INCONTRO INFORMATIVO SU

**ADOZIONE LINEE GUIDA
GESTIONE DEL TERRITORIO IN
AREE INTERESSATE
DA
LIQUEFAZIONE DEI SUOLI**

(DGR nr. 535 del 18/06/2021)

Genova, 29 settembre 2021
Sala Colombo - Via Fieschi, 15

LIQUEFAZIONE: ASPETTI GEOTECNICI

Prof.Ing. Riccardo Berardi

(Federaz. Ordine Ingegneri della Liguria)

✉ riccbera@gmail.com ; riccardo.berardi@unige.it

Cenni introduttivi

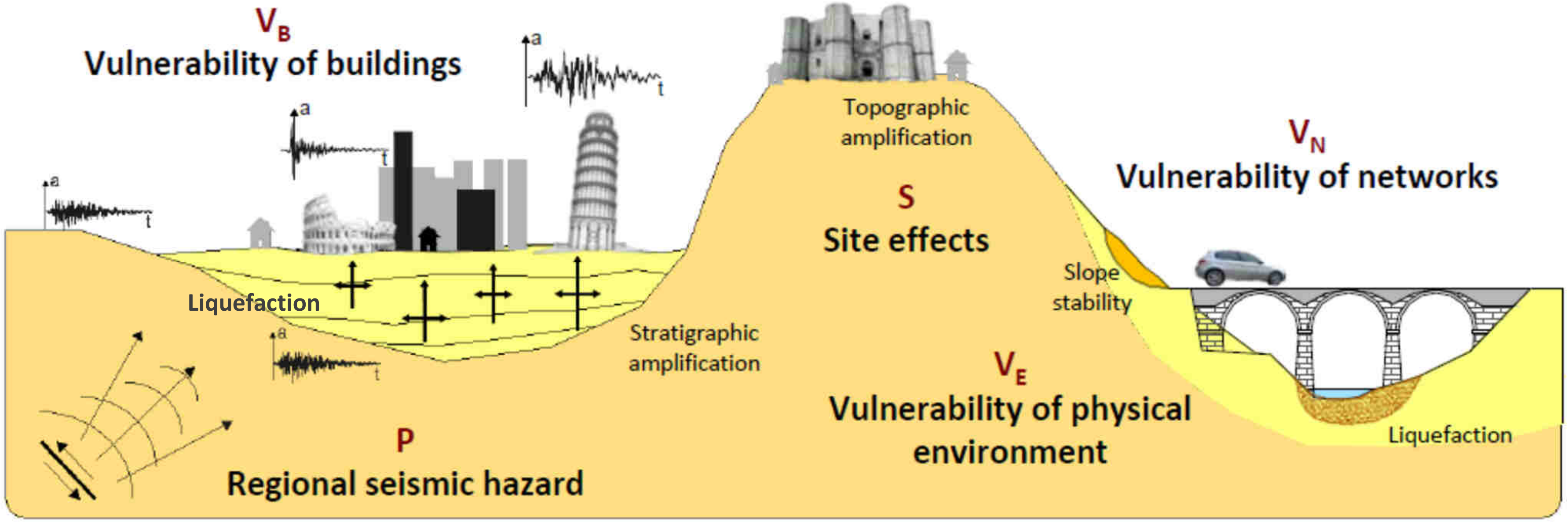
Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento

Interventi di mitigazione del rischio

Seismic risk: $R = HxVxE = (PxS)x(V_E V_B V_N)x(E)$



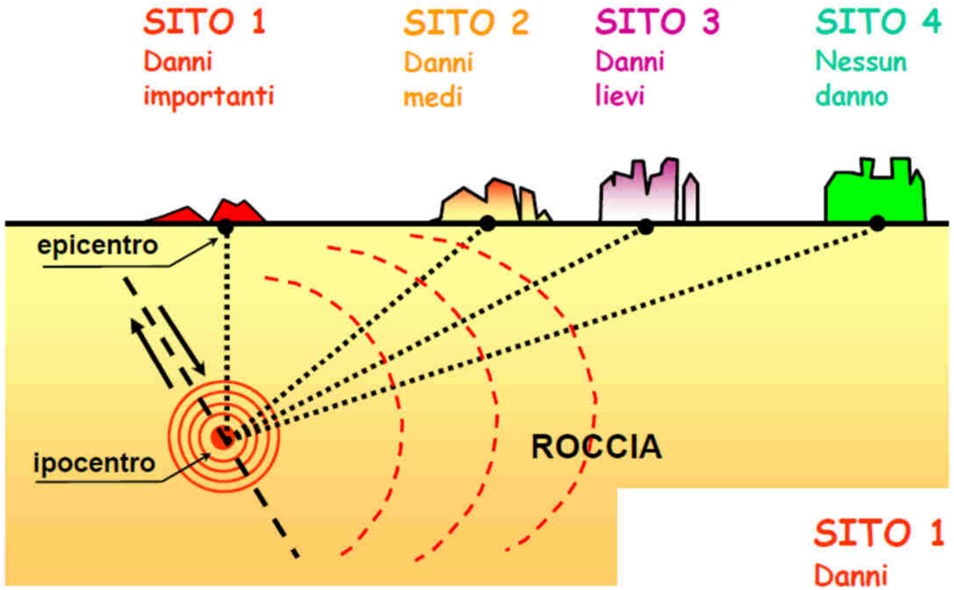
(from F. Silvestri)

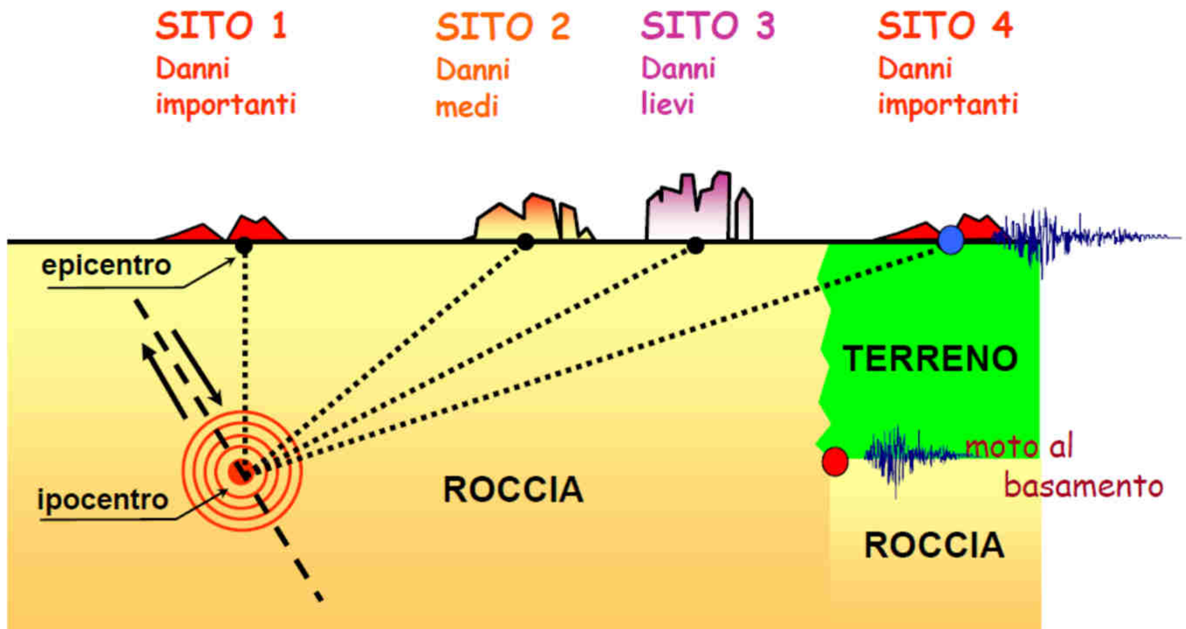
- **Seismic Hazard** (**H**)
  Probability of occurrence of a given seismic event in time and space
  = **Regional seismic hazard** (**P**) x **Site effects** (**S**)

- **Vulnerability** (**V**)
  Damage sensitivity of an engineered system
  = V. of physical Environment x V. of Building heritage x V. of Networks/infrastructures =
  = (V_E) **x** (V_B) **x** (V_N)

- **Exposure** (**E**)
  Loss felt by a social community and related resources

IMPORTANCE OF GROUND CHARACTERISTICS





(from F. Silvestri)

Liquefaction: buildings (bearing capacity and settlements)

Liquefaction: buildings (bearing capacity and settlements)

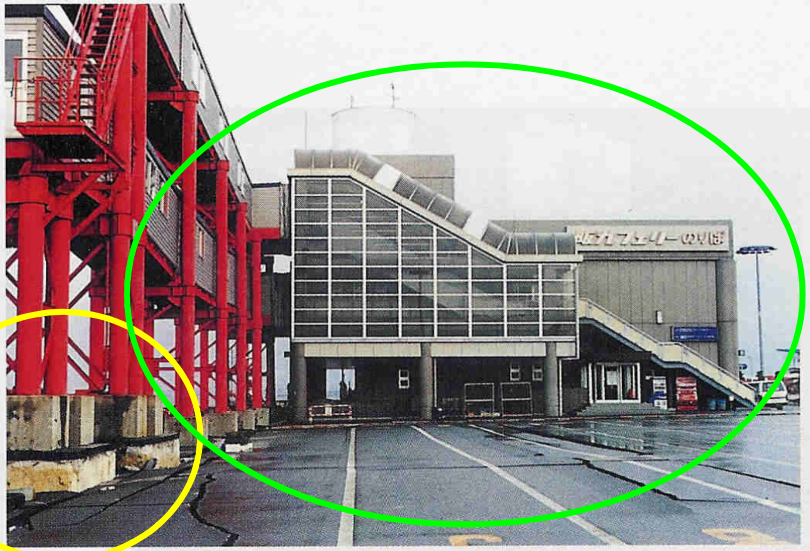
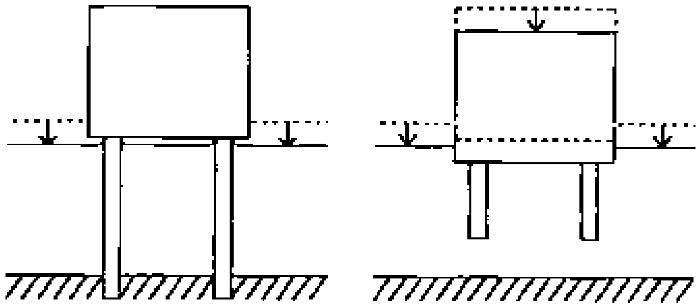


Photo 20. Building with friction piles showing no apparent differential settlement

In the northeast section of the waterfront on Port Island, a ferry terminal building supported by friction piles settled equally with the ground surface, maintaining a relative level between the two. It should be noted that differential settlement up to 40 cm occurred at the two leftmost footings of the boarding bridge supported by point bearing piles. (by K. Tokimatsu, Tokyo Institute of Technology)



(a) Projection of pile due to ground settlement

(b) Small vertical gap at building base

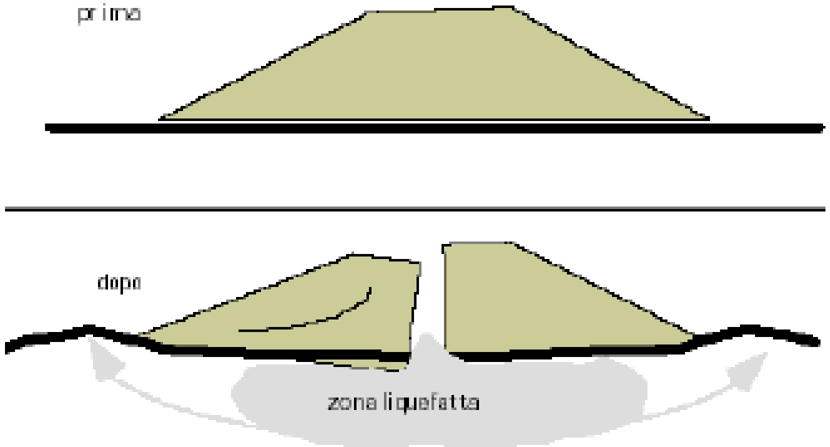
Liquefaction: buildings (bearing capacity and settlements)

Liquefaction: infrastructures

Liquefaction: infrastructures

Liquefaction: slope stability



Cenni introduttivi

Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento

Interventi di mitigazione del rischio

Definizione e Key-Words

Si definisce **LIQUEFAZIONE** la riduzione di resistenza e/o rigidezza causata (durante il moto sismico) dall' aumento di pressioni interstiziali in terreni saturi non coesivi, tale da provocare deformazioni permanenti significative o persino da indurre nel terreno condizioni di collasso per annullamento degli sforzi efficaci.

A decrease in the shear strength and/or stiffness caused by the increase in pore water pressure in saturated cohensionless materials during earthquake ground motion, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil, shall be referred to as **LIQUEFACTION** (EC8)

Key-Words

- ✓ Terreno saturo → **Fase solida + acqua**
- ✓ Terreno «non coesivo» → **Resistenza solo attritiva ($c' \approx 0$)**
- ✓ Azione sismica indotta → **Condizioni NON DRENATE($\Delta u \neq 0$)**

<u>innesco</u> della LIQUEFAZIONE dipende da :

<u>innesco e sviluppo</u> della LIQUEFAZIONE dipende da :

- **SOIL «STATE» del terreno : addensamento (D_R) + stato tensionale efficace iniziale (p')**
- **AZIONE SISMICA (a_{max}) and N° of CICLI (N_c)**

POSSIBLE LIQUEFABLE SANDY SOILS

distribuzione granulometrica interna alle zone indicate nella Figura

a) nel caso di terreni con coefficiente di uniformità *U*c < 3,5

b) nel caso di terreni con coefficiente di uniformità *U*c > 3,5

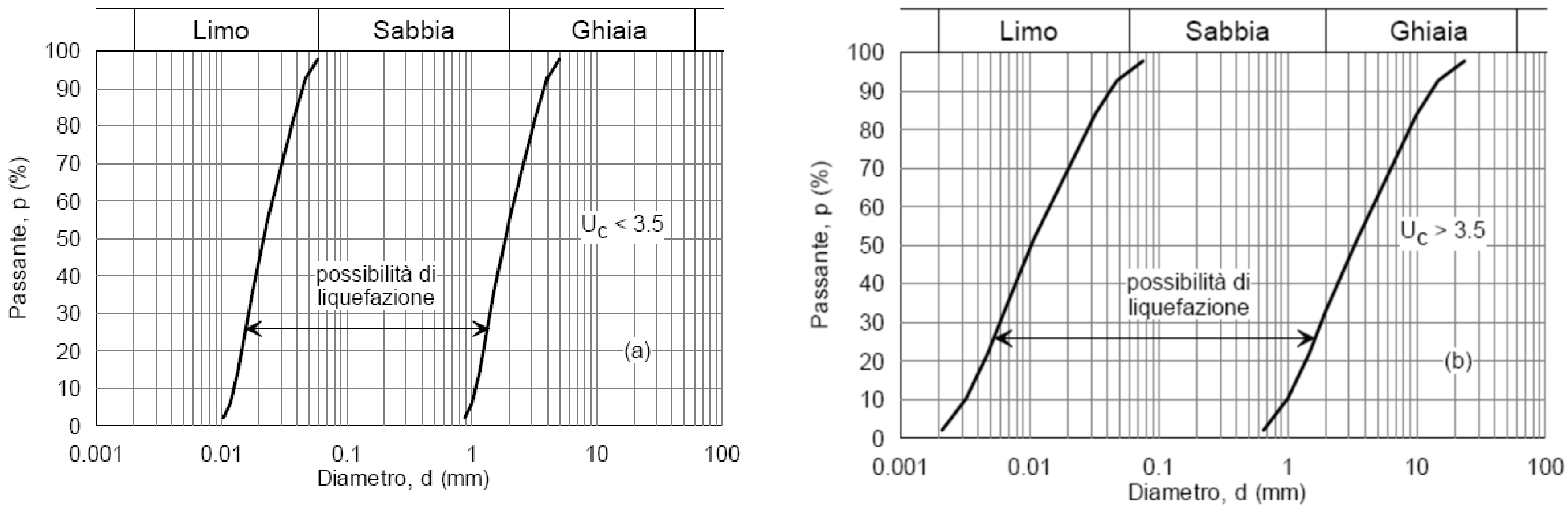
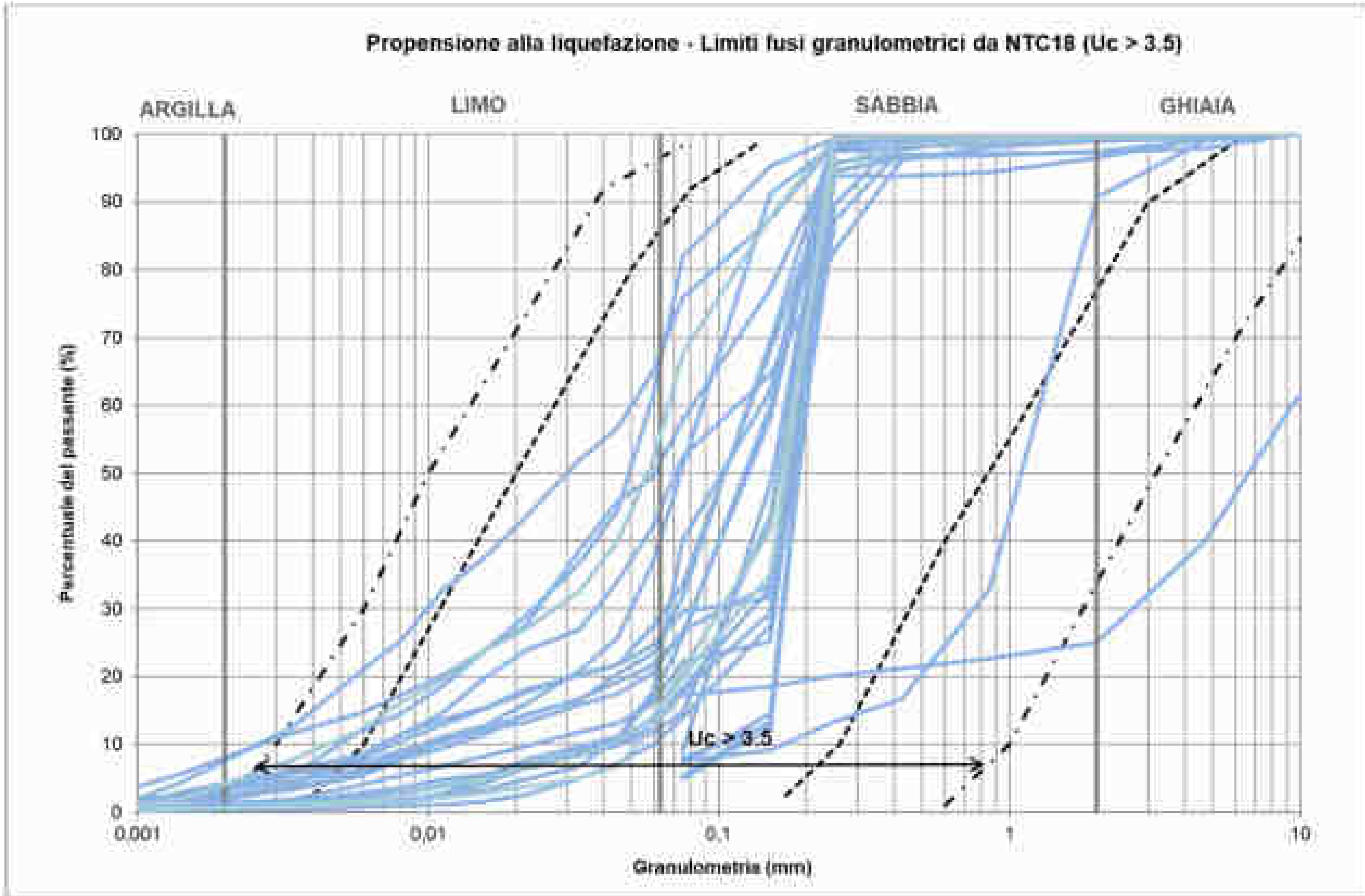


Figura 7.11.1 – *Fusi granulometrici di terreni suscettibili di liquefazione.*



Fuso granulometrico con Uc > 3.5

CYCLIC LOAD IN UNDRAINED CONDITION – SATURATED SAND

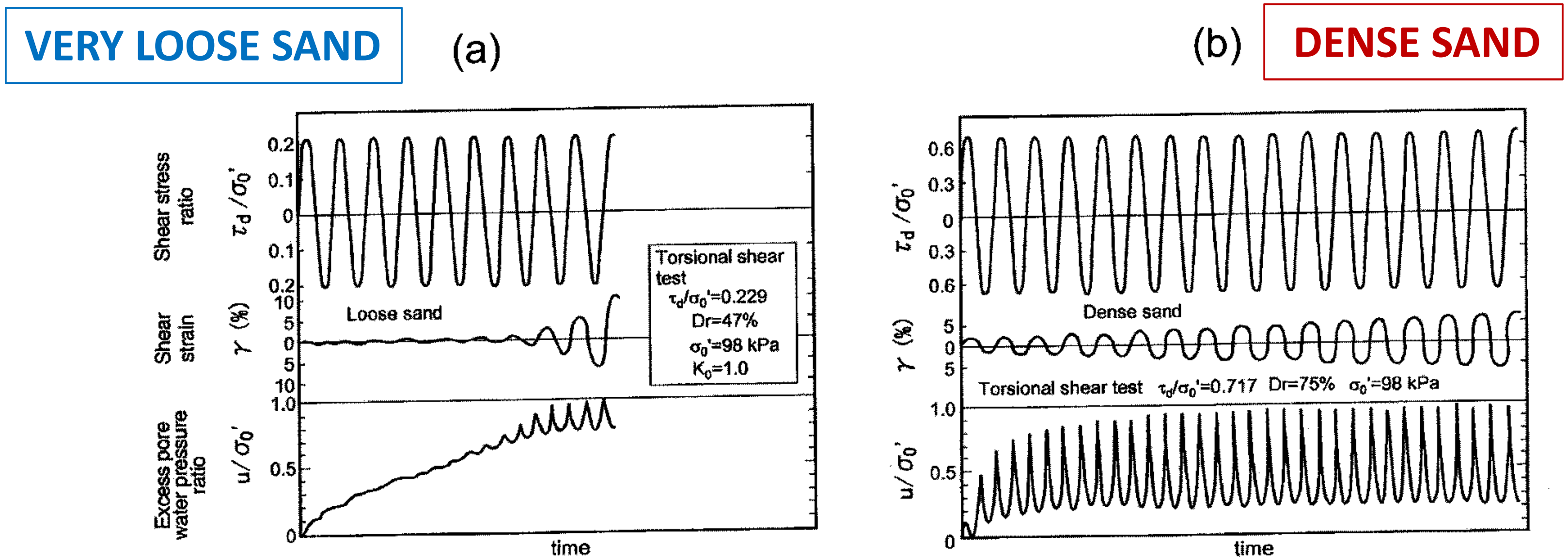
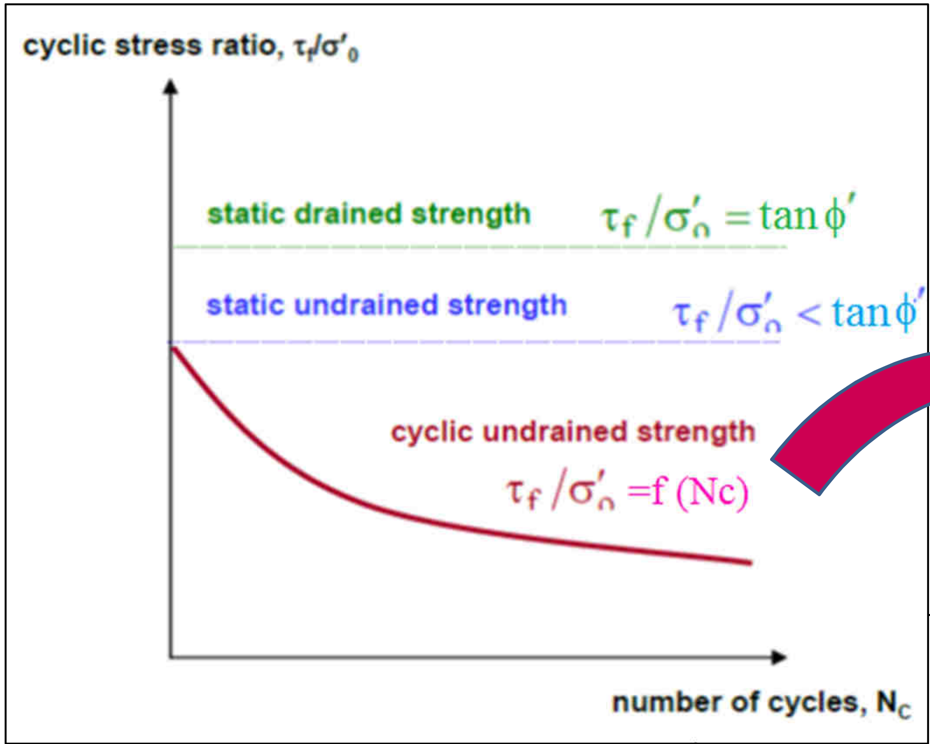


Figura 2.8 - Cicli sperimentali di tensione, deformazione e sovrapressione interstiziale in una sabbia sciolta (a) ed una densa (b) (prove di torsione ciclica su sabbia del fiume Fuji, da Ishihara, 1985).

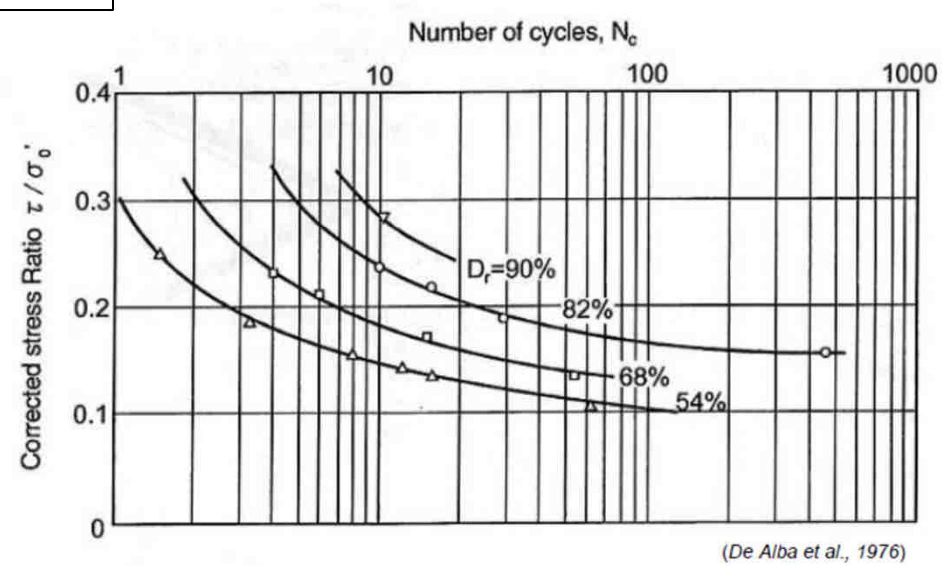
$$\tau_f = (\sigma - u)\tan\phi' \equiv \left[\sigma_0 - (u_0 + \Delta u)\right]\tan\phi' = (\sigma'_0 - \Delta u)\tan\phi'$$

$$\tau_f \to 0 \quad \text{per} \quad r_u = \frac{\Delta u}{\sigma'_0} \to 1$$

CRR Cyclic Resistance Ratio: soil strength



$$CRR = \frac{\tau}{\sigma'_0} = f(N_c)$$



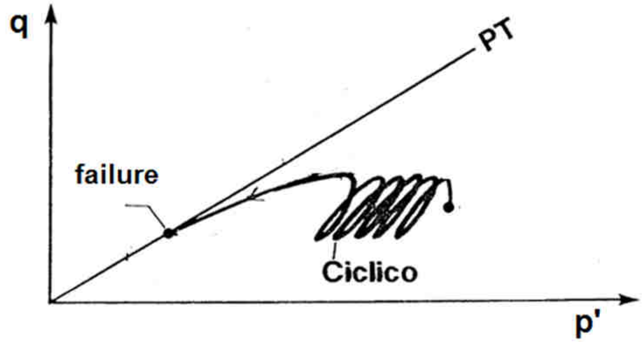
LIQUEFAZIONE DEL TERRENO

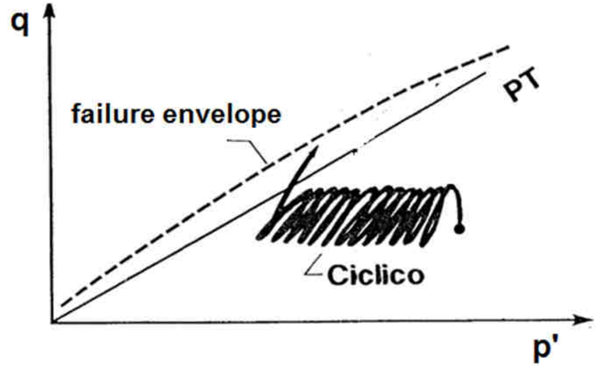
- liquefazione: riduzione di resistenza per sviluppo di incrementi di pressione interstiziale durante il sisma
- la riduzione di resistenza produce un **comportamento instabile**: spostamenti grandi e improvvisi
- riduzione di resistenza e **comportamento stabile**: mobilità ciclica
- la possibilità che si verifichi liquefazione dipende da:

  stato iniziale
  - indice dei vuoti
  - stato tensionale

  entità della sollecitazione
  - ampiezza
  - numero di cicli

LIQUEFACTION MECHANISMS





Cyclic loading-shear wave propagation

Pore pressure buid-up

Undrained conditions–
u increases u –σ' decreases

se Δu→ σ'_c : soil starts deforming *(INITIAL LIQUEFACTION*)

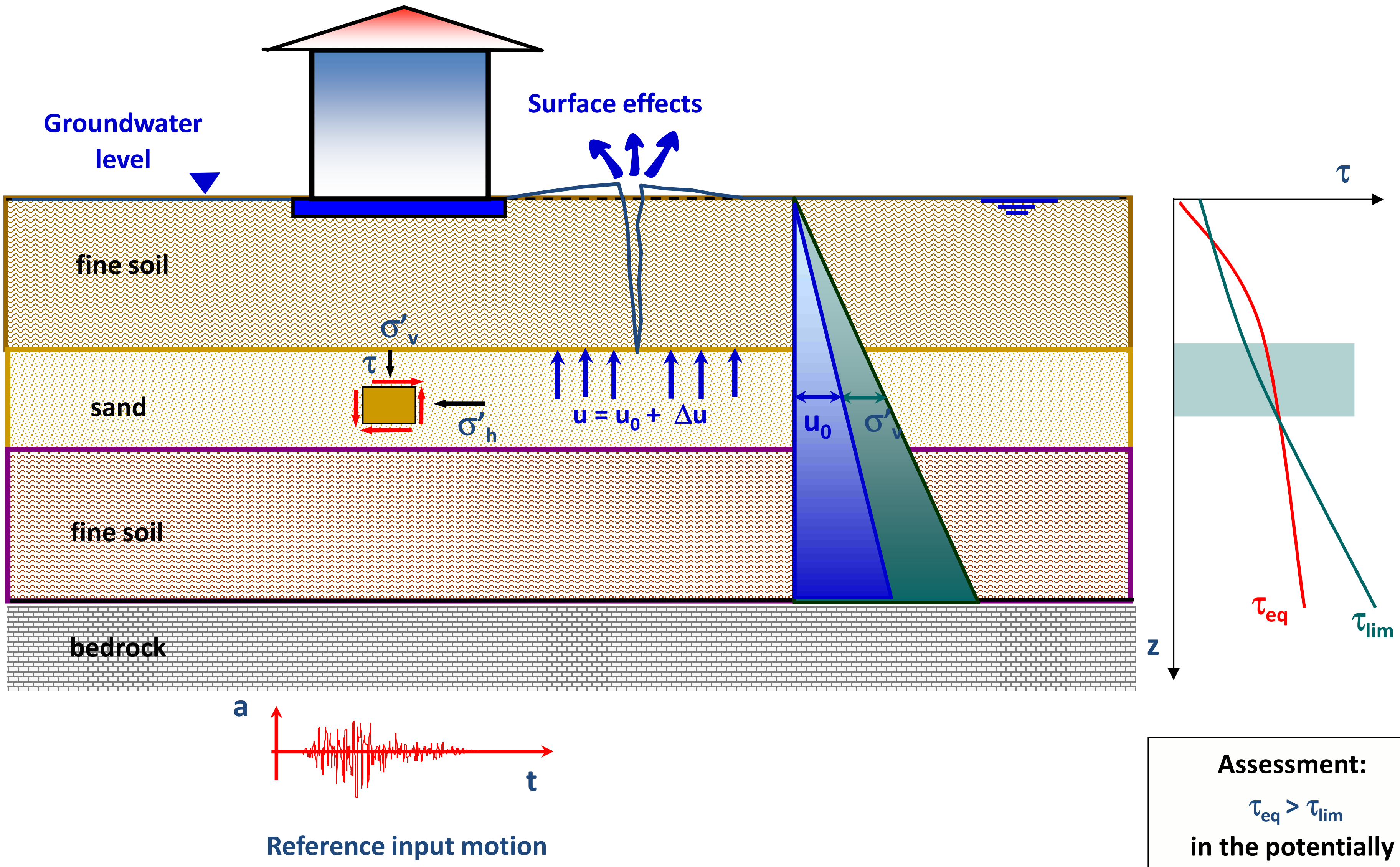
«LOOSE SANDS»
u = σ'_C – LARGE STRAINS AND FAILURE

LIQUEFACTION – $\tau = \sigma' \cdot \tan\varphi' = 0$

"DENSE" SANDS
u < σ'_C – LARGE STRAINS

«CYCLIC MOBILITY» - $\tau = \sigma' \cdot \tan\varphi' \neq 0$

Liquefaction: general approach for the analysis



Assessment:

$\tau_{eq} > \tau_{lim}$

in the potentially liquefiable layers

Cenni introduttivi

Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento

Interventi di mitigazione del rischio

Liquefaction assessment

CRR(z) > CSR (z)

CRR Cyclic Resistance Ratio: soil strength > CSR Cyclic Stress Ratio: seismic action

(1) For footing–type foundations, the soil volume to consider for liquefaction susceptibility should be related to the width of the entire bearing area rather than to the width of individual footings, Figure B1.

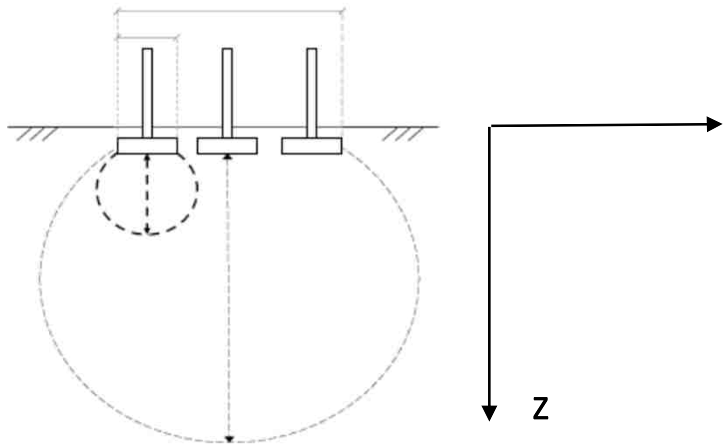


Figure B.1 Sketch (not on scale) of involved soil volume for footing-type foundations (big circle)

Semi-empirical methods

Laboratory tests very sensitive to experimental factors → not used in practice!

ISSMGE-TC4 (1999) recommends simplified procedures based on field tests and observed behaviour during earthquakes → charts

Classic empirical approach:

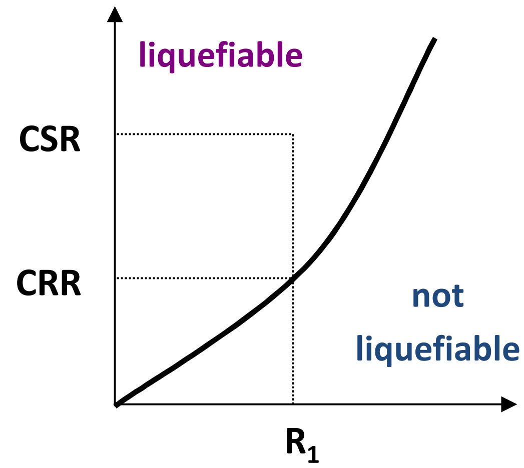
⇨ **step 1: evaluation of seismic action**
(cyclic stress ratio, CSR)

⇨ **step 2: correction & normalization of in-situ measurements**
(normalized strength, R_1)

⇨ **step 3: use of liquefaction charts**
cyclic resistance ratio, CRR = $f(R_1)$

$\Downarrow$

CSR > CRR ⇒ liquefiable soil

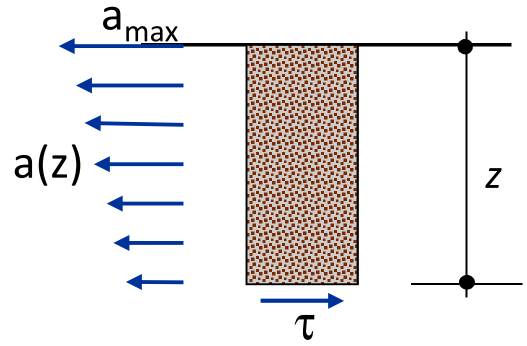


Step 1 Evaluation of seismic action

Empirical relationship (*Seed & Idriss, 1971*):

$$CSR = \frac{\tau_{eq}}{\sigma'_{v0}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{v0}}{\sigma'_{v0}} r_d$$

The cyclic load is correlated to the maximum shear stress, τ_{max}, induced by the earthquake motion at the depth of interest



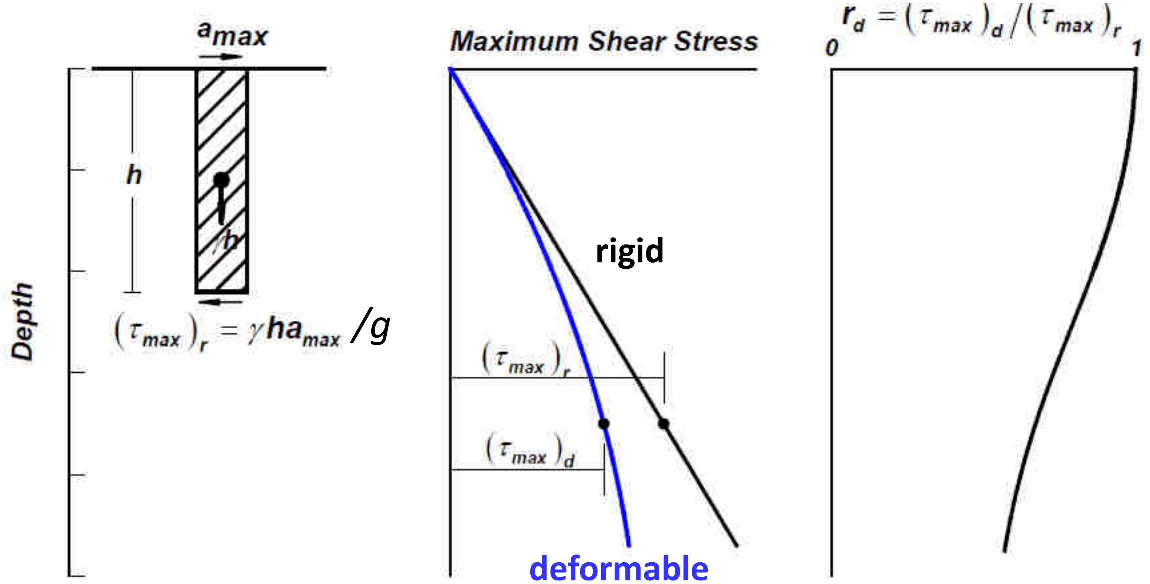
where it is assumed β=0.65 and

a_{max}= peak horizontal ground acceleration

σ_{v0} = lithostatic total stress

σ'_{v0} = lithostatic effective stress

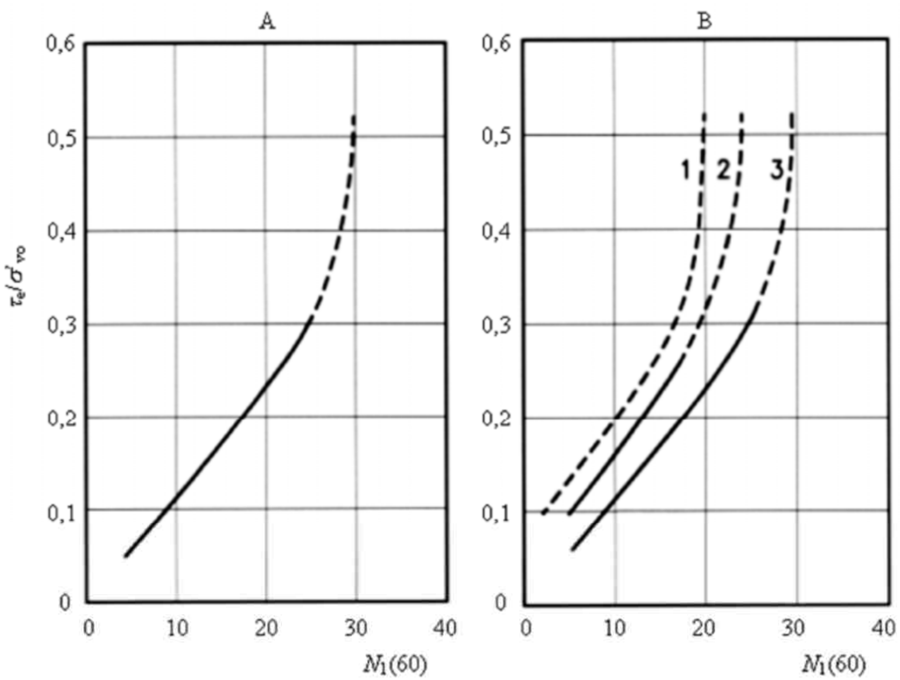
r_d = shear stress reduction factor
at depth z (deformability effect)



Step 3 Standard liquefaction charts: SPT tests

In the charts a curve separates the data points for which liquefaction has been observed from those where it did not occur.

$$D_R = \sqrt{\frac{(N_1)_{60}}{60}}$$



Key

τ_e/σ'_{vo} – cyclic stress ratio

A – clean sands;

B – silty sands

curve 1: 35 % fines

curve 2: 15% fines

curve 3: < 5% fines

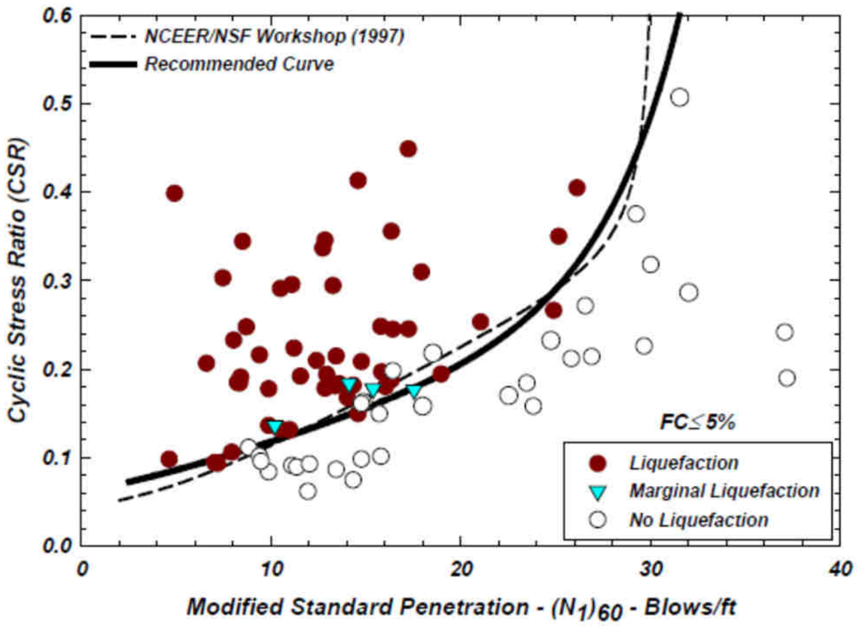
Figure B.1 — Relationship between stress ratios causing liquefaction and $N_1(60)$ values for clean and silty sands for M_S=7,5 earthquakes.

Step 3 Standard liquefaction charts: SPT tests

In the charts a curve separates the data points
for which liquefaction has been observed
from those where it did not occur.

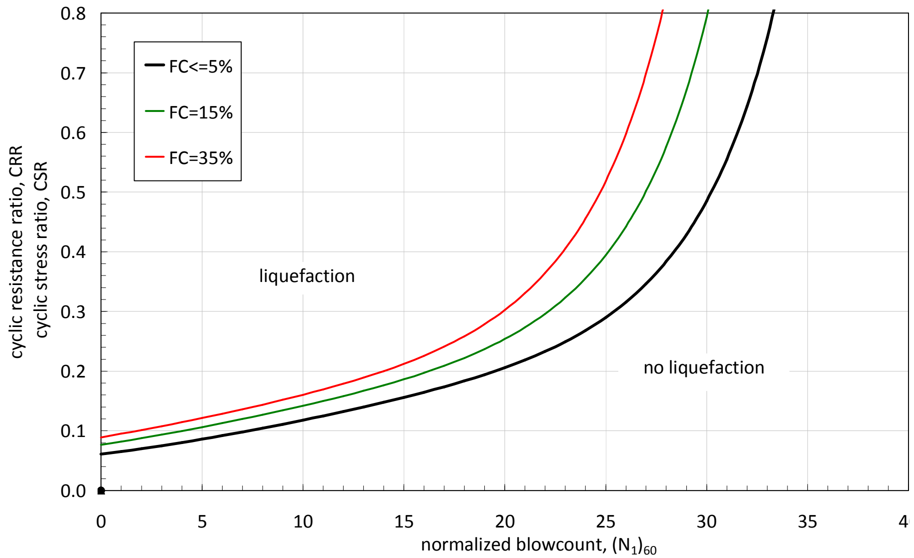
The 'standard charts' are referred to:

- moment magnitude M_w=7.5
- clean sands, with fine content FC ≤ 5%



$$CRR \cong \exp\left[\frac{(N_1)_{60cs}}{14.1}+\left(\frac{(N_1)_{60cs}}{126}\right)^2-\left(\frac{(N_1)_{60cs}}{23.6}\right)^3+\left(\frac{(N_1)_{60cs}}{25.4}\right)^4-2.8\right]$$

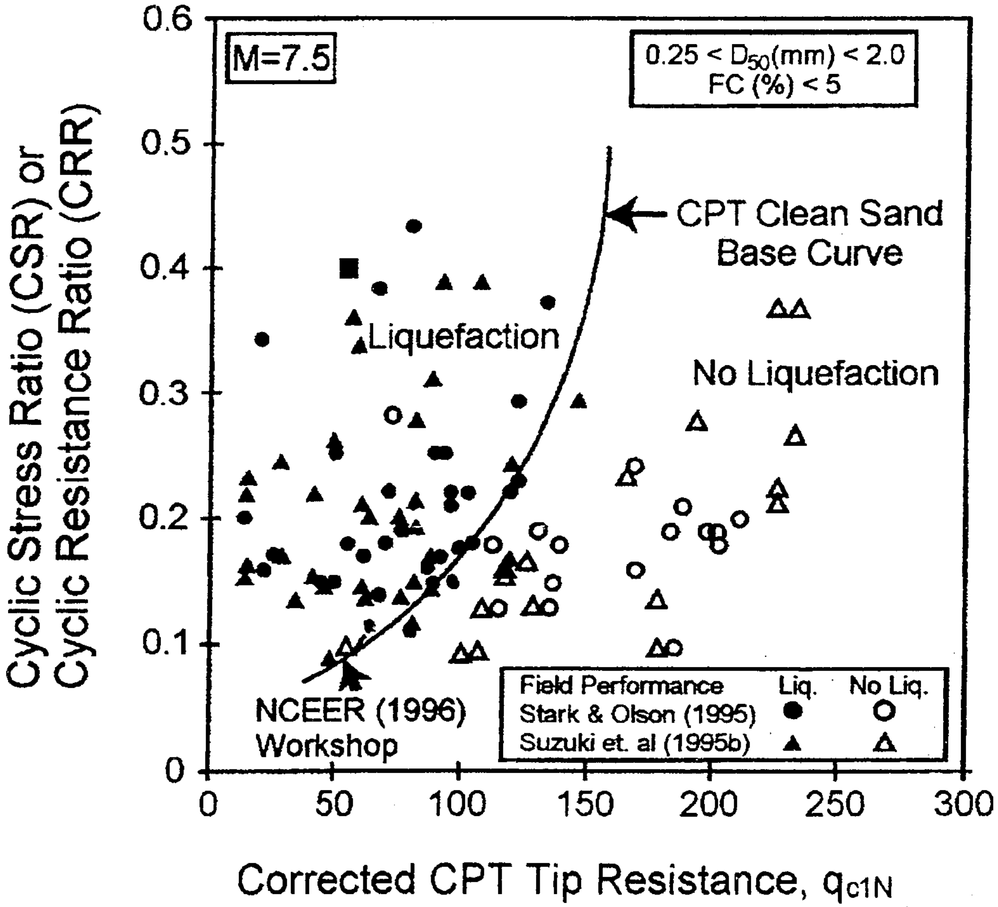
The increase of strength with the fine fraction (F_C) corresponds to a virtual increment of $(N_1)_{60cs}$

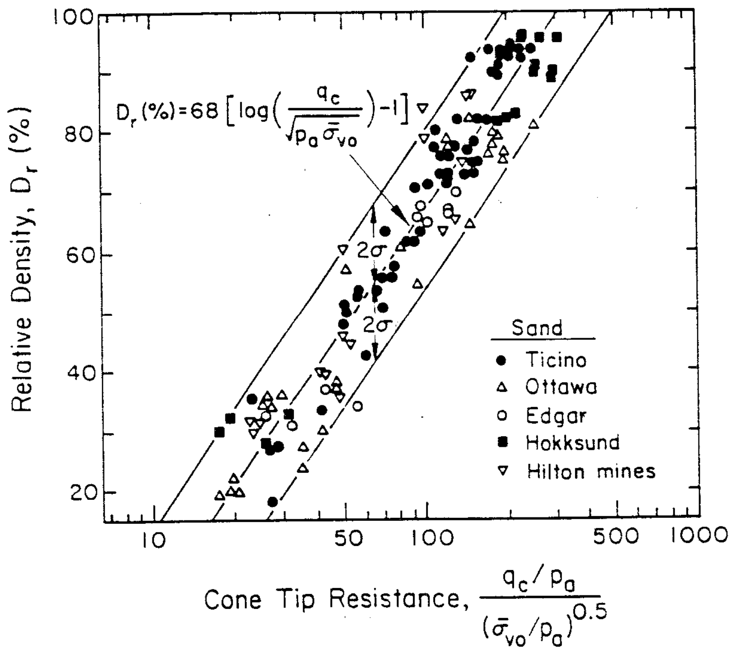


$$(N_1)_{60cs}=(N_1)_{60}+\exp\left(1.63+\frac{9.7}{F_c}-\left(\frac{15.7}{F_c}\right)^2\right)$$

(Idriss & Boulanger, 2004)

Step 3 Standard liquefaction charts: CPT tests





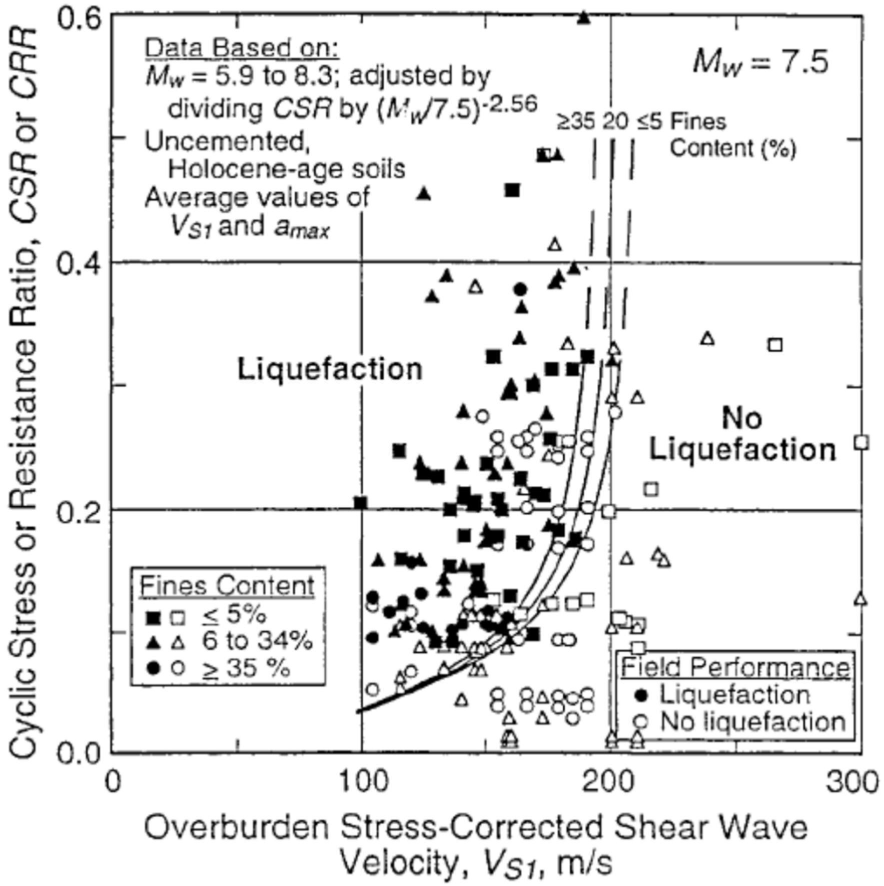
(Lancellotta, 1993)

$$q_{c1N} = \left(\frac{q_c}{p_a}\right) \cdot C_q$$

$$C_q = \left(\frac{p_a}{\sigma'_{vo}}\right)^{0.5}$$

$$p_a = 100kN/m^2$$

Step 3 V_S based standard liquefaction charts



M=7.5 chart

(Andrus & Stokoe, 2000)

⇨ **Advised for gravelly soils (penetration tests impossible)**

⇨ **Suitable for soils with crushable grains (for which penetration tests underestimate strength)**

⇨ **In weakly cemented soils, the liquefaction potential can result underestimated (cementation affecting stiffness more than strength)**

$$CRR = 0.022\left(\frac{V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{S1c} - V_{S1}} - \frac{1}{V_{S1c}}\right)$$

$$V_{s1} = V_s\left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25}$$

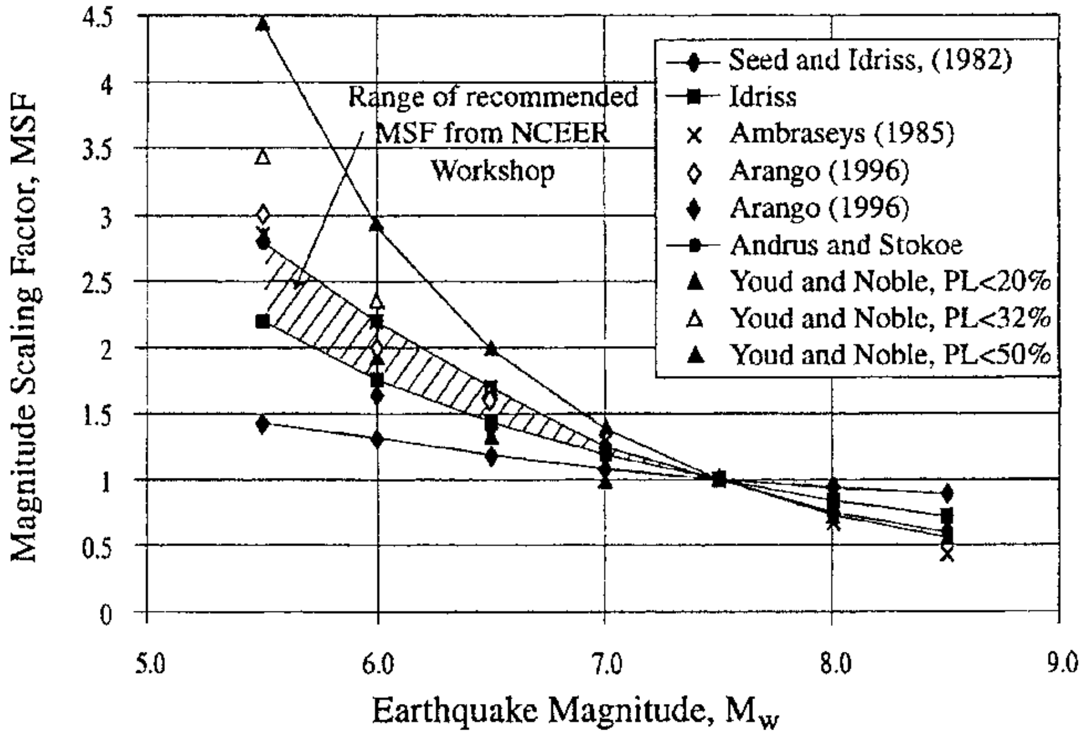
What magnitude for the liquefaction assessment? (2/3)

$$CRR^{M}(z) = CRR^{M=7.5}(z) \cdot MSF$$

$$MSF = \frac{10^{2.24}}{M_w^{2.56}}$$

Table B.1 — Values of factor MSF

M_S	MSF
5,5	2,86
6,0	2,20
6,5	1,69
7,0	1,30
8,0	0,67



Deterministic approach

Maximum magnitude, $\mathbf{M_{max}}$, potentially influencing the site seismicity, to be obtained:

- from the seismic history of the site and the seismogenic zone
- from empirical relationships (e.g. *Wells & Coppersmith, 1994*), referred to fault size and tectonic style

LIQUEFACTION SUSCEPTIBILITY : SEMI-EMPIRICAL APPROACH

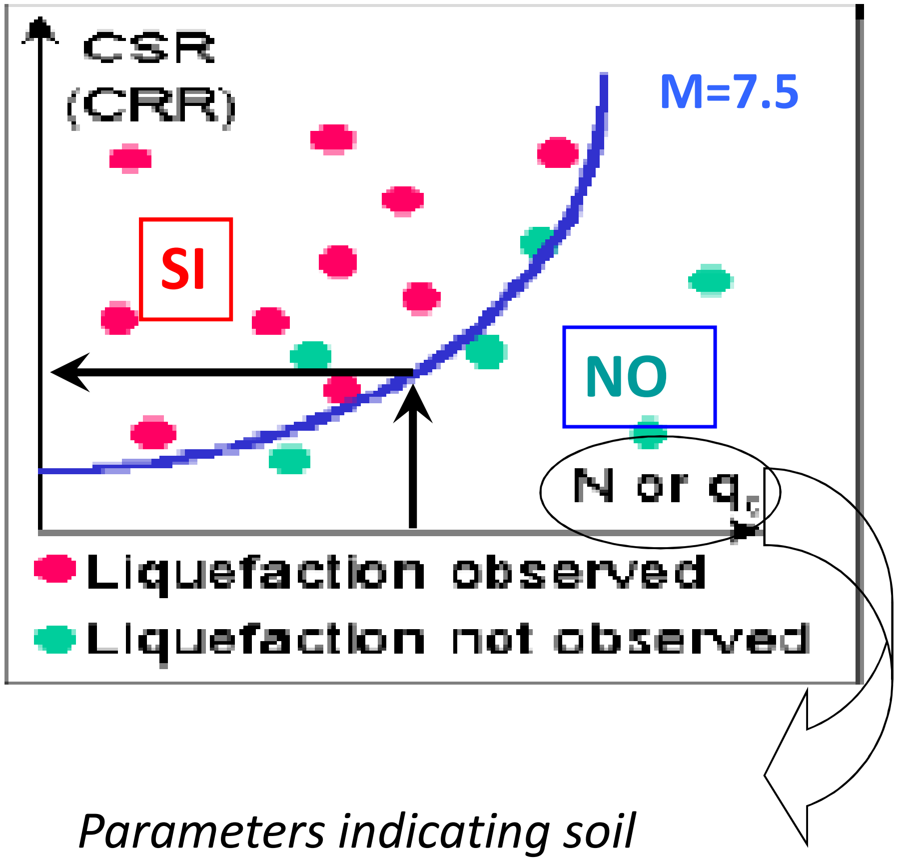
Cyclic Stress Ratio:

$$CSR(z) = \frac{\overline{\tau}(z)}{\sigma_v^{'}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma_v^{'}}\right) r_d(z)$$

Cyclic Resistance Ratio:

$$CRR^{M}(z) = CRR^{M=7.5}(z) \cdot MSF$$

$$FS(z) = \frac{CRR^{M}(z)}{CSR(z)} \geq 1$$



Parameters indicating soil relative density D_R

Step 3 Comparison among standard liquefaction charts

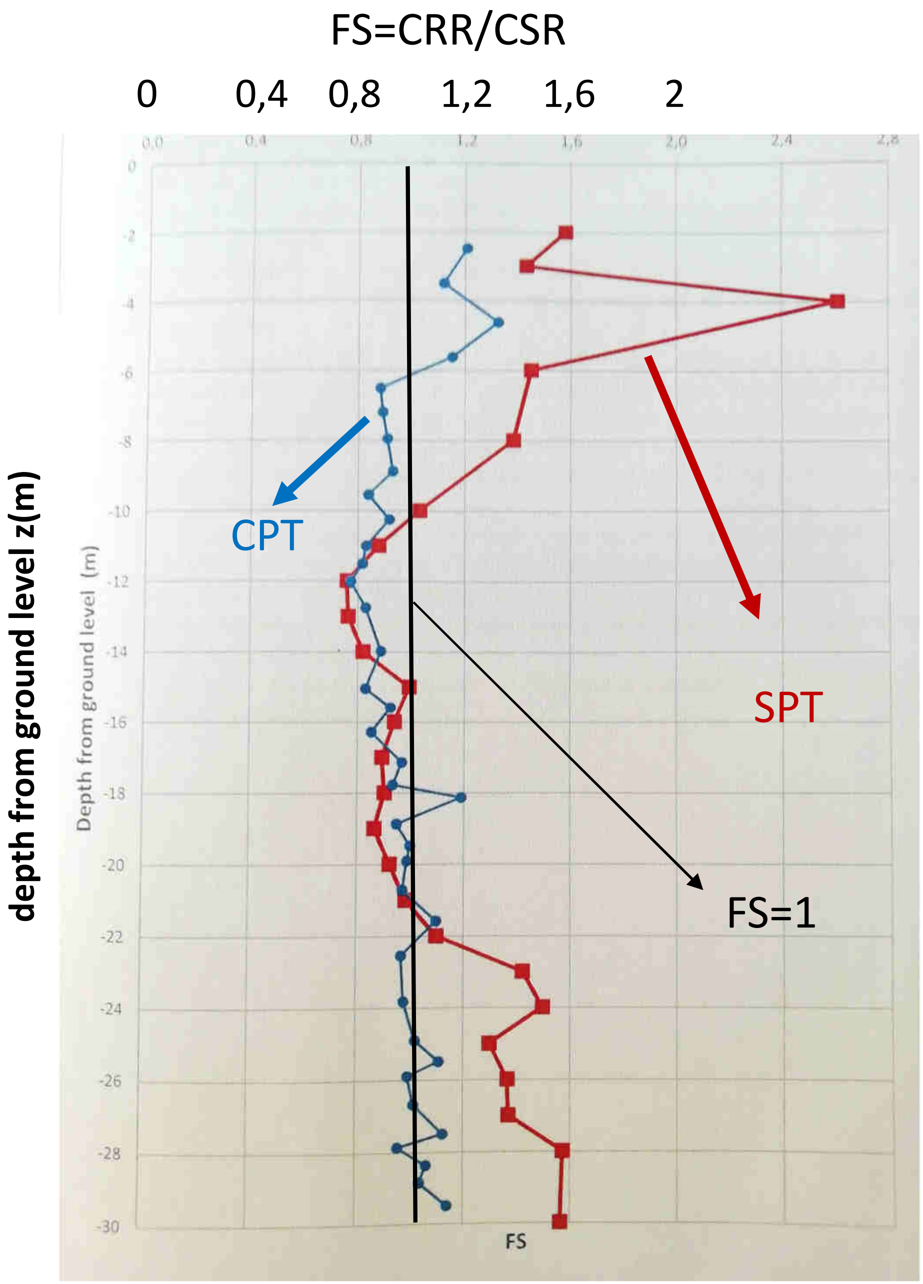
Comparative performance of various field tests (Youd & Idriss, 1997)

Feature	Test Type		
	SPT	CPT	V_S
Number of test measurements at liquefaction sites	Abundant	Abundant	Limited
Type of stress-strain behaviour influencing test	Partially drained, large strain	Drained, large strain	Small strain
Quality control and repeatability	Poor to good	Very good	Good
Detection of variability of soil deposits	Good	Very good	Fair
Soil types in which test is recommended	Non-gravel	Non-gravel	All
Test provides sample of soil	Yes	No	No
Test measures index or engineering property	Index	Index	Engineering property

⇨ **The suggested practice is to apply two or more field-based procedures to achieve reliable evaluation of liquefaction resistance**

SPT vs CPT APPROACH

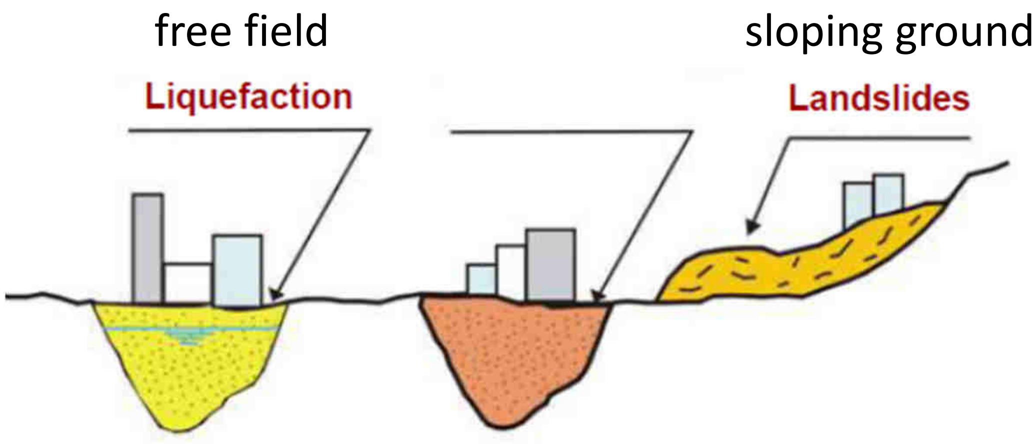
L'approccio CPT è spesso più affidabile e accurato, poiché consente di tenere conto della stratigrafia in modo più preciso e continuo



OSSERVAZIONI : in condizioni non «free-field» (es. pendii)

- Semi-empirical analysis $\rightarrow$ simplified approach with some correction factors

- Dynamic analysis $\rightarrow$ possible to account for actual geometry /external loads



LIQUEFACTION EFFECTS : SEMI-EMPIRICAL APPROACH (2/2)

Index of potential liquefaction I_L

$$I_L = \int_0^{20} F(z)w(z)dz$$

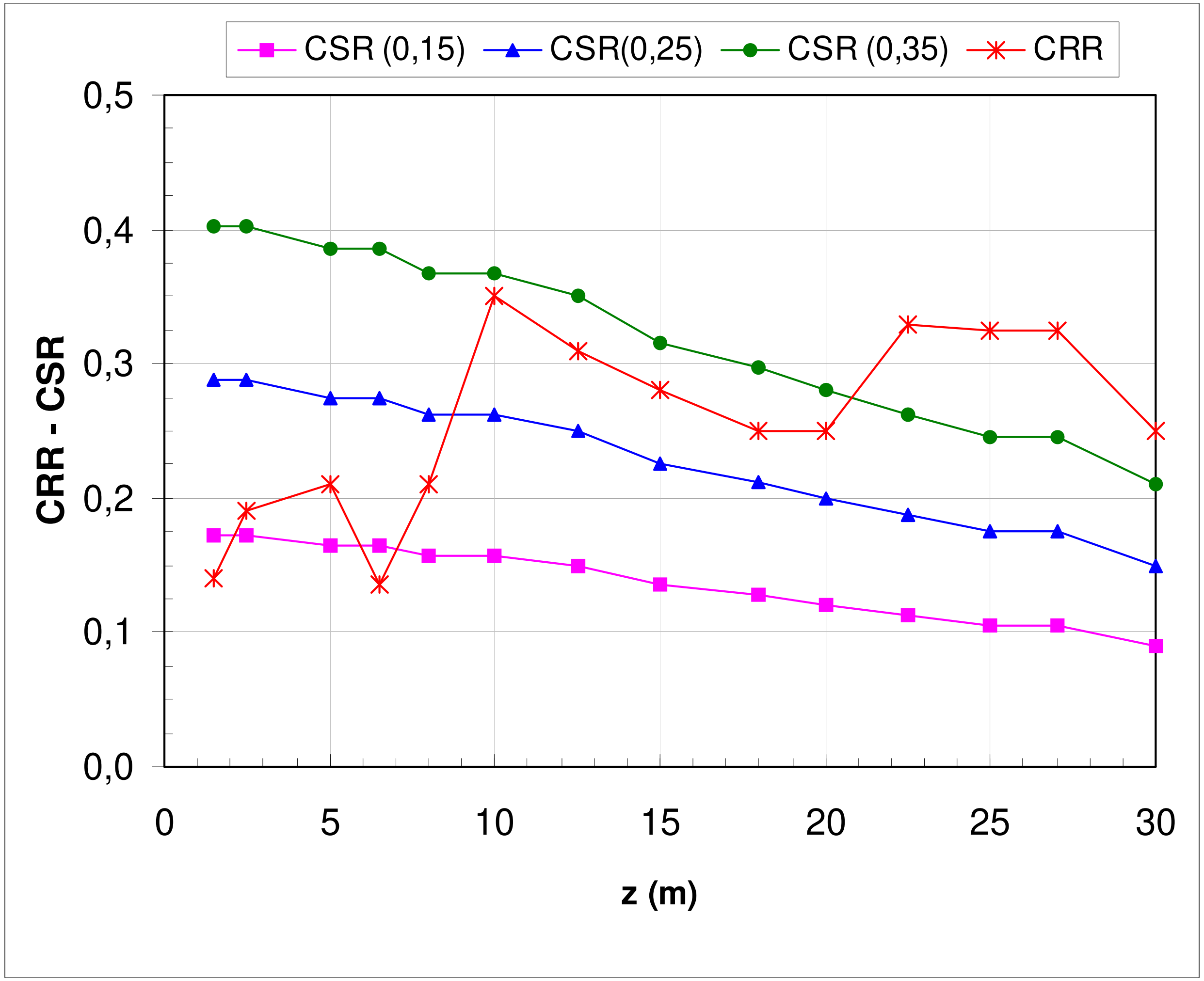
$$w(z) = 10 - 0.5z \qquad z\ [m]$$

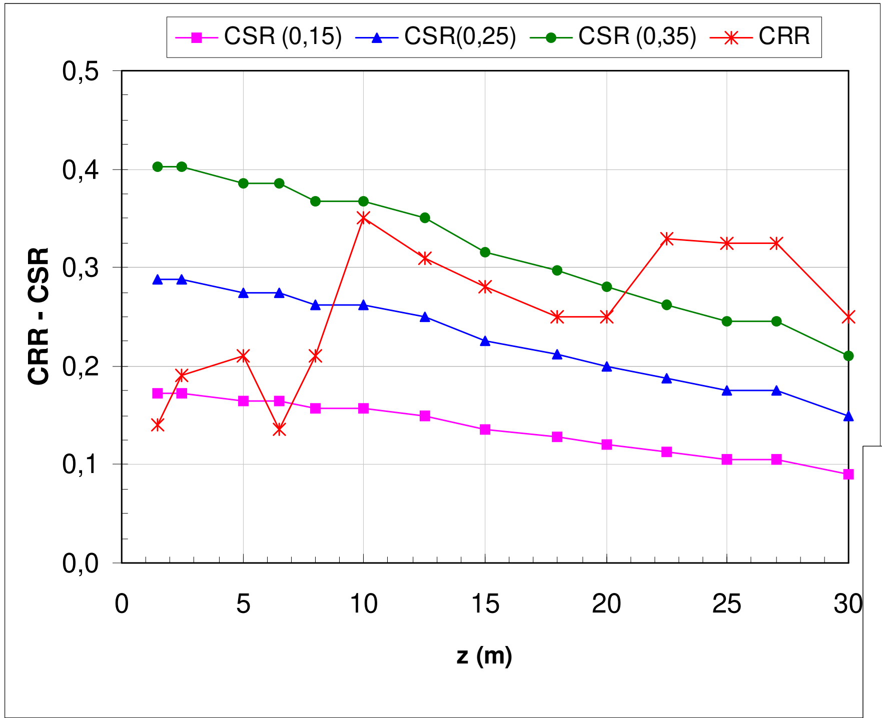
$$F = 1 - F_S \quad if \quad F_S \leq 1$$

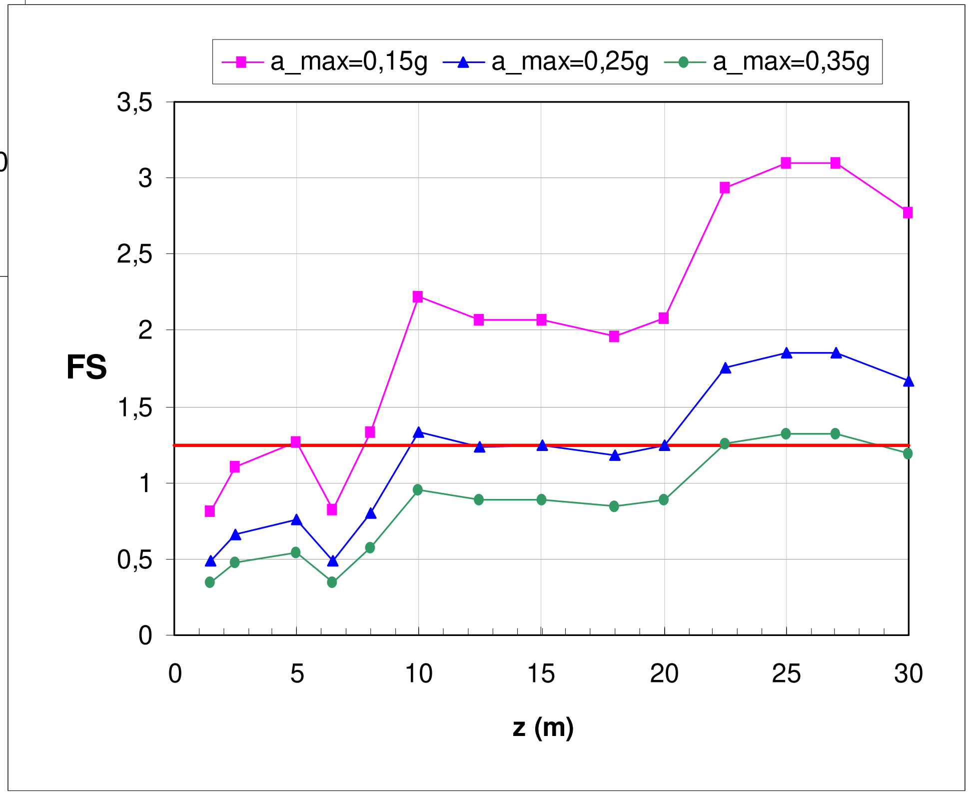
$$F = 0 \quad if \quad F_S > 1$$

- **$I_L \leq 5$ low potential risk**
- **$5 < I_L \leq 15$ high potential risk**
- **$I_L \geq 15$ extremely high potential risk**

(Iwasaki et al., 1982)


CSR (0,15)
CSR(0,25)
CSR (0,35)
CRR
0,5
0,4
0,3
0,2
0,1
0,0
CRR - CSR
0
5
10
15
20
25
30
z (m)


CSR (0,15)
CSR(0,25)
CSR (0,35)
CRR
CRR - CSR
0,5
0,4
0,3
0,2
0,1
0,0
0
5
10
15
20
25
30
z (m)


a_max=0,15g
a_max=0,25g
a_max=0,35g
FS
3,5
3
2,5
2
1,5
1
0,5
0
0
5
10
15
20
25
30
z (m)

z(m)	FS (0,25)	FS (0,35)	FS (0,15)	Delta z	w(z)	F (0.25)	F(0.35)	F (0.15)	F w(z) Dz	F w(z) Dz	F w(z) Dz
0	**0.25**	**0.35**	**0.15**	**(m)**					**0.25**	**0.35**	**0.15**
1.5	0.49	0.35	0.81	1.5	9.25	0.513	0.652	0.188	7.12	9.05	2.61
2.5	0.66	0.47	1.10	1	8.75	0.339	0.528	0.000	2.97	4.62	0.00
5	0.76	0.55	1.27	2.5	7.5	0.236	0.455	0.000	4.43	8.52	0.00
6.5	0.49	0.35	0.82	1.5	6.75	0.509	0.649	0.182	5.15	6.57	1.84
8	0.80	0.57	1.33	1.5	6	0.200	0.429	0.000	1.80	3.86	0.00
10	1.33	0.95	2.22	2	5	0.000	0.048	0.000	0.00	0.48	0.00
12.5	1.24	0.89	2.07	2.5	3.75	0.000	0.114	0.000	0.00	1.07	0.00
15	1.24	0.89	2.07	2.5	2.5	0.000	0.111	0.000	0.00	0.69	0.00
18	1.18	0.84	1.96	3	1	0.000	0.160	0.000	0.00	0.48	0.00
20	1.25	0.89	2.08	2	0	0.000	0.107	0.000	0.00	0.00	0.00

IL (0.25)	IL(0.35)	IL (0.15)
21.5	**35.3**	**4.5**

- **$I_L \leq 5$ low potential risk**
- **$5 < I_L \leq 15$ high potential risk**
- **$I_L \geq 15$ extremely high potential risk**

Cenni introduttivi

Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento (1/2)

Interventi di mitigazione del rischio

Pericolosità sismica: scuotimento atteso in un dato sito, avente data probabilità di superamento, in un certo tempo, nell'ipotesi **dell'affioramento di una formazione rigida e pianeggiante.**

Pericolosità sismica locale: scuotimento atteso in un dato sito, avente data probabilità di superamento, in un certo tempo, **tenendo conto delle possibili modificazioni indotte localmente dalle caratteristiche topografiche , stratigrafiche, geotecniche.**

In termini molto schematici il moto sismico *U(t)* che definisce la pericolosità sismica locale in un dato sito è esprimibile come:

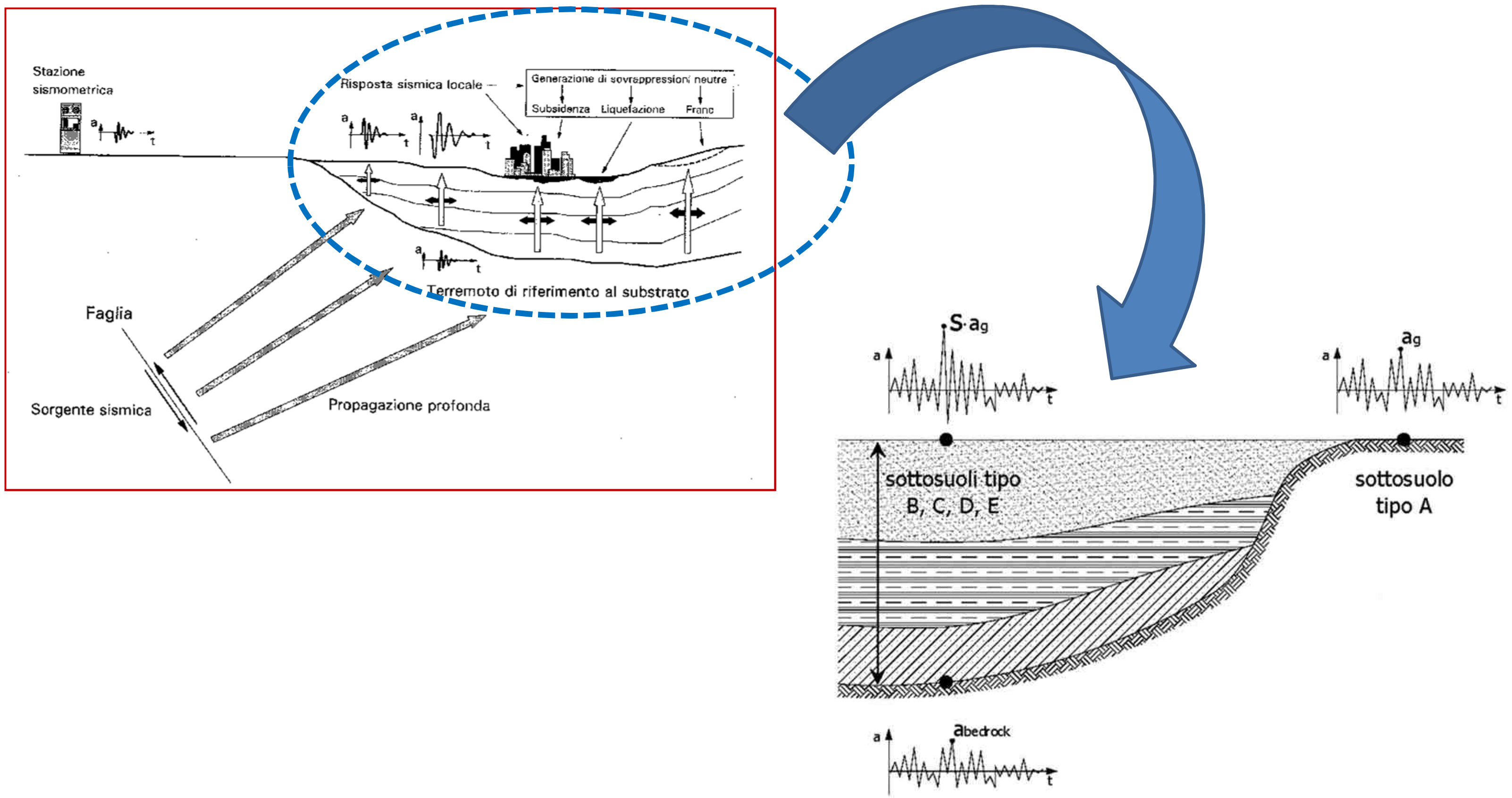
$$U(t) = S(t) \cdot F$$

S(t): segnale di ingresso al sito sulla formazione rigida di riferimento, dipende dai processi di generazione e di propagazione delle onde sismiche dalla sorgente;

F: "funzione di trasferimento" dipende dalle caratteristiche morfologiche e stratigrafiche e dalle proprietà dinamiche dei depositi superficiali (condizioni "locali")

Risposta sismica locale

Effetti (di sito) stratigrafici: (1D) le <u>modifiche al moto sismico</u>, propagandosi in direz. verticale, sono attribuibili a fenomeni di risonanza tra onde sismiche e terreni, in relazione a stratigrafia, alle caratteristiche fisico meccaniche, al contenuto in frequenza.



Ipotesi :

•Comportamento elastico lineare

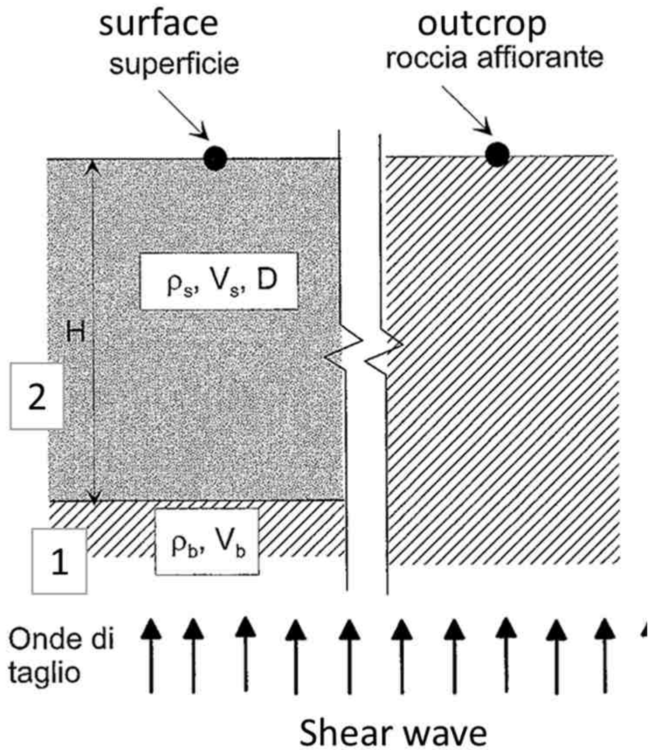
•Onde di taglio verticali incidenti sinusoidali di frequenza *f*

- Su roccia affiorante:

Onda di frequenza *f* e ampiezza $a_{max,r}$

- In superficie:

Onda di frequenza *f* e ampiezza $a_{max,s}$



$$\boxed{Z = \frac{E_{ed}}{V_P} = \frac{\rho V_P^{2}}{V_P} = \rho V_P}$$ = '*Seismic Impedance*'

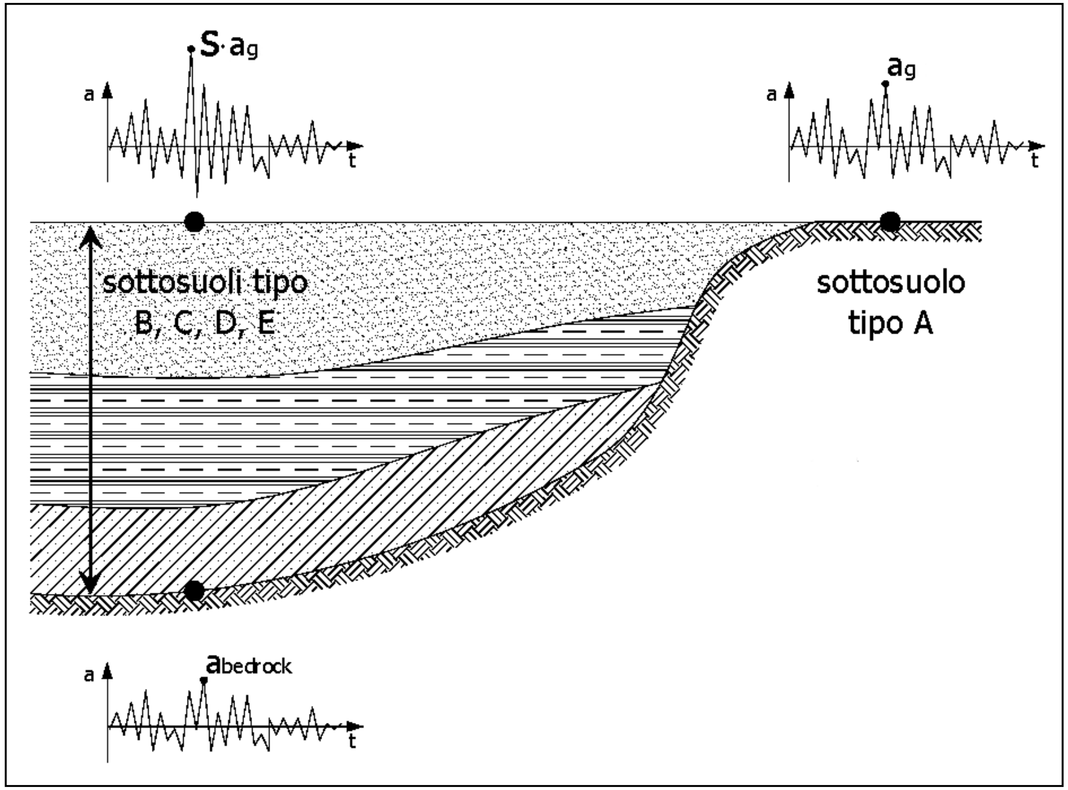
$$\boxed{\mu = \frac{1}{I} = \frac{Z_2}{Z_1} = \frac{\rho_2 V_2}{\rho_1 V_1}}$$ = '*Impedance ratio*'

Fattore amplificazione

$$S = \frac{a_{\max,s}}{a_{\max,r}} = \mathcal{F}\left(f, D, I = \frac{\rho_b V_b}{\rho_s V_s} \right)$$

$$N.B.: \quad a_{\max,r} \equiv S(t); \quad a_{\max,s} \equiv U(t)$$

Seismic amplification (local ground effects) – NTC - EC



$$a_{max,s} = S_s \cdot a_g$$

Ss depends on soil characteristics (Vs) and stratigraphical sequence
Reflects spatial variation with depth and possible amplification effects

Tab. 3.2.II – *Categorie di sottosuolo che permettono l'utilizzo dell'approccio semplificato.*

Categoria	Caratteristiche della superficie topografica
A	*Ammassi rocciosi affioranti o terreni molto rigidi* caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	*Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti,* caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 m/s e 800 m/s.
C	*Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti* con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 180 m/s e 360 m/s.
D	*Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti,* con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	*Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D,* con profondità del substrato non superiore a 30 m.

NTC 2018

$$V_{S,eq} = \frac{H}{\sum_{i=1}^{N} \frac{h_i}{V_{S,i}}}$$

Categoria sottosuolo	S_S
A	1,00
B	$1{,}00 \le 1{,}40 - 0{,}40 \cdot F_o \cdot \frac{a_g}{g} \le 1{,}20$
C	$1{,}00 \le 1{,}70 - 0{,}60 \cdot F_o \cdot \frac{a_g}{g} \le 1{,}50$
D	$0{,}90 \le 2{,}40 - 1{,}50 \cdot F_o \cdot \frac{a_g}{g} \le 1{,}80$
E	$1{,}00 \le 2{,}00 - 1{,}10 \cdot F_o \cdot \frac{a_g}{g} \le 1{,}60$

Cenni introduttivi

Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento (2/2)

Interventi di mitigazione del rischio

GRADI DI DANNO - CLASSI PRESTAZIONE

TERREMOTI DI PROGETTO

La filosofia “prestazionale” richiede l’analisi della risposta di un sistema a più terremoti di diversa intensità (soddisfacimento SLE e SLU).

a) terremoto “probabile” L1 $\rightarrow$ prob. eccedenza P1 in arco temporale T_R

b) terremoto “severo” L2 $\rightarrow$ prob. eccedenza P2<P1 in arco temporale T_R

p.es. P1 =50% - P2 =10% - T_R=50 anni (vita opera)

In generale il sistema dovrà , in relazione a L1, avere una prestazione elevata (requisiti più stringenti) identificabili con “stato limite di danno” SLD;

$$T_R = \frac{-V_R}{\ln\left(1 - P_{vR}\right)}$$

Tempo di ritorno:
L1 $\rightarrow T_{R1}$=72 anni
L2 $\rightarrow T_{R2}$=475 anni

(*) GRADI DI DANNO - CLASSI PRESTAZIONE

E' necessario quindi definire a priori le prestazioni da soddisfare, ad es. sotto forma di grado di danno ammissibile.

Classe di prestazione	Terremoto di livello L_1	Terremoto di livello L_2
A	Danno I: sistema agibile	Danno I: sistema agibile
B	Danno I: sistema agibile	Danno II: sistema riparabile
C	Danno I: sistema agibile	Danno III: sistema non riparabile

GRADI DI DANNO - CLASSI PRESTAZIONE

All'aumentare della complessità dell'analisi, (***modelli lineari*** - ***lineari equivalenti*** - ***non lineari***) aumentano il **dettaglio** e **la qualità** delle **informazioni** **da ottenere** (ed ottenibili).

Classe di prestazione	Terremoto di livello L_1	Terremoto di livello L_2
A	Danno I: sistema agibile	Danno I: sistema agibile
B	Danno I: sistema agibile	Danno II: sistema riparabile
C	Danno I: sistema agibile	Danno III: sistema non riparabile

Tipo di analisi	Classe di prestazione		
	C	B	A
Pseudostatica e metodi empirici			
Dinamica semplificata			
Dinamica completa			

Metodologie di analisi e classe di prestazione dell'opera.

Legenda:

	Progettazione finale (definitiva/esecutiva)
	Progettazione preliminare o zona sismica di bassa intensità

Tabella 3.3. Metodologie di analisi e caratterizzazione geotecnica.

	Problemi geotecnici		Metodo di calcolo	Azione sismica in ingresso	Proprietà terreni	Metodi di indagine geotecnica: Stratigrafia e dati esistenti	In situ: Prove pen.	In situ: Prove geof.	Laboratorio: Prop. indice	Laboratorio: Prove statiche	Laboratorio: Prove cicliche e dinamiche P	Laboratorio: Prove cicliche e dinamiche R
Metodi empirici ed analisi pseudo-statiche	RSL		Classificazione sito	$a_{max,r}$ $S_{a,r}(T)$	Profilo strat. N_{SPT}, $q_c \rightarrow V_S$							
	LIQ		Abachi SPT/CPT		N_{SPT} / q_c							
	STP	Stabilità	Pseudo-statico - LEM	$a_{max,s}$	Resistenza terreni							
		Spostamenti	Correlazioni		a_t							
	ITS	Sistemi rigidi	Pseudo-statico – LEM Correlazioni spostamenti		Resistenza terreni e interfaccia							
		Sistemi def.	Spettro di risposta	$S_{a,s}(T)$	Curve p:y							
Analisi dinamiche semplificate	RSL		Tens. tot. 1D	$a_r(t)$	Profilo $V_S(z)$							
					Curve G:γ, D:γ							
	LIQ	Prove in situ	Abachi SPT/CPT/V_S	$a_{max,s}$	N_{SPT} / q_c / V_S							
	STP	Stabilità	Pseudo-statico - LEM	$a_{max,s}$	Resistenza terreni							
		Spostamenti	Blocco di Newmark	$a_s(t)$	a_t							
	ITS	Sistemi rigidi	Pseudo-statico Blocco di Newmark		Resistenza terreni e interfaccia							
		Sistemi def.	Spettro di risposta	$S_{a,s}(T)$	Curve p:y							
Analisi dinamiche complete	RSL	Esclusa dal modello ITS	Tens. tot. 1D/2D	$a_r(t)$	Profilo $V_S(z)$							
					Curve G:γ, D:γ							
			Tens. eff. 1D		Legame tensio-deformativo ciclico a drenaggio impedito							
		Inclusa nel modello ITS	FEM/FDM									
	LIQ	Esclusa dal modello ITS	Tens. eff. 1D	a(t) alla frontiera del dominio d'analisi								
		Inclusa nel modello ITS	FEM/FDM									
	STP	Esclusa dal modello ITS	Newmark modificato									
		Inclusa nel modello ITS	FEM/FDM									
	ITS		FEM/FDM									

LEGENDA: RSL = Risposta Sismica Locale; LIQ = Liquefazione; STP = Stabilità Pendio; ITS = Interazione Terreno-Struttura
LEM = Metodi Equilibrio Limite; FEM = Metodo Elementi Finiti; FDM = Metodo Differenze Finite
P = Prove a livelli tensio-deformativi lontani dalla rottura; R = Prove a rottura

uso imprescindibile

uso in dipendenza dalle condizioni di progetto

2

Types of analysis: a hyerarchy

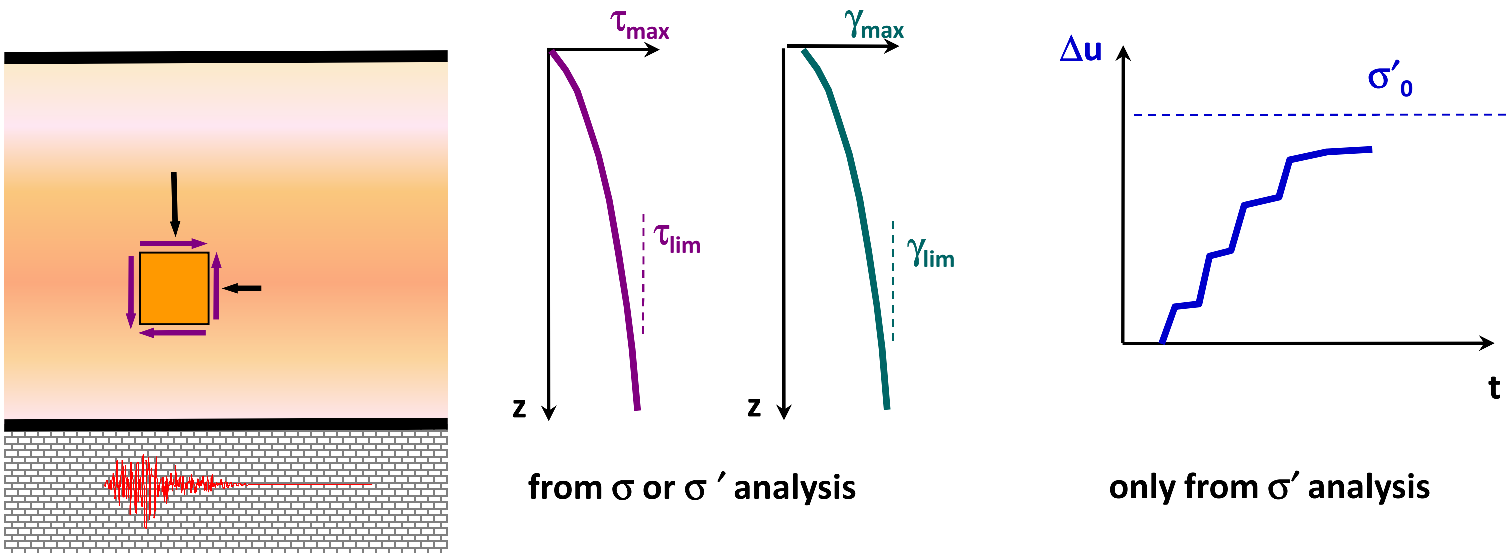
Analysis	Constitutive model	Method	Assessment
Semi-empirical	Basic (rigid-plastic)	Pseudo-static Charts	Action vs. strength
Simplified Dynamic	Simplified (visco-elastic, elasto-plastic)	Dynamic with simplified geometry	Stress, strain, pore pressure
Advanced dynamic	Advanced (elastic-plastic with hardening)	Dynamic with complex geometry (sometimes including structure)	Distributions of effective stress, pore pressure, strain, permanent displacements. Evaluation of the failure mechanism

Types of analysis: a hyerarchy

Analysis	Constitutive model	Method	Assessment
Semi-empirical	Basic (rigid-plastic)	Pseudo-static Charts	Action vs. strength
Simplified Dynamic	Simplified (visco-elastic, elasto-plastic)	Dynamic with simplified geometry	Stress, strain, pore pressure
Advanced dynamic	Advanced (elastic-plastic with hardening)	Dynamic with complex geometry (sometimes including structure)	Distributions of effective stress, pore pressure, strain, permanent displacements. Evaluation of the failure mechanism

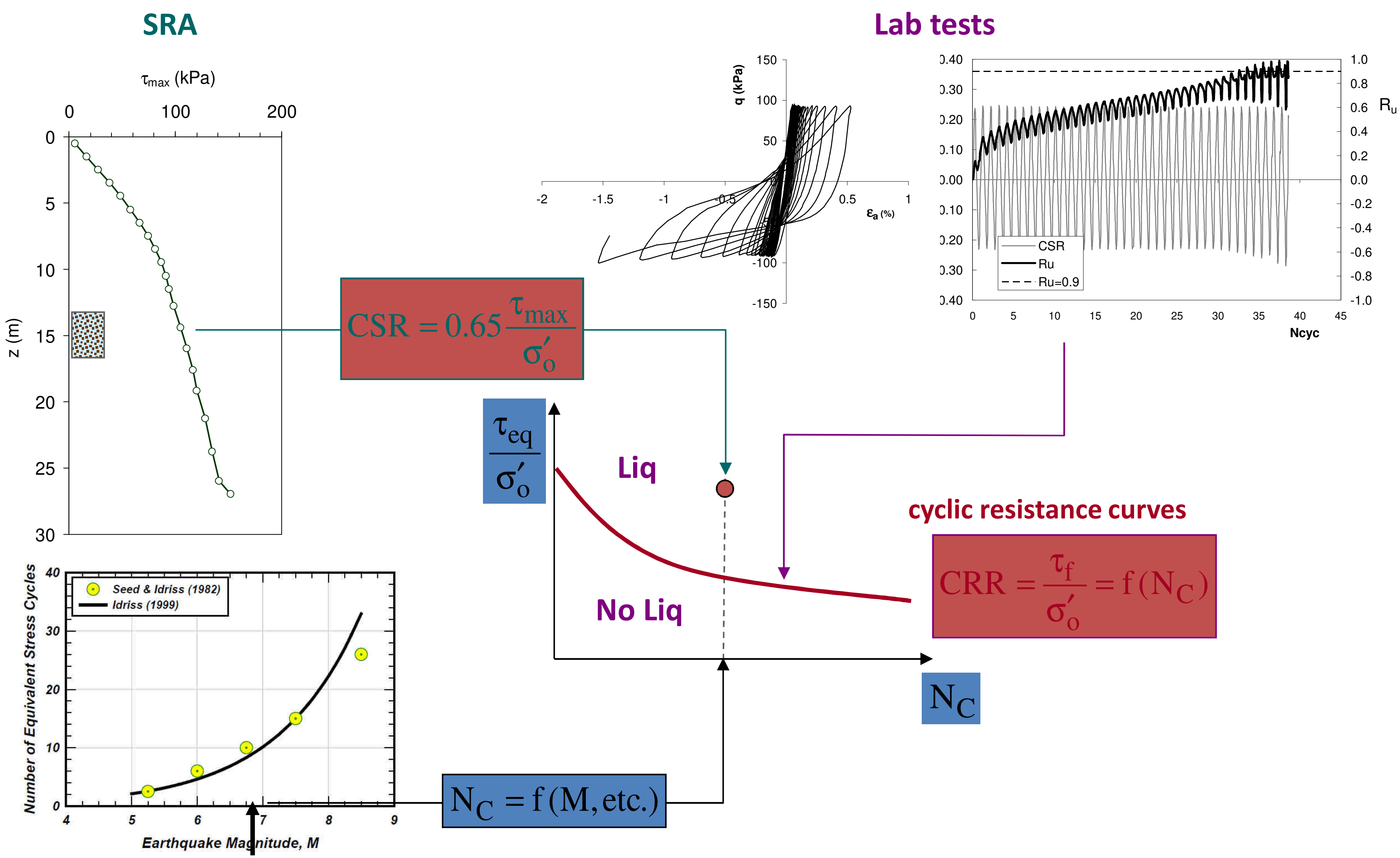
Dynamic liquefaction analysis

Step 1) evaluation of **CSR** through SRA <u>Seismic response analysis</u>



Step 2) assessment in terms of: stress ($\tau_{eq} < \tau_{lim}$), strain ($\gamma_{eq} < \gamma_{lim}$)
or (for effective stress SRA) pore pressure ratio ($\Delta u / \sigma'_0 < 1$)

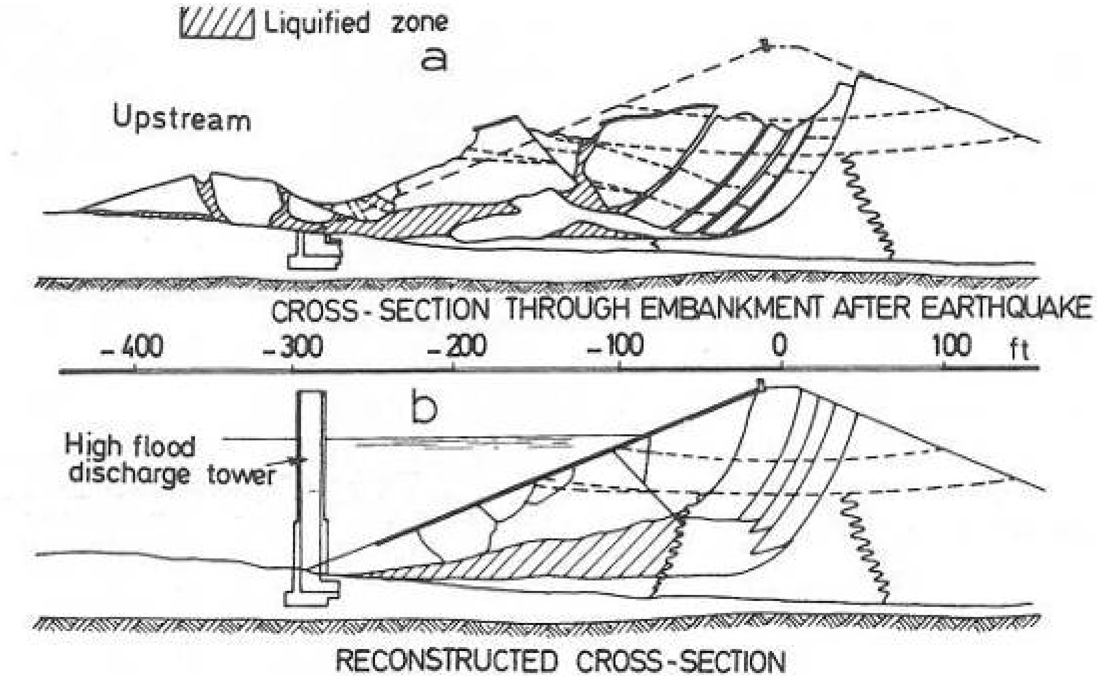
Dynamic Simplified analysis – General procedure

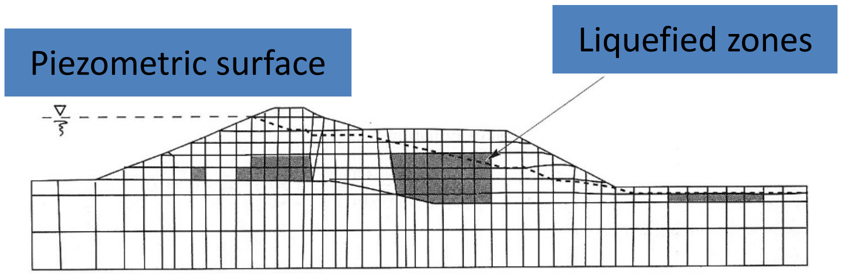


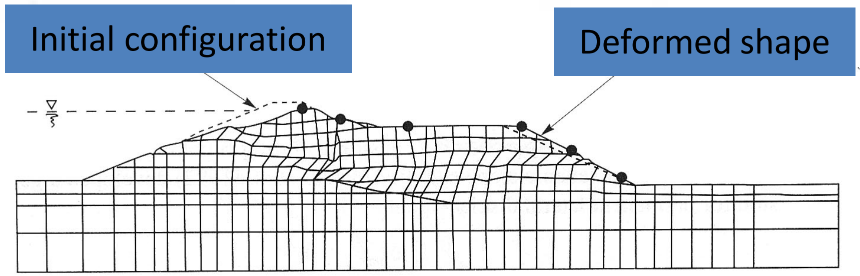
Magnitude – equivalent N_{cycles} relationship

Dynamic advanced analysis

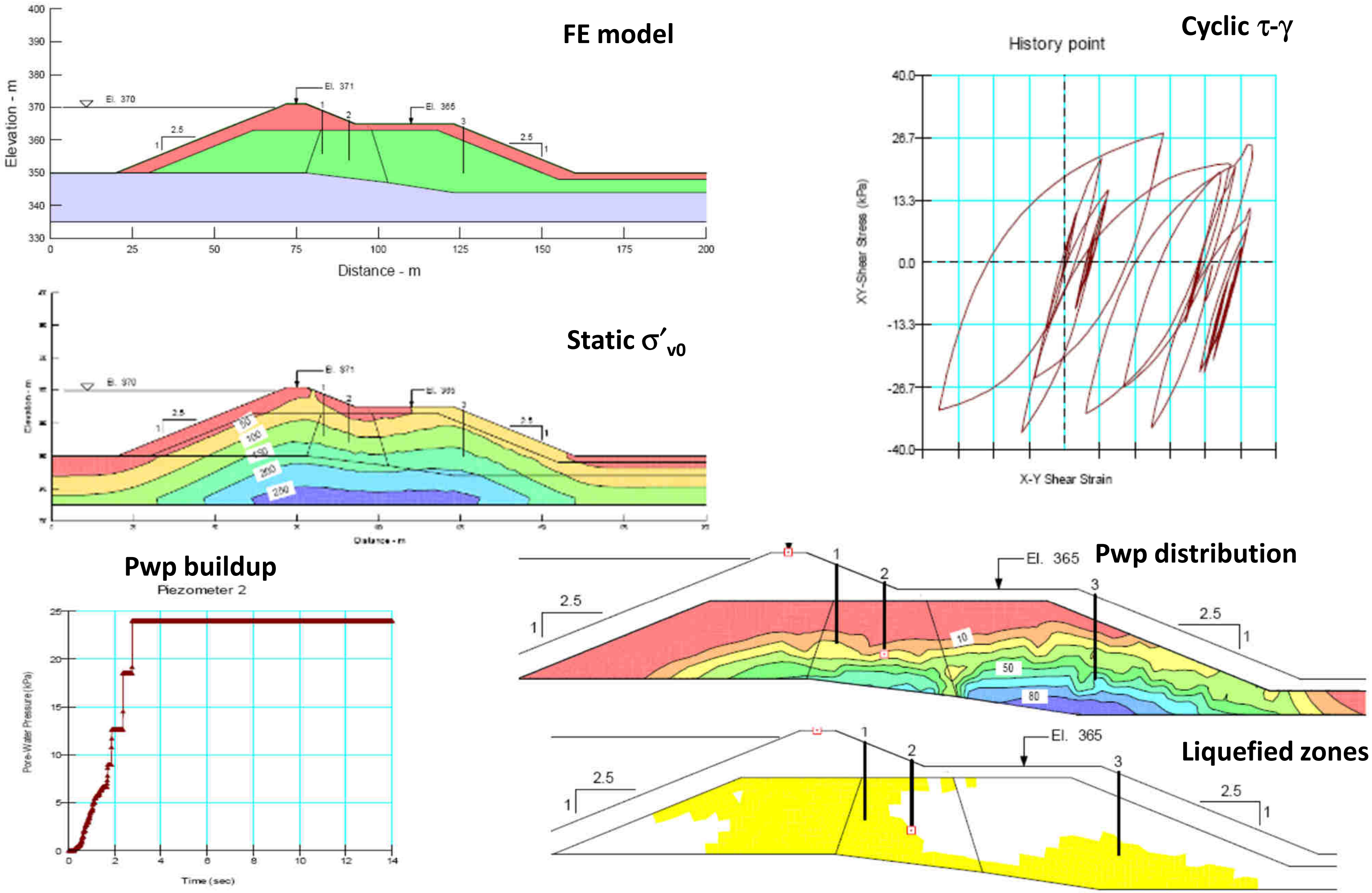
The multiphase nature of soils must be taken into account, considering the solid-fluid coupling, and with constitutive models adequately describing the soil behaviour in cyclic conditions.







Example of dynamic advanced analysis on San Fernando Dam



Cenni introduttivi

Definizioni e condizioni di innesco

Stima del potenziale di liquefacibilità del sito

Qualche approfondimento

Interventi di mitigazione del rischio

(12)P If soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures, such as ground improvement and piling (to transfer loads to layers not susceptible to liquefaction), shall be taken to ensure foundation stability.

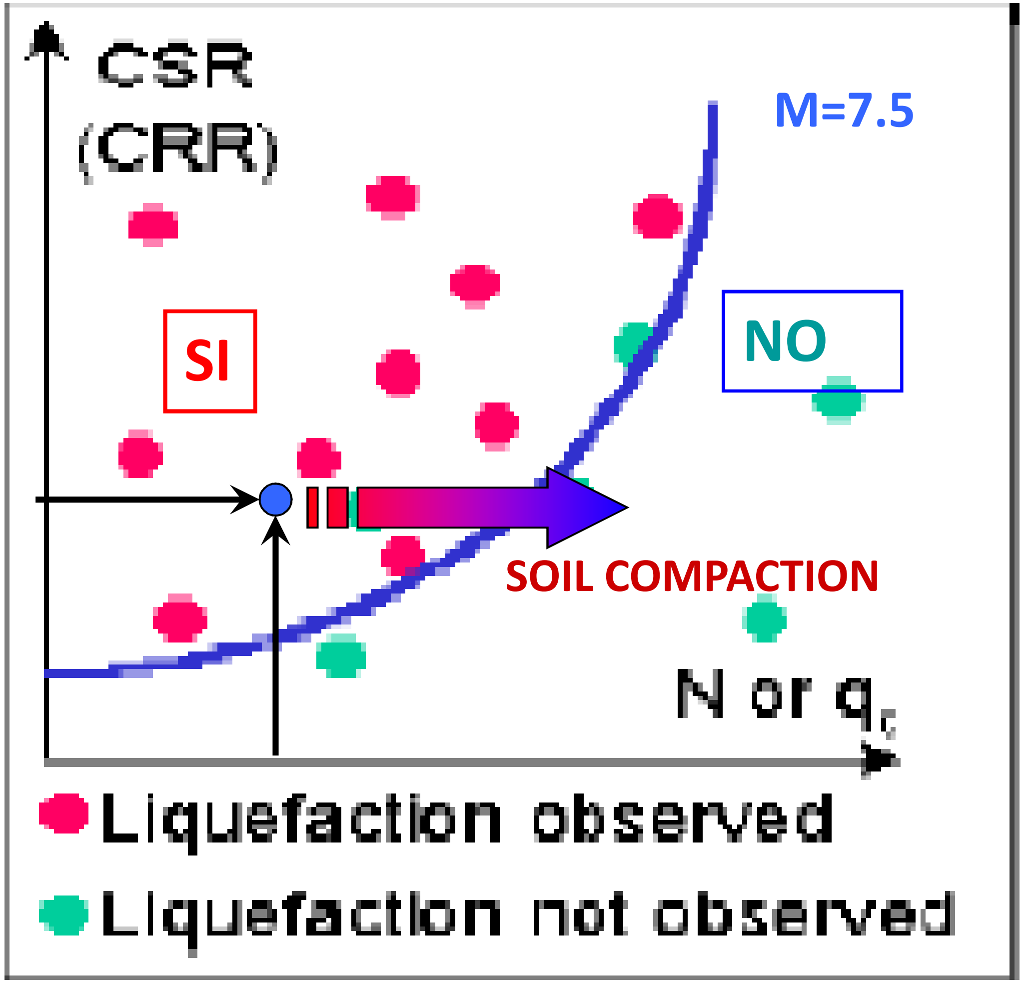
(13) Ground improvement against liquefaction should either compact the soil to increase its penetration resistance beyond the dangerous range, or use drainage to reduce the excess pore-water pressure generated by ground shaking.

NOTE The feasibility of compaction is mainly governed by the fines content and depth of the soil.

(14) The use of pile foundations alone should be considered with caution due to the large forces induced in the piles by the loss of soil support in the liquefiable layer or layers, and to the inevitable uncertainties in determining the location and thickness of such a layer or layers.

Possible mitigation works related to liquefaction.

CSR



OTHER POSSIBLE INTERVENTION APPROACHES:

- **Vertical drains**
- **Load transfer by piles**

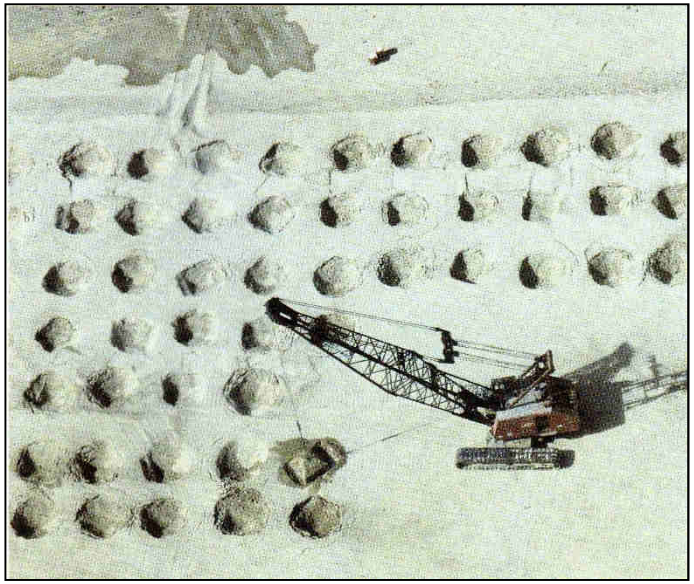
(12)P If soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures, such as ground improvement and piling (to transfer loads to layers not susceptible to liquefaction), shall be taken to ensure foundation stability.

(13) Ground improvement against liquefaction should either compact the soil to increase its penetration resistance beyond the dangerous range, or use drainage to reduce the excess pore-water pressure generated by ground shaking.

NOTE The feasibility of compaction is mainly governed by the fines content and depth of the soil.

(3) When ground improvement is implemented, reduction of liquefaction susceptibility of the deposit to an acceptable level should be demonstrated. This may be accomplished by using post-improvement in-situ tests to show increases in cyclic resistance to a satisfactory level.

COMPACTION BY HEAVY TAMPING OR DEEP DYNAMIC COMPACTION (DCC)



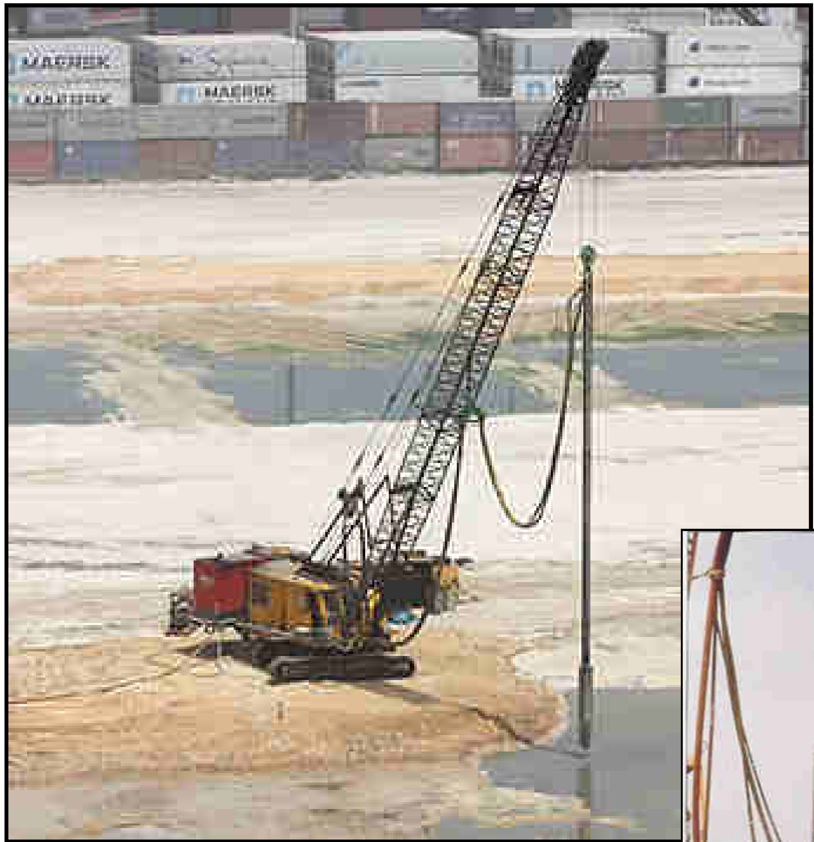
•**<u>INCREASING D_R :</u>**

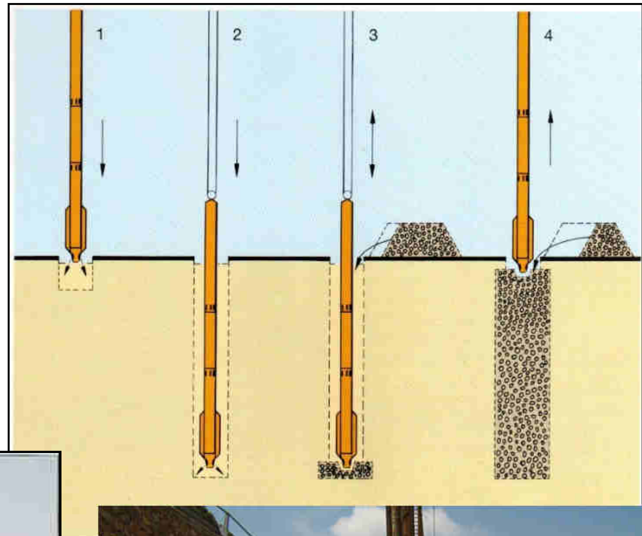
- **STRENGTH INCREASES**
- **STIFFNESS INCREASES**

SOIL COMPACTION BY VIBRATION (SANDY SOILS)

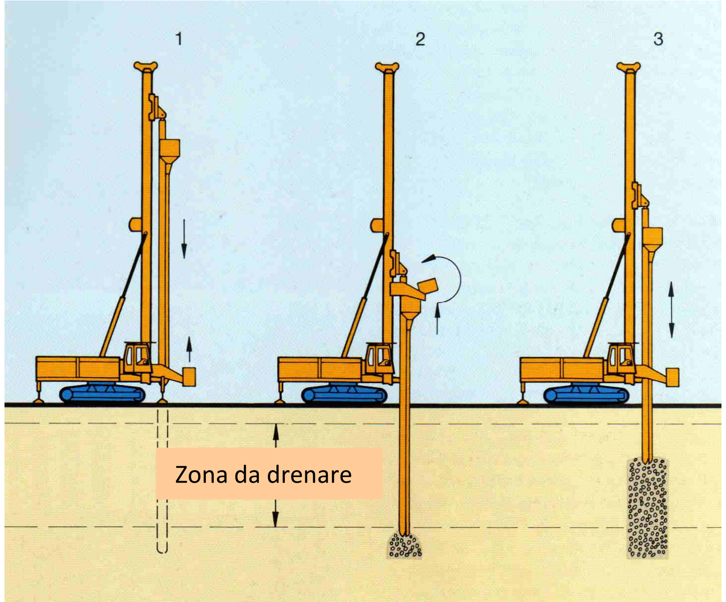
VIBROCOMPACTION : LOOSE SANDS

✓ <u>**COMPACTION AND DRAINAGE**</u>

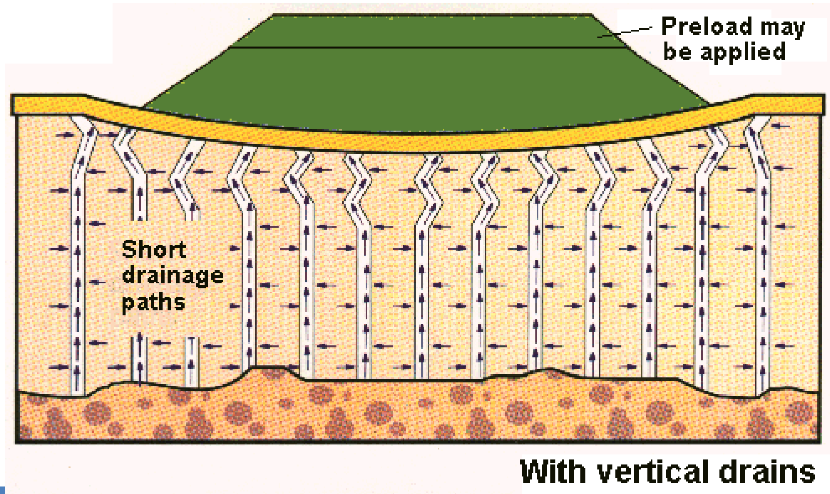




DRAINAGE BY VERTICAL DRAINS

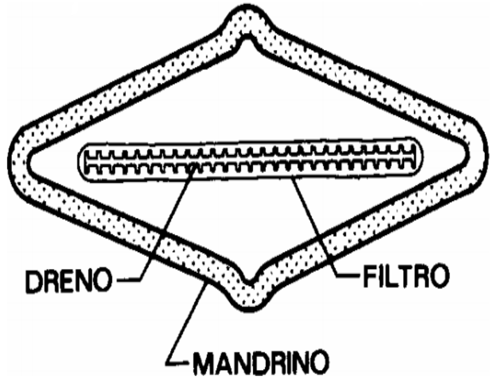


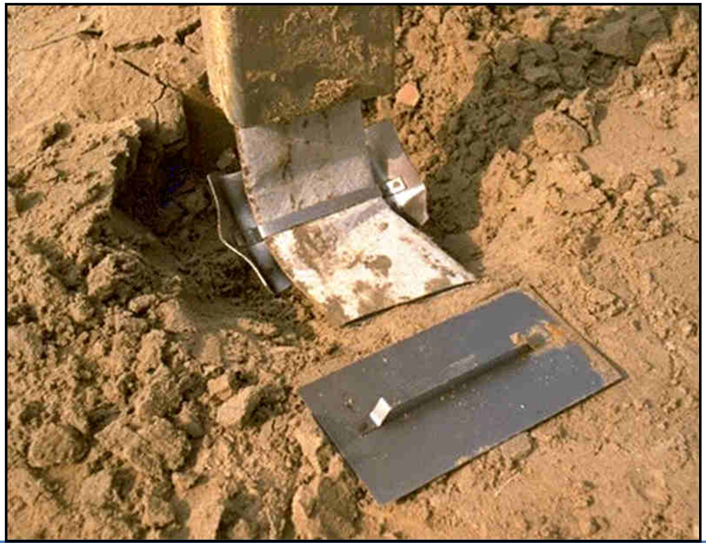
SAND COLUMNS



STRIP DRAINS (GEOSYNTHETI

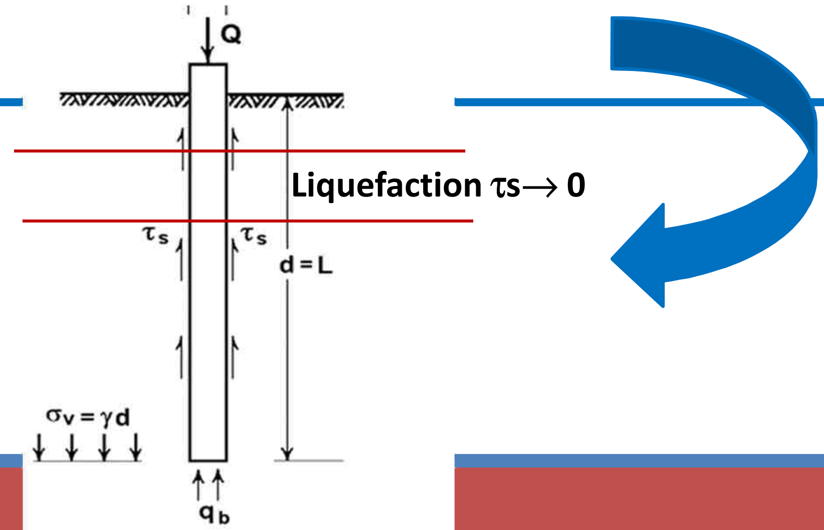
STRIP DRAINS





(12)P If soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures, such as ground improvement and piling (to transfer loads to layers not susceptible to liquefaction), shall be taken to ensure foundation stability.

(14) The use of pile foundations alone should be considered with caution due to the large forces induced in the piles by the loss of soil support in the liquefiable layer or layers, and to the inevitable uncertainties in determining the location and thickness of such a layer or layers.



Grazie per l’attenzione

✉ riccbera@gmail.com ; riccardo.berardi@unige.it