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CHARACTERIZATION OF GRAVELLY GEOMATERIALS

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LECTURE OUTLINE

- *INTRODUCTORY REMARKS*

- *STRESS-STRAIN BEHAVIOUR:*

  PHENOMENOLOGICAL FRAMEWORK

- *IN SITU TESTS* } *SELECTED TOPICS*

- *LABORATORY TESTS*

- *CLOSING REMARKS*

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INTRODUCTORY REMARKS

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TENTATIVE CLASSIFICATION OF "CLEAN" GRAVELLY GEOMATERIALS

SOIL	GC* %	SC* %	FC* %	
CLEAN SAND	< 10	≥ 85	< 5	
GRAVELLY SAND	< 50	≥ 50	< 5	For low plasticity, fines matrix FC < 15%
SANDY GRAVEL	≥ 50	< 50	< 5	
CLEAN GRAVEL	≥ 85	< 10	< 5	

● Drained response under monotonic and very low frequency cyclic loading ●

● Excess pore pressure generated under higher frequency cyclic loading ●

(*) content of gravel (GC), sand (SC) and fines (FC) respectively
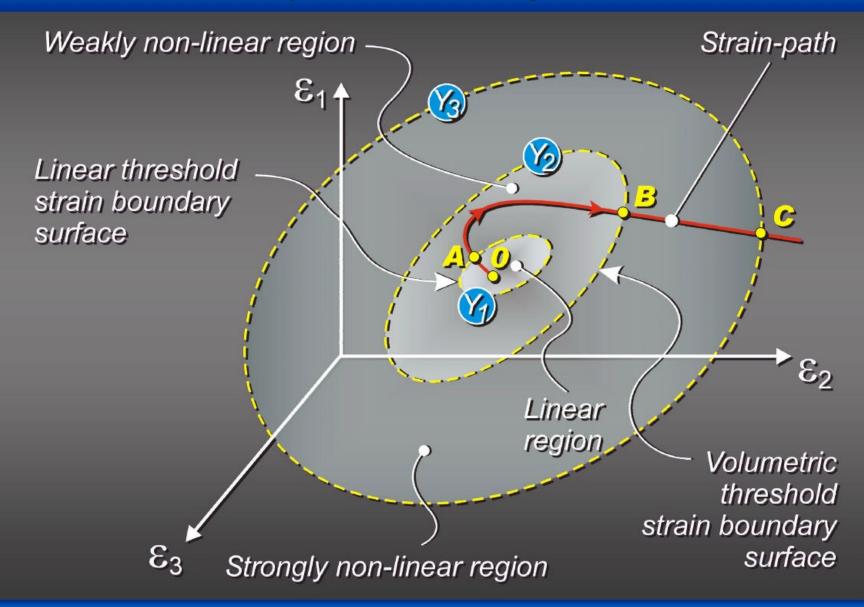
GRAVELLY DEPOSIT OVERLAYING SOFT ROCK

SANDY GRAVEL

GRAVELLY SAND

WEAKLY CEMENTED SAND

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GRAVELLY GEOMATERIALS ENGINEERING RELEVANCE

- *Important transportation infrastructures, growing demand for optimized design of direct foundations.*

  e.g.: Akashi Kaikyo (q = 1.0 MPa), Messina Straits (q = 1.5 Mpa) suspension bridges.

- *Construction materials for embankment dams, highways, high speed railways.*

  Must be checked against vibratory loadings originated by earthquakes wind, and traffic.

- *Susceptibility of natural deposits to liquefaction and cyclic mobility.*

  Many examples of gravelly deposits liquefaction, e.g.: Andrus (1994), Ishihara (1996), Andrus and Stokoe (2000)

COARSE GRAINED SOILS
STRESS-STRAIN BEHAVIOUR
A PHENOMENOLOGICAL FRAMEWORK

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THRESHOLD BOUNDARY SURFACES IN PRINCIPAL STRAIN SPACE (QUALITATIVE)

Jamiolkowski and Lai (2000)

Weakly non-linear region

Strain-path

ε_1

Y3

Y2

Linear threshold strain boundary surface

B

C

A • 0

Y1

ε_2

Linear region

Volumetric threshold strain boundary surface

ε_3

Strongly non-linear region
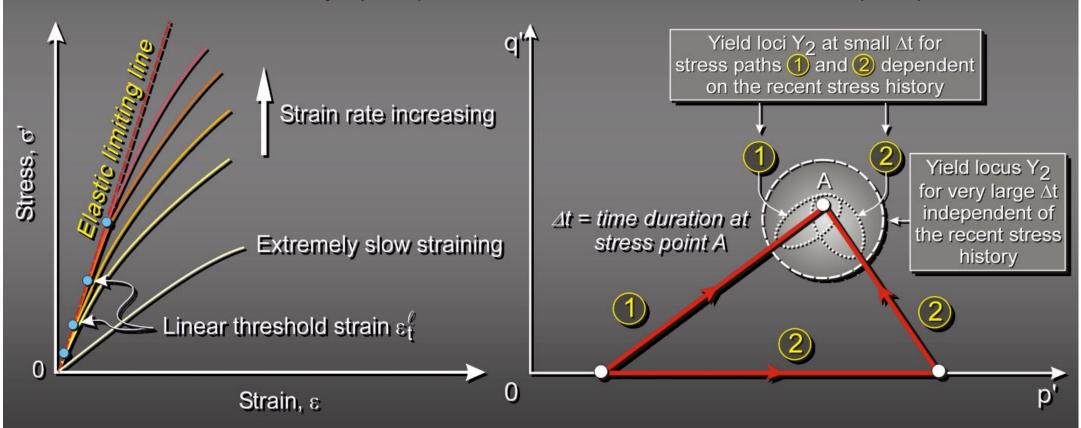
VISCOUS NATURE OF THRESHOLD STRAINS

Influence of strain rate on ε_t^ℓ

Tatsuoka and Shibuya (1991)

Elastic limiting line

Strain rate increasing

Extremely slow straining

Linear threshold strain ε_t^ℓ

Stress, σ'

Strain, ε

Influence of creep on yield locus Y_2

Tatsuoka et al. (1997)

Yield loci Y_2 at small Δt for stress paths ① and ② dependent on the recent stress history

Yield locus Y_2 for very large Δt independent of the recent stress history

Δt = time duration at stress point A

q'

p'
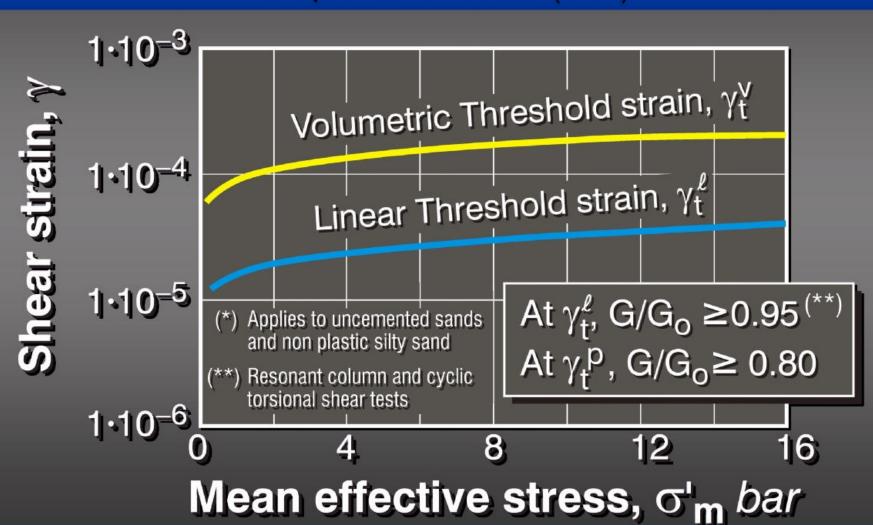
INFLUENCE OF MEAN EFFECTIVE STRESS ON THRESHOLD STRAINS OF NC SANDS*
Adapted from Derendeli (2001)
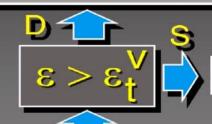
Volumetric Threshold strain, γ_t^v

Linear Threshold strain, γ_t^ℓ

(*) Applies to uncemented sands and non plastic silty sand

(**) Resonant column and cyclic torsional shear tests

At γ_t^ℓ, $G/G_o \geq 0.95$ (**)

At γ_t^p, $G/G_o \geq 0.80$

Shear strain, γ

Mean effective stress, σ'_m *bar*
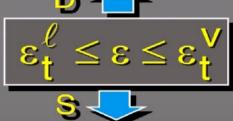
AFTER DEPOSITIONAL REDUCTION OF POROSITY

Palmer and Barton (1987), Barton et al. (1993), Cresswell (1999)
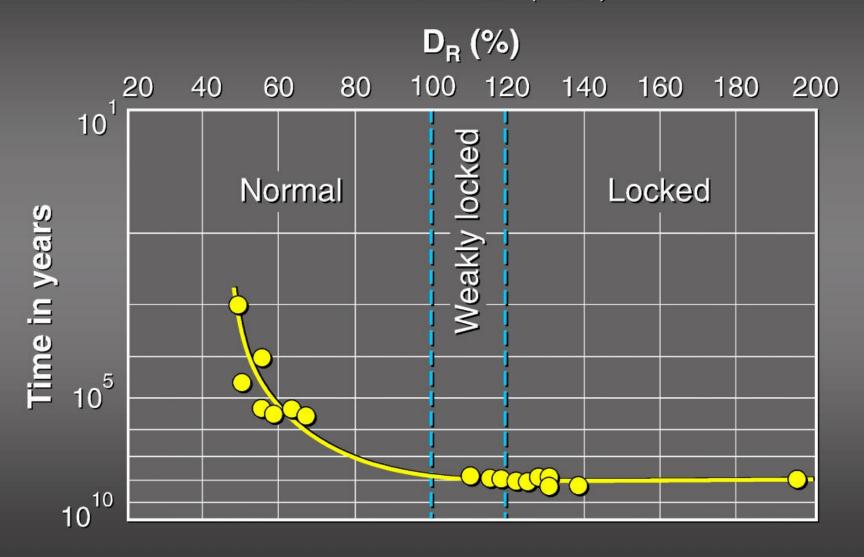
MECHANICAL* :   *reorientation and repacking*

CHEMICAL* :   *intergranular pressure dissolution, contact area and roughness increase*

CEMENTATION :   *precipitation of minerals in pore space*

(*) Occurrence of locked sands and gravels, characterized by D_R > 100% Desseault and Morgenstern (1979), Palmer and Barton (1987).
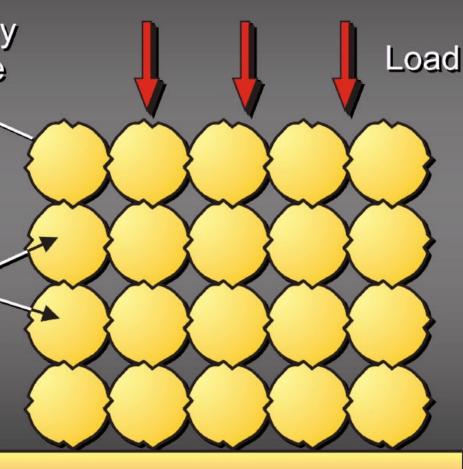
TWO-DIMENSIONAL MODEL OF LOCKED SAND
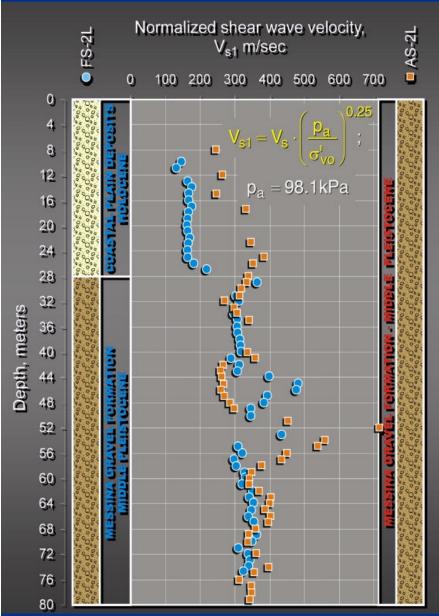
Cresswell (1999)
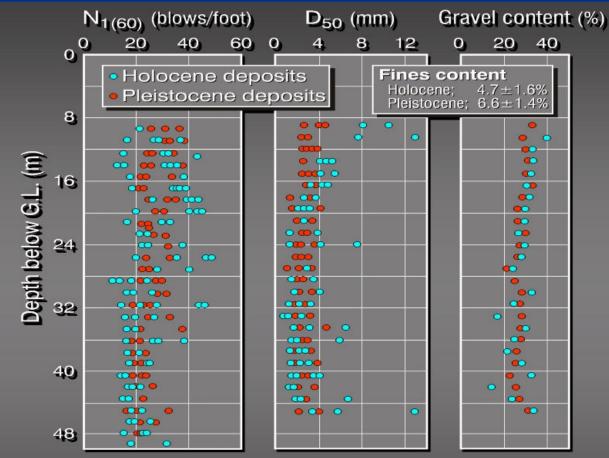
Grain can be easily removed by gentle brushing

Load

Grain held in by interlocking asperities
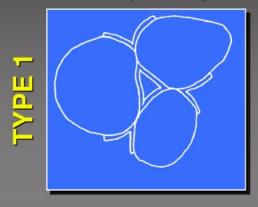
GRAVELLY DEPOSITS ON SICILIAN SHORE

Silica sand and gravel of Holocene (≅10000y) and Pleistocene (>500000y) age:

- Similar grading and mineralogical composition
- Deposited in similar environment; deltaic and shallow sea
- Comparable penetration resistance
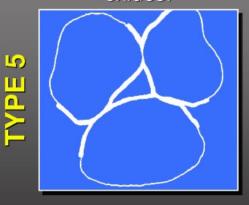- Quite different shear wave velocity

MESSINA STRAIT CROSSING
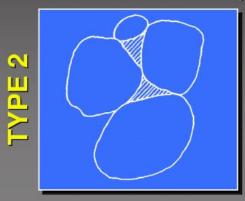# TYPES OF CEMENTATION IN MESSINA GRAVELS FORMATION
(Adapted after Bosi, 1990)

TYPE 1 — $CaCO_3$ coatings.

TYPE 2 — Voids filled with $CaCO_3$.

TYPE 3 — Sand matrix cemented with $CaCO_3$.
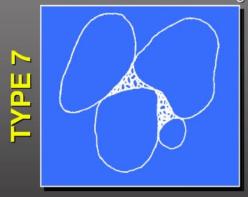
TYPE 5 — Coatings of Fe and Mn oxides.
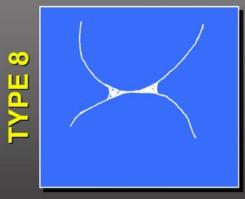
TYPE 7 — Silty clay matrix lightly cemented with $CaCO_3$.

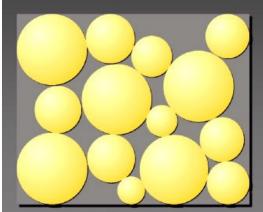**TYPE 8** — Silty clay matrix lightly cemented with Fe and Mn oxides.

Type 4, intermediate between 3rd and 5th;   Type 6, intermediate between 5th and 7th.
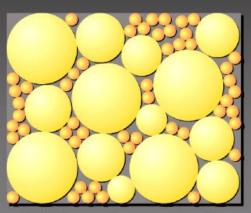
MULTIPLE-SIZED COARSE GRAINED GEOMATERIAL
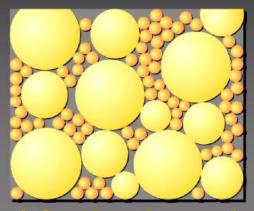
Lin (1986), Kuerbis et al. (1988), Thevanayagam (1998,1999a, 1999b), Thevanayagam and Liang (2001)
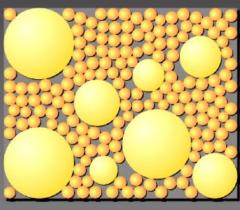
a) Gravel, SC<10%	b) Sandy gravel, $SC<SC_t$	c) Sand and gravel, $SC_t<SC<SC_\ell$	d) Gravelly sand, $SC>SC_\ell$

SC=*Sand fraction;* **GC**=*Gravel fraction;* SC_t=*Threshold sand fraction;* SC_ℓ =*Limit sand fraction*

- For mixes **a** and **d** the global void ratio e reflects the internal interparticle contacts

- For mixes **b** and **c** it is necessary to refer to the equivalent sand fraction e_s and gravel fraction e_g void ratio respectively, which control the number of interparticle contacts.

Mechanical response of coarse grained geomaterials primarily affected by contacts density per grain

GRAVELLY GEOMATERIALS
IN SITU TESTING

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PENETRATION TESTING
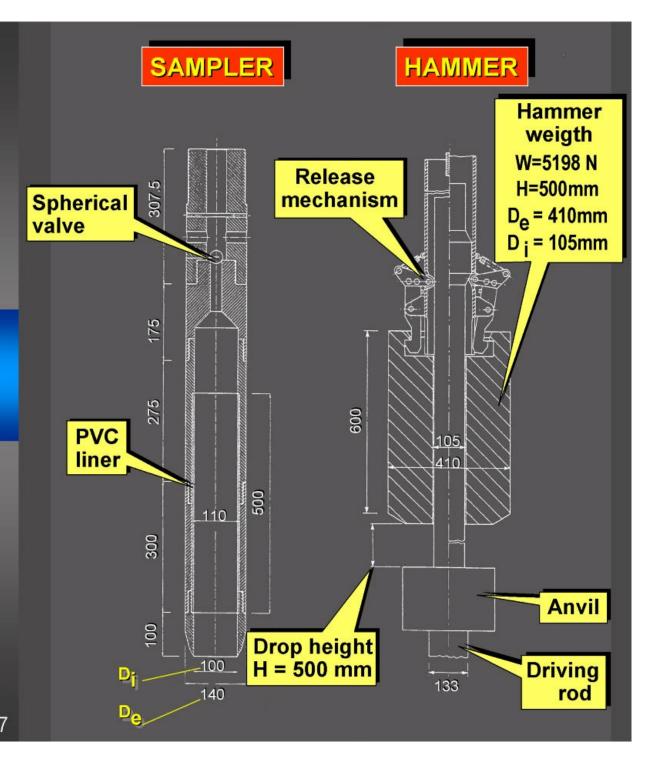
- Selection of penetration methods controlled first by size of gravelly particles, then by in situ soil density

- Every-day practice mainly limited to SPT and LPT

- N_{SPT} and N_{LPT} blow-count normalization with respect to energy transmitted to driving rods and to effective overburden stress:

$$(N_1)_{60} = C_N \cdot \frac{ER}{60} \cdot N; \quad N = N_{SPT} \text{ or } N_{LPT}$$

- How particle size affects N_{SPT} vs N_{LPT} results still to be clarified*, Daniel (1999), Daniel et al. (2004)

- In special circumstances, CPT, S-CPT, DMT, S-DMT, can be attempted

(*) Relevant when employing engineering correlation for soil properties.

LARGE PENETRATION TEST USED IN ITALY
Crova et al. (1992)

SAMPLER   HAMMER

Spherical valve

Release mechanism

Hammer weigth
W=5198 N
H=500mm
D_e = 410mm
D_i = 105mm

PVC liner

Drop height
H = 500 mm

Anvil

Driving rod

307.5
175
275
300
100
110
500
100
D_i
140
D_e

600
105
410
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N_{SPT} VERSUS N_{LPT} IN GRANULAR DEPOSITS

Location	Deposits	ER_{SPT} (%)	ER_{LPT} (%)	$\dfrac{N_{SPT}}{N_{LPT}}$	$\dfrac{[N_{1(60)}]_{SPT}}{[N_{1(60)}]_{LPT}}$	D_{50} (mm)
San Prospero*	Po River sand	56	84	1.35 ± 0.43	0.91 ± 0.27	0.2 to 0.6
Messina strait	Holocene sand and gravel	65	85	1.12 ± 0.58	0.85 ± 0.47	0.5 to 21.3
	Pleistocene sand and gravel	60	85	1.45 ± 0.32	1.02 ± 0.25	0.2 to 44

(*) Site at which LPT has been calibrated against SPT.

RELATIVE DENSITY OF GRAVELLY SOILS FROM SPT AND LPT

Cubrinowski and Ishihara (1999)

$$\frac{(N_1)_{78}}{D_R^2} = C_D = \frac{9}{(e_{max} - e_{min})^{1.7}}$$
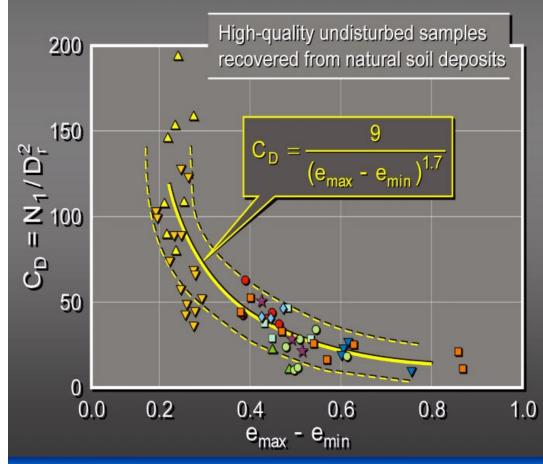
- *Checked against laboratory tests on undisturbed samples*

- *Lack of specific ER measurements*

- *Postulated that the Japanese $N_{LPT} = N_{SPT}$ with ER = 78%*

- *e_{max} and e_{min} assessed by Kokusho and Tadashi (1997) method for gravelly soils*

Further validation of Cubrinowski and Ishihara (1999) approach involving ER measurements strongly recommended.

RELATIVE DENSITY OF COARSE GRAINED SOILS FROM SPT AND LPT
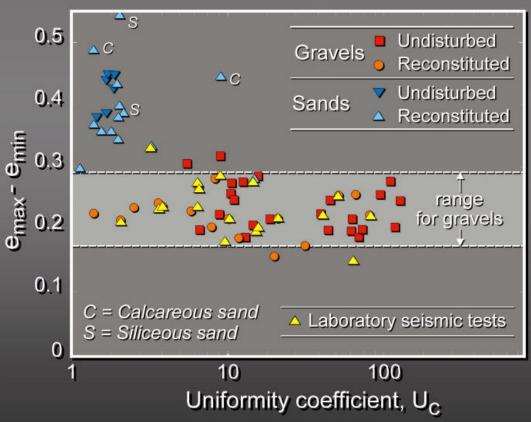
Relationship between N_1/D^2_R and void ratio range

Cubrinowski and Ishihara (1999)

High-quality undisturbed samples recovered from natural soil deposits

$$C_D = \frac{9}{(e_{max} - e_{min})^{1.7}}$$

$C_D = N_1/D_r^2$ (y-axis)

$e_{max} - e_{min}$ (x-axis)
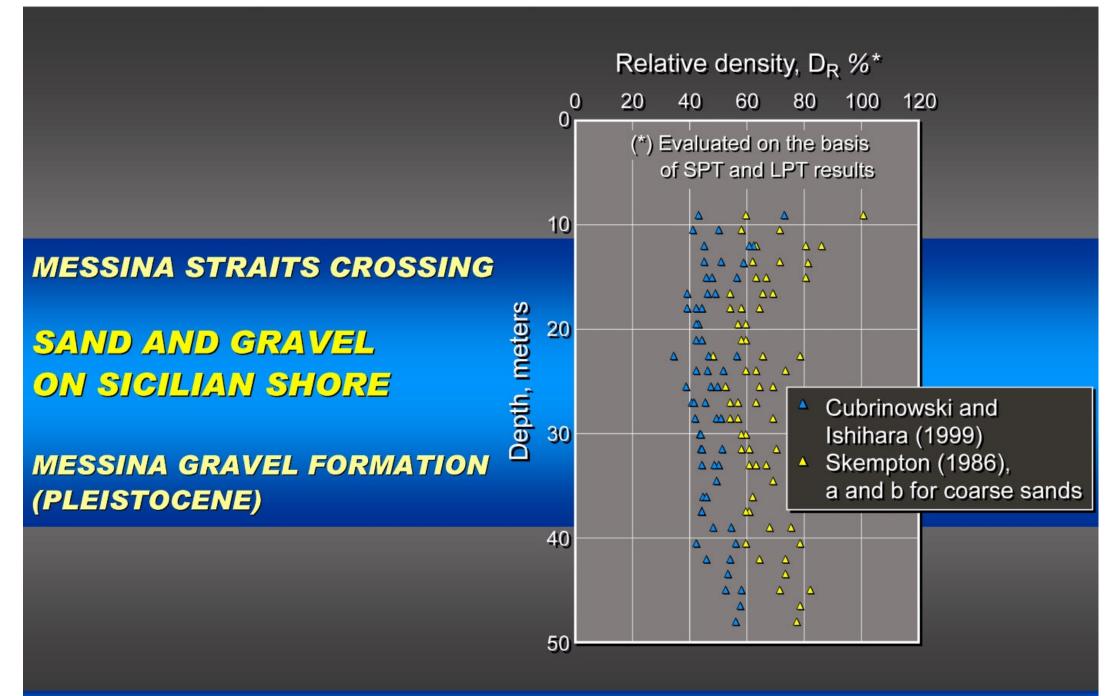
Maximum and minimum void ratio of coarse grained soils

Adapted after Cubrinowski and Ishihara (1999)

$e_{max} - e_{min}$ (y-axis)

Uniformity coefficient, U_C (x-axis)

Gravels	Undisturbed
	Reconstituted
Sands	Undisturbed
	Reconstituted

range for gravels

C = Calcareous sand
S = Siliceous sand

Laboratory seismic tests

MESSINA STRAITS CROSSING

SAND AND GRAVEL
ON SICILIAN SHORE

MESSINA GRAVEL FORMATION
(PLEISTOCENE)

Relative density, D_R %*

(*) Evaluated on the basis of SPT and LPT results

Depth, meters

△ Cubrinowski and Ishihara (1999)
△ Skempton (1986), a and b for coarse sands

GEOPHYSICAL METHODS

- **Crucial in hard–to–sample geomaterials like gravelly soils**

- **Continuously growing relevance, applied to broad spectrum of problems**

  e. g.: Small strain stiffness (G_o, M_o) and damping ratio (D_o), Porosity (n),

  Saturation line*, Liquefaction susceptibility $f(V_s)$, Quality control (V_s)

  of laboratory samples and of soil improvement, …

- **Of main Interest:**

  Borehole methods: CHT, DHT,** …

  Penetration methods: S-CPT**, S-DMT**, …

  Surface methods: SASW

  (*)   Applicable in loose deposits, $V_p < 1600$ m/s

  (**) Two receivers mandatory

SOIL STIFFNESS FROM SEISMIC TESTS

■ **Shear modulus:**

$$G_o = \rho \cdot V_s^2$$

■ **Constrained modulus:**

$$M_o = \rho \cdot V_p^2$$

■ **Young's modulus:**

$$E_o = \rho \cdot V_p'^2$$

V_s = shear wave

V_p = constrained compression wave

V_p' = unconstrained compression wave

⎫ VELOCITIES

BASIC ASSUMPTIONS: ⟹

Medium is infinite

Material is linear and elastic

Wave radius of curvature is infinite

ADVANCED SEISMIC METHODS
ACTIVE SURFACE TECHNIQUE

Rix et al. (2001), Lai et al. (2002), Lai (2004)

MULTI-STATION METHOD AS OPPOSED TO CURRENT TWO-STATION METHOD

UNCOUPLED OR COUPLED INVERSION

{ *OF PHASE VELOCITY* ➔ *SHEAR WAVE VELOCITY PROFILE*

OF ATTENUATION CURVE ➔ *MATERIAL DAMPING RATIO PROFILE*

■ **DEPTH LIMITED BY RESTRICTION IN POWER OF IMPULSIVE AND HARMONIC SOURCES USED**

■ **BETTER MEDIUM RESPONSE FUNCTIONS; DISPLACEMENT TRANSFER FUNCTIONS, i. e.: DISPLACEMENT PHASE AND DISPLACEMENT AMPLITUDES.**

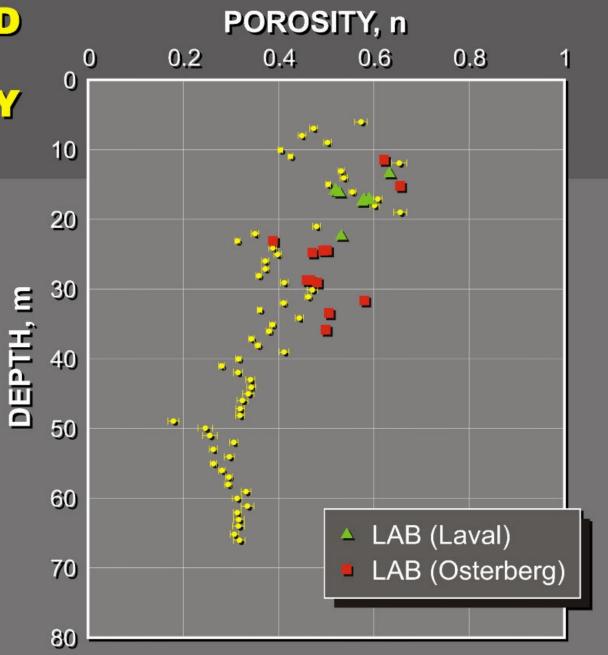# POROSITY OF GEOMATERIALS FROM MEASURED SEISMIC WAVE VELOCITIES

Lancellotta (2001), Foti et al. (2002)

$$n = \frac{\rho_s - \left[\rho_s^2 - \dfrac{4(\rho_s - \rho_f)K_f}{V_p^2 - 2\left(\dfrac{1-\nu_s}{1-2\nu_s}\right)V_s^2}\right]^{0.5}}{2(\rho_s - \rho_f)}$$

$\Rightarrow$ **Connected porosity**

ρ_s = soil particles $\Big\}$ mass
ρ_f = pore fluid $\Big\}$ density
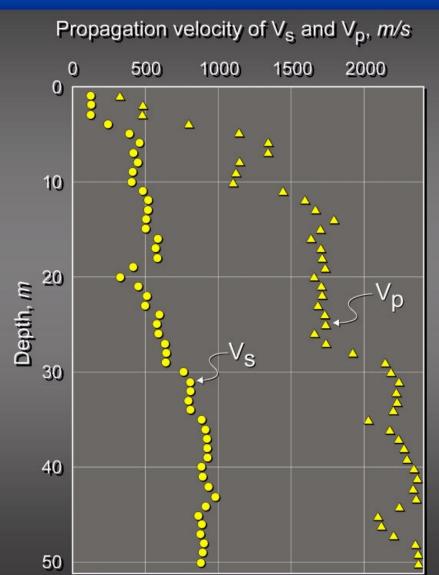
ν_s = Poisson ratio of soil skeleton
K_f = bulk modulus of pore fluid

- Valid in fluid saturated media

- Medium excited in low frequency range, no relative motion between fluid and solid phases (Biot, 1956), $f_c < (40 \text{ to } 50) \, 10^3 \, Hz$

- Influence of $0.10 \leq \nu_s \leq 0.20$ on computed n very small

POROSITY PREDICTED AT PISA SITE USING BIOT's THEORY
Foti et al (2001)

POROSITY PREDICTED AFTER FOTI ET AL. (2001) AT FIRENZE SITE USING BIOT'S (1956) THEORY

SMALL STRAIN POISSON RATIO FROM SEISMIC TESTS
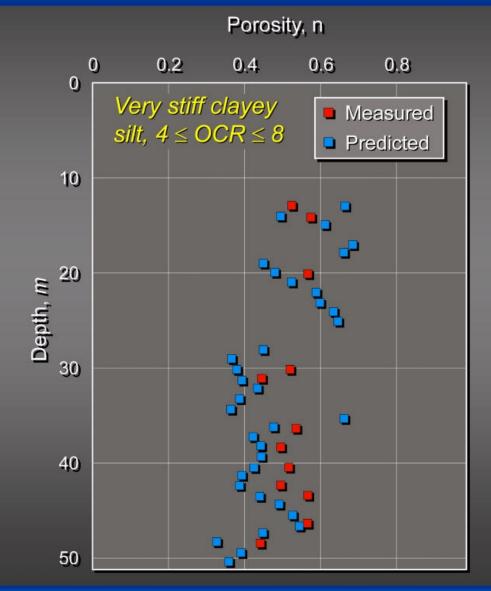
- **MONOPHASE MEDIUM**:

$$\nu'_o = \frac{(V_p/V_s)^2 - 2}{2(V_p/V_s) - 2}$$

Yields soil skeleton ν'_o in dry and far from saturation soils

- **TWO-PHASE SATURATED MEDIUM IN LOW FREQUENCY RANGE***,
Biot (1956), Foti et al. (2002):
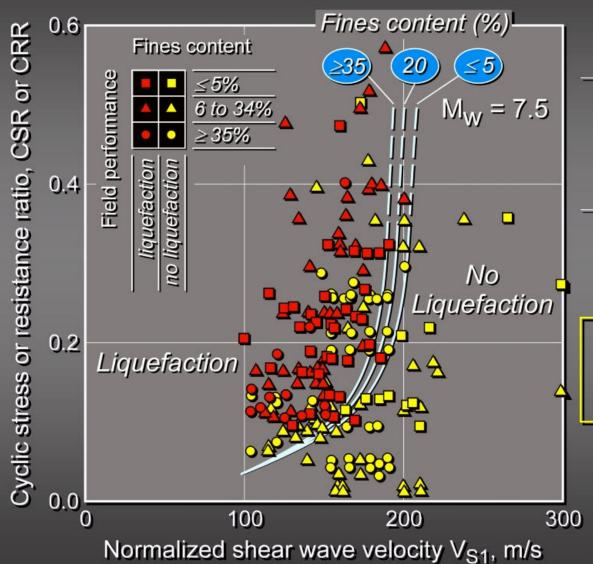
$$V_p = f(K_s, K_f, G, n)$$

K_f =	pore fluid } bulk	G =	Shear modulus of soil skeleton
K_s =	soil particles } modulus	ρ =	porous media mass density

- **To evaluate ν'_o three independent measurements (V_p, V_s, n) required**
- **In coarse grained geomaterials $0.12 < \nu'_o \leq 0.20$, affected by $\sigma'_{vc}/\sigma'_{hc}$**

(*) No relative movement occurs between fluid and solid phases.

CURVES RECOMMENDED FOR CALCULATION OF CRR FROM SHEAR WAVE VELOCITY MEASUREMENTS - BASED ON CASE HISTORY DATA

Andrus et al. (1999), Andrus and Stokoe (2000).

Data base: Uncemented Holocene coarse grained soils, $5.9 \leq M_W \leq 8.3$, MSF and r_d after Idriss (1999)
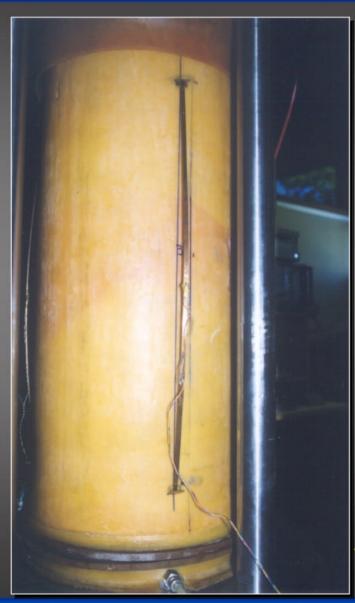
Curves for FC $\leq$ 20% applicable to gravelly soils. In first approximation:

$V_{s1}^t = 215$ m/s when FC $\leq$ 5%;

$V_s^t = 215 - 0.5$ (FC-5) m/s when FC>5%

$V_{s1}^t =$ limiting upper value of V_{s1} for liquefaction occurrence

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GRAVELLY GEOMATERIALS
LABORATORY TESTING

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LABORATORY EQUIPMENT

LARGE DIMENSIONS APPARATUSES

$(D/D_{max} > 6)$

Wu Chi Kao (1986) **RC**

Yasuda and Matsumoto (1993, 1994) **TS**

Hoque (1996) **TX**

Anh Dan (2001) **TT**

Meng and Stokoe (2003) **RC – FF**

IMPORTANT REQUIREMENTS:

- Local axial and radial strain measurement

- Low compliance, frictionless loading frame

- Actuator able to apply constant rate of strain as well as small and large cyclic loading

Technical University of Torino triaxial cell, H = 600 mm, D = 300 mm

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CAI-07

G-133   CAI-11

SMALL STRAIN YOUNG'S MODULUS EMPIRICAL RELATIONSHIP* ($\varepsilon \leq \varepsilon_t^{\ell}$)

$$E_o(V) = C_E(V)\,F(e)\,(\sigma'_v)^{nv}\,p_a^{(1-nv)}$$

$$E_o(H) = C_E(H)\,F(e)\,(\sigma'_h)^{nh}\,p_a^{(1-nh)}$$

C_E = dimensionless anisotropic material constant; p_a = 1 bar = reference stress

$F(e) = \dfrac{1}{e^x}$ = Void ratio function, $1.3 \leq x \leq 1.5$

$\left.\begin{array}{l} \sigma'_v \\ \sigma'_h \end{array}\right\}$ vertical and horizontal effective stress respectively

$\left.\begin{array}{l} nv \\ nh \end{array}\right\}$ experimental stress exponents function of U_c

(*) Applies to uncemented geomaterials

SMALL STRAIN SHEAR MODULUS
EMPIRICAL RELATIONSHIP* ($\gamma \leq \gamma_t^{\ell}$)

$$G_o(VH) = C_G(VH)\, F(e)\, (\sigma'_v)^{nv}\, (\sigma'_h)^{nh}\, p_a^{(1-nv-nh)}$$

$$G_o(HH) = C_G(HH)\, F(e)\, (\sigma'_h)^{2nh}\, p_a^{(1-2nh)}$$

C_G = dimensionless anisotropic material constant; p_a = 1 bar = reference stress

$F(e) = \dfrac{1}{e^x}$ = Void ratio function, $1.3 \leq x \leq 1.5$

$\left.\begin{array}{l} \sigma'_v \\ \sigma'_h \end{array}\right\}$ vertical and horizontal effective stress respectively

$\left.\begin{array}{l} nv \\ nh \end{array}\right\}$ experimental stress exponents function of U_c

(*) Applies to uncemented geomaterials

SMALL STRAIN SHEAR MODULUS OF GRAVELLY SOIL
UNDISTURBED vs. RECONSTITUTED* SPECIMENS

Hatanaka et al. (1988), Goto et al. (1987, 1992, 1994), Kokusho and Tanaka (1994), Yasuda et al. (1994), Hatanaka and Uchida (1995)

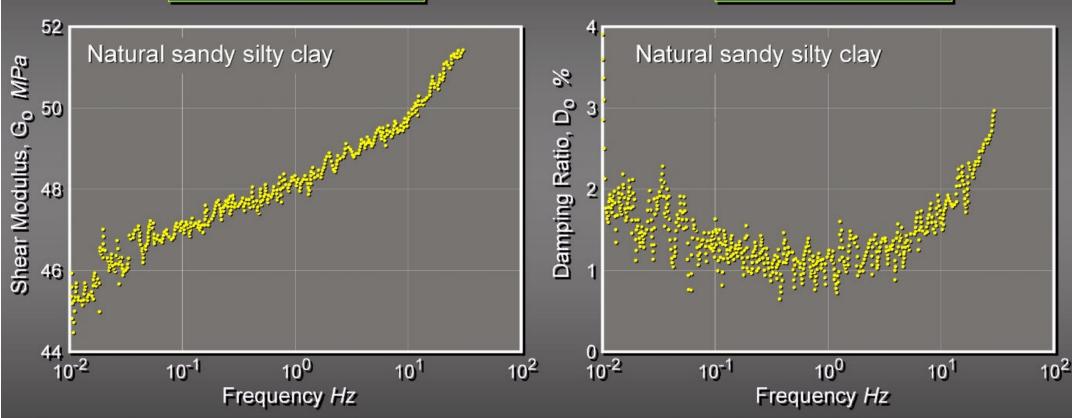|	D_{50} (mm)	U_c (-)	$\dfrac{G_0 (U) **}{G_0 (R)}$
HOLOCENE DEPOSITS	5.3 to 73.0	54 to 121	1.2 to 1.3
PLEISTOCENE DEPOSITS	1.8 to 53.0	9 to 60	2.0 to 3.0

(*) At the same state (e_0, σ'); (**) Samples retrieved by means of freezing technique

A NON-RESONANCE METHOD MEASURING DYNAMIC SOIL PROPERTIES

Rix and Meng (2004)

- Small strain shear modulus G_0 little affected by tenfold change of frequency
- Small strain damping ratio D_0 more sensitive to tenfold change of frequency

LABORATORY ASSESSMENT OF G_0 EMPLOYING PIEZOELECTRIC TRANSDUCERS

- **NEAR-FIELD EFFECTS**, see: Sanchez – Salinero et al. (1986), Jovcic (1997), Arroyo et al. (2003).......................... $2 < d/\lambda < 4$; d = travel distance

- **ENERGY TRANSMISSION THROUGH CONTINUUM**: Fam and Santamarina (1995)............. $\lambda >> D_{50}$

- **LABORATORY SPECIMENS = HOMOGENEOUS MEDIUM**: Tanaka et al. (1994, 1998, 2000), Anh Dan et al. (2002), Maqbooll et al. (2004)* .... $\lambda/D_{50} > 200$ to 500
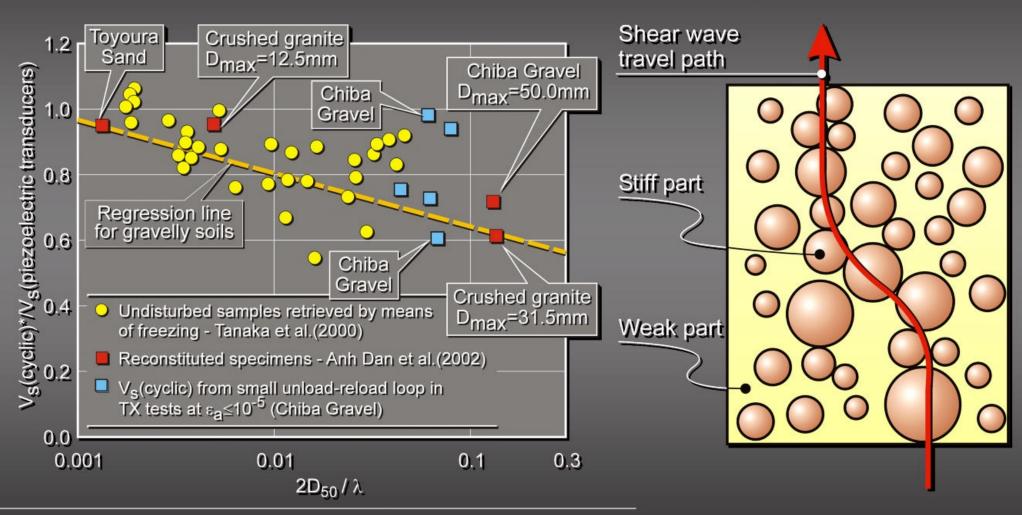
> **Above issues not sufficient to explain scatter observed in experimental data, Arroyo (2003)**

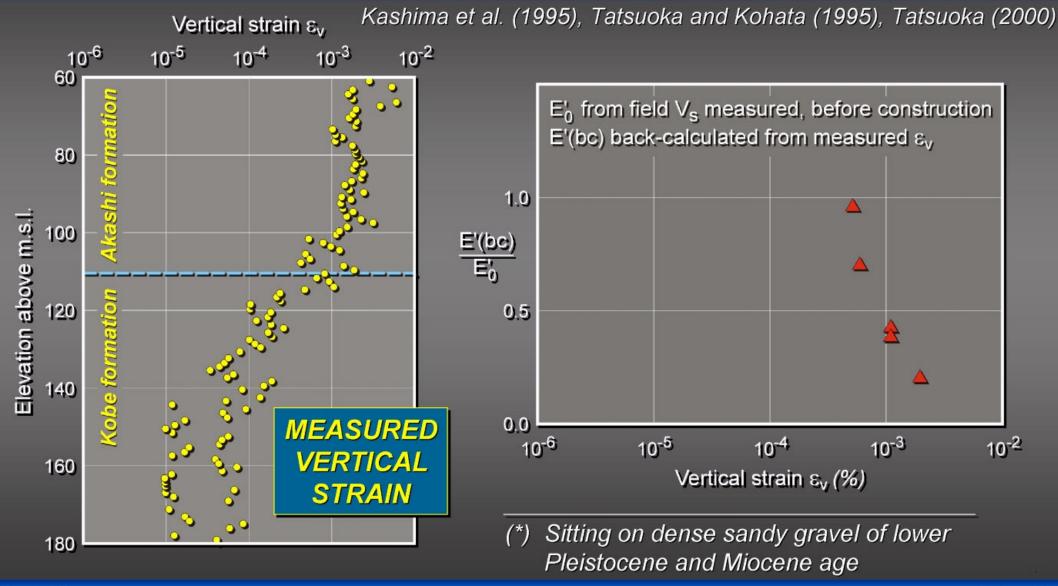(*) Especially relevant for gravelly geomaterials

EFFECT OF GRAIN SIZE ON LABORATORY SEISMIC TESTS RESULTS

Tanaka et al. (1994, 1998, 2000), Anh Dan et al. (2002), Maqbool et al. (2004)
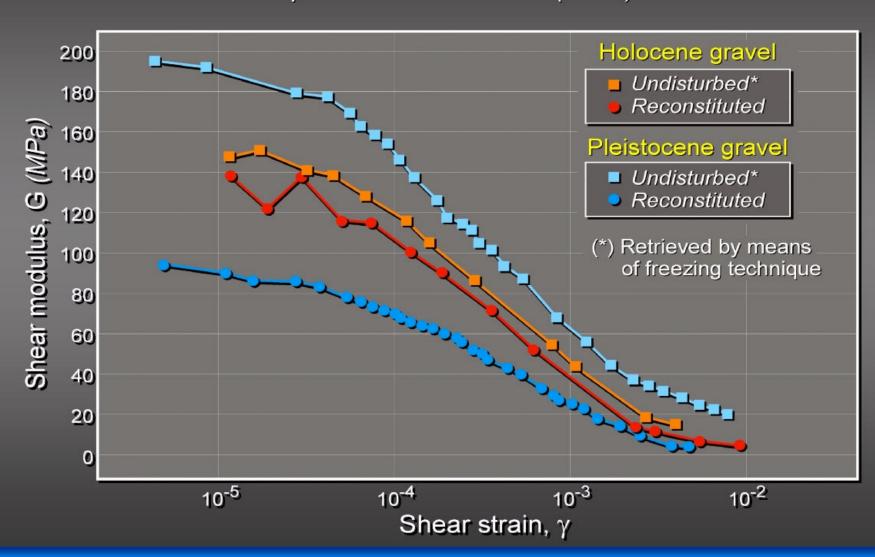

(*) V_S(cyclic) from G_0 (small unload-reload loop)

VERTICAL STRAIN AND YOUNG MODULUS DECAY UNDER PIER 2P*-AKASHI KAIKYO BRIDGE

Kashima et al. (1995), Tatsuoka and Kohata (1995), Tatsuoka (2000)

Vertical strain ε_v

Akashi formation

Kobe formation

Elevation above m.s.l.

MEASURED VERTICAL STRAIN

E'_0 from field V_s measured, before construction
$E'(bc)$ back-calculated from measured ε_v

$\dfrac{E'(bc)}{E'_0}$

Vertical strain ε_v (%)

(*) Sitting on dense sandy gravel of lower Pleistocene and Miocene age

SHEAR MODULUS OF TWO GRAVELLY SOILS FROM TRIAXIAL TESTS

Adapted after Goto et al. (1994)

Holocene gravel
- ■ Undisturbed*
- ● Reconstituted

Pleistocene gravel
- ■ Undisturbed*
- ● Reconstituted

(*) Retrieved by means of freezing technique
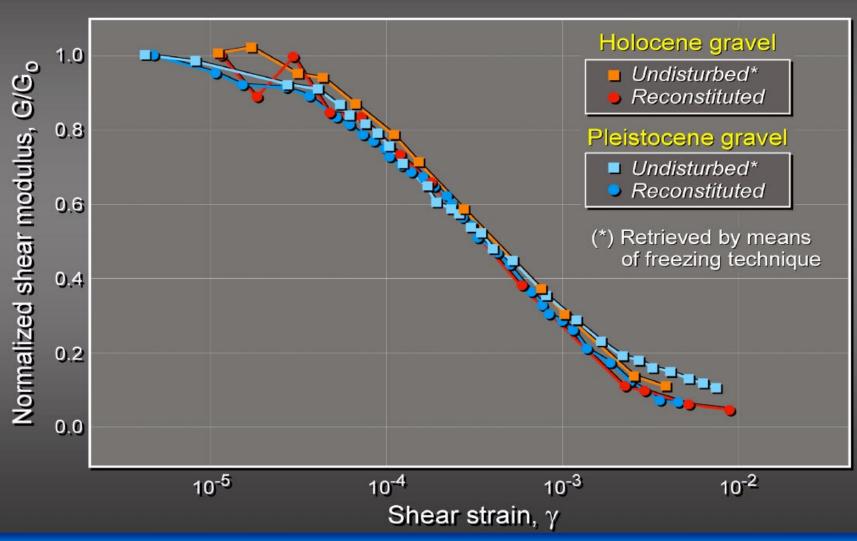
NORMALIZED SHEAR MODULUS OF TWO GRAVELLY SOILS FROM TRIAXIAL TESTS

Adapted after Goto et al. (1994)
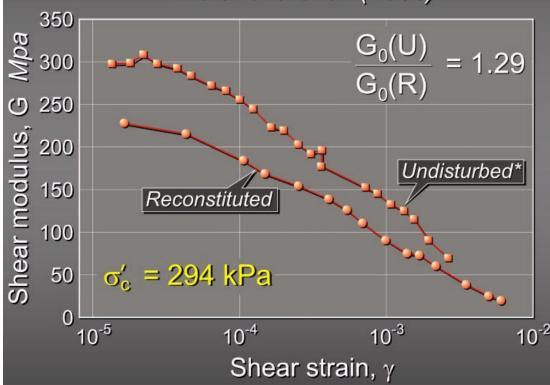
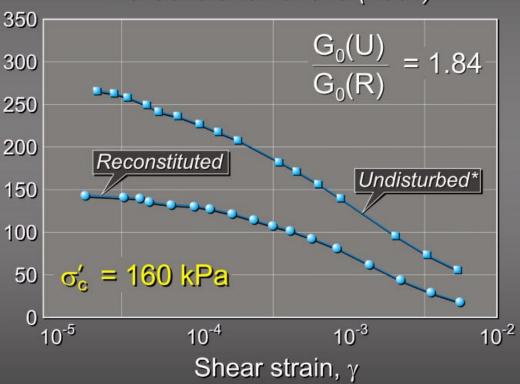# SHEAR MODULUS OF TWO GRAVELLY SOILS FROM CYCLIC TRIAXIAL TESTS

Holocene Tokyo Gravel
Hatanaka et al. (1988)

$$\frac{G_0(U)}{G_0(R)} = 1.29$$

Reconstituted

Undisturbed*

$\sigma'_c = 294$ kPa

Shear modulus, G Mpa

Shear strain, γ

Pleistocene Gravel, K-site
Kokusho and Tanaka (1994)

$$\frac{G_0(U)}{G_0(R)} = 1.84$$
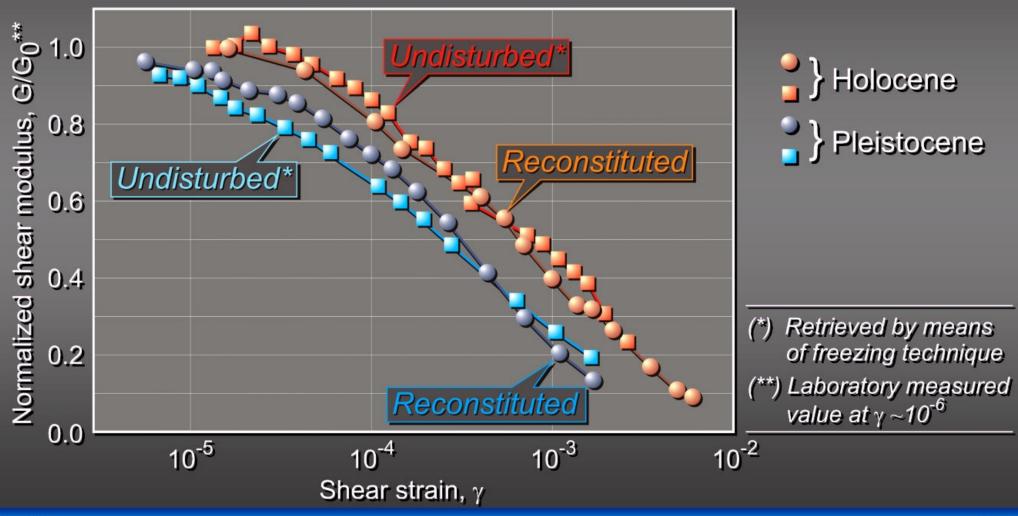
Reconstituted

Undisturbed*

$\sigma'_c = 160$ kPa

Shear strain, γ

(*) Retrieved by means of freezing technique

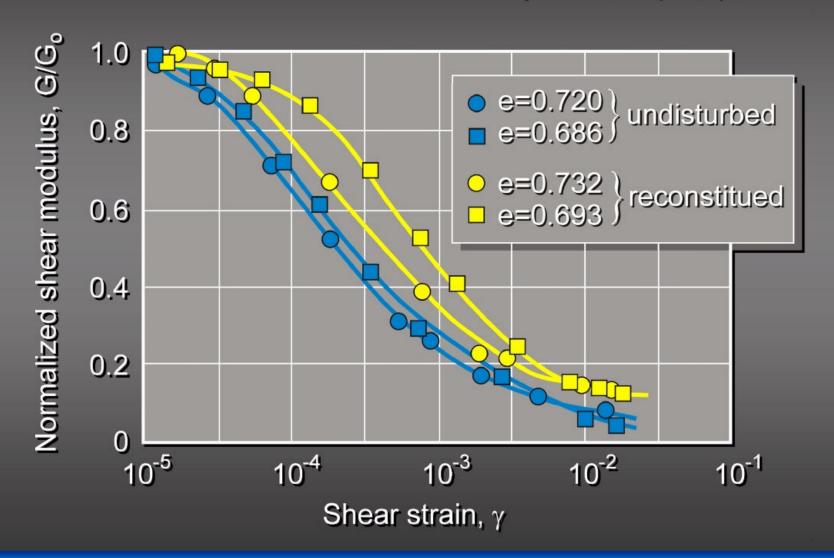# NORMALIZED SHEAR MODULUS OF TWO GRAVELLY SOILS FROM CYCLIC TRIAXIAL TESTS

Adapted after Hatanaka et al. (1988) and Kokusho and Tanaka (1994)

Undisturbed*

Undisturbed*

Reconstituted

Reconstituted

} Holocene

} Pleistocene

(*) Retrieved by means of freezing technique

(**) Laboratory measured value at $\gamma \sim 10^{-6}$

Normalized shear modulus, G/G_0**

Shear strain, γ

NORMALIZED SHEAR MODULUS OF FUJISAWA SAND UNDISTURBED vs. RECONSTITUTED SAMPLES

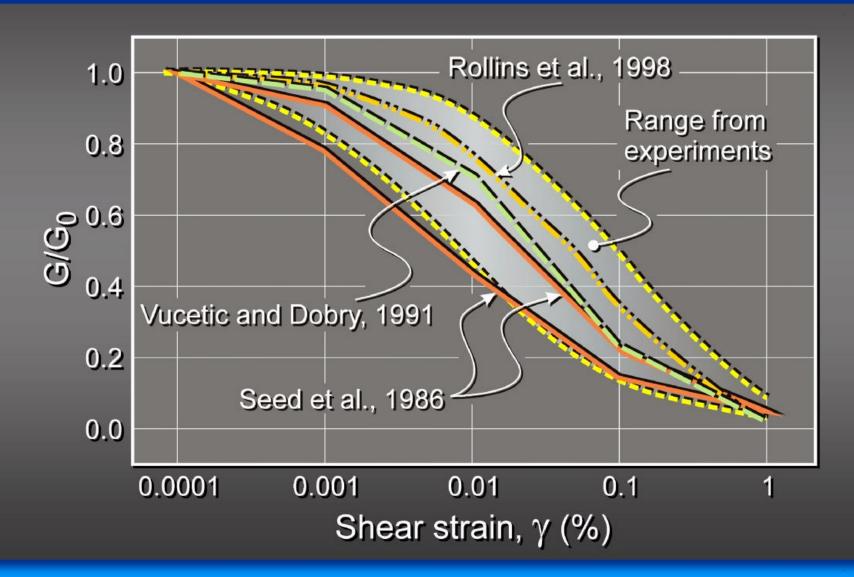*Katayama et al. (1986), Ishihara (1996)*
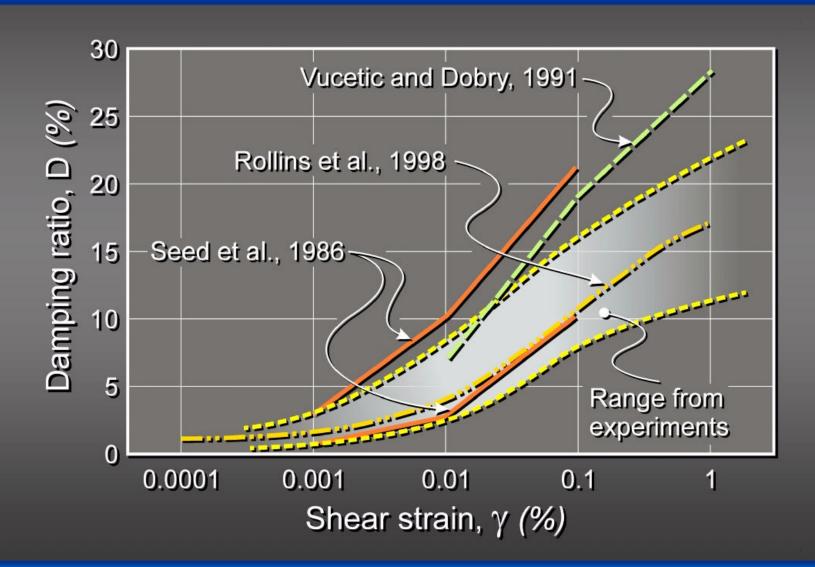
LABORATORY DETERMINED STIFFNESS DEGRADATION CURVES

Comparison of $G/G_O - \log\gamma$ (undisturbed) vs. $G/G_O - \log\gamma$ (reconstituted)*:

- *Very similar, Goto et al. (1992, 1994), Yasuda et al. (1994), Hatanaka and Uchida (1995), Ishihara (1996)*

- *Degradation of $G/G_O - \log\gamma$ of undisturbed soil can be more pronounced than for reconstituted* material, Ishihara (1996)*

- *Similar contradictory experimental results can be found as far as increase of damping ratio D with increasing $\log\gamma$ is concerned*

(*) Reconstituted at the <u>same state</u> (e_0 , σ') and normalized with respect to <u>field determined G_O</u>
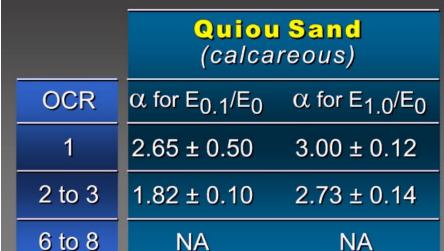
RELATIONSHIP BETWEEN G/G_0 AND SHEAR STRAIN OBTAINED FROM CYCLIC TRIAXIAL TESTS ON GRAVEL

RELATIONSHIP BETWEEN DAMPING RATIO AND SHEAR STRAIN OBTAINED FROM CYCLIC TRIAXIAL TESTS ON GRAVEL
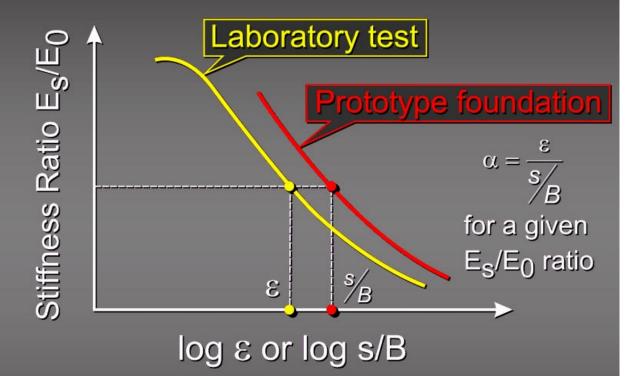
SOIL NON-LINEAR STIFFNESS: SIMPLE DESIGN PROCEDURE

Quiou Sand
(calcareous)

OCR	α for $E_{0.1}/E_0$	α for $E_{1.0}/E_0$
1	2.65 ± 0.50	3.00 ± 0.12
2 to 3	1.82 ± 0.10	2.73 ± 0.14
6 to 8	NA	NA

Ticino Sand
(silica)

OCR	α for $E_{0.1}/E_0$	α for $E_{1.0}/E_0$
1	2.91 ± 0.33	3.72 ± 0.60
2 to 3	1.49 ± 0.21	3.41 ± 0.29
6 to 8	1.19 ± 0.21	1.95 ± 0.13

(*) See also Barbero (2001) and Bovolenta (2003)

Atkinson (2000), Barbero (2001), Bovolenta (2003)

Laboratory test

Prototype foundation

Stiffness Ratio E_s/E_0

log ε or log s/B
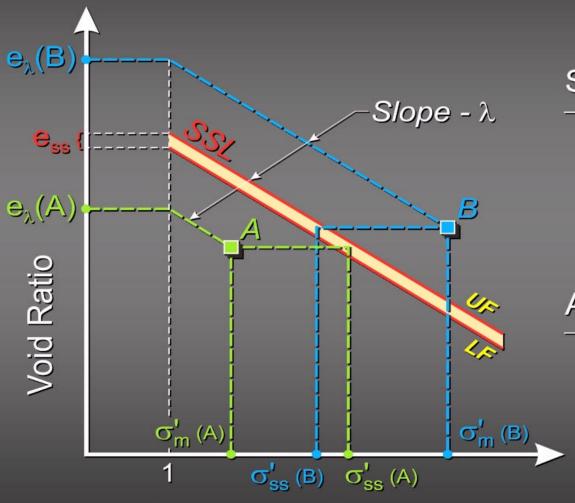
$$\alpha = \frac{\varepsilon}{s/B}$$

for a given E_s/E_0 ratio

E_0 at $\varepsilon \leq \varepsilon_t^\ell$

$E_{0.1}$ at $\varepsilon = 0.1\%$ or s/B=0.1

$E_{1.0}$ at $\varepsilon = 1\%$ or s/B=1

SIGNIFICANCE OF STATE PARAMETER FOR COARSE GRAINED GEOMATERIALS

Jefferies and Been (1985), Hird and Hassona (1986), Konrad (1993), Leroueil and Hight (2002)

State parameter, ξ:

$$\xi_A = e_\lambda(A) - e_{ss} \longrightarrow \text{negative}$$

$$\xi_A = e_\lambda(B) - e_{ss} \longrightarrow \text{positive}$$

A substitute of OCR:

$$\frac{\sigma'_{ss}}{\sigma'_m} = \exp\left(2.3\frac{\xi}{\lambda}\right)$$

Figure labels: $e_\lambda(B)$, e_{ss}, $e_\lambda(A)$, SSL, Slope - λ, A, B, UF, LF, Void Ratio, Mean effective stress, $\sigma'_m(A)$, $\sigma'_m(B)$, 1, $\sigma'_{ss}(B)$, $\sigma'_{ss}(A)$

CLOSING
REMARKS

G-130

BAN-26

CLOSING REMARKS

- *Phenomenological framework behaviour as proposed by Jardine (1985) and Jardine et al. (1991) can satisfactorily describe stress-strain response of gravelly geomaterials*

- *Among penetration tests, SPT and LPT, normalizing the results with respect to the energy delivered to the driving rods and to the effective overburden stress, appear most suitable for the soils in question*

CLOSING REMARKS

■ *On the increase applications of in situ geophysical methods for multipurpose geotechnical site characterization*

■ *Of them, the most relevant are:*

- *Assessment of small strain shear modulus*

- *Evaluation of in situ porosity in fully saturated deposits*

- *Simultaneous evaluation of small strain shear modulus and damping ratio profiles from multi station SASW method*

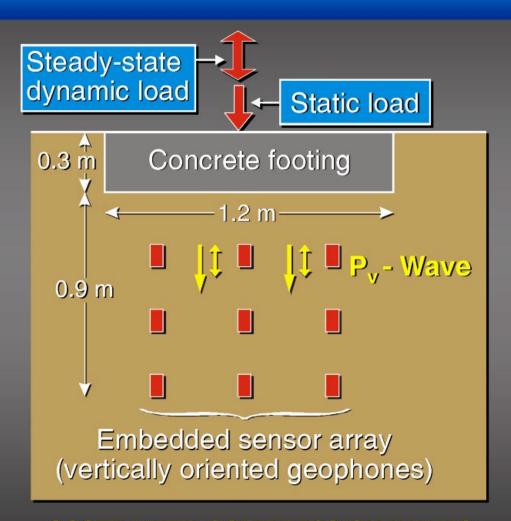- *Use of shear wave velocity for evaluating liquefaction susceptibility*

CLOSING REMARKS

- Initial small strain ($\varepsilon \leq \varepsilon_t^{\ell}$) shear (G_o) and Young's (E_o) are reliably assessed from laboratory tests on <u>undisturbed</u> samples of gravelly soils when <u>local strains</u> are measured

- Stiffness of <u>undisturbed</u> gravelly geomaterials at small and intermediate strain is higher than those reconstituted, with same void ratio and subject to same consolidation stress, difference increases with increasing deposit's age

- Use of seismic tests in laboratory is subject to a number of stringent requirements as far as near-field effects and specimen homogeneity conditions are concerned
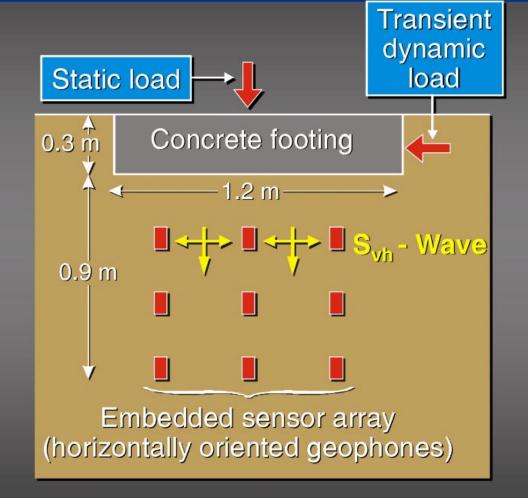
CLOSING REMARKS

- G/G_o vs. $\log\gamma$ degradation curves* as obtained from laboratory tests on undisturbed samples are representative for in situ stiffness non-linearity

- The same hypothesis applied to the G/G_o vs. $\log\gamma$ curves* obtained from laboratory tests on reconstituted specimens still questionable, see Ishihara (1996)

(*) In both cases reference is made to the field determined G_o

APPROACH TO LARGE-STRAIN DYNAMIC IN SITU TEST
Stokoe et al (2001)

Steady-state dynamic load

Static load

0.3 m

Concrete footing

1.2 m

0.9 m

P_v - Wave

Embedded sensor array
(vertically oriented geophones)

COSTRAINED COMPRESSION WAVES

Static load

Transient dynamic load

0.3 m

Concrete footing

1.2 m

0.9 m

S_{vh} - Wave

Embedded sensor array
(horizontally oriented geophones)

SHEAR WAVES

UNDISTURBED SAMPLE OF GRAVELLY SOIL

By courtesy of Tokyo Soil Research Co. Ltd.

G-130

BAN-51

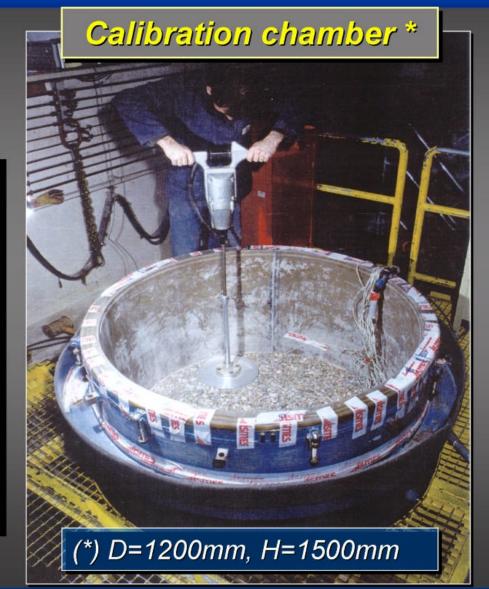G-133   CAI-05

SEISMIC TESTS IN CALIBRATION CHAMBER RECONSTITUTED TICINO RIVER GRAVELLY SOILS
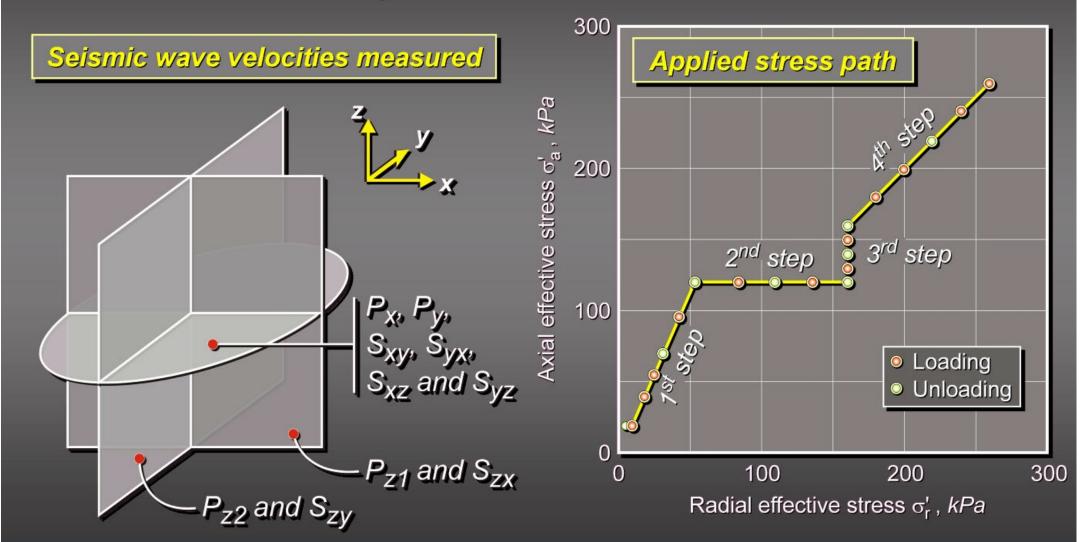
Brignoli et al. (1997), Ismes (1998)

Test material

Mix -	A	C	E	Ba
GC %	18.3	38.2	65.1	77.5
SC %	81.7	61.8	34.9	17.8
FC %	3.8	3.2	2.1	4.9
D_{50} mm	0.433	0.693	3.751	7.965
U_c -	6.1	17.2	32.1	25.2
e_{max} -	0.633	0.433	0.382	0.477
e_{min} -	0.362	0.258	0.177	0.246

Calibration chamber *

(*) D=1200mm, H=1500mm

SEISMIC TESTS IN CALIBRATION CHAMBER RECONSTITUTED TICINO RIVER GRAVELLY SOILS

Brignoli et al. (1997), Ismes (1998)

Seismic wave velocities measured

P_x, P_y, S_{xy}, S_{yx}, S_{xz} and S_{yz}

P_{z1} and S_{zx}

P_{z2} and S_{zy}

Applied stress path

Axial effective stress σ'_a, kPa

Radial effective stress σ'_r, kPa

1st step

2nd step

3rd step

4th step

Loading
Unloading

SEISMIC TESTS IN CALIBRATION CHAMBER
RECONSTITUTED TICINO RIVER GRAVELLY SOILS
EMPIRICAL EQUATIONS FITTING EXPERIMENTAL DATA

Roesler (1979), Bellotti et al. (1996), Weston (1996)

$$V_s = C_s \sqrt{F(e)} (\sigma'_a)^{na} (\sigma'_b)^{nb} p_a^{(1-na-nb)}$$

$$na + nb = 0.22 \, U_c^{0.09}$$

$$C_s = 207 \, U_c^{-0.08}$$

$$x = -0.54$$

$$R^2 = 0.97$$

$$s_{yx} = 21 \, m/s$$

$$V_p = C_p \sqrt{F(e)} (\sigma'_a)^{na} (\sigma'_b)^{nb} p_a^{(1-na-nb)}$$

$$na + nb = 0.22 \, U_c^{0.07}$$

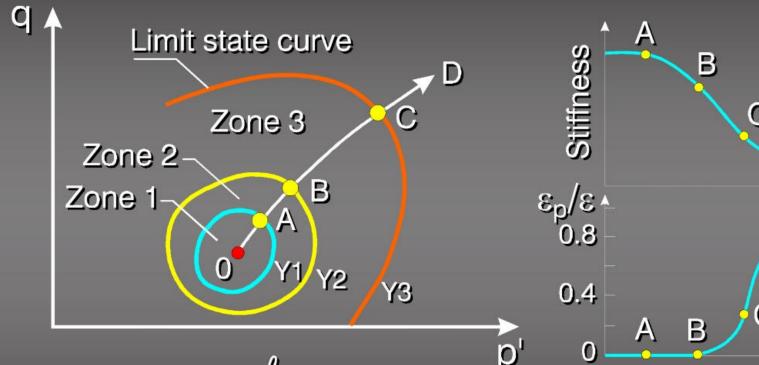$$C_p = 350 \, U_c^{-0.03}$$

$$x = -0.39$$

$$R^2 = 0.96$$

$$s_{yx} = 36 \, m/s$$

MULTIPLE YIELD SURFACES AND SOIL RESPONSE
Jardine et al (1991), Hight and Higgins (1994), Leroueil and Hight (2003)
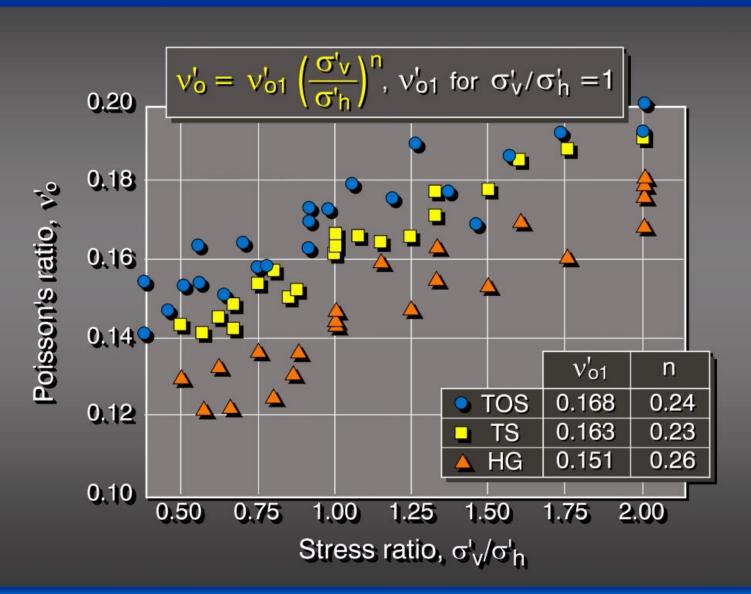
Zone 1: $\varepsilon \leq \varepsilon_t^\ell$

Zone 2: $\varepsilon_t^\ell < \varepsilon \leq \varepsilon_t^v$

Zone 3: $\varepsilon > \varepsilon_t^v$

POISSON'S RATIO ν'_o AT SMALL STRAIN AS FUNCTION OF STRESS RATIO

SHEAR MODULUS AND DAMPING RATIO OF FUJISAWA SAND UNDISTURBED vs. RECONSTITUTED SAMPLES

Katayama et al. (1986), Ishihara (1996)