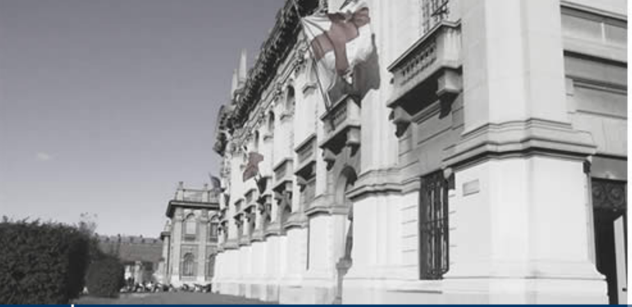

Workshop
Eurocodes – Background and Applications

Brussels, February 18-20, 2008

POLITECNICO DI MILANO





Eurocode 8
Part 5: Foundations, retaining structures and geotechnical aspects (EN1998-5: 2003)

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Outline

- Object and salient characteristics of EN1998-5:2003
- Ground properties (strength, stiffness and damping parameters)
- Design seismic action and its dependence on ground type
- Requirements of construction site and of foundations soils
- Foundation system: shallow and deep foundations
- Earth retaining structures

Object of EN1998-5:2003

The norm:

- "rules for the siting and foundation soil of structures for earthquake resistance", and
- "covers the design of different foundation systems, .. of earth retaining structures ……under seismic actions"

From Part 1: *"…. It shall be verified that both the foundation elements and the foundation soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations. "*

Salient characteristics and innovative aspects

- Complementarity with Eurocode 7 (EN 1997e), which does not cover earthquake resistant design.
- Introduction and use of dynamic soil properties (τ_{cyc}, shear wave velocity v_s and damping) in addition to standard static properties ($tan\ \phi'$, c_u, q_u).

"4.2.2 (2) The profile of the shear wave propagation velocity in the subsoil shall be regarded as the most reliable predictor of the site dependent characteristics of the seismic action at stable sites"

- Different approaches to safety and strength verifications, depending on seismicity level and type of soil.

"4.1.2.3 (8) Simplified methods (of slope stability analysis), such as the pseudo-static ones, shall not be used for for soils capable of developing high pore water pressures or significant degradation of stiffness under cyclic loading".

- In situ investigations, or gathering of reliable equivalent data, to determine the elastic design response spectrum.
- Recognition of seismically-induced permanent ground deformations as a design criterion.

Ground properties

- Static undrained parameter values (c_u, $\tan\phi'$) can generally be used for strength verifications, with recommended values of material partial factors (γ_m).
- For cohesionless soil the strength parameter is the cyclic undrained shear strength $\tau_{cy,u}$ which should take the possible pore pressure build-up into account.
- The soil shear modulus:

$$G = \frac{w}{g} {v_S}^2$$

and the soil damping ratio are introduced, for use in SSI calculations, as well as their dependence on the seismic shear strain in the soil (through the design ground acceleration).

- For the evaluation of the liquefaction potential, the cyclic soil resistance against liquefaction (based on field performance in past earthquakes) is used, which depends on SPT blowcount or cone penetration resistance.

Design seismic action: dependence on ground type

The reference ground motion model at a point on the surface is the acceleration elastic response spectrum:

$$S_e(T) = a_g\, f\,(T; S, T_B, T_C, T_D)$$

The dependence is established through:

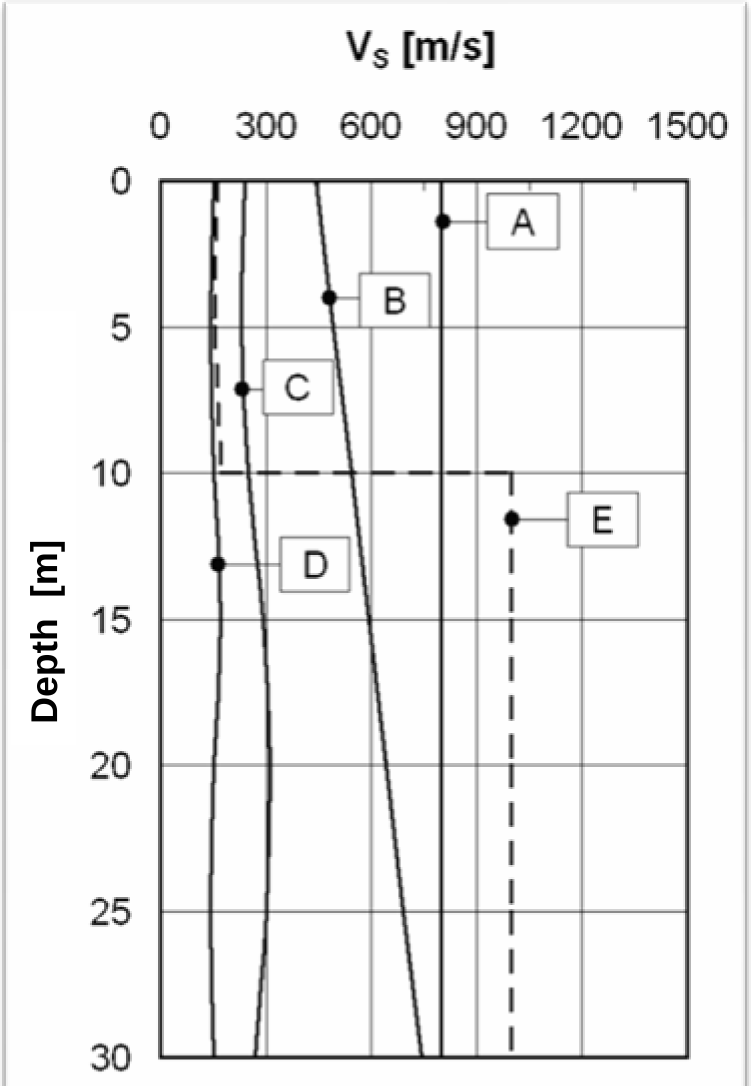
- The constant soil factor S, which does not modify the spectrum shape but only amplifies it, and takes a single value for each ground type ;
- The "control periods" T_B, T_C, T_D which modify the shape by enlarging the spectral plateau

Remark: it would be desirable to make S a smoothly varying function of V_{s30}, e. g.:

$$S = (V_{S30}/V_a)^{b_V}$$

with b_V and V_a constants

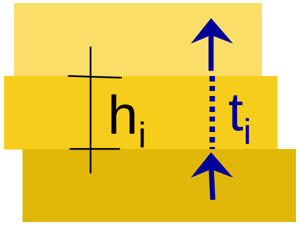
Identification of ground types



$$V_{S,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_{S_i}}}$$

Weighted V_S

$$\sum_{i=1,N} t_i$$



Particular cases:

Types S1,S2 → Special studies

Seismic action: dependence on ground type

In seismic geotechnical verifications, such as:

- Slope stability
- Liquefaction hazard occurrence
- Stability of retaining works

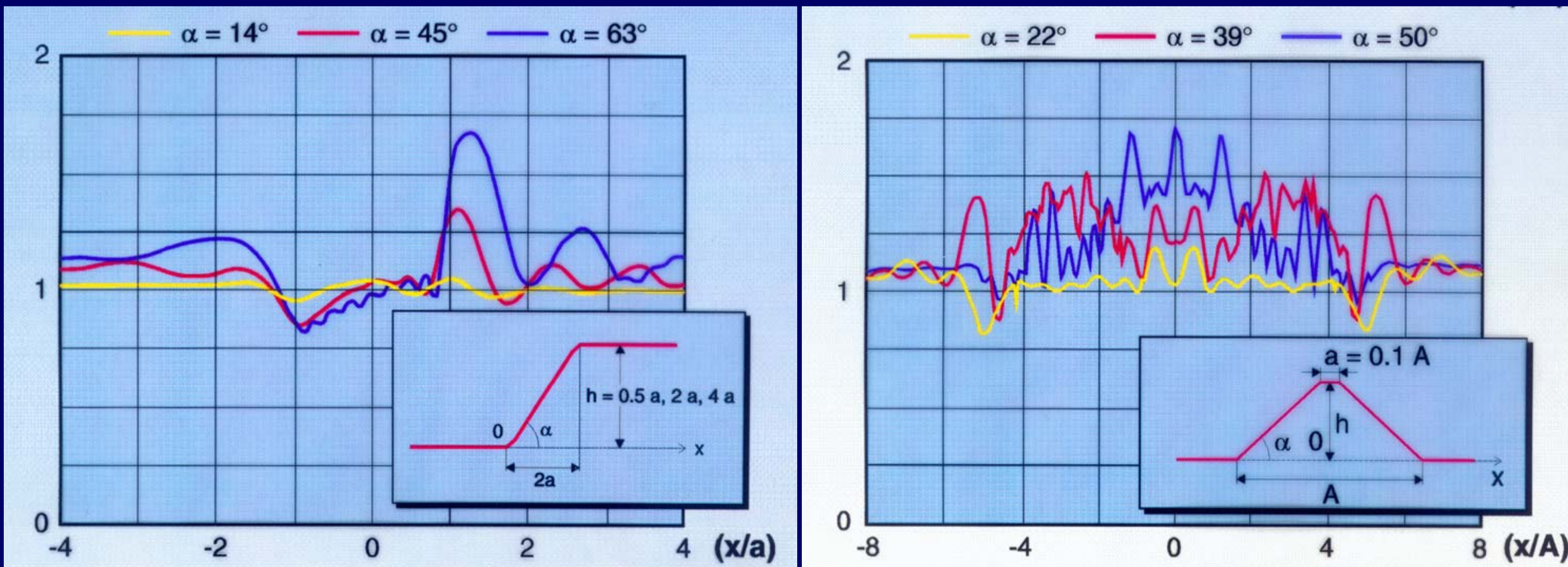
the sesmic action is directly determined by the design ground acceleration $a_g S$, multiplied by a reductive coefficient, as follows:

- $(0.5\ a_g S)/g$ seismic coefficient for pseudo-static slope stability verifications
- $0.65\ a_g S$ effective acceleration for checking the liquefaction potential in a saturated sand deposit
- $[(a_g/g)S)]/r$ seismic coefficient for computing the dynamic thrust in the pseudo-static verification of a retaining work.

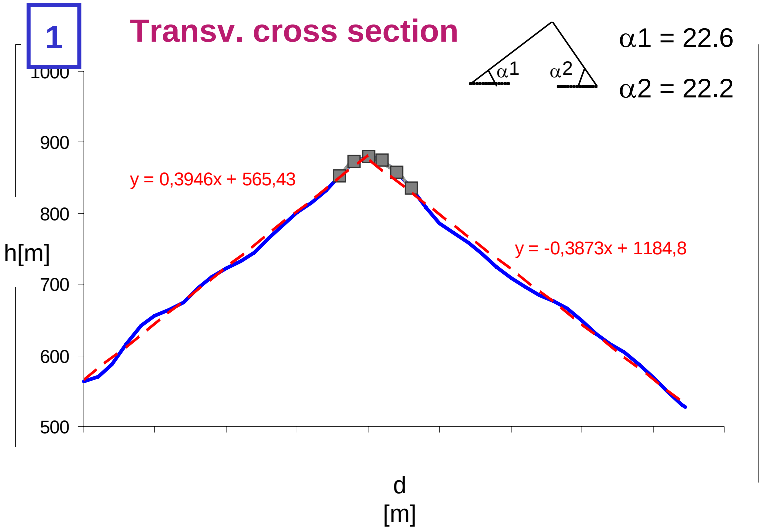
Requirements for siting and foundation soils

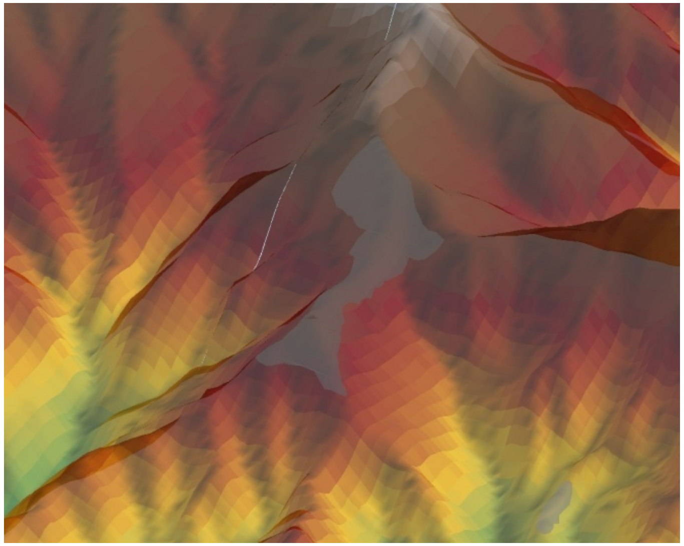
[redacted]

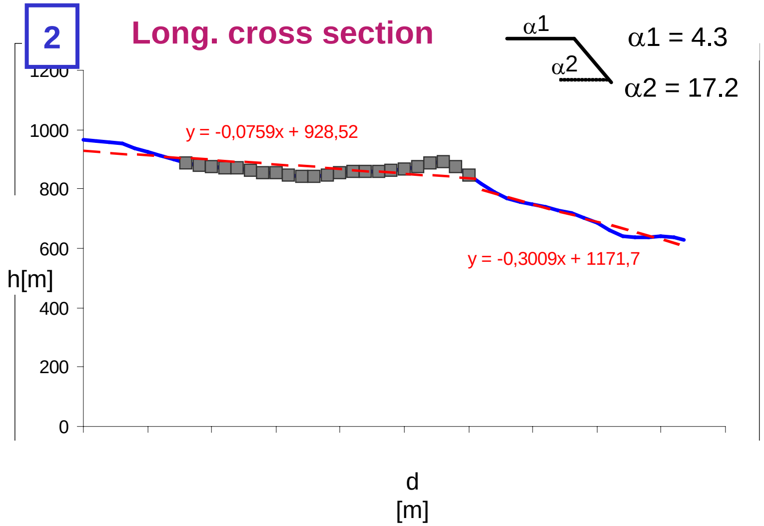
Amplitude response (averaged over frequency) along slopes of different geometry

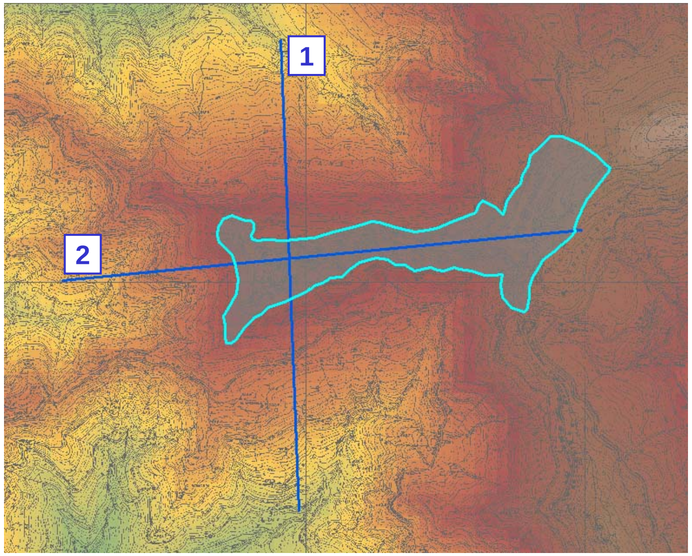


Example of simplified 3D representation of a settlement on a crest: Baiardo









Requirements of the construction site: Proximity to seismically active surface faults

"(1) Buildings of importance classes II, III, IV defined in EN 1998-1:2004, 4.2.5, shall not be erected in the immediate vicinity of tectonic faults recognised as being seismically active in official documents issued by competent national authorities.

(2) An absence of movement in the Late Quaternary may be used to identify non active faults for most structures that are not critical for public safety.

(3) Special geological investigations shall be carried out for urban planning purposes and for important structures to be erected near potentially active faults in areas of high seismicity, in order to determine the ensuing hazard in terms of ground rupture and the severity of ground shaking.."

Izmit, Turkey, M 7.3 earthquake of August 1999
Fault rupture (double stranded) near Golcuk

Izmit, Turkey, M 7.3 earthquake of August 1999
Details of fault rupture

Izmit, Turkey, M 7.3 earthquake of August 1999
Damages to buildings

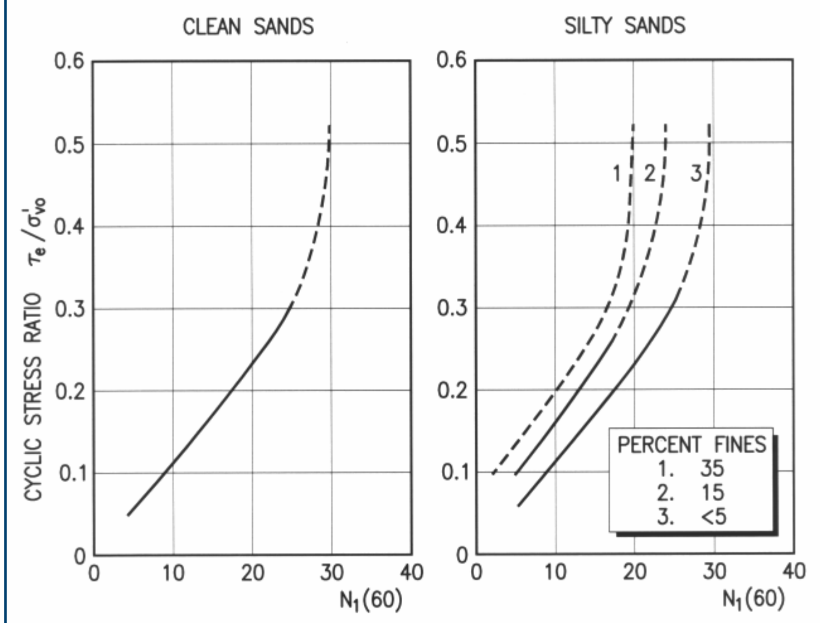
Requirements for siting and foundation soils

POTENTIALLY LIQUEFIABLE SOILS

Detailed guidelines and charts (Annex B) are provided for evaluating the liquefaction susceptibility of saturated cohesionless foundation soils through the well known empirical method based on N_{SPT} or *CPT* resistance.

The guidelines are not unduly conservative, because evaluation of liquefaction susceptibility can be omitted if:

- The sandy soil layer or lens lies at more than 15 m depth from ground surface
- The design ground acceleration is less than 0.15 *g* and, at the same time: N_{SPT} is sufficiently high, or the content of plastic fines in the soil is sufficiently high.



Relationship between stress ratios causing liquefaction and N1 (60) values for clean and silty sands for M=7,5 earthquakes.

A minimum safety factor of 1.25 (in terms of shear stresses) is recommended.

Ground investigations and studies

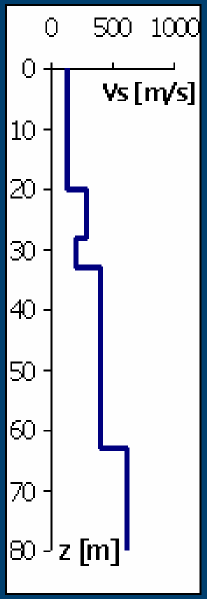
OBJECTIVES: Determine average subsoil profile for selecting design response spectrum and dynamic soil properties

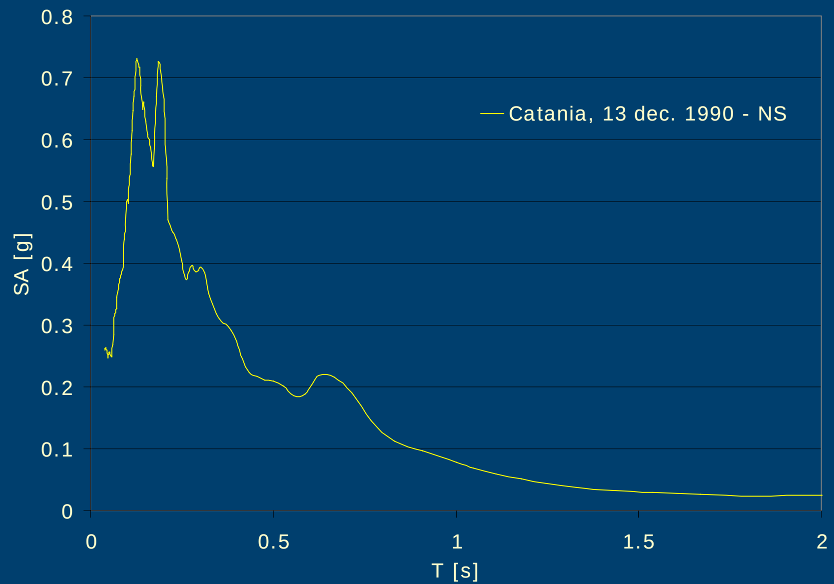
CRITERIA:

- The v_s profile at the site is taken as "the most reliable predictor of the site dependent characteristics of the seismic action at stable sites"
- Estimating the v_s profile by empirical correlations (e. g. with N_{SPT}) is allowed, provided the inherent scatter is taken into account

Vs profile at Catania strong motion station on deep sediments

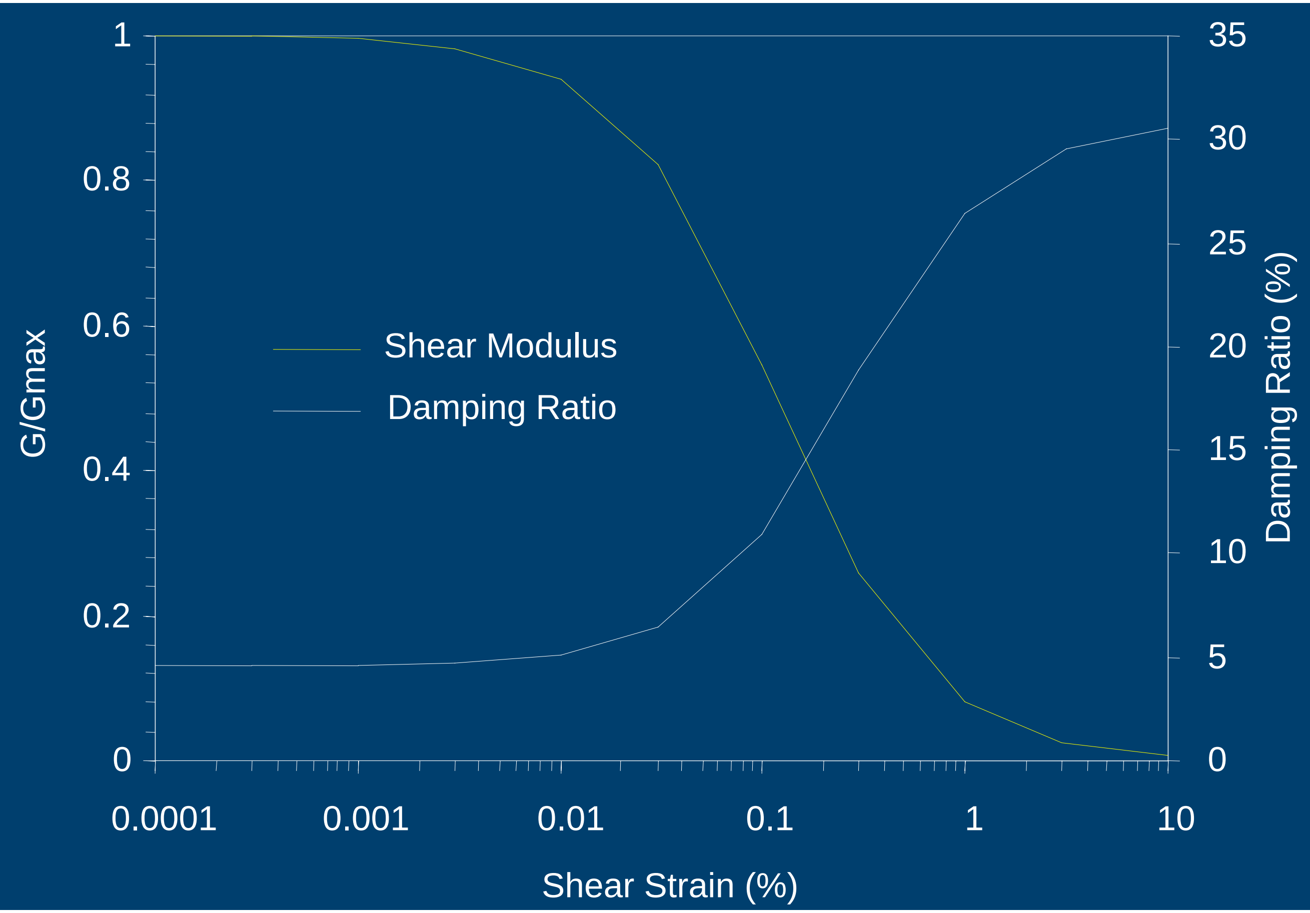
(ground type C/D)





Ground investigations and studies

Indicative reduction factors for v_s or G_{max}, and for internal damping in the soil are provided as a function of shear strain amplitude (through the peak ground acceleration)



Foundation system

BASIC RULE

Only one foundation type to be used for the same structure, unless this consists of dynamically independent units. E. g. use of piles and shallow foundations in the same structure must be avoided, except for bridges and pipelines.

DESIGN ACTION EFFECTS

Action effects transmitted to the foundations are evaluated according to capacity design considerations for *dissipative structures* (high ductility), while allowable seismic action combination applies for *non-dissipative structures* (essentially responding in the elastic range).

DIRECT FOUNDATIONS (Footings)

Stability of footings is to be checked against sliding failure, i. e.

$$V_{sd} \leq N_{sd} \tan \delta + E_{pd}$$

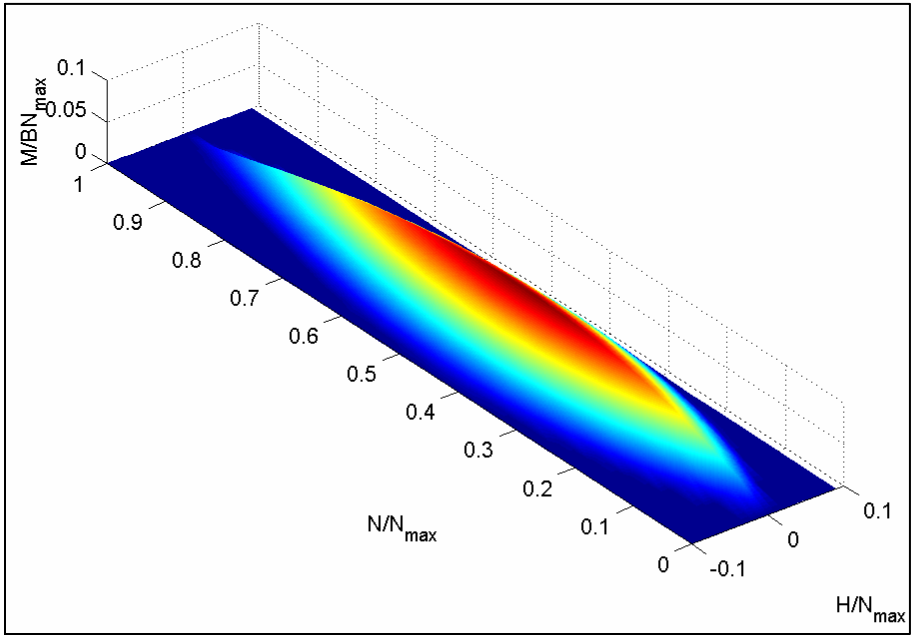
shear force | friction resistance | lateral resistance

and against bearing capacity failure (Annex F, see following two slides). Factors to be taken into account include: Inclination and eccentricity of structural loiad, inertia forces in the soil, pore pressure effects, non-linear soil behaviour.

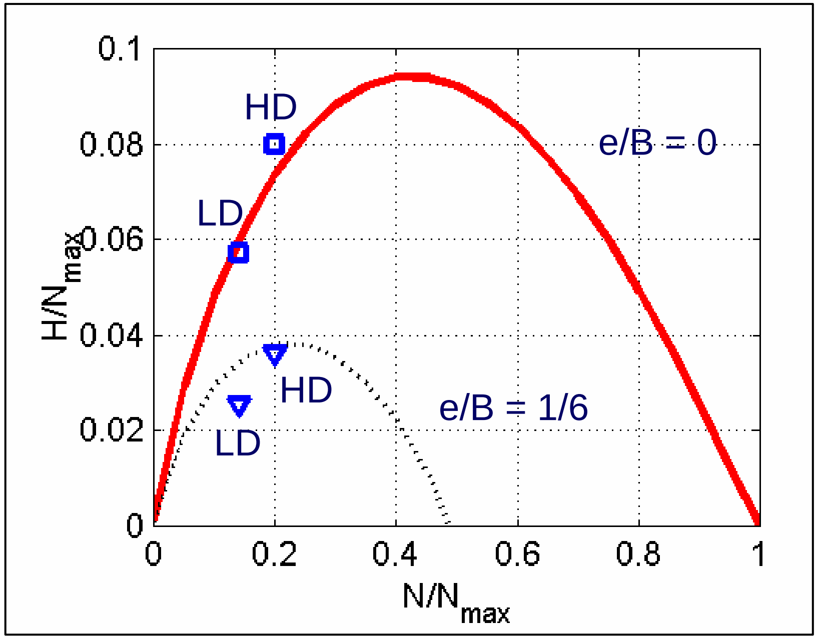
Foundation system

Bounding surface of external loads

A single expression has been obtained for both cohesive and granular soils; it is introduced in Annex F of Part 5



3D view of bounding surface



Cross-sections for vanishing F and different load eccentricities

Foundation system

Piles and piers

Should be designed to resist both:

(a) *Inertia forces* from the superstructure, and

(b) *Kinematic forces* due to the eartquake-induced soil deformations.

The latter apply if all of the following conditions occur:

b1. Class D, S1, or S2 soil profile with consecutive layers of sharply contrasting stiffness

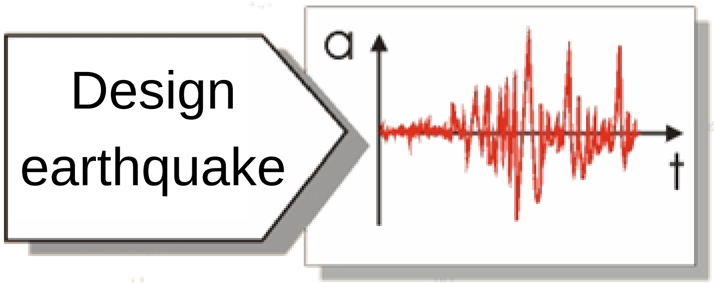
b2. Design ground acceleration > 0.10 g, and

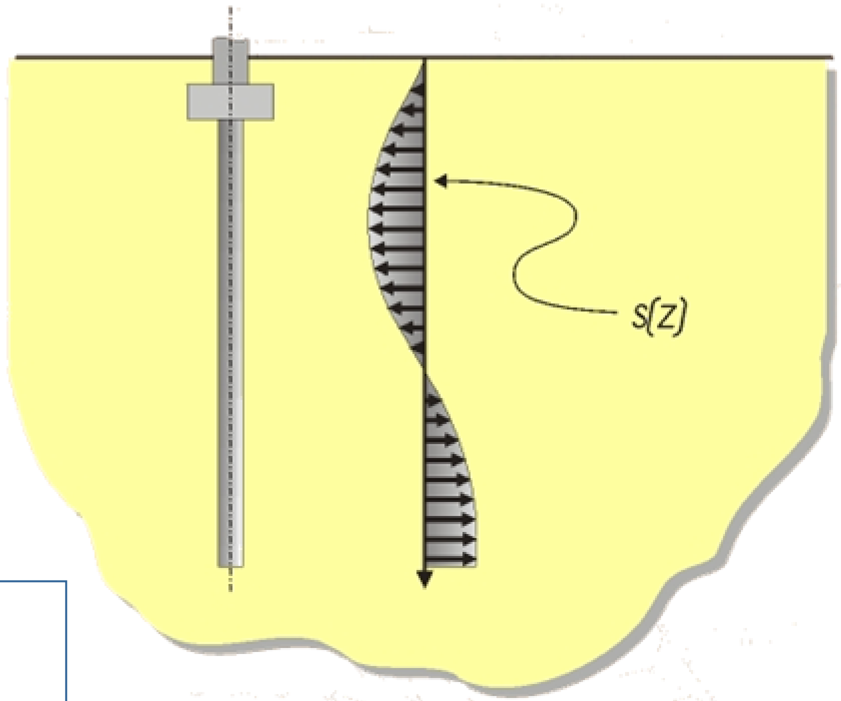
b3. The supported structure is of importance category III or IV.

Although piles will generally be designed to remain elastic, they may under certain conditions develop plastic hinges at their head.

Kinematic forces on piles

Kinematic forces induced by earthquake on piles in “stable” soils

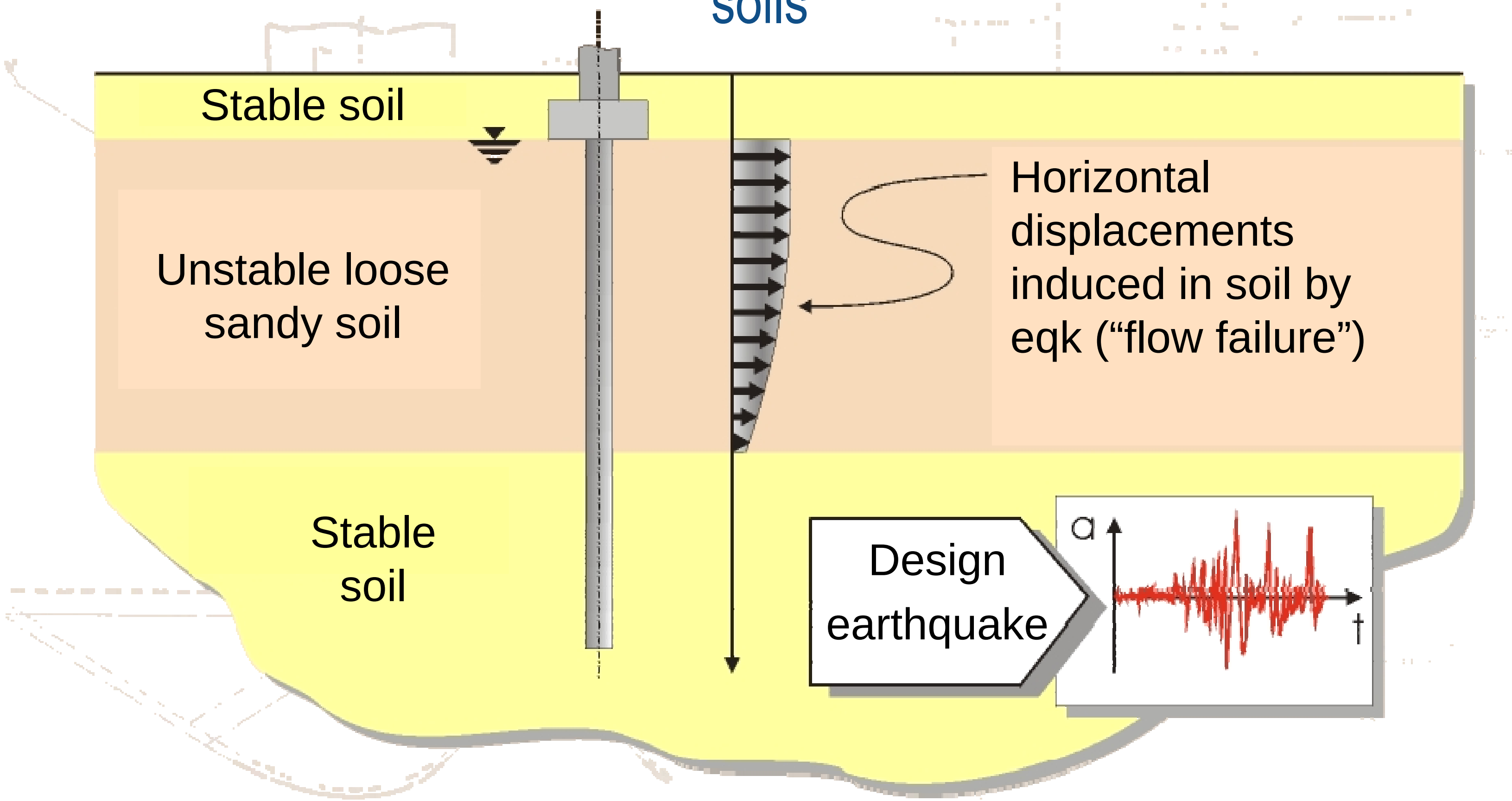




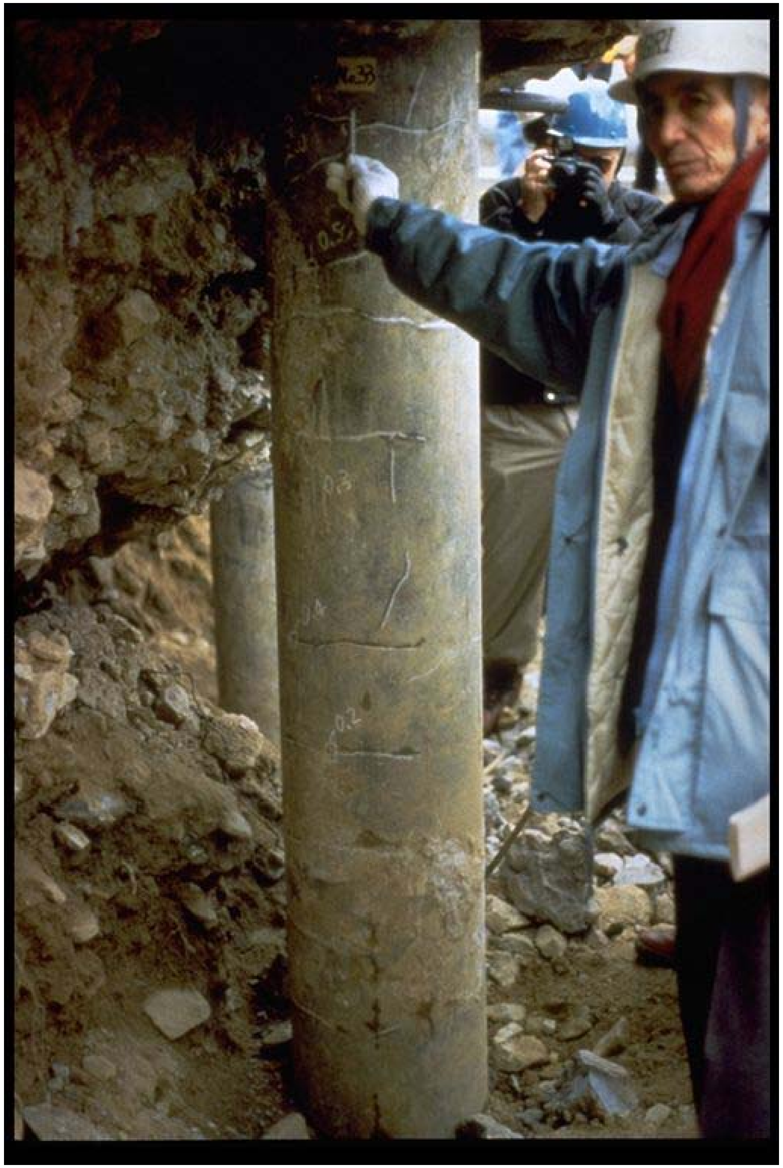
s(z): seismically induced horizontal displacements in the soil

Kinematic forces on piles

Kinematic forces induced by earthquake on piles in “unstable” loose soils



Effects of kinematic forces on piles



(NISEE slide collection)

Earth retaining structures

General requirements and considerations

- Permanent displacements/tilting may be acceptable, provided functional or aesthetic requirements are not violated
- Structural choice is based on static loads , but seismic action may lead to different solution
- Build-up of significant PWP in backfill or supported soil is to be absolutely avoided
- Methods of analysis should in principle account for:
    - inertial and interaction effects between structure and soil (even non-linear)
    - hydrodynamic effects in the presence of water (in the soil, and on outer face of wall)
    - compatibility of deformations of soil, wall, and free tendons)

NB: After introducing requirements for general methods of analysis, the code only provides prescriptions for pseudo-static verifications

Experimental observations (flexible retaining walls)

1) Effects on retaining walls in historical earthquakes

- Liquifiable soils (harbour facilities) → Collapse (0.6 – 4m displacements)

  Loma Prieta (California, 1989) – M_W=7.1

  Kobe (Giappone, 1995) – M_W=6.9

  Bhuj (India, 2001) – M_W=7.6

- Non-liquifiable soils → Good behaviour (<10cm displacements)

  Northridge (California, 1994) – M_W=6.8

  Kobe (Giappone, 1995) – M_W=6.9

  Taiwan (1999) – M_W=7.6

2) Dynamic experimental tests

- Shaking table
- Dynamic centrifuge tests

Earth retaining structures

Simplified (pseudo-static) analysis: the seismic action can be reduced by kind of ductility factor r:

$$k_h = a_g \gamma_I S/r$$

$k_v = \pm 0{,}5\, k_h$ (Spectrum Type 1) $k_v = \pm 0{,}33\, k_h$ (Spectrum Type 2)

Values of reduction factor *r* and residual displacement

Type of retaining structure	*r*	**Acceptable residual displacement,** d_r (mm)
Free gravity walls that can accept a displacement	**2**	**$300 a_g \gamma_I S / g$**
As above, but less"tolerant"	**1,5**	**$200 a_g \gamma_I S / g$**
Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	**1**	**0**

For loose, saturated granular soils, r = 1 and FS against liquefaction not less than 2.

The provisions for pseudo-static analysis follow a standard approach (Mononobe and Okabe), given in Annex E.

Earth retaining structures

Resistance and stability verifications

- **Foundation soil** The following need be verified:
  - Stability of slope
  - Stability w. r. to failure by sliding and loss of bearing capacity, for shallow foundation.

    Design actions: permanent gravity loads, horizontal thrust E_d , seismic action.
- **Anchorages** They shall assure equilibrium and have a sufficient capacity to adapt to the seismic deformations of the ground . The distance L_e between the anchor and the wall shall exceed the distance L_s, required for non-seismic loads :

$$L_e = L_s\,(1 + 1.5\,S\,a_g)$$

- **Backfill material** must be immune from liquefaction.
- *Structural strength* under the combination of the seismic action with other possible loads, equilibrium must be achieved without exceeding the strength of any structural element:

$$R_d > S_d$$

R_d : design resistance of the element, evaluated as for the non seismic situation

S_d : design value of the action effect, as obtained from the analysis.

Examples of numerical analyses: simple flexible wall

(from M. Eng. Thesis of O. Zanoli, Politecnico di Milano, Dec. 2007)

Soil profile properties

Ground categories B, C, D: governed by $V_{s,30}$

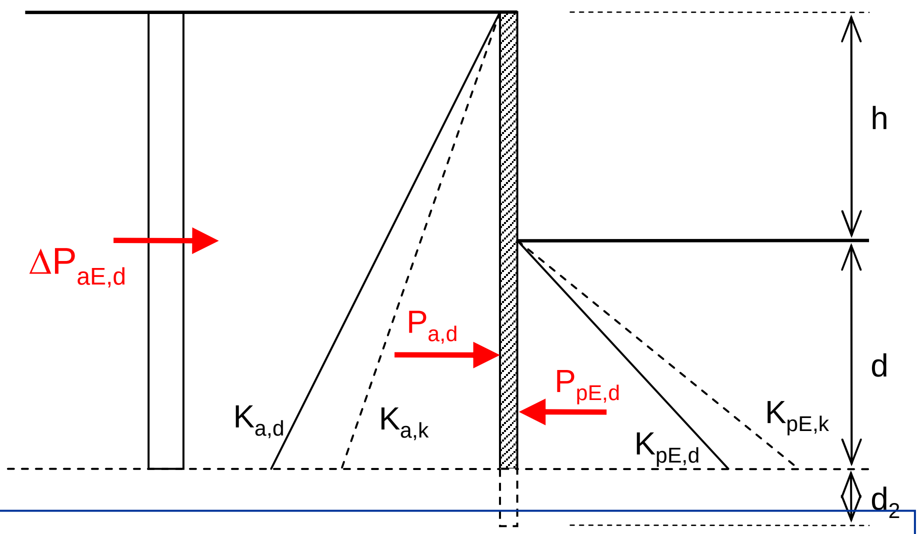
Coarse grained, dry, γ_S=20kN/m^3, Φ=34°,

c'=0, ψ=0°, δ_A=0°, δ_P= Φ

RC flexible wall

Unit weight γ_{RC}=25kN/m^3, ν_{RC}=0.3, E_{RC}=28GPa

Excavation depth h=3, 5, 7m + Embedment (d)



<u>Pre-dimensioning (EC 8)</u>

1) Design strength : $\tan \varphi_d = \tan \varphi / 1.25$

2) Seismic coefficients

$$\begin{cases} k_v = 0 \\ k_h = \dfrac{1}{r} S \dfrac{a_{gR}}{g} \end{cases}$$

3) Thrusts

$$\begin{cases} P_{AE,d} = P_{A,d} + \Delta P_{AE,d} = \dfrac{1}{2}\gamma(h+d)^2 K_A + \gamma(h+d)^2 \Delta K_{AE} \\ P_{PE,d} = \dfrac{1}{2}\gamma d^2 K_{PE} \end{cases}$$

4) Rotational equilibrium w. r. to base

5) 20% increase in embedment d

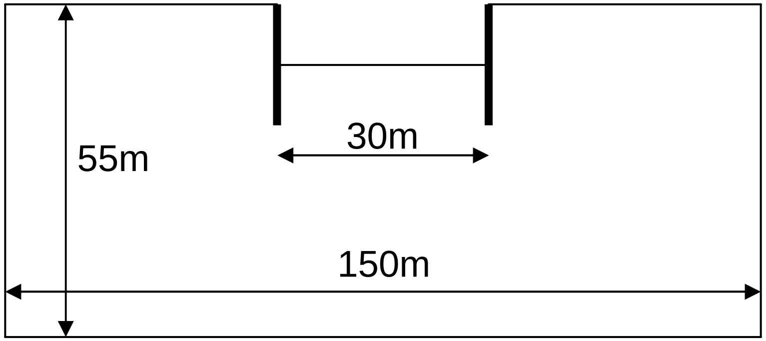
	h3d5	h5d8	h7d11
Excavation d. [m]	3	5	7
Embedment [m]	5	8	11
H total [m]	8	13	18

Examples of numerical analyses: simple flexible wall (from M. Eng. Thesis of O. Zanoli, Politecnico di Milano, Dec. 2007)

Material models

Soil: coarse grained, homogeneous, V_s=156m/s, elastic-plastic non associated constitutive model, Mohr-Coulomb rupture criterion(f=32°).

Flexible wall: 13m height, 5m excavation, linear elastic behaviour, E=28 GPa, ν=0.3.



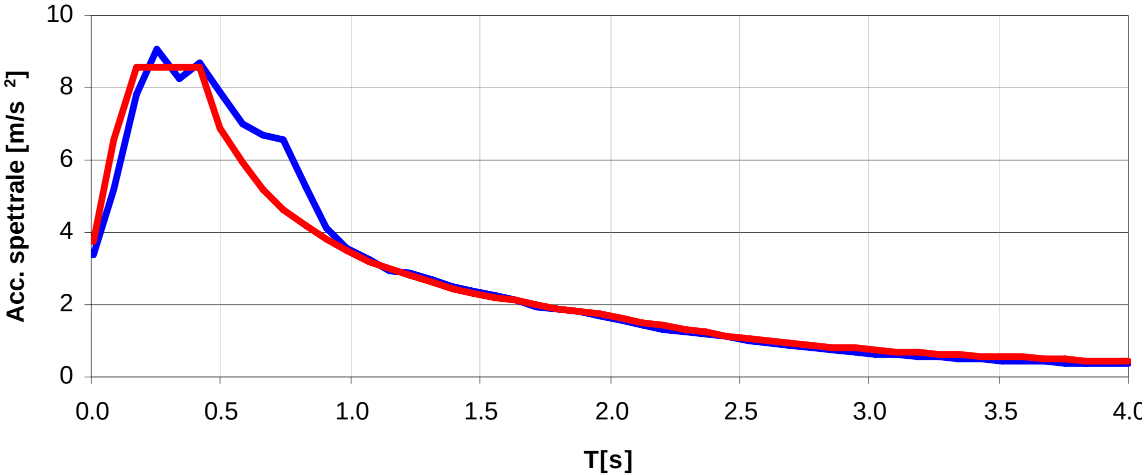
Base excitation

2 groups of 7 accelerograms on ground type A

Zone I
(a_{max}=0.35g)

Zone II
(a_{max}=0.25g)

Average response spectra matching EC8 elastic spectrum



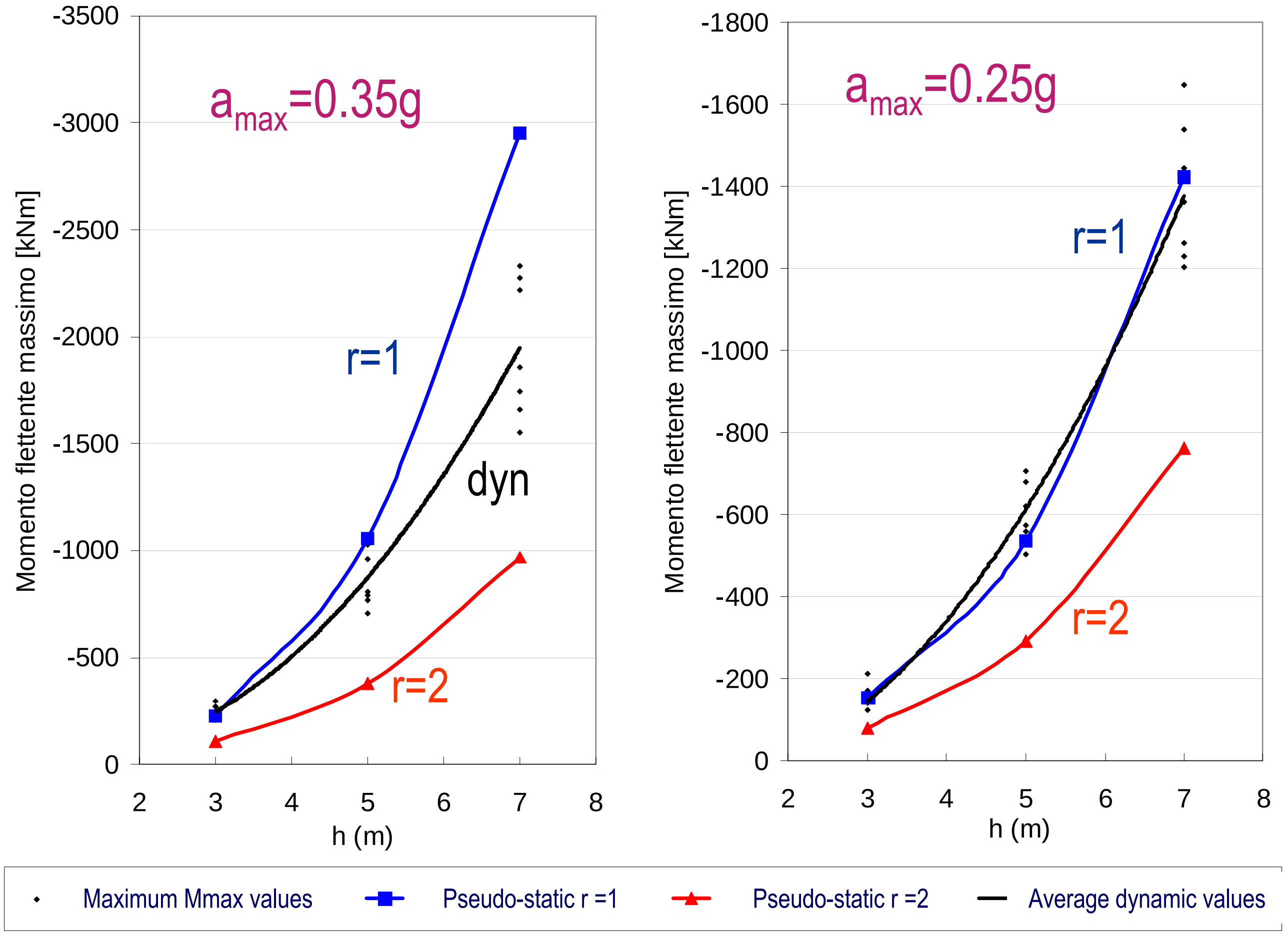
Results of EC8 pseudo-static vs. 2D dynamic (FEM) analyses
(from the M. Eng. Thesis of O. Zanoli, Politecnico di Milano)

Bending moment profiles in retaining wall

- **Eurocode 8:** pseudo-static analyses with assigned k_h values for r = 1, 2;
- **Dynamic analyses:** average M_{max} values over groups of input accelerograms

Ground type B

(Both pseudo-static and dynamic FEM analyses performed with commercial software FLAC)



Results of EC8 pseudo-static vs. 2D dynamic (FEM) analyses

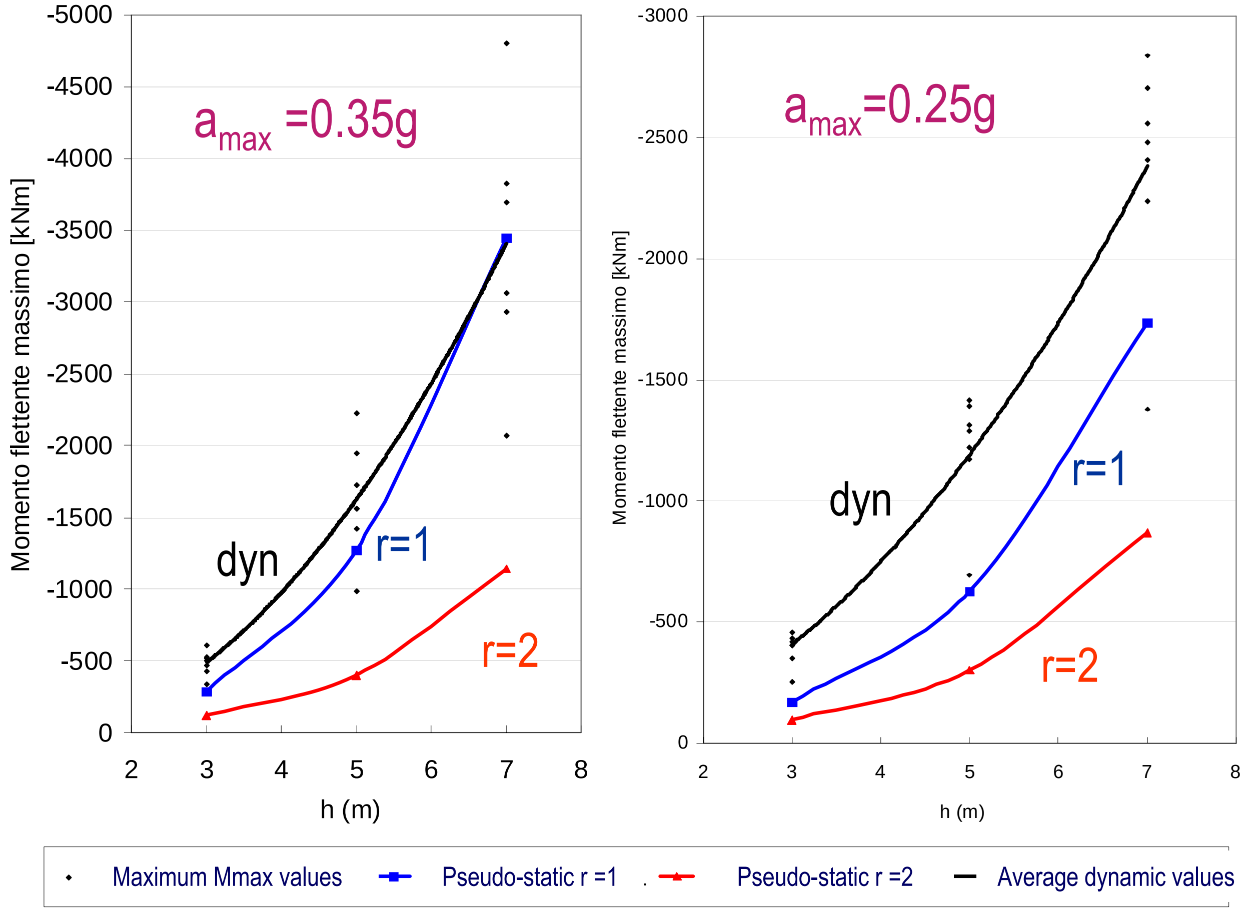
(from the M. Eng. Thesis of O. Zanoli, Politecnico di Milano)

Bending moment profiles in retaining wall

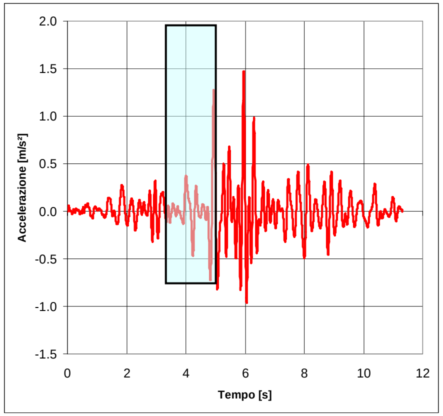
- **Eurocode 8: pseudo-static analyses with assigned k_h values for r=1, 2;**
- **Dynamic analyses: average M_{max} values over groups of input accelerograms**

Ground type D

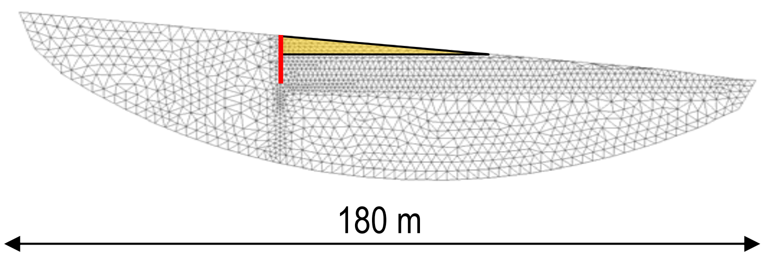
(Both pseudo-static and dynamic FEM analyses performed with commmercial software FLAC)



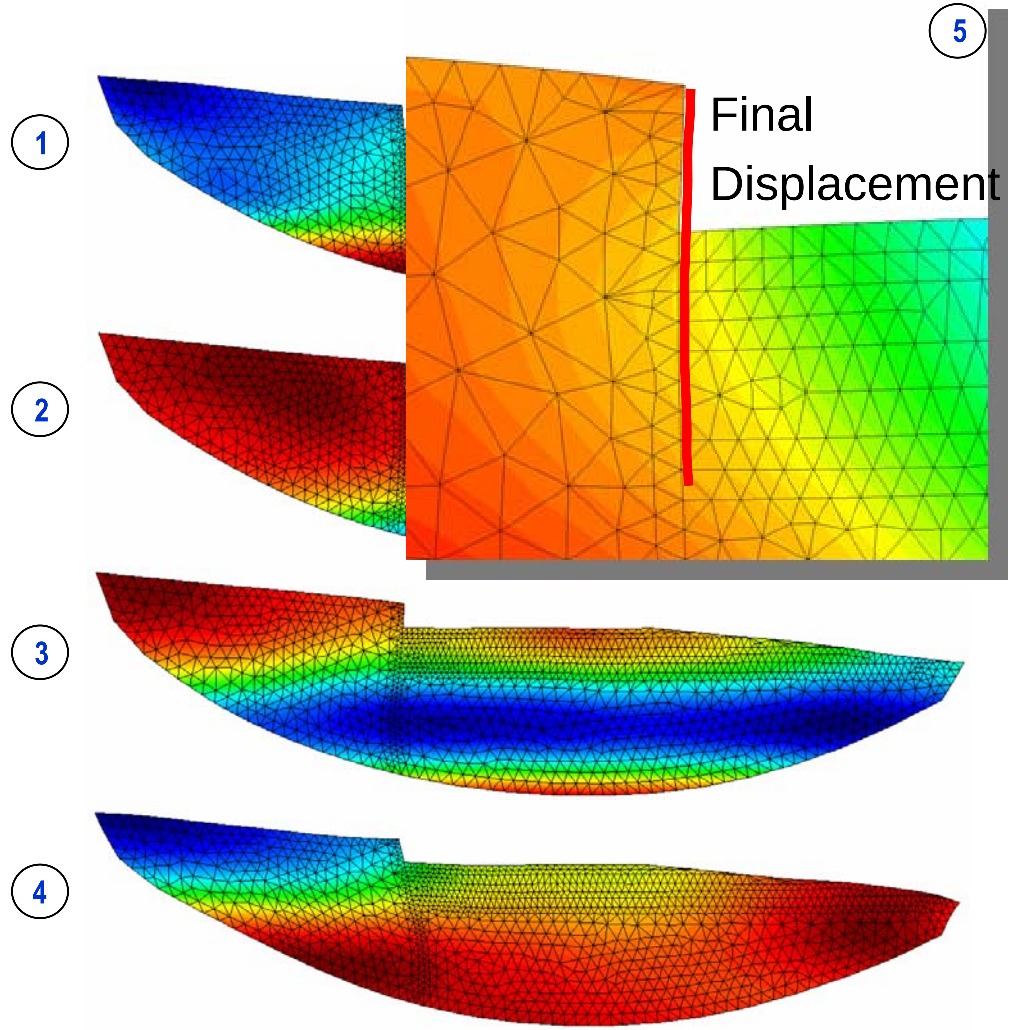
Examples of numerical analyses
Complexity of wave propagation phenomena in soil – flexible wall systems
(other example with a_{max} = 0.15g)



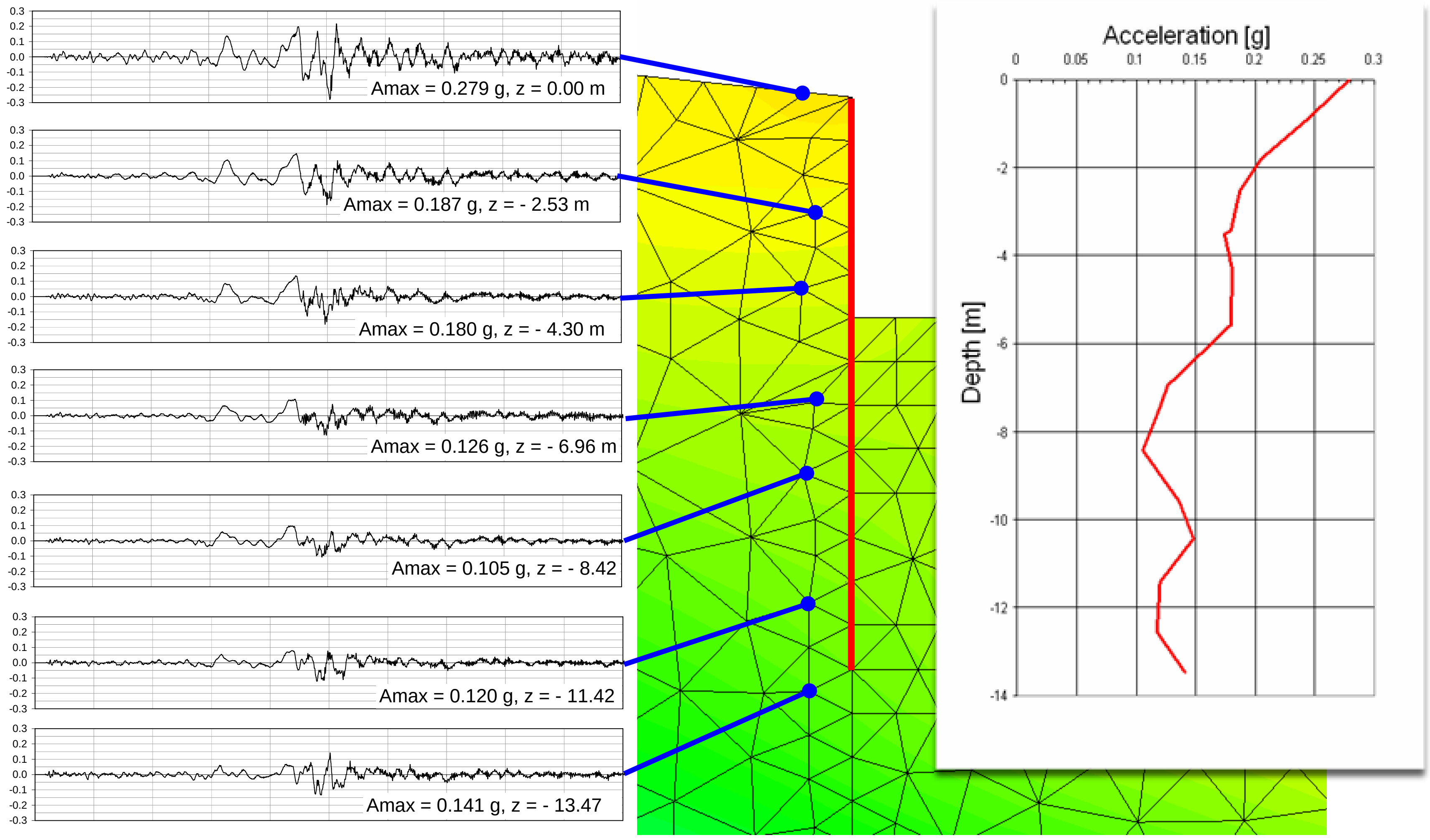
Nonlinear dynamic analyses performed with FEM code Gefdyn



Snapshots of displacement field at different instants



Examples of numerical analyses
Acceleration profile showing amplification of soil-wall seismic motion from depth to surface (example with a_{max} = = 0.23g)



Suggested reference:

Designers' Guide to EN 1998-1 and EN 1998-5

Eurocode 8: Design of structures for earthquake resistance. General rules, seismic actions, design rules for buildings, foundations and retaining structures

Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioli, Paolo Pinto and Andre Plumier.

Published by Thomas Telford, UK, 2006