Design guidelines for lattice type soil improvement with deep mixing for liquefaction mitigation

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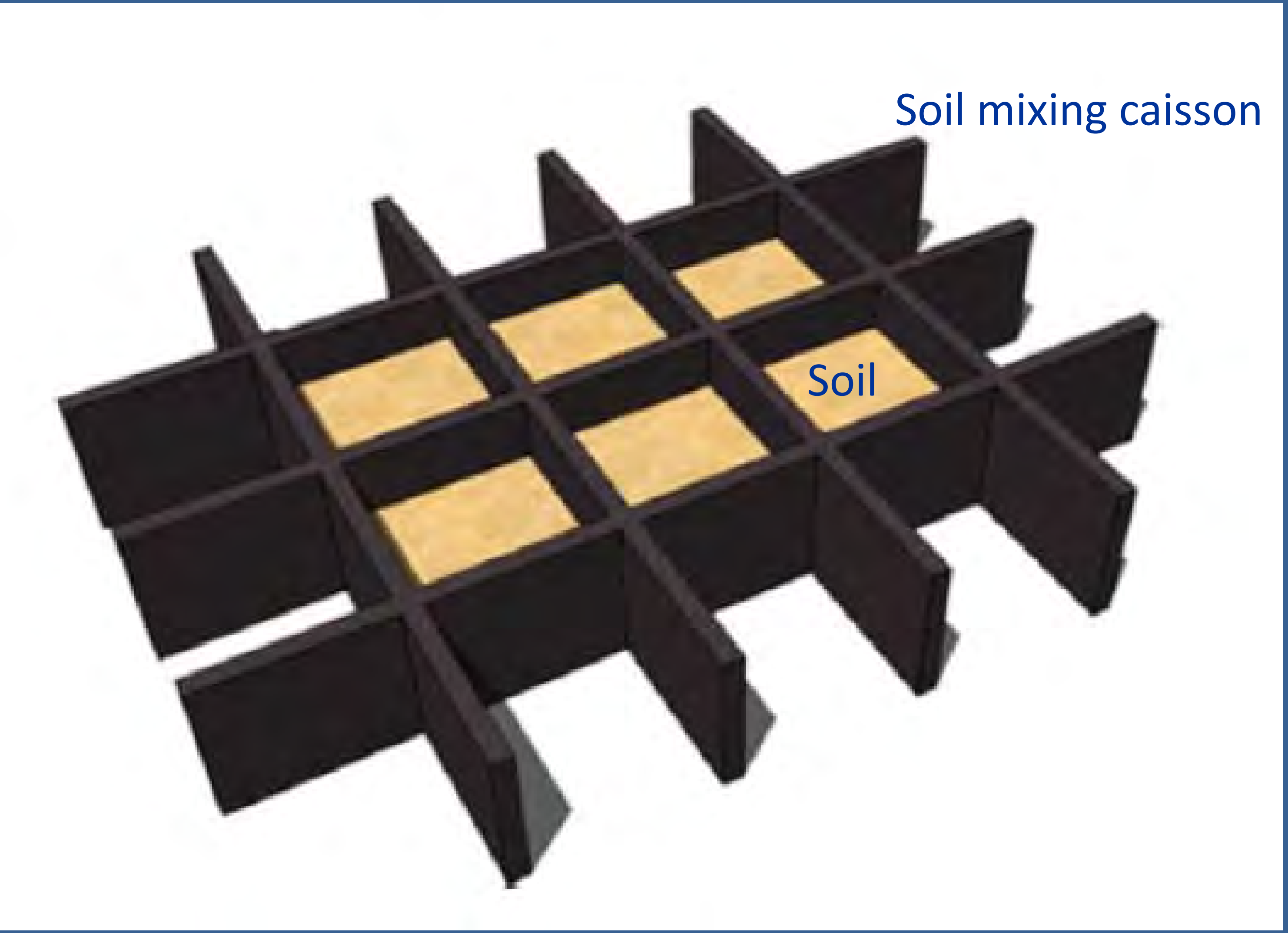
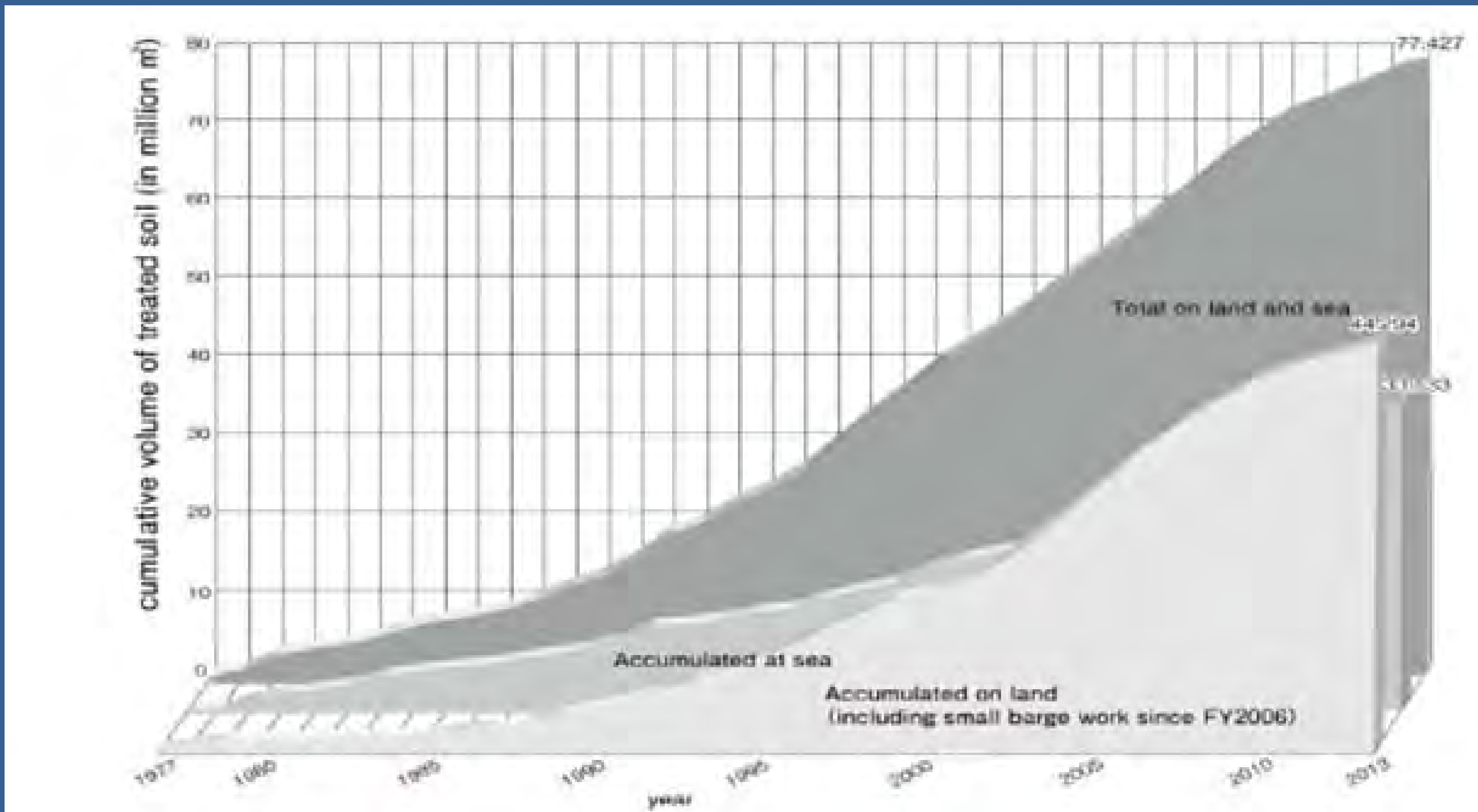
LATTICE TYPE SOIL IMPROVEMENT WITH DEEP MIXING



OUTLINE OF THE PRESENTATION

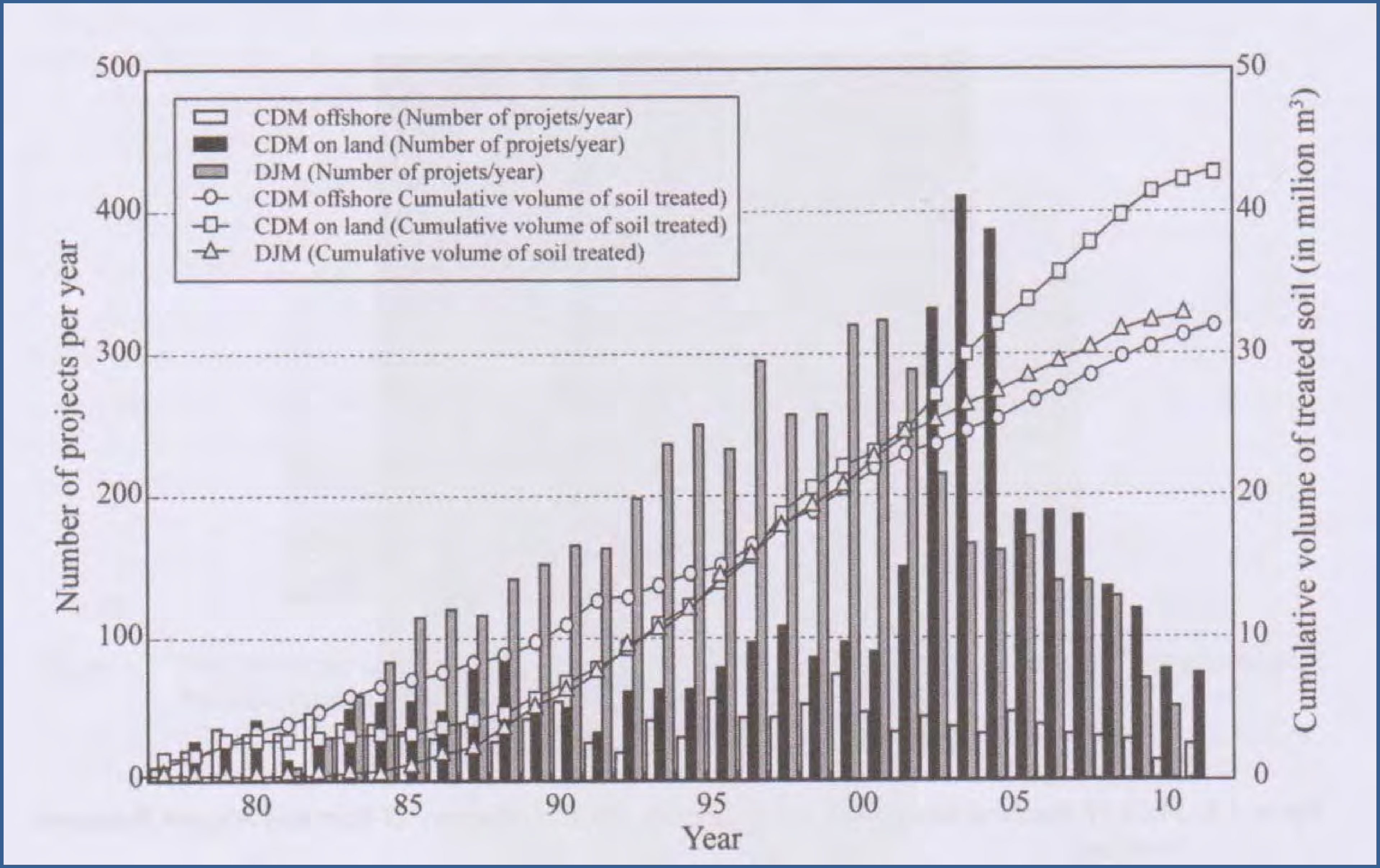
- Motivation for the study and some examples
- Fundamentals of seismic behavior of lattice type deep soil mixing
- Rapid overview of some previous studies and limitations
- The research program : experiments and numerical analyses
- Liquefaction assessment
  - seismic capacity
  - seismic demand
- Design procedure
- Conclusions

VOLUME OF TREATED SOIL BY SOIL MIXING



Tokunaga et al. (2015)

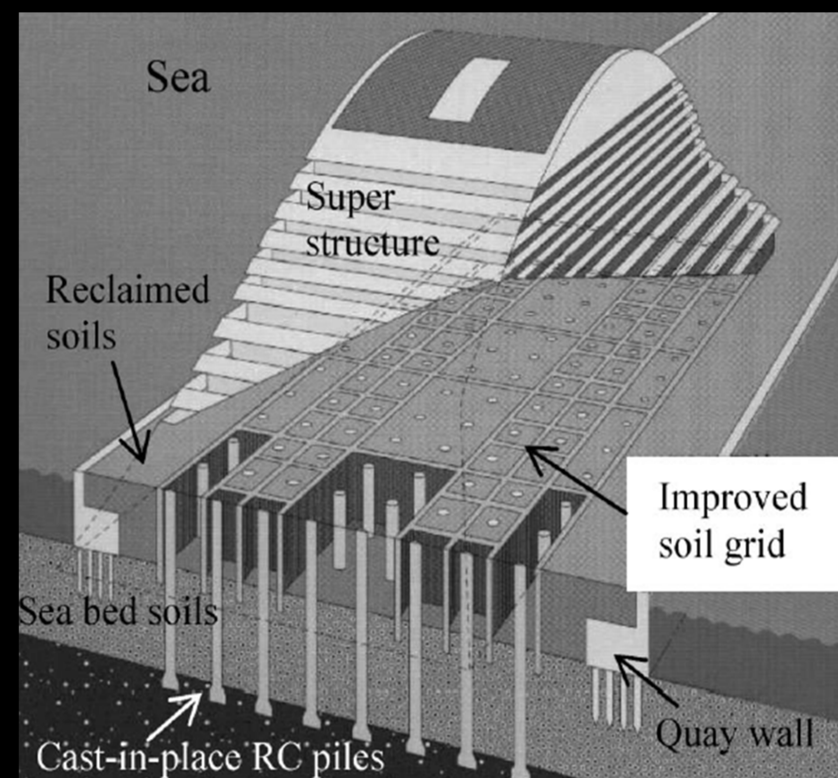
STATISTICS OF DEEP MIXING WORKS IN JAPAN



Kitazume – Terashi (2013)

POST-EARTHQUAKE OBSERVATIONS

- During the Yoko-ken-Nanbu earthquake
  - 14-story Oriental Hotel, founded on piles protected by a deep soil mixing grid (DSM) survived the earthquake

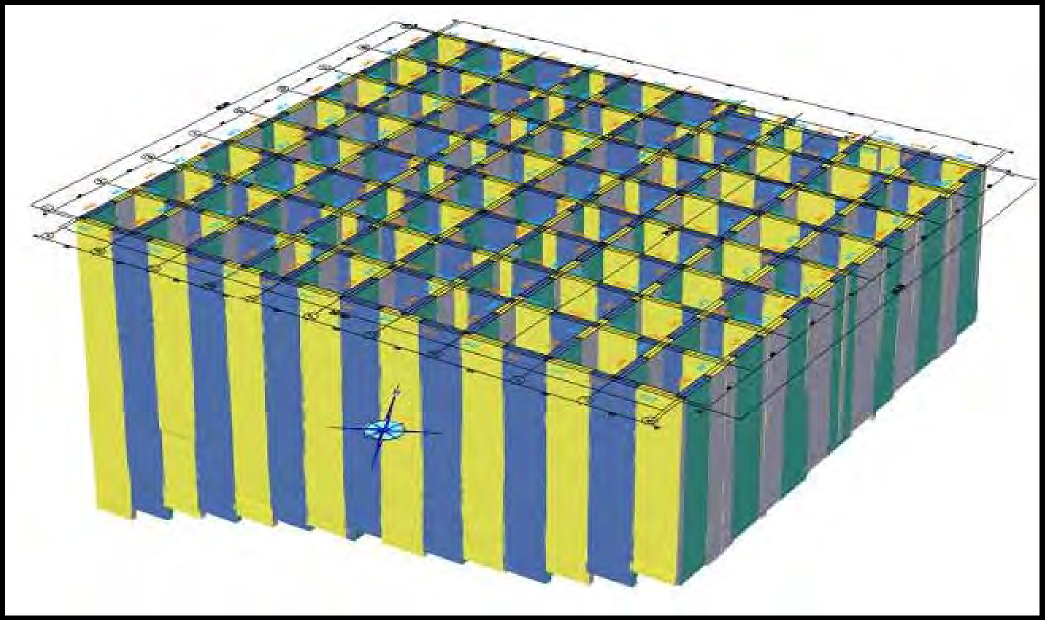


PREFECTURE FORT de FRANCE

Soletanche-Bachy

- Reclaimed hydraulic fill (9-17m)
- Sloping rock surface towards the sea ➡ lateral spreading
- Highly seismic area : M=7.5, pga=0.36g

PREFECTURE FORT de FRANCE



ANSE DU PORTIER - MONACO

Bouygues

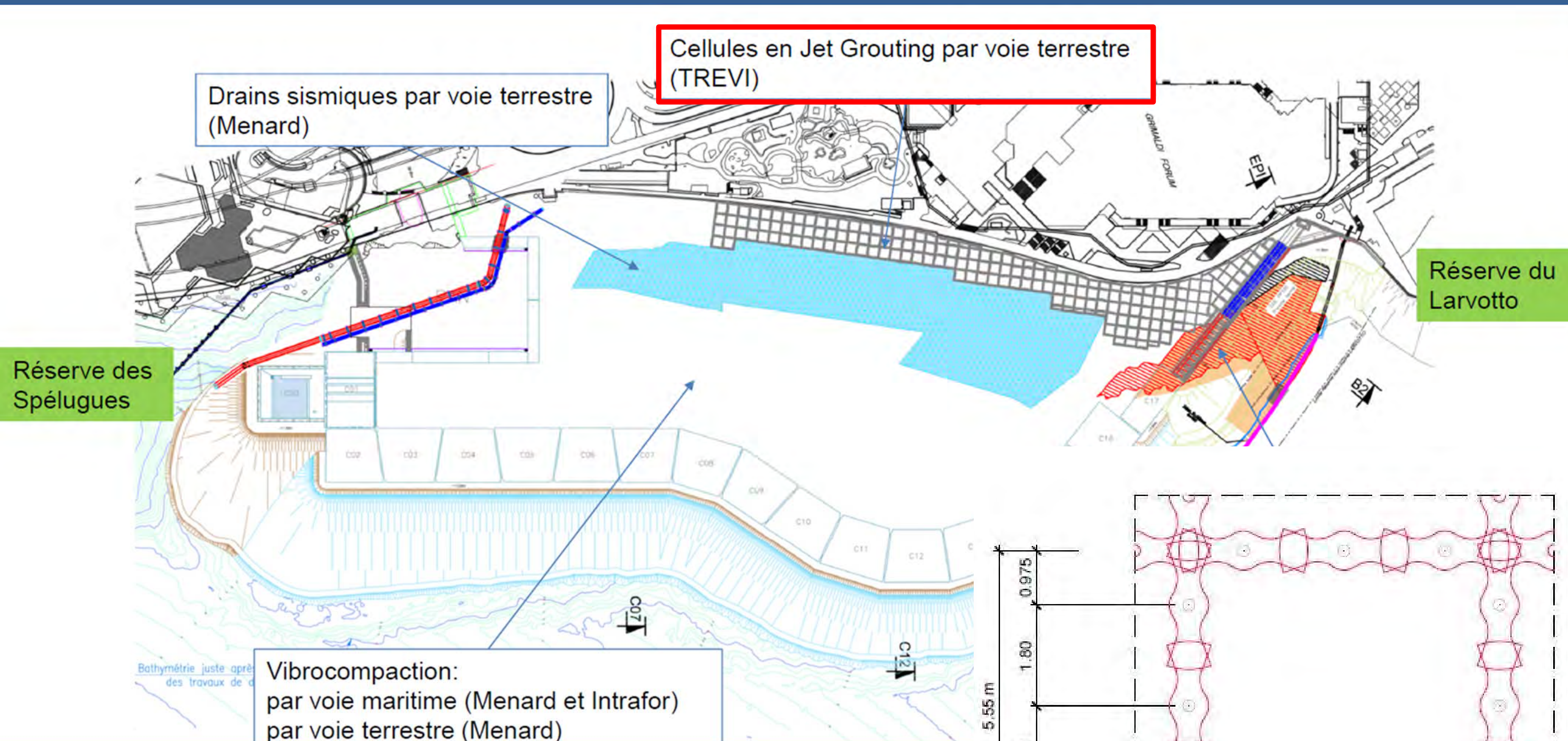
Perini-Borsellino-Jourdren : Webinar CFMS(2020)

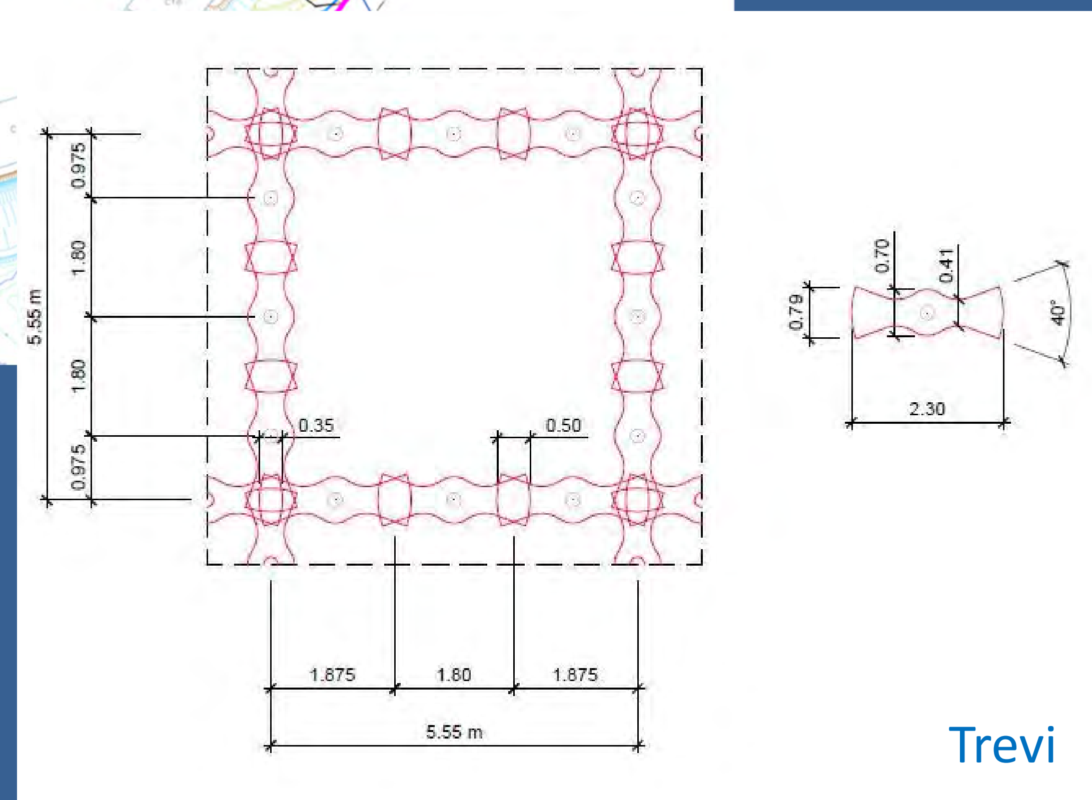
ANSE DU PORTIER – MONACO

Construction site

Perini-Borsellino-Jourdren : Webinar CFMS(2020)

SOIL IMPROVEMENT TECHNIQUES

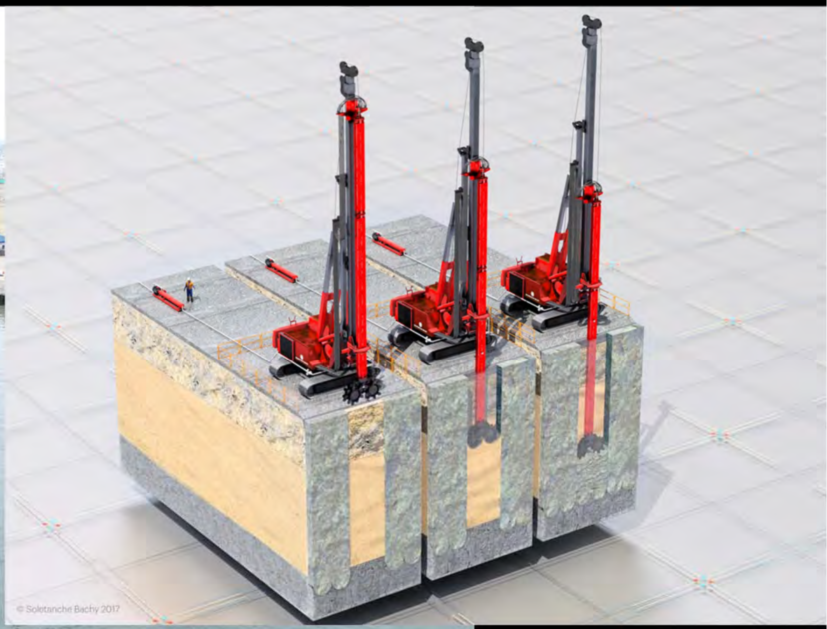




Perini-Borsellino-Jourdren : Webinar CFMS(2020)

PUERTO BOLIVAR HARBOUR

Soletanche-Bachy



T. Jeanmaire : CFMS (11/06/2024)

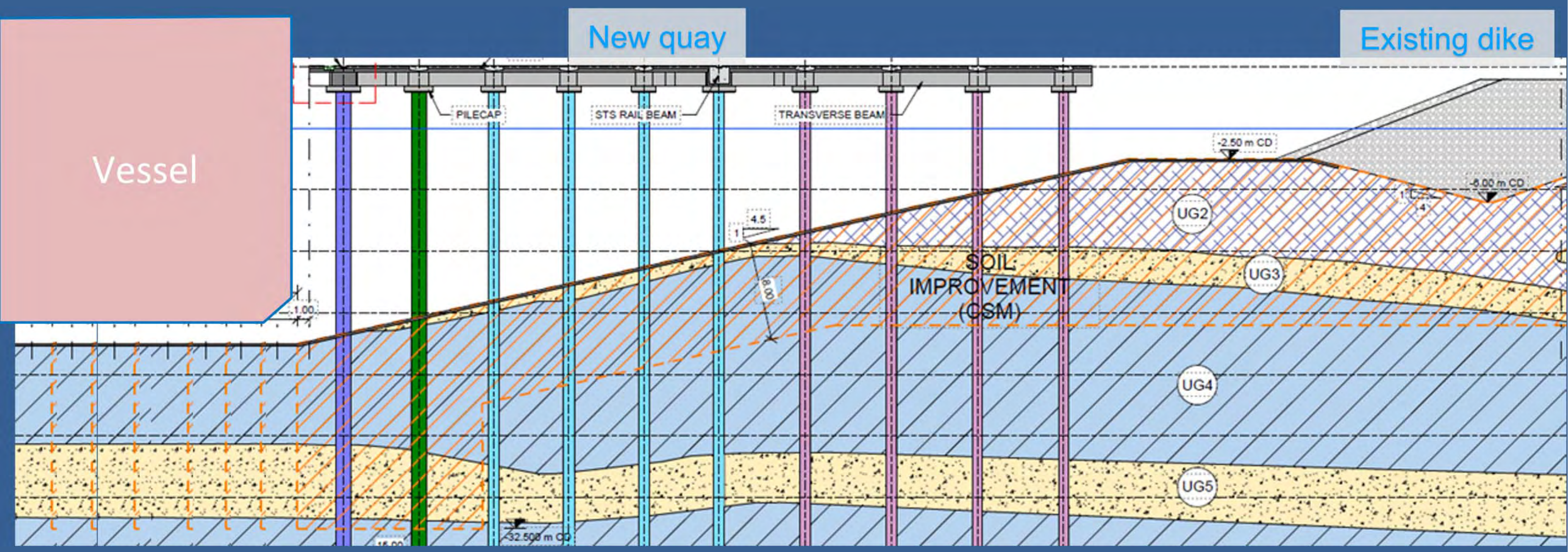
SOIL IMPROVEMENT FOR LIQUEFACTION MITIGATION AND SLOPE STABILSATION

Characteristics:

1500 CSM Pannels (45 500 m3)

2.8m x 0.8m - average height : 10m

R_c = 2.6 MPa @90 days



T. Jeanmaire : CFMS (11/06/2024)

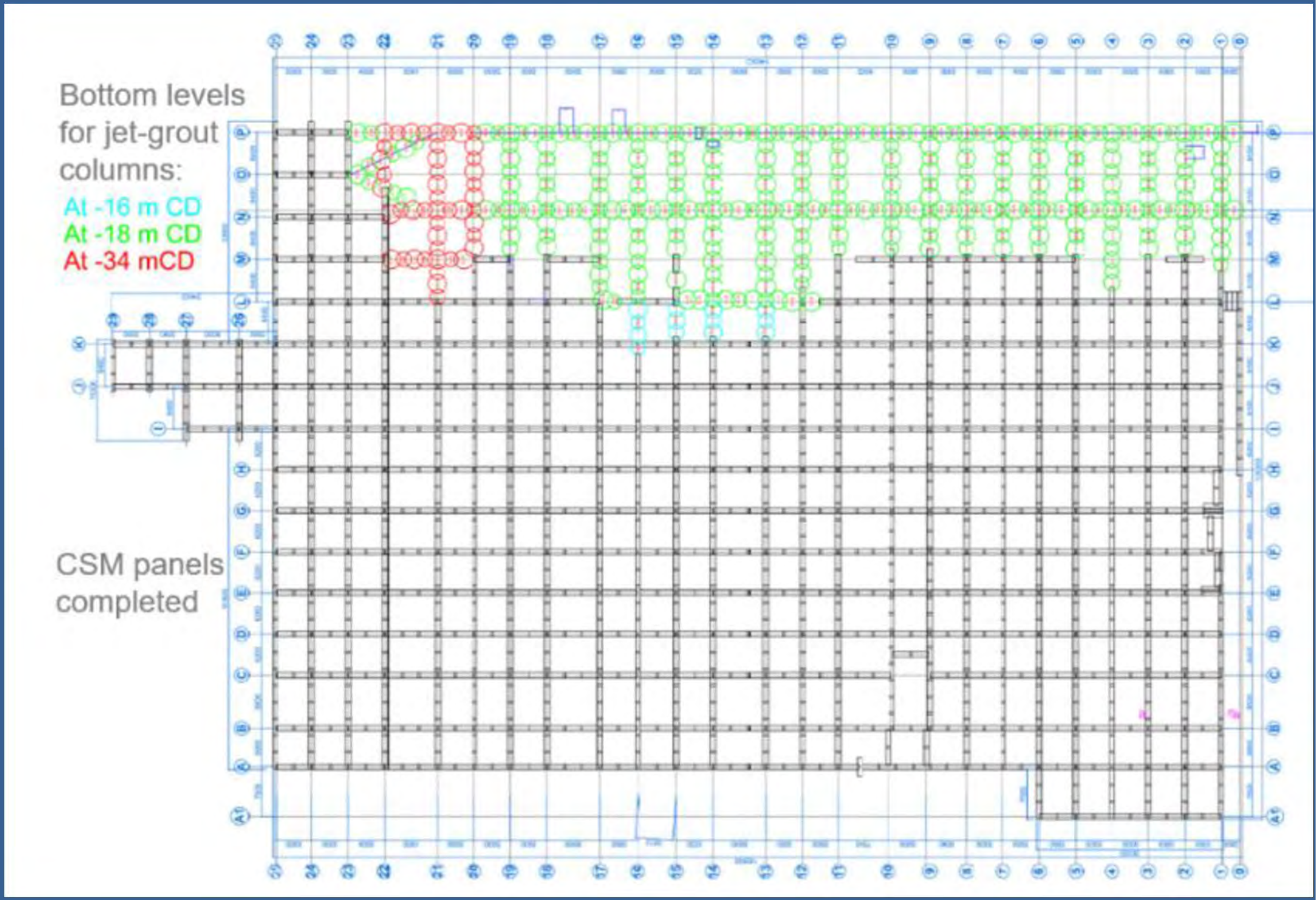
SOIL IMPROVEMENT TECHNIQUES

Difficulty due presence of blocks on east side : CSM panels replaced by Secant jet grouted columns

Same purpose

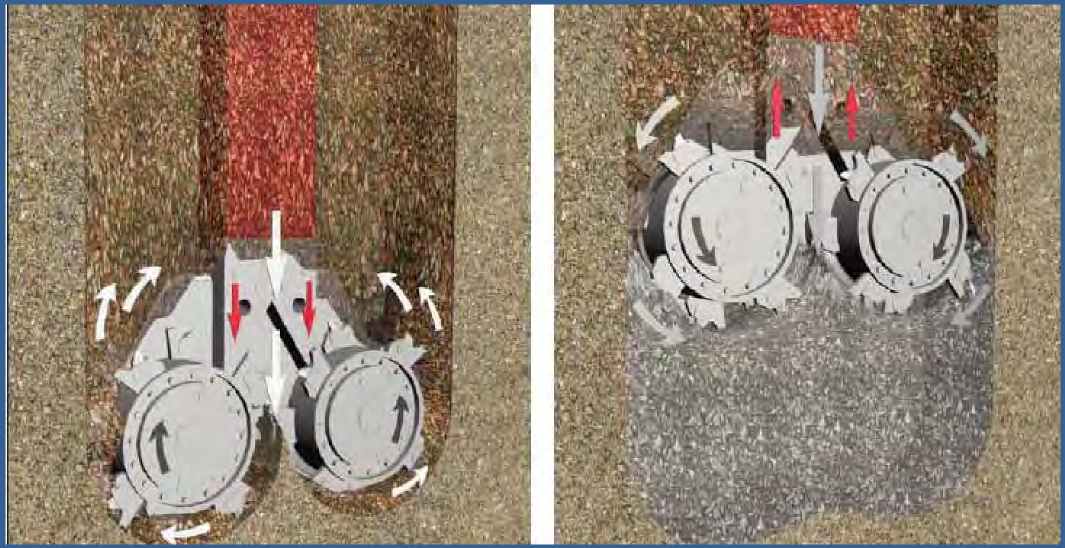
Slope stabilisation

Liquefaction mitigation



FOCUS ON LATTICE TYPE DSM TECHNIQUES

Geomix technique



Trenchmix technique

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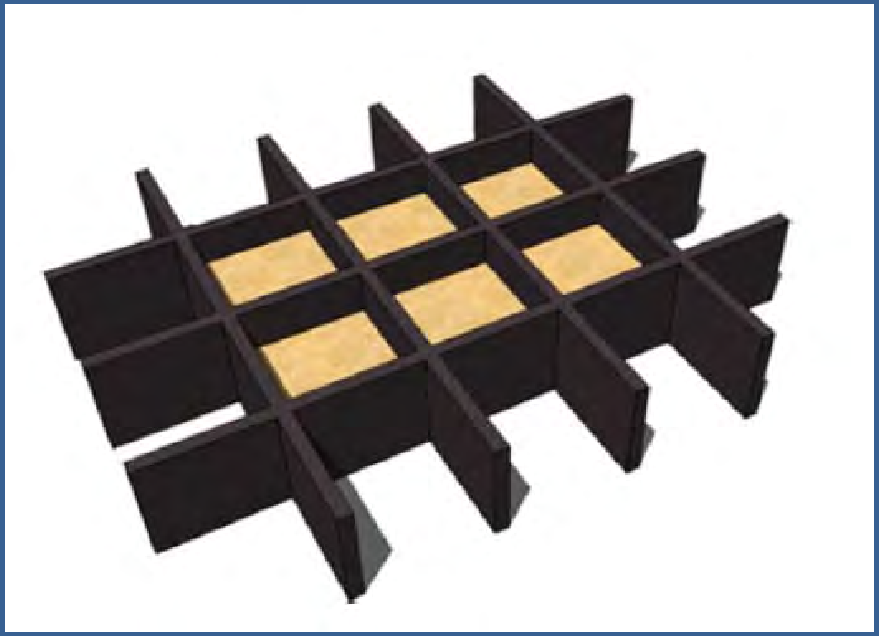
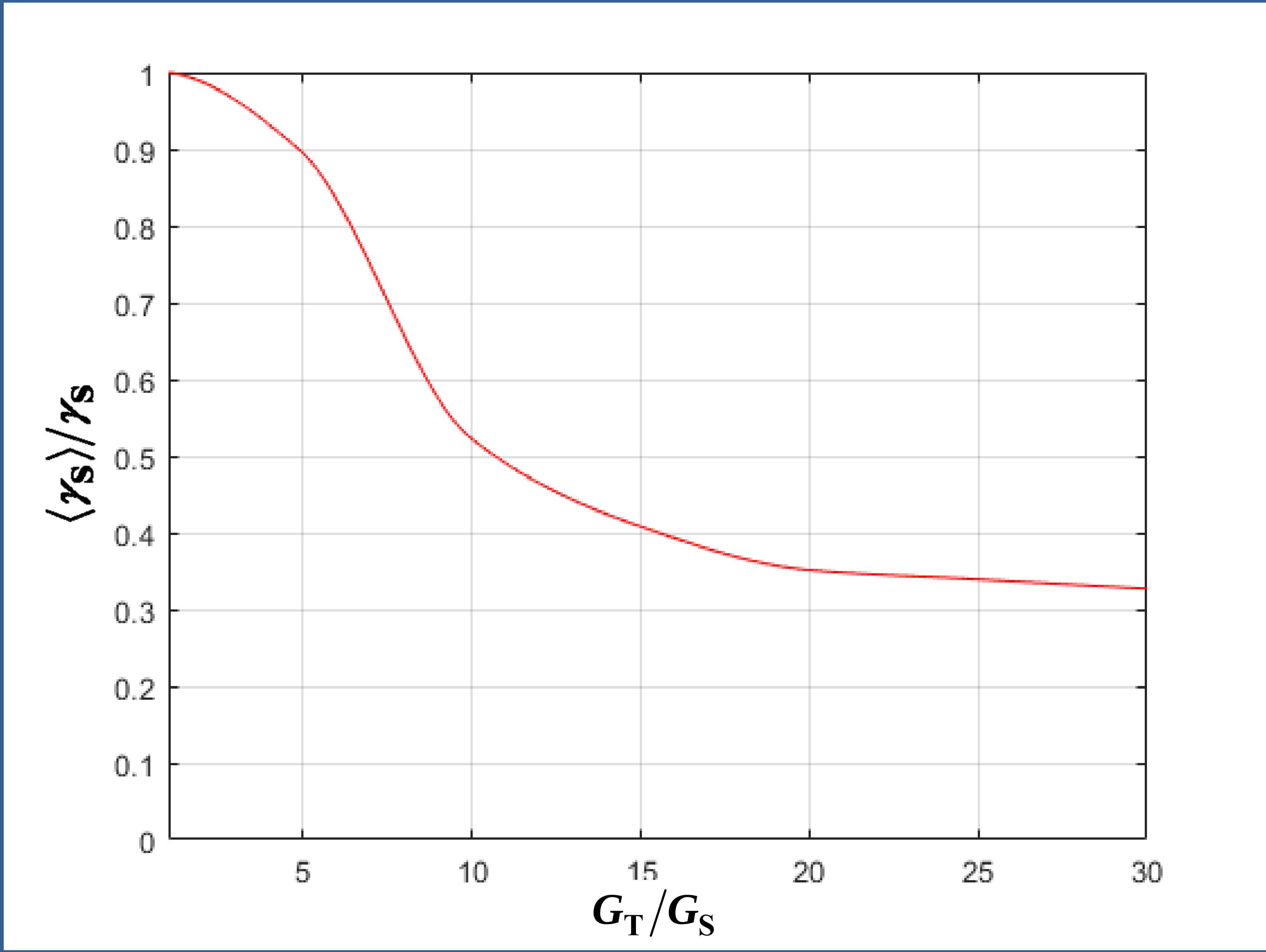
SEISMIC BEHAVIOUR OF DSM GRIDS

- Increases the lateral confinement of improved soils
- Limits the shear strains in the confined soil
- Provides resistance against lateral and vertical deformation
- Prevents pore water pressure migration from adjacent liquefied zones

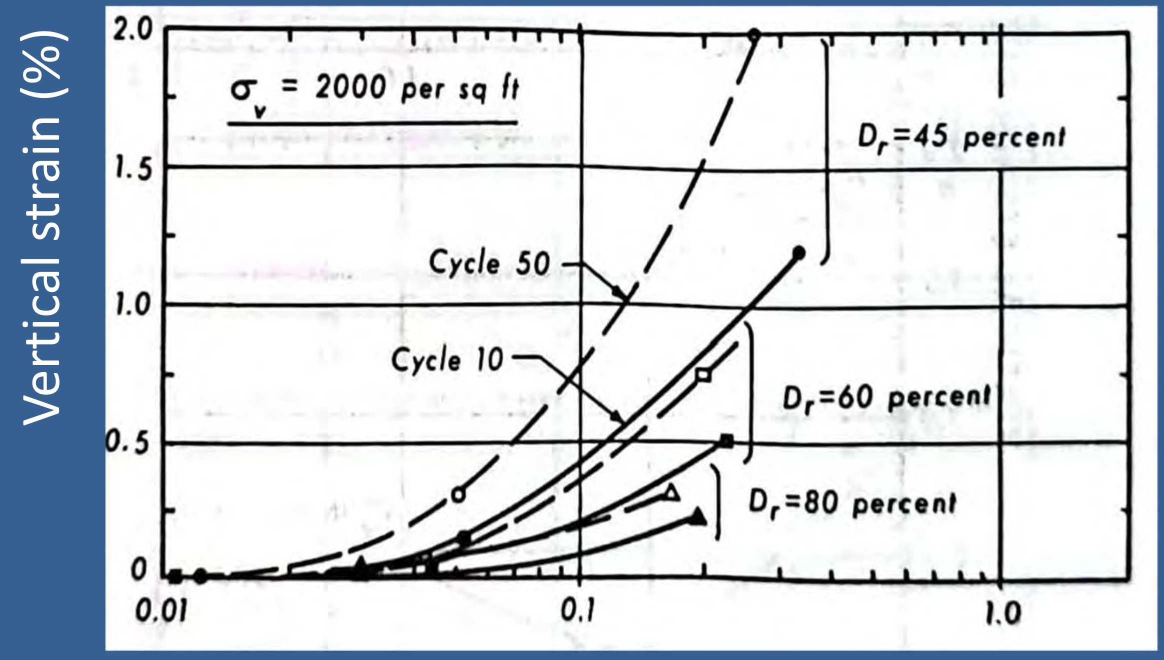


REDUCTION OF FREEFIELD SHEAR STRAIN

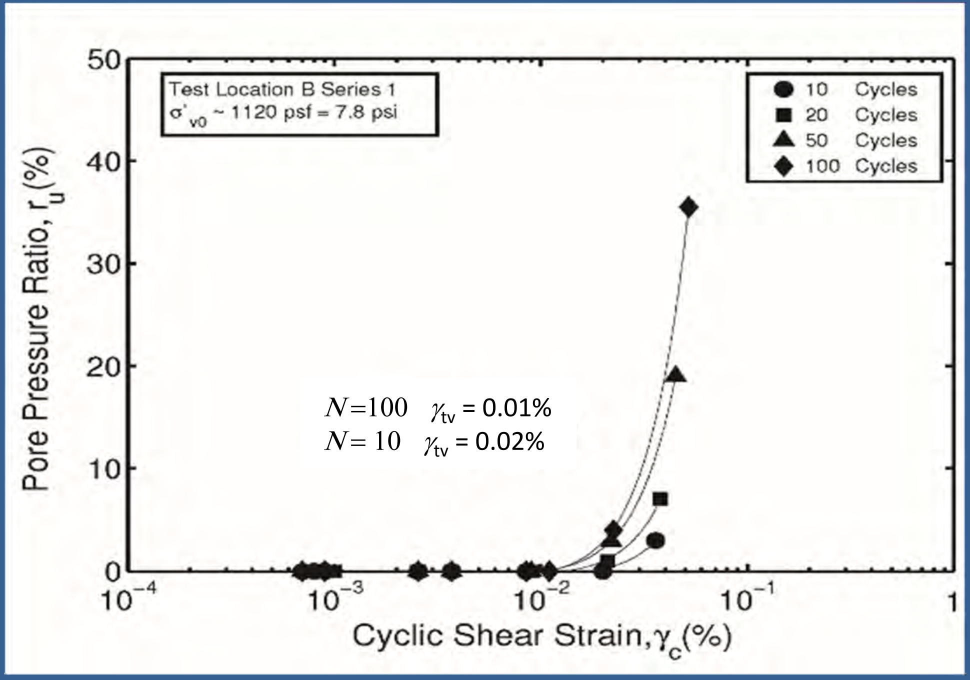


CYCLIC SHEAR STRAIN vs VOLUMETRIC STRAIN/PORE PRESSURE

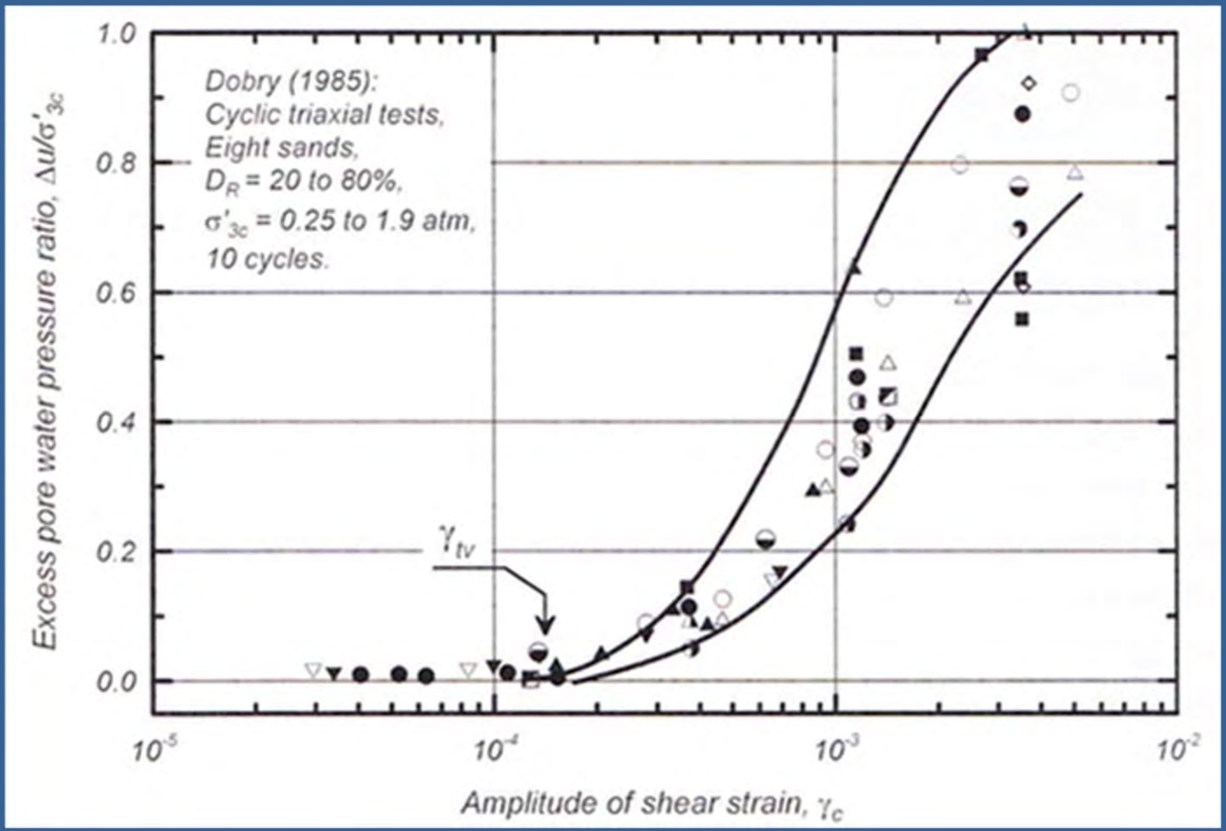
After Silver-Seed (1971)



After Dobry-Abdoun (2015)



BEHAVIOUR OF LATTICE TYPE DEEP SOIL MIXING



Increases the stiffness of the soil layer

⇒Decreases the cyclic shear strain

⇒ Decreases pore pressure build up

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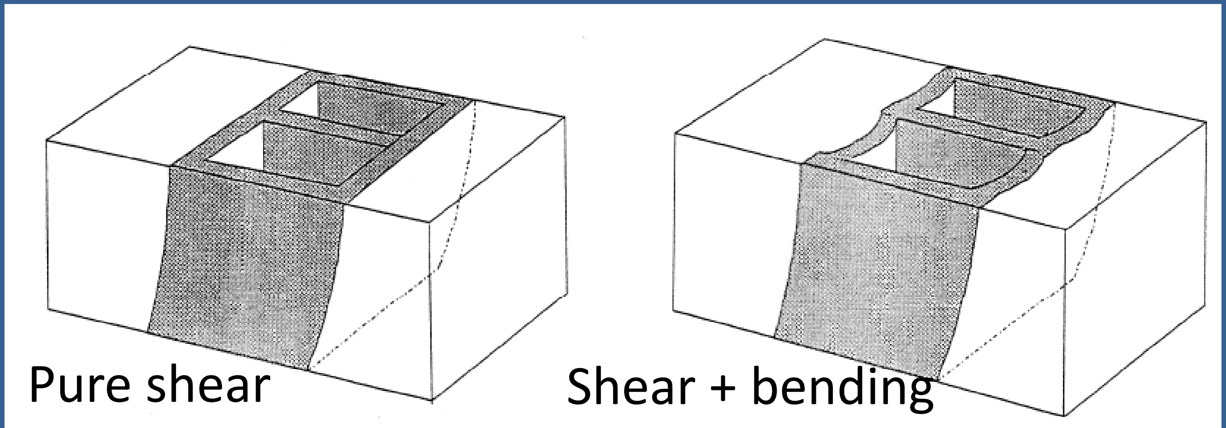
COMMON ASSUMPTIONS

- Soils improved with DSM grids experience pure shear deformation
- Shear strains in the confined soil and in the soil mixing trenches are equal : strain compatibility assumption

However

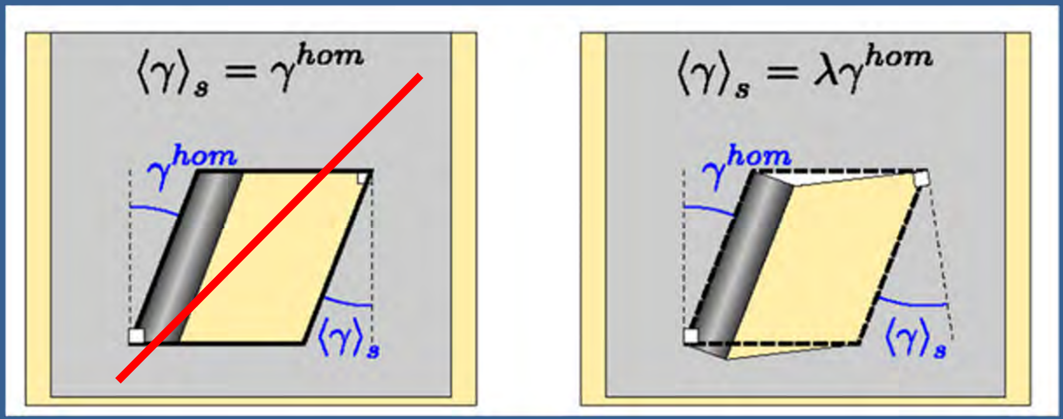
- Bending flexure of the trenches plays a major role in the response (O'Rourke et Goh, 1997)



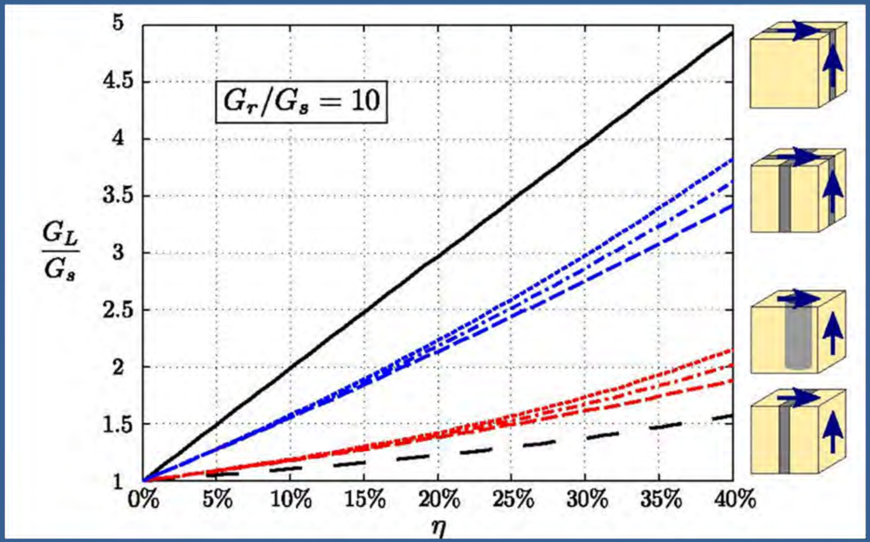
DSM GRIDS : SATE OF THE ART

- Based on soil elastic behaviour and stress-based approach
  - Nguyen et al (2013), Gueguin et al (2013)
- Findings

No strain compatibility



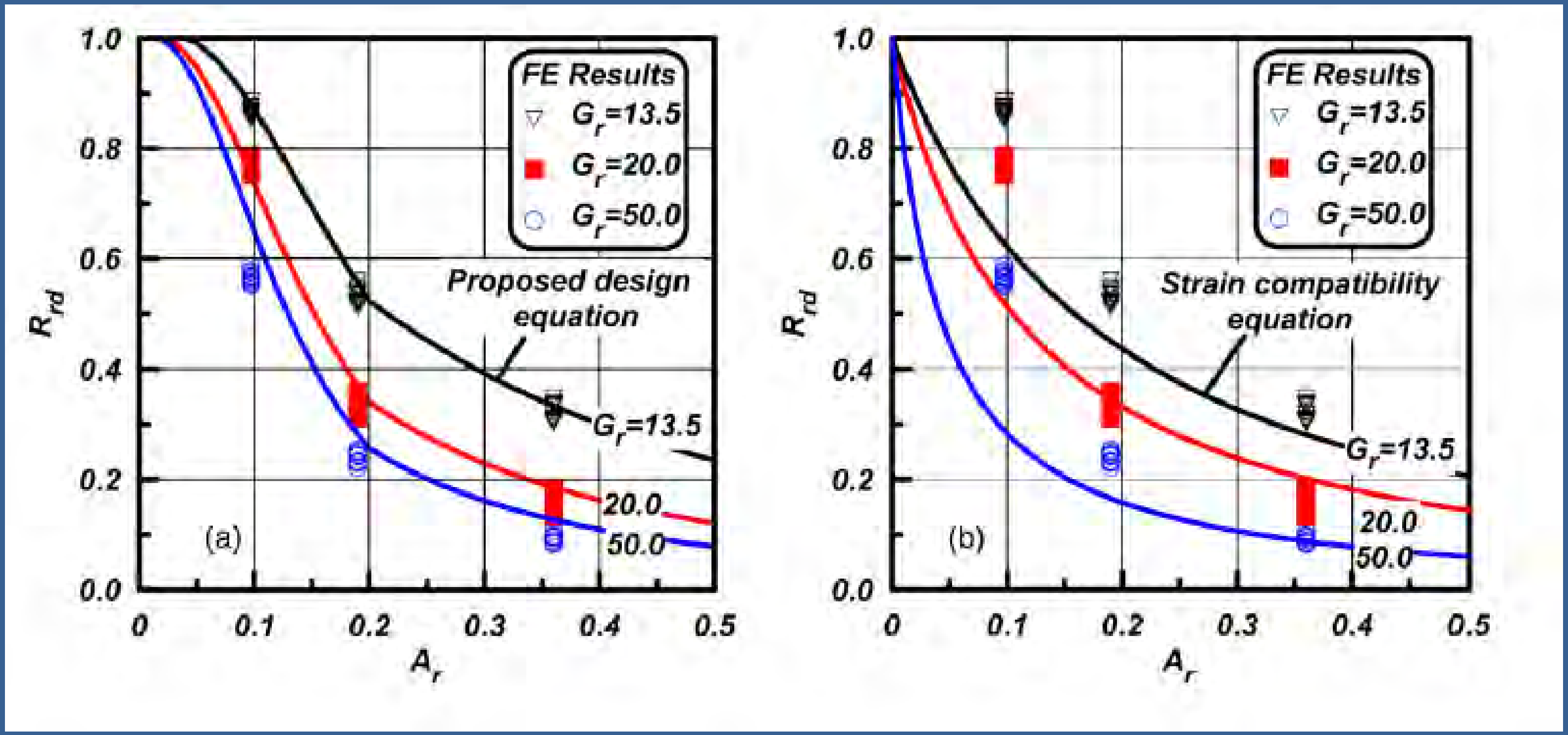
Stiffening effect
$\Rightarrow$ decrease in volumetric shear strain



NGUYEN's PROPOSALS

$$R_{\mathrm{rd}} = \frac{\tau_{\mathrm{improved}}}{\tau_{\mathrm{nonimproved}}} = \min\left(\frac{1}{G_{\mathrm{r}}\left[A_{\mathrm{r}} C_{\mathrm{G}} \gamma_{\mathrm{r}} + \frac{1}{G_{\mathrm{r}}}(1 - A_{\mathrm{r}}) \right]}, 1 \right)$$

C_G , γ_r calibrated on FEM analyses



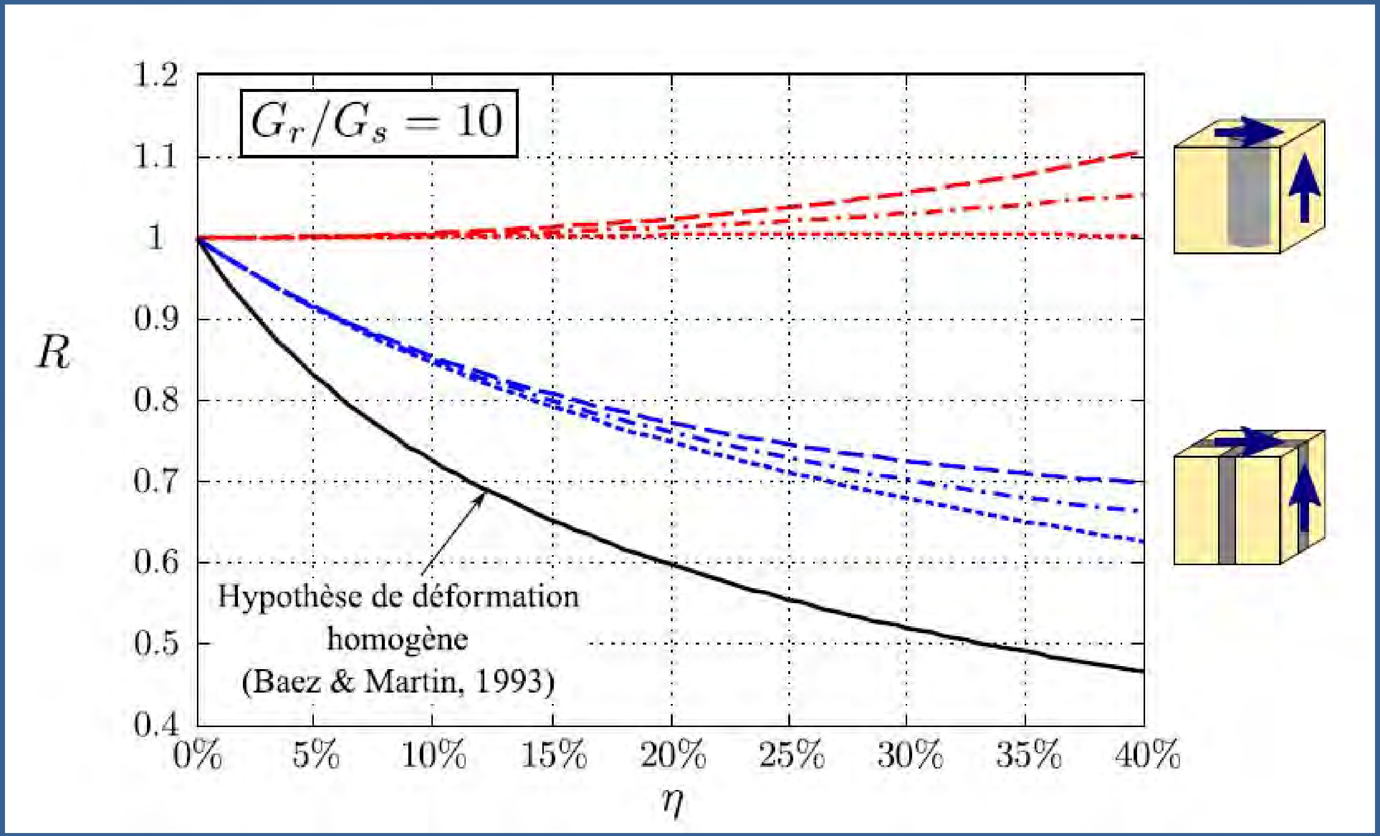
No strain compatibility (Nguyen et al)

Strain compatibility (Baez-Martin)

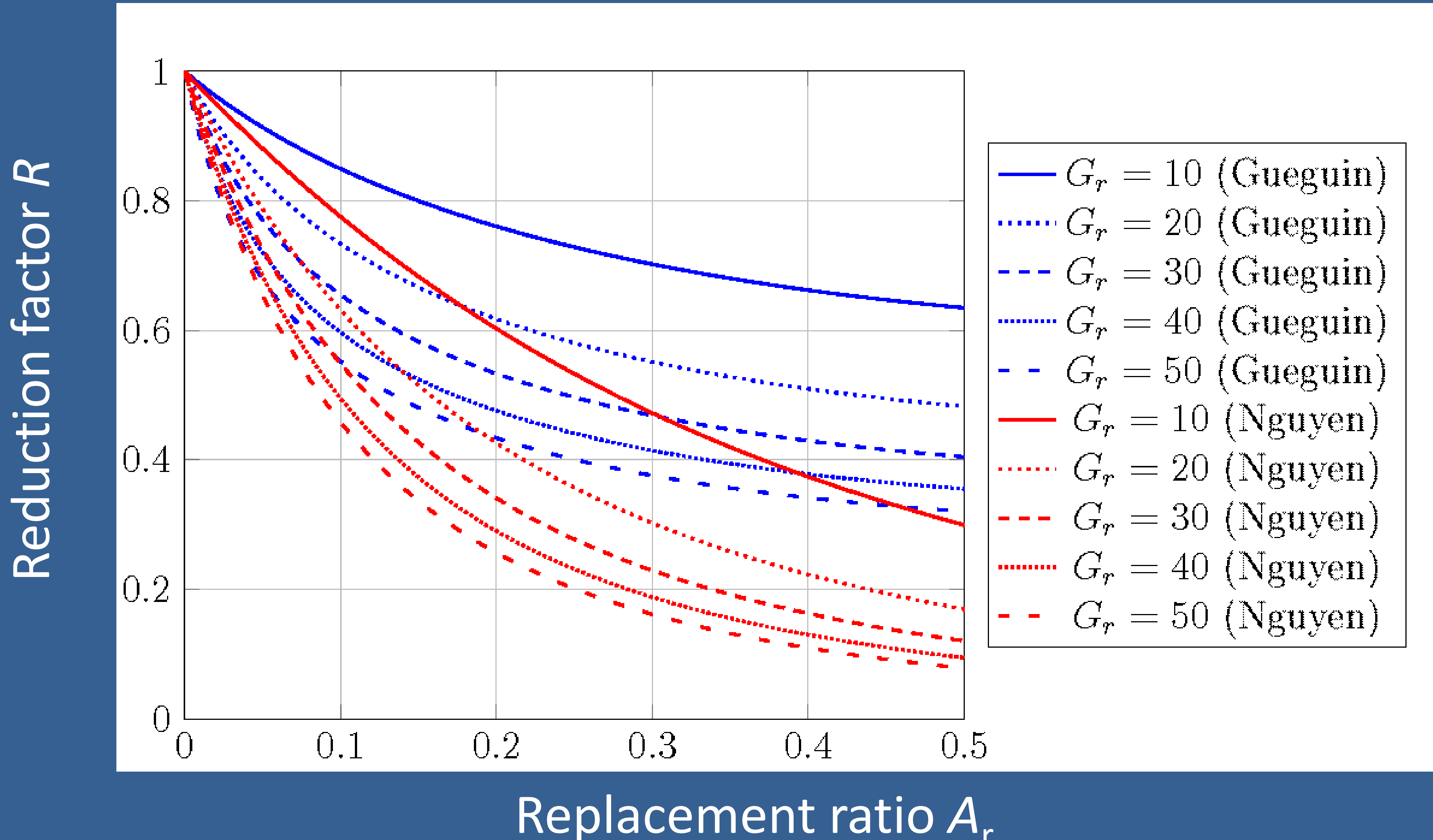
GUEGUIN PROPOSALS

$$R = \frac{\langle \gamma_S \rangle}{\gamma_S} = \lambda \sqrt{\frac{G_S}{G_L}}$$

λ *Localization factor for shear strain*



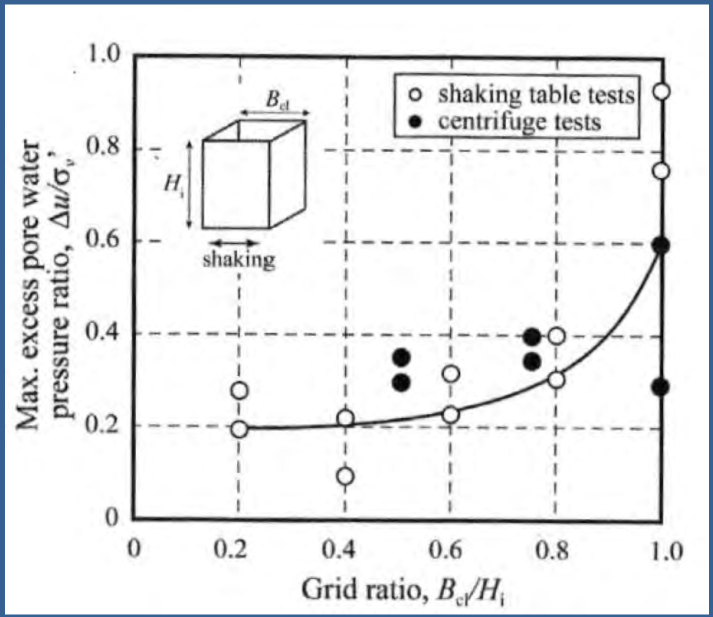
COMPARISON

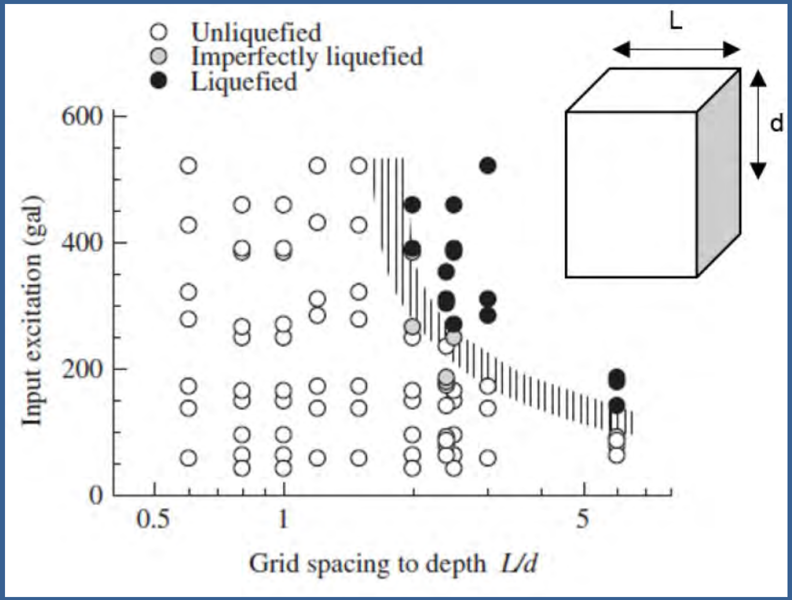


Reduction factor R
1
0.8
0.6
0.4
0.2
0
0
0.1
0.2
0.3
0.4
0.5
G_r = 10 (Gueguin)
G_r = 20 (Gueguin)
G_r = 30 (Gueguin)
G_r = 40 (Gueguin)
G_r = 50 (Gueguin)
G_r = 10 (Nguyen)
G_r = 20 (Nguyen)
G_r = 30 (Nguyen)
G_r = 40 (Nguyen)
G_r = 50 (Nguyen)
Replacement ratio A_r

EXISTING GUIDELINES

- North America : FHWA (Federal Highway Administration)
  1. Based on database of excess pwp, normalized by freefield excess pwp for various configurations
  2. Calculate the freefield excess pwp
  3. Multiply 2 by 1
- Japan : Kitazume-Terashi





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RESEARCH PROGRAM

- Objectives : establish guidelines for the preliminary design of soil improvement with DSM grids
- Funding : Fédération Nationale des Travaux Publics (FNTP)

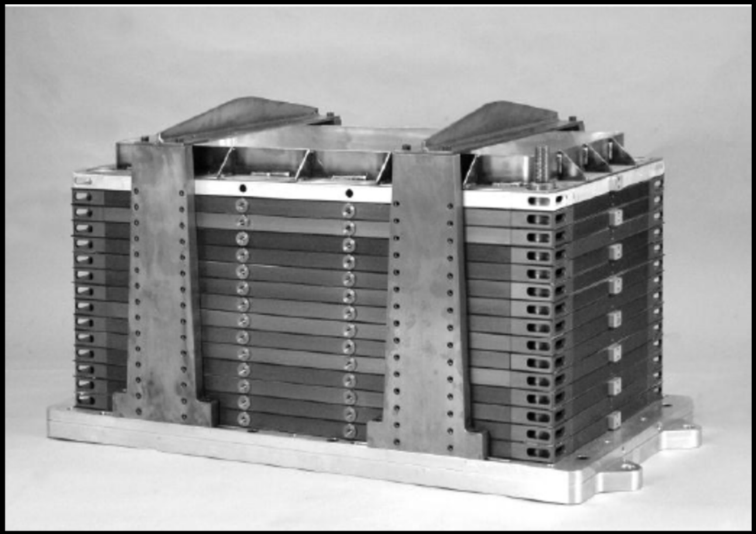
  Soletanche-Bachy
- Means : Combination of:

  Model tests in centrifuge facility (Institut Gustave Eiffel)

  Nonlinear dynamic numerical analyses

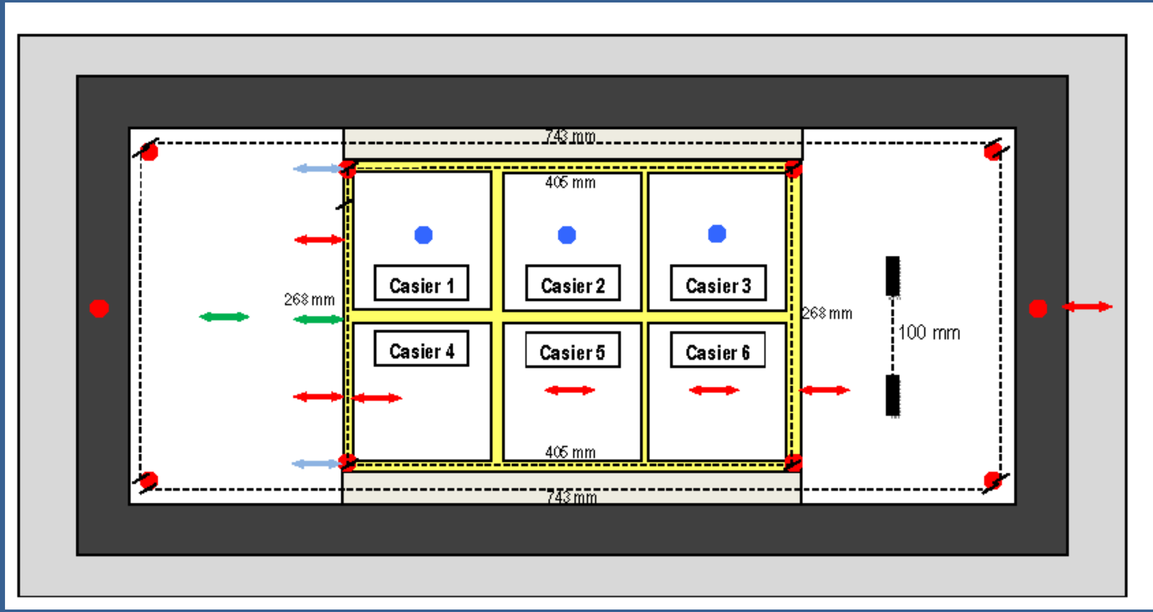
Shaker

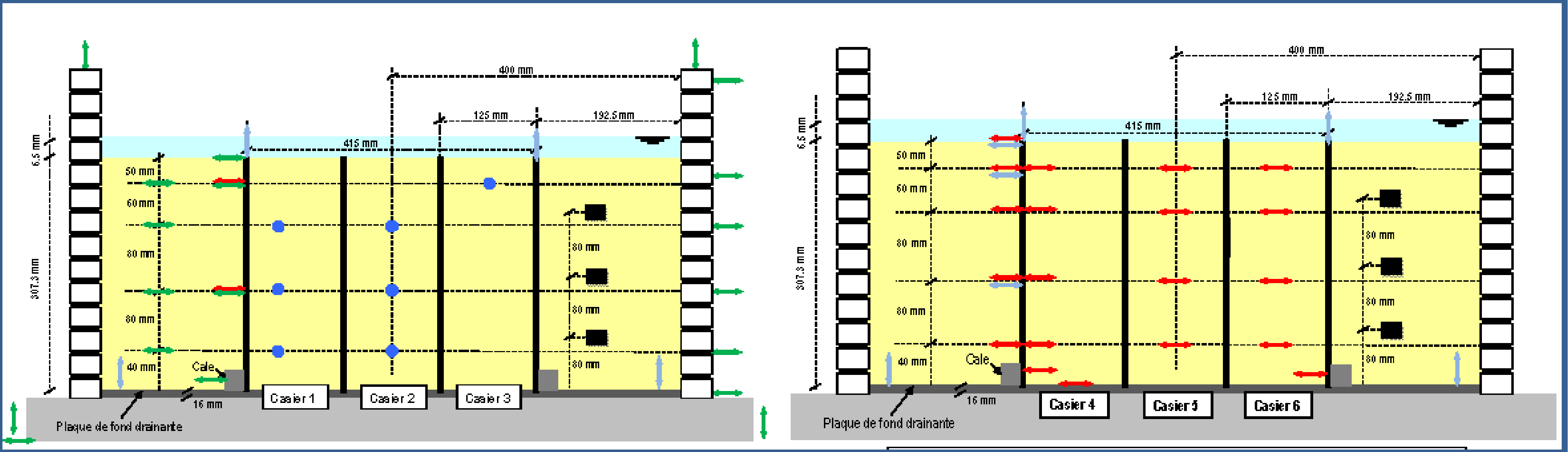
ESB box



Gustave Eiffel (ex- IFFSTAR)
Centrifuge facility

EXPERIMENTAL SET-UP





TESTED CONFIGURATIONS

- Soil column (freefield) and improved soil (DSM grid)
- DSM grid made of plastic resin (G = 1 to 2 MPa)
- Soil : Fontainebleau sand at D_R = 57%
- 32g and 48g : allows to test 2 grid spacings (4m x 4m and 6m x 6m) with the same “structural” model
- 10 cycles of sinusoidal loading (pga = 0.05g, 0.01g, 0.015g, …to 0.30g)
- 2 real accelerograms: Landers and Northridge scaled to pga= 0.05g and 0.2g

CONCLUSIONS FROM CENTRIFUGE TESTS

- Several experimental difficulties prevent a thorough interpretation of the tests and a definitive validation of the improvement technique:
  - ➢The ESB container creates boundary effect on the outside unimproved soil, which in turn induce motions of the DSM grid
  - ➢Unperfect fixity of the DSM grid at the bottom creates rotational motions of the DSM cells, which would not happen in reality due to the lower aspect ratio of real DSM grids
  - ➢Smooth contact between the DSM and the soil is different from the actual rough contact
  - ➢…….
- Nevertheless, some tests could be used to calibrate the numerical models

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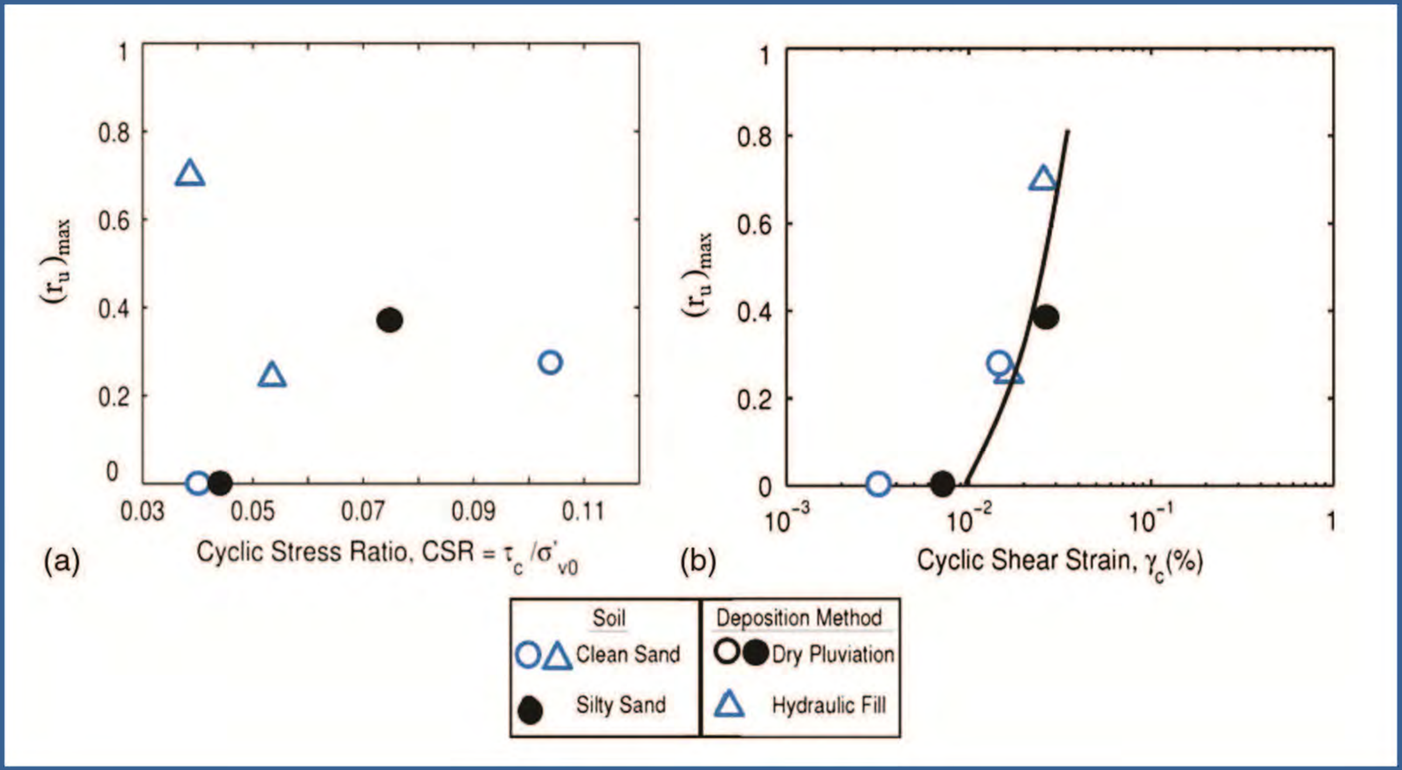
EVALUATION OF SOIL LIQUEFACTION

- Stress based approach (NCEER, Idriss-Boulanger)
  - ➢Definition of seismic demand CSR
  - ➢Evaluation of seismic capacity CRR
  - ➢Calculation of safety factor CRR/CSR

- Strain based approach ⇐ Followed in this presentation
  - ➢Definition of seismic demand : induced cyclic shear strain $<\gamma_s>$
  - ➢Evaluation of cyclic shear strain triggering liquefaction γ_{cl}

LIQUEFACTION ASSESSMENT
Back to the fundamentals

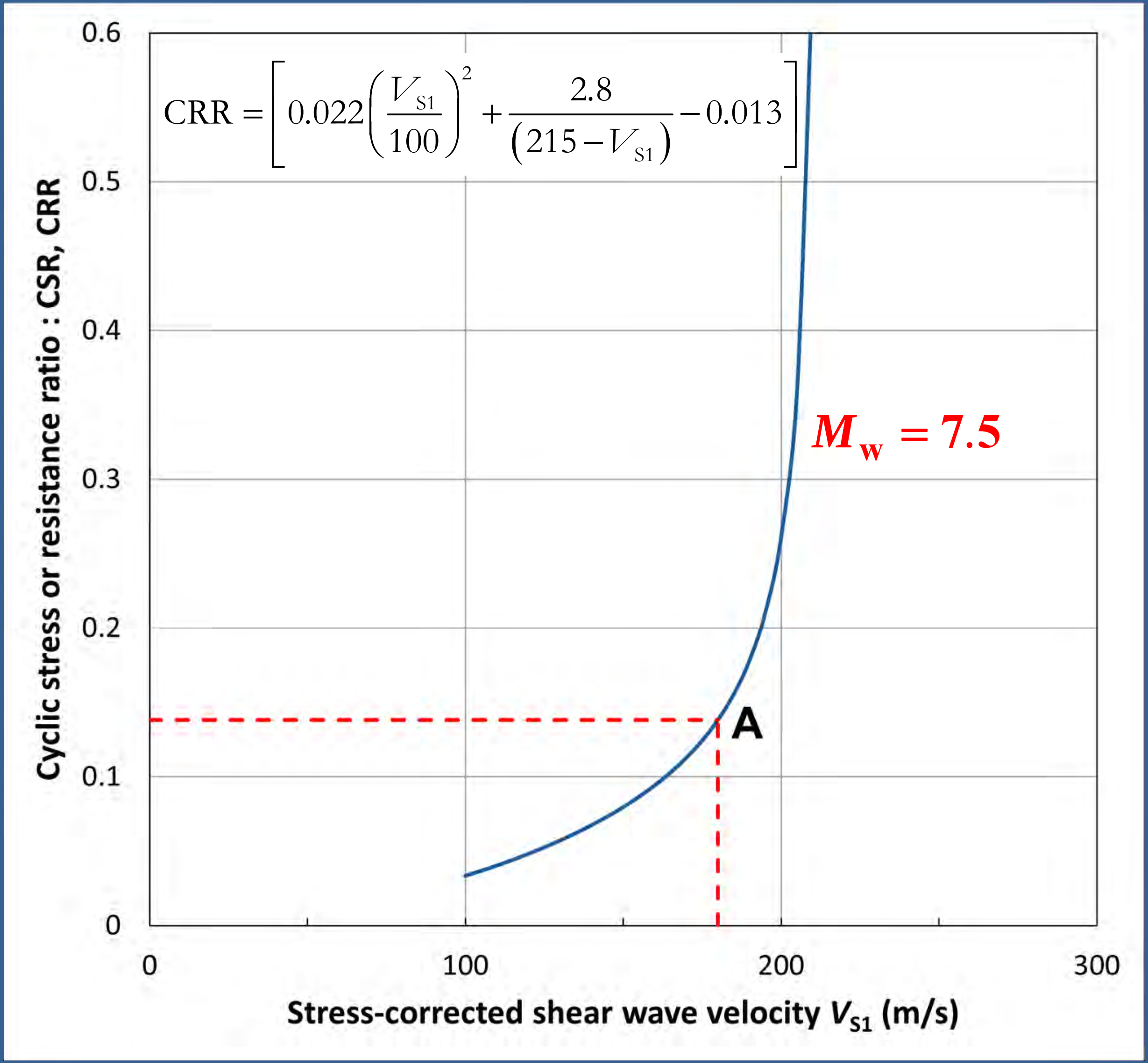
- Liquefaction is a strain governed phenomenon



"Results of undrained cyclic strain-controlled tests in the laboratory …show practical advantages of using γ instead of CSR to characterize the pore pressure response of sands to cyclic shear loading"
Dobry-Abdoun, 2015

LIQUEFACTION TRIGGERING STRAIN γ_{cl}

after Dobry-Abdoun (2015)



$$CSR = \tau / \sigma'_{v0} = G\gamma_c / \sigma'_{v0}$$

$$= \gamma_c G_{max} (G/G_{max})_{|\gamma_c} / \sigma'_{v0}$$

$$= \gamma_c \rho V_{S1}^2 (G/G_{max})_{|\gamma_c} / \sqrt{p_a \sigma'_{v0}}$$

CRR

$$= \gamma_{cl} \rho V_{S1}^2 (G/G_{max})_{|\gamma_{cl}} / \sqrt{p_a \sigma'_{v0}}$$

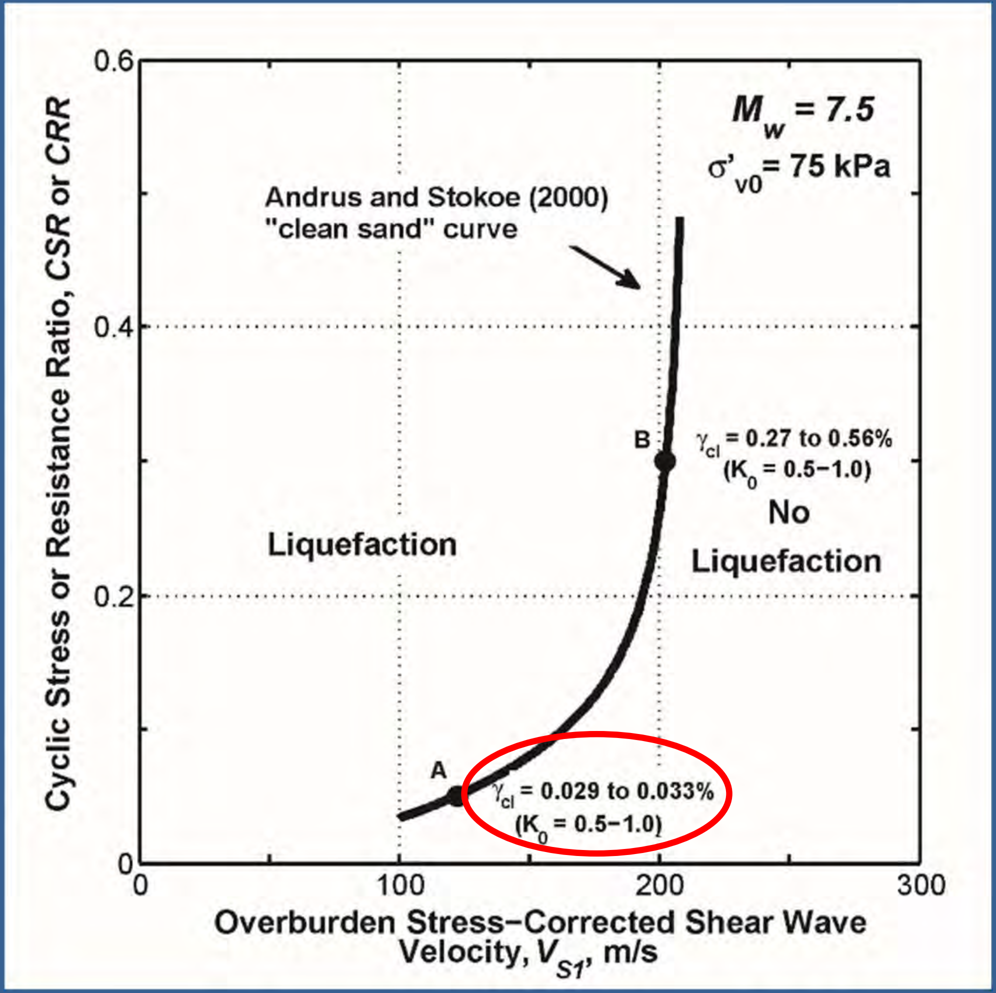
$$G/G_{max} = 1/(1 + \gamma/\gamma_r)^{\alpha}$$

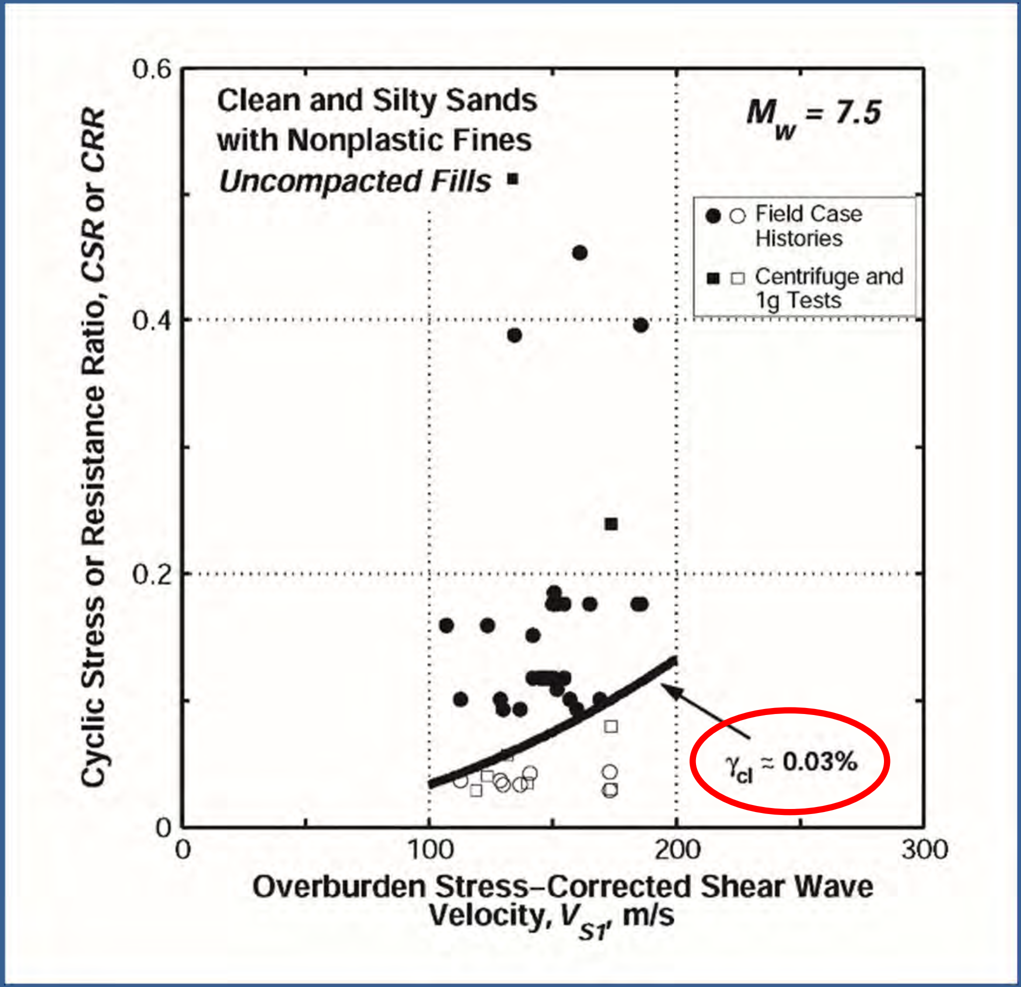
e.g. Darendeli (2001)

Iterative solution → γ_{cl}

SEISMIC CAPACITY for $M_w = 7.5$

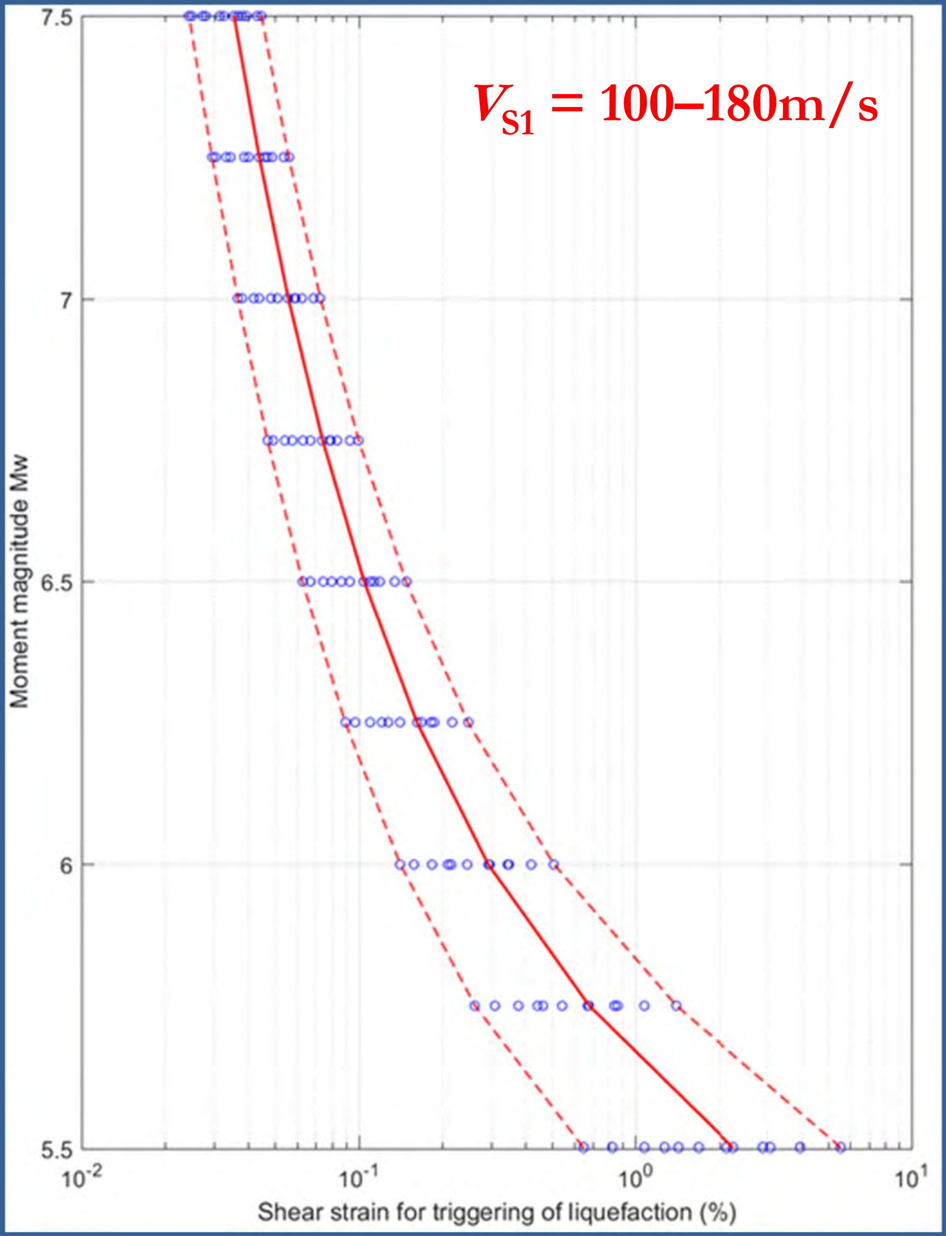
after Dobry-Abdoun (2015)





$K_0 = 0.5\text{–}1.0$, $\sigma'_{v0} = 50\text{–}100\text{kPa}$, $V_S = 100\text{–}180\text{m/s}$

DEFINITION OF LIQUEFACTION TRIGGERING STRAIN



For loose recent sand deposits this strain is constant $\gamma_{cl} = 3.0\ 10^{-4}$ for $M_w = 7.5$ (Dobry-Abdoun, 2015)

Dobry's procedure extended to other M_w

$$\gamma_{cl} = \frac{0.113 M_w^2 - 1.596 M_w + 5.711}{M_w - 5.343}$$

γ_{cl} replaces CRR

SEISMIC DEMAND

- Dynamic analyses of one cell (periodic scheme)
  - 3D elastoplastic soil behaviour (Prevost's model)
- Validation of the numerical model vs tests (centrifuge experiments)
- Extensive nonlinear FE analyses with 30 recorded time histories with $6.0 \leq M_w \leq 7.0$
- Development of a relationship between average induced strain in the cell, freefield strain, DSM characteristics....

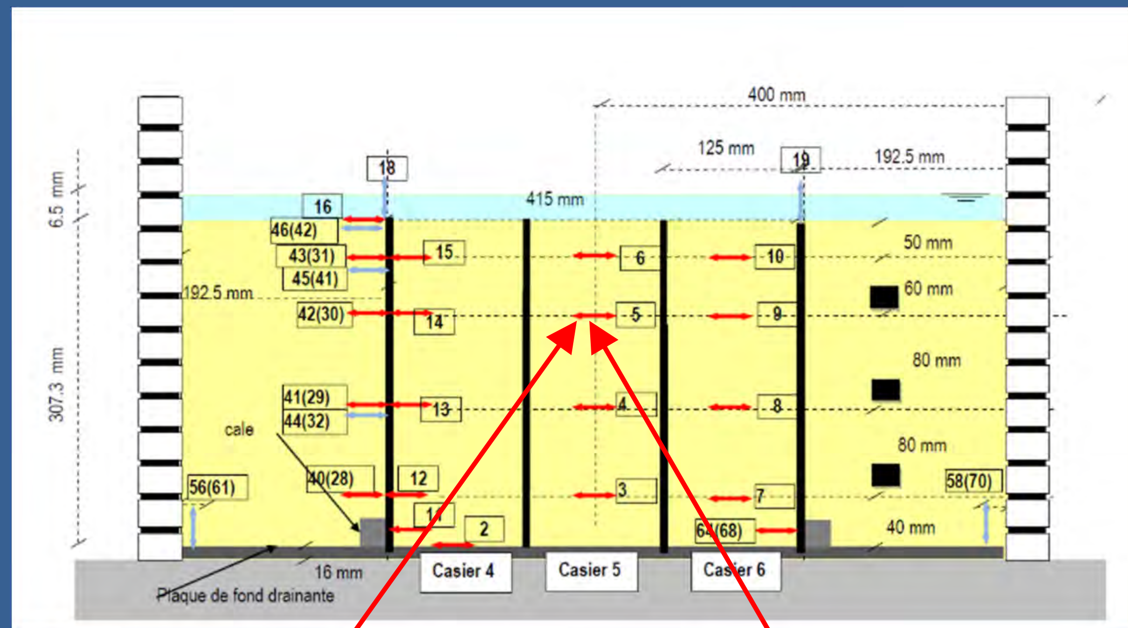
$$< \gamma_S > = \Phi\left(\gamma_S, G, G_T, B, e, \eta, pga...\right)$$

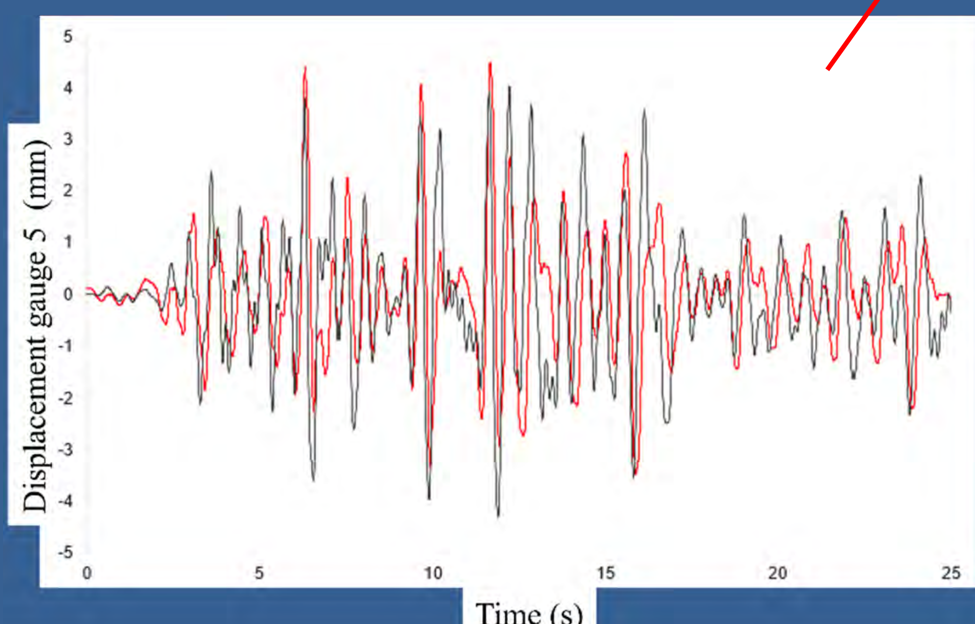
SOIL CHARACTERISTICS : Prevost's model

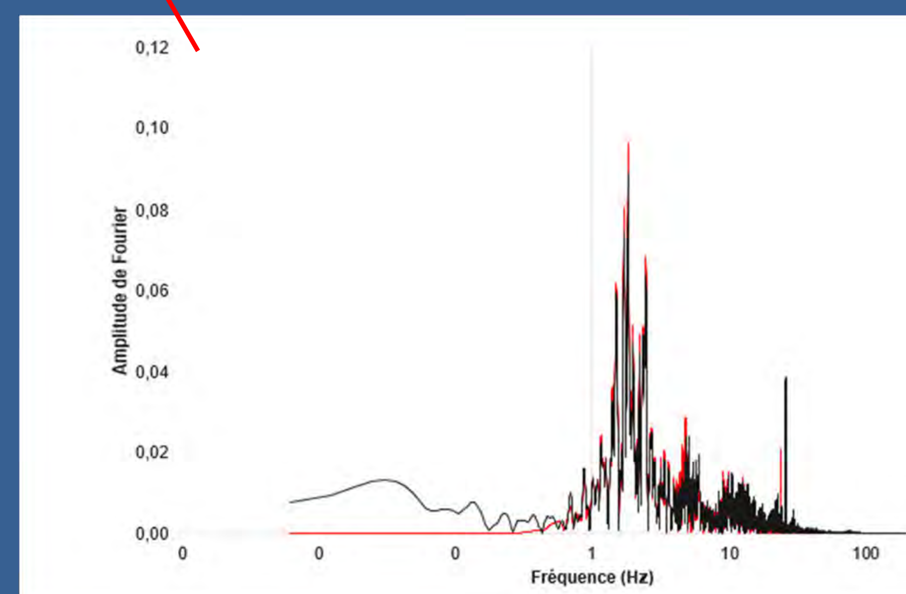
Parameter	Symbol	Values		Type
		Backfill	Sand	
Mass density (t/m³)	ρ_S	1.9	2.0	State parameter
Shear modulus (MPa)	G_1	75, 90, 110	75, 90, 110	Elastic parameters
Bulk modulus (MPa)	B_1	140, 165, 200	1500, 2200, 2700	
Reference stress (MPa)	p_a	0.10	0.10	
Exponent	n	0.5	0.5	
Friction angle (°)	ϕ	35	35	Plastic parameters
Cohesion (kPa)	C_u	5	5	
Definition of stress-strain curve	α, x_l and x_u	see Figure 5-3		Nonlinear parameters
Characteristic angle (°)	ψ	33	33	Dilation parameters
Dilation parameter	X_{PP}	0.05	0.10	

Note : $G_S = G_1\left(\sigma'_m / p_a\right)^n \quad B_S = B_1\left(\sigma'_m / p_a\right)^n$

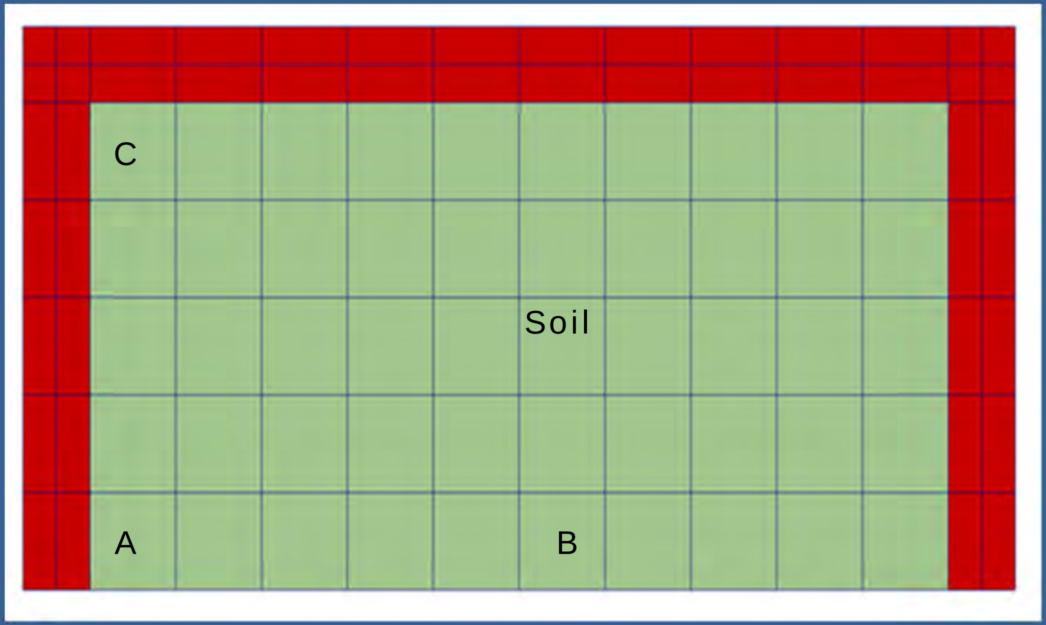
NUMERICAL ANALYSES vs EXPERIMENTS



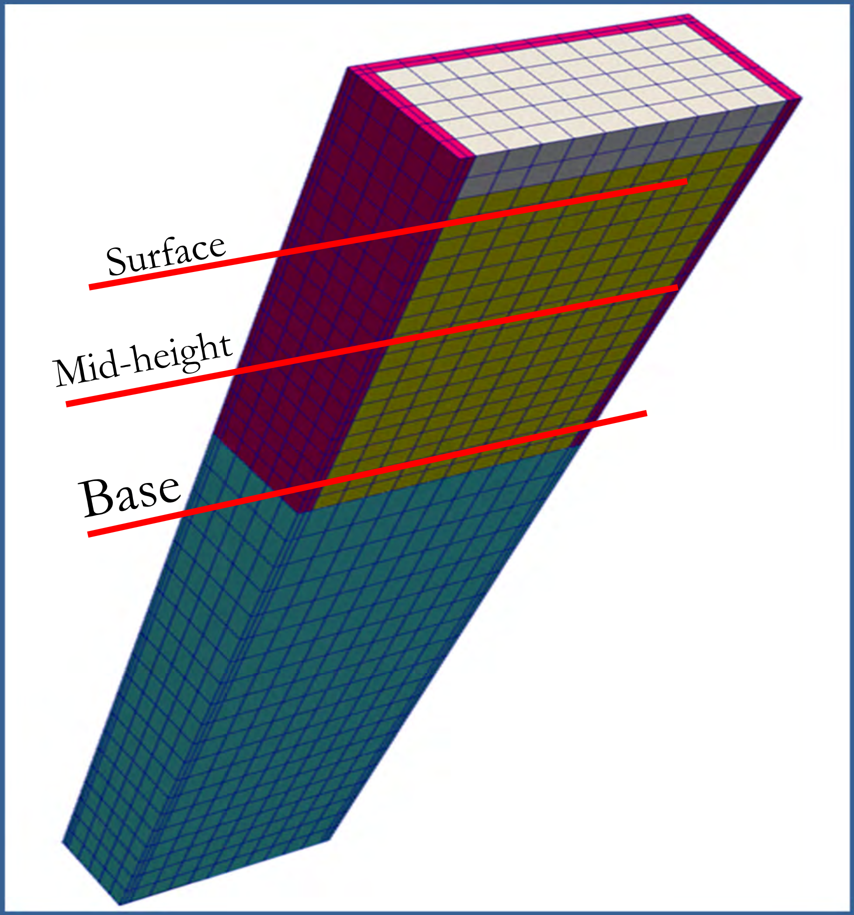




FINITE ELEMENT MODEL



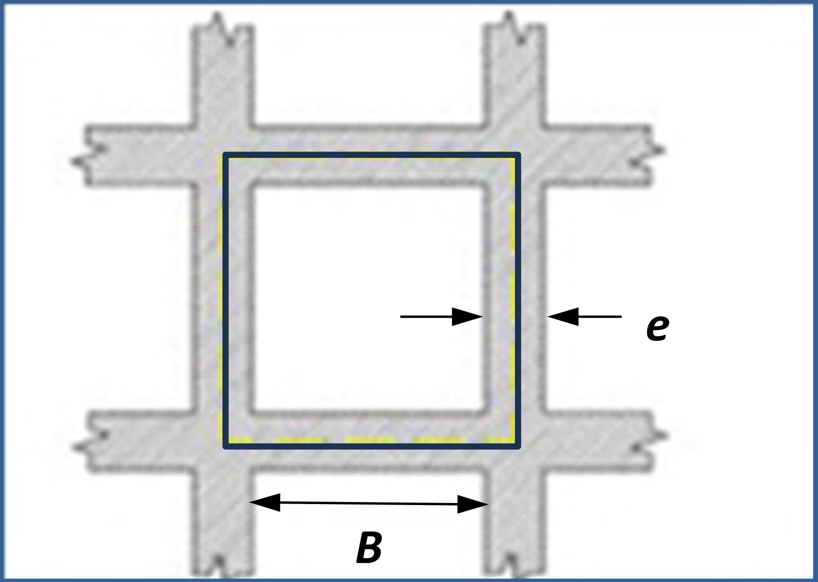
Top view



3D perspective

VARIABLES INVOLVED in MODEL

SUBSYSTEM	PHYSICAL QUANTITIES	SYMBOL
Soil	Soil shear modulus	G_1
	Mass density	ρ_S
	Height of soil column	H_S
	Maximum freefield cyclic shear strain	γ_S
Trenches	Shear modulus	G_T
	Mass density	$\rho_T \sim \rho_S$
	Height of treatment	H_T
	Horizontal spacing	B
	Trench thickness	e
	Maximum average cyclic shear strain within a cell	$\langle\gamma_S\rangle$
Ground motion	Freefield peak ground surface acceleration	pga
	Magnitude	M_w



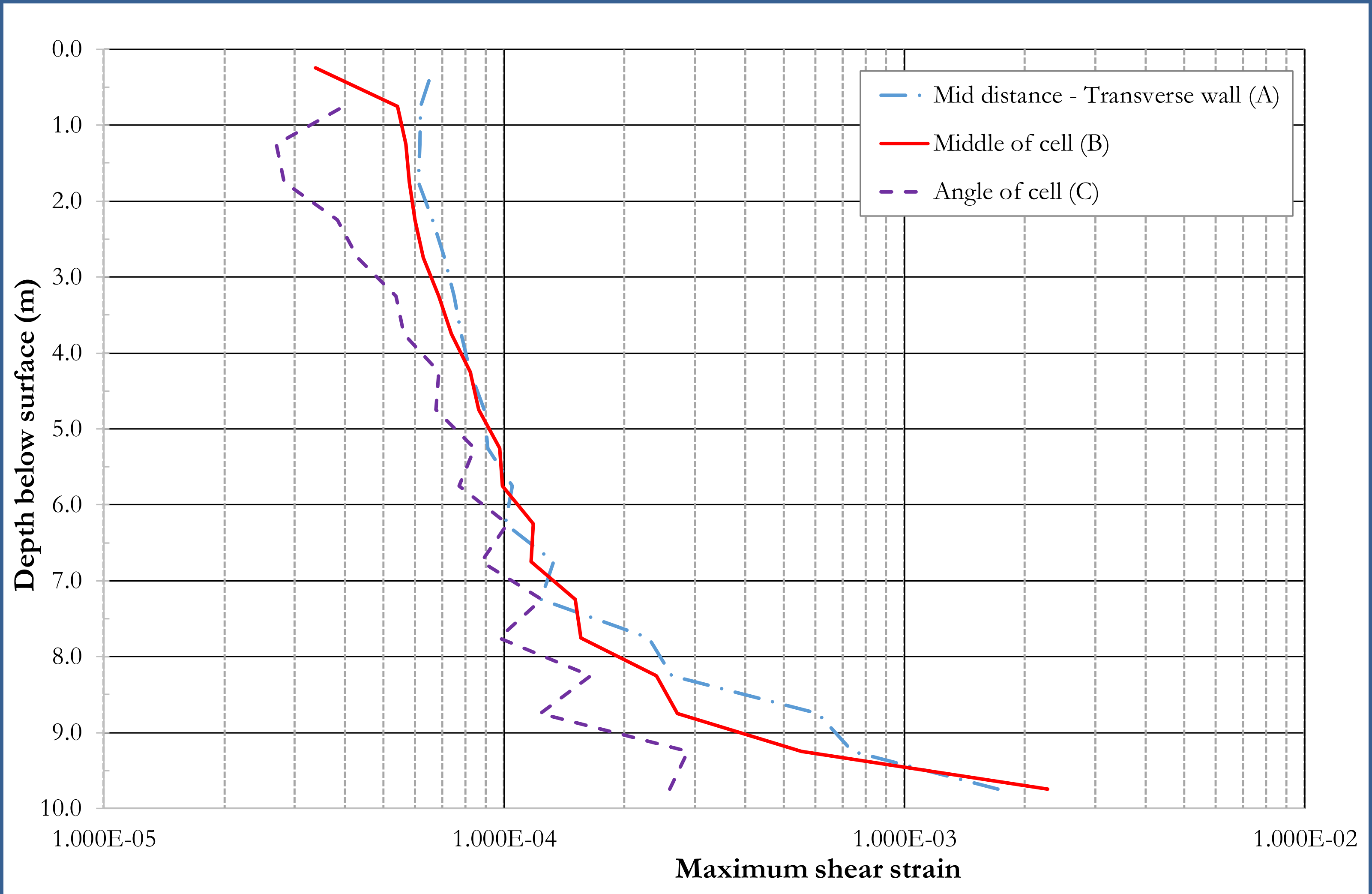
RANGE OF INVESTIGATED PARAMETERS

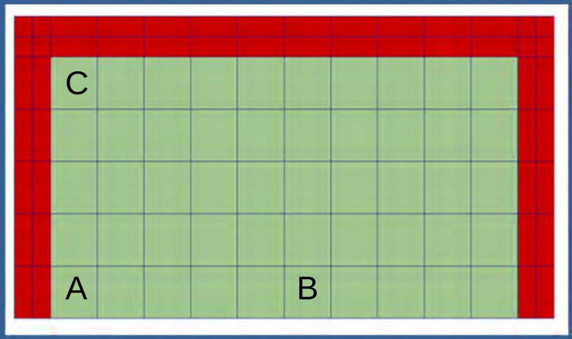
Subsystem	Physical quantity	Symbol	Range of values
Soil	Shear modulus (MPa)	G_1	75, 90, 110
	Mass density (t/m^3)	ρ_S	2.0
	Height of soil column (m)	H_S	30
	Maximum freefield cyclic shear strain	γ_S	calculated
DSM trenches	Shear modulus (MPa)	G_T	1125, 1700, 2000
	Mass density (t/m^3)	$\rho_T \sim \rho_S$	2.0
	Height of treatment (m)	H_T	10, 15, 20
	Horizontal spacing (m)	B	4, 6, 8
	Trench thickness (m)	e	0.4, 0.6, 0.8, 1.0
	Maximum average cyclic shear strain within a cell	$< \gamma_S >$	calculated
Ground motion	Freefield ground surface acceleration (m/s^2)	Pga	Calculated from outcrop acceleration at depth H_S
	Magnitude	M_W	6.0, 6.5, 7.0

DIMENSIONNAL ANALYSIS
Vaschy-Buckingham theorem

Dimensionless parameter	Expression	Physical meaning
Π_1	G_T / G_1	Relative stiffness of soil mixing to freefield soil
Π_2	$\eta = (2Be + e^2)/(B + e)^2$	Replacement ratio (in decimal)
Π_3	$(B + e)/H_T$	Aspect ratio
Π_4	M_w	Earthquake magnitude
Π_5	$V_S^2/(H_S . pga)$	Fundamental frequency of soil column
Π_6	H_T / H_S	Relative height of soil treatment to soil depth
Π_7	γ_S	Freefield cyclic shear strain
Π_8	$< \gamma_S >$	Average cyclic shear strain within a cell

VERTICAL PROFILES OF MAXIMUM CYCLIC SHEAR STRAIN





PLANE VIEW OF MAX SHEAR STRAIN (half a cell at depth *Z*)

1.00E-04	8.76E-05	8.85E-05	8.85E-05	8.93E-05	9.02E-05	9.09E-05	9.25E-05	9.26E-05	1.07E-04
1.04E-04	9.08E-05	9.01E-05	8.98E-05	9.16E-05	9.28E-05	9.52E-05	9.77E-05	1.02E-04	1.20E-04
9.80E-05	9.19E-05	9.16E-05	9.07E-05	9.42E-05	9.43E-05	9.89E-05	1.01E-04	1.08E-04	1.22E-04
9.40E-05	9.21E-05	9.27E-05	9.11E-05	9.64E-05	9.51E-05	1.02E-04	1.03E-04	1.11E-04	1.24E-04
9.10E-05	9.23E-05	9.29E-05	9.14E-05	9.75E-05	9.53E-05	1.03E-04	1.04E-04	1.13E-04	1.25E-04

DESIGN EQUATION

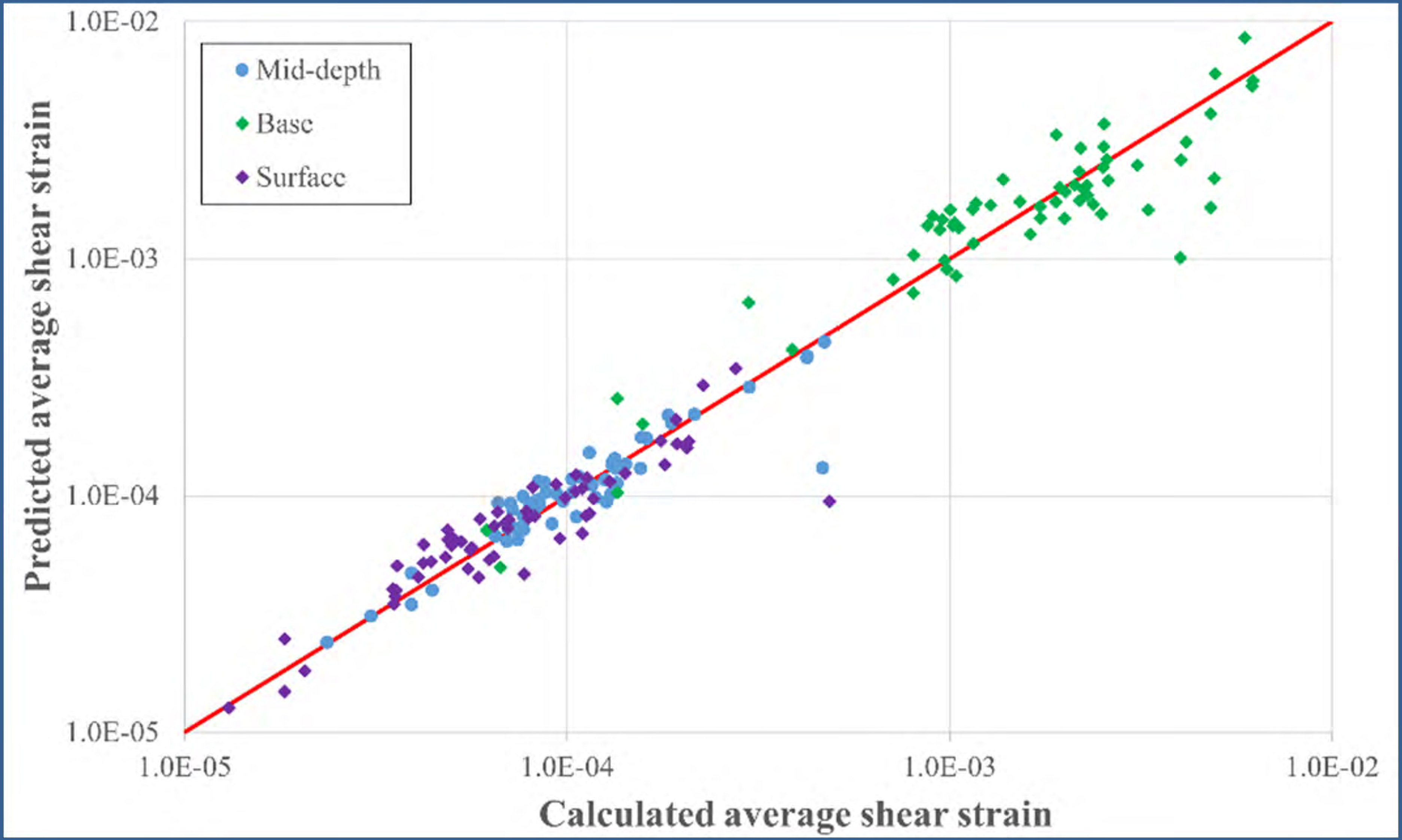
- Extensive 3D numerical analyses
  + Dimensional analyses

Average shear strain within a cell function of freefield shear strain

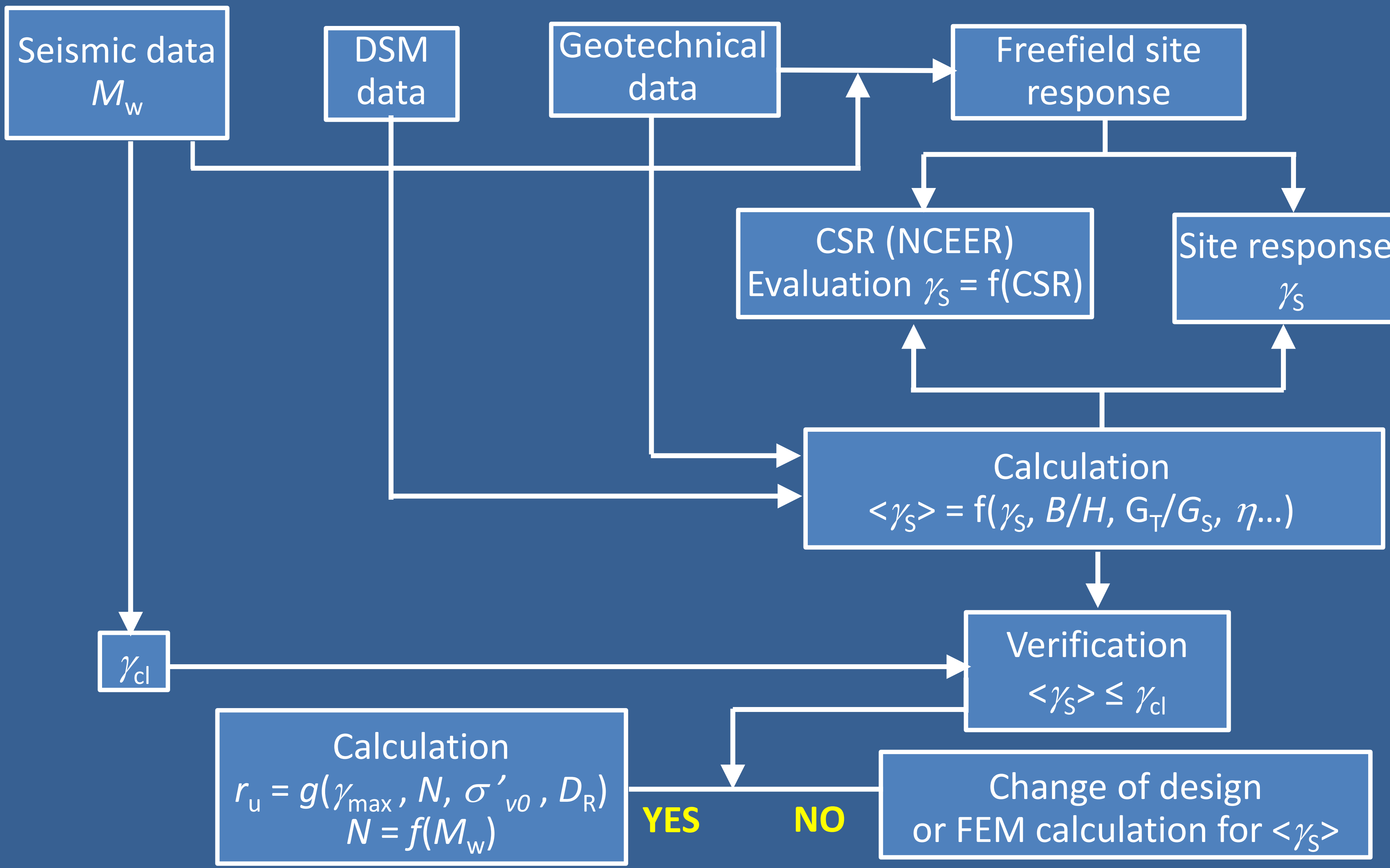
$$< \gamma_S > = \lambda \left(\frac{G_T}{G_1} \right)^{\theta_1} (1-\eta)^{\theta_2} \tanh^{\theta_3} \left(\frac{B+e}{H_T} \right) M_w^{\theta_4} \left(\frac{G_S}{\rho_S H_S \, pga} \right)^{\theta_5} \left(1 - \frac{H_T}{H_S} \right)^{\theta_6} \gamma_S^{\theta_7}$$

θ_1 to θ_7 numerical parameters determined from analyses, depth dependent

CALCULATIONS vs PREDICTIONS



DESIGN FLOWCHART



PORE PRESSURE MODEL

Cetin K. O., and Bilge H. T. (2012)

Dobry's model

$$\ln\left(r_{u,N}\right) = \ln\left\{\frac{p f N F\left(\gamma_{max,N} - \gamma_{tv}\right)^{s}}{1 + f N F\left(\gamma_{max,N} - \gamma_{tv}\right)^{s}}\right\}$$

$r_{u,N}$ excess pore-water pressure ratio in *N*th loading cycle

$\gamma_{max,N}$ maximum shear strain amplitude in *N*th loading cycle (%) = $<\gamma_S>$

N number of equivalent cycles at strain amplitude $\gamma_{max,N}$

γ_{tv} volumetric threshold shear strain below which $\Delta u = 0.0$

f model parameter = 1.0 for one dimensional loading and 2.0 for bidirectional loading

p, F, s are model parameters

p	1.00
F	$3810\ (V_S)^{-1.55}$
γ_{tv} (%)	0.01
s	$(FC+1)^{0.1252}$

CONCLUSIONS

- Promising perspectives are offered by the technique of soil reinforcement with DSM trenches : cost, efficiency
- Based on extensive numerical 3D nonlinear dynamic analyses, validated by comparison with centrifuge experiments, a design procedure is established
  - Liquefaction triggering is assessed from a strain-based approach
  - The safety margin is evaluated with respect to the seismically induced pore water pressure and not from a conventional safety factor

CONCLUSIONS

- Real configurations may differ from those analyzed in these recommendations (layered profiles with significant stiffness variations, sloping ground surface…)
  - ➢Design equation for the seismic demand may need more detailed studies (e.g. FE analyses)
- A last comment : the consequences development of high pore water pressure locally within a cell are less dramatic than for unimproved soil ; the gravel mattress allows for the redistribution of the loads on the DSM grid and the DSM walls prevent pore water pressure migration.

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