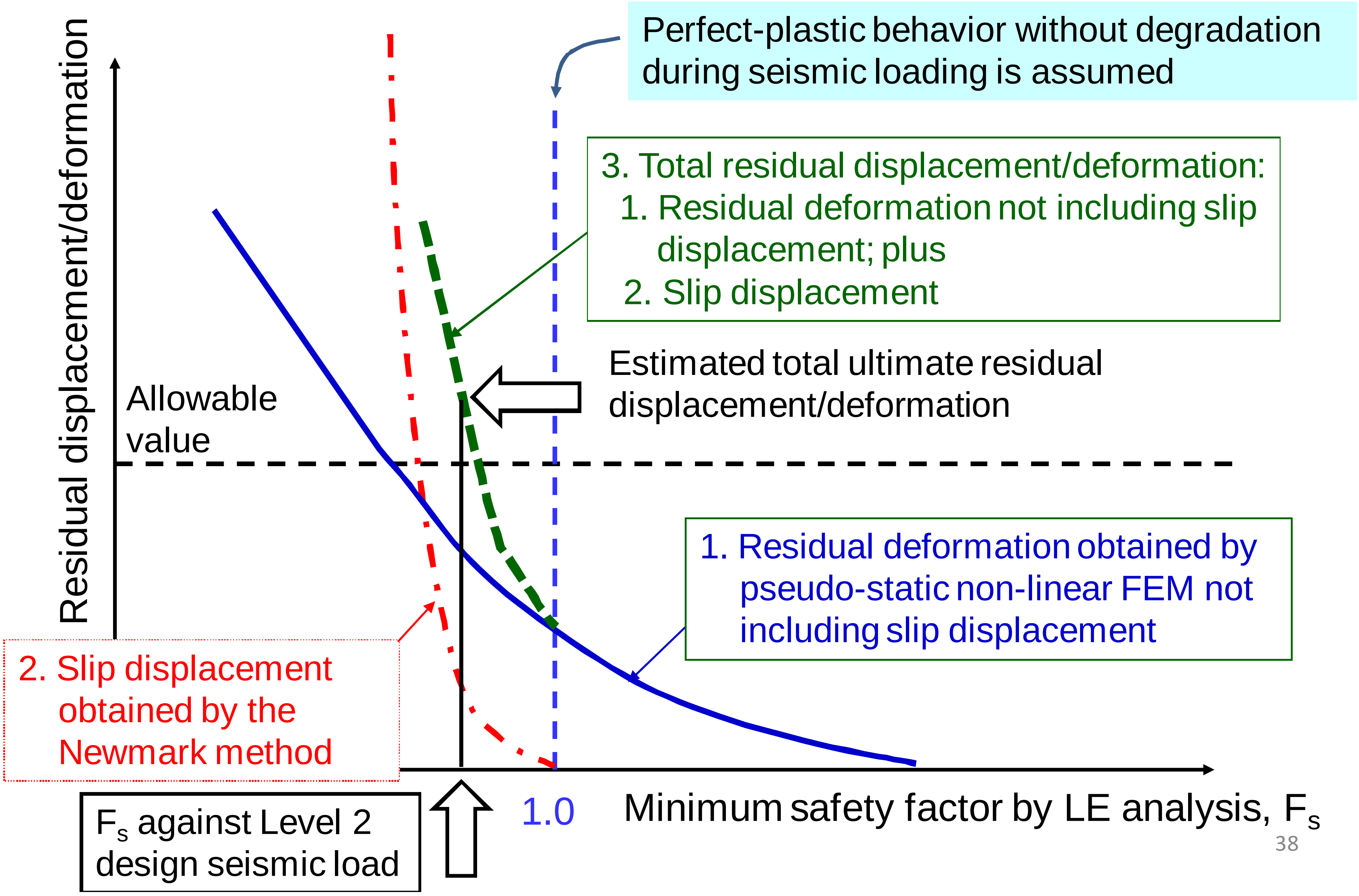
C: undrained cyclic loading strength necessary to develop 15 % double amplitude shear strain

Toyoura sand; σ'_{vc}= σ'_{vc}= 98 kPa

These undrained shear strengths, B & C, are necessary, but not sufficient, to evaluate the residual deformation by:

a) slip displacement analysis by Newmark-D method; and
b) residual deformation analysis by pseudo-static non-linear FEM.

Undrained stress- strain behaviour of saturated soil to evaluate the residual displacement/deformation-1



Perfect-plastic behavior without degradation during seismic loading is assumed

3. Total residual displacement/deformation:
   1. Residual deformation not including slip displacement; plus
   2. Slip displacement

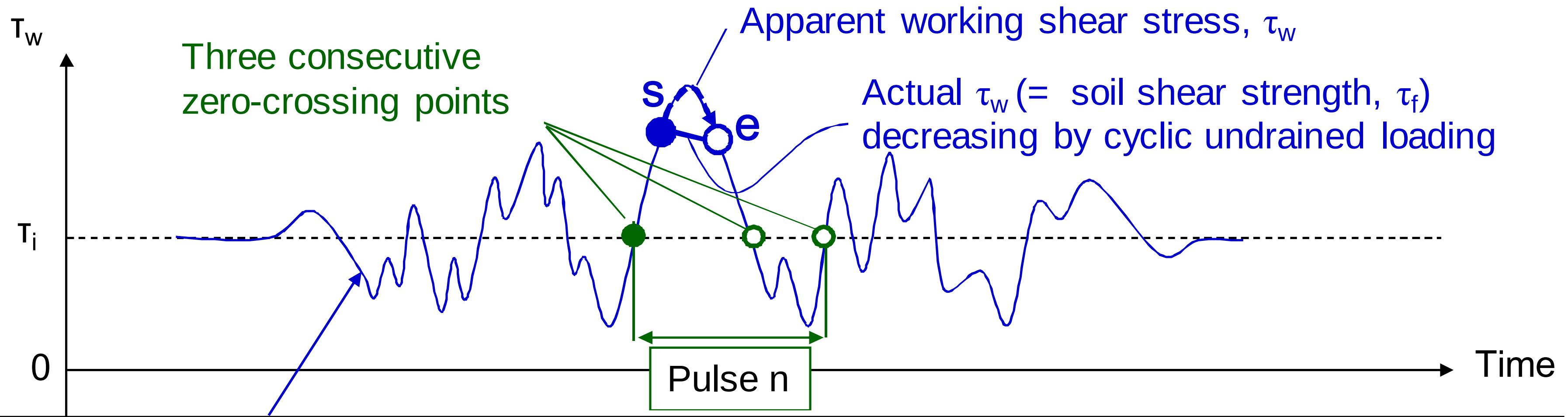
Estimated total ultimate residual displacement/deformation

1. Residual deformation obtained by pseudo-static non-linear FEM not including slip displacement

2. Slip displacement obtained by the Newmark method

F_s against Level 2 design seismic load

Undrained stress- strain behaviour of saturated soil to evaluate the residual displacement/deformation - 2



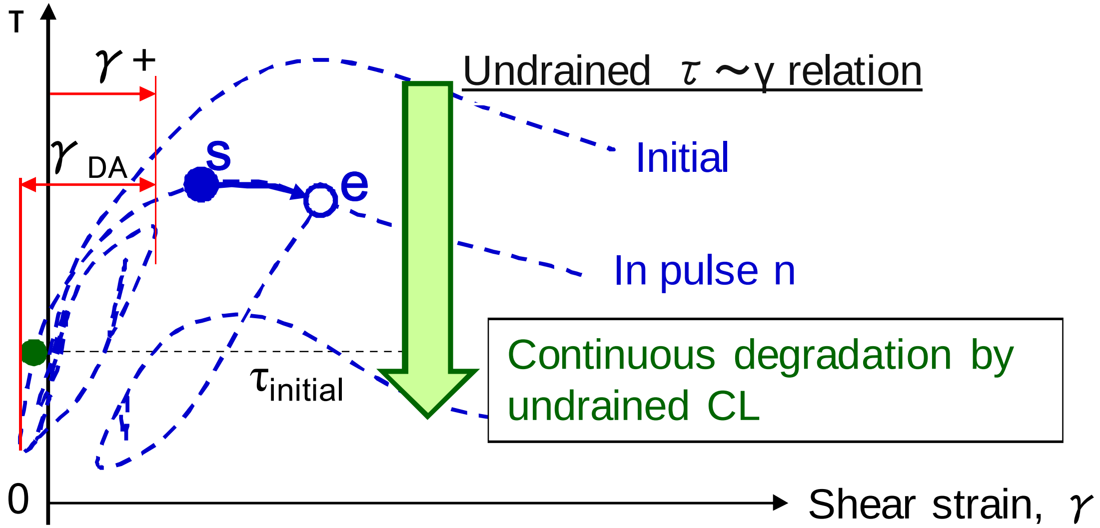
Three consecutive zero-crossing points

Apparent working shear stress, τ_w

Actual τ_w (= soil shear strength, τ_f) decreasing by cyclic undrained loading

Time history of apparent irregular working stress τ_w obtained by total stress seismic response analysis not taking into account both strength degradation by undrained CL and slip failure

Actual $\tau \sim \gamma$ behavior of soil



Undrained $\tau \sim \gamma$ relation

Initial

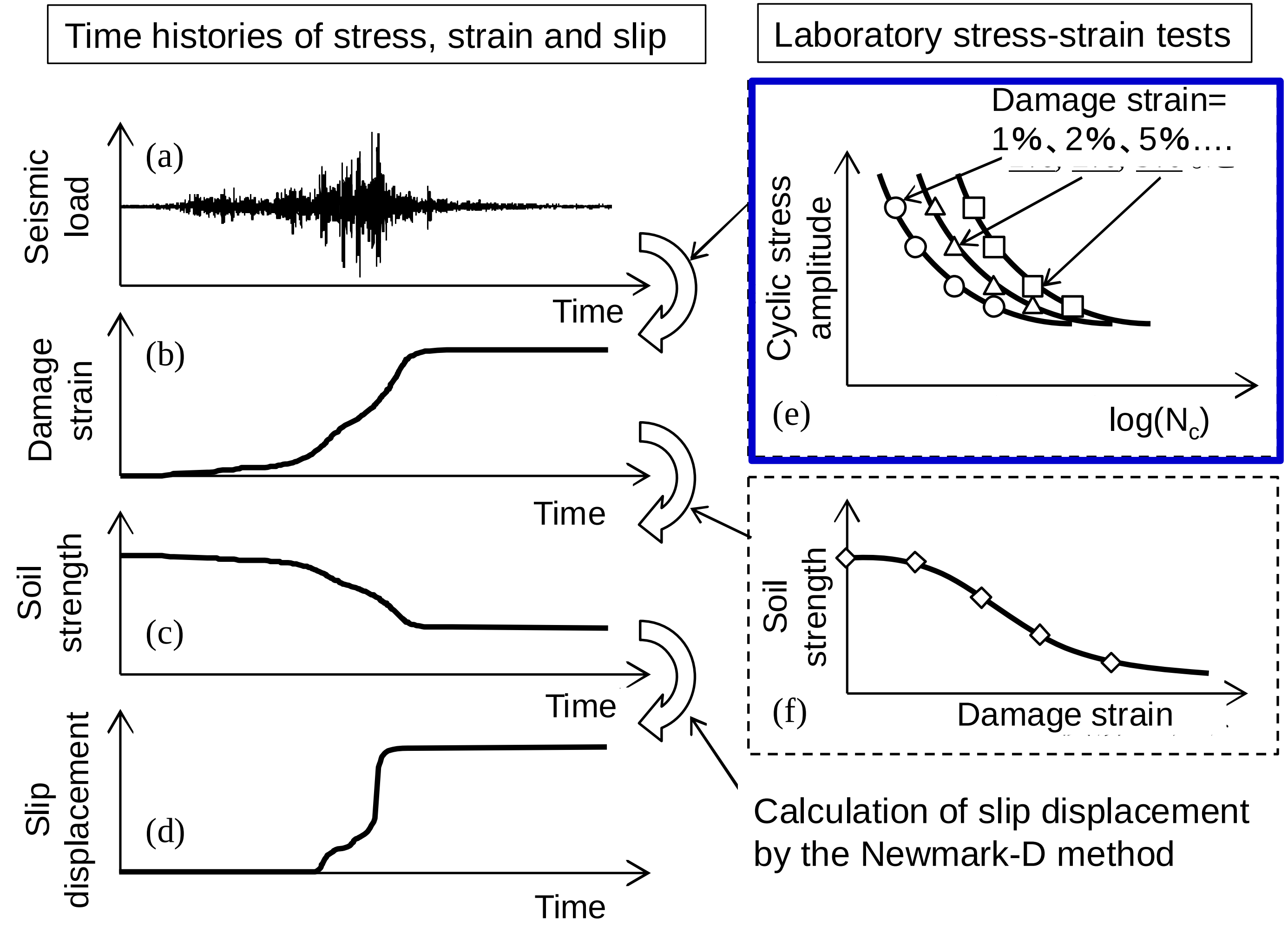
In pulse n

Continuous degradation by undrained CL

"Increments of slip displacement" in all pulses where slip takes place (such as s→e) are integrated to obtain the ultimate residual slip displacement

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 1:



Time histories of stress, strain and slip

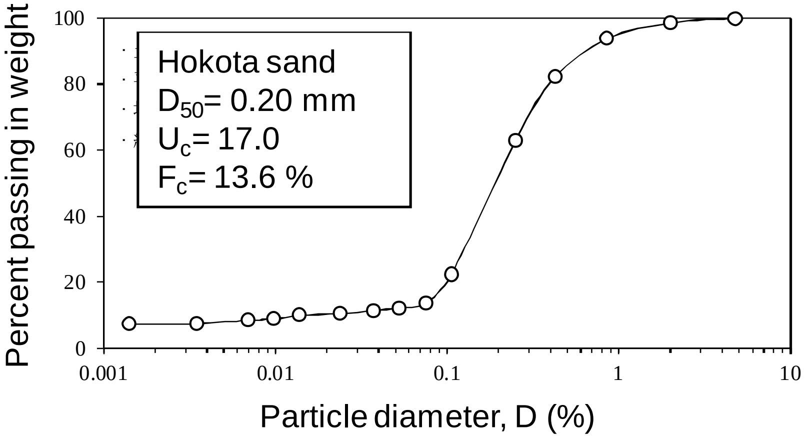
Laboratory stress-strain tests

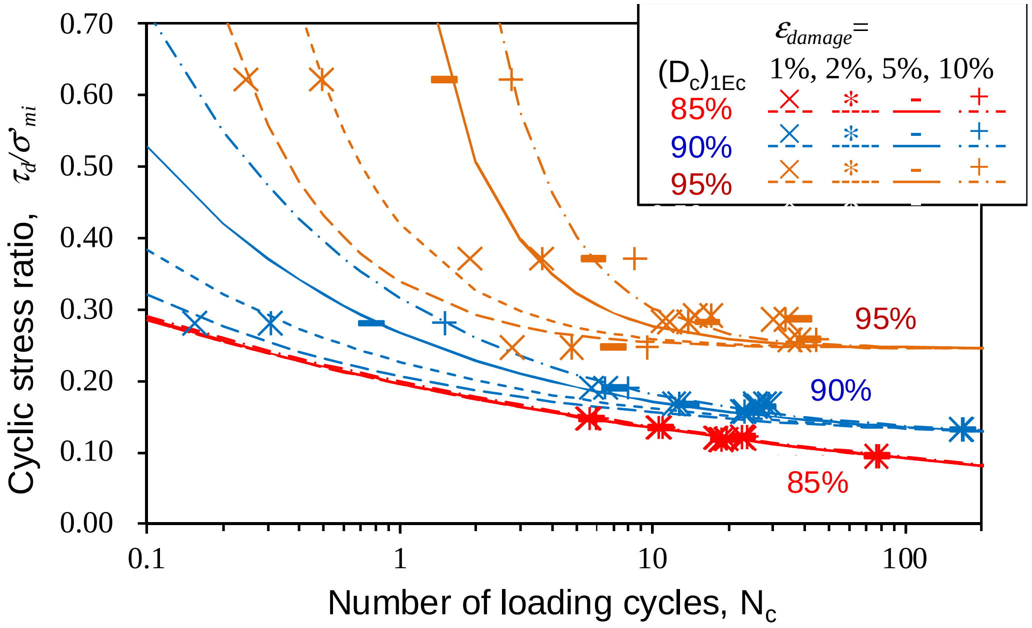
Calculation of slip displacement by the Newmark-D method

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 2:

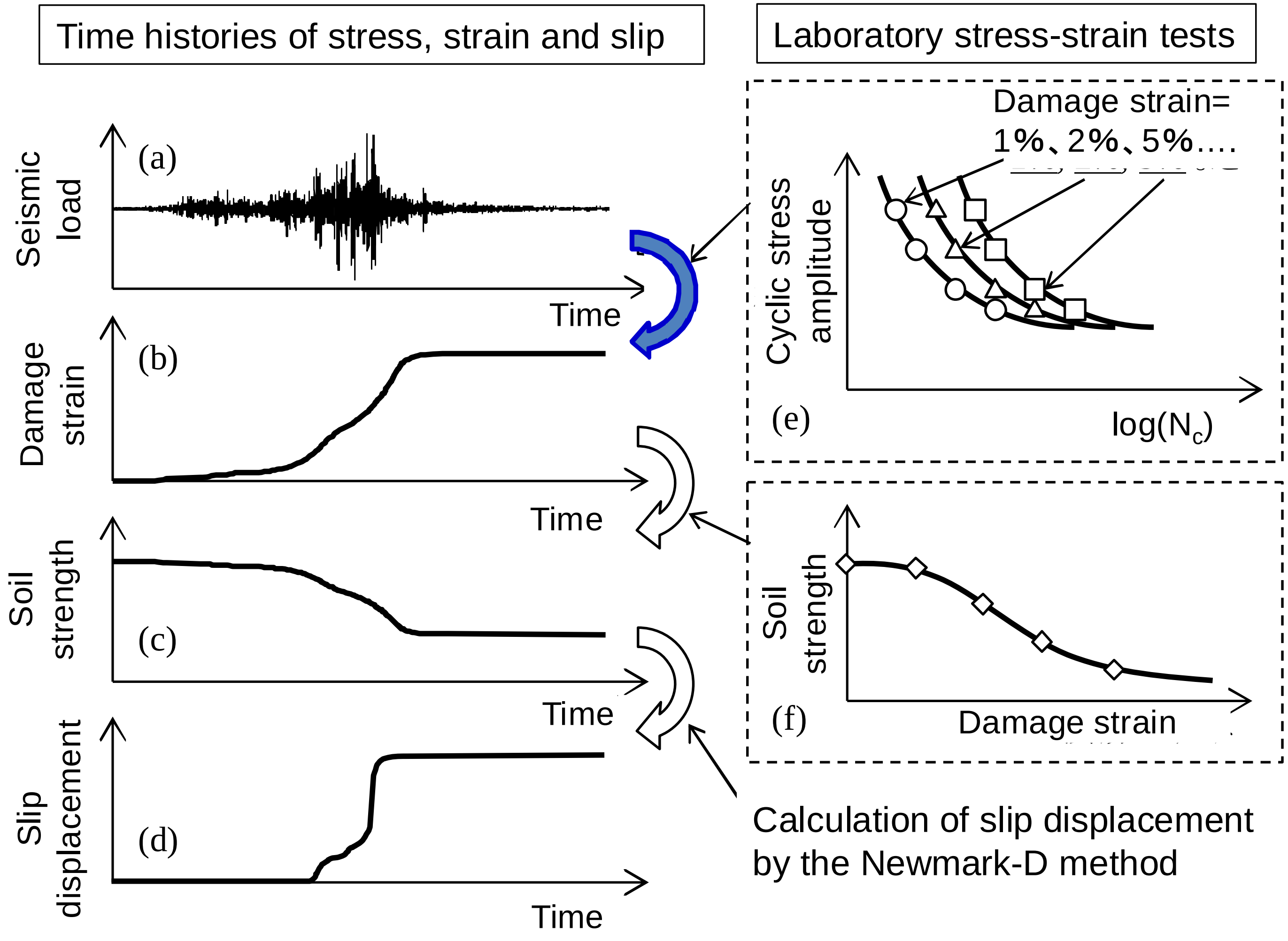
Typical example of cyclic undrained triaxial tests on isotropically consolidated specimens compacted to $(D_c)_{1Ec}$= 85 %; 90 % and 95 %





Undrained stress- strain behaviour of saturated soil

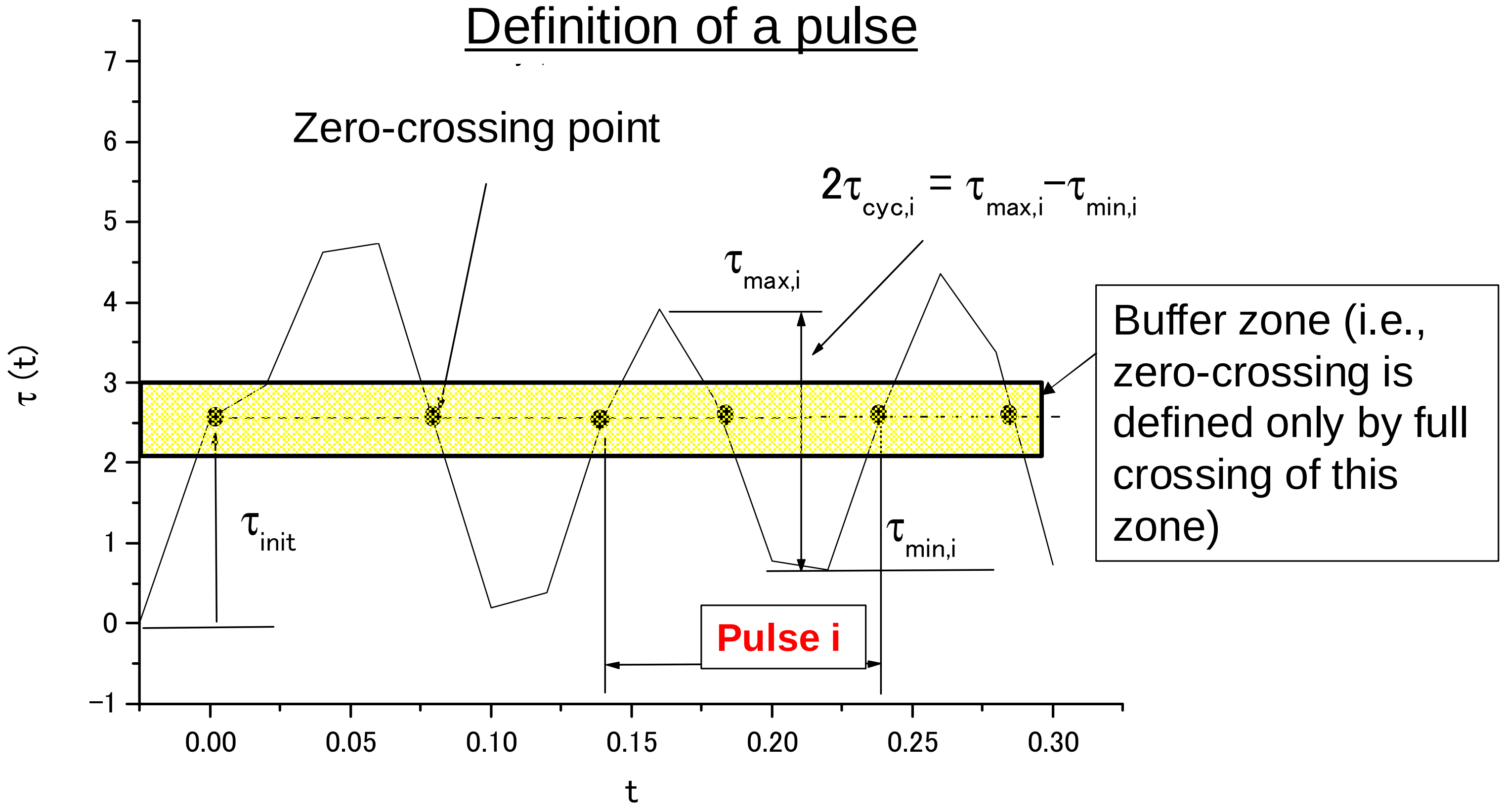
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 3:



Calculation of slip displacement by the Newmark-D method

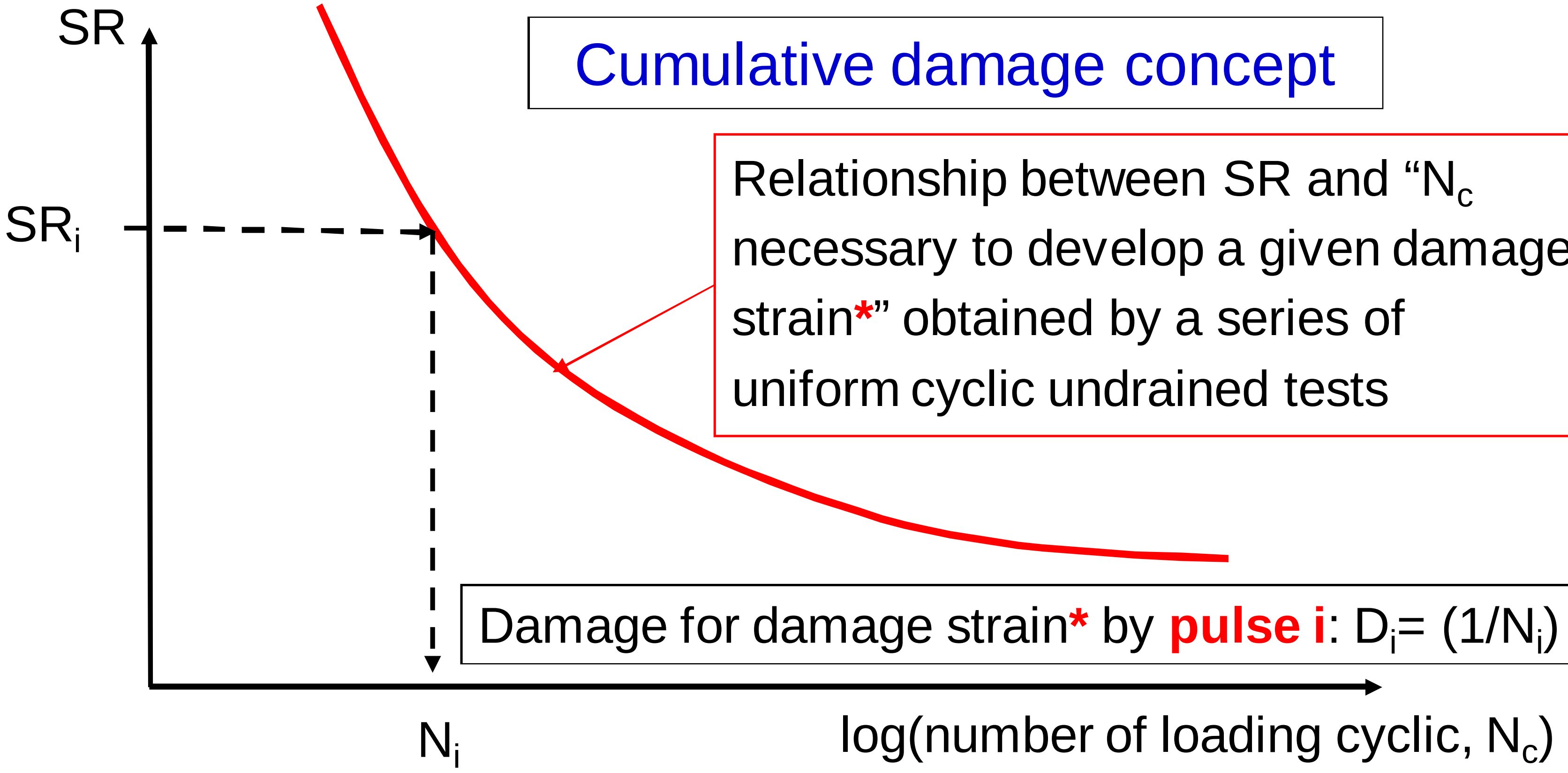
Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 4:



Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 5:



Cumulative damage concept

Relationship between SR and “N_c necessary to develop a given damage strain*” obtained by a series of uniform cyclic undrained tests

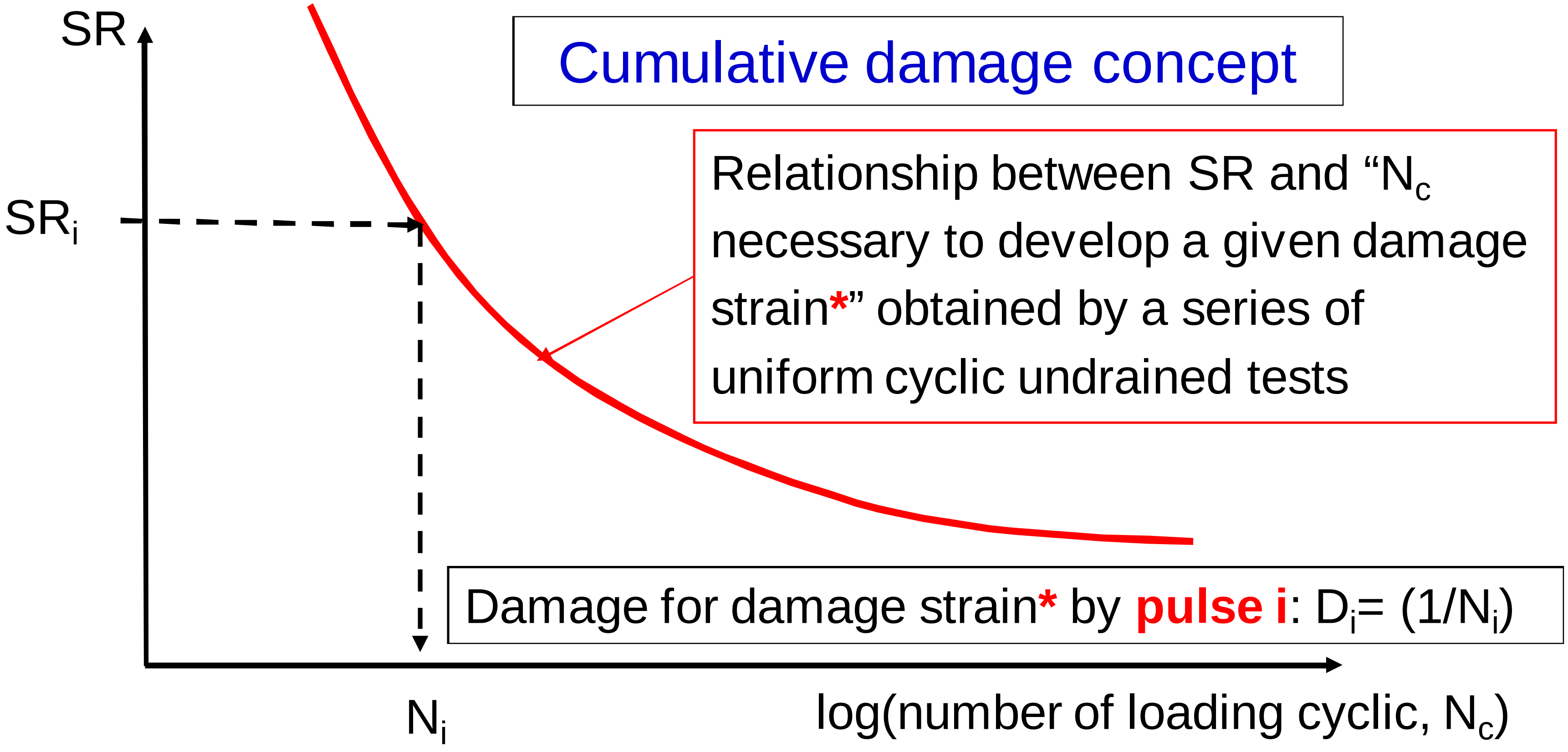
Damage for damage strain* by **pulse i**: $D_i = (1/N_i)$

$SR_i = \tau_{cyc,i}/\sigma'_0$: cyclic stress ratio of **pulse i**

$\tau_{cyc,i}$: shear stress amplitude; and σ'_0: initial effective confining stress

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 6:



Cumulative damage concept

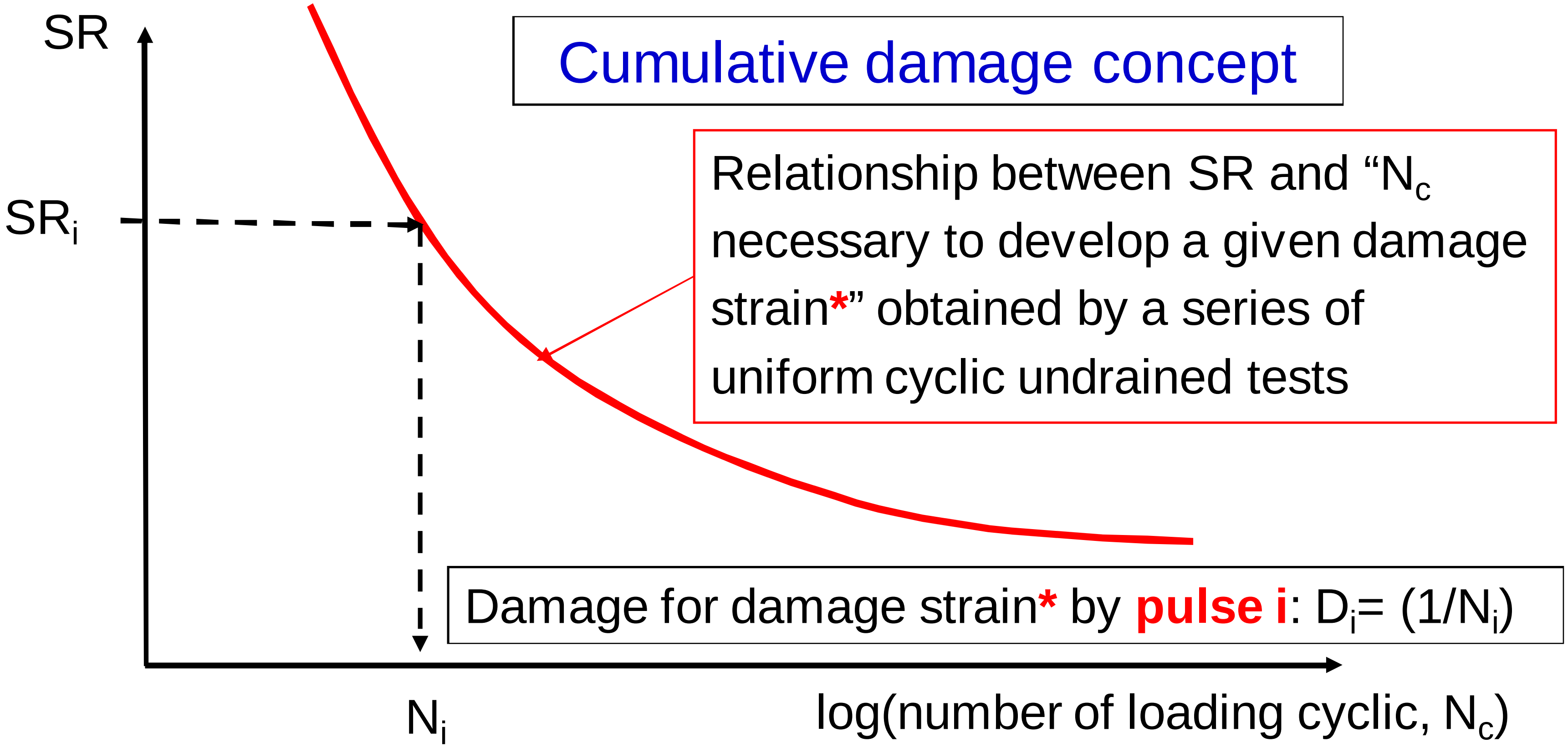
Relationship between SR and "N_c necessary to develop a given damage strain*" obtained by a series of uniform cyclic undrained tests

Damage for damage strain* by **pulse i**: $D_i = (1/N_i)$

Total damage for damage strain* caused
by a series of irregular pulses until the end of pulse n: $D = \sum_{i=1}^{n} D_i = \sum_{i=1}^{n} \frac{1}{N_i}$
If D becomes 1.0 at the end of pulse n,
it is assumed that this damage strain * takes place in pulse n.

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 7:



Cumulative damage concept

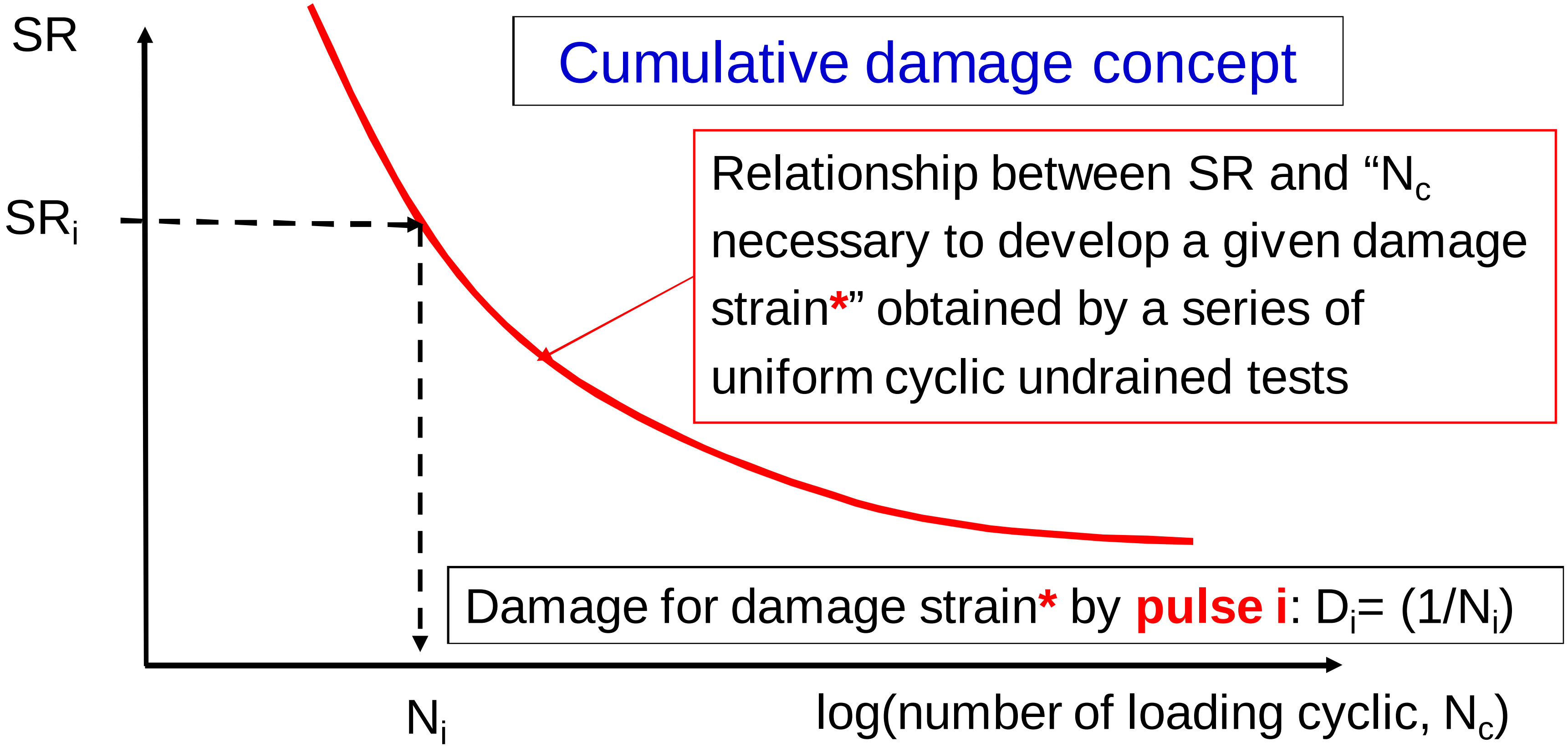
Relationship between SR and "N_c necessary to develop a given damage strain*" obtained by a series of uniform cyclic undrained tests

Damage for damage strain* by **pulse i**: $D_i = (1/N_i)$

Then, we can find the damage strain at the end of pulse n at which the total damage $D = \sum_{i=1}^{n} \frac{1}{N_i}$ becomes 1.0.

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 8:



Cumulative damage concept

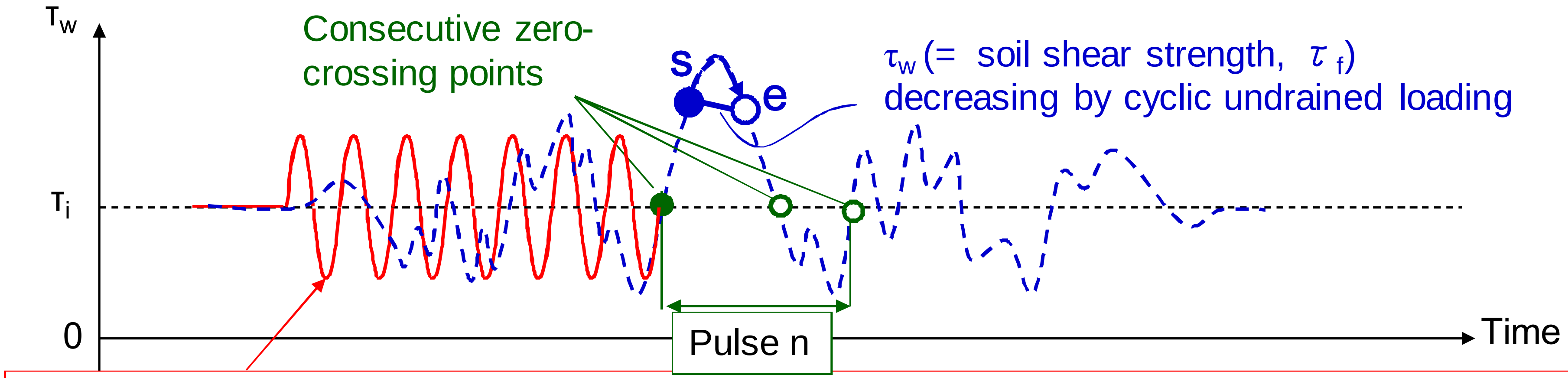
Relationship between SR and "N_c necessary to develop a given damage strain*" obtained by a series of uniform cyclic undrained tests

Damage for damage strain* by **pulse i**: $D_i = (1/N_i)$

By this procedure, "a given time history of irregular cyclic stresses causing a certain damage strain can be converted to "uniform cyclic stresses with an arbitrary combination of SR & N_c that develops the same damage strain".

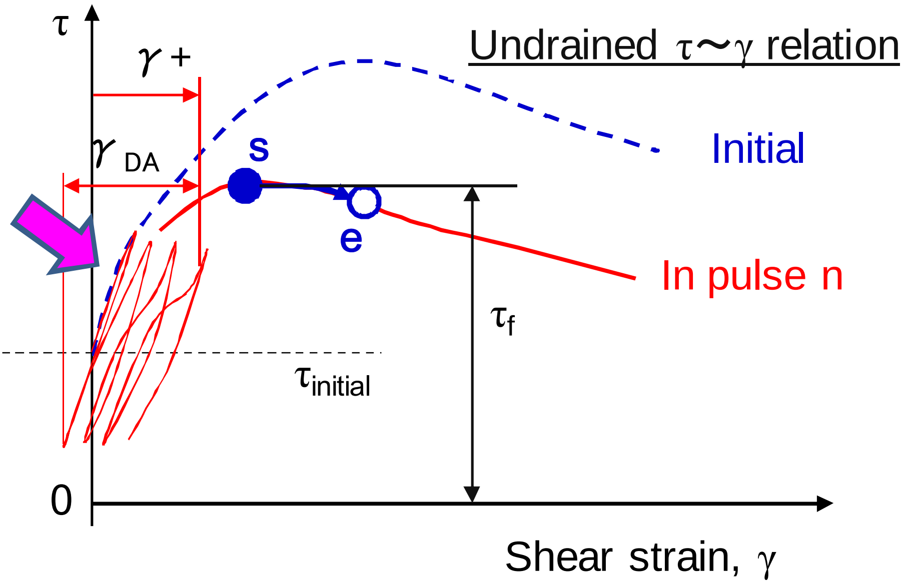
Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 9:



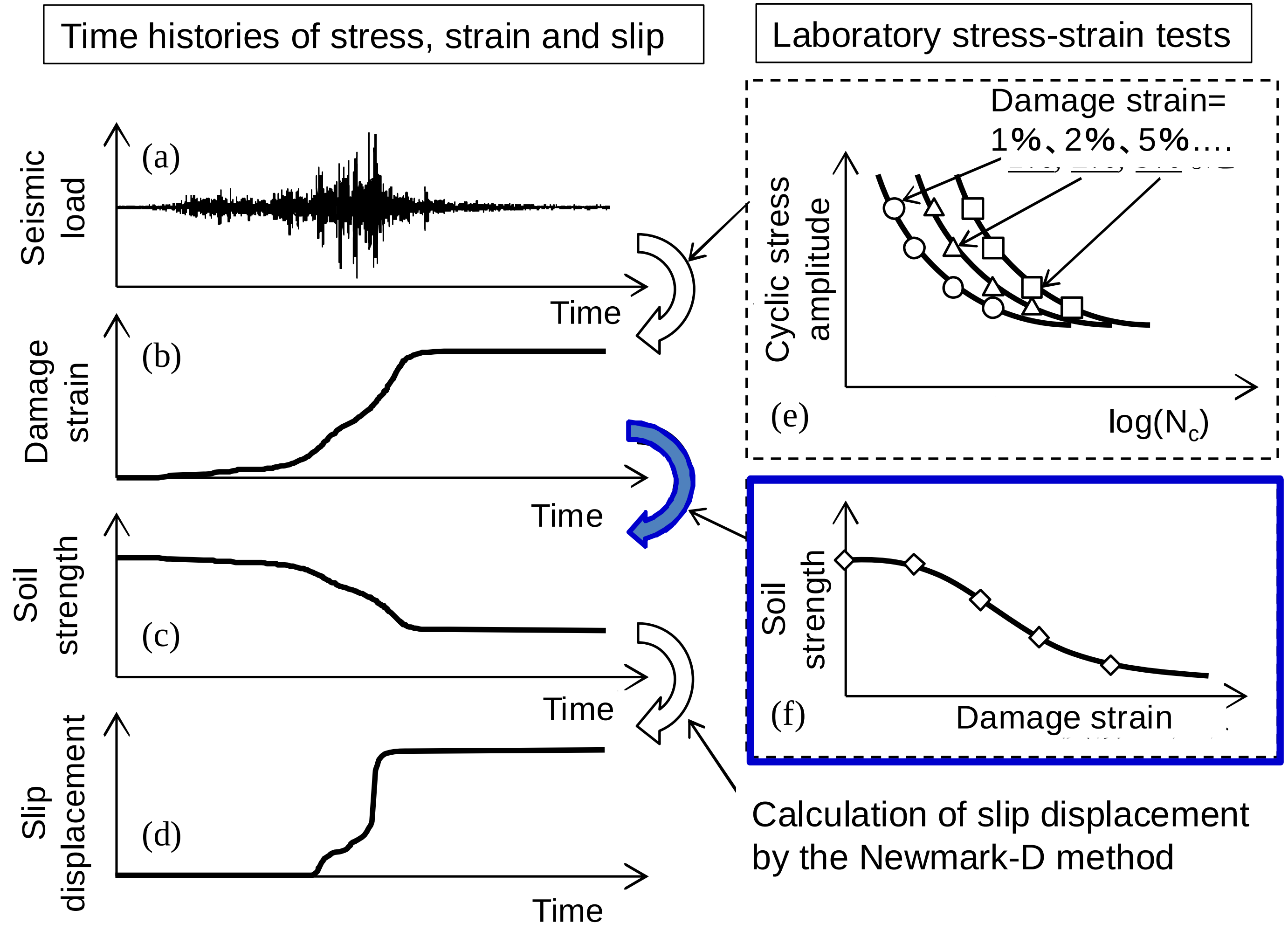
Uniform cyclic stresses equivalent to “irregular working stresses before the start of pulse n” obtained by the cumulative damage concept.

Equivalent τ∼γ behavior after have been subjected to “*equivalent uniform cyclic stresses obtained by the cumulative damage concept*”



Undrained stress- strain behaviour of saturated soil

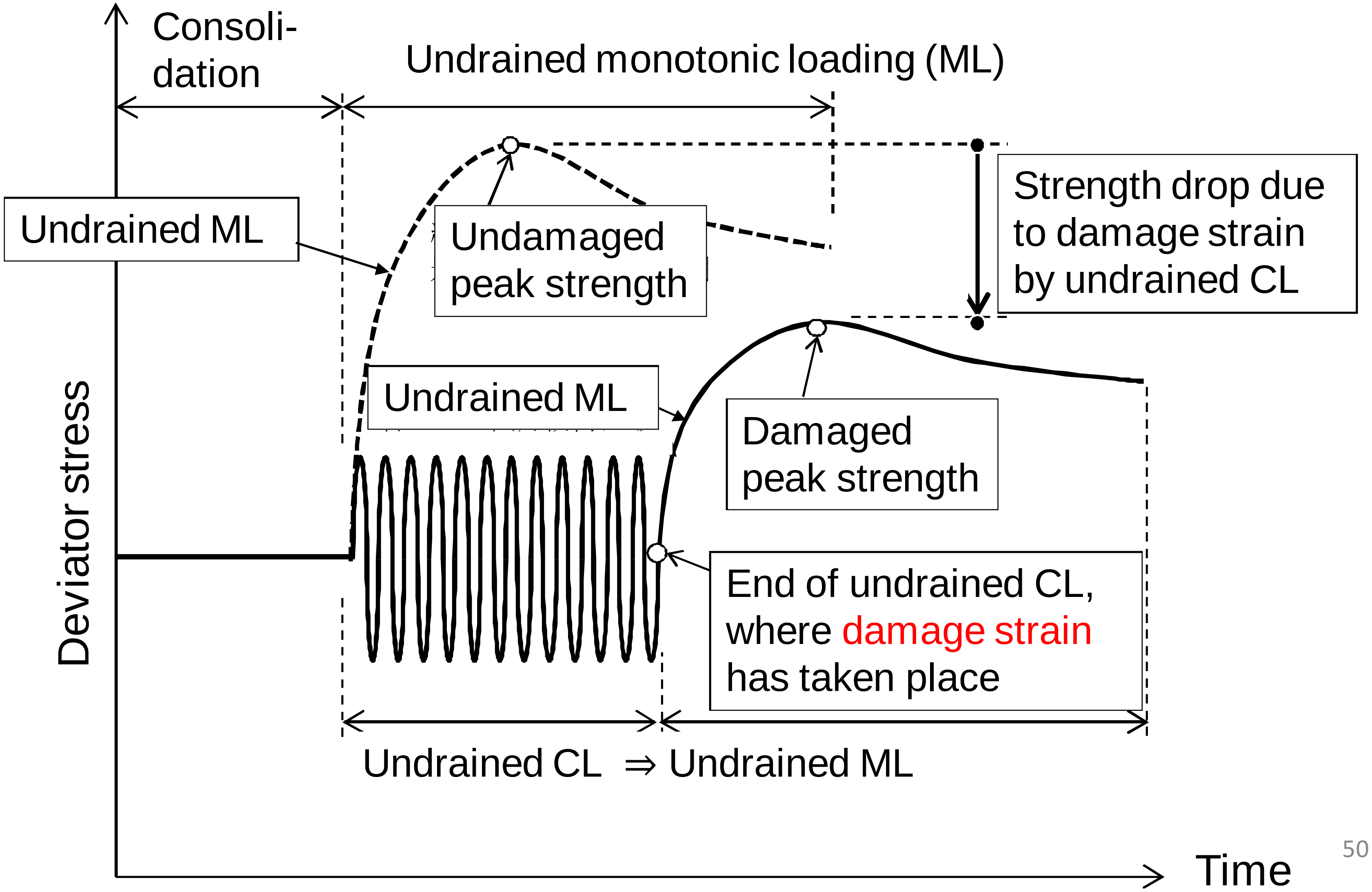
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 10:



Calculation of slip displacement by the Newmark-D method

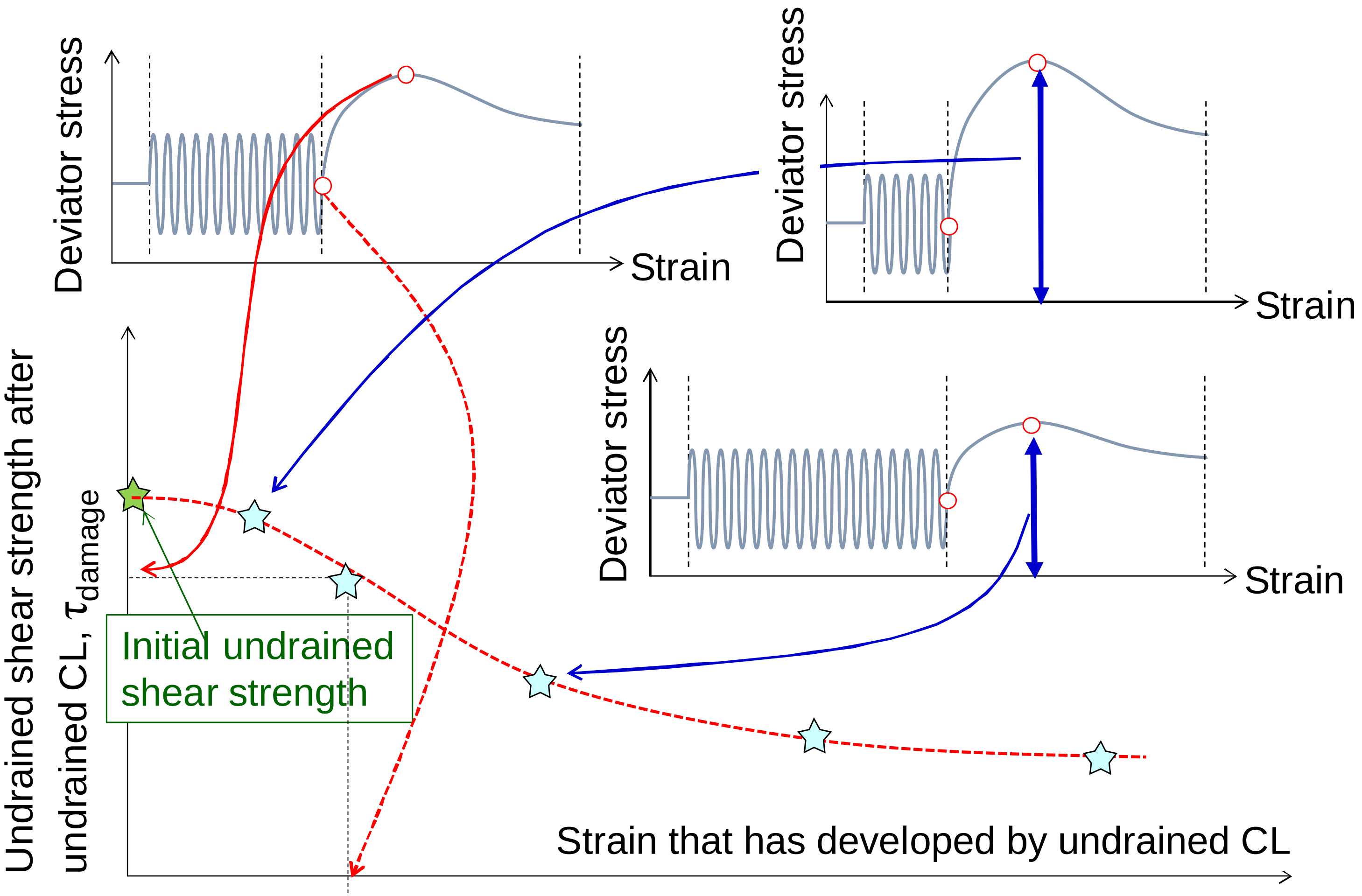
Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 11:



Undrained stress- strain behaviour of saturated soil

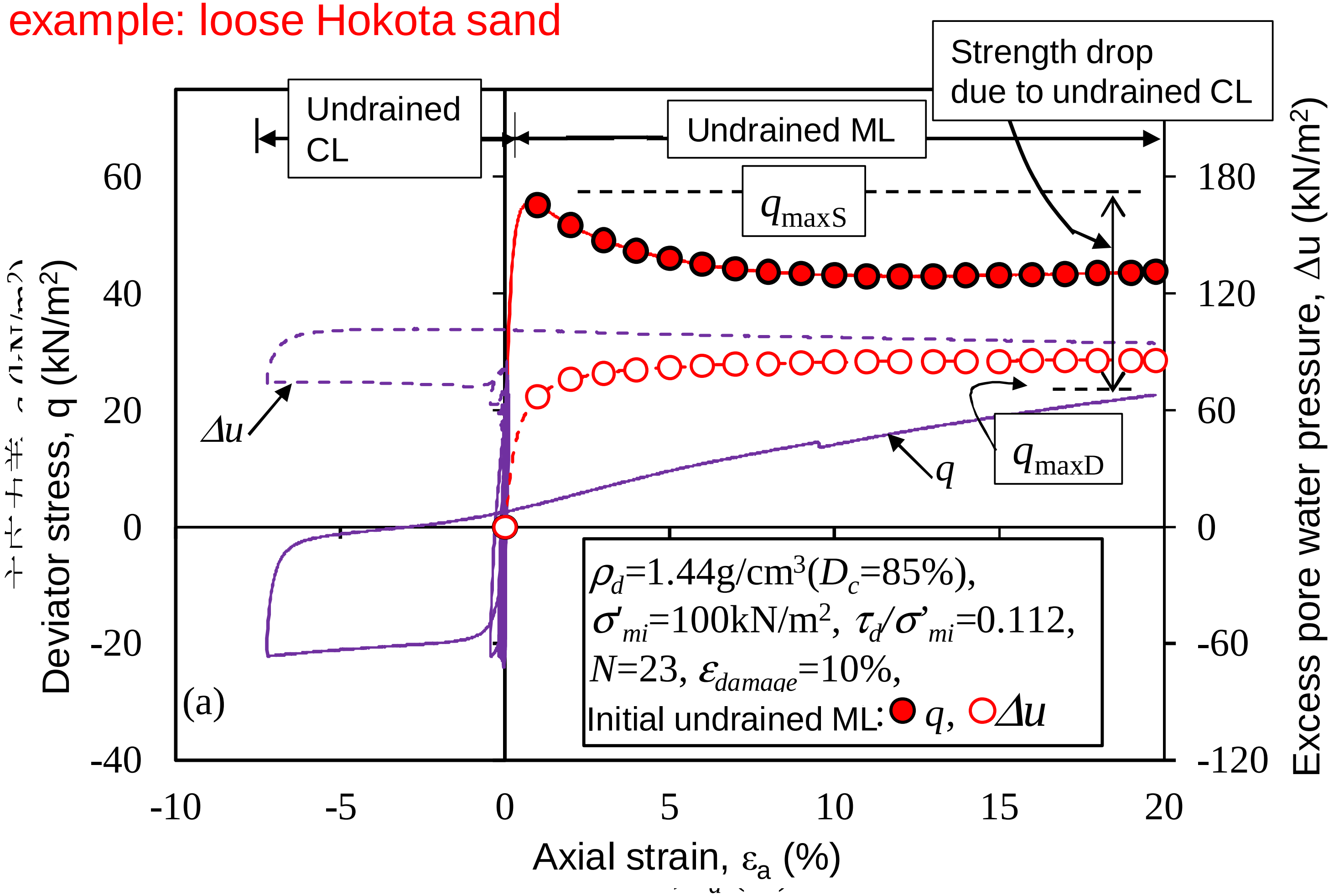
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 12:



Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 13:

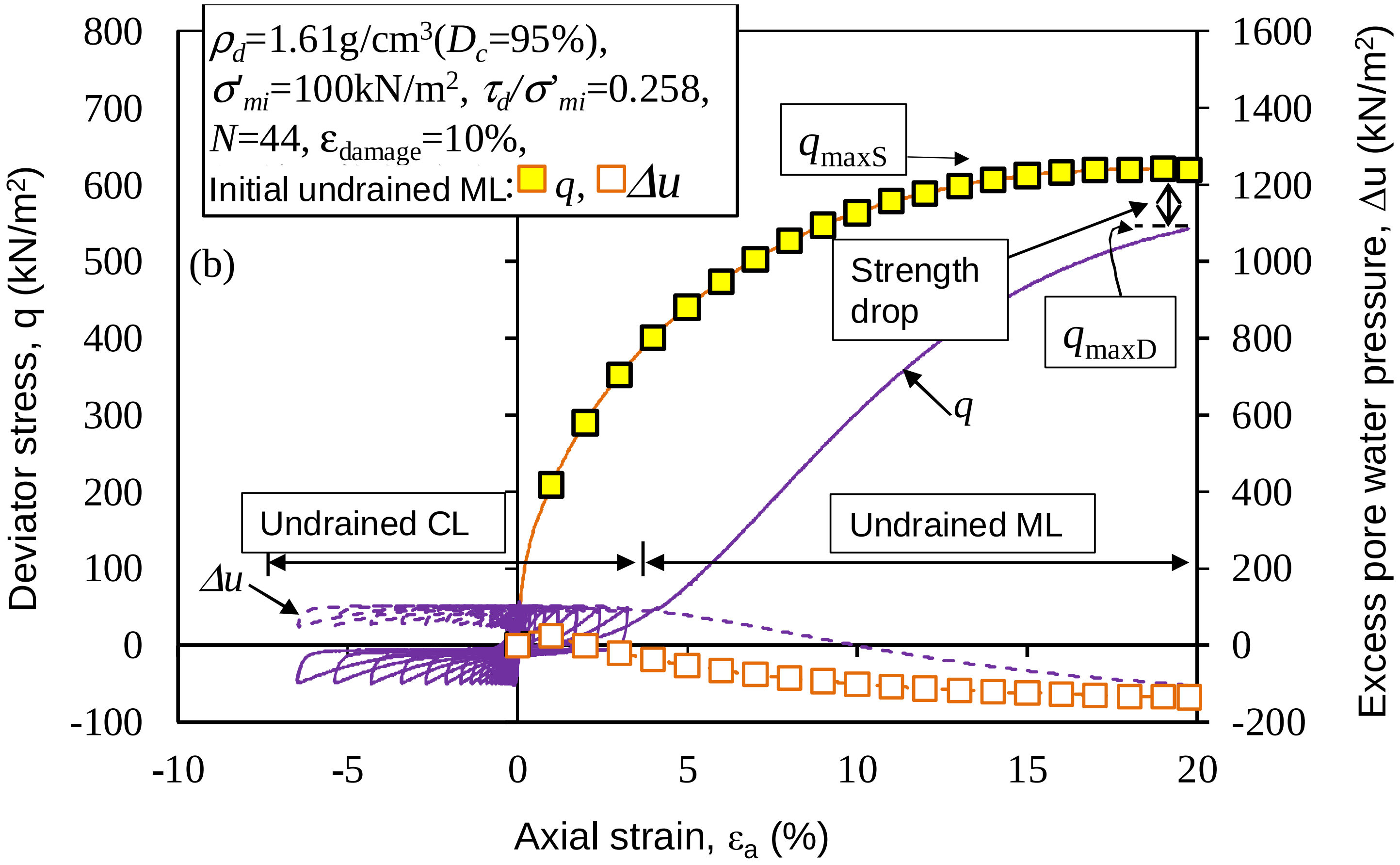
Typical example: loose Hokota sand



Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 14:

Typical example: dense Hokota sand



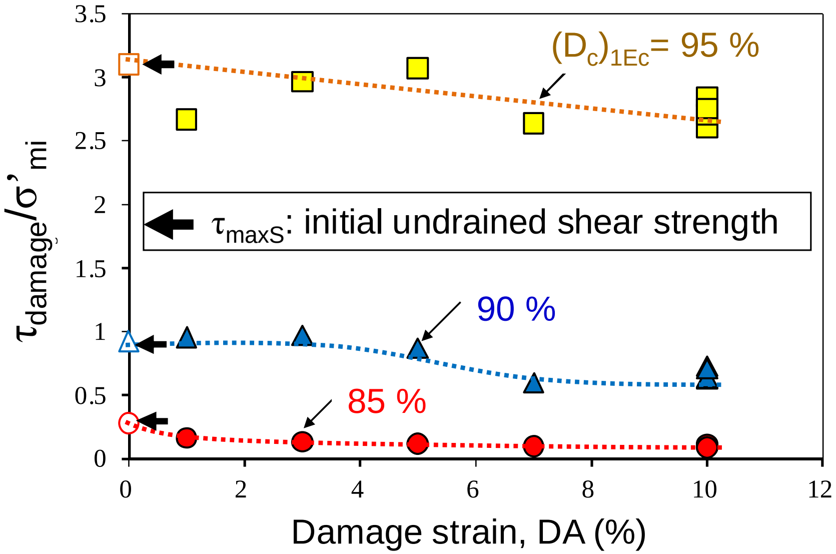
Undrained stress- strain behaviour of saturated soil

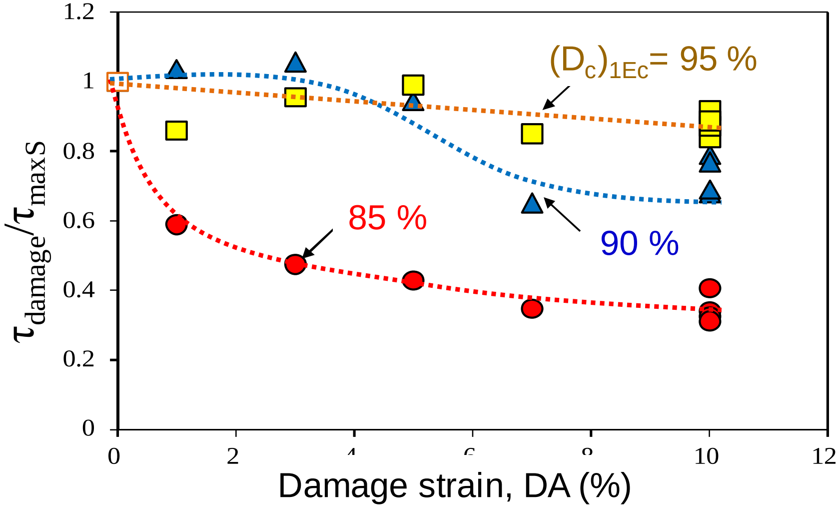
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 15:

τ_{damage}: undrained shear strength after undrained CL
σ'_{mi}: initial mean effective stress”

Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:

1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.



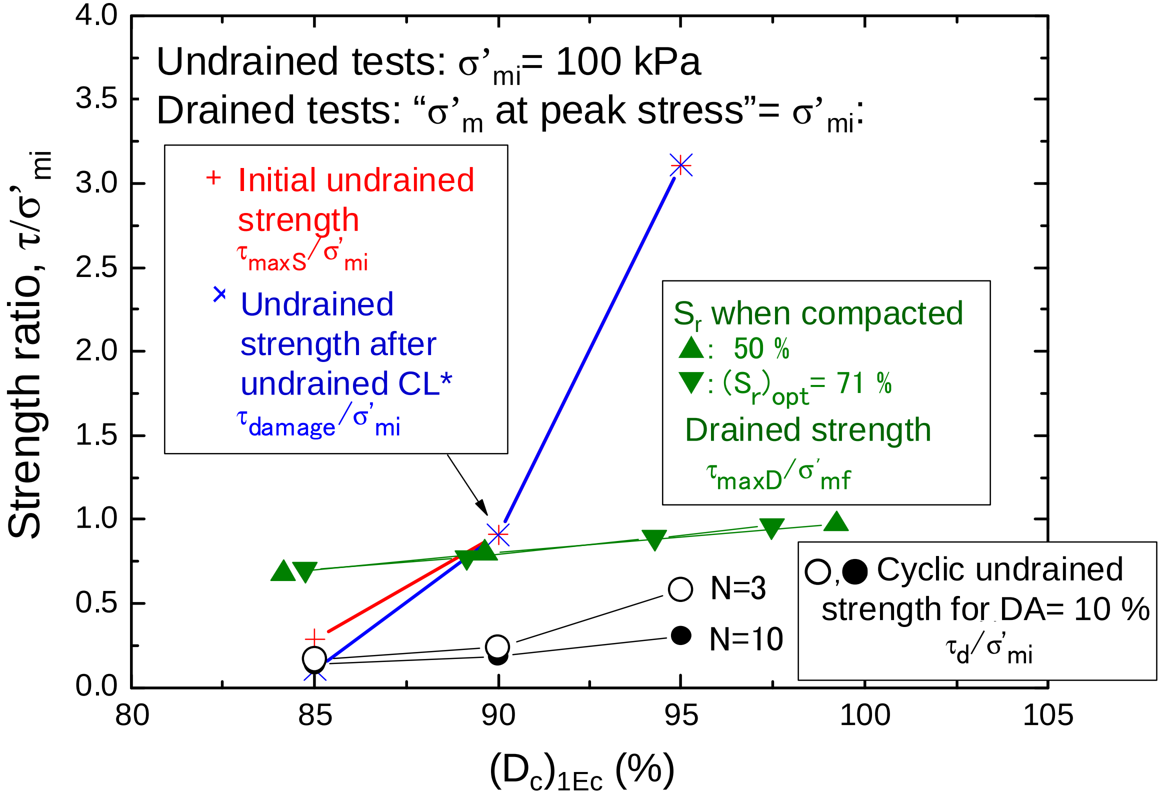


Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 16:

Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:

1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.



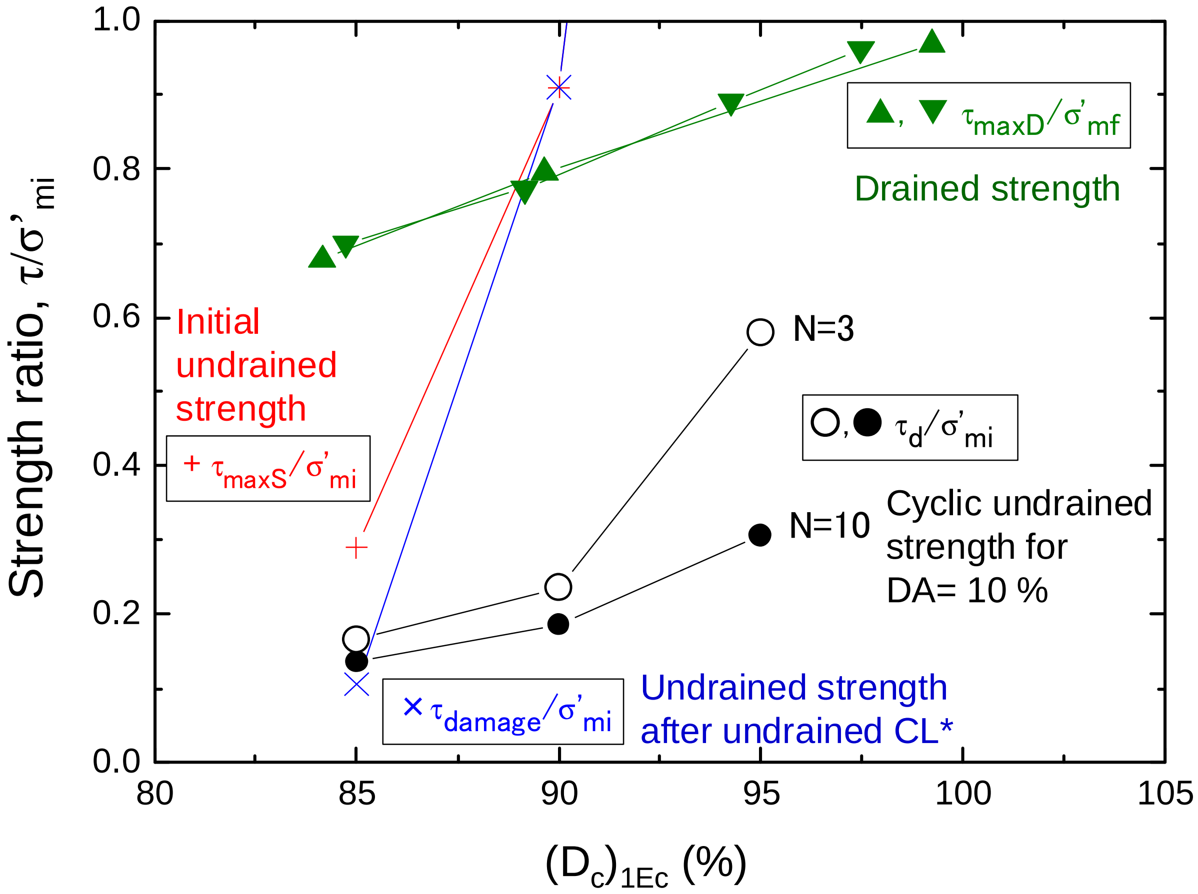
* τ_d/σ'_{mi}= 0.19; N_c= 2

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 17:

Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:

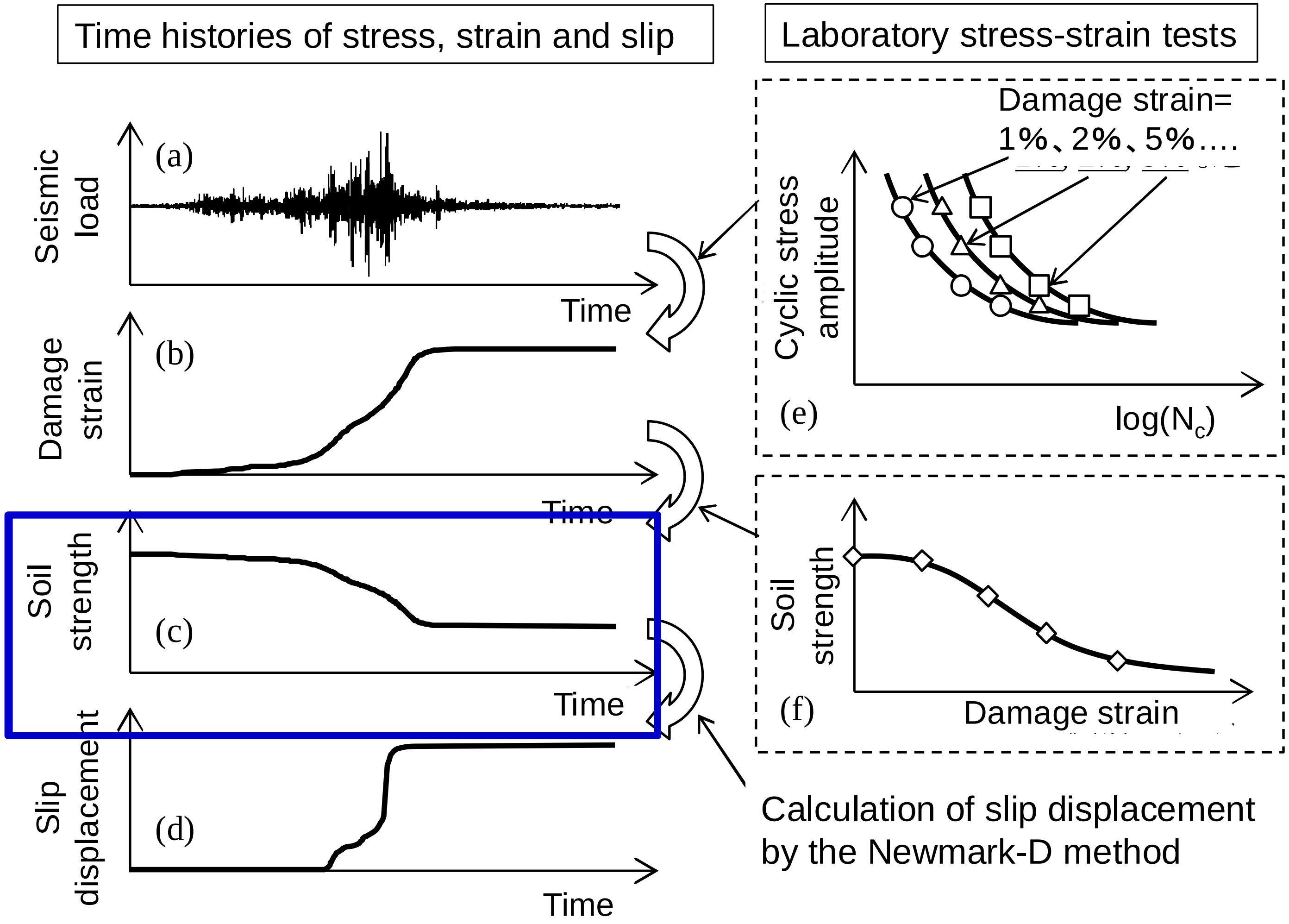
1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.



* $\tau_d/\sigma'_{mi} = 0.19$; $N_c = 2$

Undrained stress- strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 18:

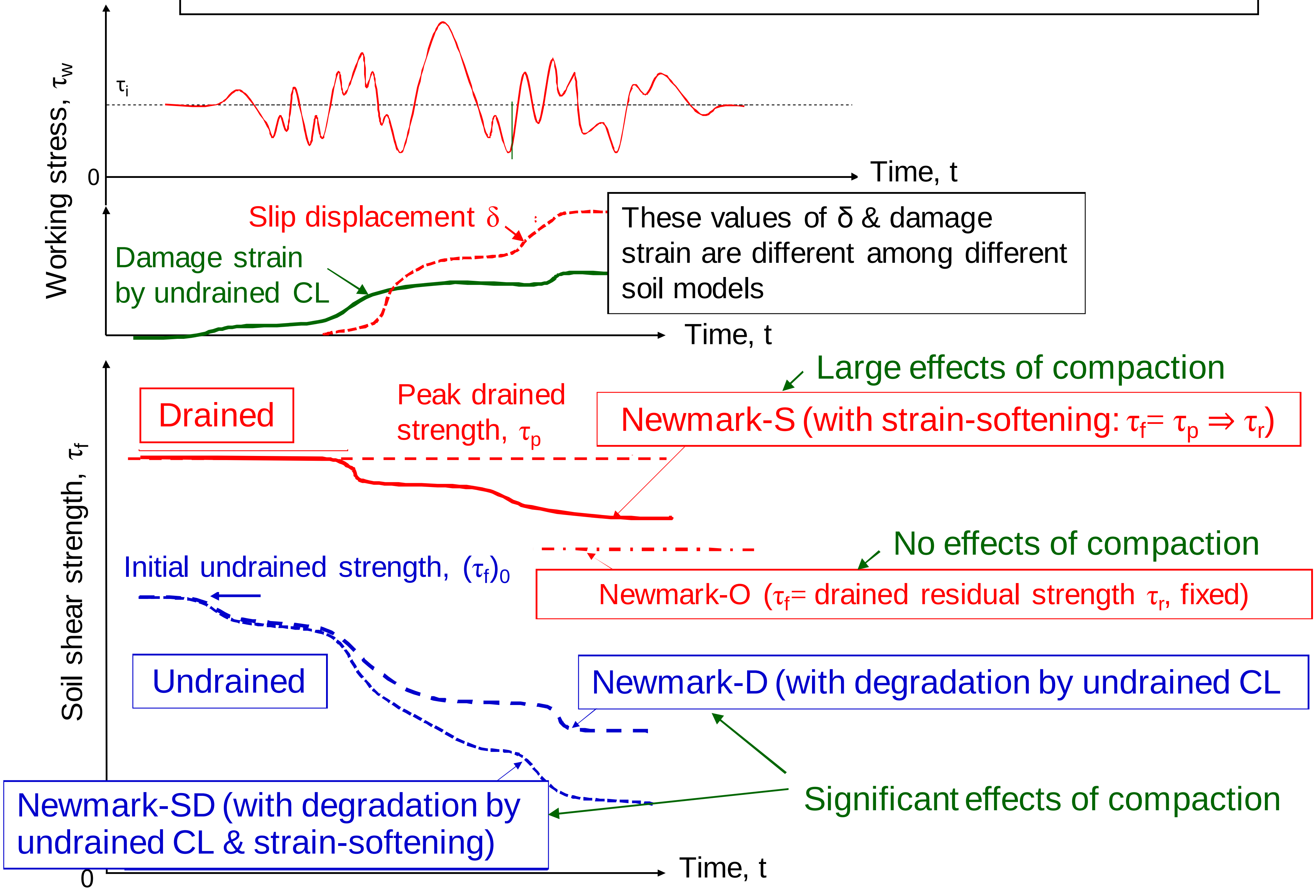


Time histories of stress, strain and slip

Laboratory stress-strain tests

Calculation of slip displacement by the Newmark-D method

Different soil models for different Newmark methods



These values of δ & damage strain are different among different soil models

Large effects of compaction

Newmark-S (with strain-softening: $\tau_f = \tau_p \Rightarrow \tau_r$)

No effects of compaction

Newmark-O (τ_f= drained residual strength τ_r, fixed)

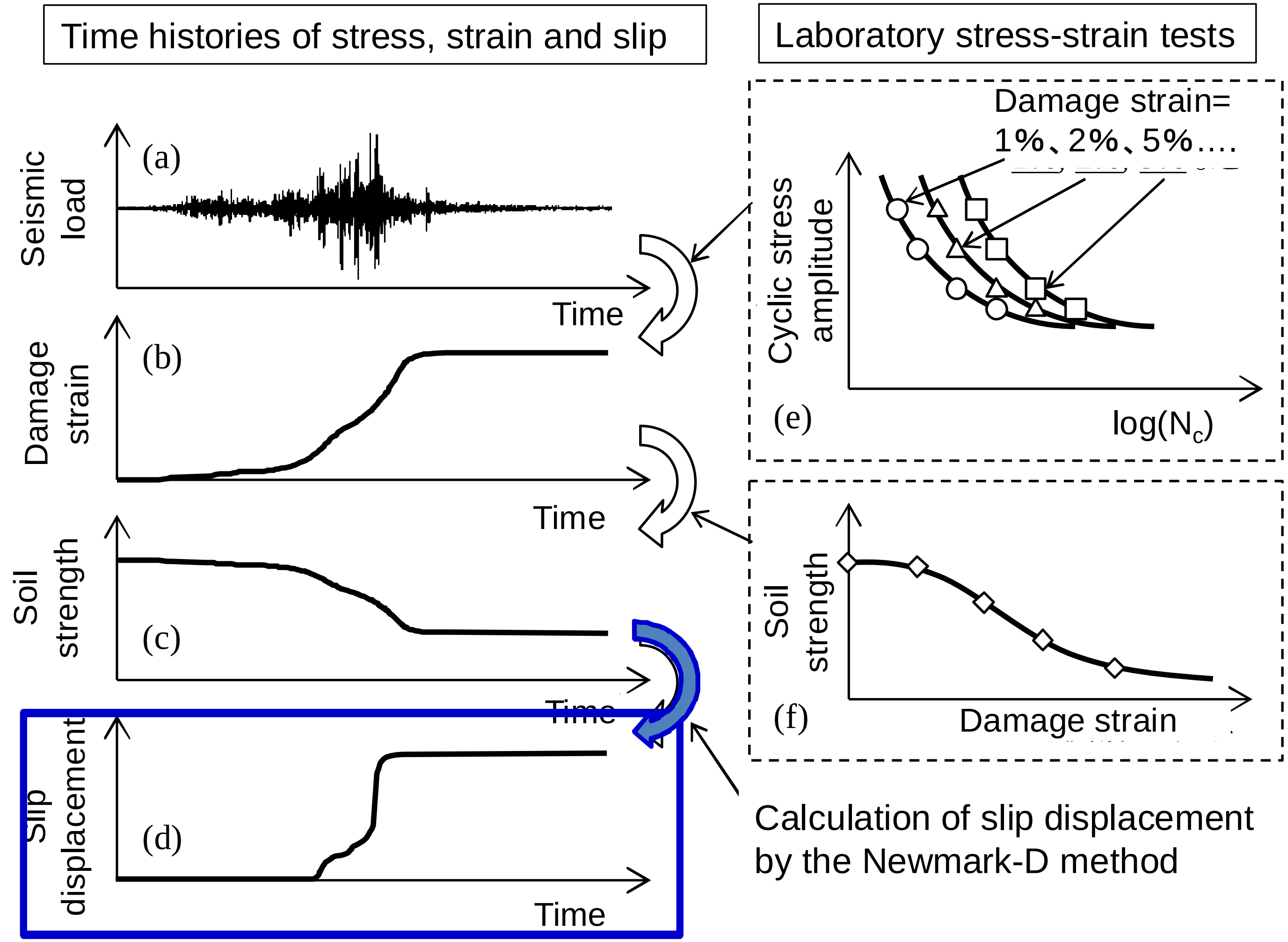
Newmark-D (with degradation by undrained CL

Newmark-SD (with degradation by undrained CL & strain-softening)

Significant effects of compaction

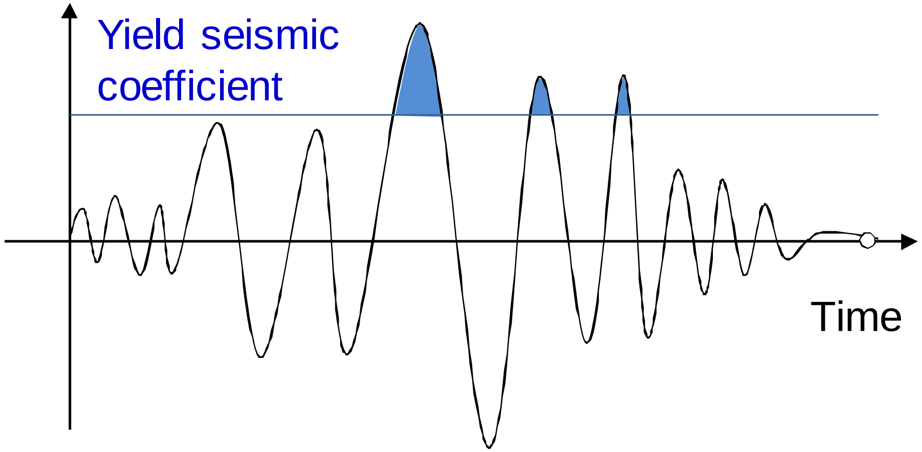
Undrained stress- strain behaviour of saturated soil

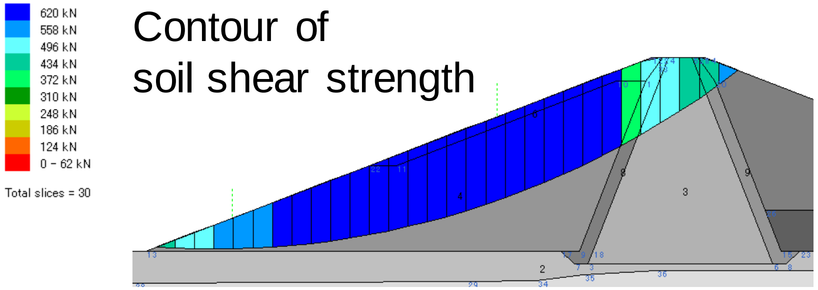
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 20:

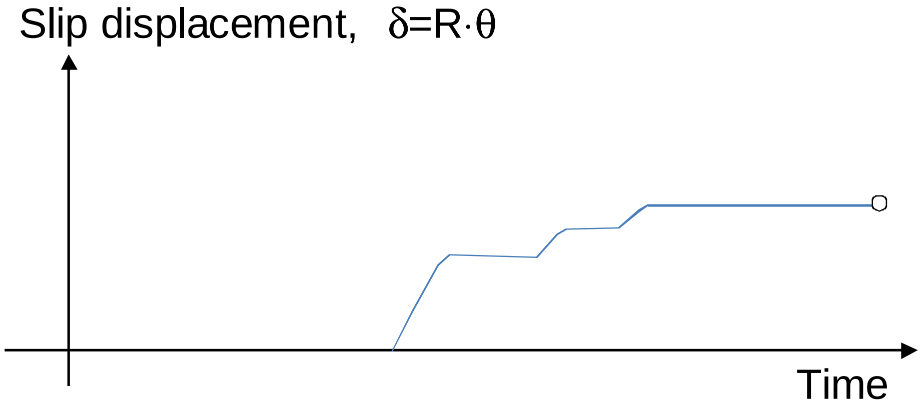


Calculation of slip displacement by the Newmark-D method

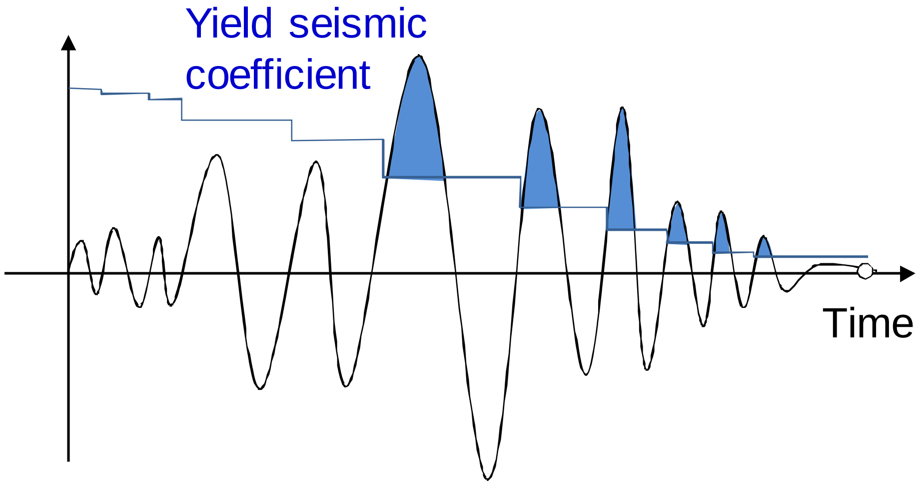
Newmark-O

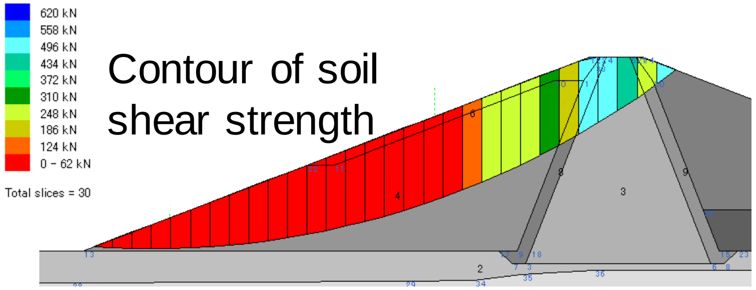


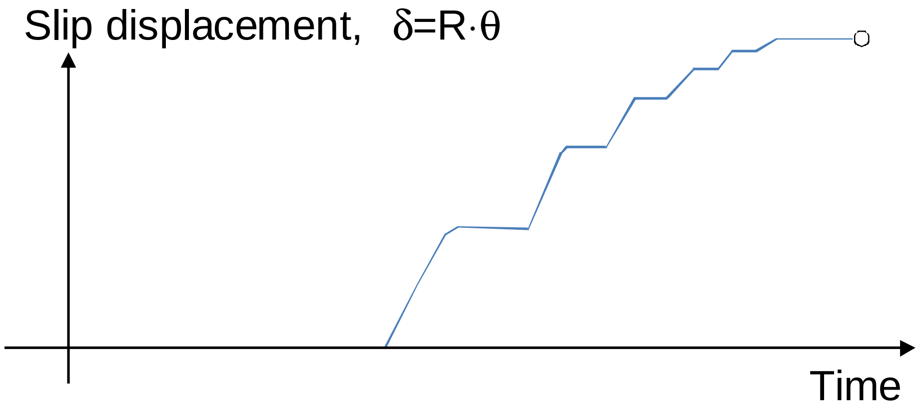




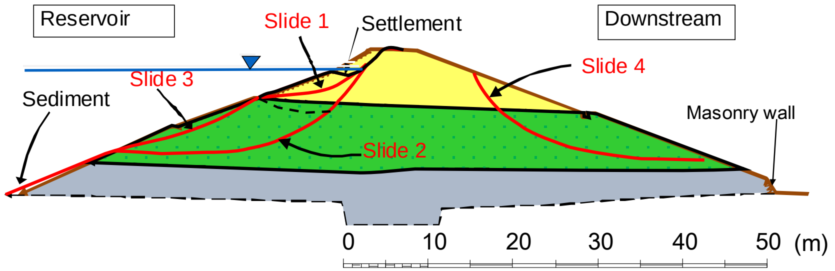
Newmark-D



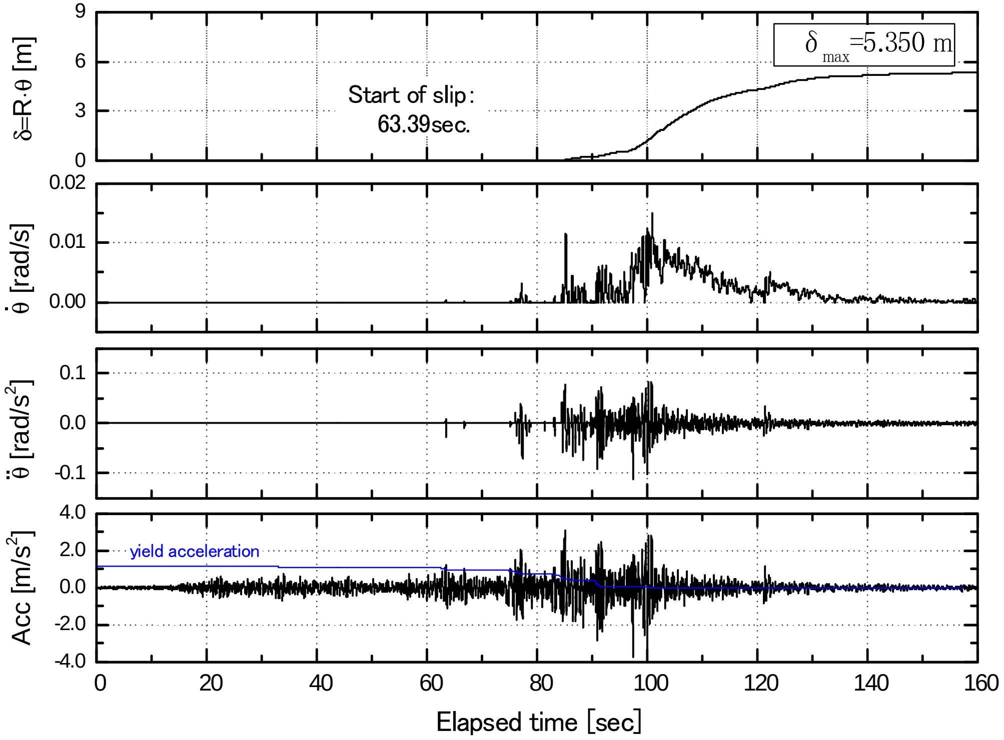




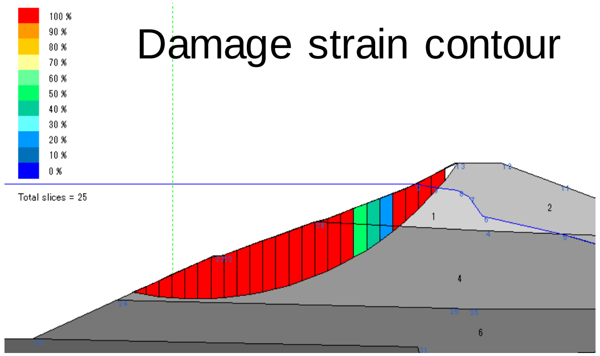
Slip displacement by Newmark-D analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake



Circular slide 2



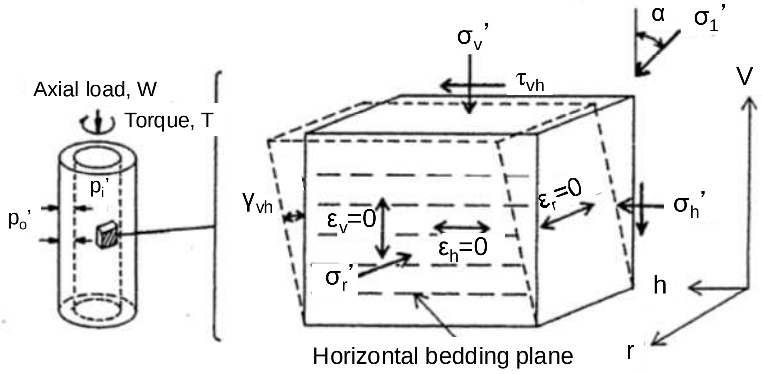
Very large ultimate slip displacement:
-Slip continues after the moment of peak acceleration (t= 97.01 s) due to continuing deterioration in the undrained shear strength.

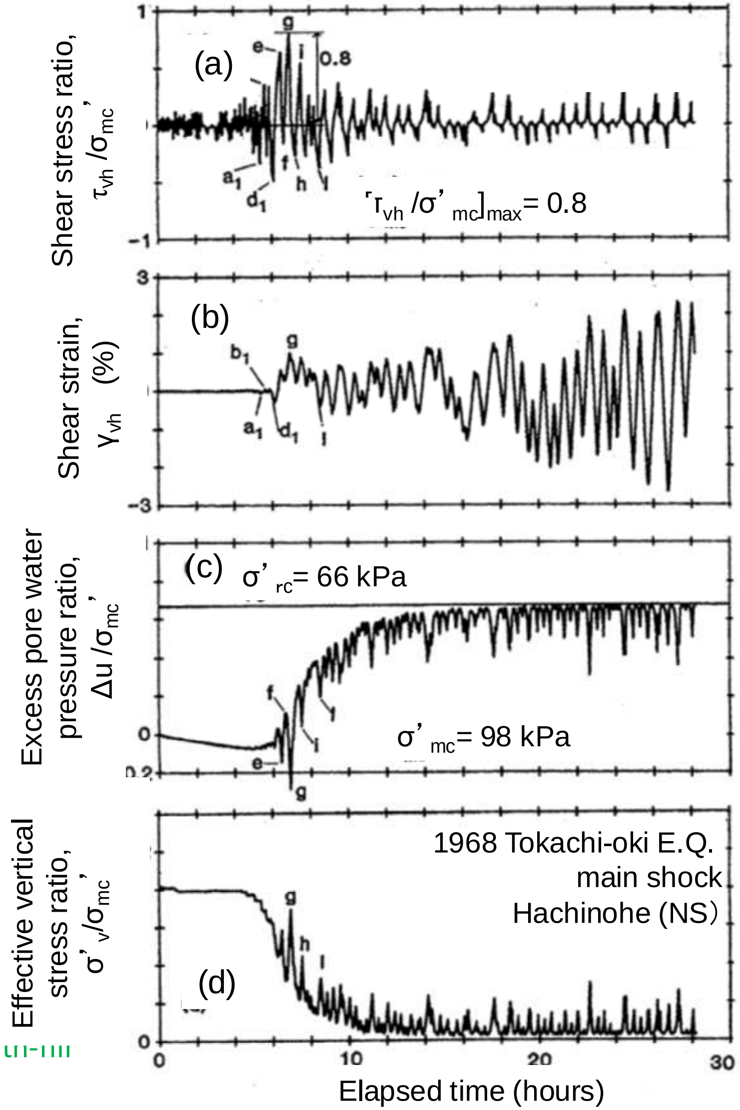


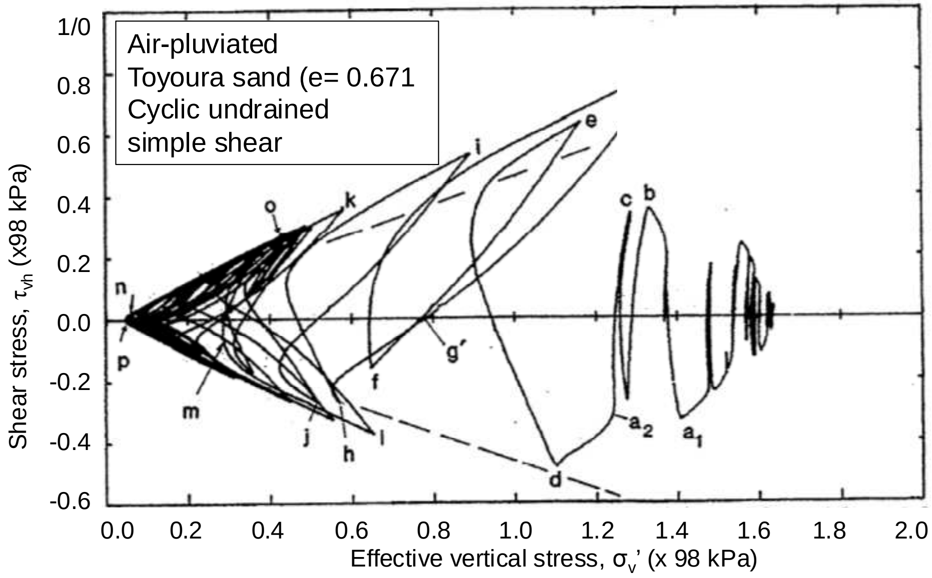
Undrained stress- strain behaviour of saturated soil

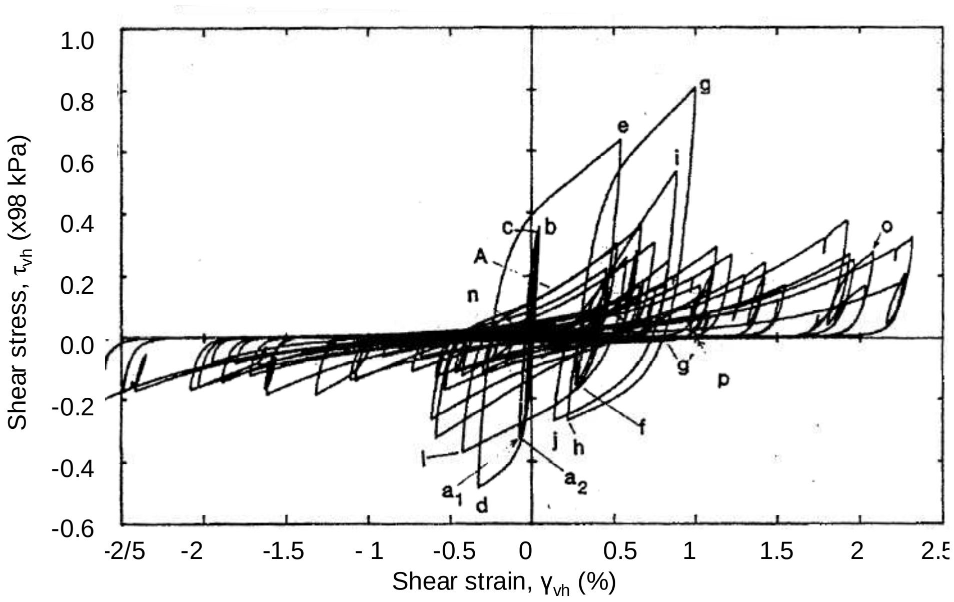
- Undrained stress – strain relation in the course of cyclic undrained loading modelled for residual deformation analysis by pseudo-static non-linear FEM - 1 :

Cyclic undrained torsional simple shear test on **dense** Toyoura sand applying ‘seismic’ random stresses at a constant shear strain rate

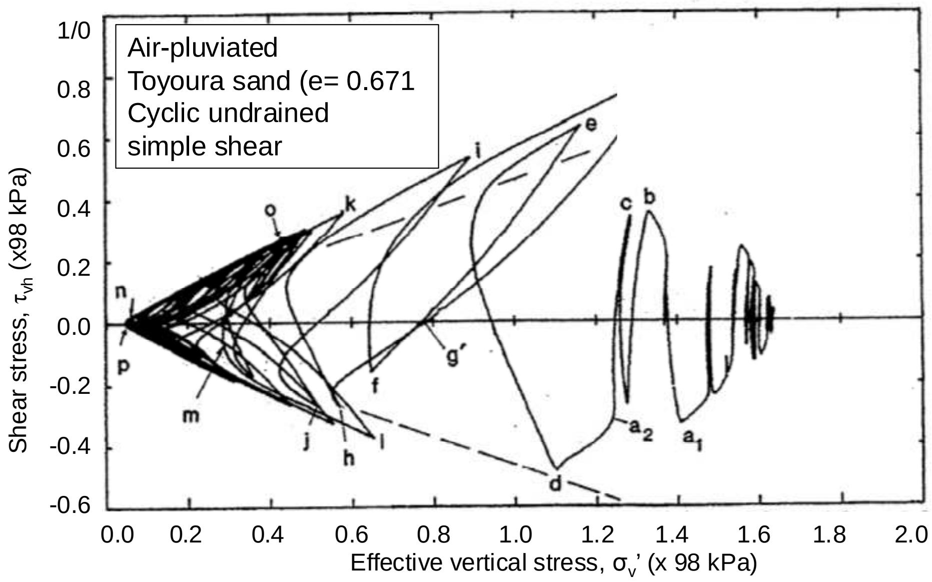


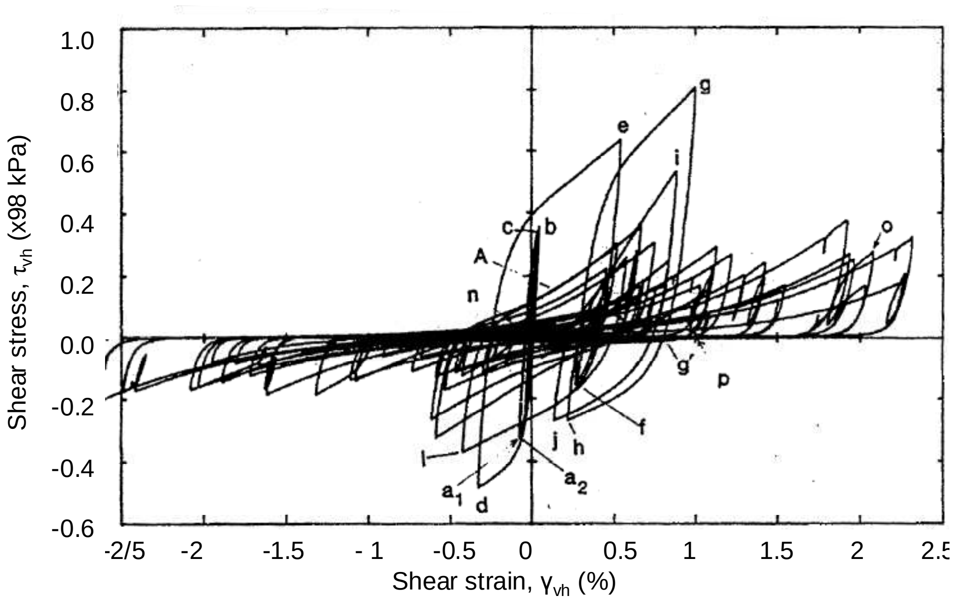


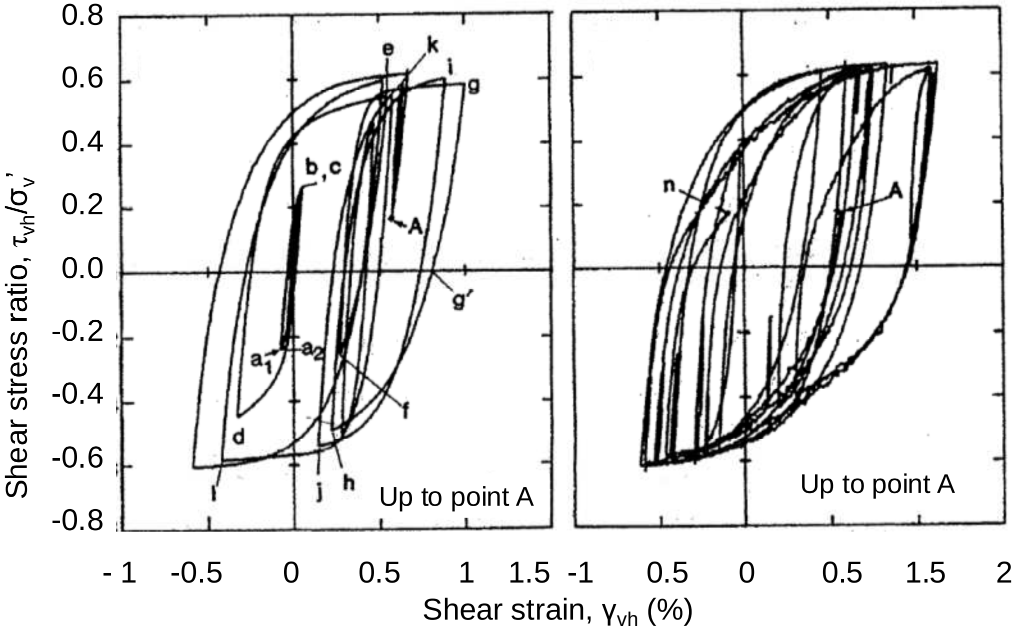




Complicated $\tau_{vh} - \gamma_{vh}$ relation !







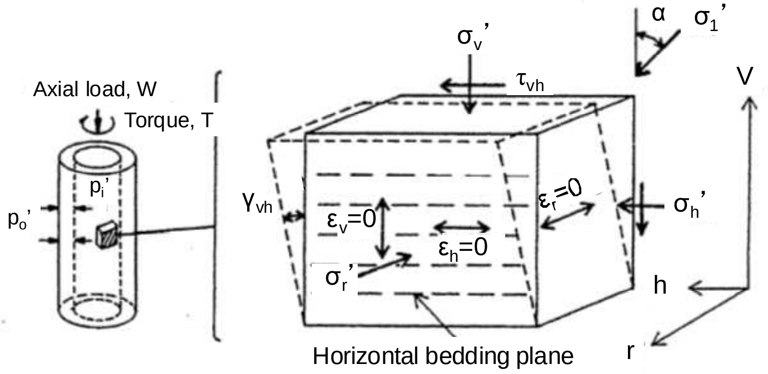
Complicated $\tau_{vh} – \gamma_{vh}$ relation, but smooth strain-hardening hysteretic $\tau_{vh}/\sigma'_v – \gamma_{vh}$ relation:

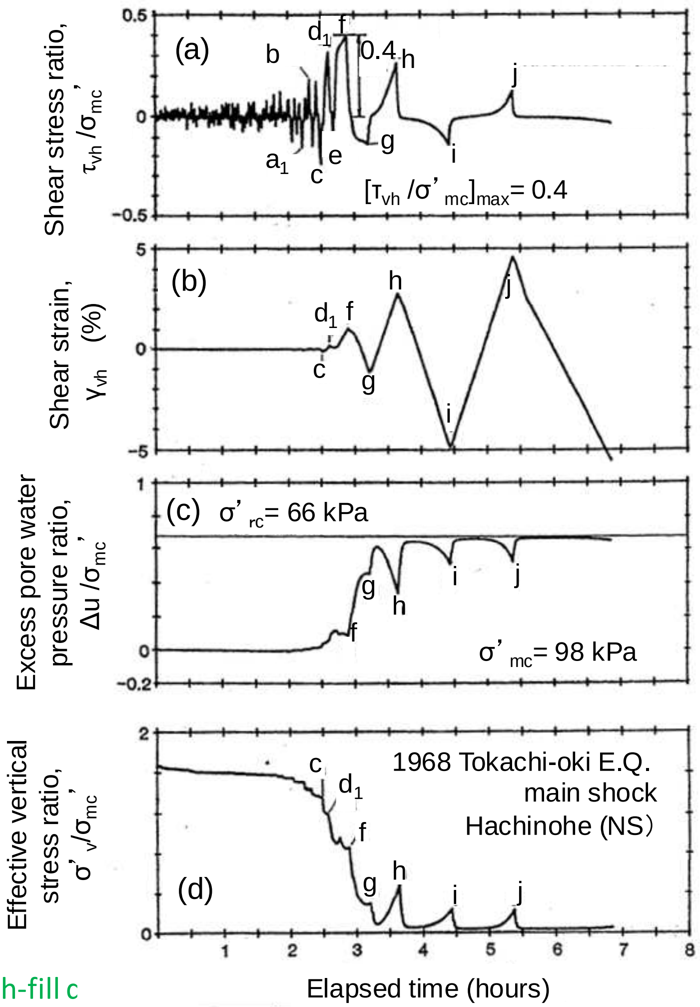
- Yielding starts when τ_{vh}/σ'_v exceeds the previous maximum value in each direction.

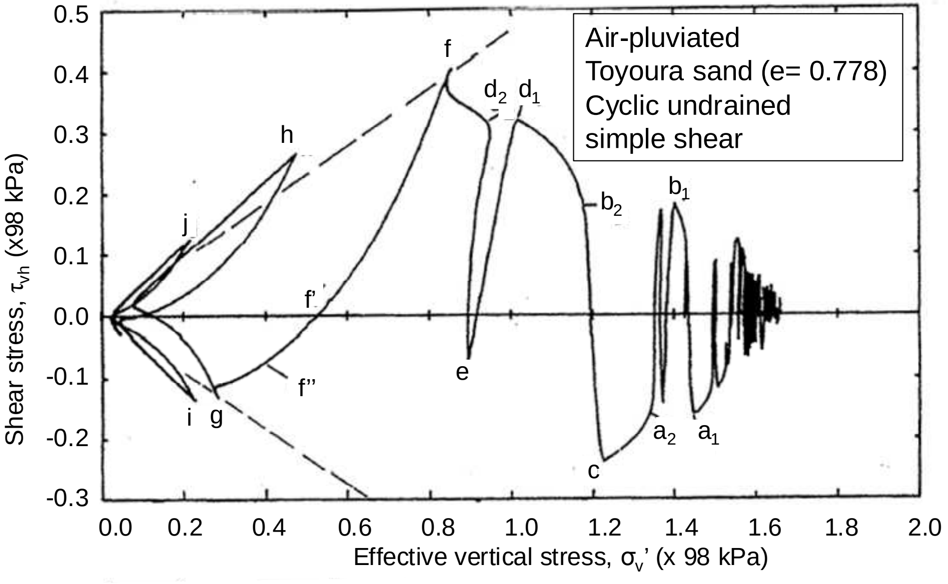
Undrained stress- strain behaviour of saturated soil

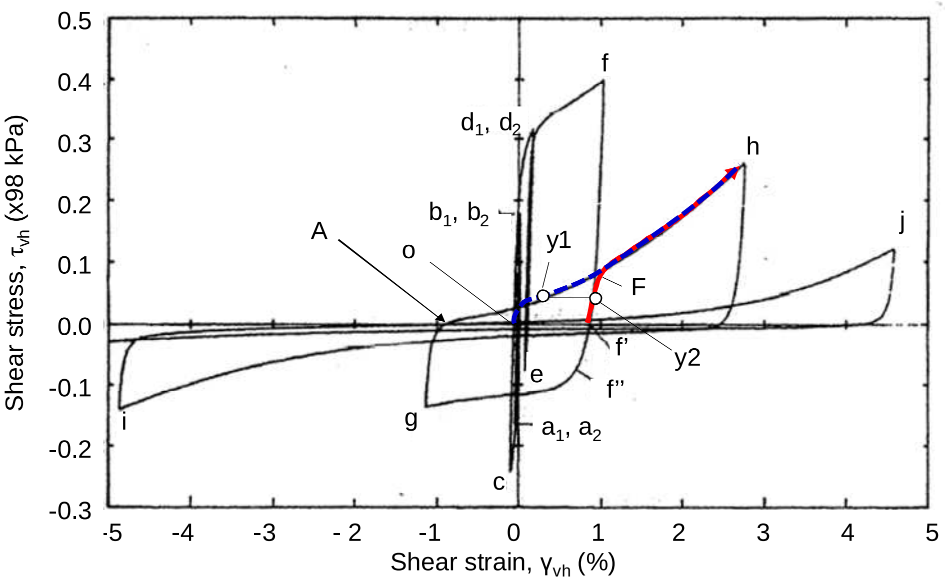
- Undrained stress – strain relation in the course of cyclic undrained loading modelled for residual deformation analysis by pseudo-static non-linear FEM - 1 :

Cyclic undrained torsional simple shear test on **loose** Toyoura sand applying ‘seismic’ random stresses at a constant shear strain rate

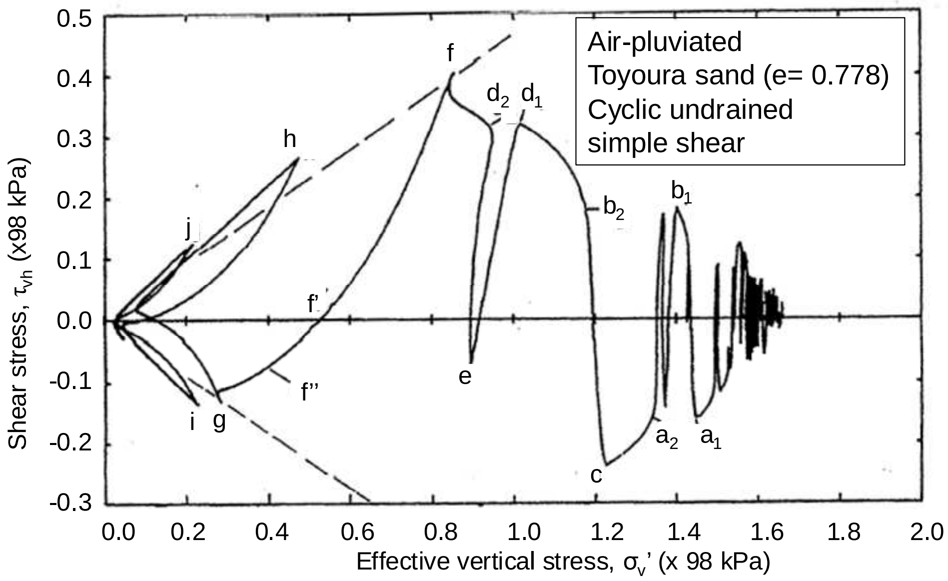


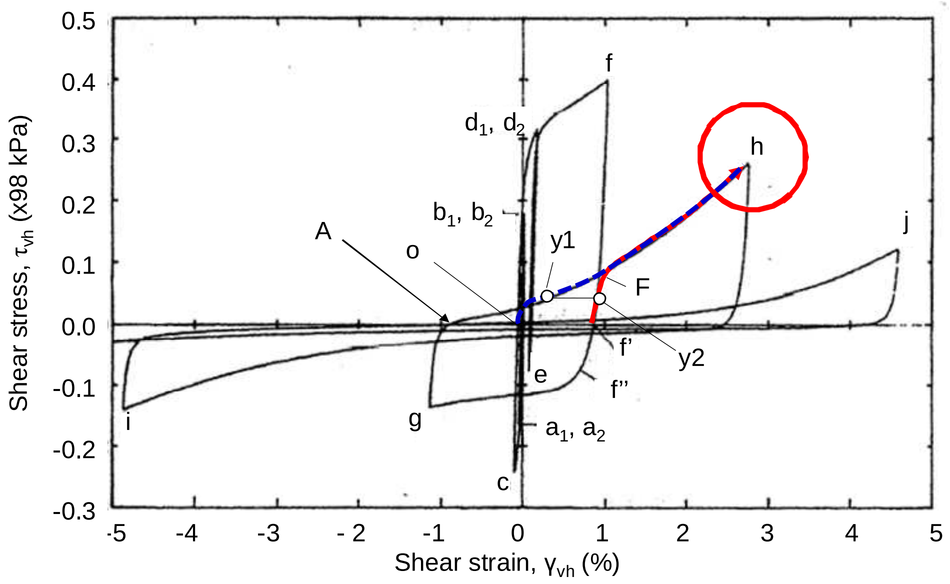






Complicated $\tau_{vh} - \gamma_{vh}$ relation !

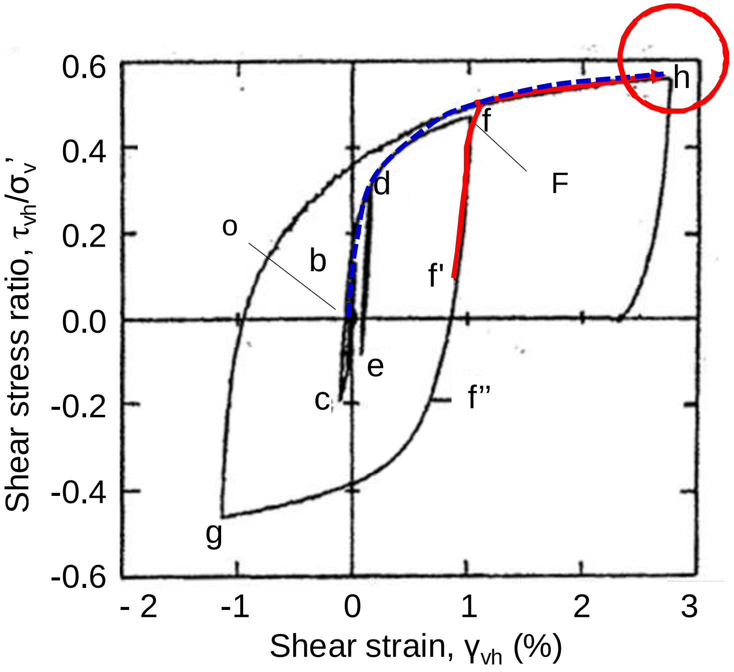


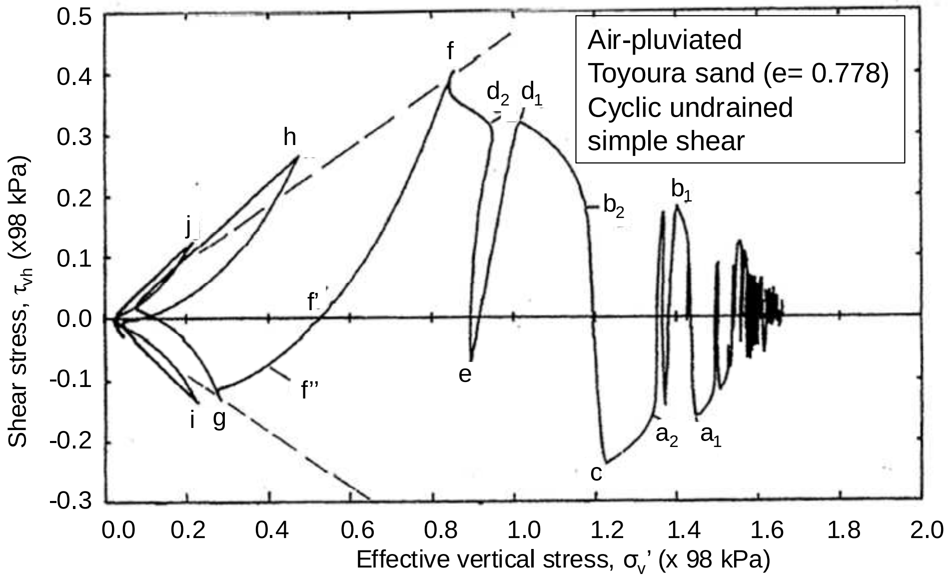


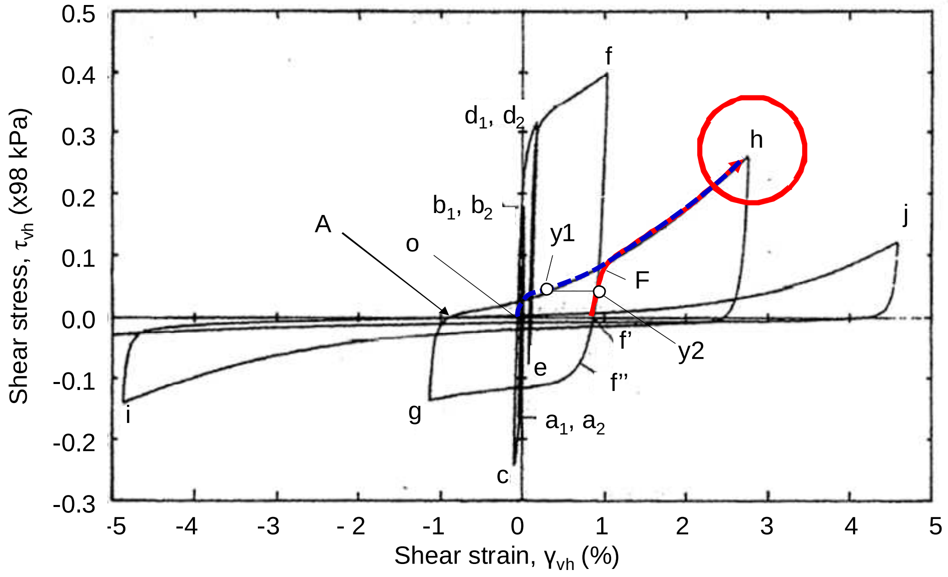
Complicated $\tau_{vh} - \gamma_{vh}$ relation, but smooth strain-hardening hysteretic $\tau_{vh}/\sigma'_v - \gamma_{vh}$ relation:

- Yielding starts when τ_{vh}/σ'_v exceeds the previous maximum value in each direction.

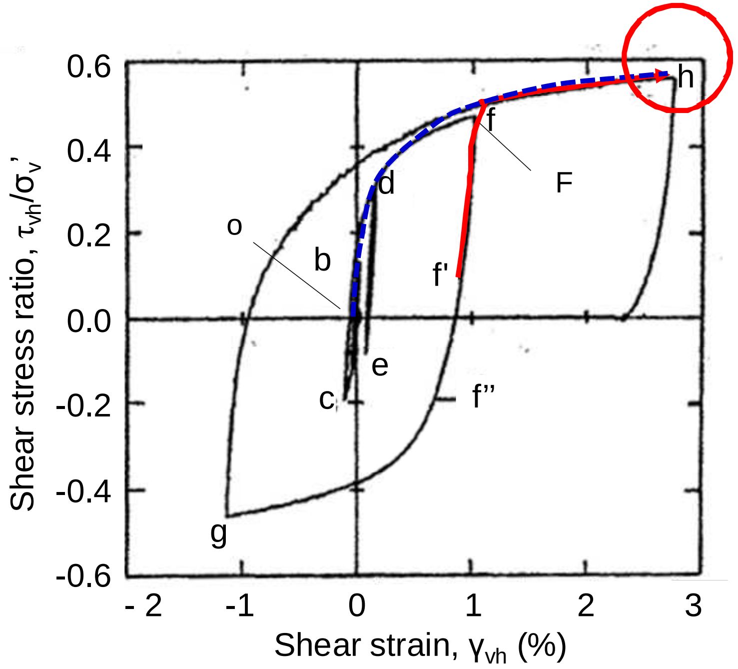
- The γ_{vh} value at the peak τ_{vh}/σ'_v state after having passed the yielding point (e.g. point h) can be determined only by the peak τ_{vh}/σ'_v value and the ML stress – strain relation starting from the origin (i.e., o→y1→F→h), not referring to previous cyclic loading histories.

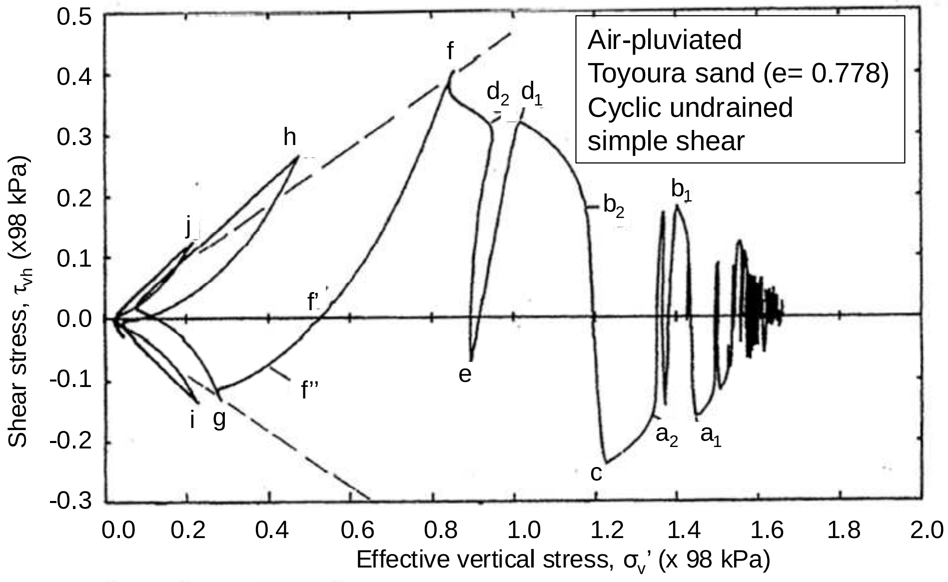


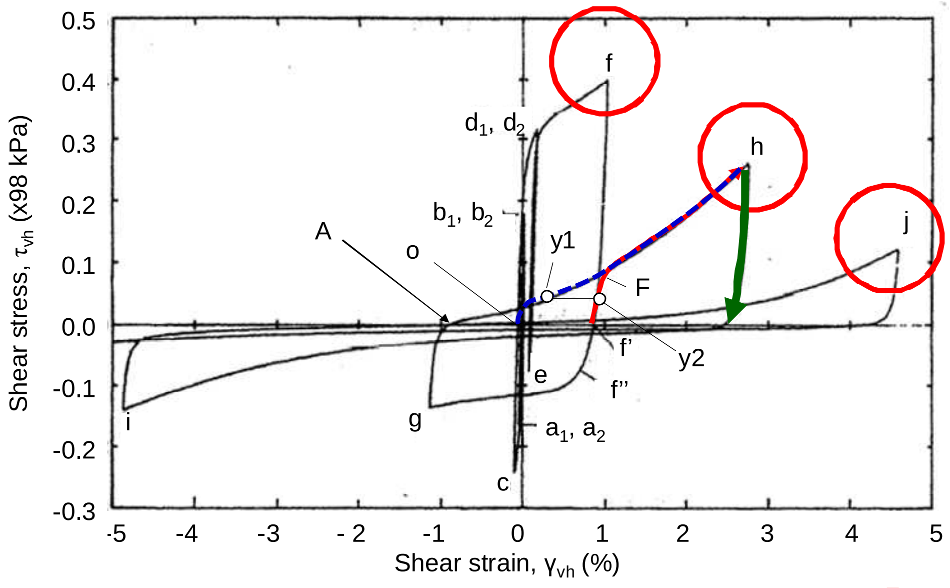




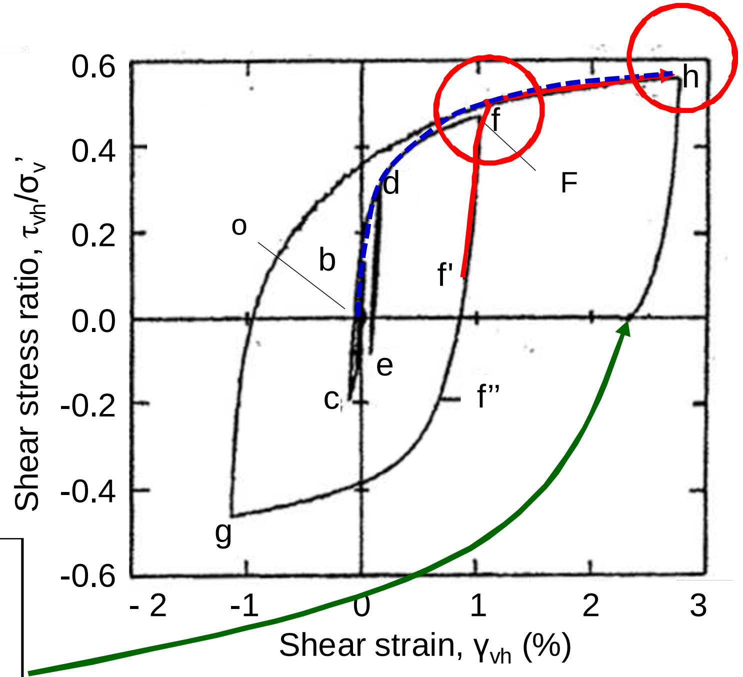
The γ_{vh} value at the peak τ_{vh}/σ'_v state after having passed the yielding point (e.g. point h) obtained by following the reloading $\tau_{vh} - \gamma_{vh}$ relation (e.g. f'→y2→F→h) is the same as the value obtained by following the monotonic loading $\tau_{vh} - \gamma_{vh}$ relation starting from the origin (e.g. o→y1→F→h) while not referring to previous cyclic loading histories.



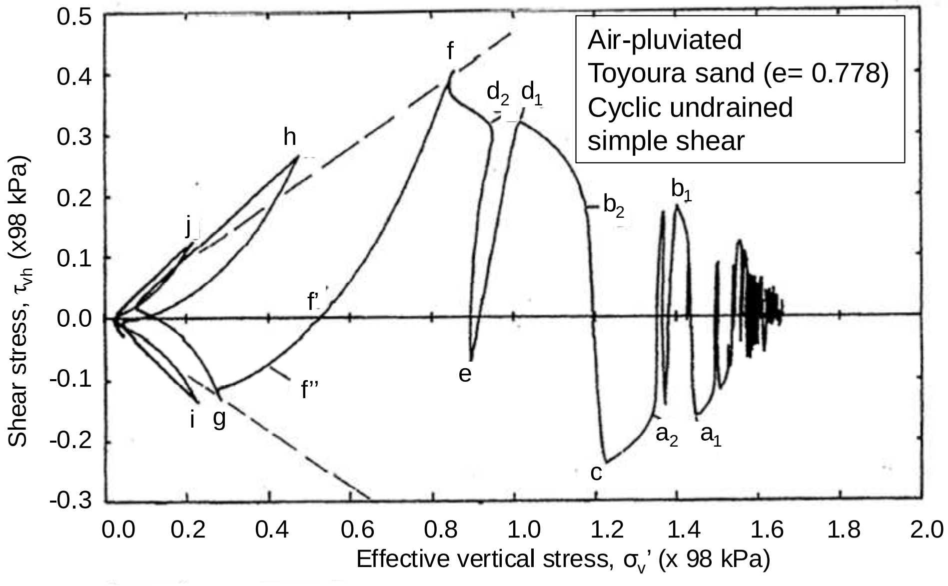


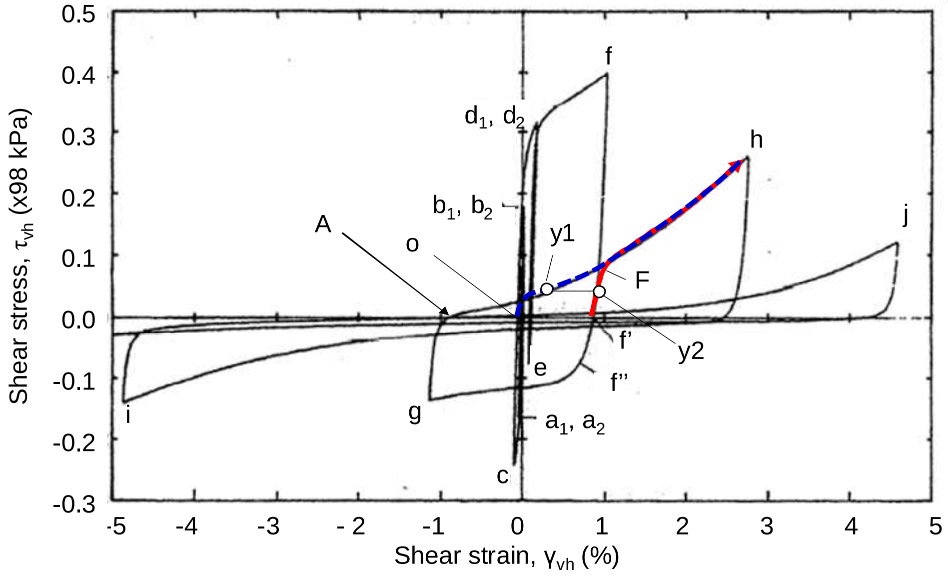


The time history of the γ_{vh} value at the peak τ_{vh}/σ'_v states after having passed the yielding point (e.g. the values at points f, h & j) can be obtained by following respective ML stress-strain relations starting from the origin, o, that have degraded by respective preceding cyclic undrained loading histories.

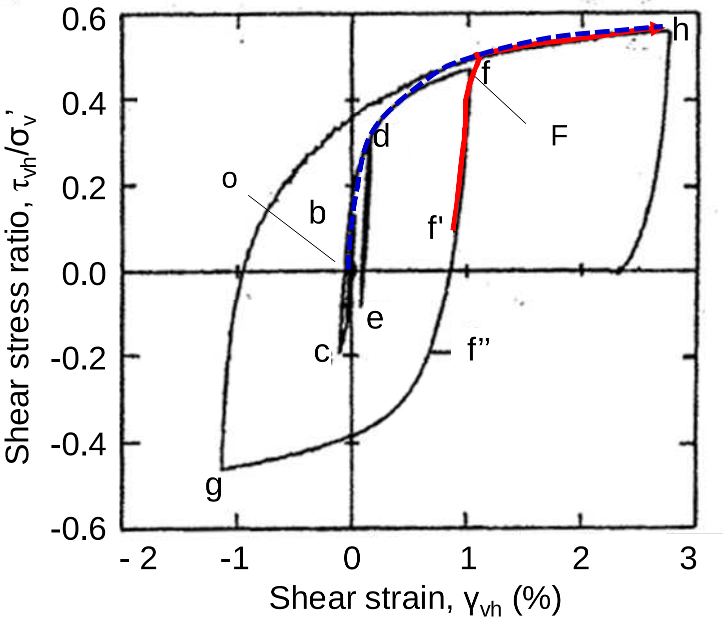


→ In a slope in which initial shear stresses are acting, most of the respective peak γ_{vh} value remains as the residual value upon unloading.



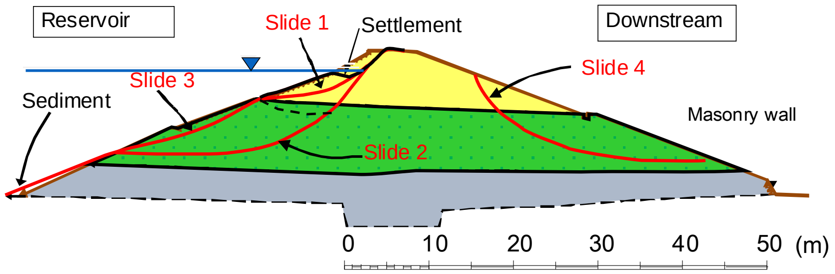


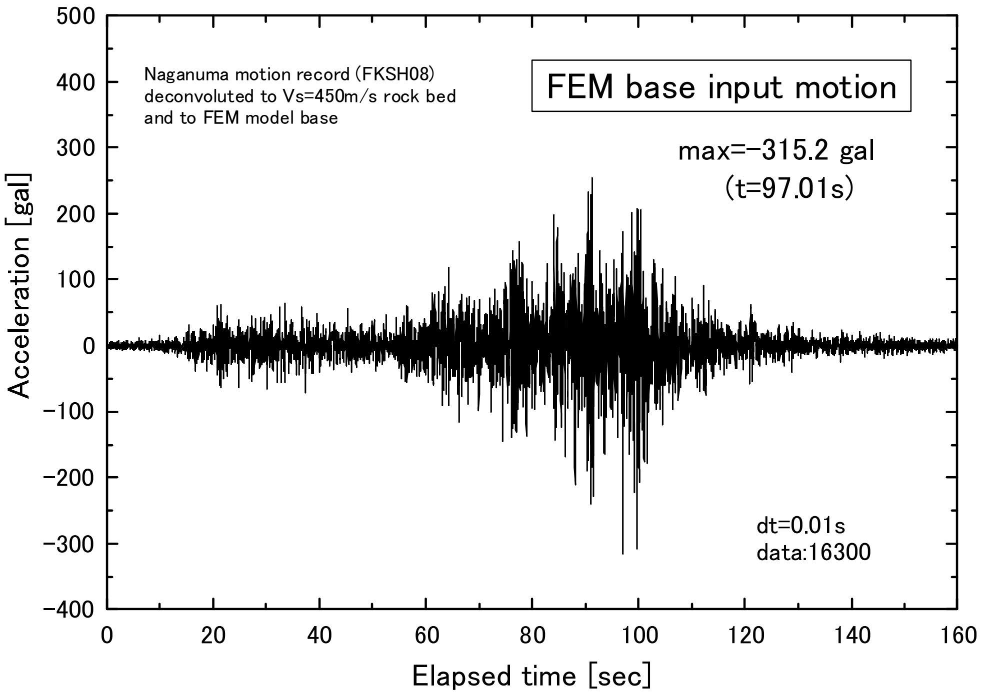
The time history of the residual deformation of a slope may be obtained by a series of pseudo-static non-linear FEM analyses incorporating gravity and seismic loads while using respective ML stress – strain relations starting from the origin.



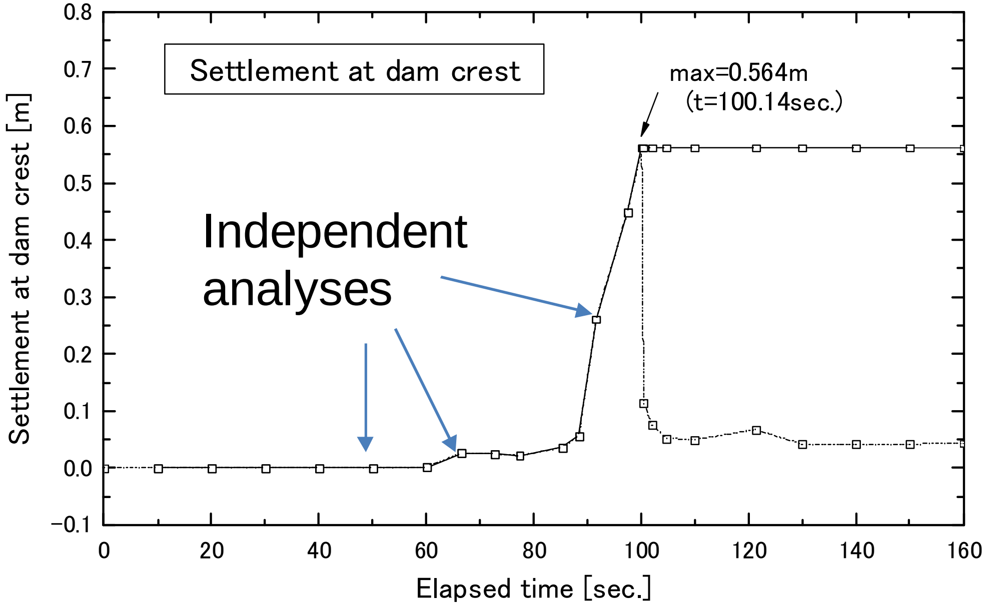
Approximately, the maximum value of this deformation can be considered as the ultimate residual deformation caused by a given seismic loading history.

A series of pseudo-static FEM analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake

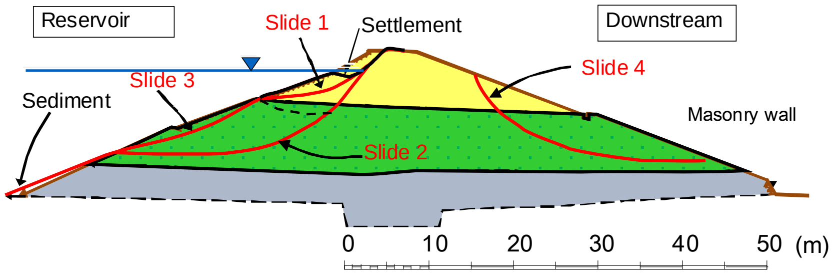


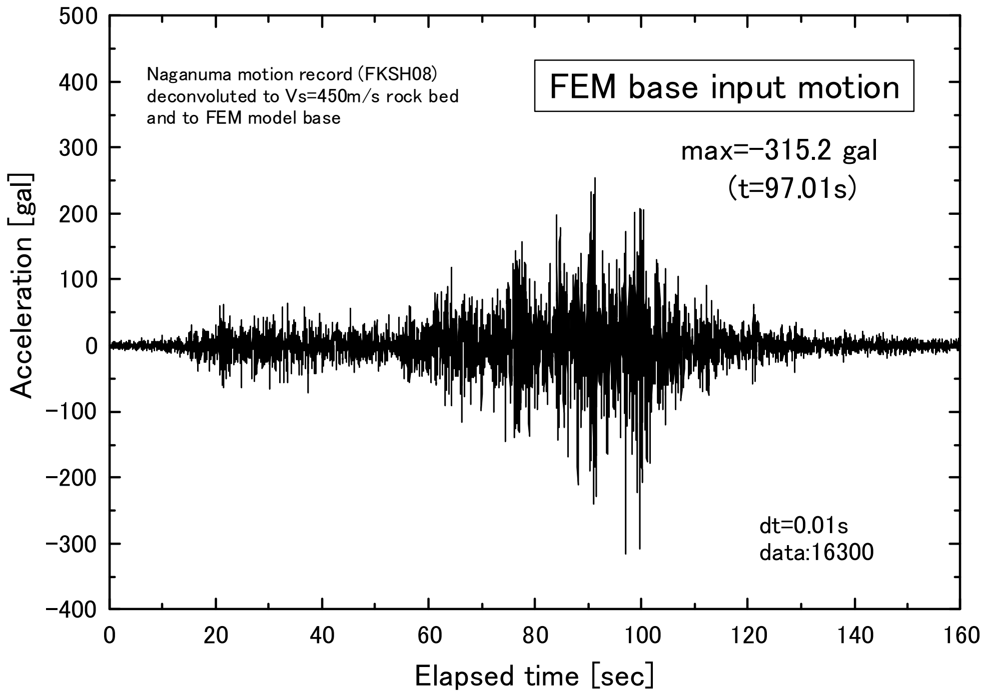


The largest deformation (at t= 100.14 s):
-not by the peak acceleration (t= 97.01 s), but later after the stress-strain relation has deteriorated more.



A series of pseudo-static FEM analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake

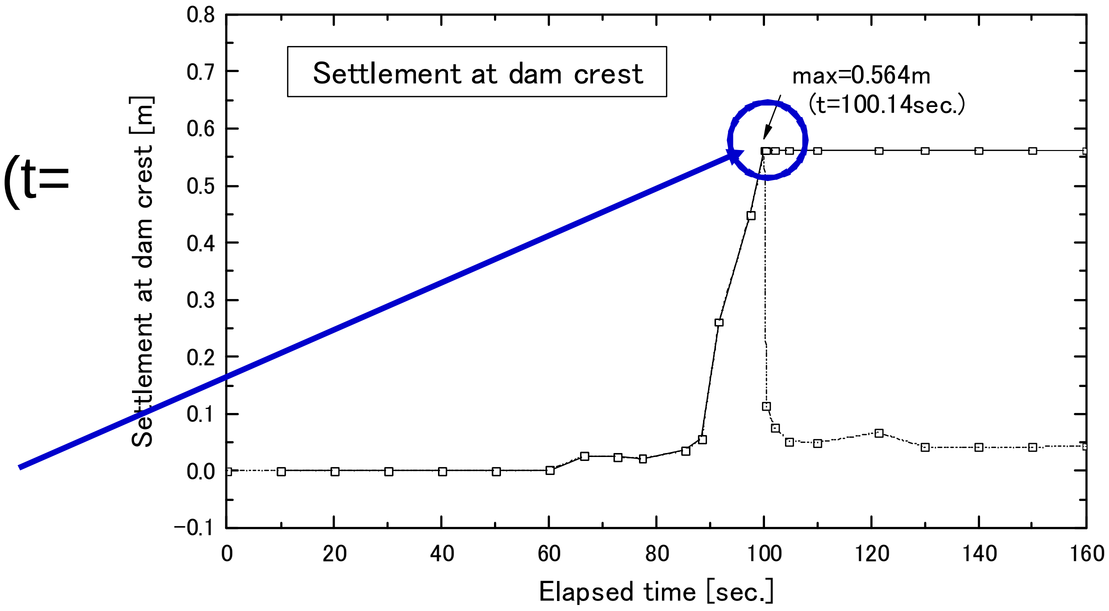




The largest deformation (at t= 100.14 s):

-not by the peak acceleration (t= 97.01 s), but later after the stress-strain relation has deteriorated more.

-This largest deformation is considered as the ultimate residual deformation.



Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

●Practical simplified seismic stability analysis needs appropriate balance among the methods chosen in the following items:

1) Criterion to evaluate of the stability:
   Global safety factor relative to a specified required minimum vs.
   Residual deformation relative to a specified allowable largest.

2) Design seismic load at a given site:
   Conventional design load vs. Likely largest load in the future

3) Stress – strain properties of soil:
   Actual complicated behavior vs. Simplified model

4) Relevant consideration of the effects of other engineering factors:
 - compacted dry density; soil type; etc.

Design seismic load at a given site:

Conventional design load:
- specified in many old seismic design codes
- defined as Level 1 design seismic load in new seismic design codes (introduced after the 1995 Great Kobe E.-Q.)

Likely largest seismic load during the lifetime of a given structure:
- defined as Level 2 design seismic motion in new seismic design codes (introduced after the 1995 Great Kobe E.-Q.)

Japanese Society for Civil Engineers (1996) :

•Level 1 design seismic motion: It is a seismic motion with a high likelihood of occurring during the design lifetime of the concerned structure. It is required that, in principle, all new structures have sufficient seismic resistance to ensure "no damage" when subjected to this seismic motion.

•Level 2 design seismic motion: It is the strongest seismic motion thought likely to occur at the location of the concerned structure during its lifetime. It is required that the structure should not collapse, although damage that renders it unusable is acceptable if its functionality can be rapidly restored.

The relationship among “the design seismic load”, “design shear strength of soil” and “stability analysis method (i.e., global Fs vs. residual deformation)” is complicated due to historical reasons.

Actual behavior during severe earthquakes

■ Well-compacted fill A:

Examples:

high rock fill dams

modern highway embankments

modern railway embankments

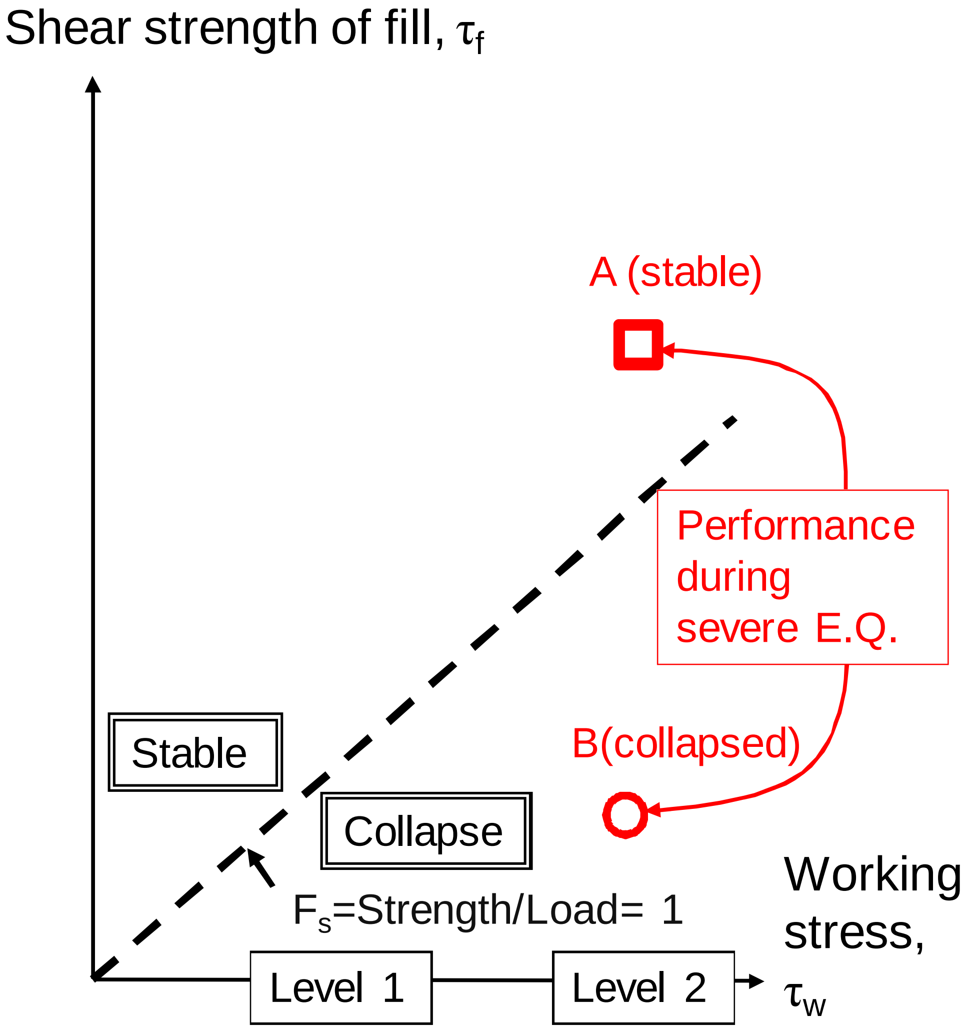
earth-fill dams

■ Poorly-compacted fill B:

Examples:

old soil structures before introduction of modern design and construction codes and methods

residential embankments



Shear strength of fill, τ_f

A (stable)

Performance during severe E.Q.

B(collapsed)

Stable

Collapse

F_s=Strength/Load= 1

Level 1

Level 2

Working stress, τ_w

Typical conventional seismic design of soil structure

Design seismic load $(\tau_w)_d$: k_h= 0.15 (Level 1 seismic coefficient)

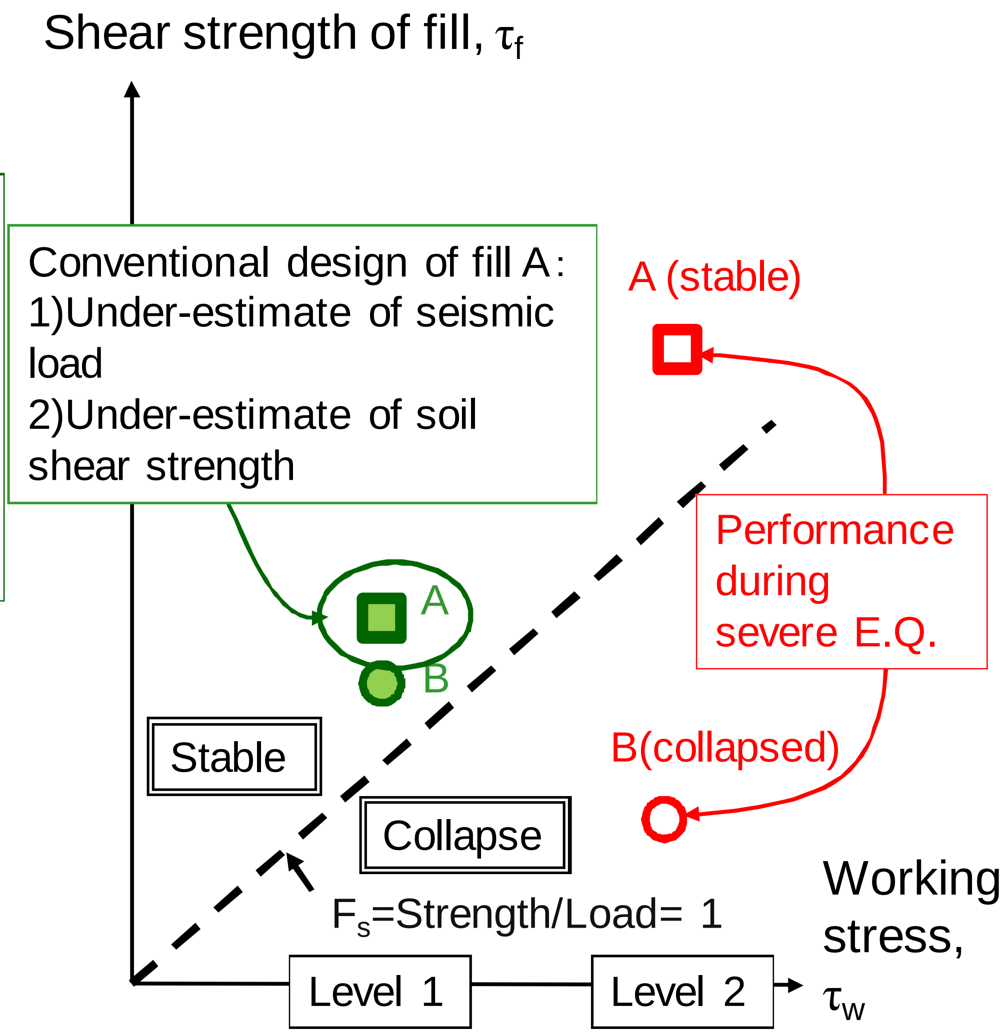
Design shear strength $(\tau_f)_d$*: Drained shear strength when D_c by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90 %)

→ Required min. F_s by limit equilibrium stability analysis = 1.2, for example

■ Well-compacted fill A:

The use of k_h=0.15 as Level 2 seismic load is on the unsafe side. However, the use of $(\tau_f)_d$* as the drained/ undrained strength of well-compacted fill is on the safe side.

→ These two factors may be balanced.



Conventional design of fill A:
1)Under-estimate of seismic load
2)Under-estimate of soil shear strength

Performance during severe E.Q.

Typical conventional seismic design of soil structure

Design seismic load $(\tau_w)_d$: k_h= 0.15 (Level 1 seismic coefficient)

Design shear strength $(\tau_f)_d$*: Drained shear strength when D_c by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90 %)

→ Required min. F_s by limit equilibrium stability analysis = 1.2, for example

■ Well-compacted fill A:

The use of k_h=0.15 as Level 2 seismic load is on the unsafe side. However, the use of $(\tau_f)_d$* as the drained/ undrained strength of well-compacted fill is on the safe side.

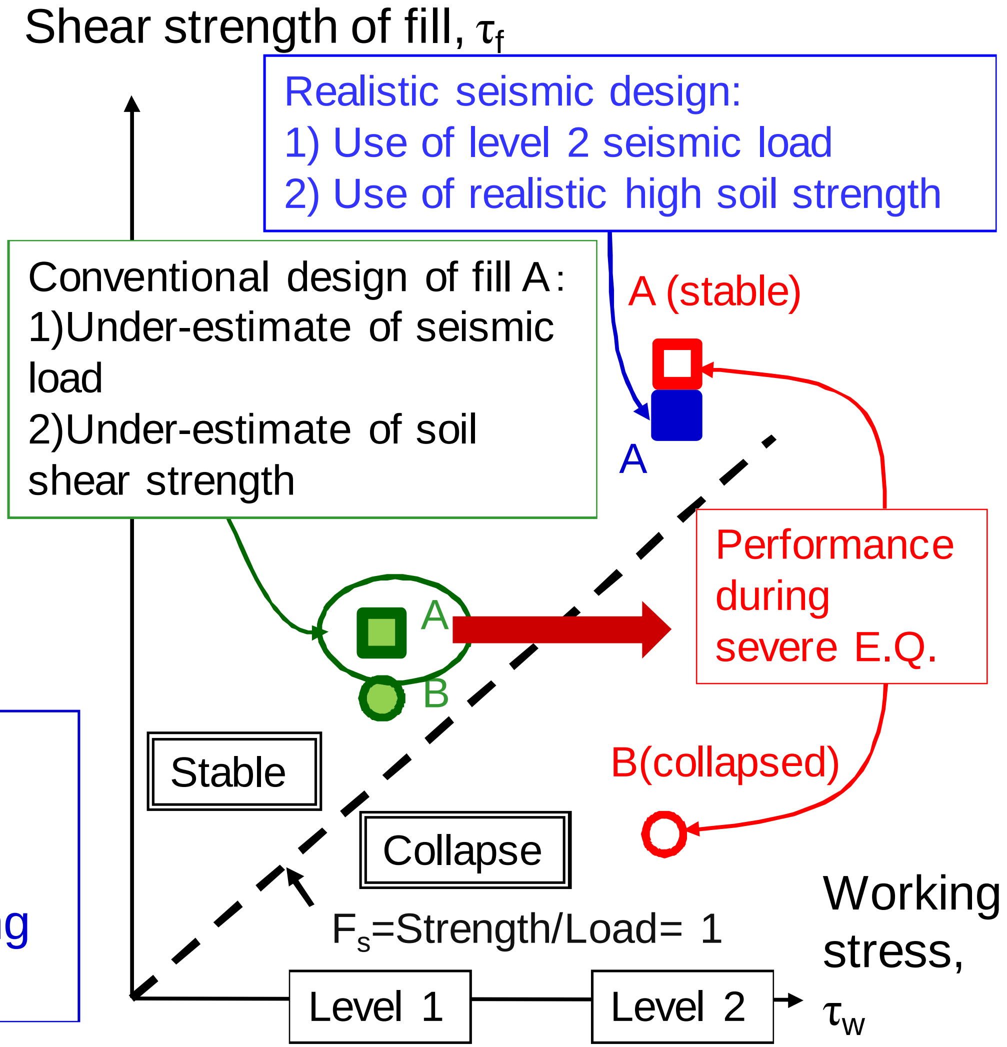
→ These two factors may be balanced.

If only k_h is increased: i.e., if only $(\tau_w)_d$ is increased⇒ Under-estimate of stability by losing balance (i.e., collapse despite actual stable performance against level) 2

Solution:

1) use of level 2 design seismic load; and

2) use of realistic high soil strength $(\tau_f)_d$ corresponding to actual D_c (> 90 %) (using undrained strength when relevant)



Realistic seismic design:

1) Use of level 2 seismic load

2) Use of realistic high soil strength

Conventional design of fill A:

1)Under-estimate of seismic load

2)Under-estimate of soil shear strength

Performance during severe E.Q.

Typical conventional seismic design of soil structure

Design seismic load $(\tau_w)_d$: k_h= 0.15 (Level 1 seismic coefficient)

Design shear strength $(\tau_f)_d$*: Drained shear strength when D_c by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90 %)

→ Required min. F_s by limit equilibrium stability analysis = 1.2, for example

■ Poorly-compacted fill B:

The use of k_h=0.15 as Level 2 seismic load is on the unsafe side.

Besides, the use of $(\tau_f)_d$* as "undrained strength of saturated poorly-compacted fill subjected to seismic load" is on the unsafe side.

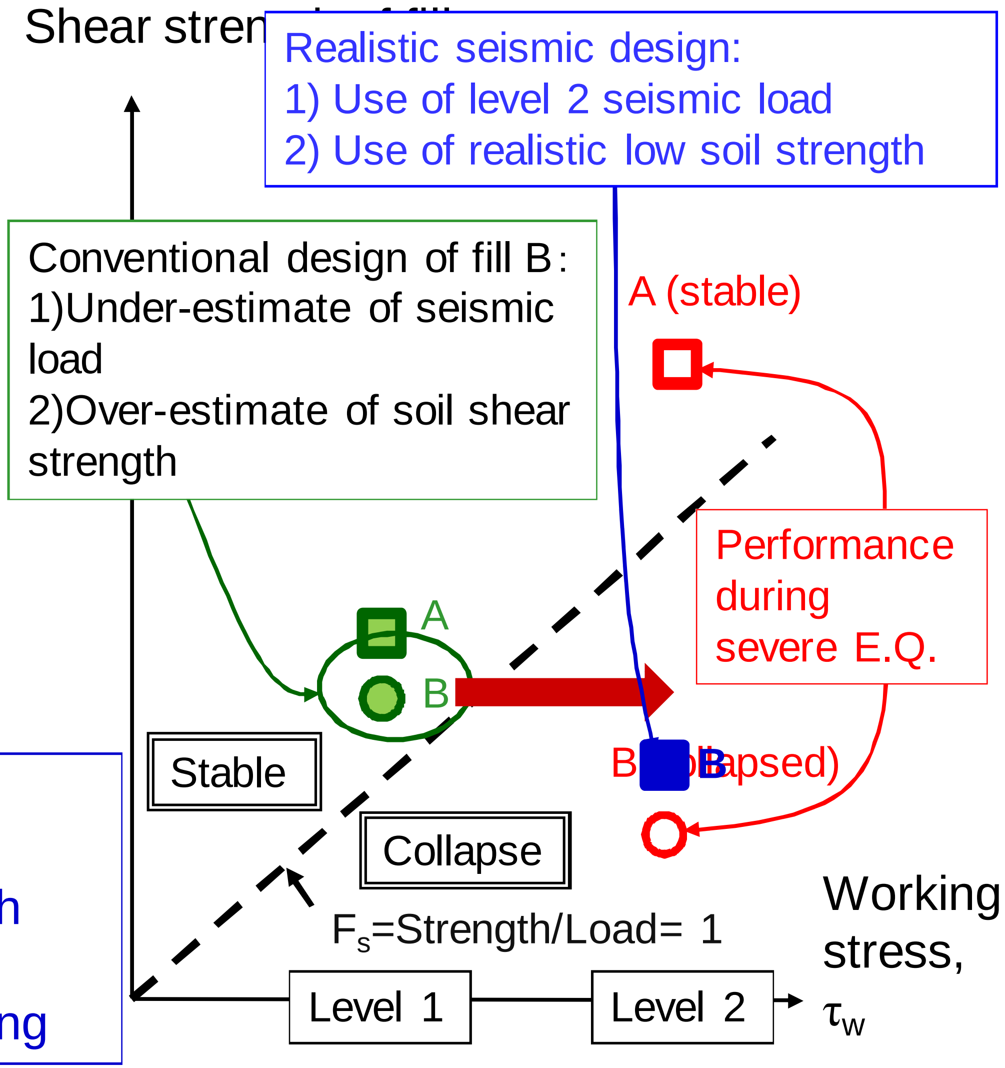
→ These two factors are not balanced.

Only an increase in k_h is not sufficient, but use of realistic low $(\tau_w)_d$ is necessary to duly evaluate the stability against level 2 seismic load.

Solution:

1) use of level 2 design seismic load and;

2) use of realistic soil strength that is much lower than the conventional value $(\tau_f)_d$* if saturated/undrained during seismic loading



Realistic seismic design:

1) Use of level 2 seismic load

2) Use of realistic low soil strength

Conventional design of fill B:

1)Under-estimate of seismic load

2)Over-estimate of soil shear strength

Performance during severe E.Q.

CONCLUDING REMARKS - 1

Several important factors that influence the drained &saturated-undrained stress-strain properties of soil when subjected to monotonic & cyclic loading histories related to the seismic stability analysis of soil structures, including earth-fill dams, were discussed. The following are the main conclusions:

1) Compacted soils exhibit significant strain-softening associated with shear banding, resulting in progressive failure in a slope.

2) As the thickness of shear band increases with D_{50}, the rate of strain-softening becomes slower with D_{50} ,so the failure tends to become less progressive, making the slope more stable.

3) Although the effect of dry density on the drained peak shear strength is large, the effect on the undrained shear strength is much more significant.

CONCLUDING REMARKS - 2

4) The drained strength of compacted soil exhibits strong inherent anisotropy in plane strain compression.

5) The strength obtained by different stress-strain tests (i.e., triaxial and plane strain compression and direct shear) could be largely different due to the effects of different angles between the σ_1 direction and the bedding plane direction; different ratios of σ_2 to σ_1 & σ_3; and different definition of friction angle.

CONCLUDING REMARKS - 3

6) For the limit equilibrium-based stability analysis of a slope under plane strain conditions, a practical simplified method that assumes the followings, yet takes into account the effects of compacted dry density and particle size, can be proposed:

a) Isotropic stress-strain properties exhibiting strain-softening of which the rate decreases with an increase in D_{50}.

b) Use of the peak & residual strengths determined by the conventional TC tests at δ= 90°,.

c) Use of the peak strength corresponding to somehow conservatively determined compacted dry density (i.e., slightly lower than the value that corresponds to the anticipated average of the actual values of D_c).

d) Progressive failure is not taken into account.

CONCLUDING REMARKS - 4

7) Under undrained monotonic loading conditions, loose & dense saturated soils exhibit the shear strength that is significantly lower and higher than the respective drained shear strengths.

8) The undrained shear strength decreases by preceding cyclic undrained loading. The effects of dry density on the damaged undrained shear strength are significant due to the following trends with an increase in the dry density:

a) the increase in the initial undrained shear strength;

b) the decrease in the damage strain by preceding cyclic undrained loading; and

c) the decrease in the degradation rate by damage strain.

CONCLUDING REMARKS - 5

9) For simplified stability analysesof a slope having saturated zones by “slip deformation by the Newmark method” ad “residual deformation by the pseudo-static non-linear FEM”, the characteristic feature of undrained stress – strain properties described above can be modelled in a unified framework from the same results of a set of monotonic and cyclic loading lundrained stress – strain tests of saturated soil..

THANK YOU FOR YOUR ATTENTIONS