Aristotle University of Thessaloniki

Civil Engineering Department

Laboratory of Soils Mechanics, Foundation and Geotechnical Earthquake Engineering

Local Site Effects
Seismic Response
Implications to Seismic Codes

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Civil Engineering Department

Presentation's General Layout

- Introduction to Site Effects
  - Definition
  - Basic Physical Concepts
- Seismic Codes & Soil Categorization
- Estimation Methods – Theoretical approach
- Liquefaction Risk Assessment

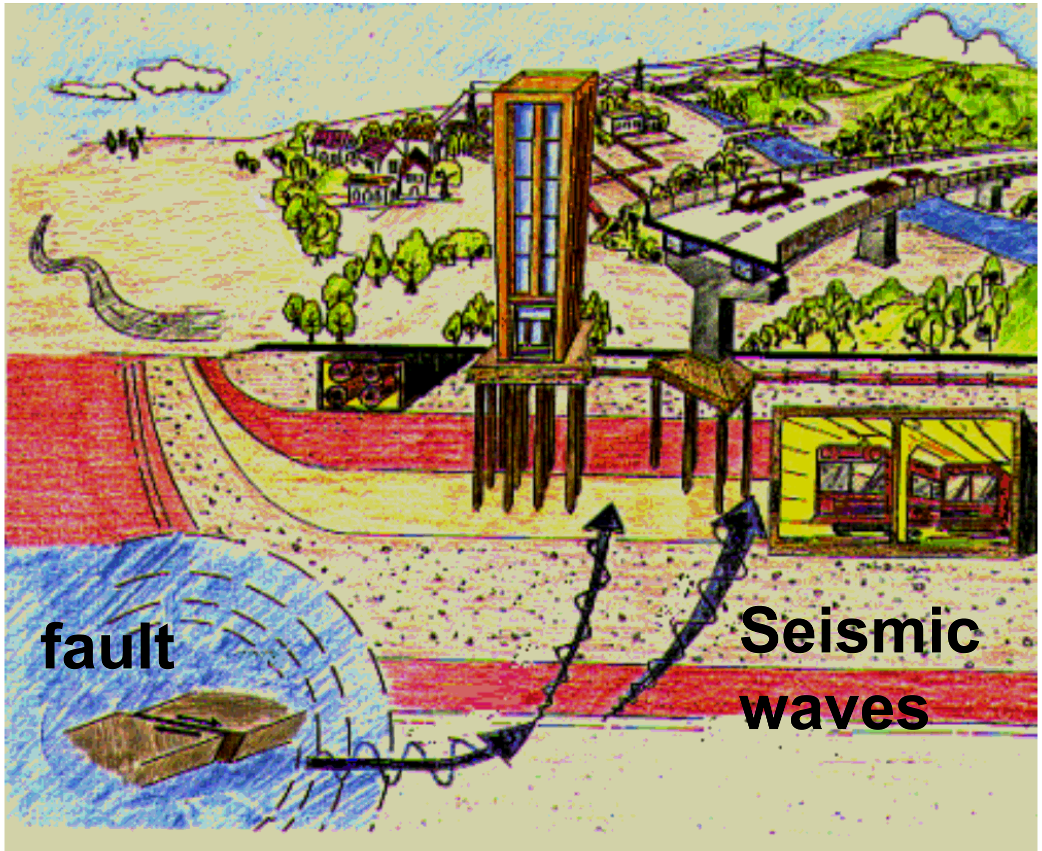
Key References :

• Steven Kramer , 1996, "Geotechnical Earthquake Engineering", Prentice Hall
• Instructional Material Complementing FEMA 451, Design Examples, Topic 15-4
• Pitilakis K.,2004, "Site Effects – Chapter 5, "Recent Advances in Earthquake Geotechnical Engineering and Microzonation, KLUWER ACADEMIC PUBLISHERS.

Contribution :

Dr. D. Raptakis, Lecturer, Civil Engineering Department
Dr. K. Makra, Researcher, Inst.of Engineering Seismology and Earthquake Engineering

‘Seismic problem’ & Geotechnical Earthquake Engineering



Typically concerned with:

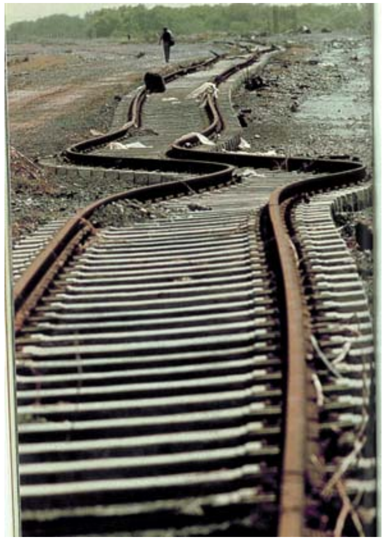
• Determining ground motions – especially as to effects of local site conditions

• Liquefaction and liquefaction-related evaluations –(settlements, lateral spreading movements, etc.)

•Seismic behavior-Design:

- Slopes/landslides evaluation
- Dams/embankments
- Design of retaining structures
- Deep and shallow foundation analysis
- Lifelines and Underground structures (tunnels, etc.)

Impact of Earthquakes



Disaster Impacts

- Economic Costs
- Humanitarian Effects
- Ecological Effects

Direct

Indirect

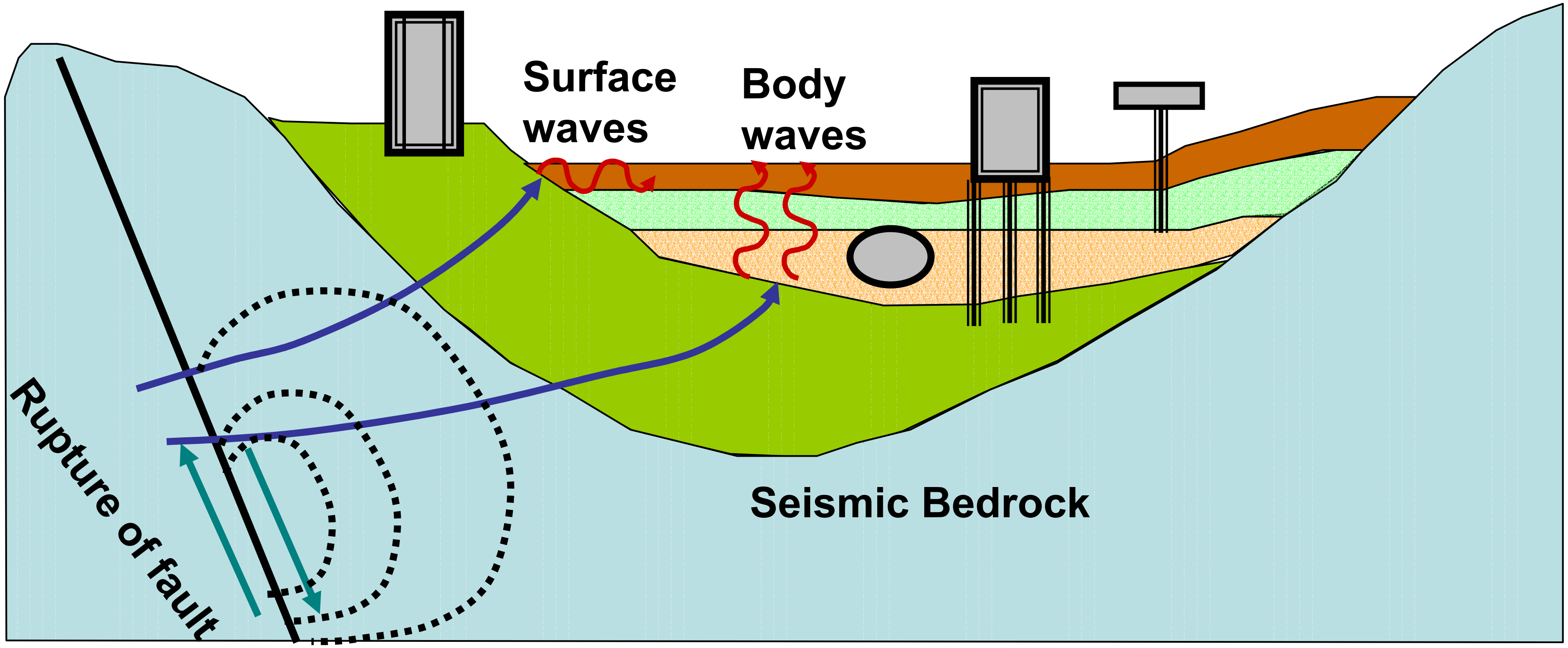
Site Effects – Some History

- *"… a movement … must be modified while passing through media of different constitutions. Therefore, the earthquake effects will arrive to the surface with higher or lesser violence according to the state of aggregation of the terrain which conducted the movement.*

Del Barrio (1855)

Introduction to Site effects

Schematic figure showing wave propagation from fault to ground surface



Earthquake recordings at soil surface include "information"

1. the source activation (fault rupture)
2. the propagation path of seismic energy
3. the effect of local geology at the recording site

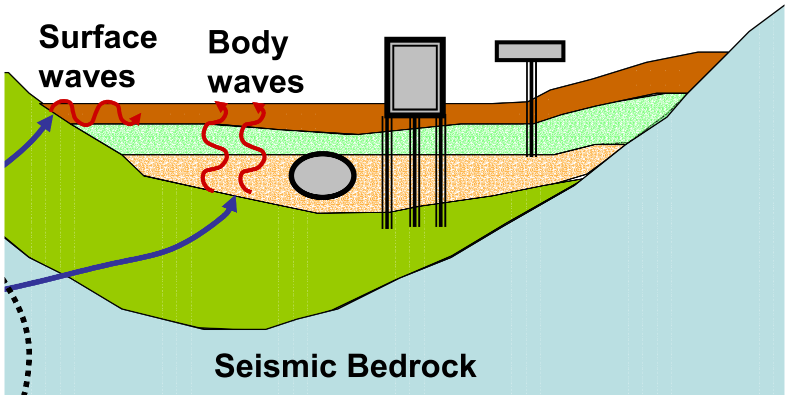
Introduction to Site effects

What do we mean with the term "**local geology**"?

- Surface soil formations
  - products of erosion, weathering and deposition processes
  - stacked in layers over more cohesive materials

- Surface topography (ridges, mountains, hills)

- Subsurface topography (valleys, basins, …)

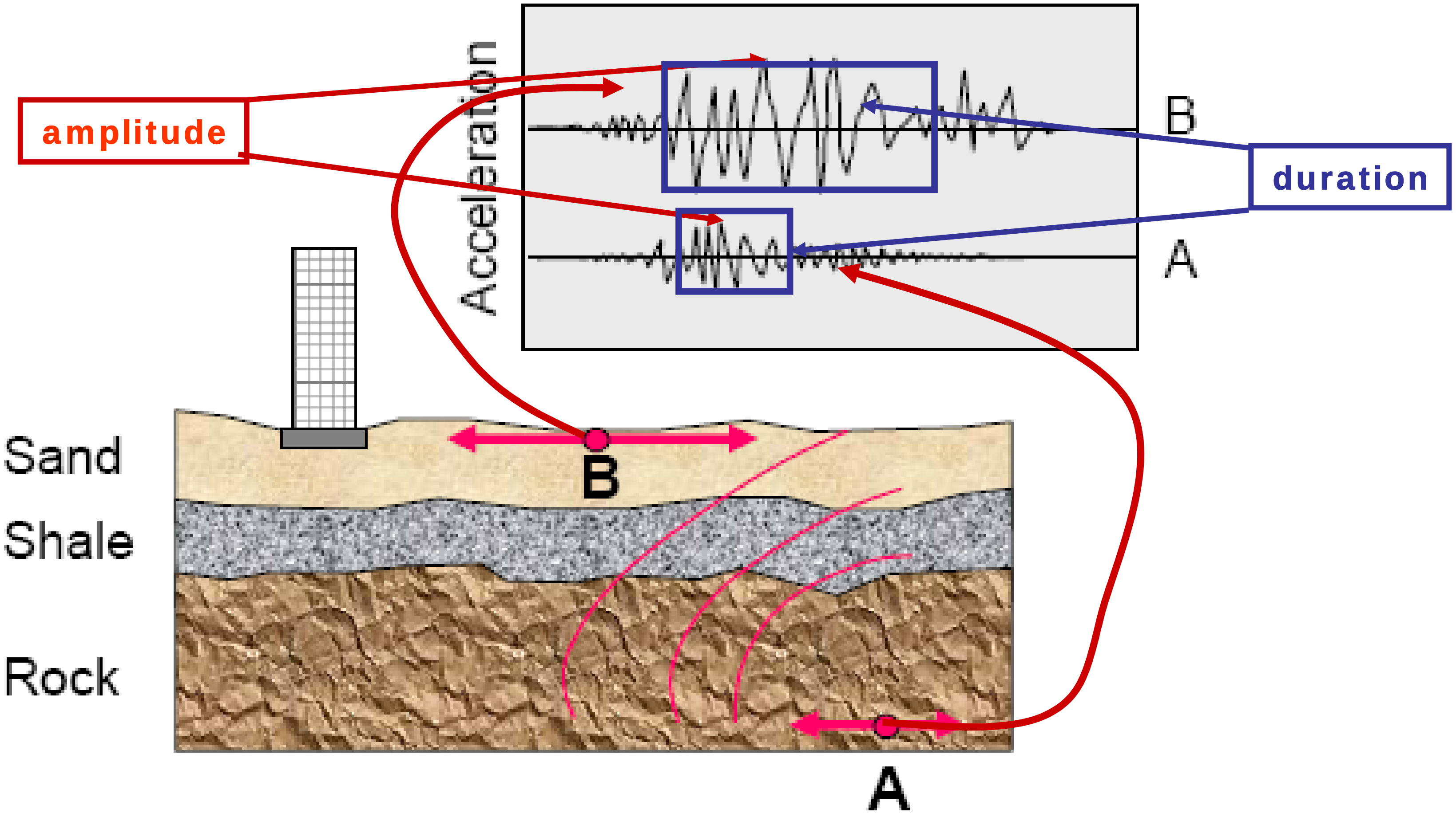


…… responsible for significant amplification and spatial variation of surface ground motion and irregular geographical distribution of damages.

Introduction to Site effects

Definition

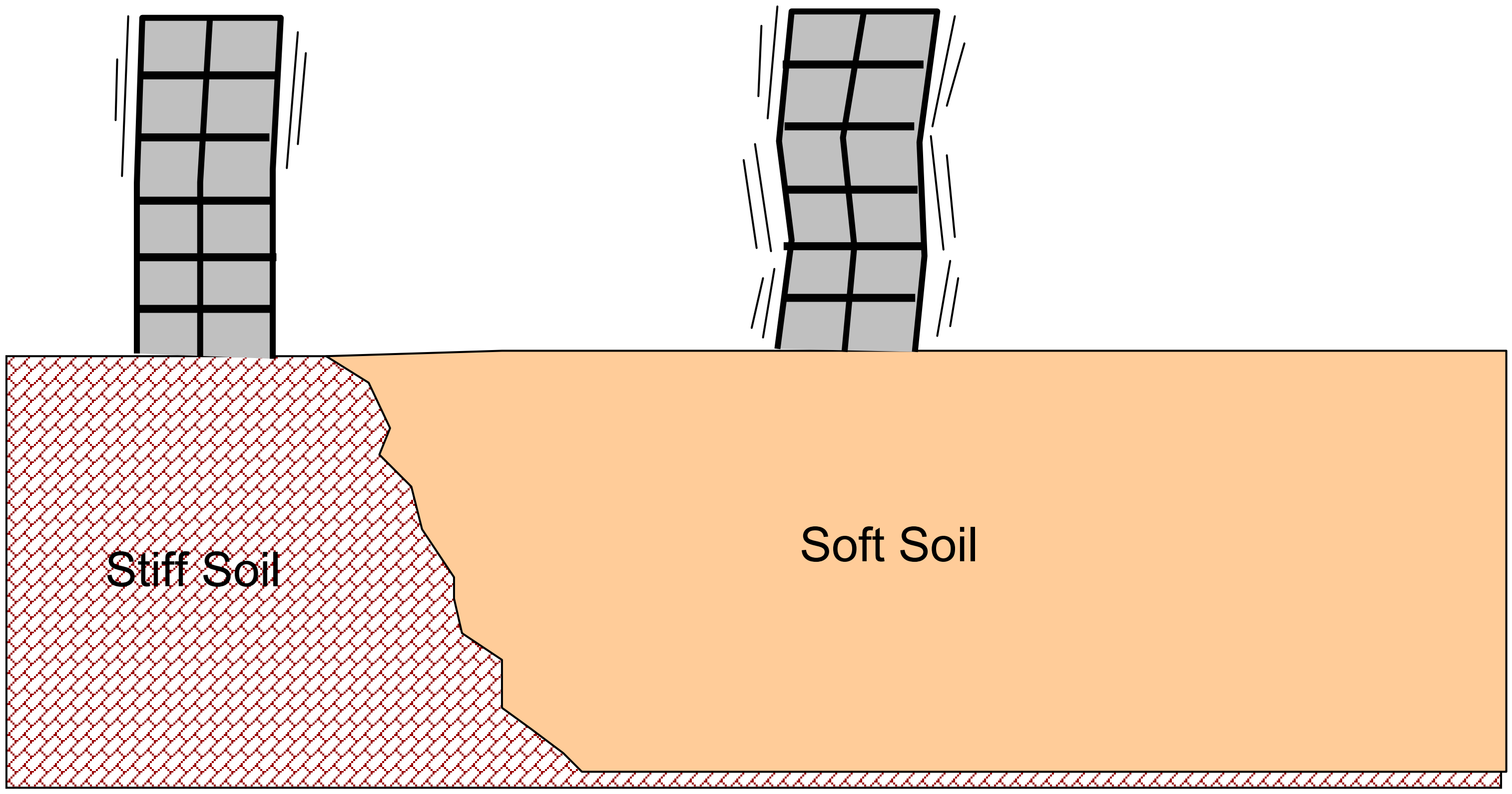
"Soil formations and topography modify the characteristics (amplitude, frequency content and duration) of the incoming wavefield having as a result the amplification or deamplification of ground motion".



Site Effects on Ground Motions

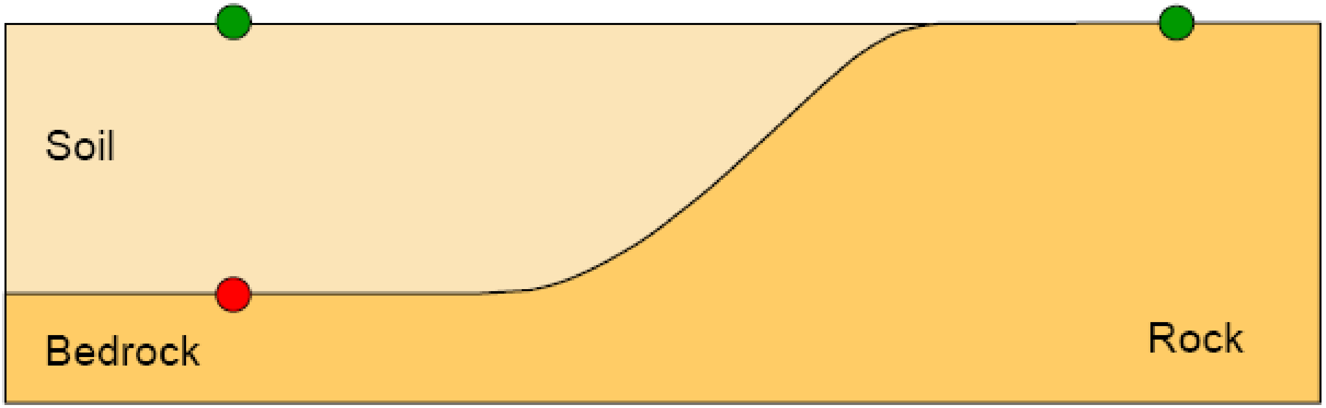
- Soil profile acts as filter
- Change in frequency content of motion
- Layering complicates the issue
- Amplification or de-amplification of ground motions can occur
- Duration of motion is increased

Site Effects on Ground Motions



Structures founded on soils, especially if soft, tend to be subjected to stronger shaking with longer-period motions.

Definition of Amplification / Deamplification



$$\text{Amplification} = \frac{\text{Free Surface}}{\text{Bedrock}}$$

$$\text{Amplification} = \frac{\text{Free Surface}}{\text{Outcrop}}$$

- Fourier amplification spectra
- Spectral amplification

$$\left| \frac{a_{\text{free surface}}(f)}{a_{\text{outcrop}}(f)} \right|$$

$$\left| \frac{S_{\text{a, free surface}}(T)}{S_{\text{a, outcrop}}(T)} \right|$$

Site Effects due to soft surface soil layers

...... the amplitude of earthquake ground motion is affected by both the properties and configuration (geometry) of the near surface soil materials through which seismic waves propagate.

These properties are **impedance** and **damping**.

the resistance, any material exhibits, to particle motion

Absorption of seismic energy

how do they affect seismic motion?

Site Effects due to soft surface soil layers

Dr. K. Makra & Dr. D. Raptakis, personal communication

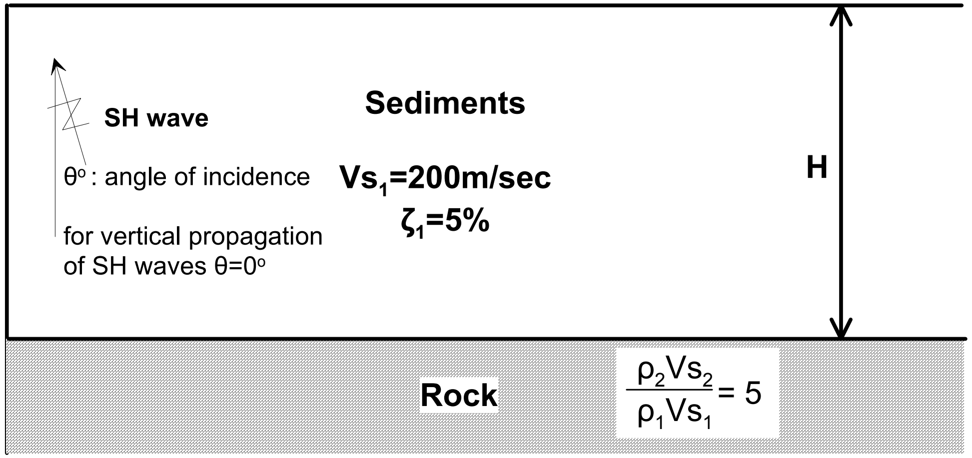
Impedance = the product of the density (ρ), the shear wave velocity (Vs) and the cosine of the angle of incidence which is defined as the angle between the vertical and the direction of seismic wave propagation

$$I = \rho \cdot Vs \cdot \cos\theta$$

$$\cos\theta \cong 1$$

⇓

$$I = \rho \cdot Vs$$



When seismic waves meet a decrease in impedance below the earth's surface, an increase in their amplitude is observed due to **resonance** as seismic waves are trapped in this layer and begin to reverberate.

The change in impedance is expressed with the impedance contrast

$$C = \frac{I_2}{I_1} = \frac{\rho_2 \cdot Vs_2}{\rho_1 \cdot Vs_1}$$

Site Effects due to soft surface soil layers

Dr. K. Makra & Dr. D. Raptakis, personal communication

damping = Absorption, anelastic attenuation

- Absorption is substantially greater on soft soils than on hard rocks
- and mitigates the increase in amplitude of seismic motion due to resonance

CONCLUSION

The fundamental phenomenon responsible for the amplification of motion over soft sediments is the trapping of seismic waves due to the impedance contrast between sediments and the underlying bedrock

The interference between these trapped waves leads to resonance

Resonance is a frequency-dependent phenomenon related with the geometrical and mechanical (density, P-wave and S-wave velocities, damping) characteristics of the soil structure.

Site Effects due to soft surface soil layers

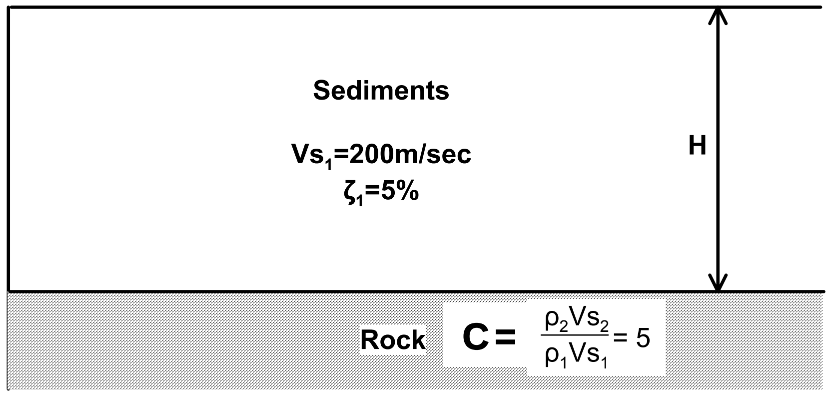
Dr. K. Makra & Dr. D. Raptakis, personal communication

Frequency domain features of the resonance phenomenon

- One horizontal layer - 1D structures

$$f_0 = \frac{Vs_1}{4 \cdot H} \quad \text{fundamental}$$

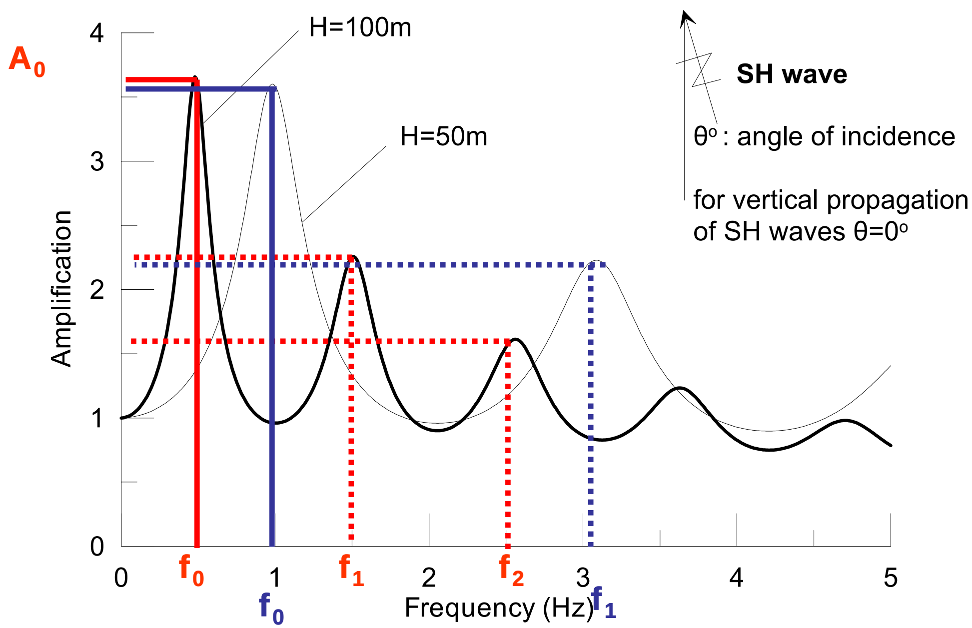
$$f_n = (2n+1) \cdot f_0 \quad \text{harmonics}$$



- f_0 = 0.2Hz - 10Hz or more

very thick deposits or extremely soft materials

very thin layers of deposits or weathered rocks



- A_0 depends on impedance contrast and material damping

$$A_0 = \frac{1}{\frac{1}{C} + 0.5 \cdot \pi \cdot \zeta_1}$$

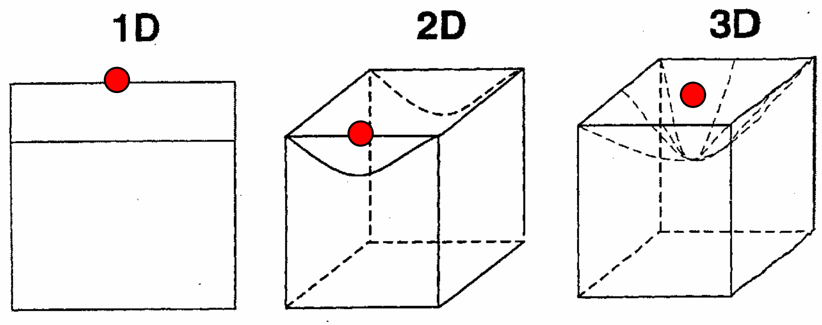
- A_0 = 6-10 (in usual cases)
- A_0 > 20 (high C value & small damping)

Site Effects due to soft surface soil layers

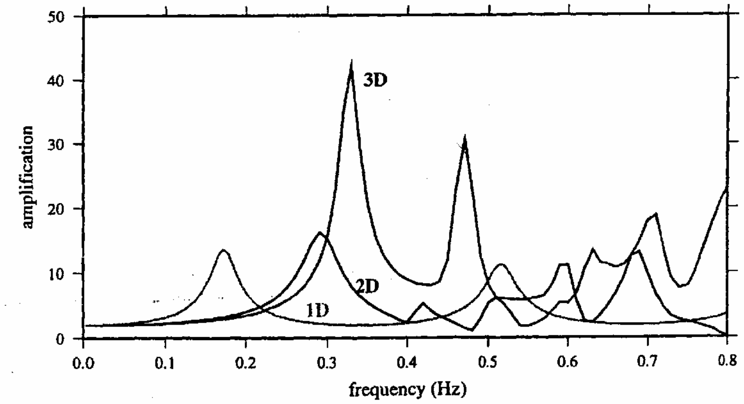
Dr. K. Makra & Dr. D. Raptakis, personal communication

Frequency domain features of the resonance phenomenon

- 2D – 3D structures
- Resonant frequencies and amplification depend also on the width of the soil structure
- Complex effects are introduced
  - consideration of the finite lateral extent
  - locally generated at the discontinuities (edges, faults, etc) and laterally propagated surface waves
- The effect of surface waves
  - $f_0=f_{0,1D}$ but $A_0>A_{0,1D}$ (shallow basins)
  - $f_0>f_{0,1D}$ and $A_0>A_{0,1D}$ (deep basins)
- The differences between 1D and 2D are much more pronounced than between 2D and 3D cases.

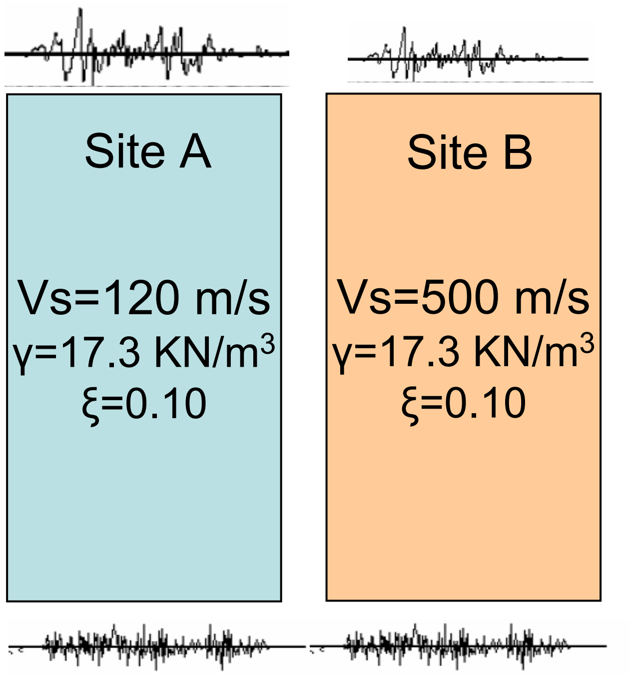


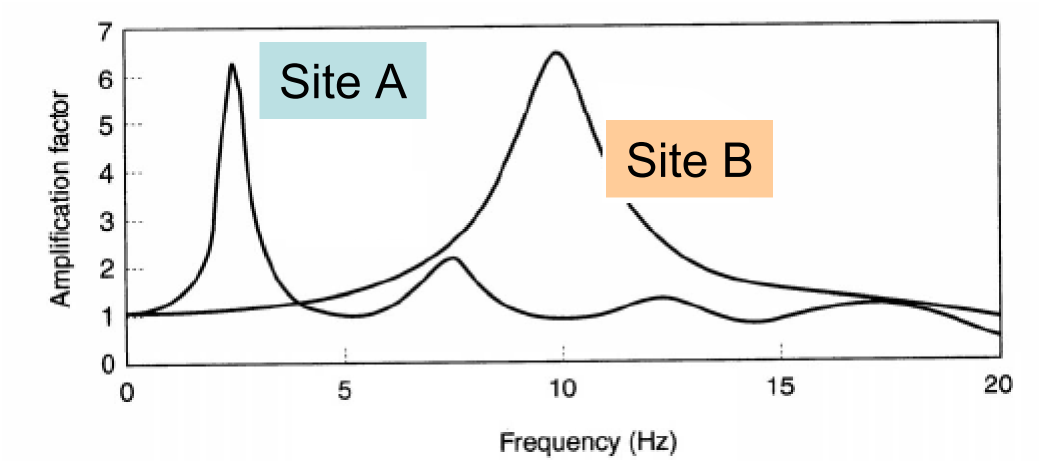
transfer functions for the central point of a sinusoidal irregularity



Site Effects due to soft surface soil layers

Time domain features of the resonance phenomenon

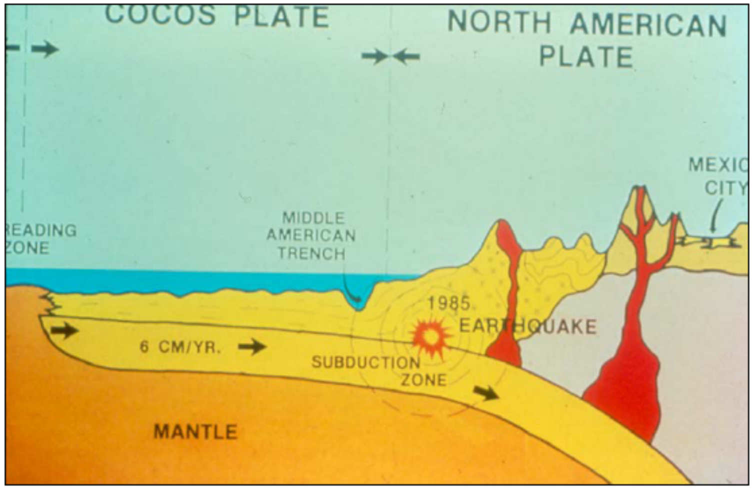




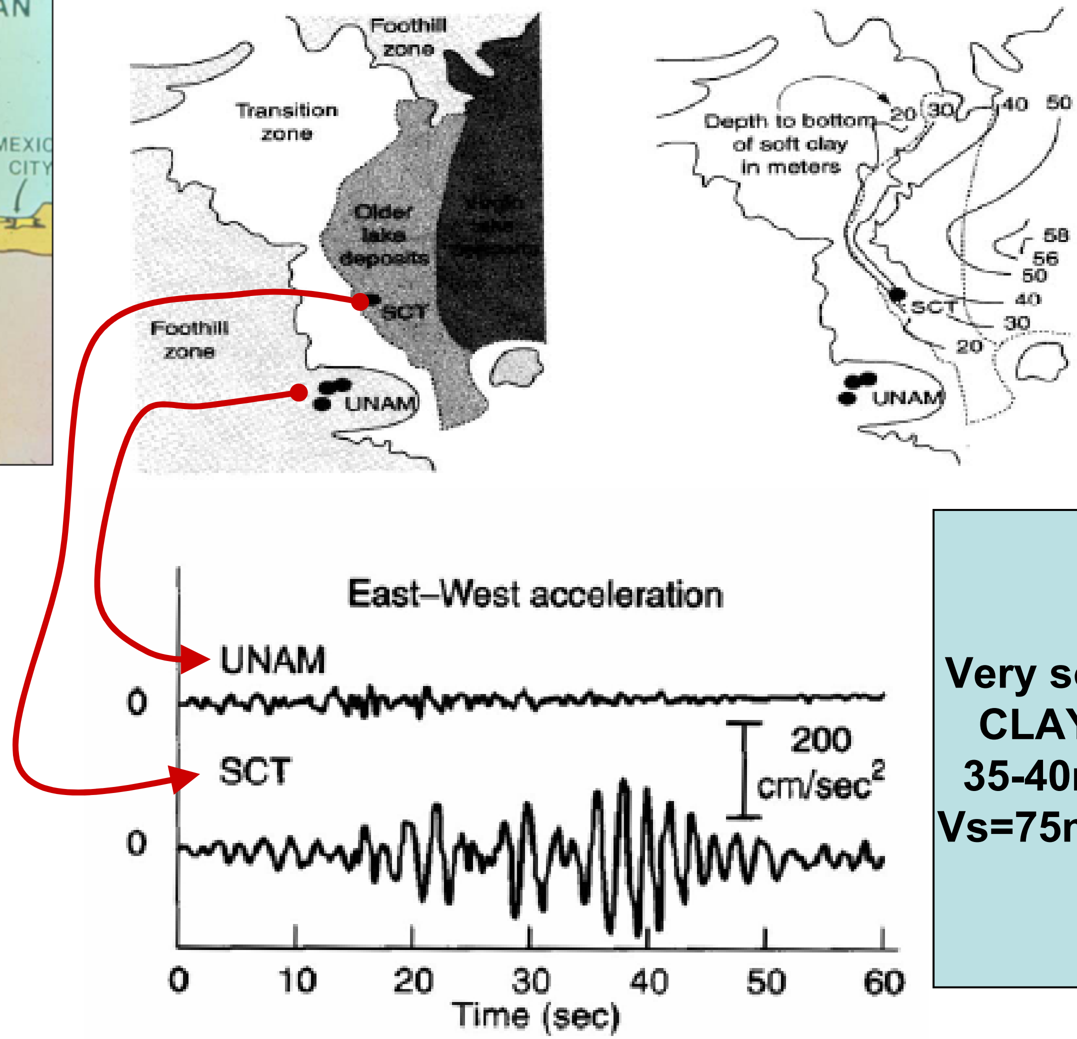
Softer Soil A will amplify low-frequency input much more strongly that will the stiffer soil of site B. At higher frequencies, the opposite behavior is expected.

Site Effects due to soft surface soil layers

1985 – Mexico City



Mexico City, 1985
Ms = 8.1 cause only moderate damage in the vicinity of the epicenter but extensive damage 350 km away.



Very soft CLAY
35-40m
Vs=75m/s

Site Effects due to soft surface soil layers

1985 – Mexico City

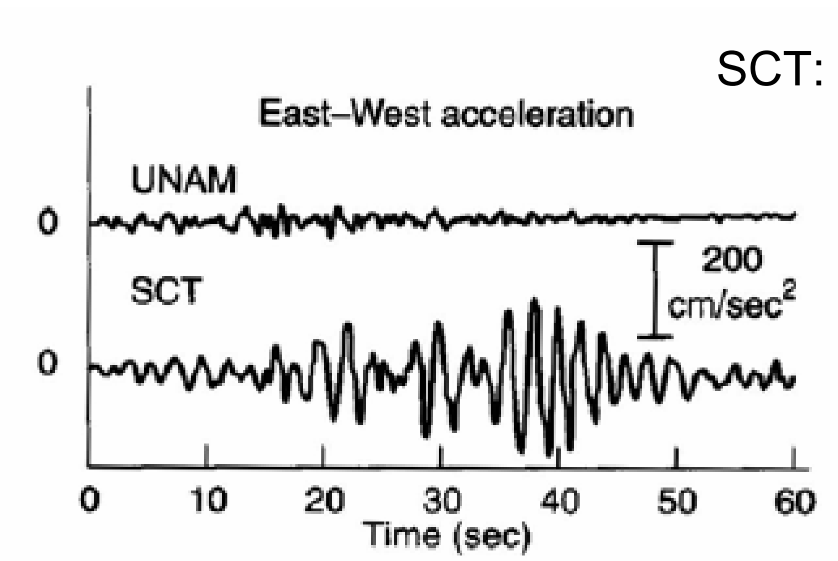
Failure of Top Floors, Hotel Continental

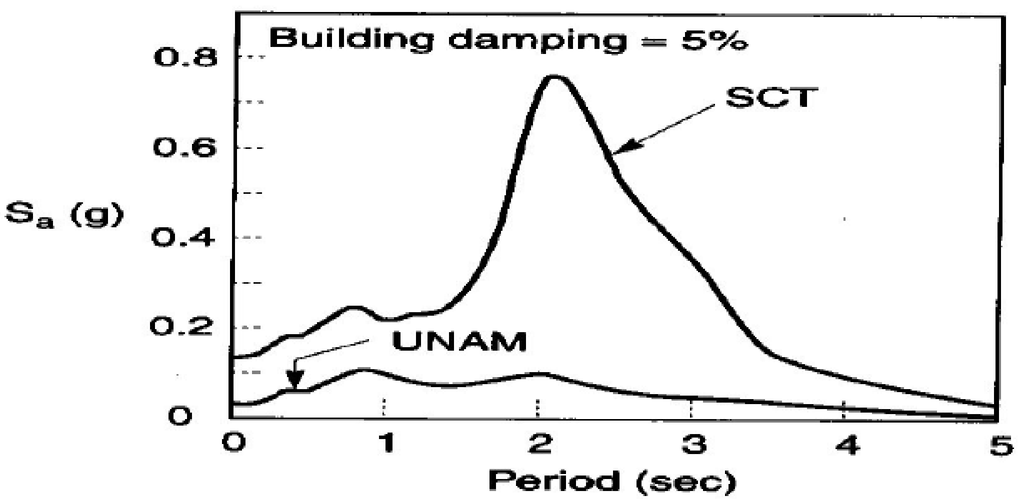
Failure of Top Floors, Hotel Continental

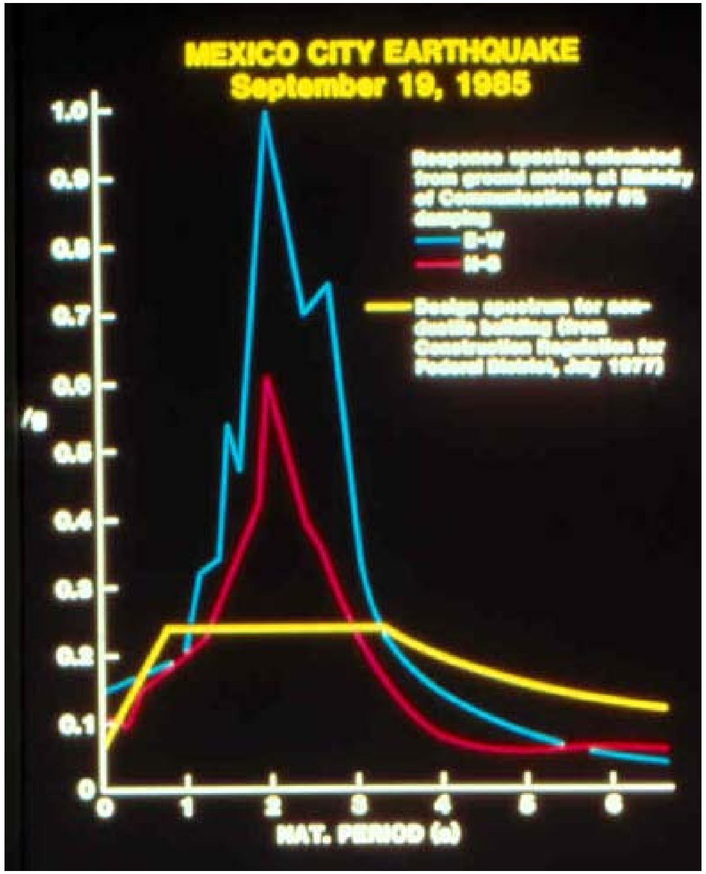
Site Effects due to soft surface soil layers

1985 – Mexico City

SCT: Site period To=4 H/Vs =4x37.5/ 75=2sec

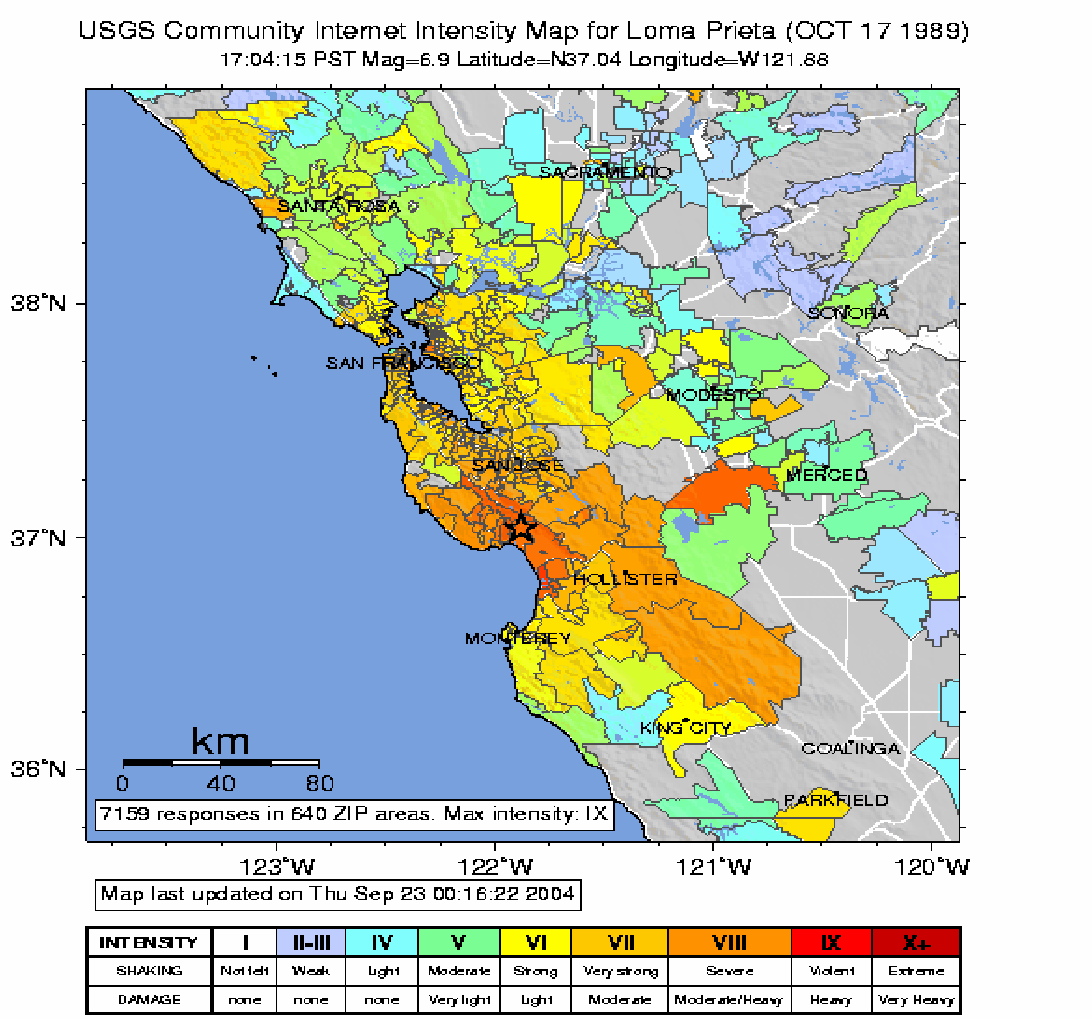


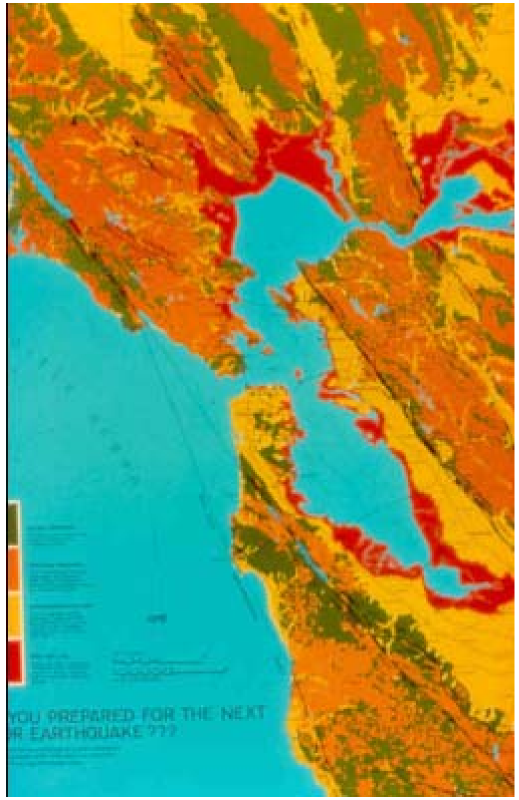




Site Effects due to soft surface soil layers

1989 – Loma Prieta

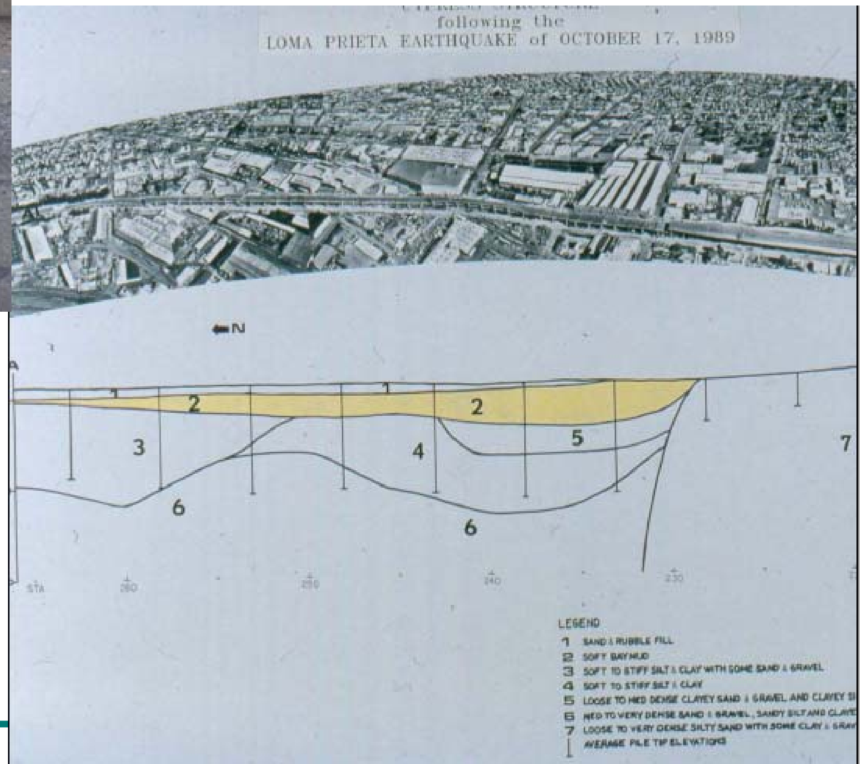




Soft deposits in red
(Bay mud)

Site Effects due to soft surface soil layers

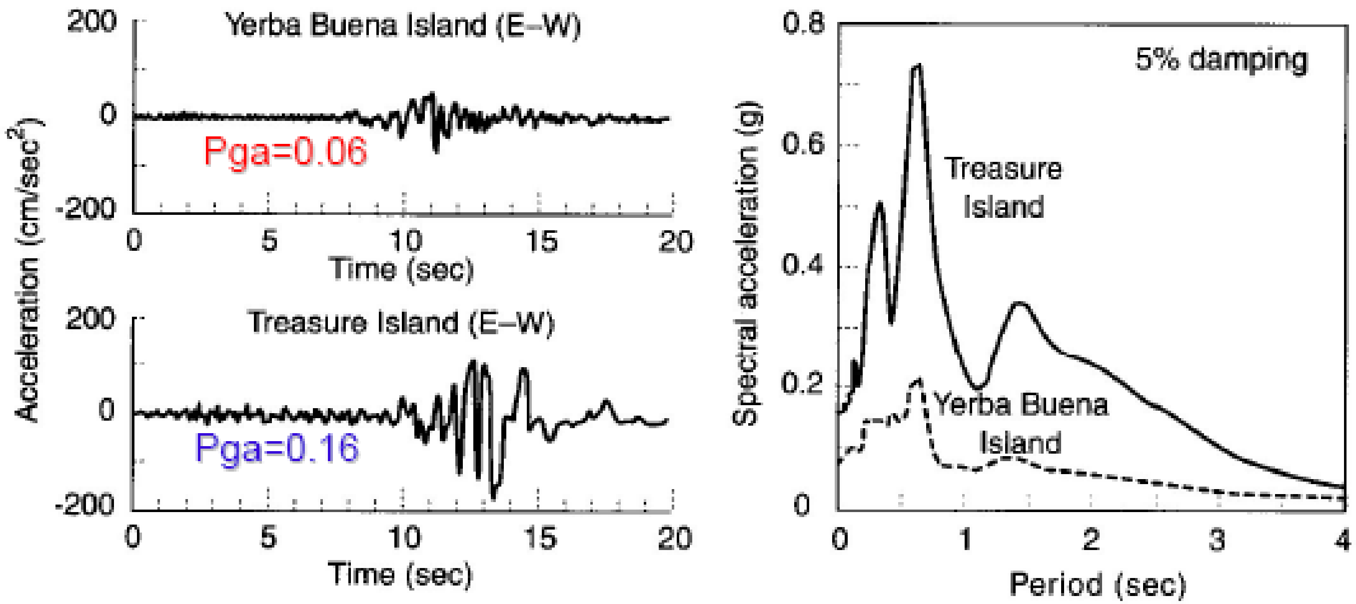
1989 – Loma Prieta



Site Effects due to soft surface soil layers

1989 – Loma Prieta

Yerba Buena Island is a rock outcrop Treasure island is 400-arce man-made hydraulic fill Underlain by 45 ft of loose sandy soil over 55 ft of San Francisco Bay Mud



The northern portion of the I-880 Cypress Viaduct that collapsed in the earthquake was underlain by San Francisco Bay Mud; the southern part that remained standing was not.

Site Effects due to soft surface soil layers

Dr. K. Makra & Dr. D. Raptakis, personal communication

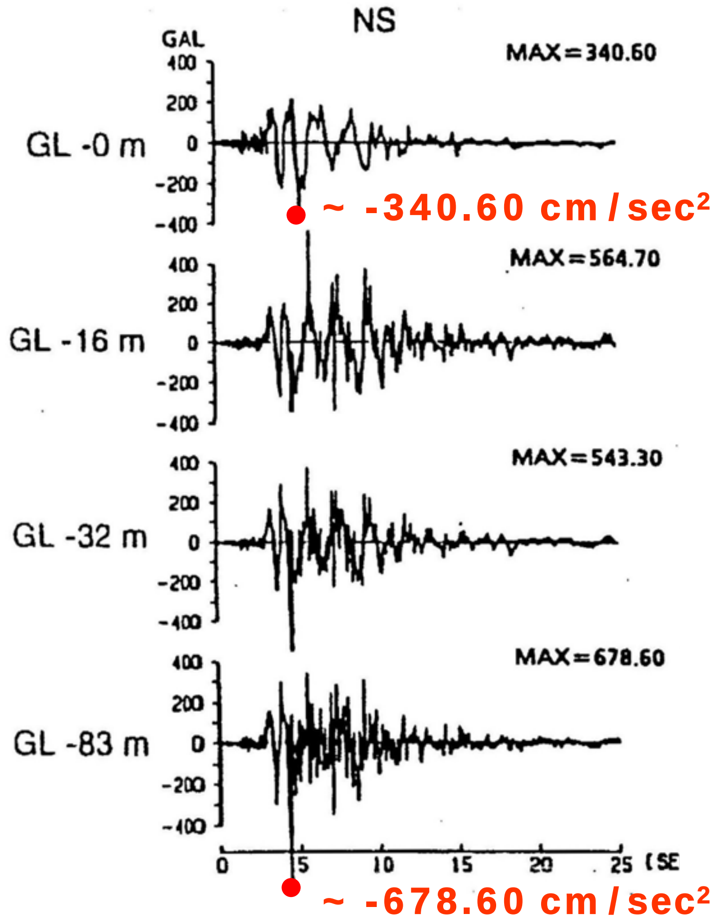
Time domain features of the resonance phenomenon

Records from recent earthquakes (Mexico, Loma Prieta, Northidge etc) showed PGAs at soil sites > 4 * PGAs at rock sites.

…… especially when f_0 of a site exceeds 2-3Hz

On the other hand ……

liquified sandy deposits induce important reduction of peak accelerations (Kobe case).



Conclusion

PGA values on sediments cannot be predicted in a straightforward manner from PGA values on rock

It depends on the input motion amplitude in combination with the non-linear behavior of soil materials

General Trend

For moderate accelerations levels (<0.2-0.3g), an amplification of PGAs is expected at soil sites relatively to rock ones

Site Effects due to soft surface soil layers

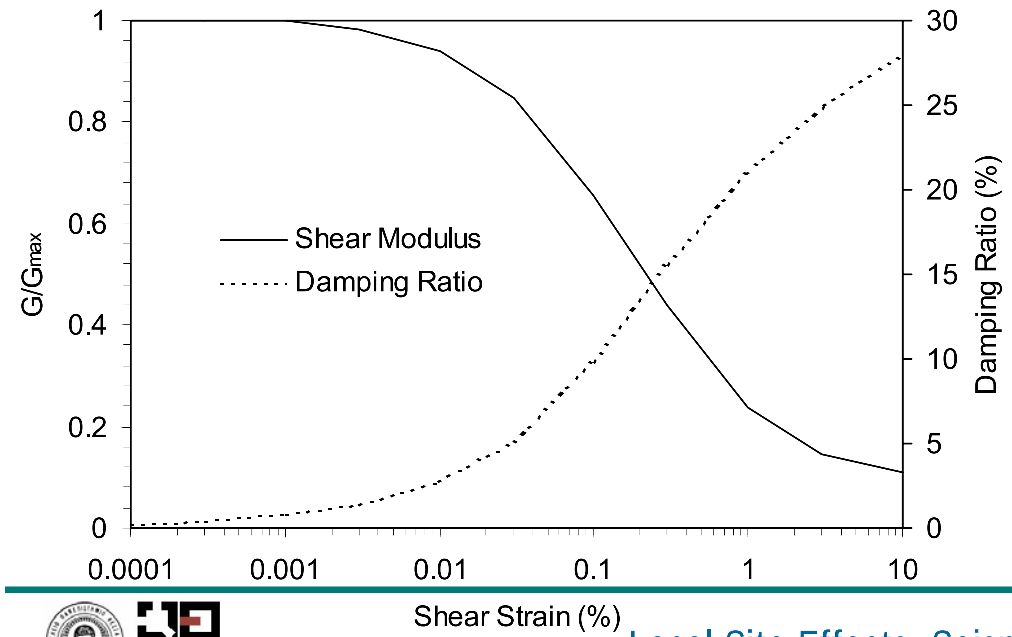
Dr. K. Makra & Dr. D. Raptakis, personal communication

Time domain features of the resonance phenomenon

This behavior of PGA amplification is attributed to

…… soils with low S wave velocity, the accumulated energy results in amplification and therefore, as the ground becomes softer, amplification becomes larger (elastic range)

…… under strong dynamic loading the ground becomes softer (shear strength decreases – nonlinear behavior)



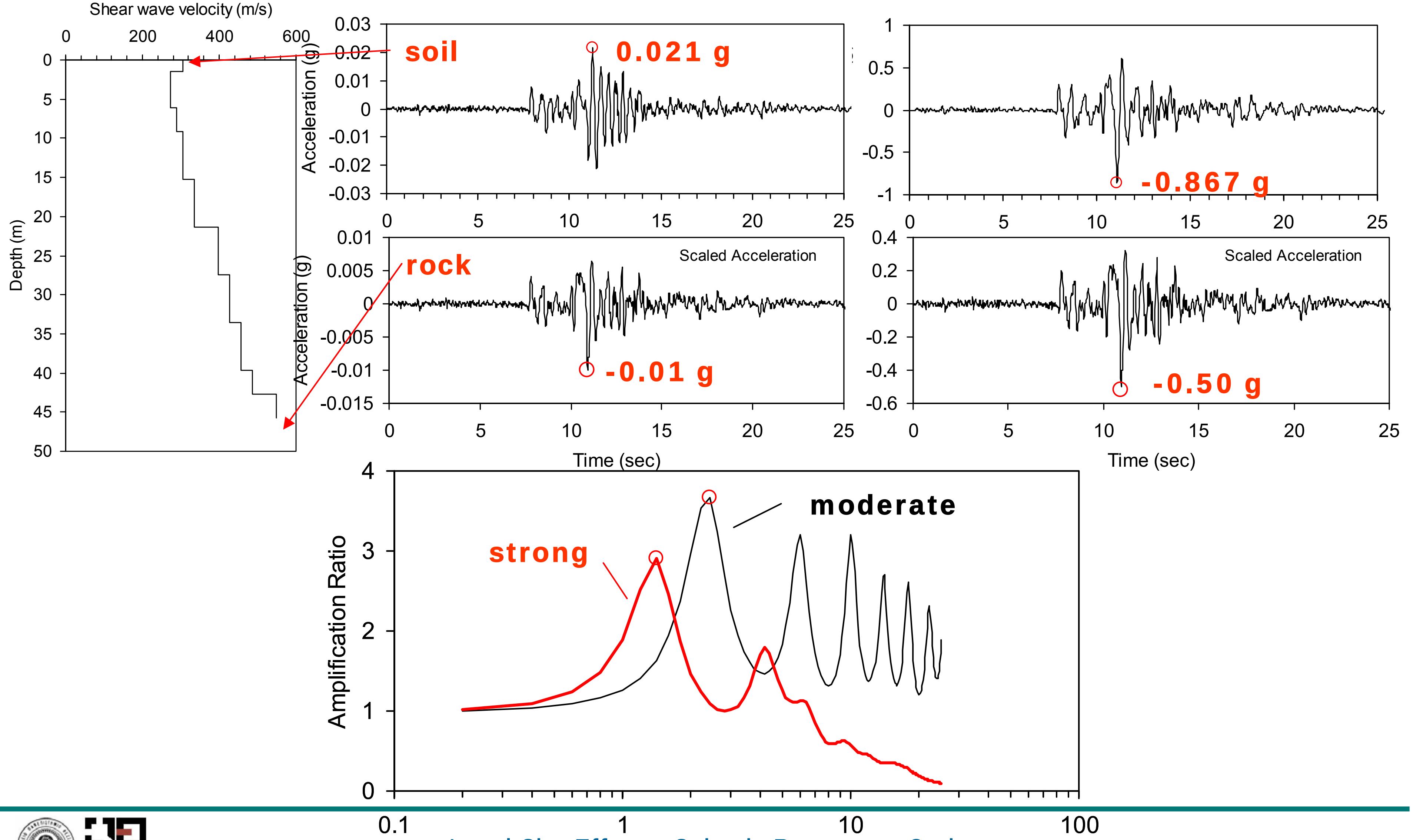
hence,

- the peak acceleration becomes smaller
- the fundamental frequency of soil profile is shifted to lower values

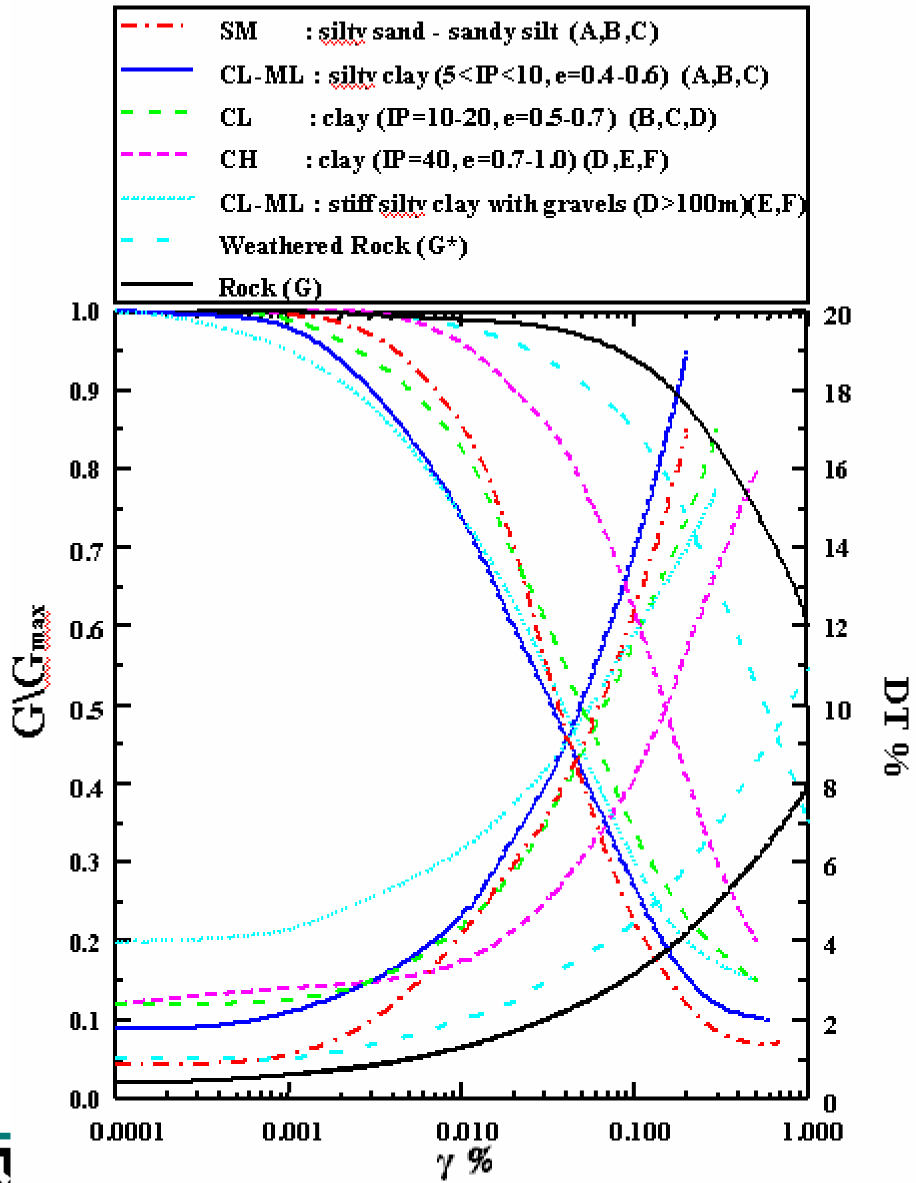
Site Effects due to soft surface soil layers

Dr. K. Makra & Dr. D. Raptakis, personal communication

$$\frac{PGA_{soil}}{PGA_{rock}} = 2.1 \qquad \frac{PGA_{soil}}{PGA_{rock}} = 1.7$$



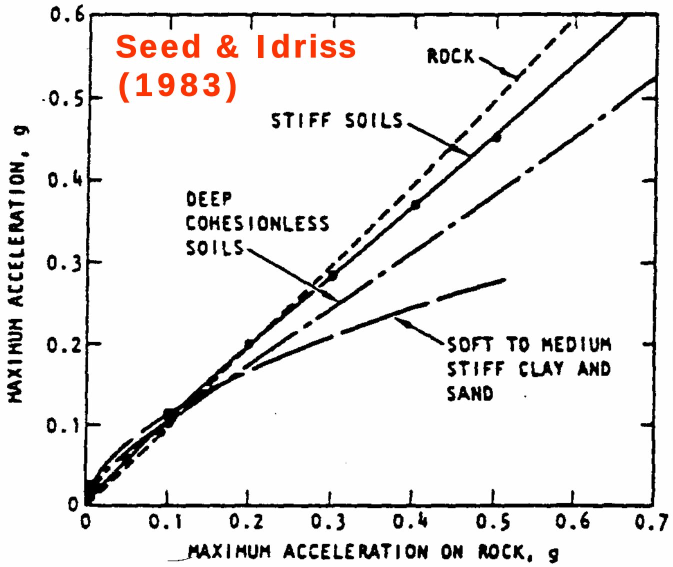
Site Effects due to soft surface soil layers & nonlinear effects

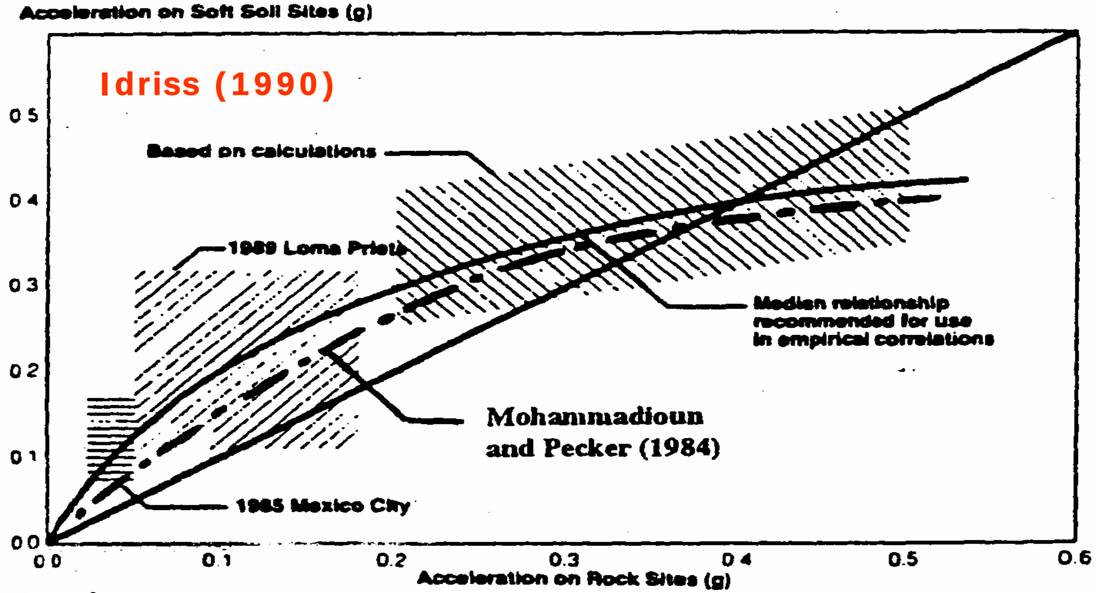


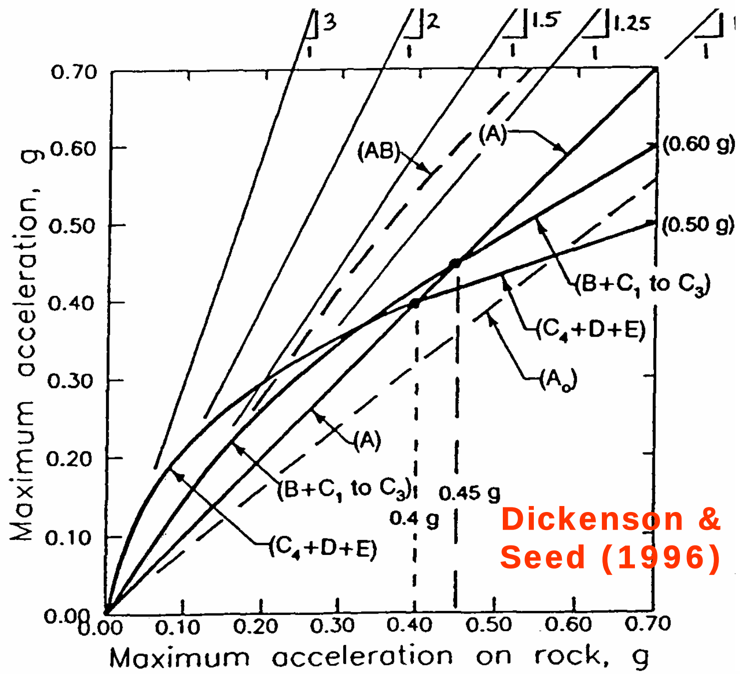
Shear modulus and damping dependency on shear strain (G/Gmax – γ% - D% curves) for the soil formations of EUROSEISTEST site

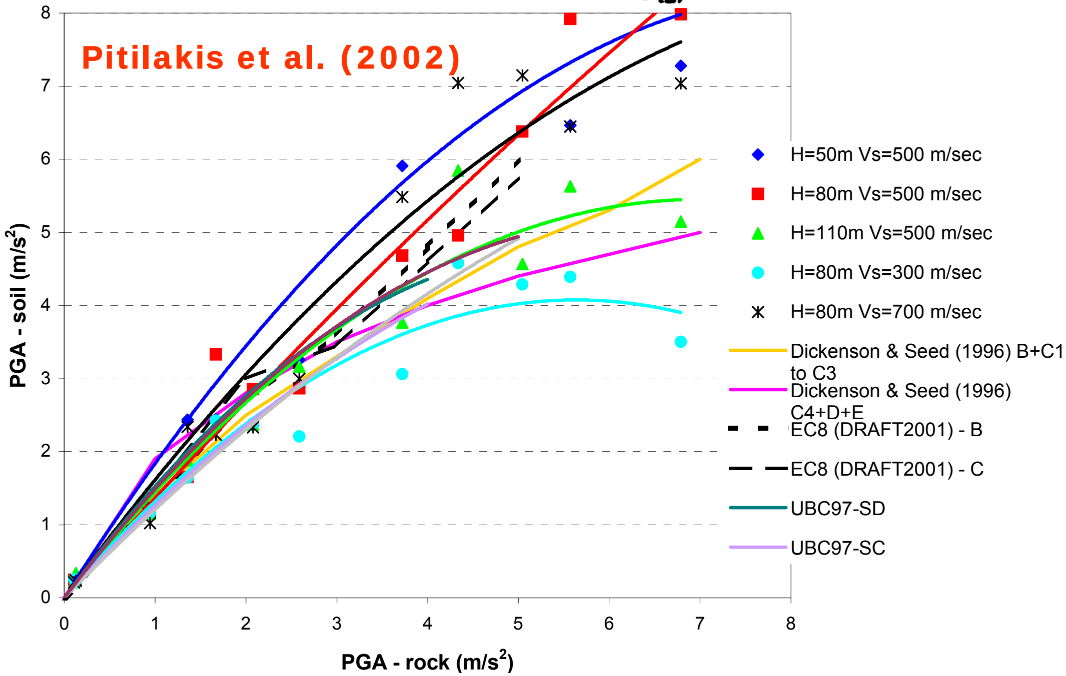
(after Pitilakis et al., 1999)

Effects of local siteconditions & nonlinear behavior of soil



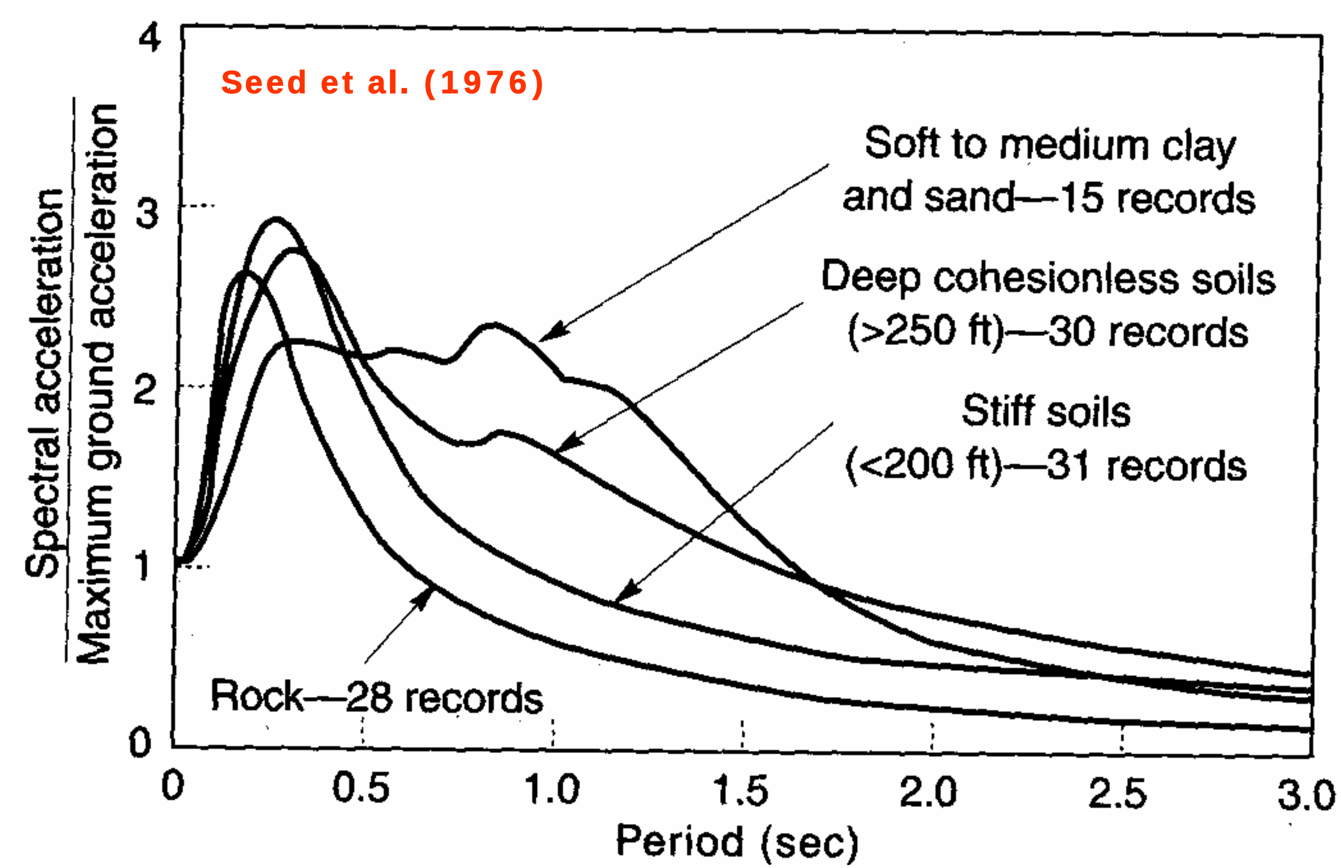






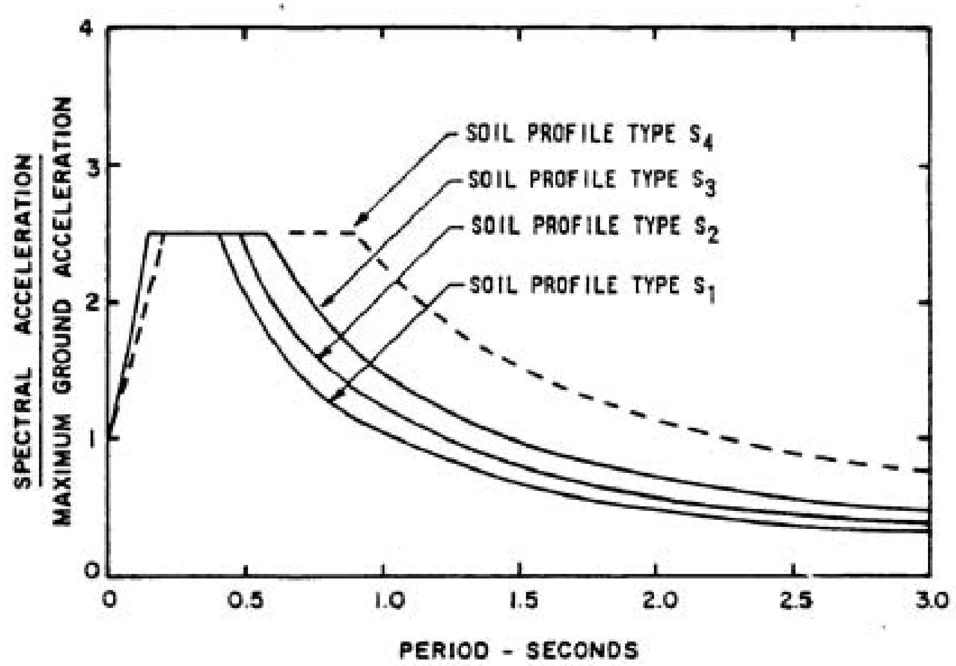
Effects of local Site conditions & nonlinear behavior of soil

Response Spectra



Effects of local site conditions & nonlinear behavior of soil

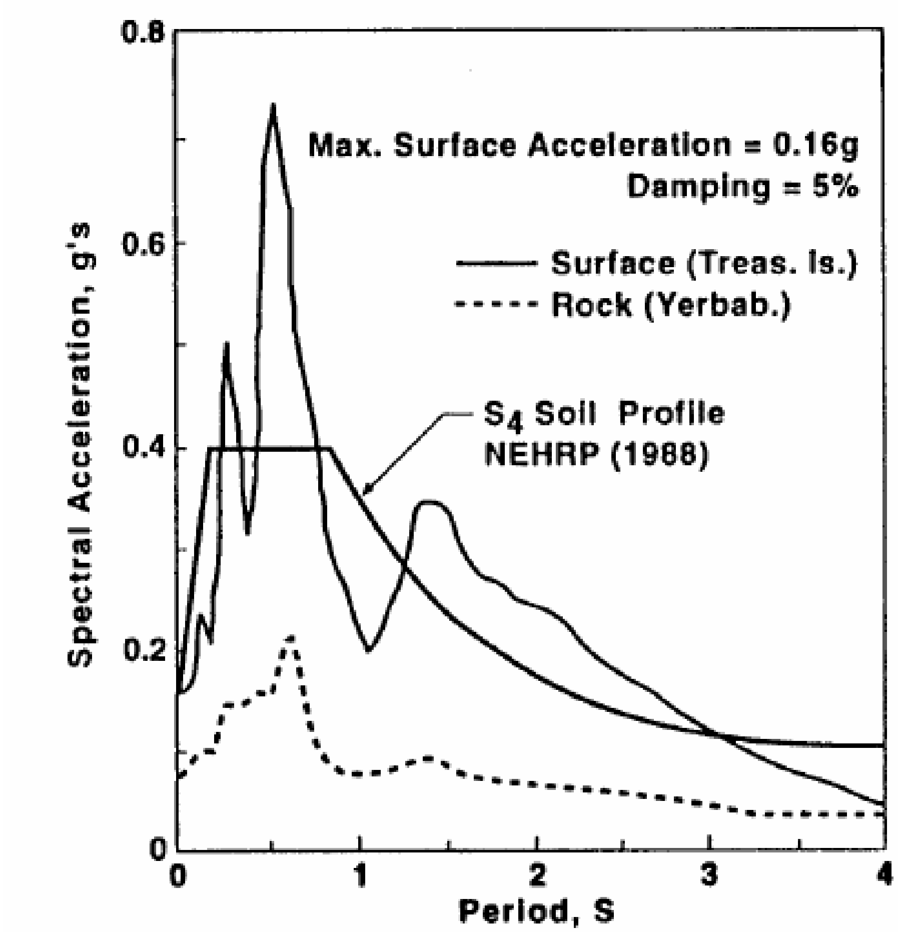
Design Response Spectra before Loma-Prieta earthquake



Soil Factor, S (NEHRP, 1988)

Type	Description	NEHRP
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet or more.	1.2
S_3	A soil profile 70 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile, characterized by a shear wave velocity less than 500 feet per second, containing more than 40 feet of soft clay.	2.0

Comparison of Response Acceleration Spectrum from 1989 Loma Prieta at deep Soft Soil Site with proposed by NEHRP-88 (S4)



This prompted the development of Category F for such soils that require site-specific analysis instead of simplified analysis (IBC 2003)

Comparison of Soil Classification in Modern Seismic Codes Worldwide

$V_{s,30}$ *(m/sec)*	180	360	760	1500	
UBC/97 I BC/2000	S_E	S_D	S_C	S_B	S_A
GREEK SEISMIC CODE EAK2000	D – C	C B A		A	
EC8 (ENV1998)	C	C B A		A	
EC8 (prEN1998) (Draft4, 2001)	D	C	B	A	
New Zealand, 2000 (Draft)	D (T>0.6s =>$V_{s,30}$<200)	C (T<0.6s =>$V_{s,30}$>200)	B	A	
Japan, 1998 (Highway Bridges)	III (T>0.6s->$V_{s,30}$<200)	II (I) (T=0.2-0.6 s -> $V_{s,30}$=200-600)		I (T<0.2s ->$V_{s,30}$>600)	
Turkey/98	$Z_4 - Z_3$	$Z_3 - Z_2$	$Z_3 - Z_2 - Z_1$	Z_1	
AFPS/90	$S_3 - S_2$	$S_3 - S_2 - S_1$	$S_1 - S_0$	S_0	

IBC 2003- Site Classification

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\overline{v}_s$, (ft/s)	Standard penetration resistance, $\overline{N}$	Soil undrained shear strength, $\overline{s}_u$, (psf)
A	Hard rock	$\overline{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \overline{v}_s \leq 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \overline{v}_s \leq 2{,}500$	$\overline{N} > 50$	$\overline{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \overline{v}_s \leq 1{,}200$	$15 \leq \overline{N} \leq 50$	$1{,}000 \leq \overline{s}_u \leq 2{,}000$
E	Soft soil profile	$\overline{v}_s < 600$	$\overline{N} < 15$	$\overline{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\overline{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa.

IBC 2003- Site Classification and Spectral Amplification Factors

TABLE 1615.1.2(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS
AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_S)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_S.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS
AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

Discussion on site effects and soil categorization: EC8

Shear wave velocity - upper 30m

$$V_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_i}}$$

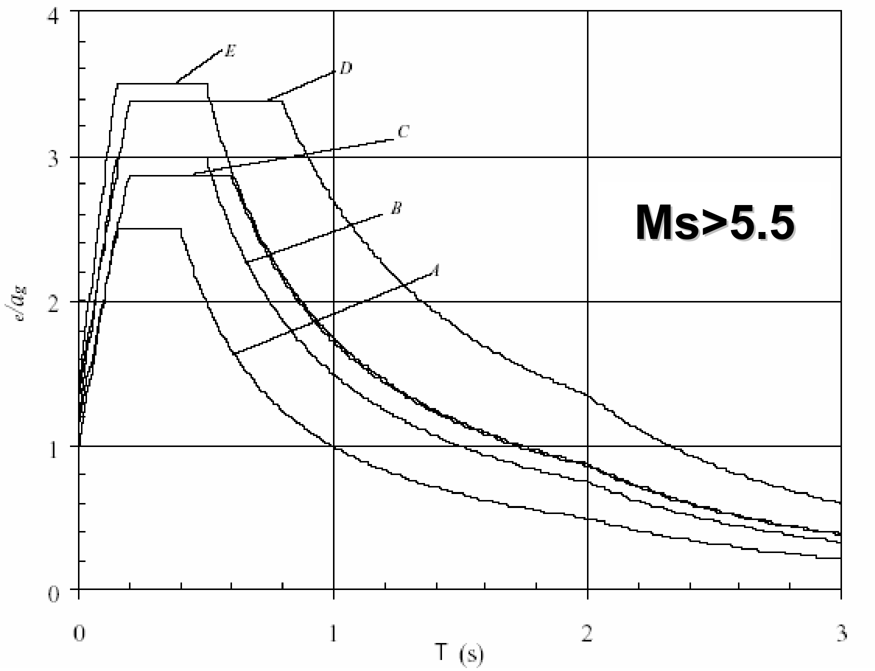
h_i, V_i thickness and velocity of i-layer up to 30m depth

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$V_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with $V_{s,30}$ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_{s,30}$ > 800 m/s			
S_1	Deposits consisting – or containing a layer at least 10 m thick – of soft clays/silts with high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A –E or S_1			

Discussion on site effects and soil categorization: EC8

Elastic Response Spectrum – Type 1



Parameters

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

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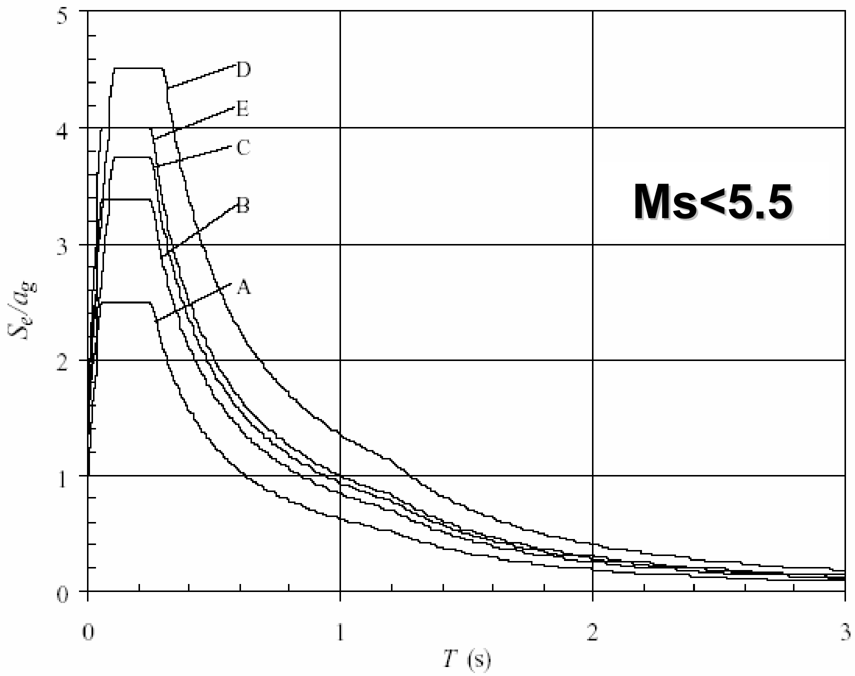
Discussion on site effects and soil categorization: EC8

Elastic Response Spectrum – Type



Parameters

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

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Site Effects Estimation Methods

- Empirical techniques
  - make use of recordings of strong ground motion

- Theoretical Methods
  - Simulation of ground motion based on real or hypothetical information for
    - the source
    - the input motion
    - the soil model

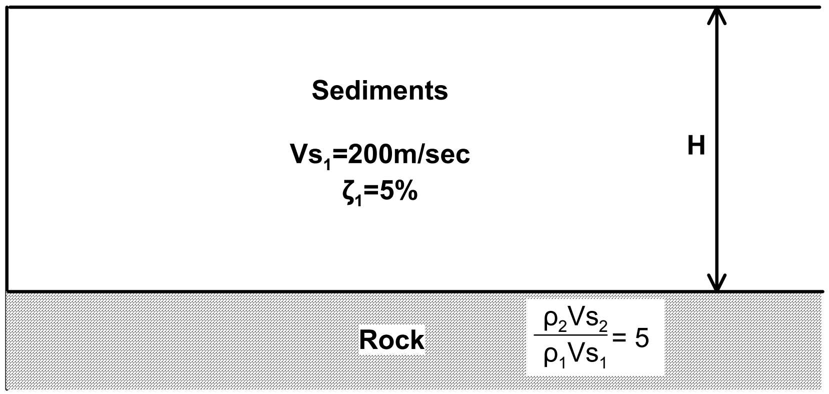
Theoretical (numerical and analytical) methods

- **Simple estimations**

- One horizontal layer - 1D structures

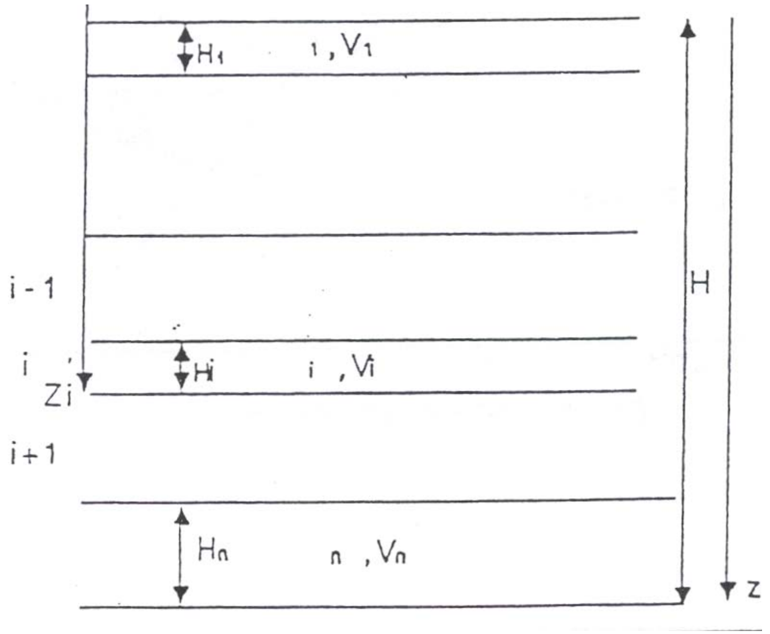
$$f_0 = \frac{Vs_1}{4 \cdot H} \qquad f_0 = \frac{1}{T_0}$$

$$A_0 = \frac{1}{\frac{1}{C} + 0.5 \cdot \pi \cdot \zeta_1}$$



- horizontal multi-layer 1D structures

only f_0 or T_0 can be approximated



Theoretical (numerical and analytical) methods

- **Simple estimations**
  - horizontal multi-layer 1D structures

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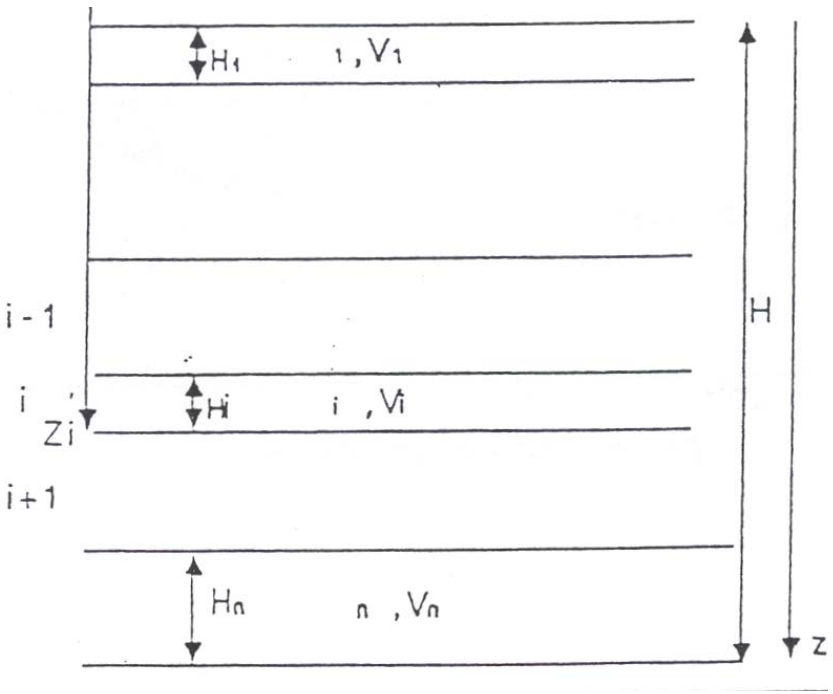


Method	Description	Mathematical Formulation
1	Weighted average of S wave velocities (β)	$\overline{\beta} = \left(\sum_{i=1}^{i=n} \beta_i h_i\right) / h$ $T_0 \approx T_1 = 4h / \overline{\beta}$
2	Weighted average of shear moduli and densities	$\overline{G} = \left(\sum_{i=1}^{i=n} G_i h_i\right) / h$ $\overline{\rho} = \left(\sum_{i=1}^{i=n} \rho_i h_i\right) / h$ $T_0 \approx T_2 = 4h / \sqrt{\overline{G}/\overline{\rho}}$
3	Sum of natural periods of each layer	$T_0 \approx T_3 = \sum_{i=1}^{i=n} 4h_i / \beta_i$
4	Linear approximation of the fundamental periods of each layer	$\omega_4^{\ 2} = \left(3\sum_{i=1}^{i=n} \beta_i^{\ 2} h_i\right) / h^3$ $T_0 \approx T_4 = 2\pi / \omega_4$
5	Simplified version of Rayleigh approach	$x_{i-1} = x_i + \dfrac{z_i + z_{i-1}}{\beta_i^{\ 2}} h_i$ $x_n = 0$ $\omega_5^{\ 2} = \dfrac{4\sum_{i=1}^{i=n} \dfrac{(z_i + z_{i-1})^2}{\beta_i^{\ 2}} h_i}{\sum_{i=1}^{i=n} (x_i + x_{i-1})^2 h_i}$ $T_0 \approx T_5 = 2\pi / \omega_5$

Theoretical (numerical and analytical) methods

- The most commonly used theoretical method in microzonation studies is the

One dimensional response of soil columns

Two Steps:

- **(1) Input data**
  - Modeling the Soil profile
  - Input motion (earthquake record)
- **(2) Output results**
  - Acceleration, Velocity, Displacement time histories at the surface of the soil profile (common) or at various levels within the profile
  - Response spectra and Amplification
  - Max acceleration, strain and stress with depth

Step 1 – Modelling soil Profile:

- Stratigraphy and dynamic properties (dynamic modulus and damping).
- 1D approach: soil depth is reasonably constant s reasonably constant beneath the structure and the soil layers and ground surface reasonably flat. Otherwise, 2D or 3D models of the site can be used.
- A range of properties should be defined for the soil layers to account for uncertainties (Unless soil properties are well constrained)

Step 2 – Calculating 'expected' motions:

- Analysis should incorporate nonlinear soil behavior (either through equivalent linear or true nonlinear methods)
- Design Input motions at outcropping bedrock conditions – compatibility with the seismotectonic of the broader area
- Assume base or halfspace (Vs>700m/s is often assumed but not always is OK). Determine 'seismic bedrock' according both to Vs and geological criteria.

Site Response Analysis

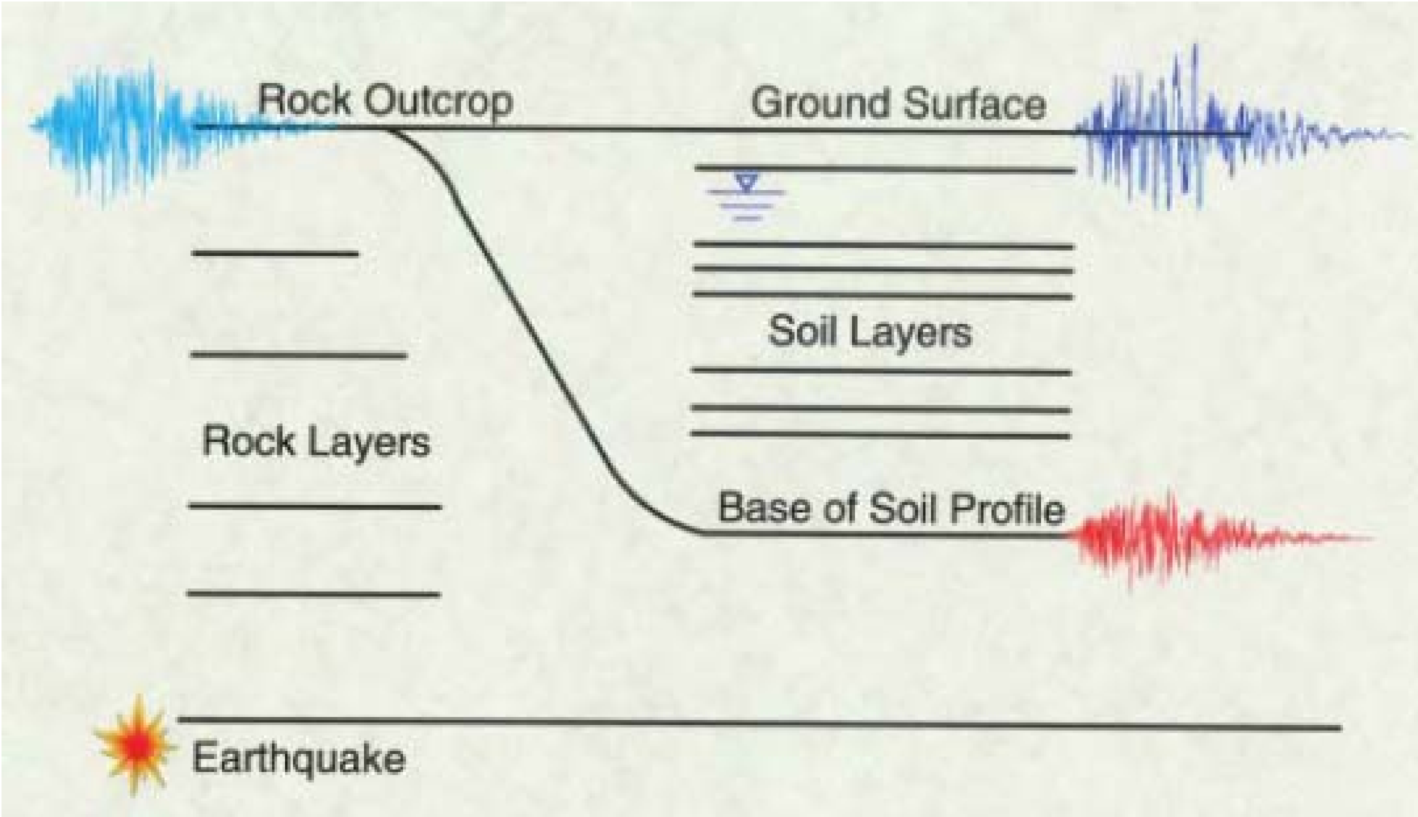
Techniques

- Linear analyses
- Quarter-wavelength approximation
- **Equivalent linear analyses**
- Nonlinear analyses

Codes

- **Equivalent linear analyses:**
  - **SHAKE (Schnabel, Seed, and Lysmer 1972;Idriss and Sun 1992)**
  - **WESHAKE (Sykora, Wahl, and Wallace 1992)**
  - **EERA (J. P. Bardet, K. Ichii, and C. H. Lin, 2000) http://geoinfo.usc.edu/gees/**
- Nonlinear analyses
  - DESRA-2 (Lee and Finn 1978), DESRA-MUSC (Qiu 1998)
  - SUMDES (Li, Wang, and Shen 1992)
  - MARDES (Chang et al. 1990)
  - D-MOD (Matasovic 1993)
  - TESS (Pyke 1992)
  - CYBERQUAKE (BRGM 1998)
  - DEEPSOIL (Hashash and Park 2001)

Equivalent-Linear Analysis (SHAKE)



Equivalent-Linear Analysis (SHAKE)

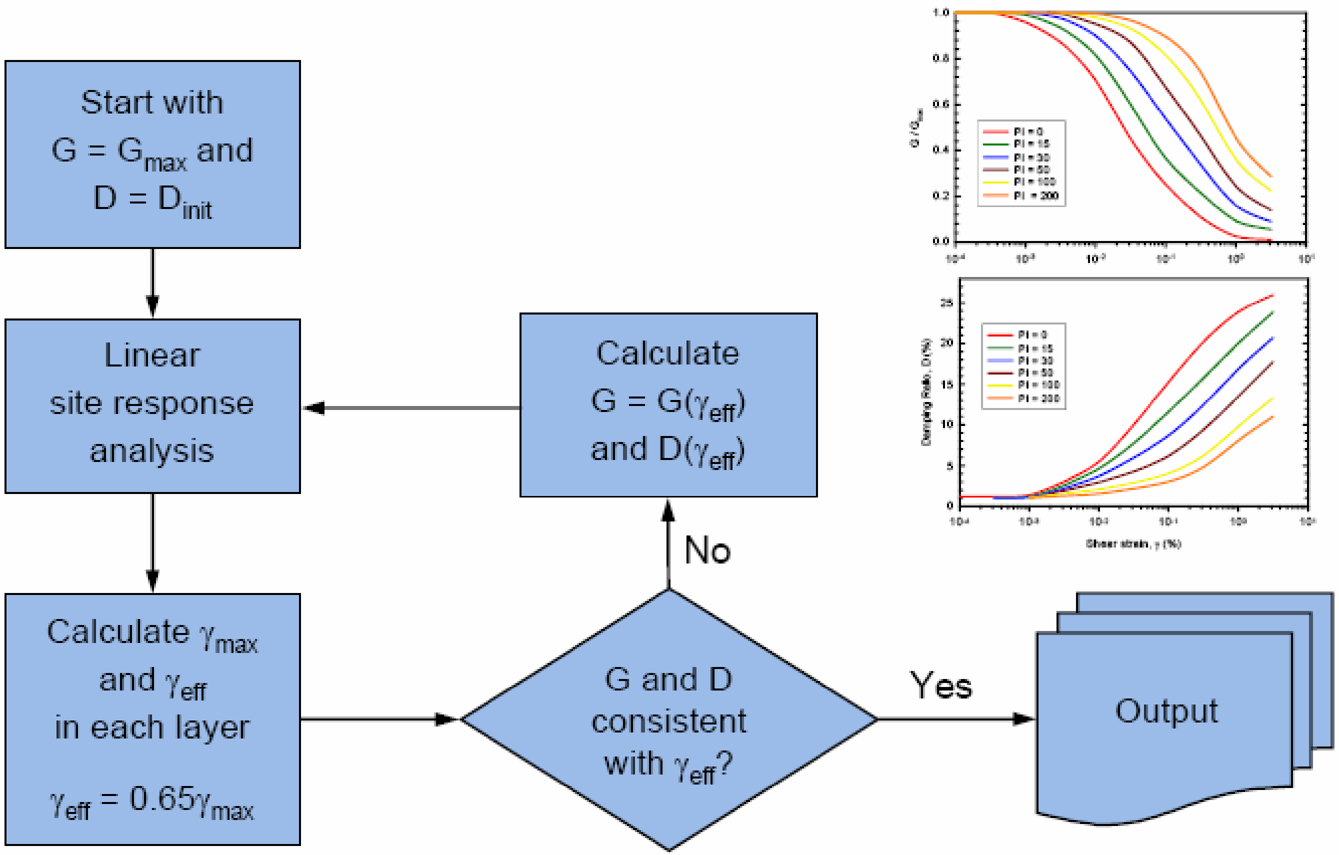
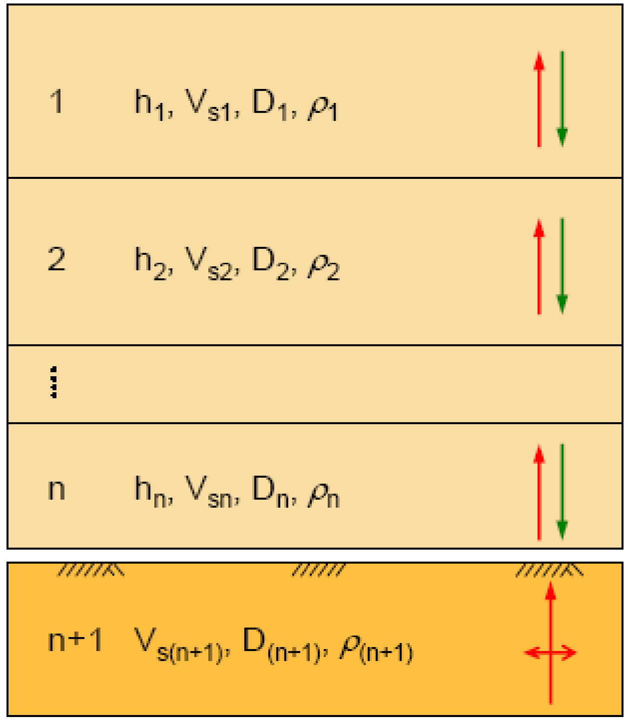


Figure adapted from Rix, G. J., (2001)

Equivalent-Linear Analysis (SHAKE)

Step 1: Modelling the Soil Profile - Layers

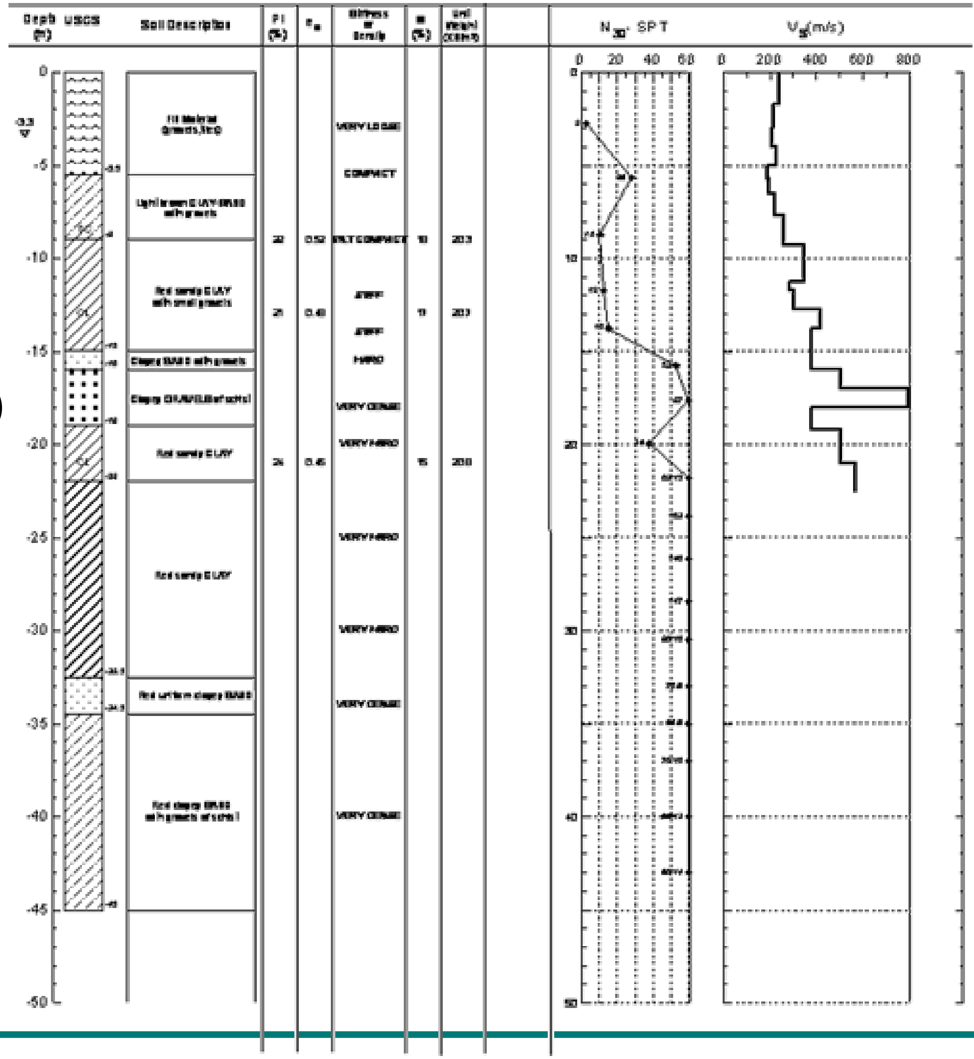
Maximum layer Thickness (H):
dependent on change in material properties
Usually Hmax=Vs/4fmax, 1-3m



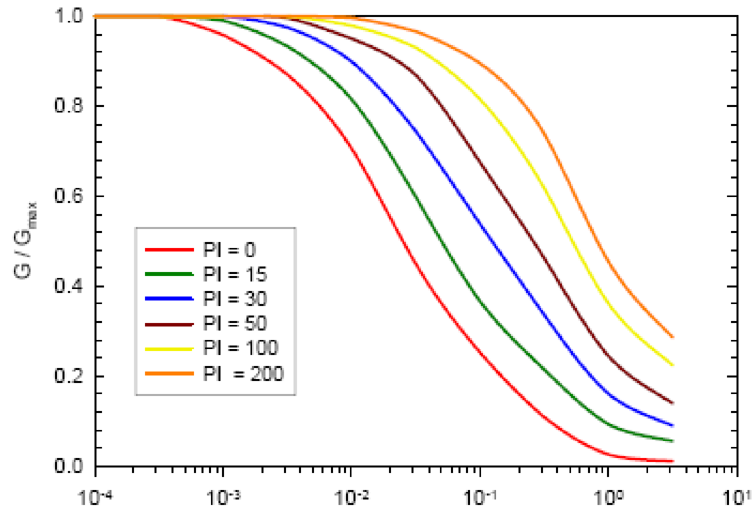
Equivalent-Linear Analysis (SHAKE)

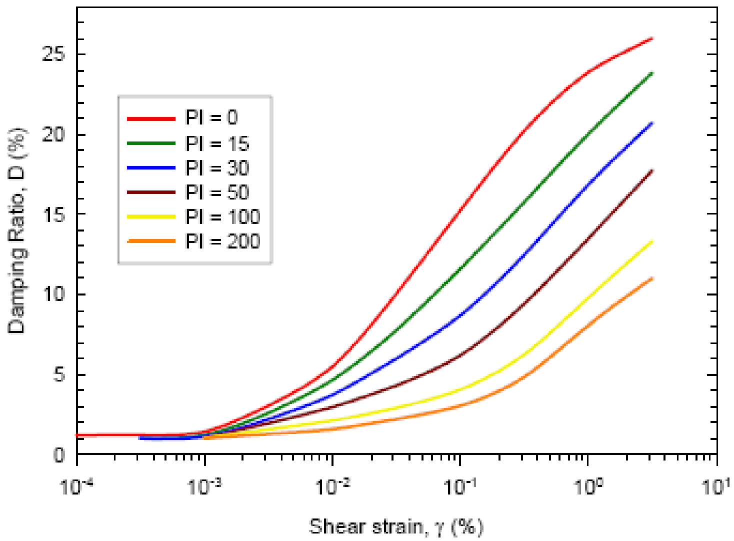
Step 1: Modelling the Soil Profile – Vs (Gmax)

- Crosshole
- Downhole/SCPT
- P-S suspension logger
- Vertical arrays
- Surface wave methods
- Empirical correlations (SPT, CPT)



Equivalent-Linear Analysis (SHAKE)





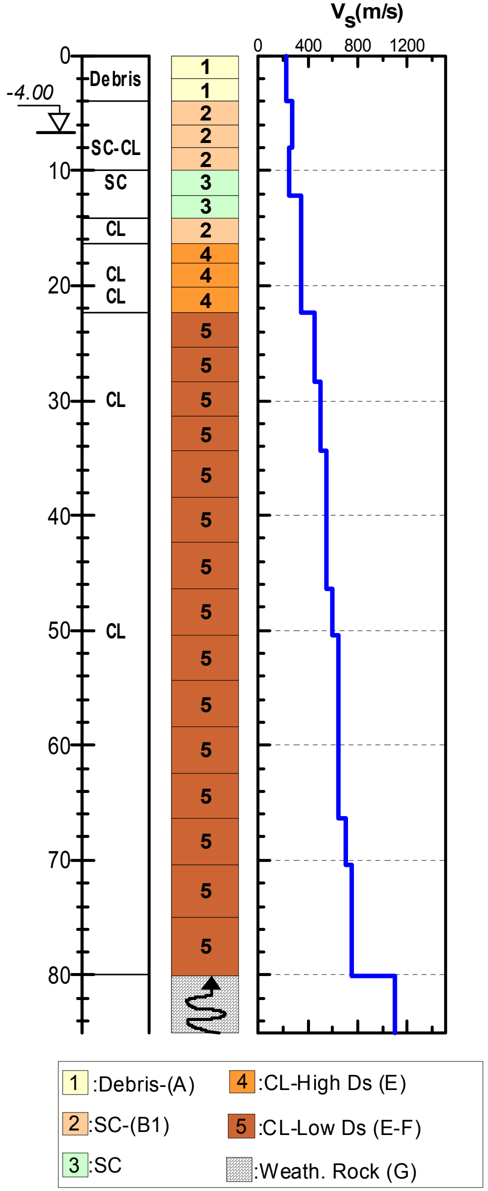
Vucetic and Dobry (1991)

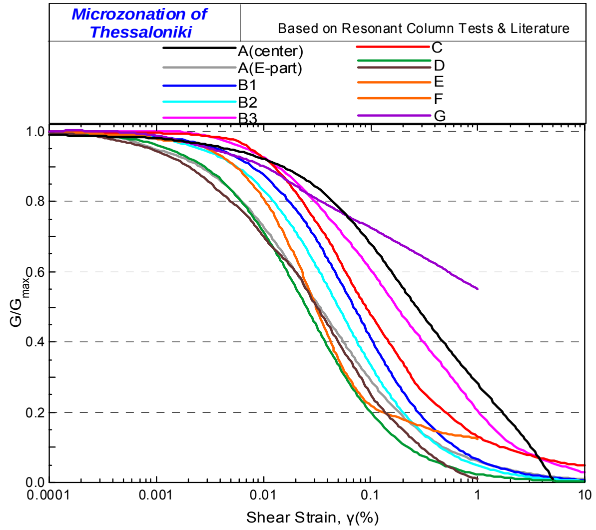
Modulus Reduction and Damping Curves

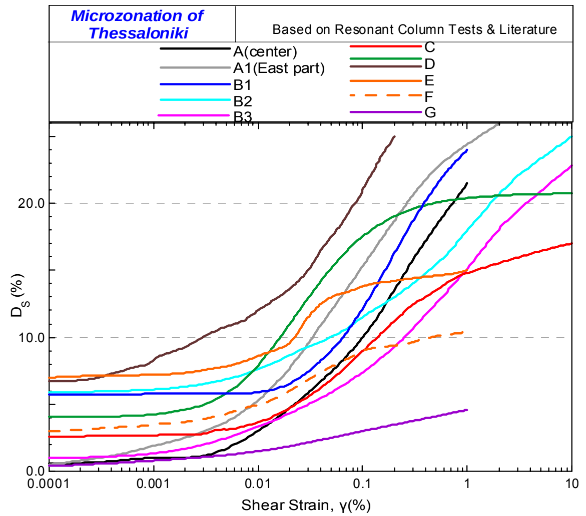
Laboratory Tests:
- Resonant column
- Torsional shear
- Cyclic simple shear
- Cyclic triaxial

- Seed et al. (1986)
- Sun et al. (1988)
- Vucetic and Dobry (1991,1993)
- Ishibashi and Zhang (1993)
- EPRI (1993)
- Hwang (1997)
- Toro and Silva (2001)
- Stokoe and Darandeli (2001)
- Roblee and Chiou (2004)

Equivalent-Linear Analysis (SHAKE) - example

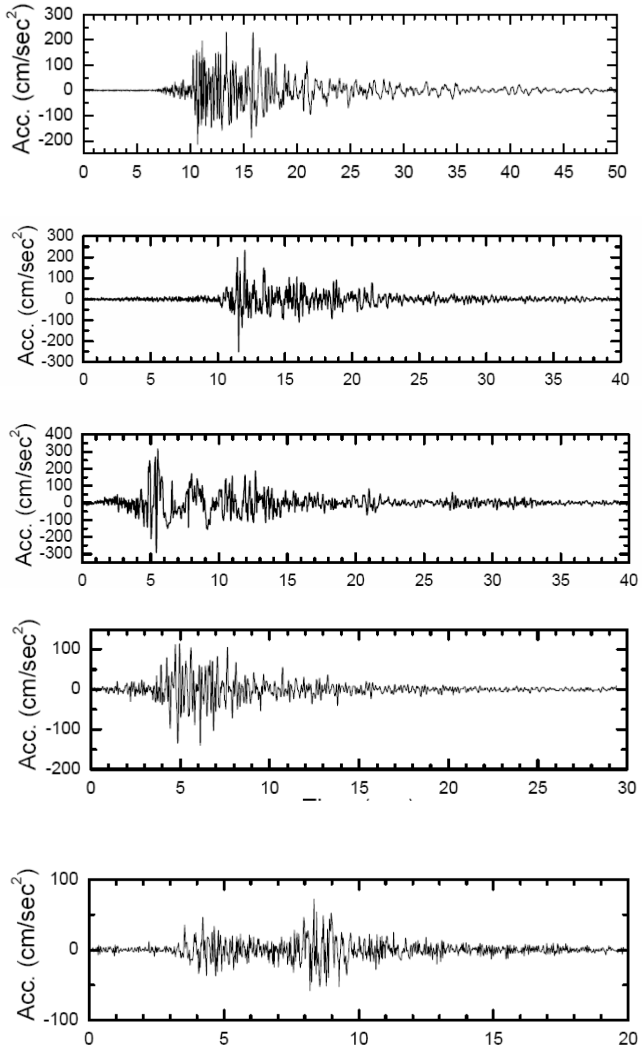




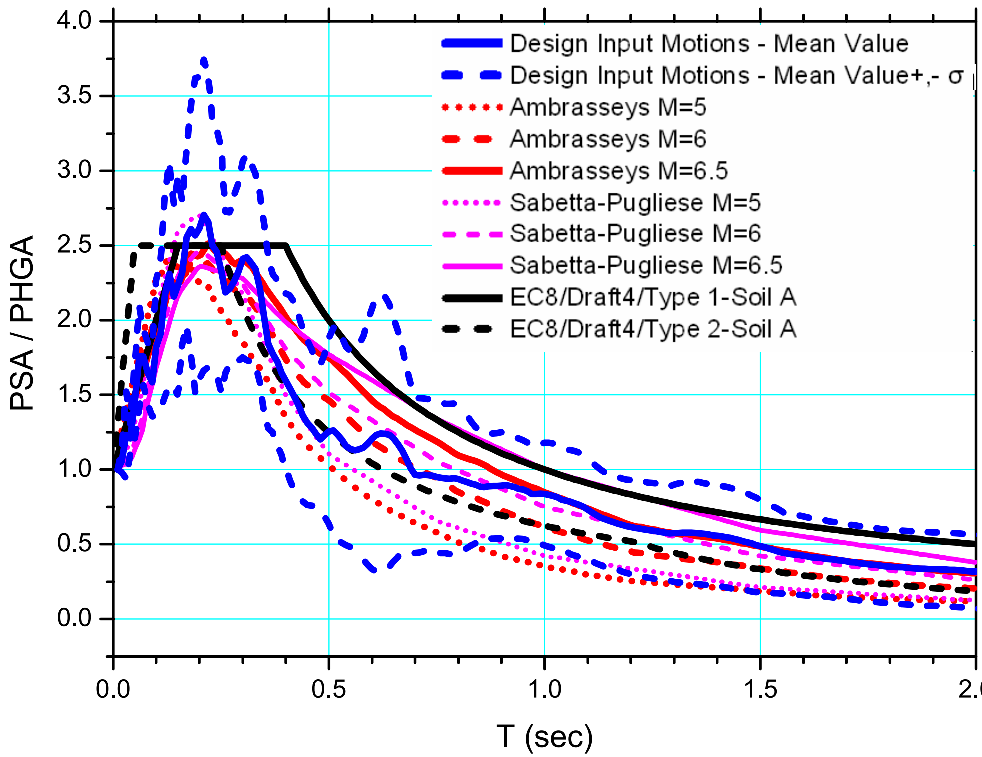


Equivalent-Linear Analysis (SHAKE) - example

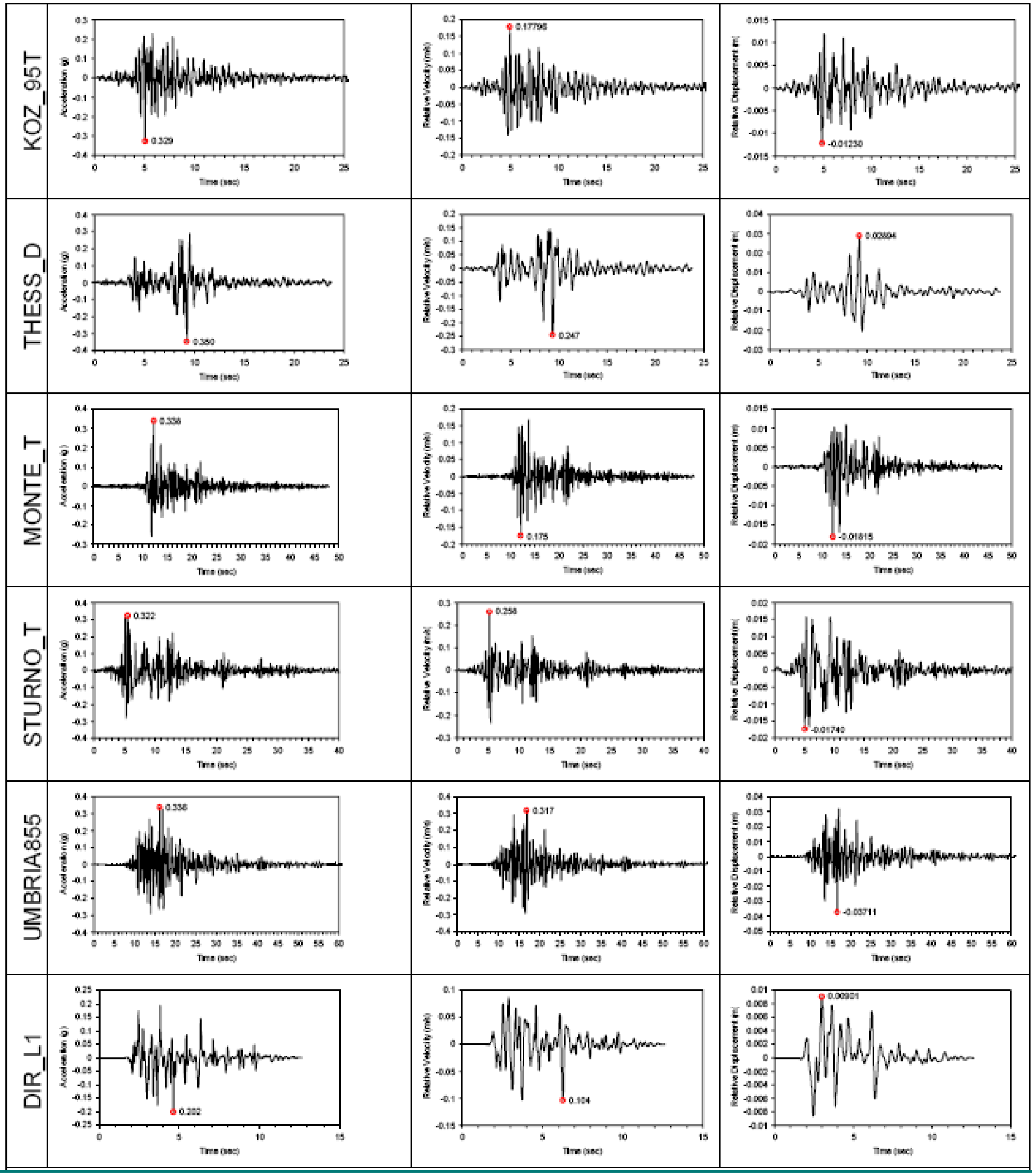
Input motion



No.	Name	Earthquake	Country	Date	Time	Lat.	Long.	Focal Depth (Km)	Magnitude Mw	Mecha-nism	Station's name	Building type	Geology	Epicentral distance R (Km)	PGA (g)
1	855-Y	Umbria-Marche	Italy	5/4/1998	15:52:20	43.19	12.72	10	4.8	normal	Cubbio-Piene	free-field	rock	18	0.235
2	MONT_T	Montenegro	Yugoslavia	15/4//1979	6 :19 :41	41.98	18.98	12	6.9	thrust	Hercegnovi Novi-O.S.D. Pav.Sch	free-field	rock	65	0.256
3	Sturno_T	Campagno Lucano	Italy	23/11/1980	18 :34 :52	40.78	15.33	16	6.9	normal	Sturno	free-field	rock	32	0.323
4	Koz95-T	Kozani	Greece	13/5/1995	8:47:15	40.18	21.66	14	6.5	normal	Kozani's Perfecture	free-field	rock	17	0.142
5	Thes78_Dec	Thessaloniki	Greece	20/6/1978	20:03:22	40.73	23.25	6	6.2	normal	The_6-City	free-field	rock	29	0.074 (0.143)



Equivalent-Linear Analysis (SHAKE) - example



Acceleration,

Velocity and

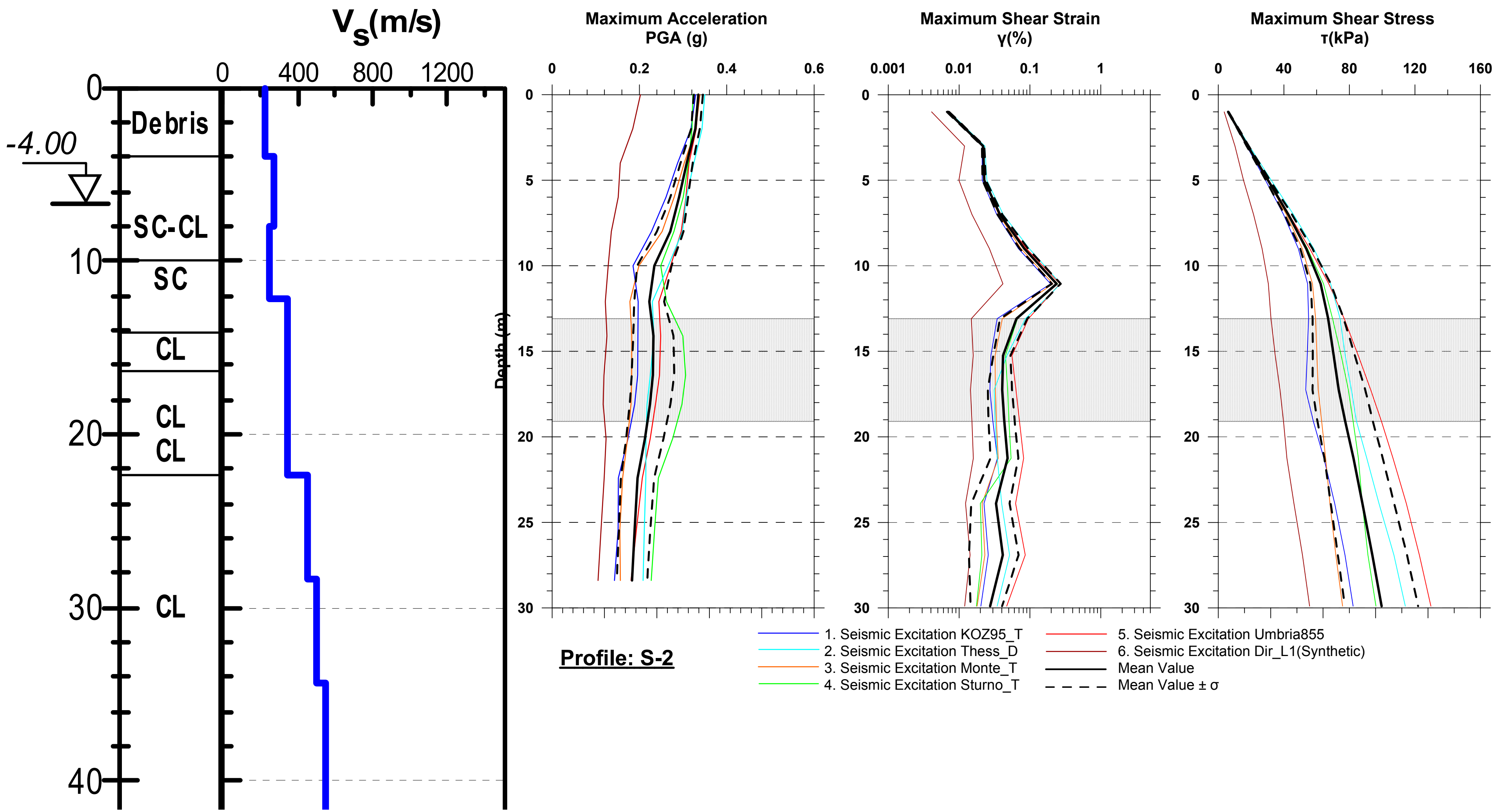
Displacement

Time Histories

at Ground Surface

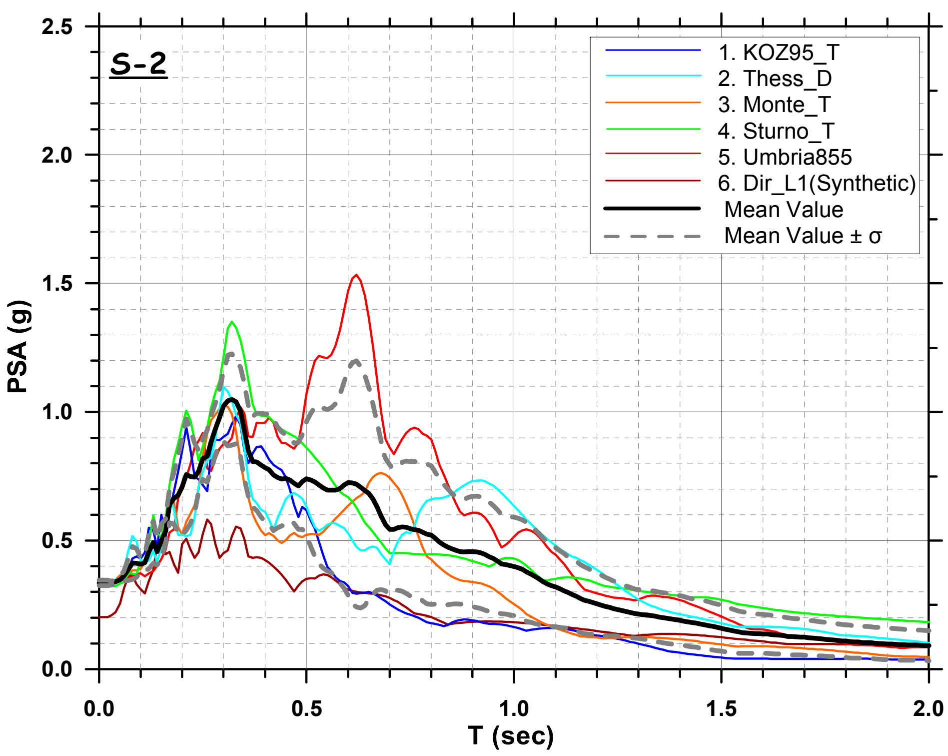
Equivalent-Linear Analysis (SHAKE) - example

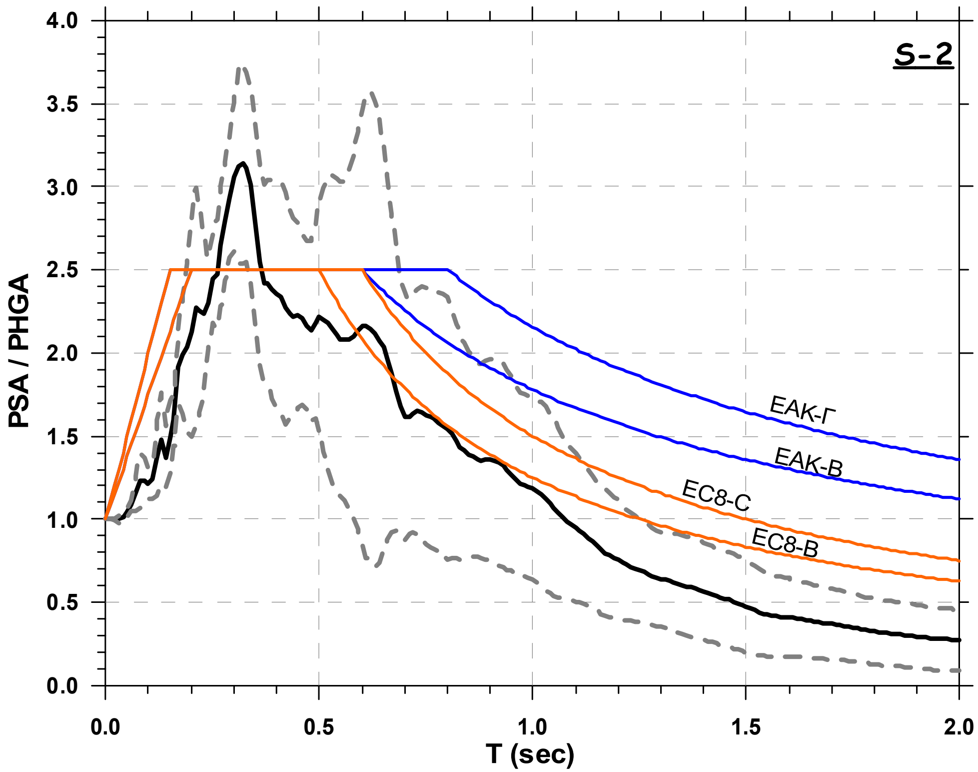
Variation with depth of PGA, Shear Strain and Shear Stress



Equivalent-Linear Analysis (SHAKE) - example

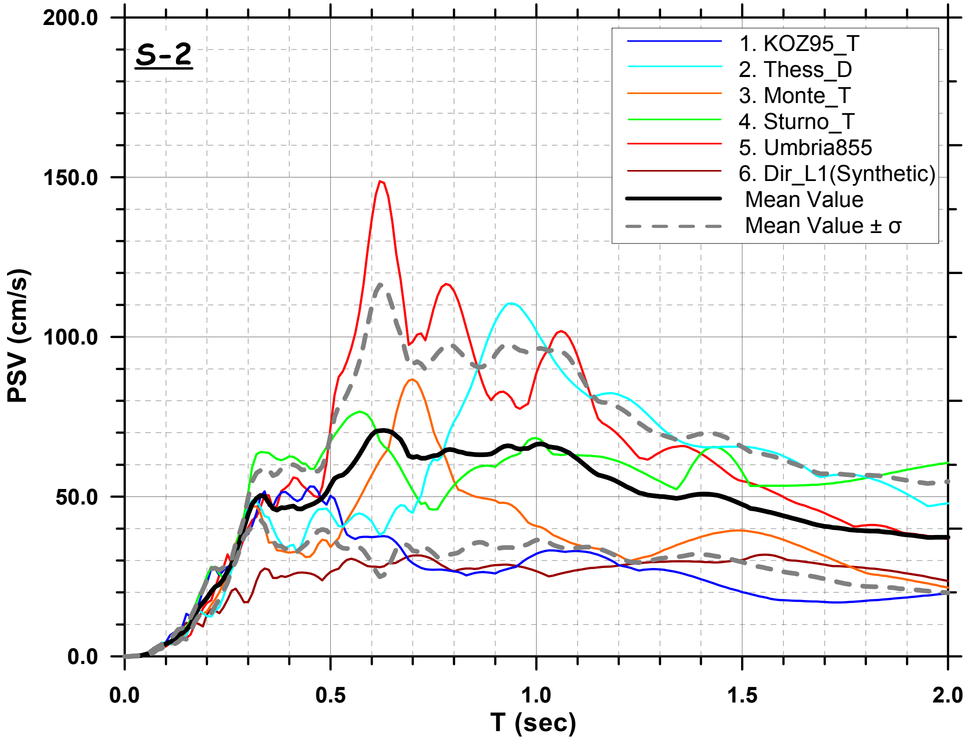
Acceleration and Normalized Acceleration Response Spectrum at free Surface

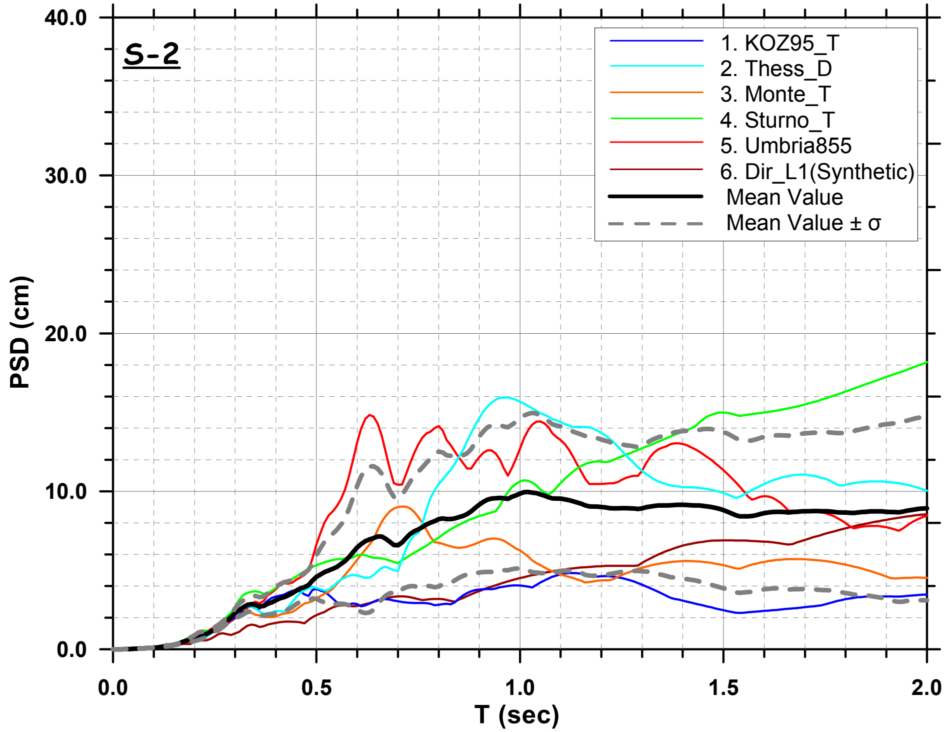




Equivalent-Linear Analysis (SHAKE) - example

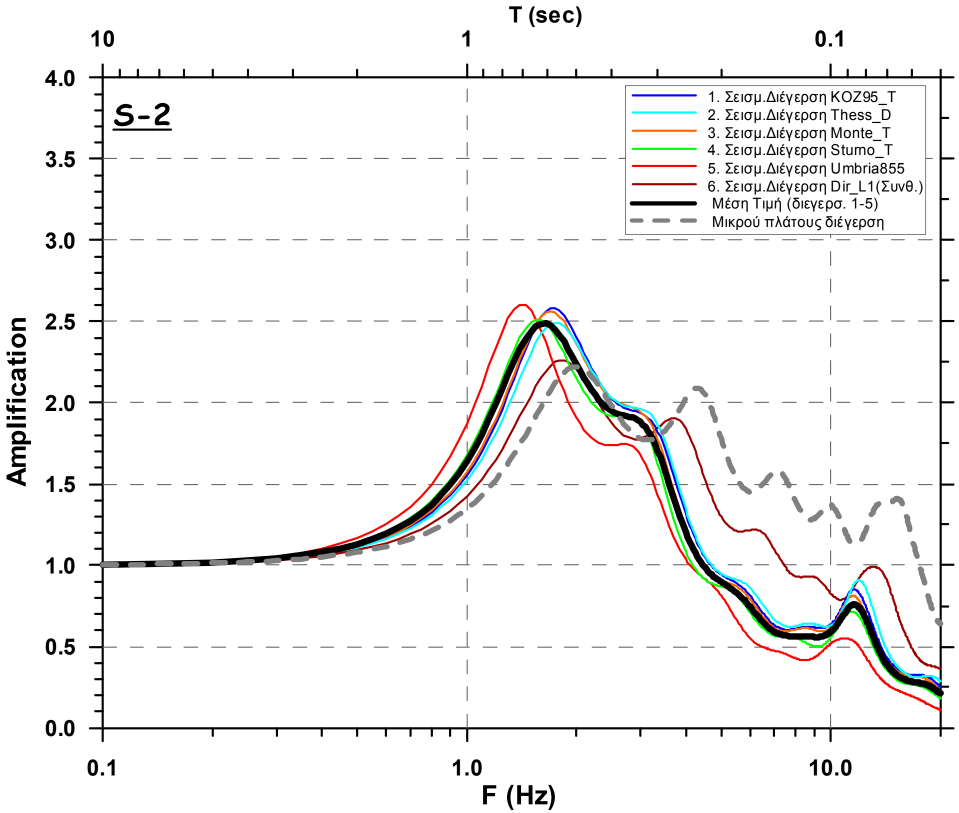
Velocity and Displacement Response Spectrum at free Surface



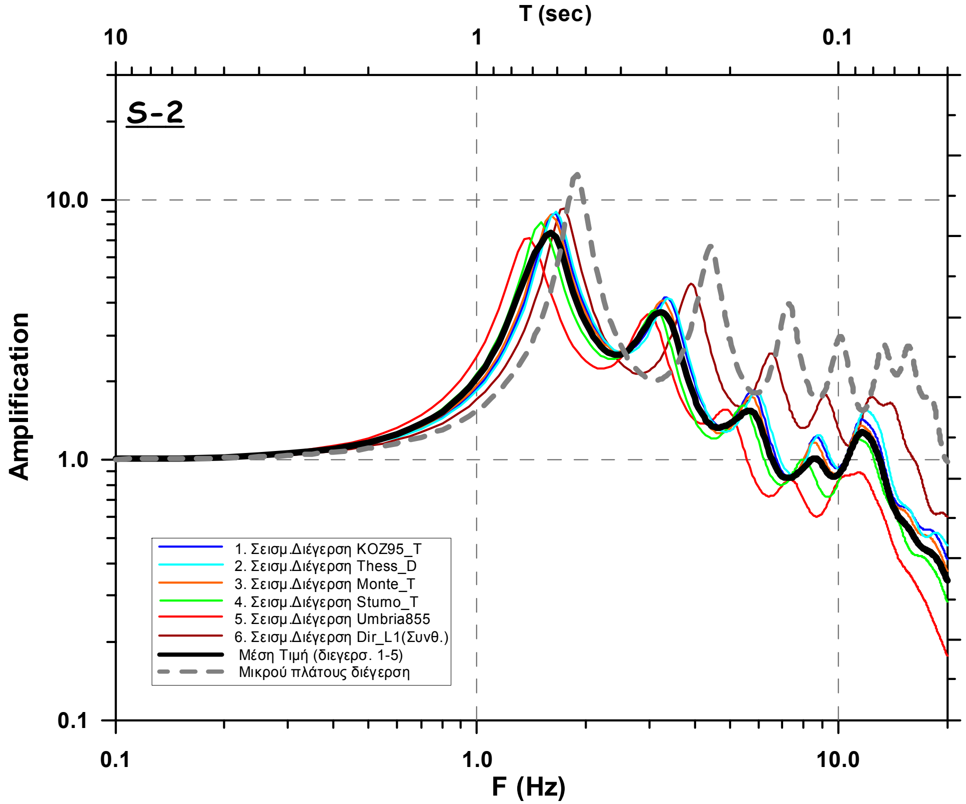


Equivalent-Linear Analysis (SHAKE) - example

Amplification ratios at free surface



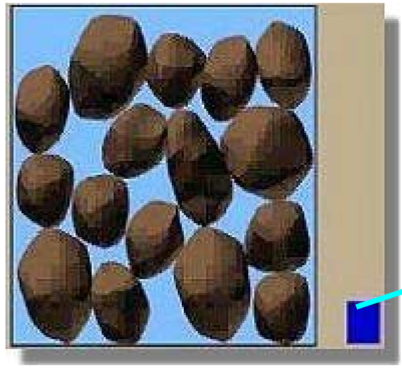
ground surface / seismic motion at "outcropping" conditions



ground surface / the incident seismic vibration

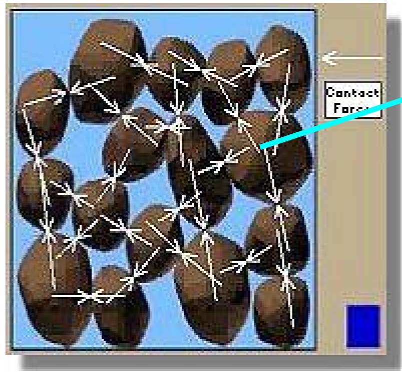
Liquefaction

Liquefaction = phenomenon in which the strength & stiffness of a soil is reduced by earthquake shaking or other rapid loading.

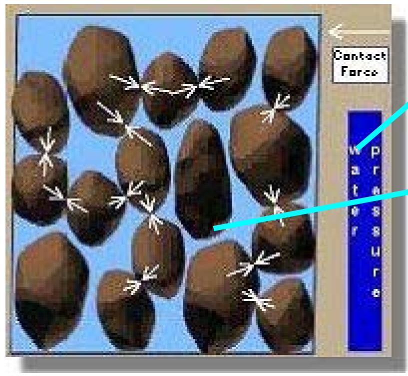


Soil grains in a soil deposit

Pore water pressure



Length of arrows=size of contact



Pore water pressure increases

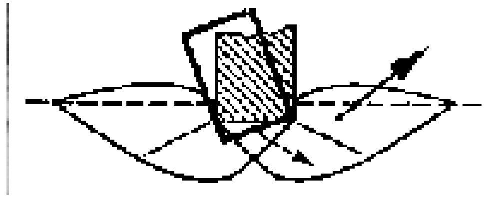
Soil particles lose contact

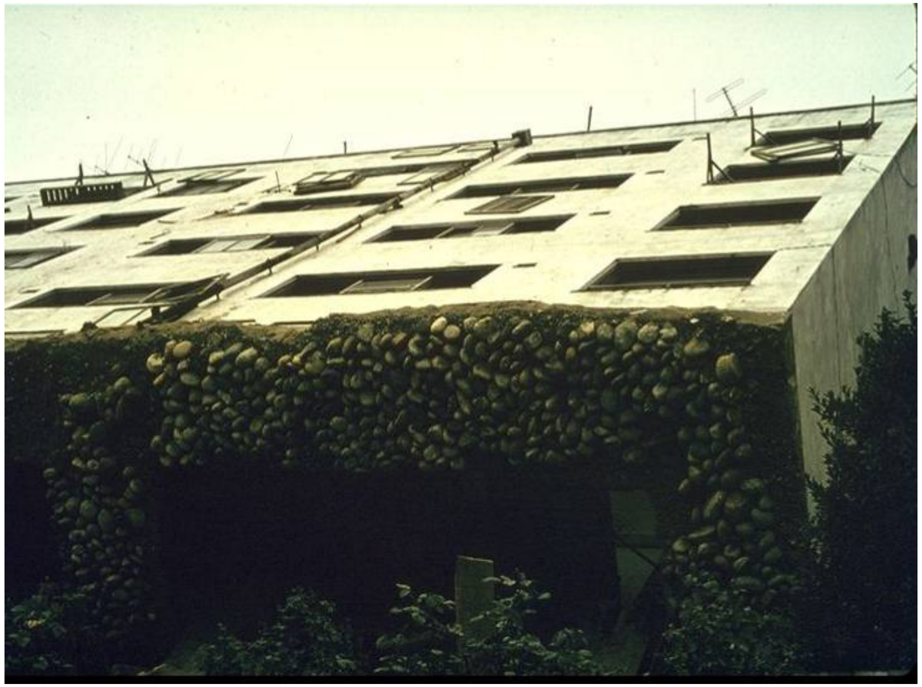
Friction~0
Strength~0

<u>Conditions:</u>

- **Saturated,uniform,loose sandy-silty layers**
- **Strong ground motion - duration**

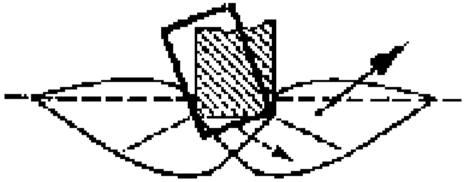
Niigata, Japan, June 16, 1964





A remarkable ground failure occurred near the Shinano river bank where the Kawagishi-cho apartment buildings suffered bearing capacity failures and tilted severely. Despite the extreme tilting, the buildings themselves suffered remarkably little structural damage.

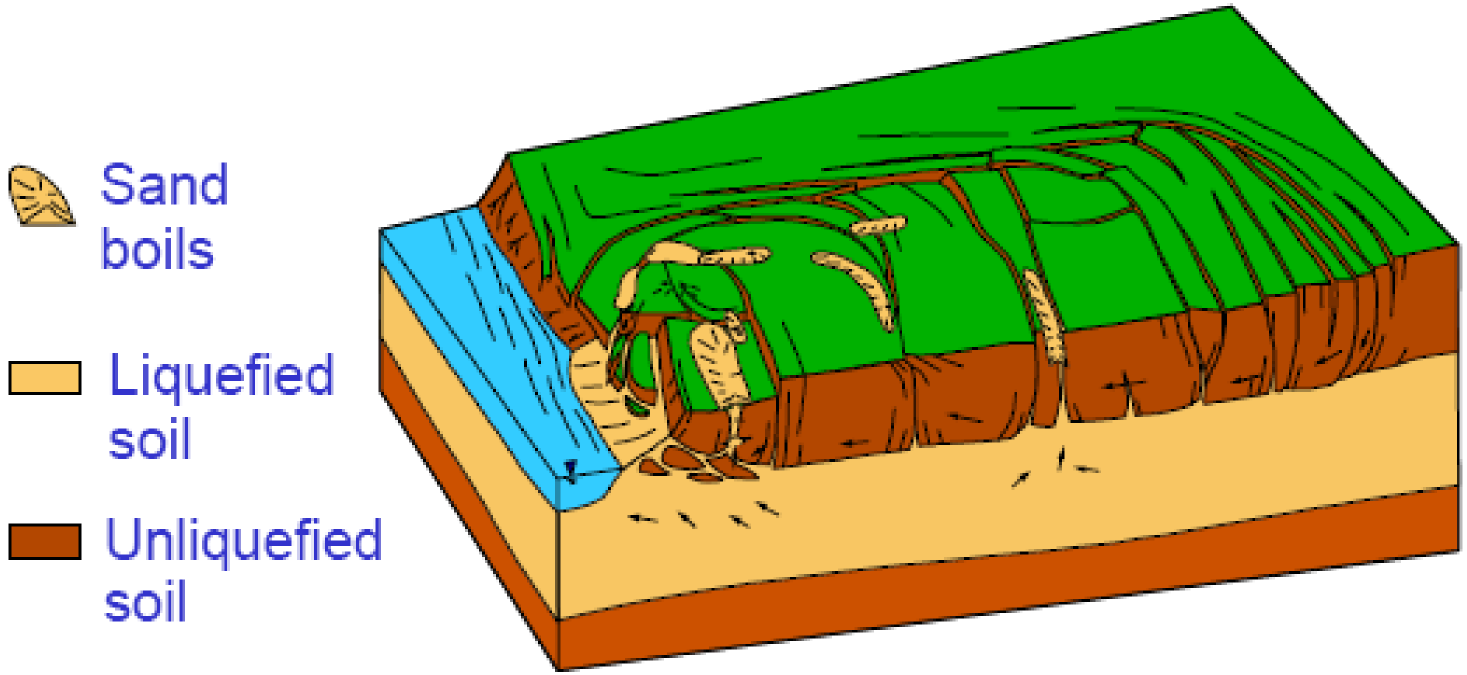
Adapazari, Koaceli - Turkey,1999



The mat foundation for this building was exposed when it overturned. This building has a relatively large height-to-width ratio, making it more susceptible to overturning failure.

This new building was not yet occupied at the time of the earthquake. Again, the bearing failure of its mat foundation was related to its relatively large height-to-width ratio.

Liquefaction - Lateral Spreading



• One of most pervasive forms of ground damage; especially troublesome for lifelines
• Mostly horizontal deformation of gently-sloping ground (< 5%) resulting from soil liquefaction

Kobe, Japan, Jan. 17, 1995

Collapse of crane due to lateral movement (~2 m) of quay wall on Rokko Island. Note settlement of 1-2 m also occurred behind wall.

Kobe, Japan, Jan. 17, 1995

A segment of this new bridge (Nishinomiya bridge) collapsed because of foundation deformations that are attributed to the effects of liquefaction. Ground cracks behind the quay walls and parallel to the water edge are indicative of the lateral ground movements that occurred. Sand boils are visible on the ground surface.

Northridge, California, Jan. 17, 1994

Kobe, Japan, Jan. 17, 1995

Typical utility pipe ruptured by lateral spreading in Granada Hills on Balboa Blvd

Pipes separated by lateral spreading between a building and adjacent concrete slab near Nakahara Wharf

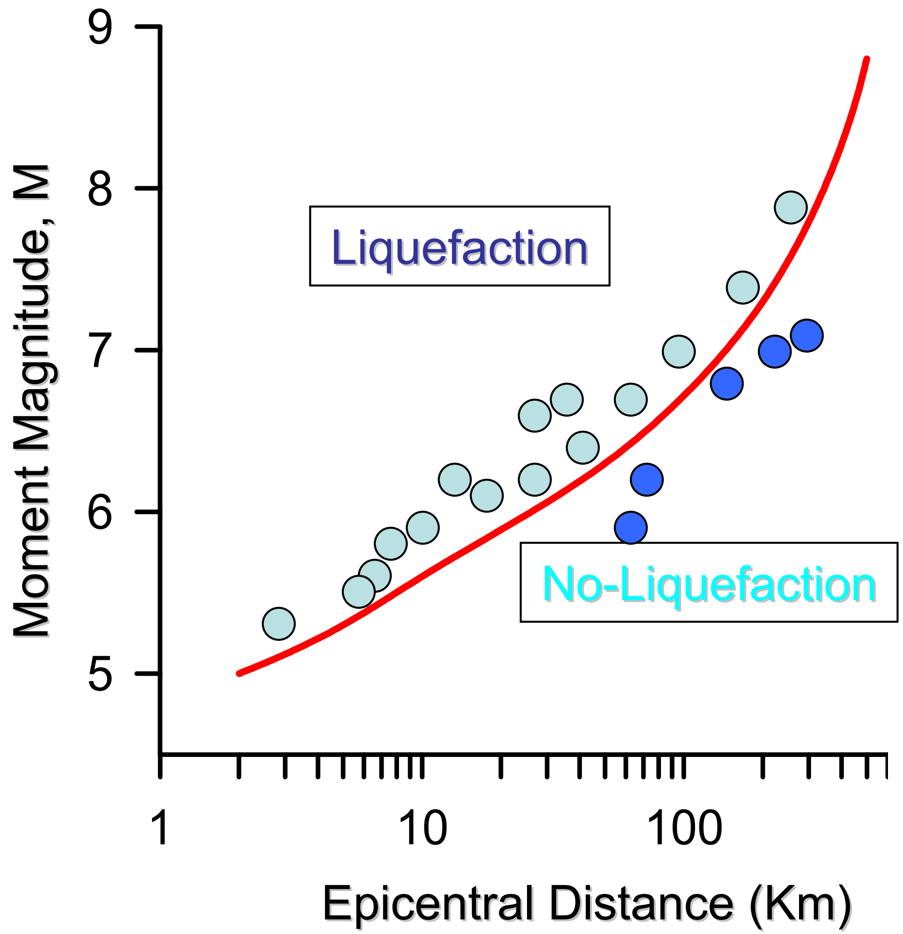
Hanshin Expressway - Jan. 1995

Liquefaction Susceptibility - Criteria

Historical Criteria

Observations from earlier earthquakes

Soils that have liquefied in the past can liquefy again in future earthquakes

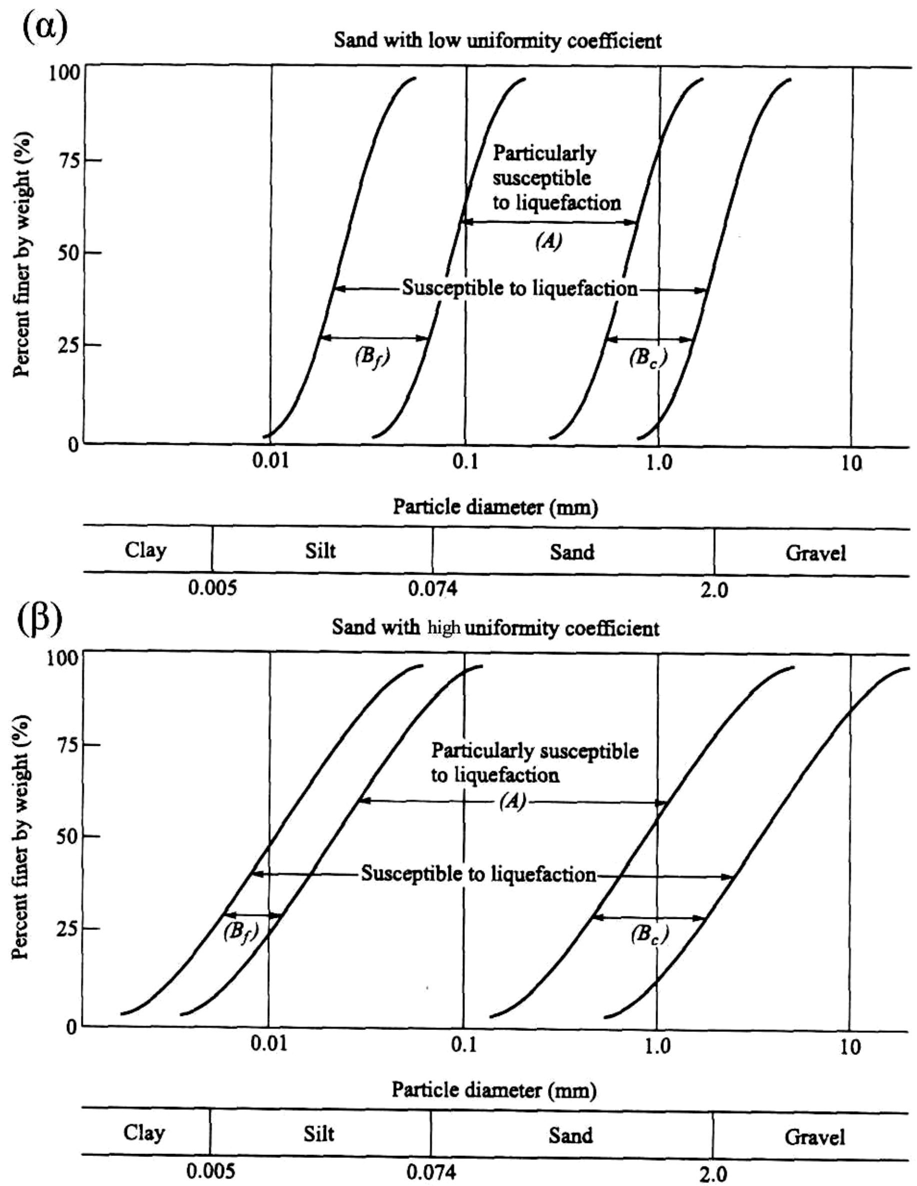


Geological Criteria:

Saturated soil deposits created by sedimentation in rivers and lakes (fluvial or alluvial deposits), deposition of debris or eroded material (colluvial deposits), or deposits formed by wind action (aeolian deposits) can be very liquefaction susceptible.

Compositional Criteria:

Fraction finer than 0.005mm ≤15%
Liquid Limit, LL ≤35%
Natural water content ≥ 0.9 LL
Liquidity index ≤ 0.75



Evaluation of Liquefaction Potential - Procedures

Stress-based : Calculation of strength and load in term of stresses

Factor safety=Capacity-'strength'/ Demand – 'load'

$$FS = \frac{CRR_M}{CSR}$$

Strain procedures - Energy-based procedures

Aseismic Codes:

EC8: S2 subsoil class liquefiable soils are described by the S2 subsoil class (Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in classes 1-E or S1)

Stress based procedure (Factor safety=Capacity-'strength'/Demand – 'load')

Greek Code: Group X soils (loose saturated sands, Evaluation of liquefaction potential by Using appropriate analytical methods based on in-situ and laboratory tests

Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Demand – Cyclic Stress Ratio – 'load':

$$CSR = \frac{\tau_{ave}}{\sigma'_{vo}} = 0.65\, \alpha_{\max} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d$$

σ_{vo}: Total vertical stress at depth z

$\alpha_{\max}$: Peak Horizontal ground acceleration, PGA in g

σ'_{vo}: effective vertical stress at depth z

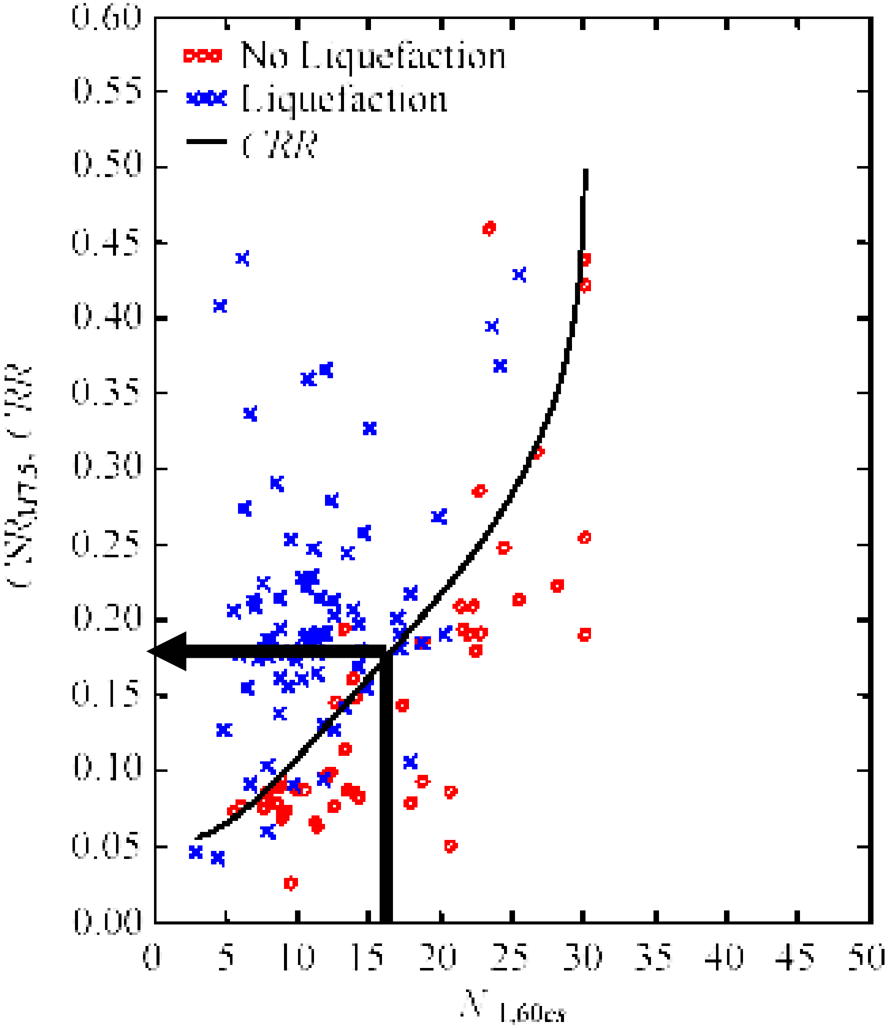
r_d: **Dimensionless parameter that accounts for the stress reduction**

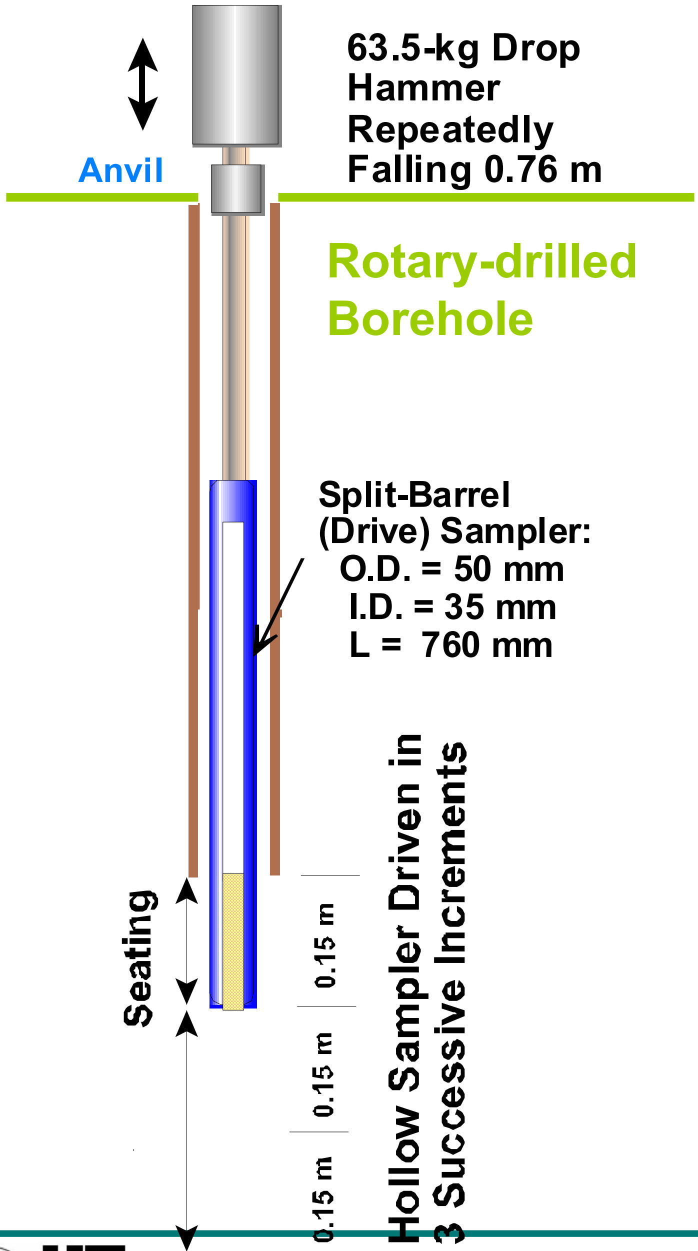
$$r_d = \begin{cases} 1.0 - 0.00765\,z & for \quad z \le 9.15m \\ 1.174 - 0.0267\,z & for \quad 9.15 < z \le 23m \\ 0.744 - 0.008\,z & for \quad 23m < z \le 30m \\ 0.5 & for \quad z > 30m \end{cases}$$

Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Empirical Correlation
N (S.P.T.)





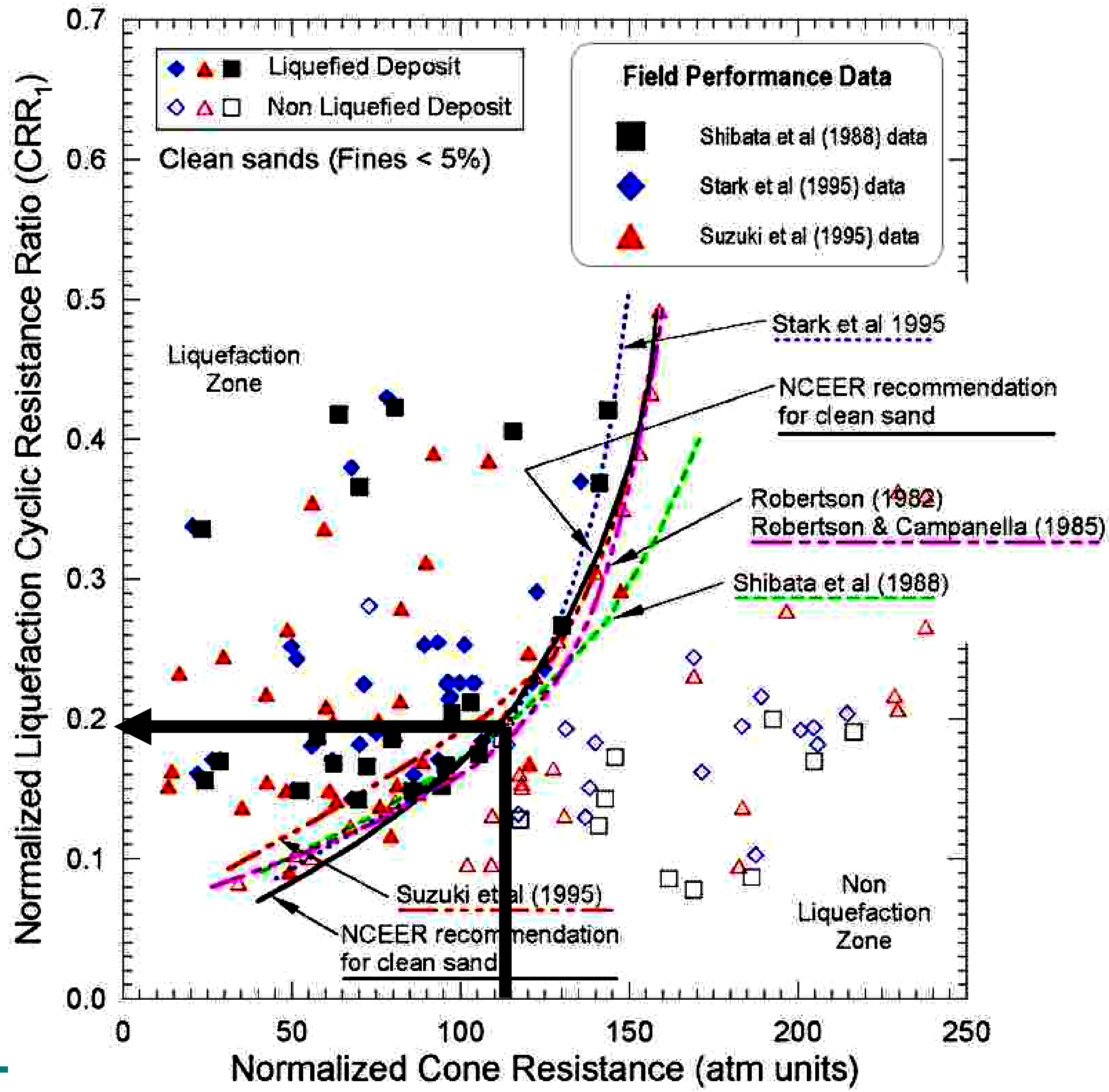
Standard Penetration Test

Standard Penetration Test (SPT)
Procedures: ASTM D 1586
N = measured Number of Blows to drive sampler 300 mm into soil.

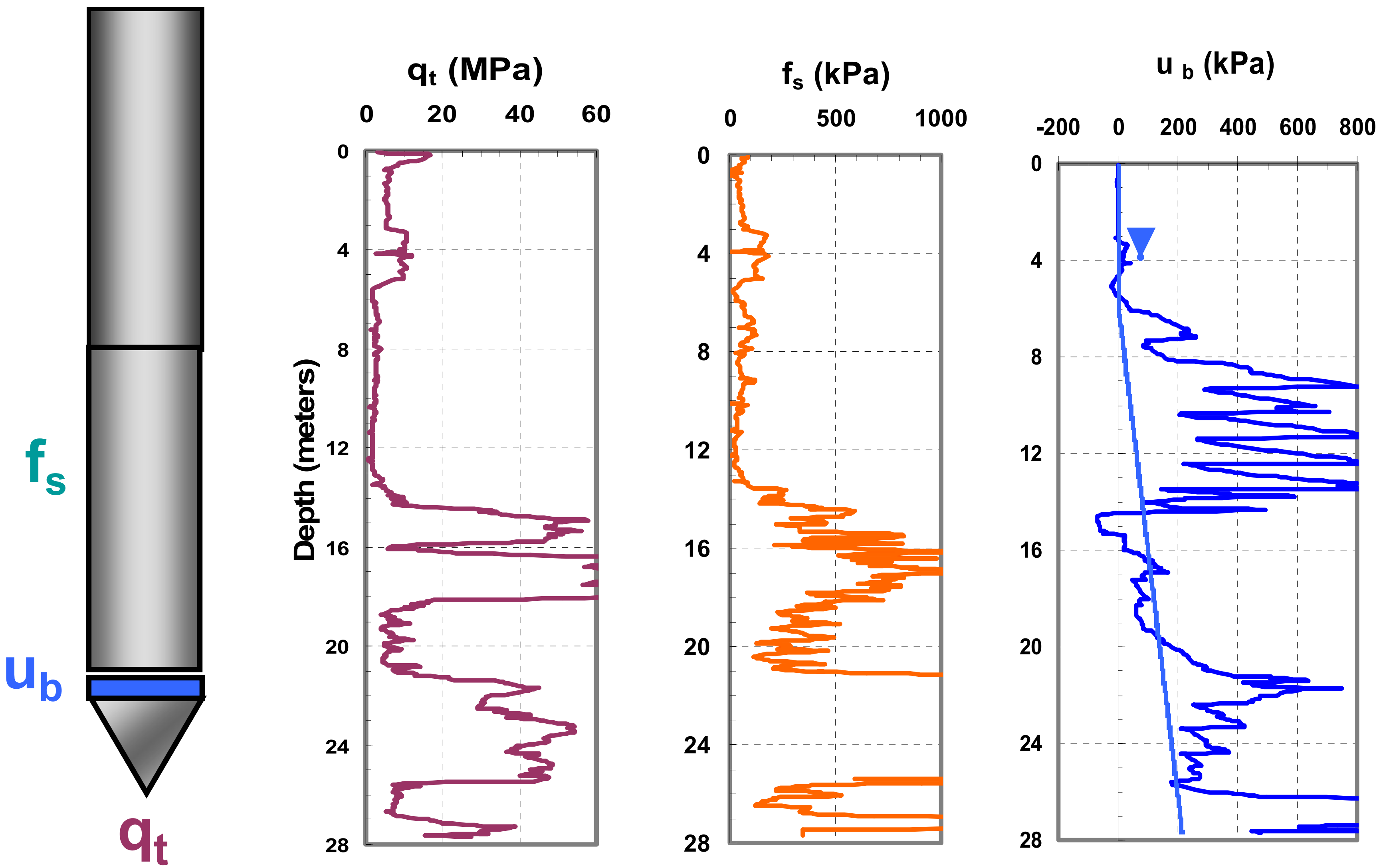
Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

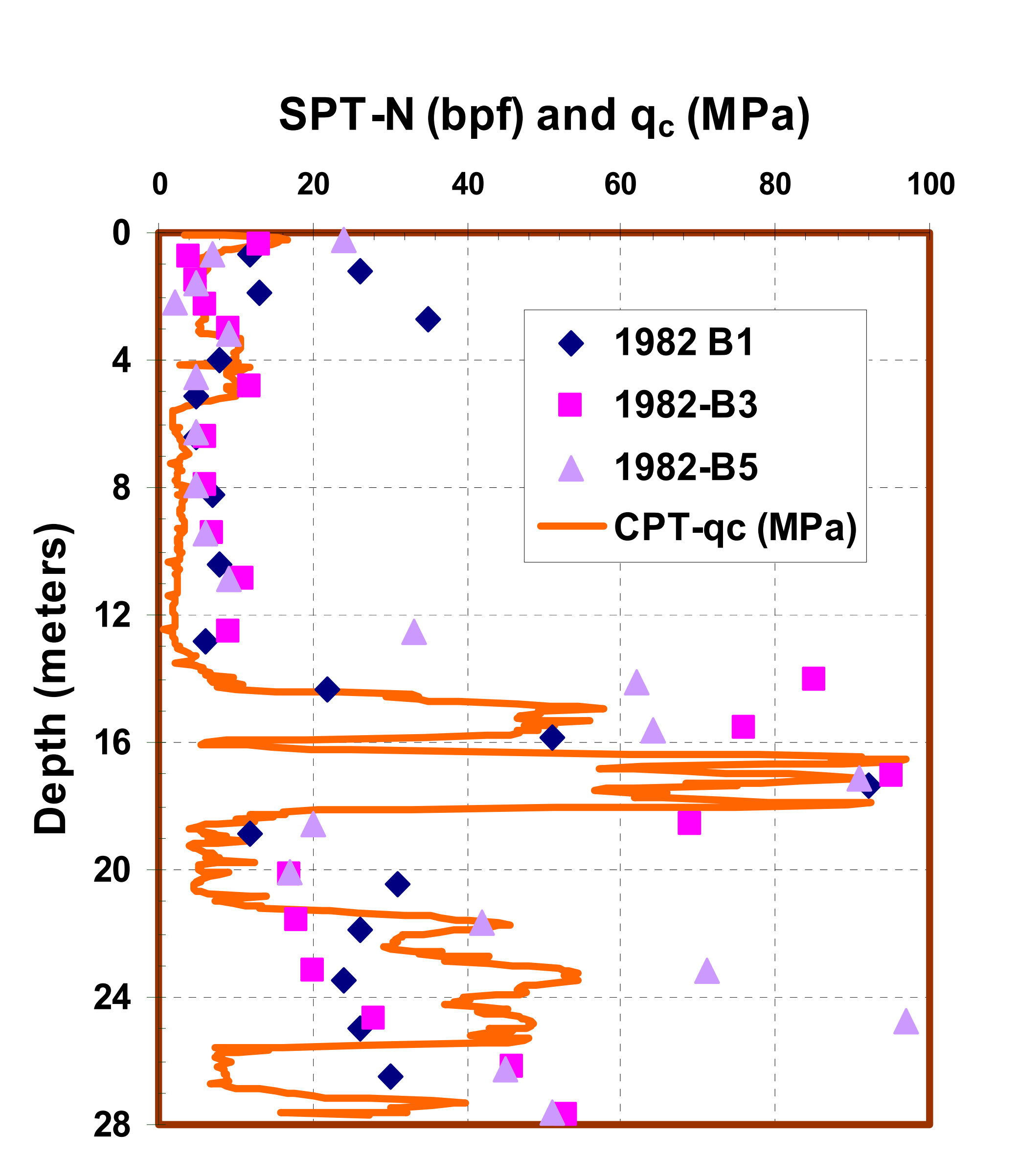
Empirical Correlation CPT

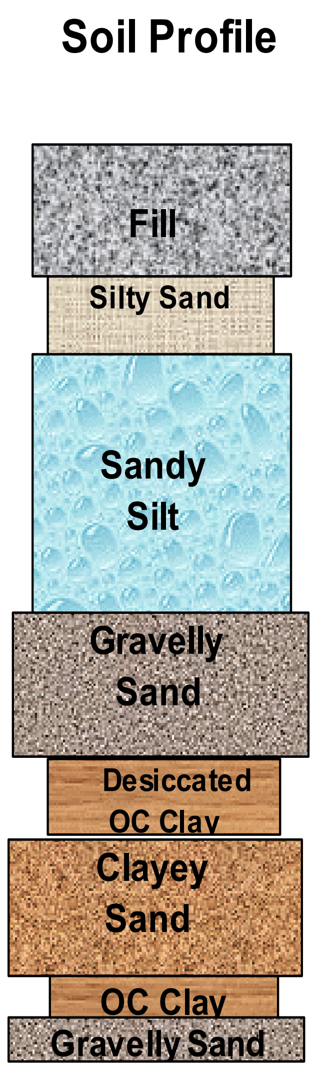


CPT Profile, Downhole Memphis



Comparison CPT and SPT - Downtown Memphis





Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Step 1: Correction of SPT blow count data

$$(N_1)_{60} = N_m \cdot C_n \cdot C_e \cdot C_b \cdot C_r \cdot C_s$$

Factor	Test Variable	Term	Correction
Overburden Pressure[1]		C_N	$\left(\frac{P_a}{\sigma'_{vo}}\right)^{0.5}$ $C_N \leq 1.7$
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-Trip Donut-Type Hammer		0.8 to 1.3
Borehole Diameter	65*mm* to 115*mm*	C_B	1.0
	150*mm*		1.05
	200*mm*		1.15
Rod Length[2]	< 3*m*	C_R	0.75
	3*m* to 4*m*		0.8
	4*m* to 6*m*		0.85
	6*m* to 10*m*		0.95
	10*m* to 30*m*		1.0
Sampling Method	Standard Sampler	C_S	1.0
	Sampler without Liners		0.1 to 1.3

(Modified from Skempton 1986 and Robertson and Wride 1998)

[1]The effective overburden pressure should be the value corresponding to that at the time of drilling and testing. A higher groundwater level might be assumed for conservatism in the liquefaction resistance calculations.

[2]Rod corrections were not applied for lengths greater than 3*m* in the formulation of the simplified procedure; therefore, corrections are not required in applying the procedure for lengths greater than 3*m*.

Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Step 2: Fines Content Correction of SPT

$$N_{1,60\,cs} = a + \beta\, N_{1,60}$$

$$a = \begin{cases} 0 & for\ FC \le 5\% \\ \exp[1.76 - (190 / FC^2)] & for\ 5\% < FC \le 35\% \\ 5.0 & for\ FC > 35\% \end{cases}$$

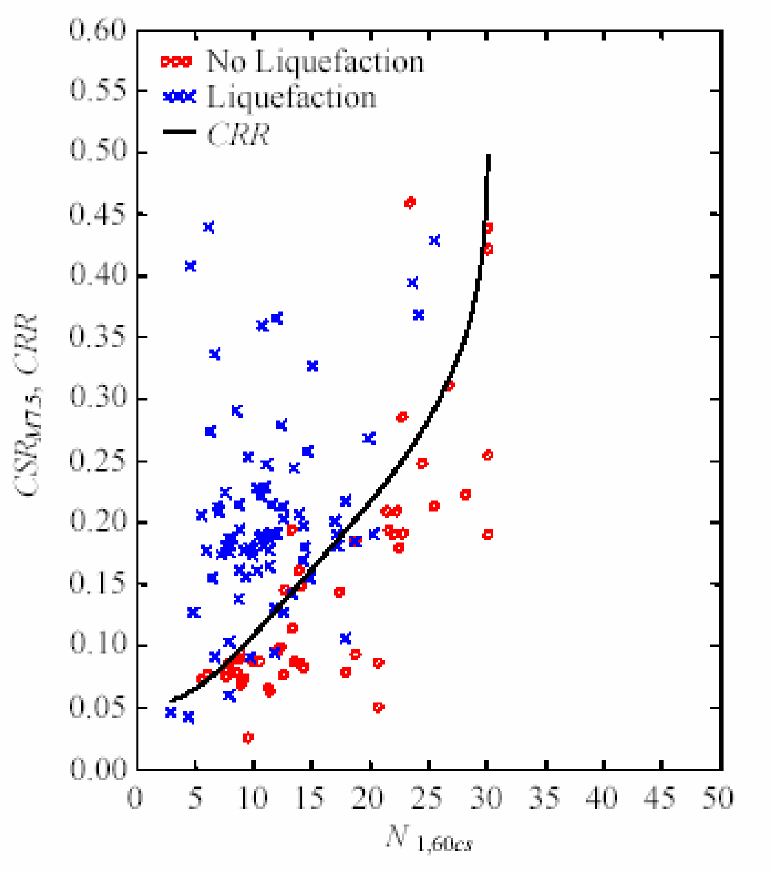
$$\beta = \begin{cases} 1.0 & for\ FC \le 5\% \\ [0.99 - (FC^{1.5} / 1000)] & for\ 5\% < FC \le 35\% \\ 1.2 & for\ FC > 35\% \end{cases}$$

Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Step 3: Calculation of CRR7.5

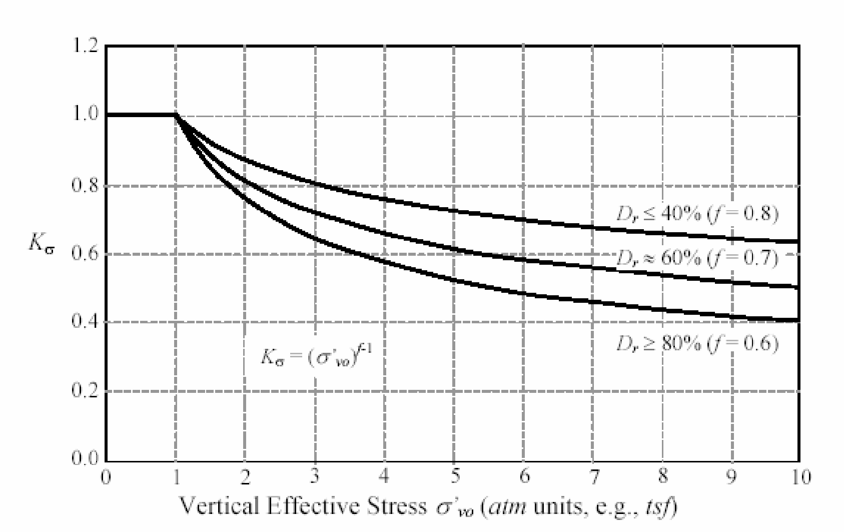
$$CRR_{7.5} = \frac{1}{34 - N_{1,60cs}} + \frac{N_{1,60cs}}{135} + \frac{50}{(10\,N_{1,60cs} + 45)^2} - \frac{1}{200}$$



Evaluation of Liquefaction Potential - Procedures

NCEER 1997: Factor of Safety:

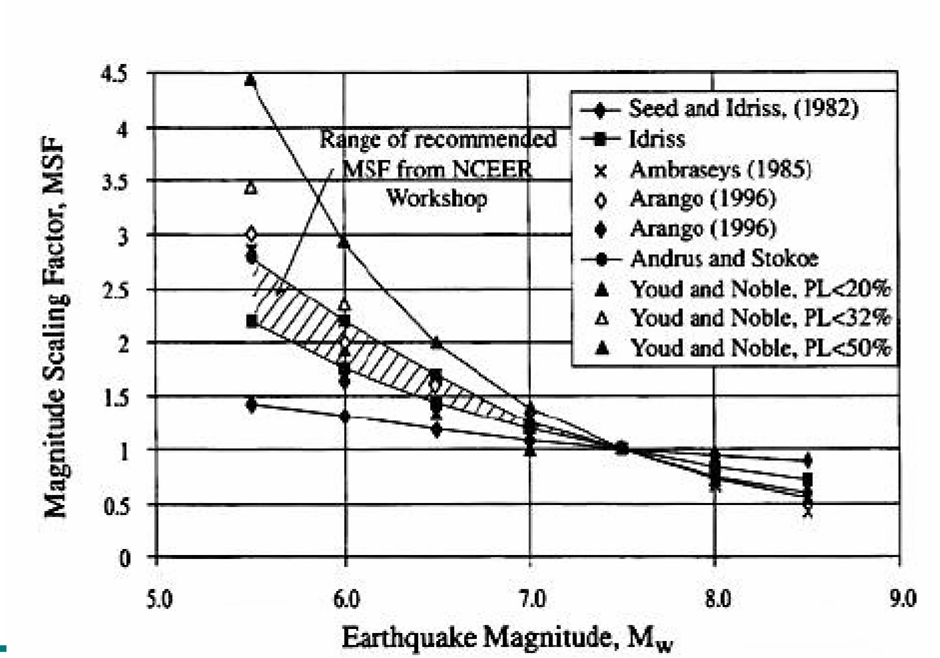
$$CRR_V = CRR_{7.5} \cdot K_\sigma \cdot K_\alpha$$



$$Capacity = CRR_M = CRR_V \cdot MSF$$

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

$$FS = \frac{CRR_M}{CSR}$$



Evaluation of Liquefaction Potential - Procedures

EC8: Demand – Cyclic Stress Ratio – 'load':

$$\tau_e = 0.65 \cdot a \cdot S \cdot \sigma_{vo}$$

a : ground acceleration ratio, i.e. ratio between the design acceleration a_g and the gravity acceleration

S : Soil profile parameter (see table 6.3)

σ_{vo} : is the total overburden pressure

Ground type	S
A	1,0
B	1,2
C	1,15
D	1,35
E	1,4

type 1 (high seismicity)

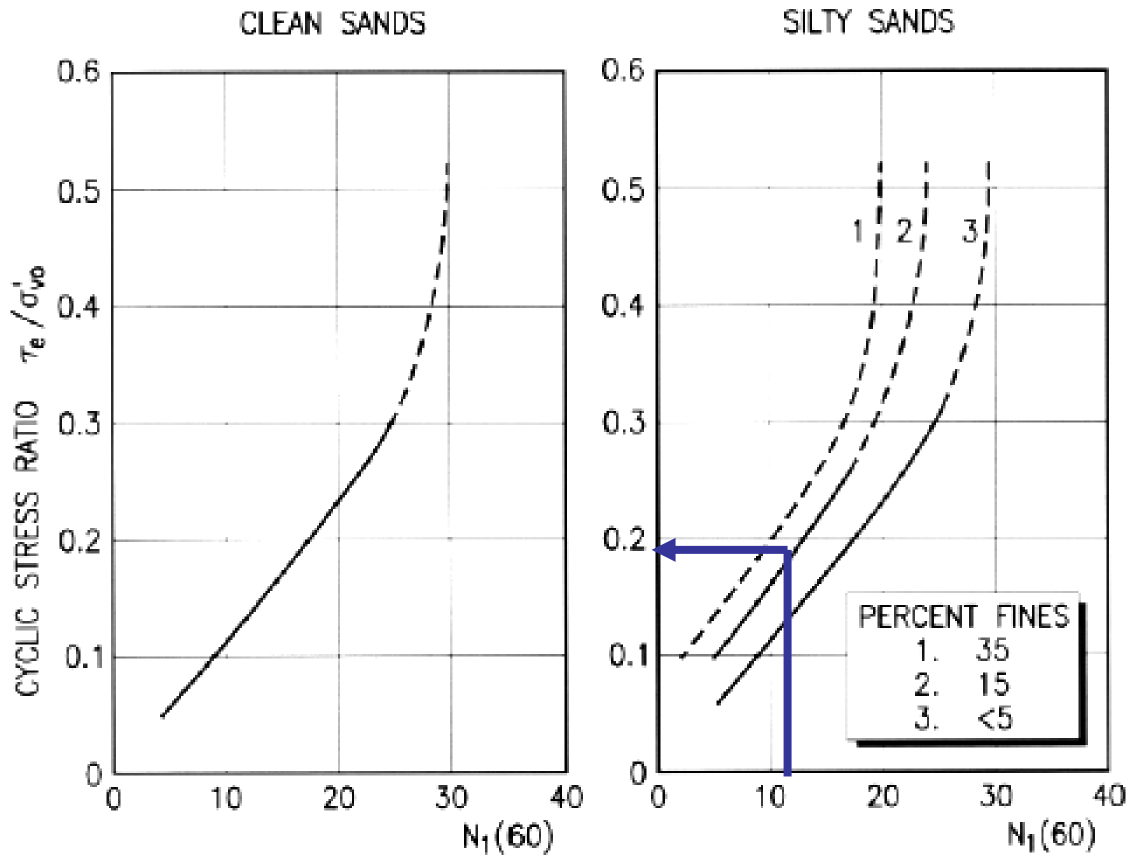
Ground type	S
A	1,0
B	1,35
C	1,5
D	1,8
E	1,6

type 2 (low seismicity)

Evaluation of Liquefaction Potential - Procedures

EC8: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Step 2: Fines Content Correction of SPT -Calculation of CRR7.5



Evaluation of Liquefaction Potential - Procedures

EC8: Capacity – Cyclic Resistance Ratio (CRR) – 'strength':

Step 3: Correction of CRR7.5 for magnitude

M	5.5	6.0	6.5	7.0	8.0
CM	2.86	2.20	1.69	1.30	0.67

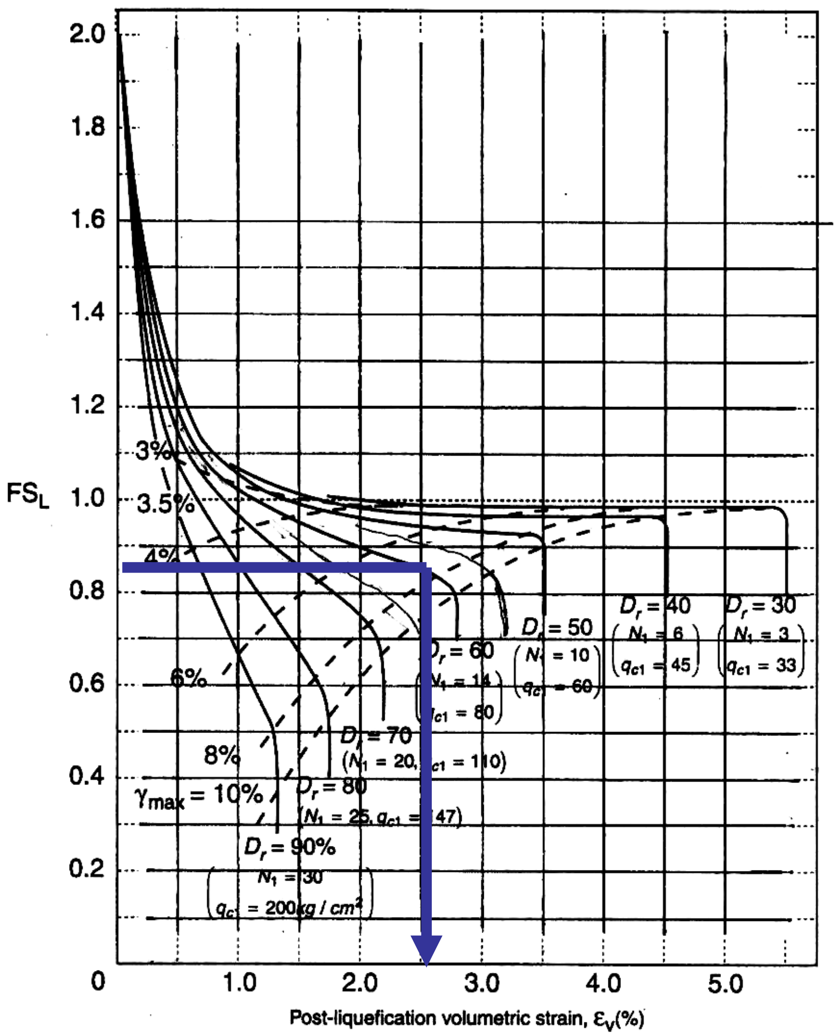
$$Capacity=CRR_M=CRR_{7,5}*CM$$

Step 4: Factor of Safety

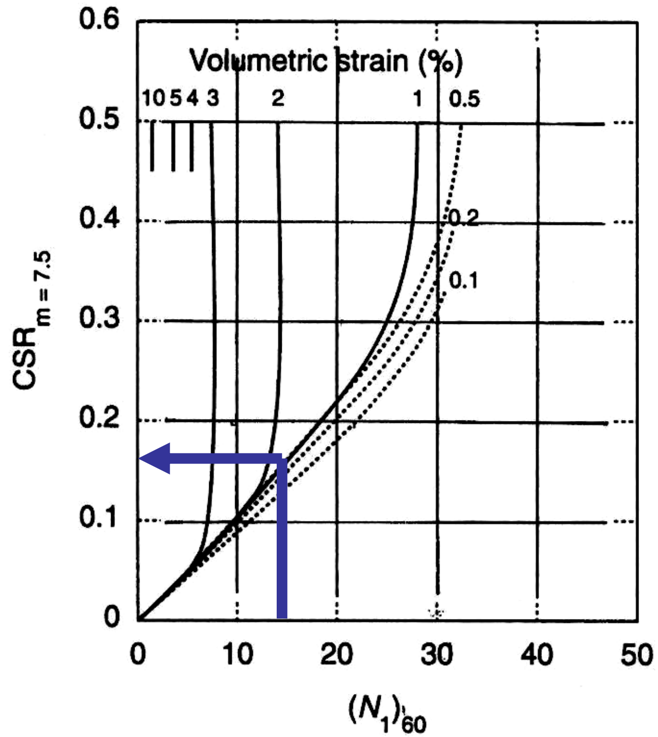
$$FS=\frac{CRR_M}{CSR}$$

A soil shall be considered susceptible to liquefaction under level ground conditions whenever the earthquake-induced shear stress exceeds a certain fraction of the critical stress known to have caused liquefaction. The recommended value is 80%, which implies a safety factor of **1.25**

Saturated layer: $S_{sat} = \frac{\varepsilon_c}{100} dz$

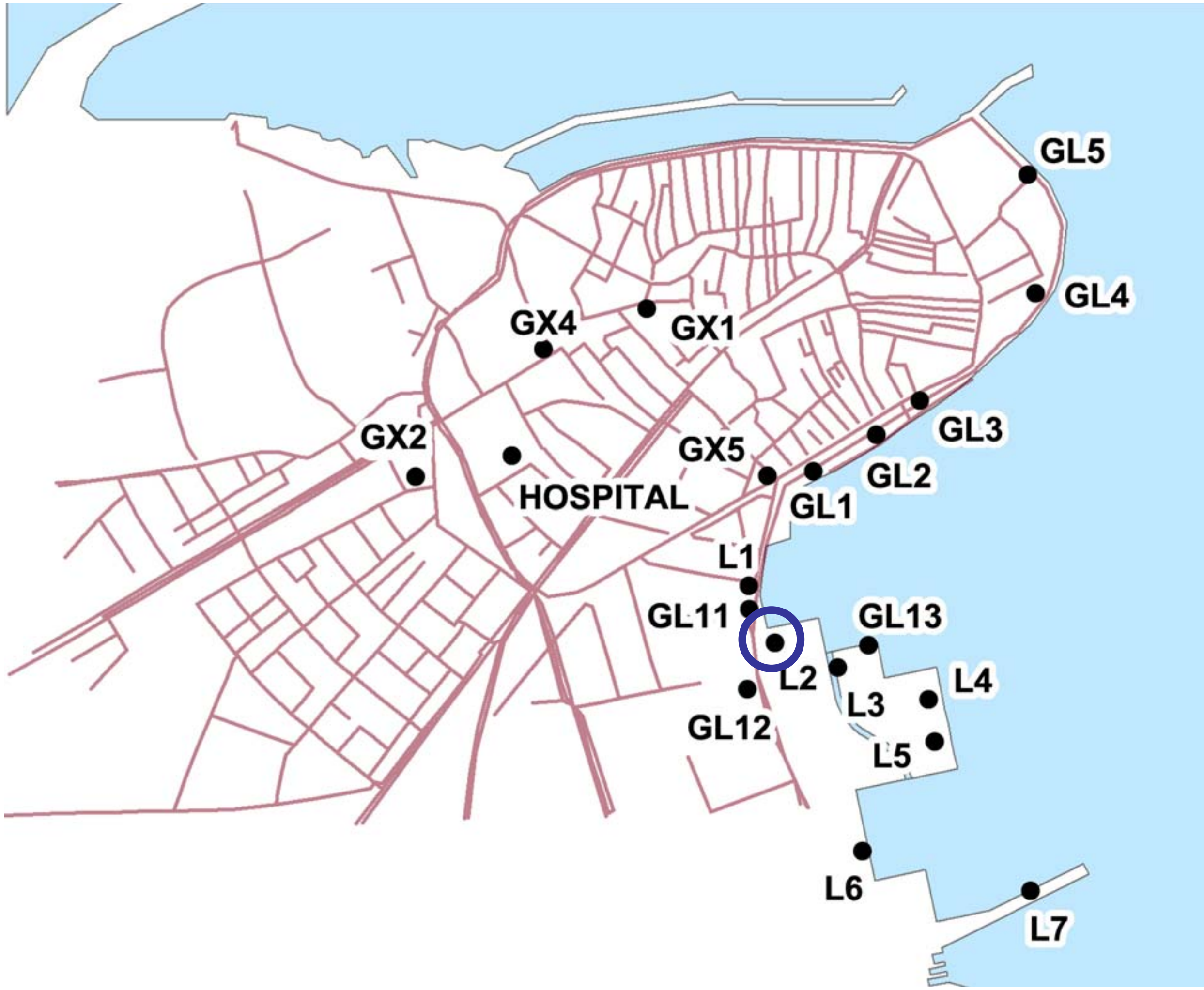


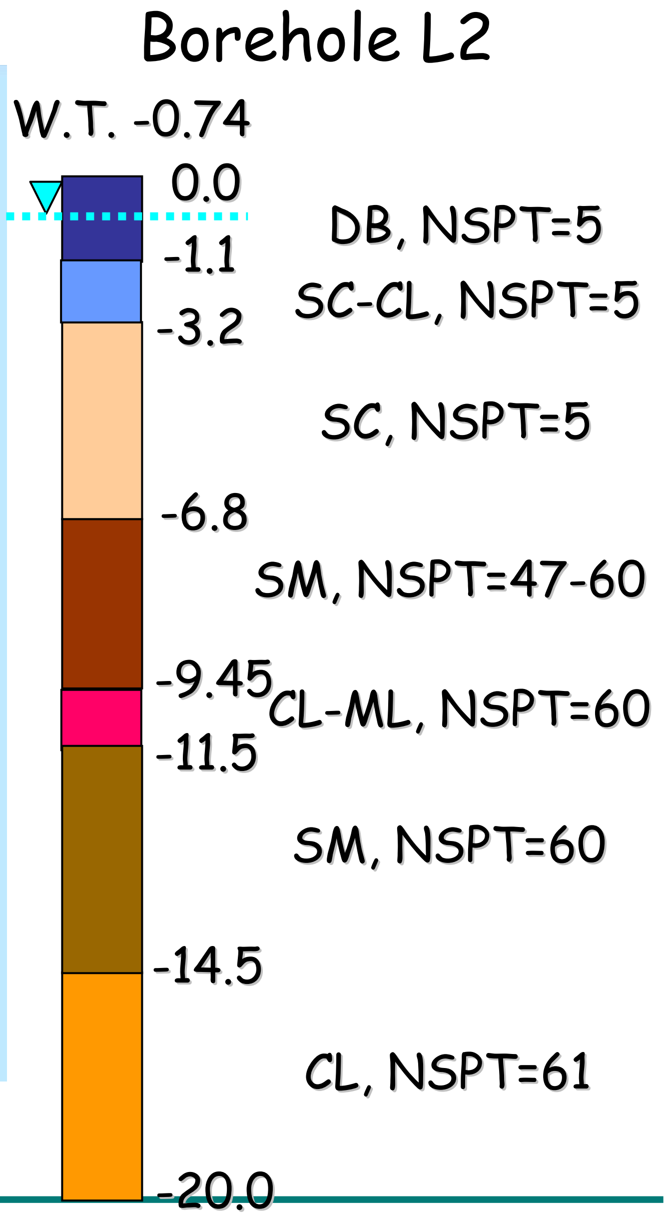
Ishihara & Yoshimine, 1992



Tokimatsu & Seed, 1987

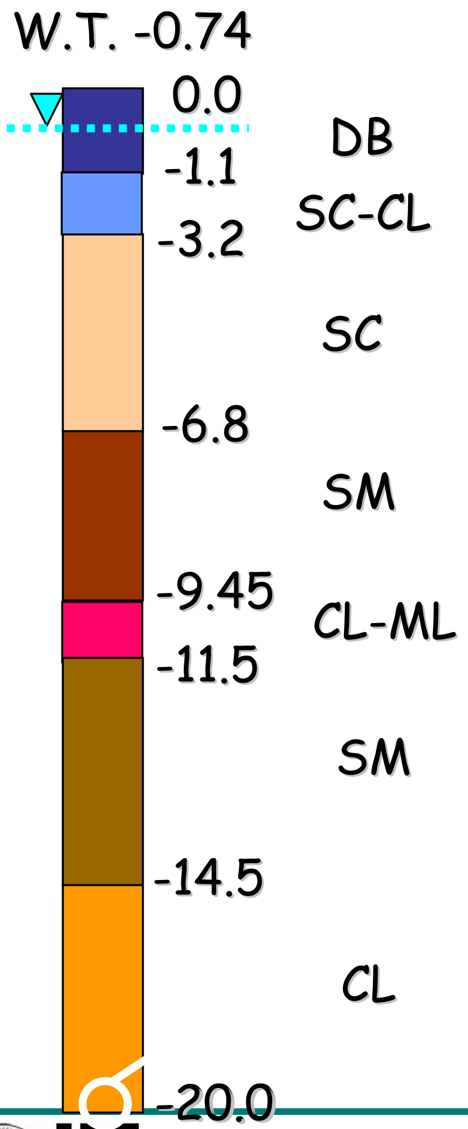
Lefkada's earthquake (2003): Liquefaction Assessment - example



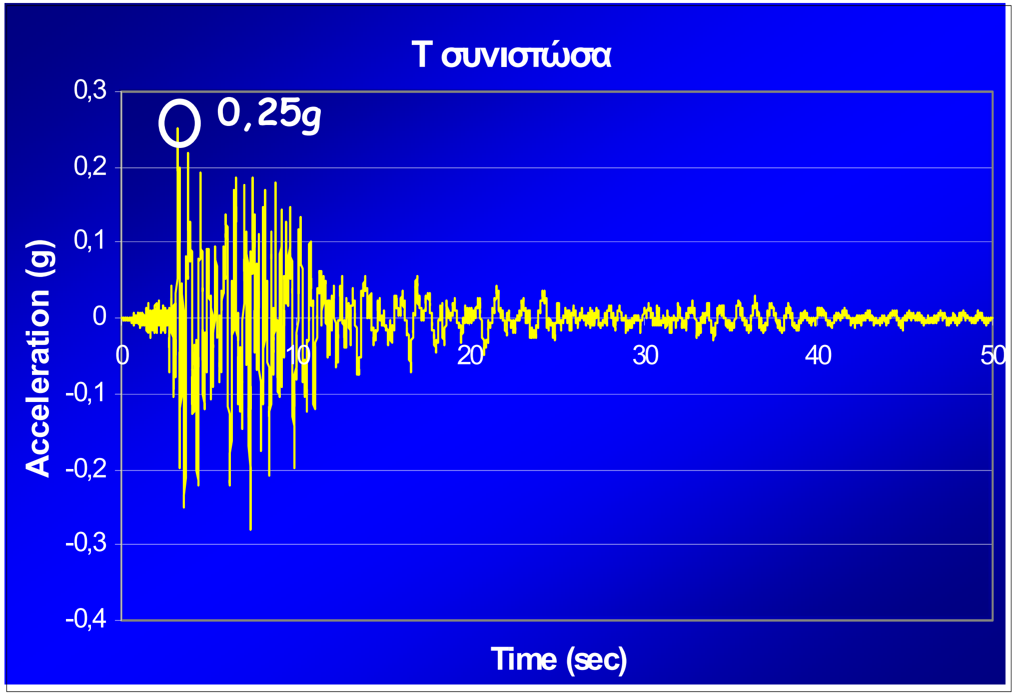


Observed permanent deformations: 30cm

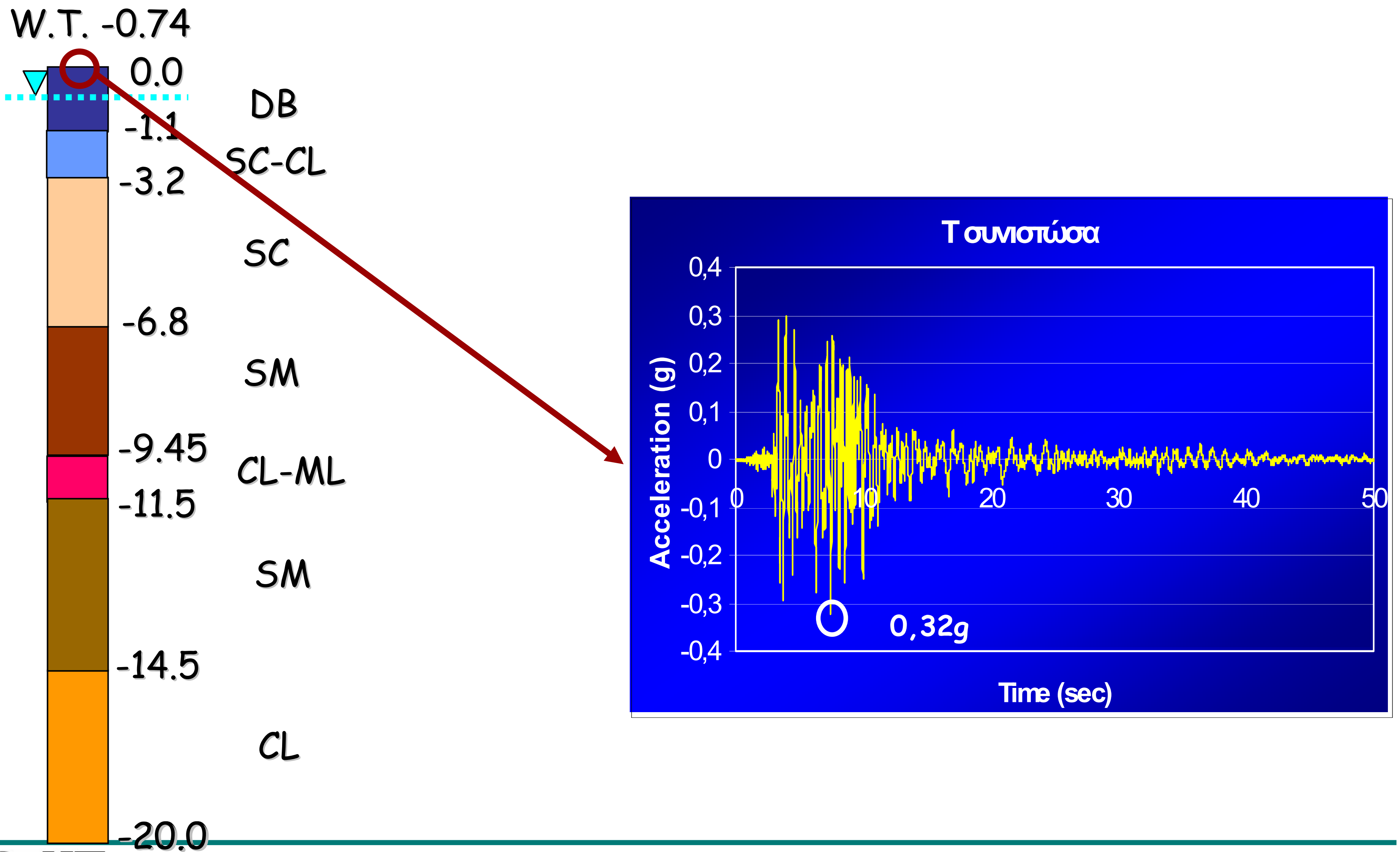
Lefkada's earthquake (2003): Liquefaction Assessment - example



1D Equivalent Linear Analysis: Input Motion=deconvolved time history of the recorded T-component at Hospital

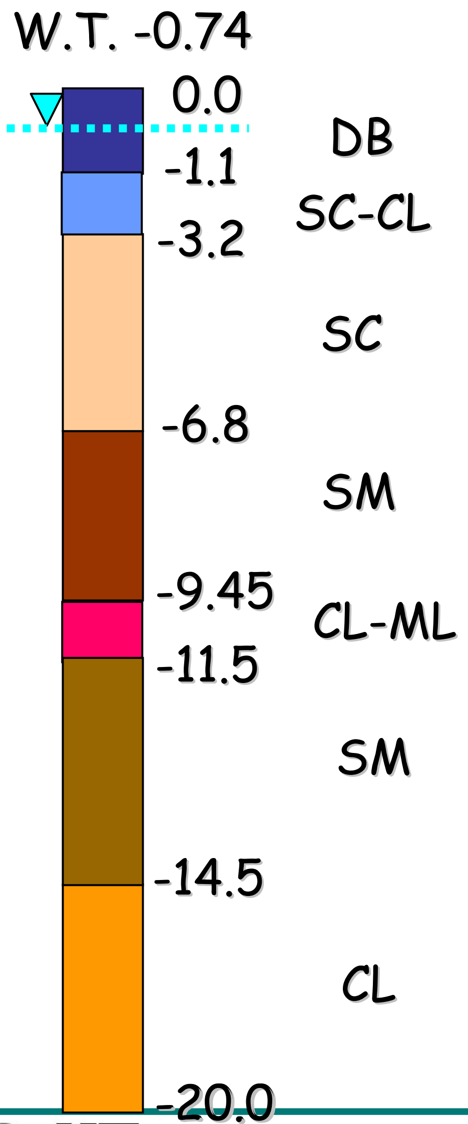


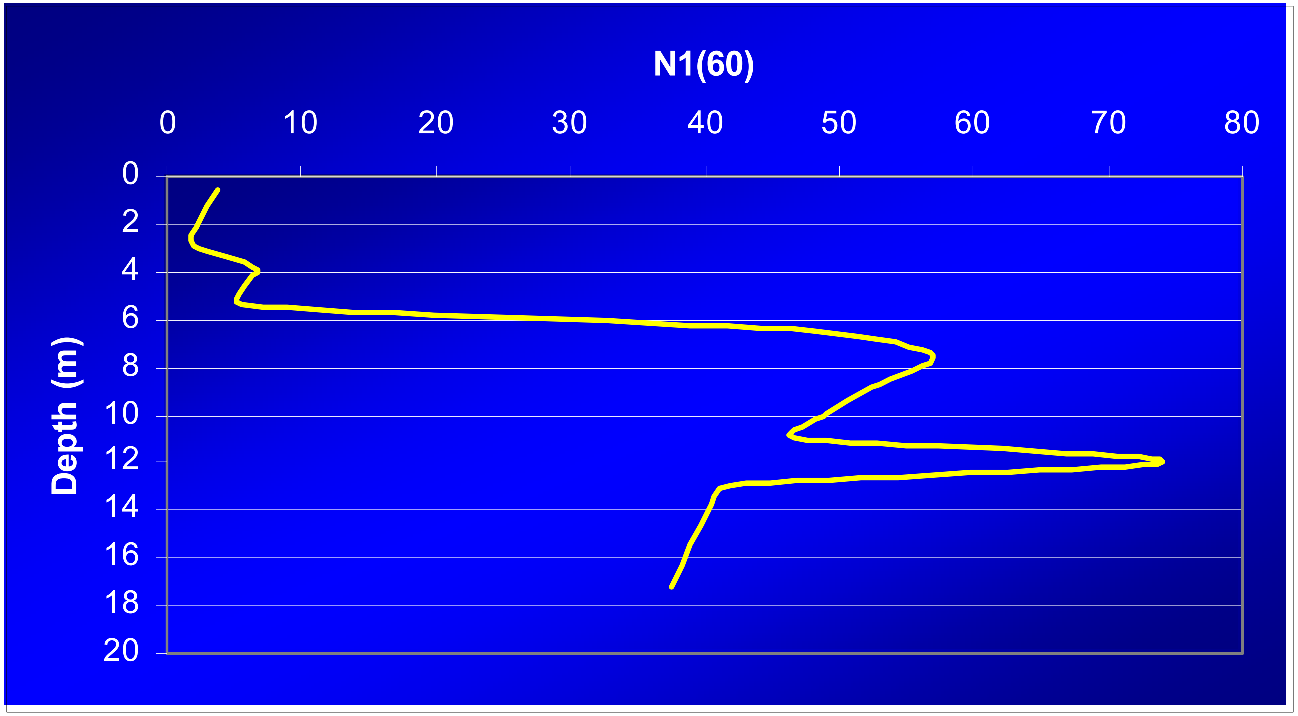
1D EQL Analysis



Lefkada's earthquake (2003): Liquefaction Assessment - example

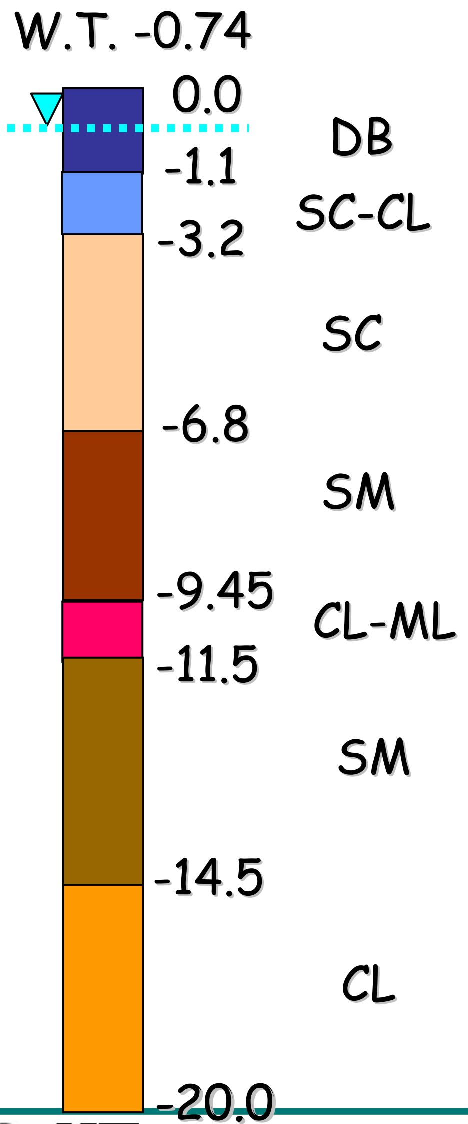
M=6.4
PGA=0.32g

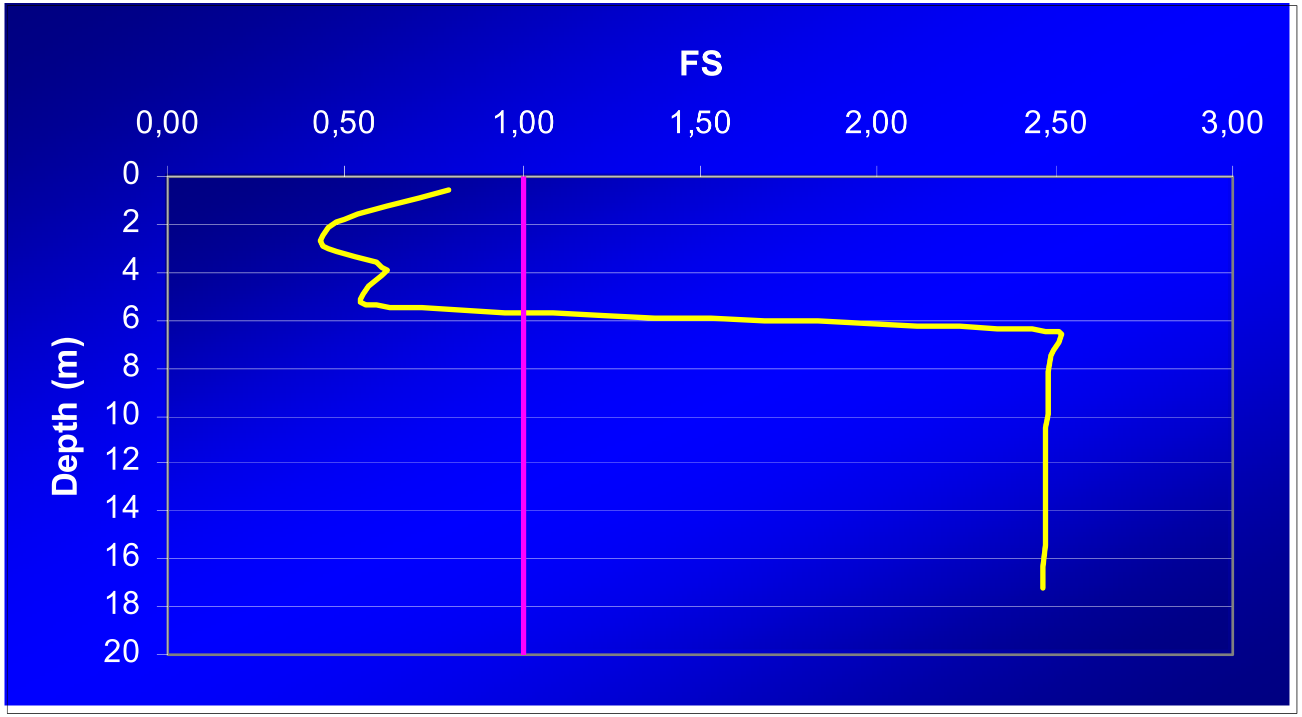




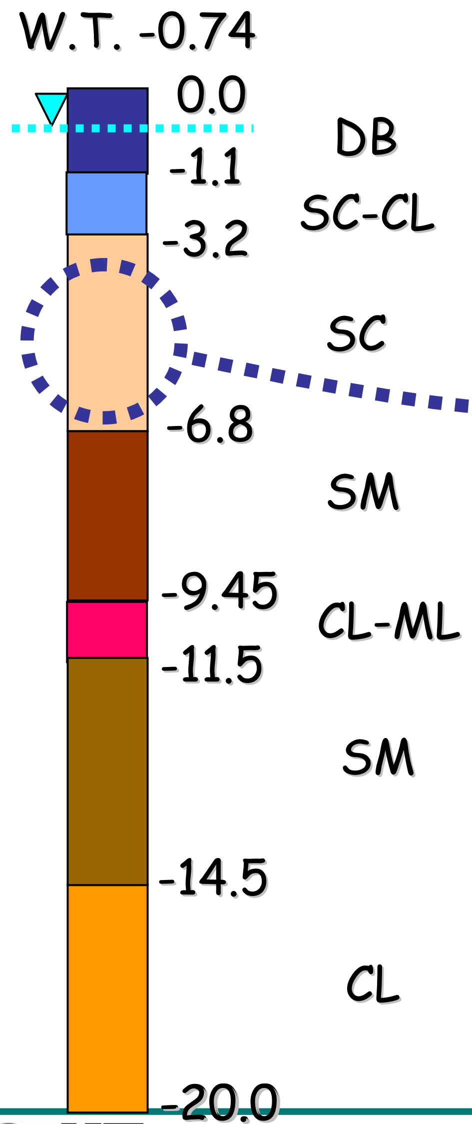
Lefkada's earthquake (2003): Liquefaction Assessment - example

M=6.4
PGA=0.32g





Lefkada's earthquake (2003): Liquefaction Assessment - example



M=6.4
PGA=0.32g

Z= -5m, N_{30}(SPT) = 5
Fines =24%, Vs=150m/sec[2]

σ_v= 123.6 kPa
u = 45.2 kPa
σ'_v= 78,4 kPa
(Equivalent linear analysis)

a_{max}=0,26g   S=0,9

$$\tau_e = 0.65 \cdot a \cdot S \cdot \sigma_{vo}$$

'load'= **CSR = 0.384**

Lefkada's earthquake (2003): Liquefaction Assessment - example

Z= -5m, N_{30}(SPT) = 5
Fines =24%, Vs=150m/sec^2

N_{SPT}=5,0

C_N=1.13

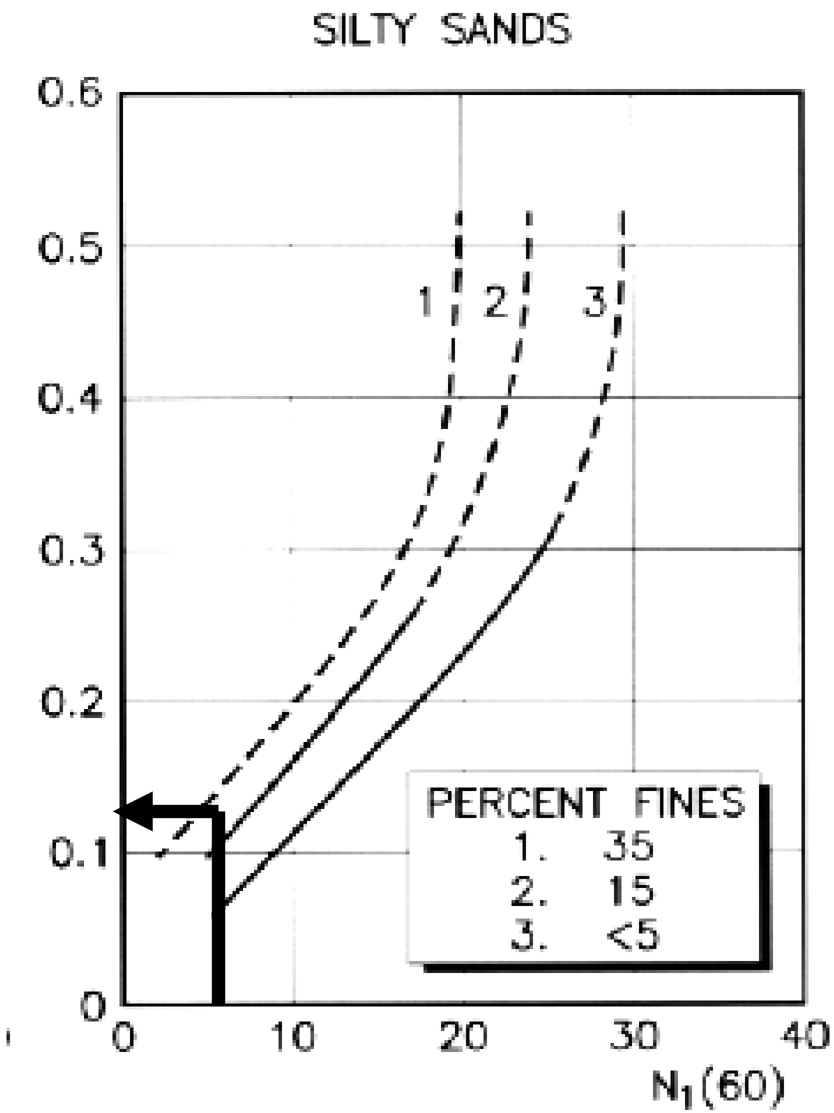
N1(60)=6,0

Fines=24%

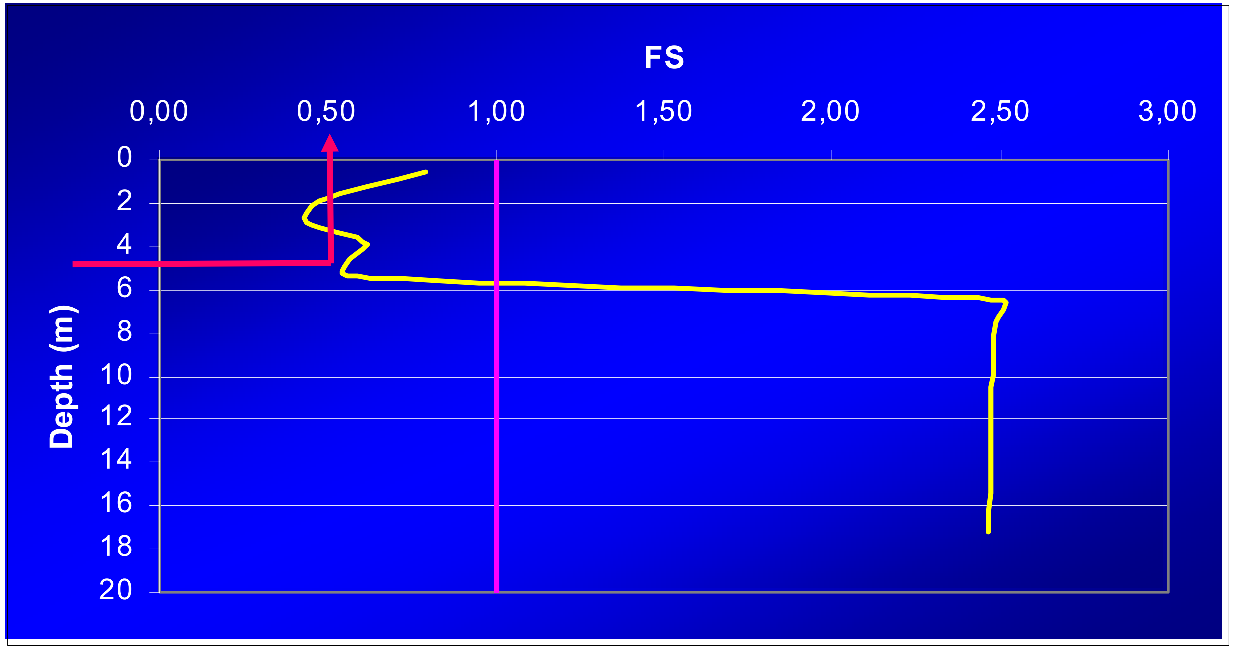
$CRR_{7.5}$ = 0.120

CM=1.792

$CRR_{6.4}$ = 0.215



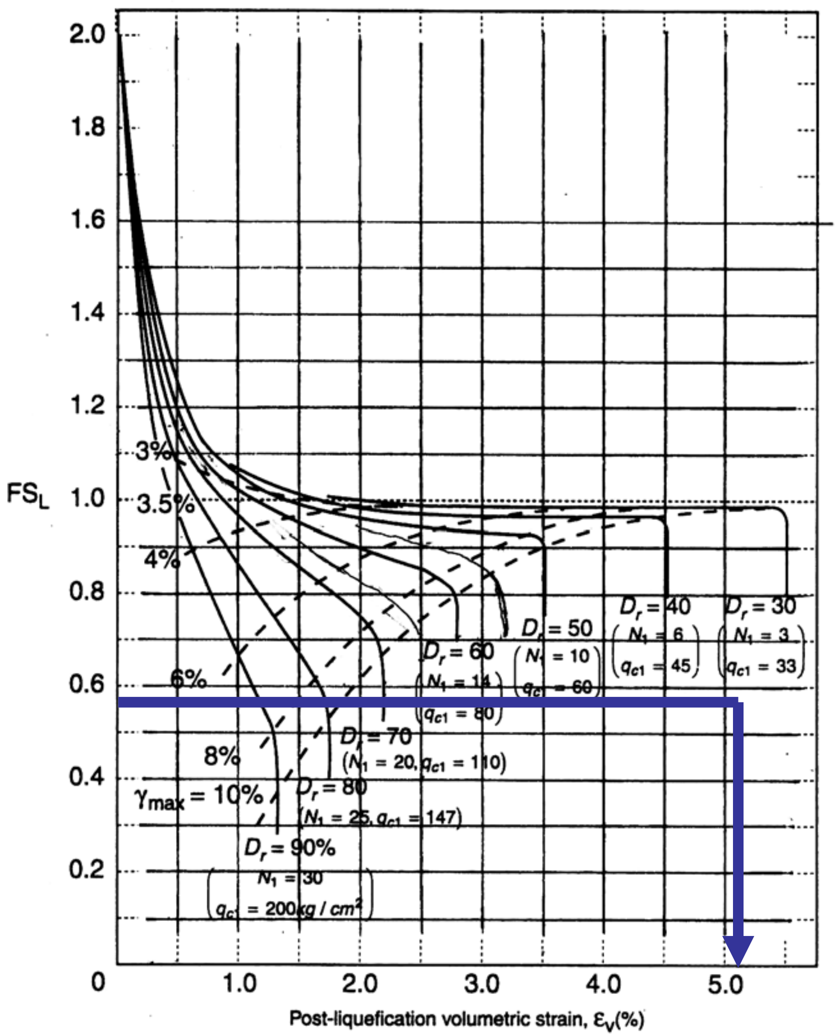
Lefkada's earthquake (2003): Liquefaction Assessment - example



$$FS = \frac{CRR_M}{CSR}$$

FS=0.215/0.384=0.560

Lefkada's earthquake (2003): Liquefaction Assessment - example



Ishihara & Yoshimine, 1992

Calculation of Settlements

N1(60)=6,0

N1=4,7

FS=0.560

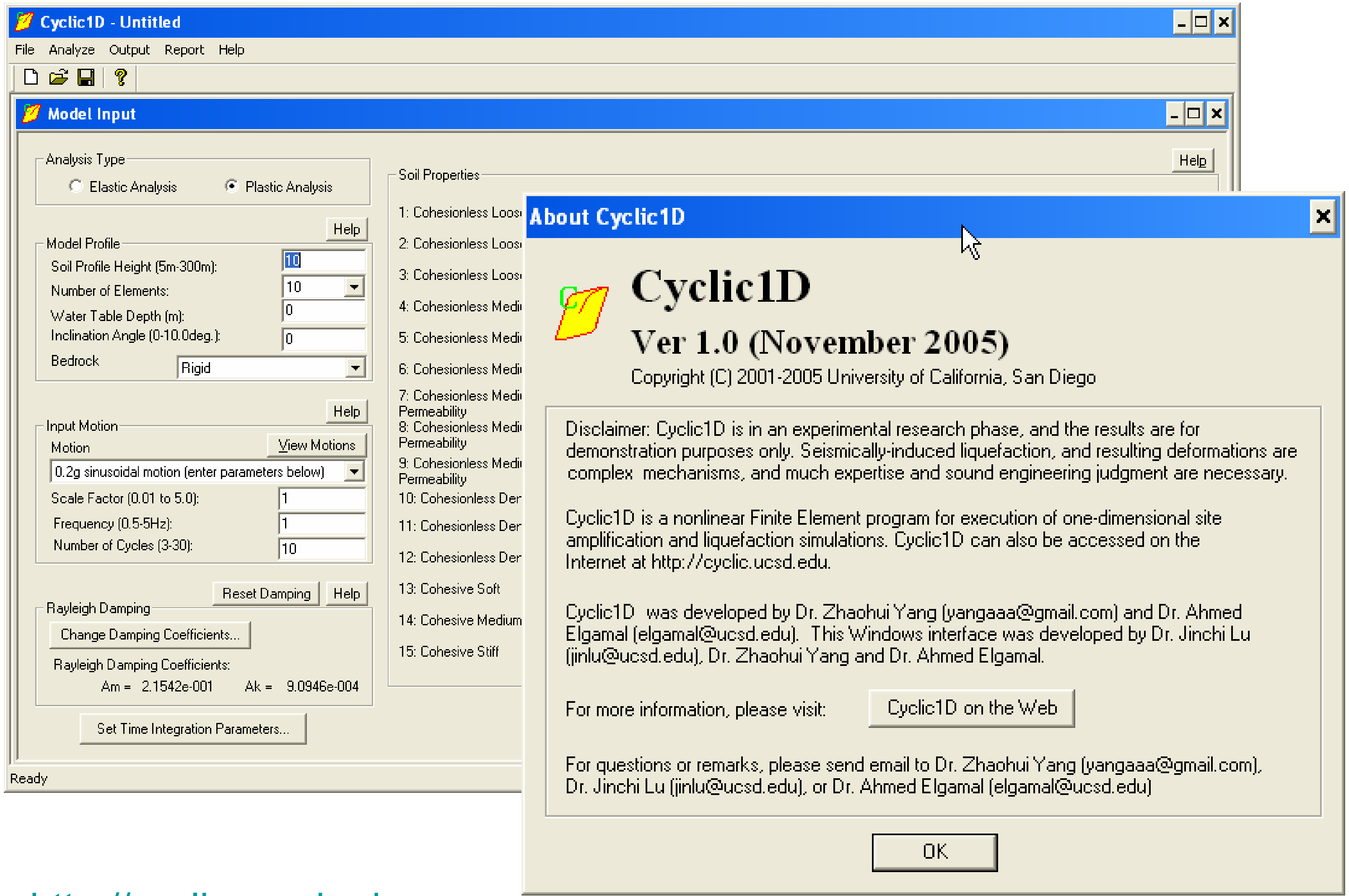
ε_v=4,9%

ΔH=0,074m

Total Settlement

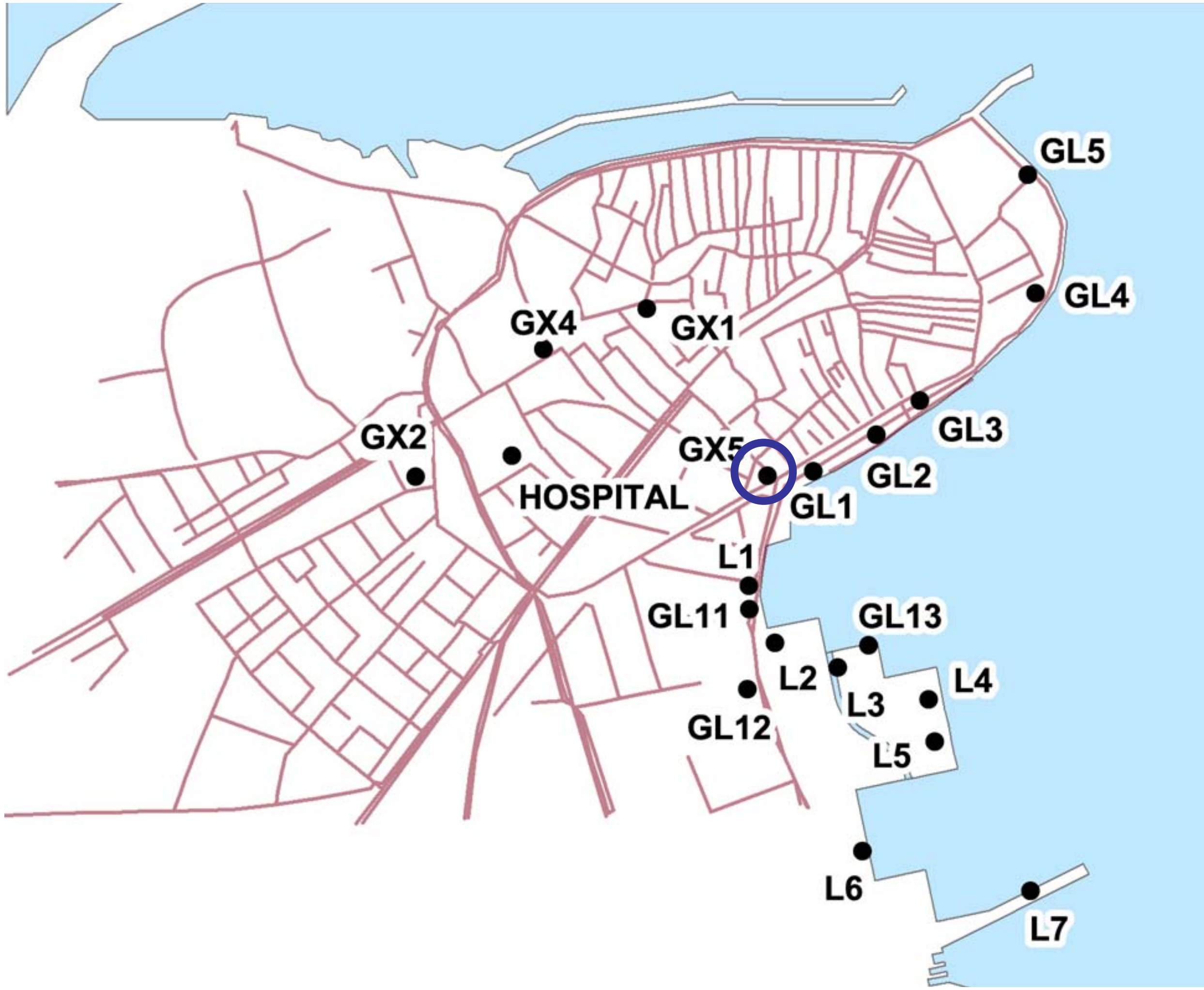
ΔΗολ=0,028m=28cm

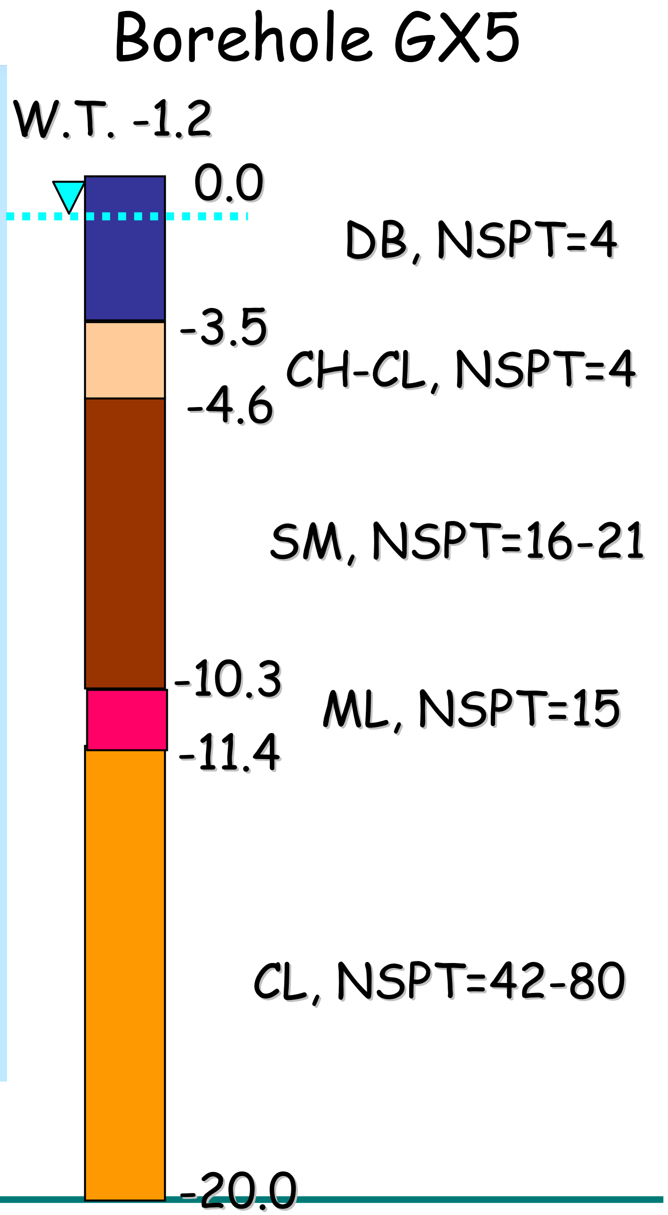
Cyclic 1D – Non Linear effective stress code



http://cyclic.ucsd.edu

Lefkada's earthquake (2003): Liquefaction Assessment - example

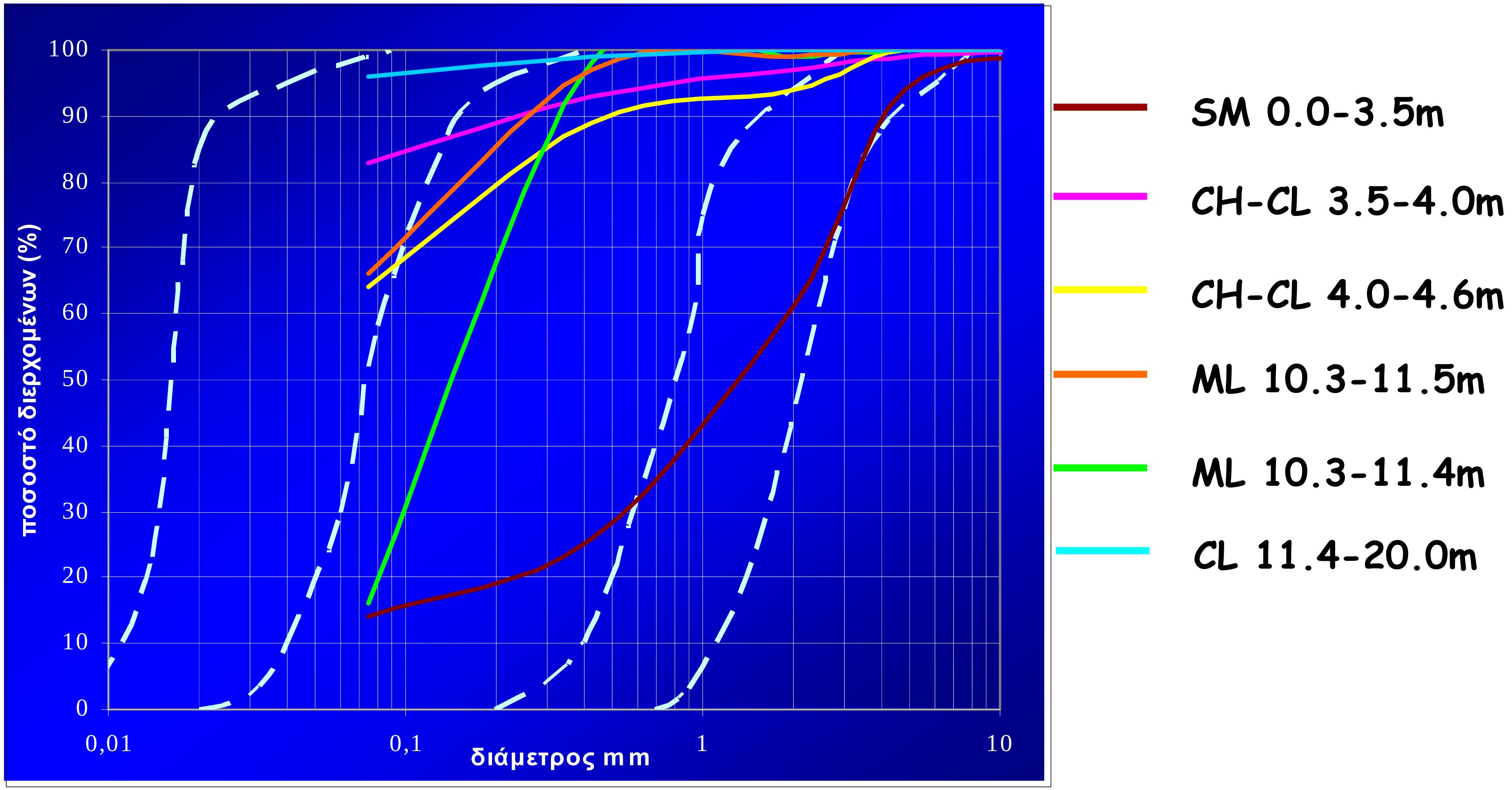




Observed permanent deformations : 2,5cm

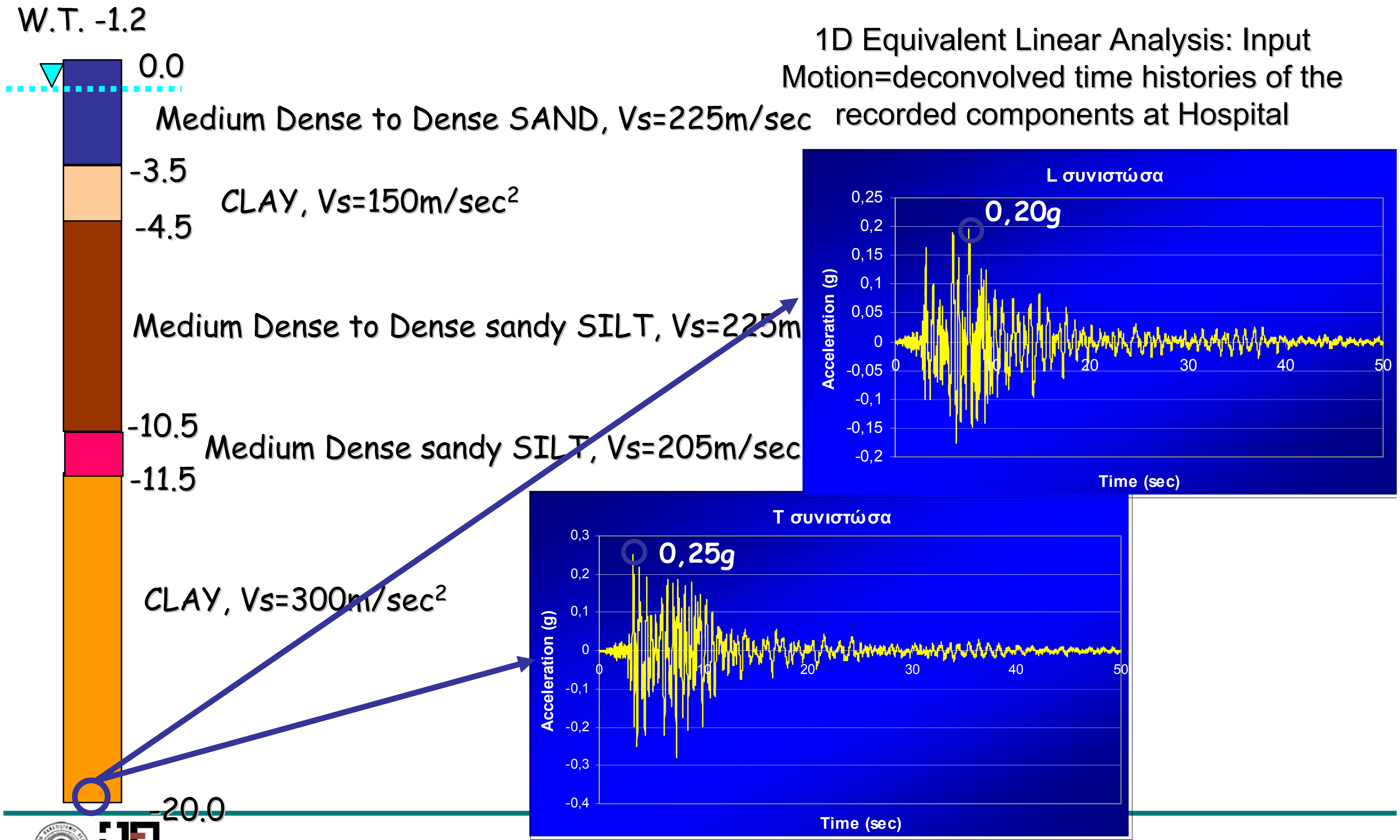
Lefkada's earthquake (2003): Liquefaction Assessment - example

Liquefaction Susceptibility



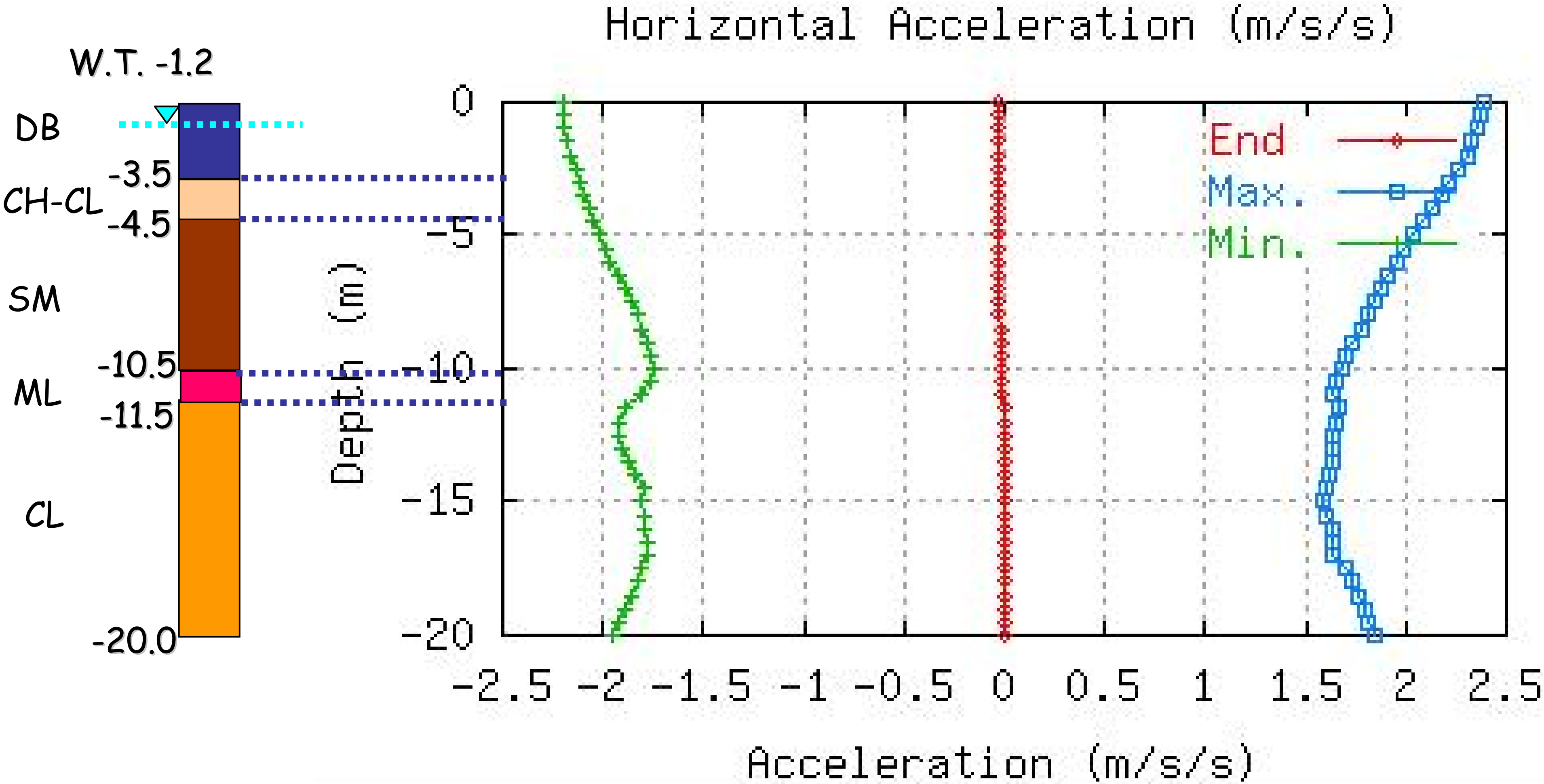
Lefkada's earthquake (2003): Liquefaction Assessment - example

Soil Model Cyclic1D



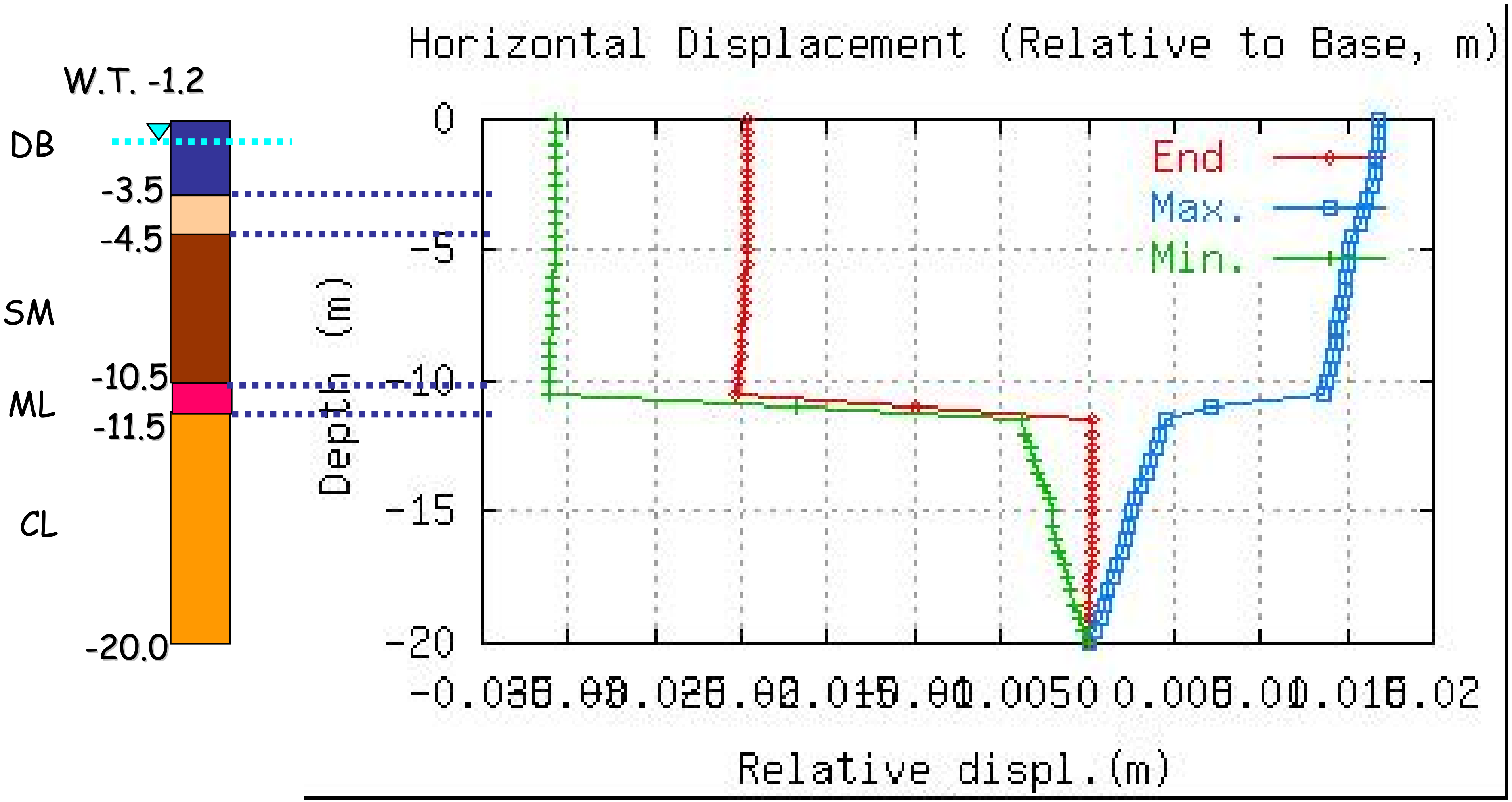
Lefkada's earthquake (2003): Liquefaction Assessment - example

T component



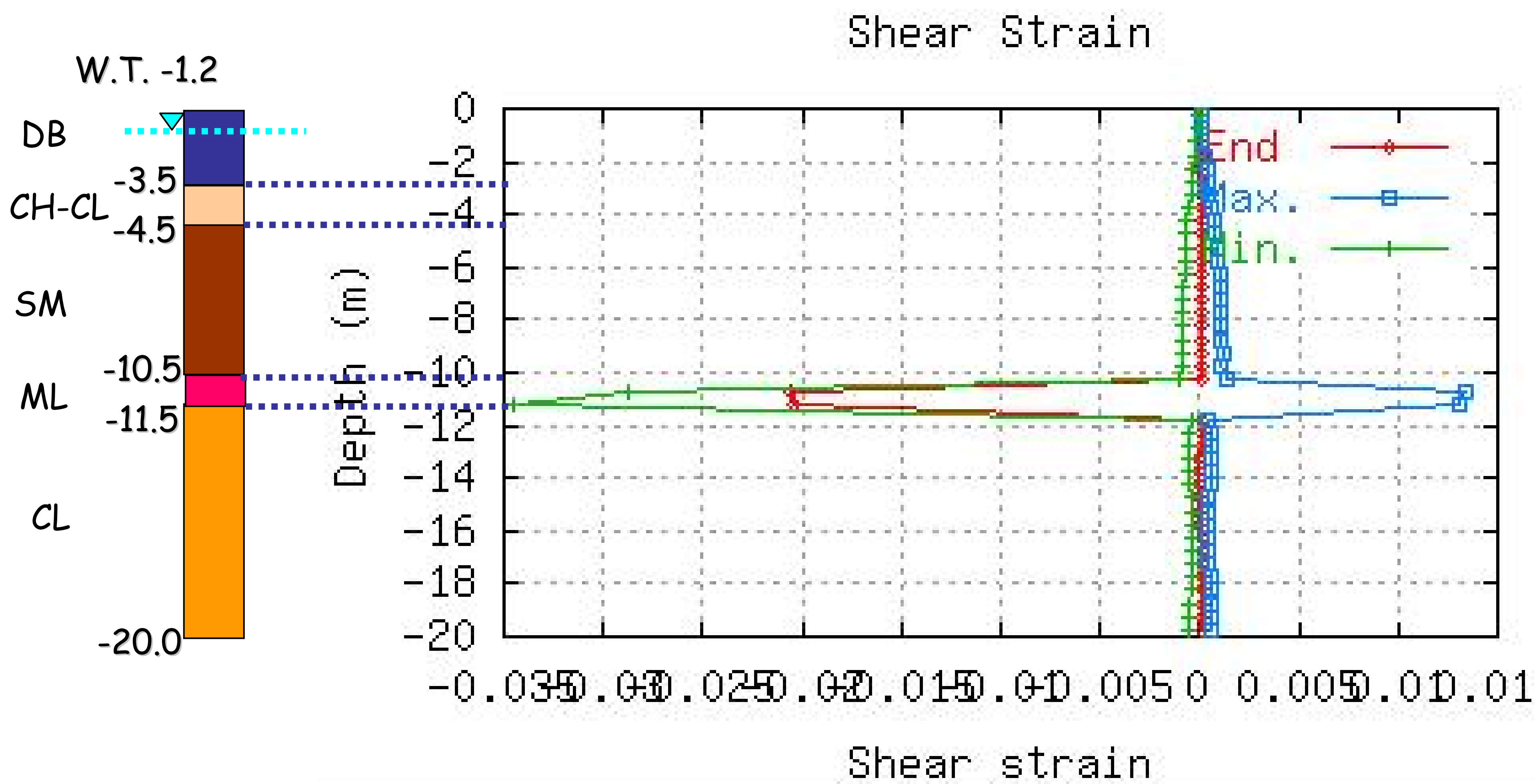
Lefkada's earthquake (2003): Liquefaction Assessment - example

T component



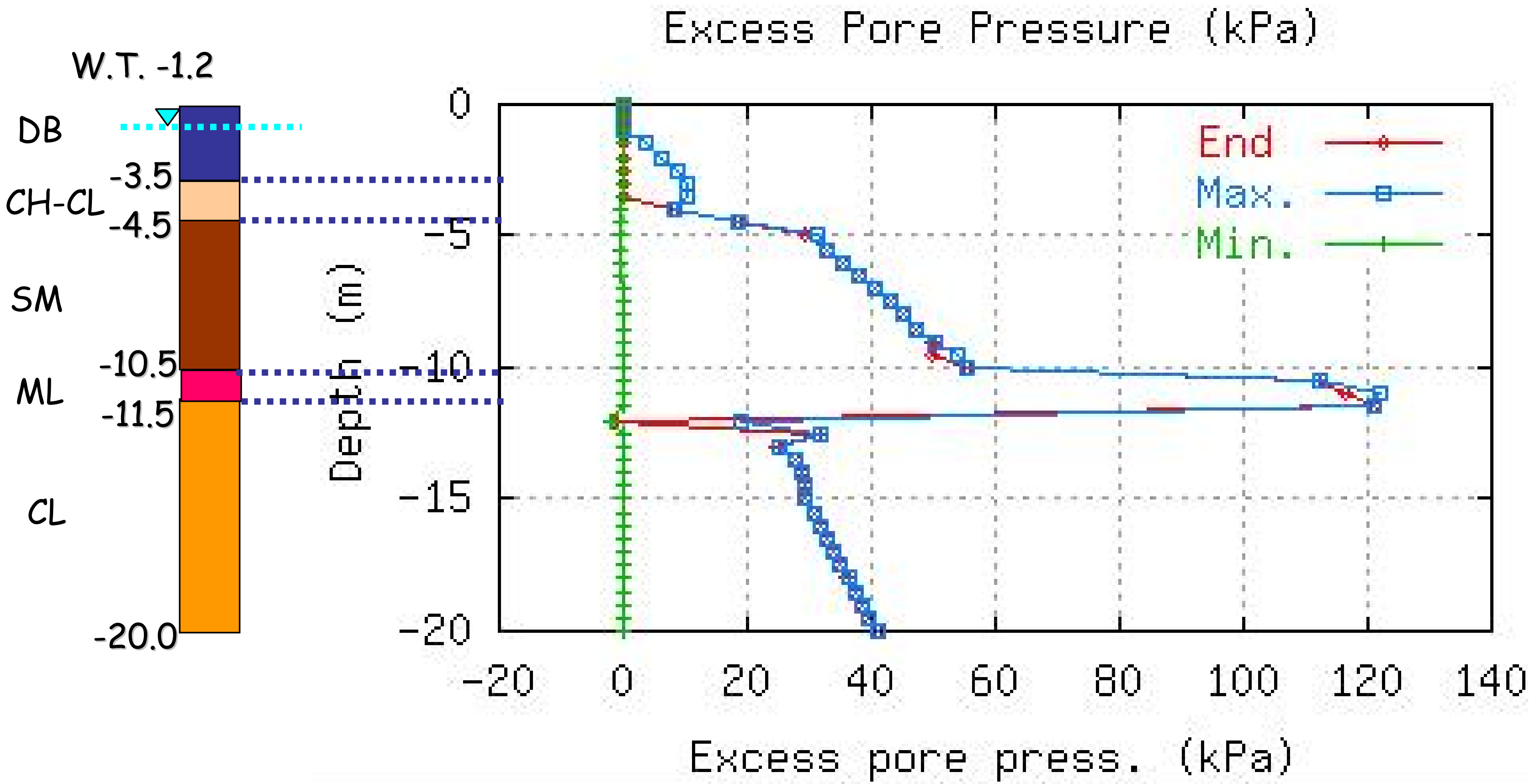
Lefkada's earthquake (2003): Liquefaction Assessment - example

T component



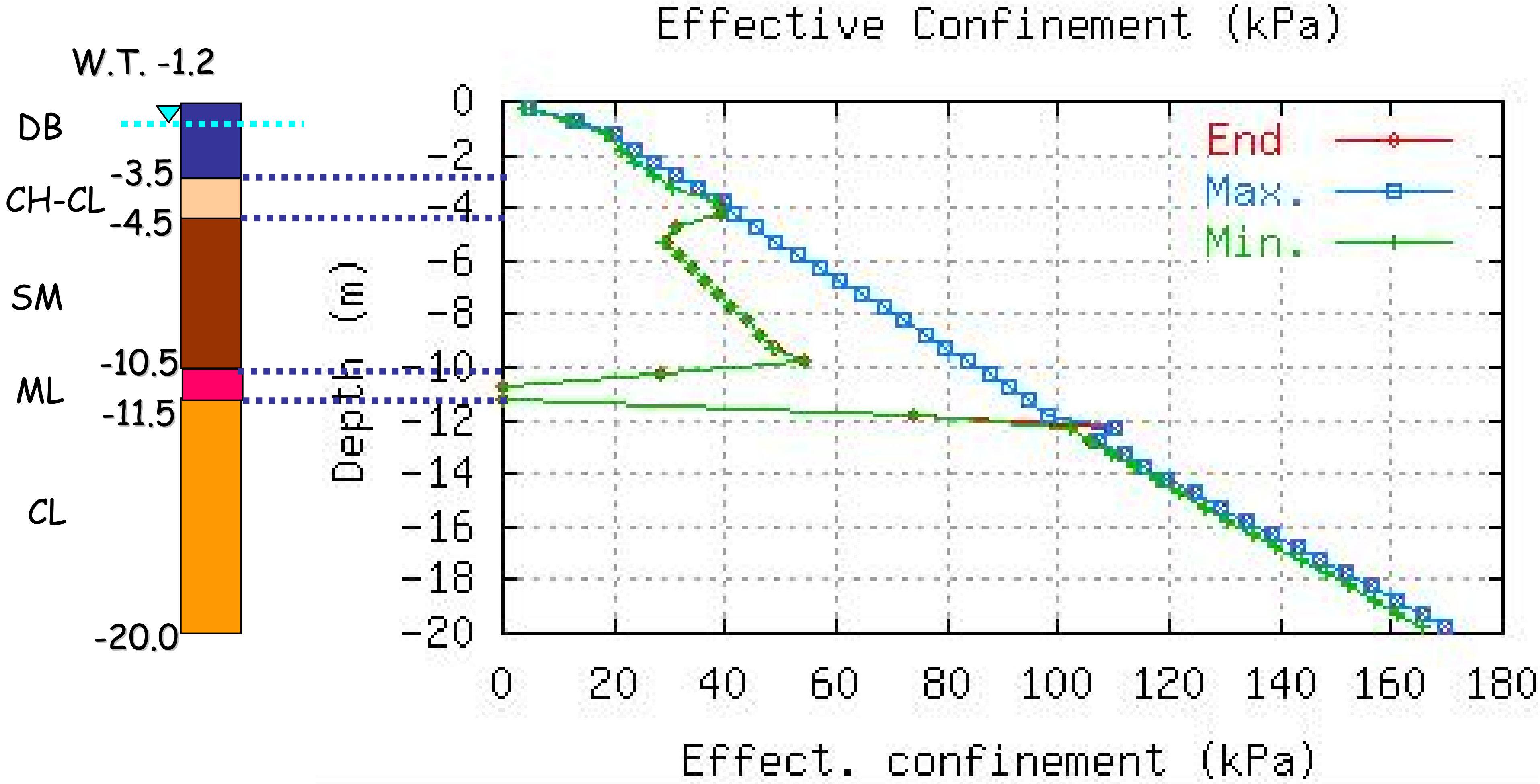
Lefkada's earthquake (2003): Liquefaction Assessment - example

T component



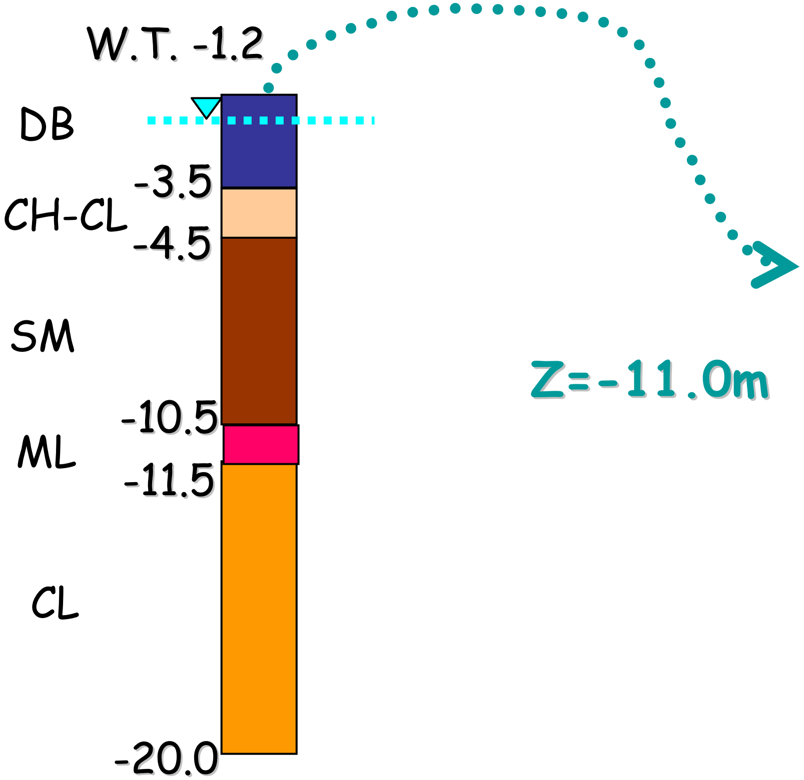
Lefkada's earthquake (2003): Liquefaction Assessment - example

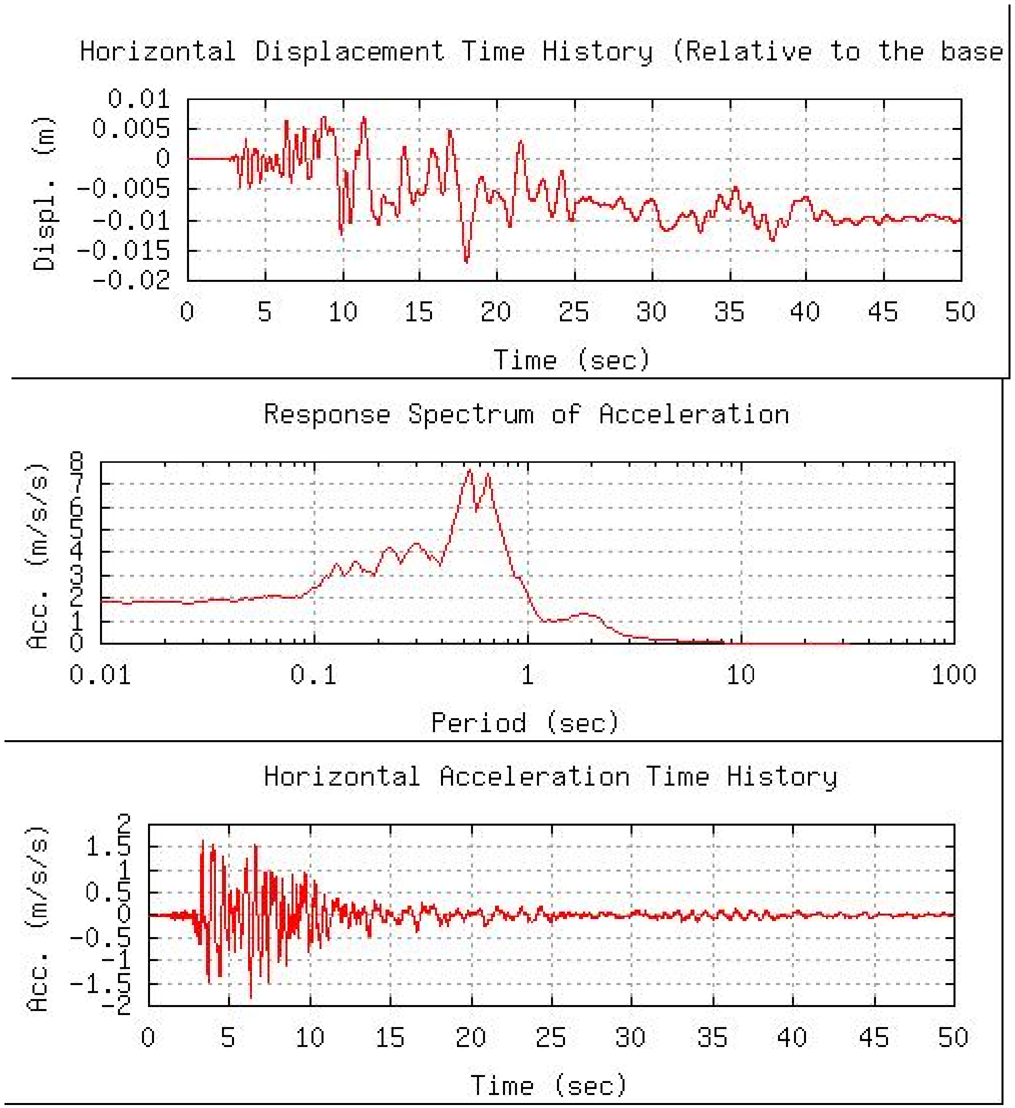
T component



Lefkada’s earthquake (2003): Liquefaction Assessment - example

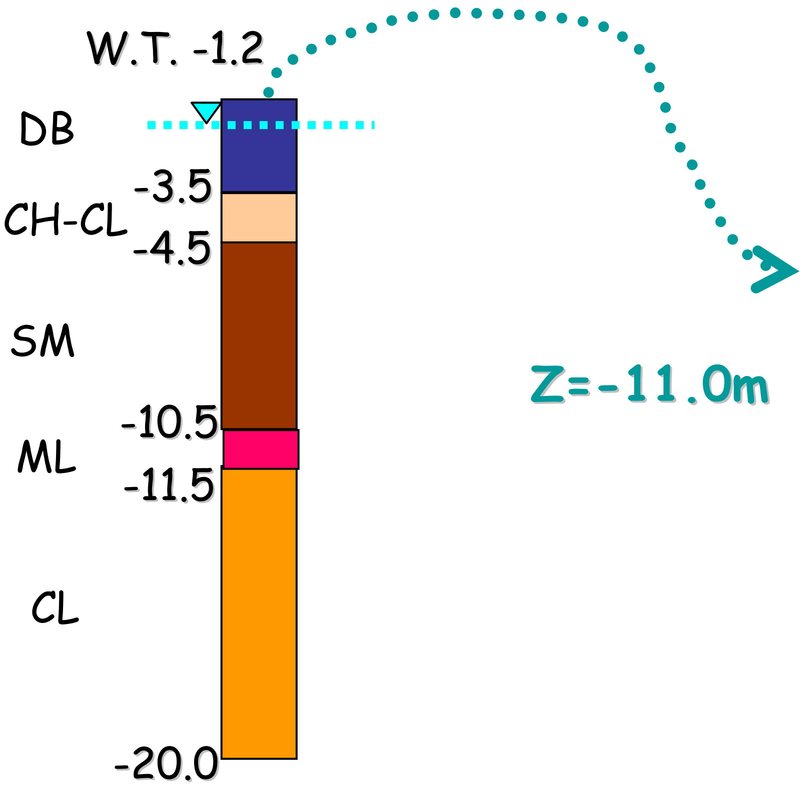
T component

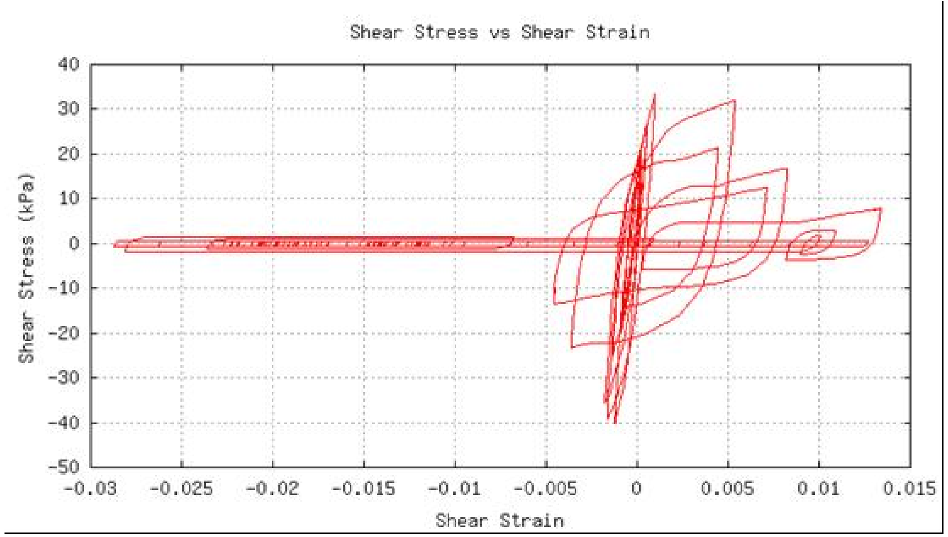


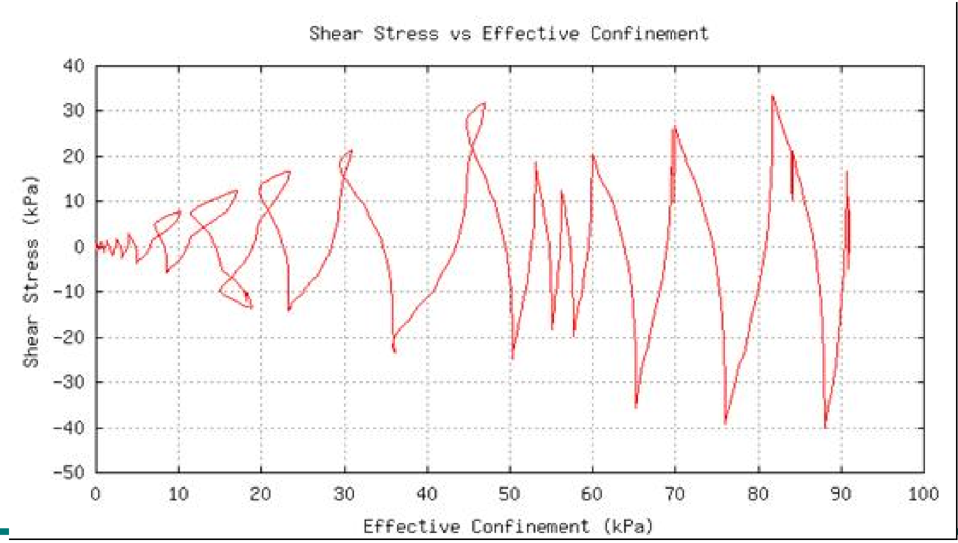


Lefkada's earthquake (2003): Liquefaction Assessment - example

T component







Recommendations For Specific Site Response Analysis Studies

A site specific analysis study requires a **methodological criterion**

a) the available information
b) Budget / available time
c) risk level of the area under study.

Taking into account …

- the majority of the techniques for the estimation of local effects
- the variation of their cost/accuracy
- the information they require (which is not always available)
- the nature of the results (quantitative, not always comparable and usable in a straightforward manner in a regulatory context)
- the required expertise in their use which is not always available

Concluding Remarks

One dimensional body wave propagation models are the basic tool for ground response analyses. In their simplest form (i.e. linear elastic or equivalent linear elastic soil behaviour) they are rather simple while they need for few parameters which are easily estimated even without performing specific dynamic field and laboratory tests, as there are many correlations with conventional geotechnical parameters (i.e. *Vs*-*SPT*, *Vs*-*CPT*, *G/Go-γ-DT%* with *PI* and *DR%* for clays and sands etc).

Generally 1D models are reliable for nearly horizontally layered deposits and in cases when the impedance contrast between soil deposits and underlying rock is the controlling parameter of ground motion. The velocity of the bedrock and the incident wave field characteristics are playing an equally important role. With the 1D modelling the higher frequency parts of the expected ground motion can be captured quite accurately. Low frequency parts are less reliable and this is an important shortcoming for the case of deep basins (>300m).

Concluding Remarks - Needs :

- Next generation of well focused and designed strong motion networks (surface, down-hole arrays)
- Improved knowledge of soil and site conditions for site effects
- Validation of existing models with well constrained data
- Development of accurate low cost In-situ survey techniques

"SITE EFFECTS" - IN ENGINEERING PRACTICE

- **CODE ORIENTED**
- **Complex site effects**
- **Microzonation-CODES**
- **MORE DATA (well designed-focused) - Test Sites**
- **Combined efforts**
- **Cooperation at European Level**

Thank you for
your attention …