Design of concrete foundation elements

Alain Pecker
Géodynamique & Structure

OUTLINE OF PRESENTATION

Object and salient features of EN 1998-5

Relations with EN 1997

Ground properties (strength, stiffness, material factors)

Requirements for construction site

Earth retaining structures

Foundation system: shallow and deep foundations

SALIENT FEATURES AND INNOVATIVE ASPECTS

Complementarity with Eurocode 7 (EN 1997)
which does not cover earthquake resistant design.

Introduction and use of dynamic soil properties:

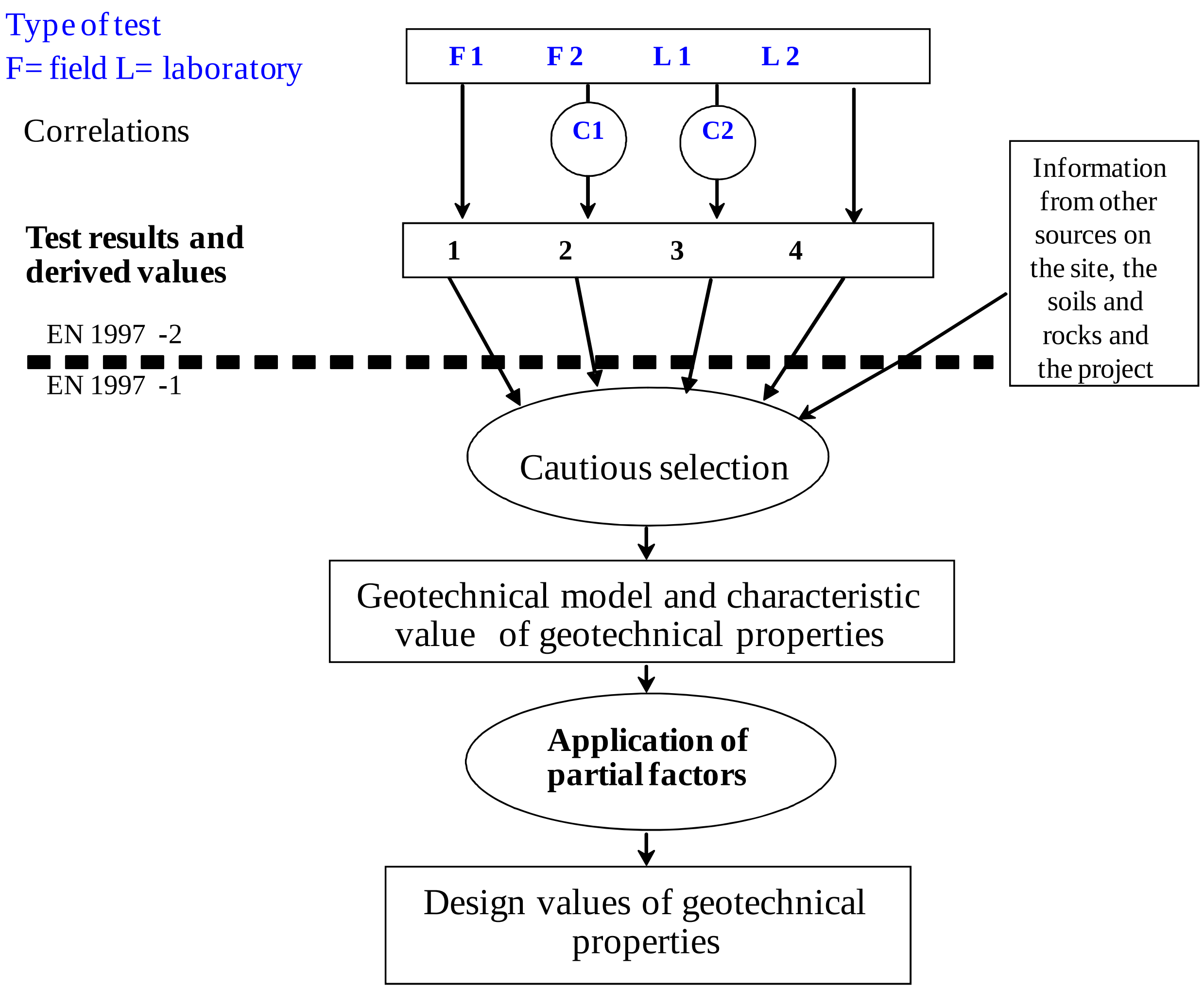
(τ_{cyc}, shear wave velocity V_s and damping) in addition to standard static properties (tan ϕ', c_u, q_u)

Different approaches to safety and strength verifications

depending on seismicity level and type of soil

Recognition of seismically-induced permanent ground deformations as a design criterion.

DETERMINATION OF DESIGN VALUES



Three possible design approaches

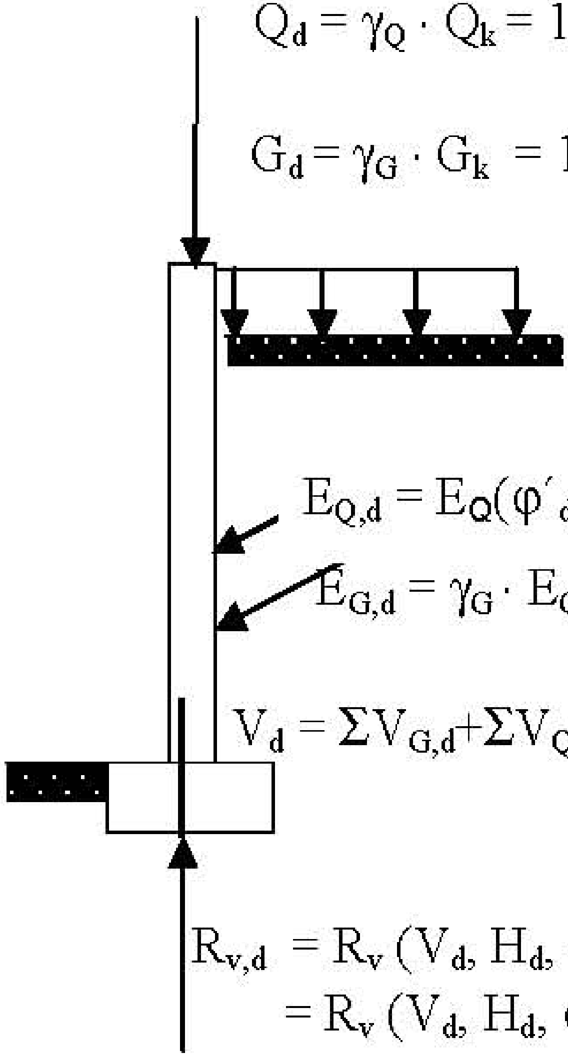
DA-1 C1 : partial factors on actions

DA-1 C2 : partial factors on ground strength parameters

DA2 : partial factors on actions (or action effects) and on global resistance

EN 1998-5

DESIGN APPROACH DA-1 C1



$$Q_d = \gamma_Q \cdot Q_k = 1.50 \cdot Q_k$$

$$G_d = \gamma_G \cdot G_k = 1.35 \cdot G_k$$

$$q_d = \gamma_Q \cdot q_k = 1.50 \cdot q_k$$

$$\gamma_{\varphi'} = \gamma_c = 1.0$$

$$\varphi'_d = \varphi'_k,\ c'_d = c'_k$$

$$E_{Q,d} = E_Q(\varphi'_d,\ c'_d,\ q_d)$$

$$E_{G,d} = \gamma_G \cdot E_G(\varphi'_d,\ c'_d) = 1.35 \cdot E_G(\varphi'_k,\ c'_k)$$

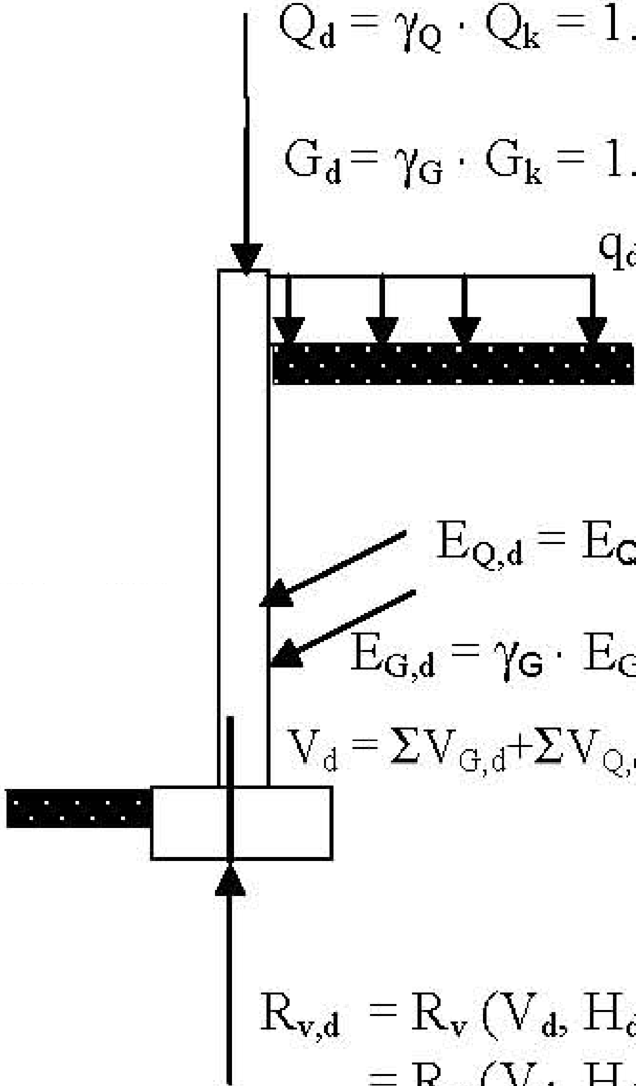
$$V_d = \Sigma V_{G,d} + \Sigma V_{Q,d}$$

$$R_{v,d} = R_v\,(V_d,\ H_d,\ \varphi'_d,\ c'_d)/\gamma_{Rv}$$

$$= R_v\,(V_d,\ H_d,\ \varphi'_k,\ c'_k)/1.0$$

(R. Frank, 2008)

DESIGN APPROACH DA-1 C2



$Q_d = \gamma_Q \cdot Q_k = 1.30 \cdot Q_k$

$G_d = \gamma_G \cdot G_k = 1.00 \cdot G_k$

$q_d = \gamma_Q \cdot q_k = 1.30 \cdot q_k$

$\tan\varphi'_d = \tan\varphi'_k/\gamma_{\varphi'} = \tan\varphi'_k/1.25$

$c'_d = c'_k/\gamma_c = c'_k/1.25$

$E_{Q,d} = E_Q(\varphi'_d, c'_d, q_d)$

$E_{G,d} = \gamma_G \cdot E_G(\varphi'_d, c'_d) = 1.00 \cdot E_G(\varphi'_d, c'_d)$

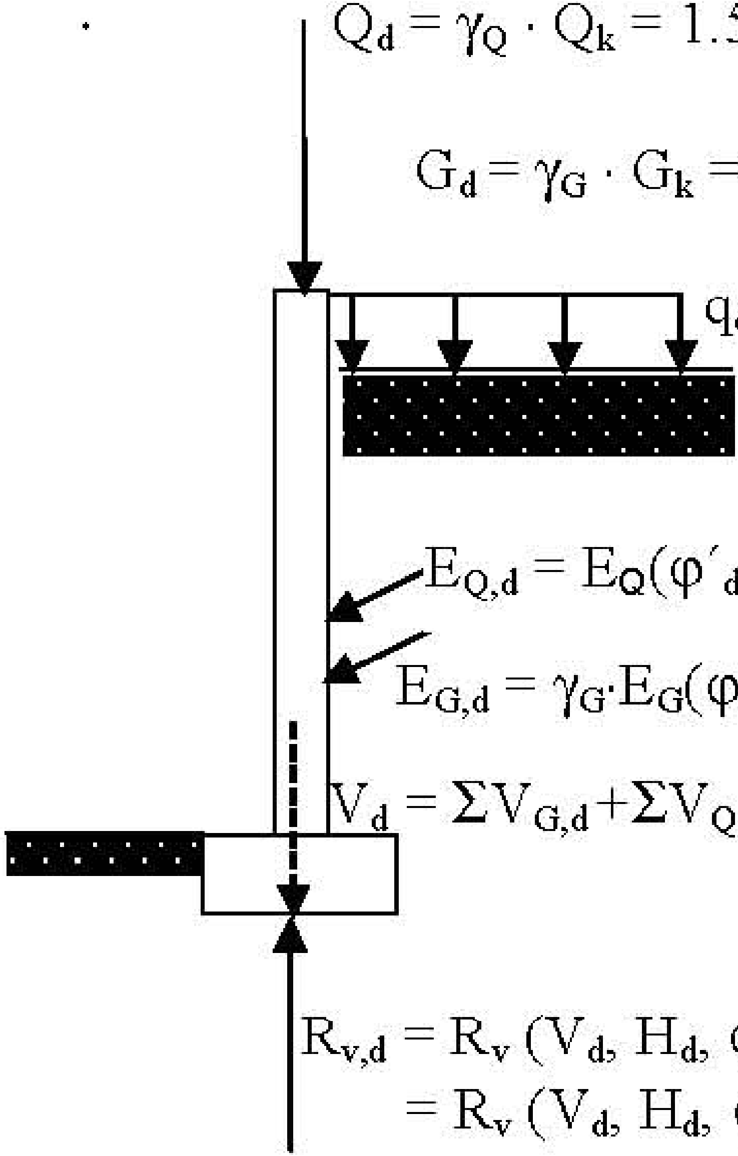
$V_d = \Sigma V_{G,d} + \Sigma V_{Q,d}$

$R_{v,d} = R_v(V_d, H_d, \varphi'_d, c'_d)/\gamma_{Rv}$

$= R_v(V_d, H_d, \varphi'_d, c'_d)/1.0$

(R. Frank, 2008)

DESIGN APPROACH DA-2



$$Q_d = \gamma_Q \cdot Q_k = 1.50 \cdot Q_k$$

$$G_d = \gamma_G \cdot G_k = 1.35 \cdot G_k$$

$$q_d = \gamma_Q \cdot q_k = 1.50 \cdot q_k$$

$$\gamma_{\varphi'} = \gamma_c = 1.00$$
$$\varphi'_d = \varphi'_k, \; c'_d = c'_k$$

$$E_{Q,d} = E_Q(\varphi'_d, c'_d, q_d)$$

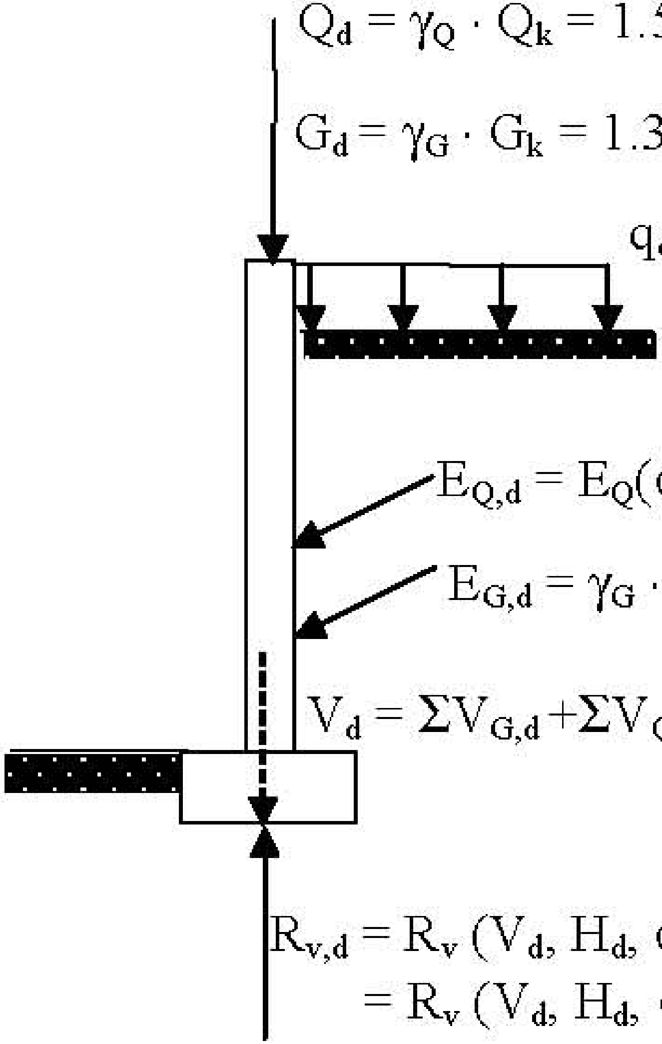
$$E_{G,d} = \gamma_G \cdot E_G(\varphi'_d, c'_d) = 1.35 \cdot E_G(\varphi'_k, c'_k)$$

$$V_d = \Sigma V_{G,d} + \Sigma V_{Q,d}$$

$$R_{v,d} = R_v(V_d, H_d, \varphi'_d, c'_d)/\gamma_{Rv}$$
$$= R_v(V_d, H_d, \varphi'_k, c'_k)/1.40$$

(R. Frank, 2008)

DESIGN APPROACH DA-3



$$Q_d = \gamma_Q \cdot Q_k = 1.50 \cdot Q_k$$

$$G_d = \gamma_G \cdot G_k = 1.35 \cdot G_k$$

$$q_d = \gamma_Q \cdot q_k = 1.30 \cdot q_k$$

$$\tan \varphi'_d = \tan\varphi'_k/\gamma_{\varphi'} = \tan\varphi'_k/1.25$$
$$c'_d = c'_k/\gamma_c = c'_k / 1.25$$

$$E_{Q,d} = E_Q(\varphi'_d, c'_d, q_d)$$

$$E_{G,d} = \gamma_G \cdot E_G(\varphi'_d, c'_d) = 1.00 \cdot E_G(\varphi'_d, c'_d)$$

$$V_d = \Sigma V_{G,d} + \Sigma V_{Q,d}$$

$$R_{v,d} = R_v (V_d, H_d, \varphi'_d, c'_d)/ \gamma_{Rv} = R_v (V_d, H_d, \varphi'_d, c'_d)/1.0$$

(R. Frank, 2008)

Characteristic values of geotechnical parameters

cautious estimate of value affecting occurrence of limit state

mean range of values covering a large volume

characteristic values cannot be fundamentally different from traditional values

if statistical methods are used : probability of exceedance of worse values < 5%

Design value

$$X_d = \frac{X_k}{\gamma_M}$$

⟵ NDP

SOIL CHARACTERISTICS

- **Strength parameters**
  - Static parameters may be used
  - Clay C_u with corrections for: cyclic degradation
    rate of loading

$\gamma_M = 1.4$

  - Sand *C'* , ϕ' or cyclic undrained shear strength for saturated sands τ_{cy}

$\gamma_{MC'} = 1.4 \qquad \gamma_{M\phi} = 1.25 \qquad \gamma_{M\tau cy} = 1.25$

SOIL CHARACTERISTICS

- **Stiffness and damping parameters**

Used for site classification
Strain dependent

Ground acceleration ratio $\alpha \cdot S$	Damping ratio	$\frac{v_s}{v_{s,max}}$	$\frac{G}{G_{max}}$
0,10	0,03	0,90(±0,07)	0,80(±0,10)
0,20	0,06	0,70(±0,15)	0,50(±0,20)
0,30	0,10	0,60(±0,15)	0,36(±0,20)

Valid for V_{Smax} < 360 m/s

REQUIREMENTS FOR CONSTRUCTION SITE

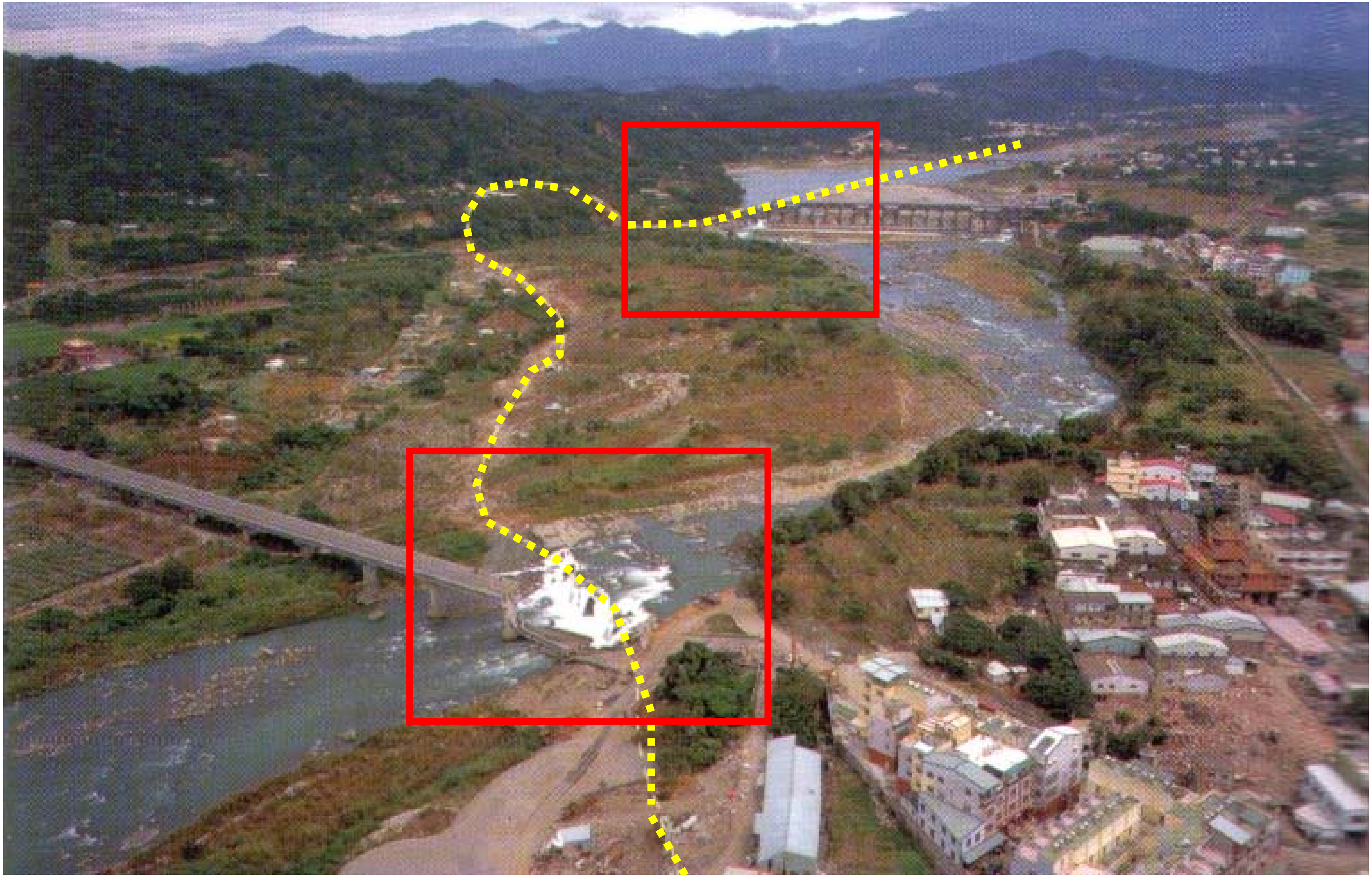
Buildings of importance classes II, III, IV shall not be erected in the immediate vicinity of seismically active tectonic faults

official documents issued by competent national authorities

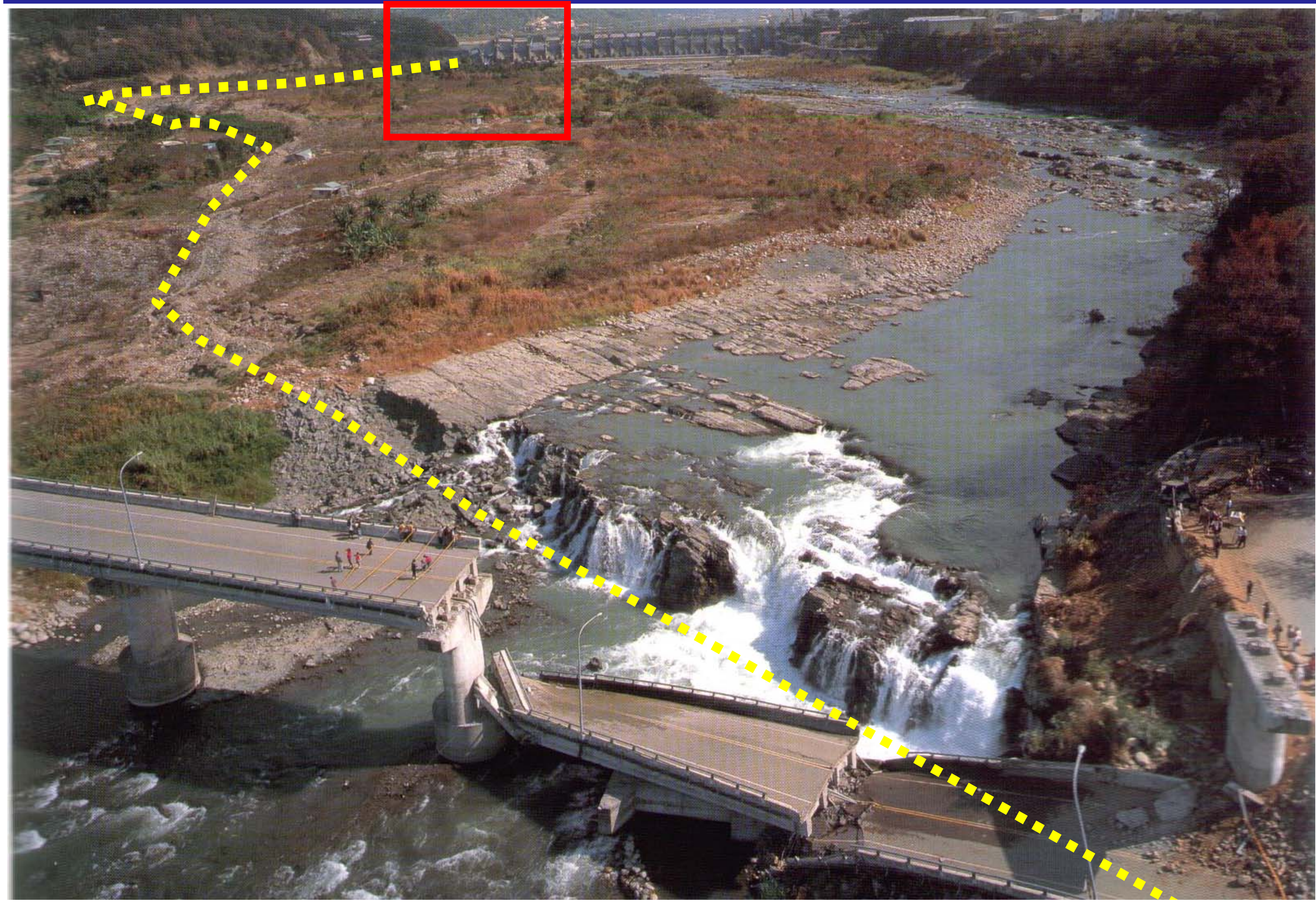
absence of movement in the Late Quaternary

Special geological investigations shall be carried out for urban planning purposes and for important

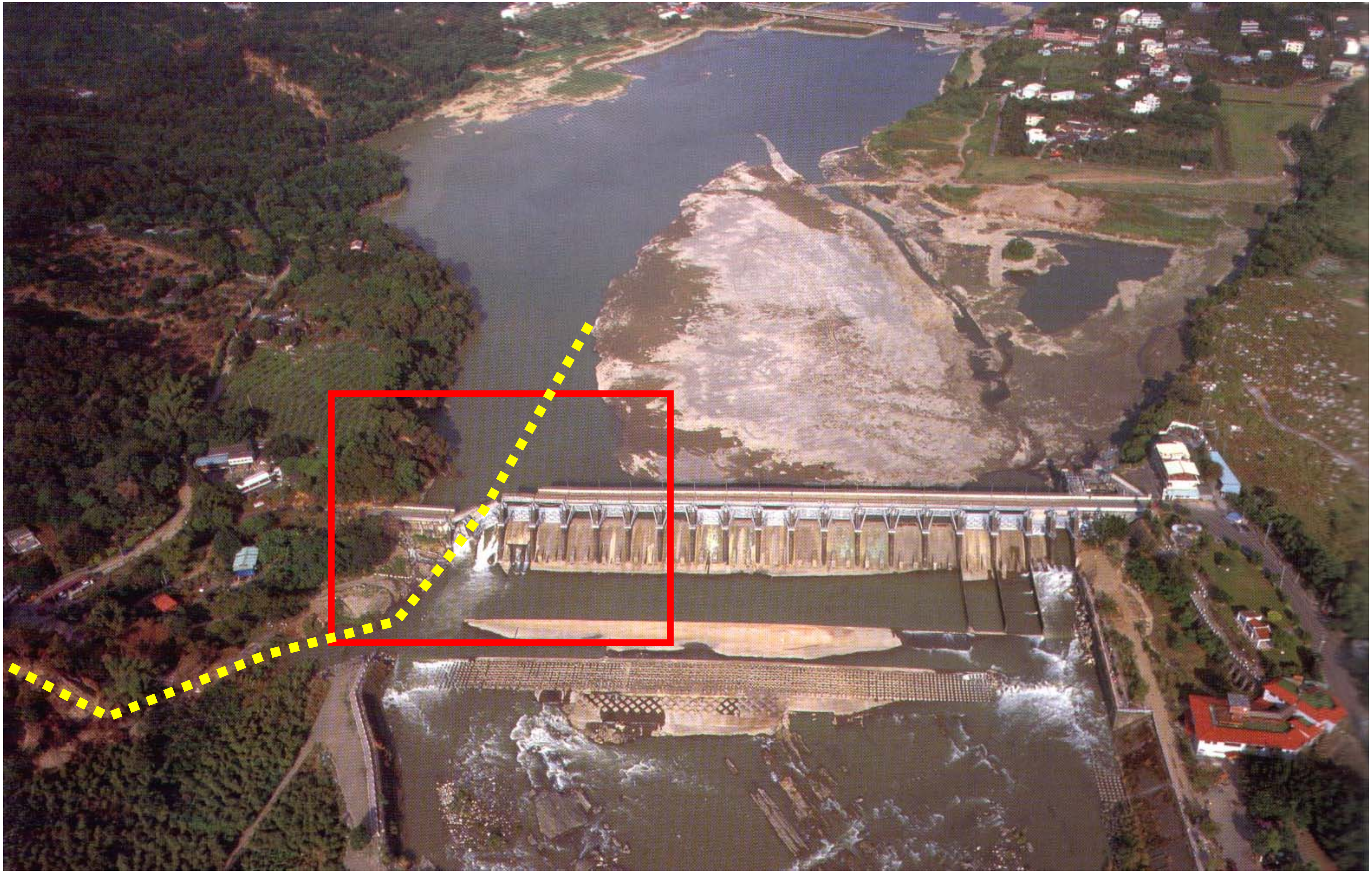
Chi-Chi, Taiwan 1999 : FAILURE OF BRIDGE AND DAM



CHIEN-MIN BRIDGE



SHIH-KANG DAM



SHIH-KANG DAM

REQUIREMENTS FOR SITING AND FOUNDATION SOILS : LIQUEFACTION

Verification carried out in free field conditions

Conditions prevailing during life time of building

Seismic demand : Seed – Idriss method (1971)

Liquefaction resistance from field tests

SPT (normative annex), CPT or V_S
with detailed corrections for overburden and energy

Required safety factor FS = 1.25 (NDP)

REQUIREMENTS FOR SITING AND FOUNDATION SOILS : LIQUEFACTION

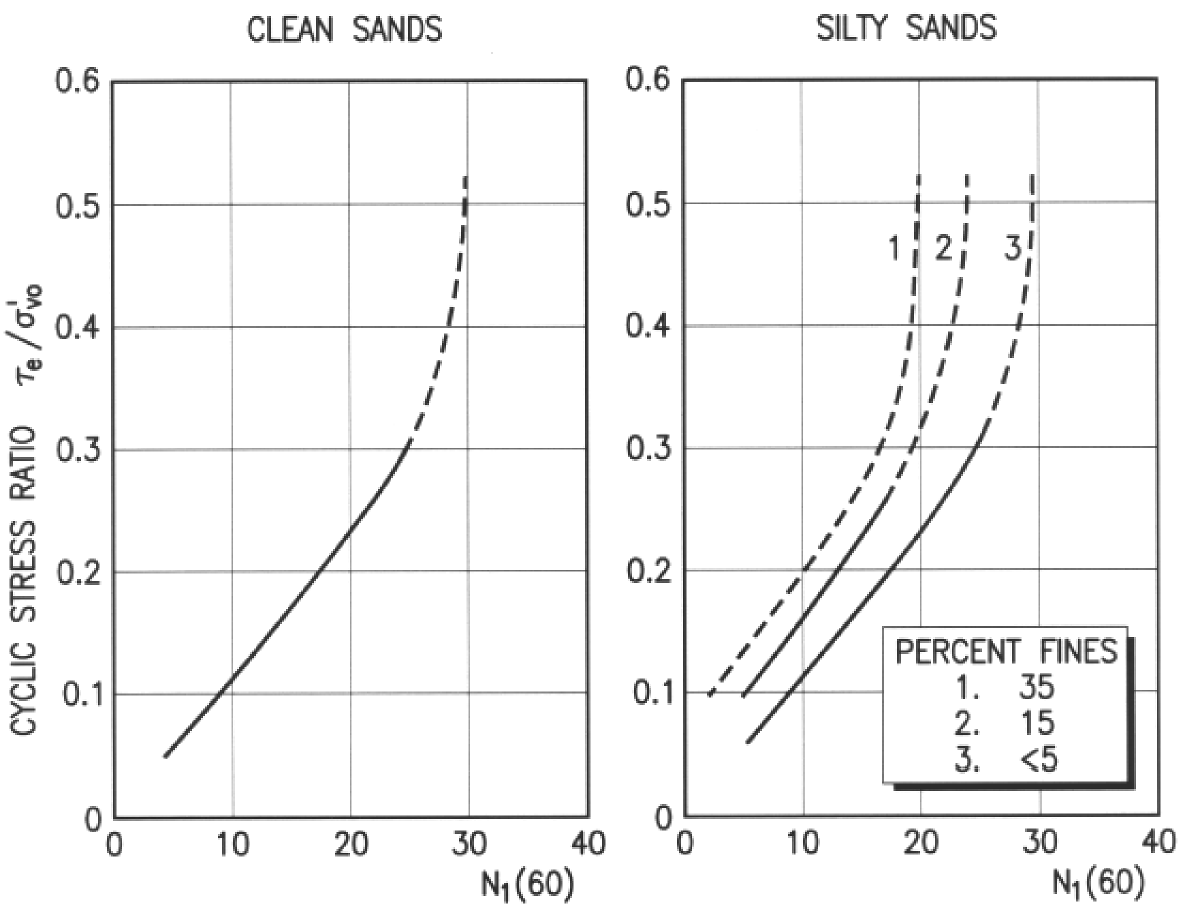
No verification requirements if

- The sandy layers are deeper than 15m
- $(a_g /g)S < 0.15$

AND either

% clay > 20% and PI > 10%
% silt > 35% and $N_1 > 20$
Clean sand and $N_1 > 30$

LIQUEFACTION CHARTS (Annex B – normative)



Charts valid for M_w=7.5 – Corrections provided for other M_w

EXAMPLES OF LIQUEFACTION DAMAGES

Port Island 1995

EXAMPLES OF LIQUEFACTION DAMAGES

REQUIREMENTS FOR SITING AND FOUNDATION SOILS : SLOPE STABILITY

The ultimate limit state (ULS) or damage limit state (DLS) is related to unacceptable large displacements

Analysis is required for all structures (except cat. I) in vicinity of a slope

Topographic amplification shall be taken into account

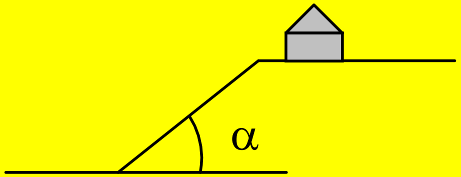
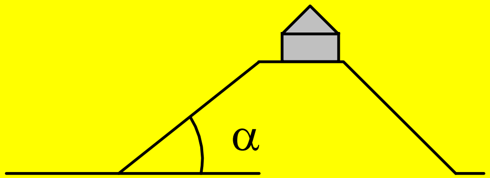
Pseudo-static analysis recommended

$$F_H = 0.5\, a_g\, S\left(W/g\right) \quad , \quad F_V = 0.33 \text{ to } 0.50\, F_H$$

Only valid if no significant loss of shear resistance

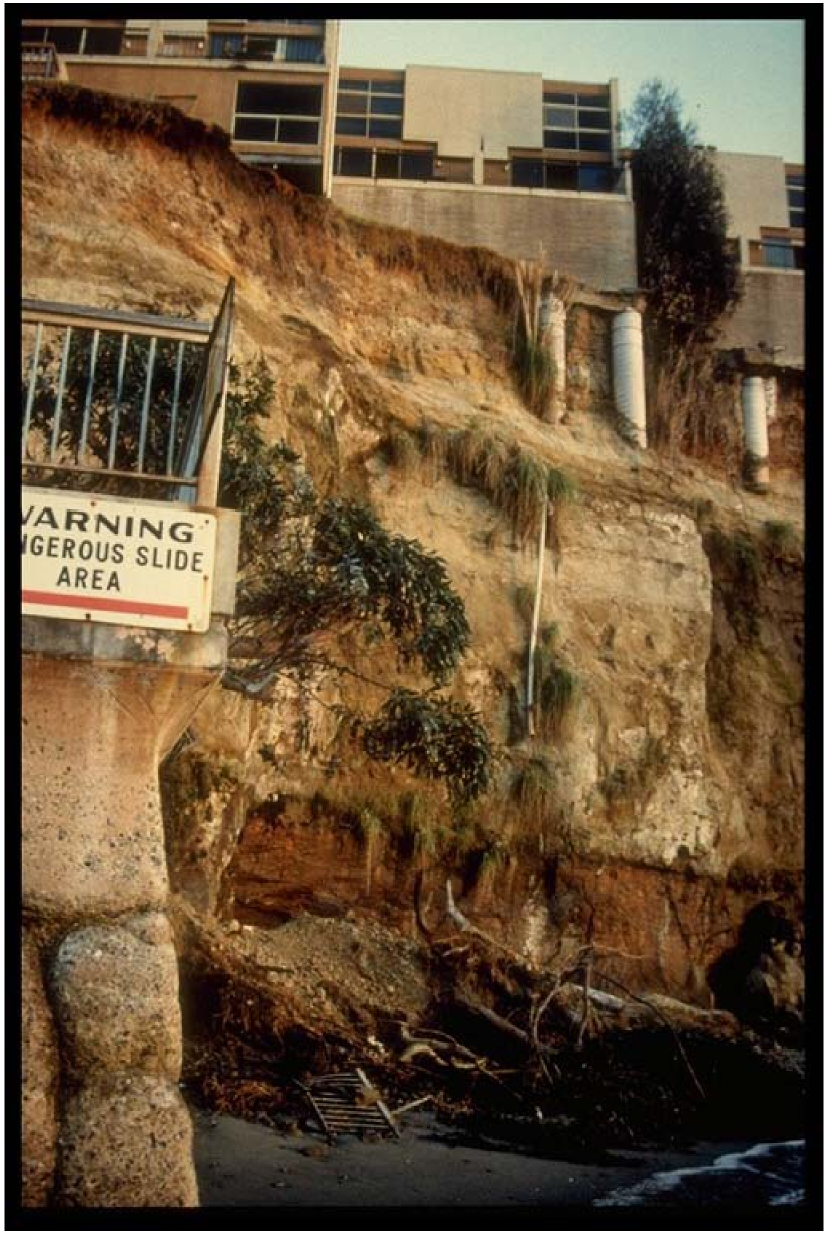
ANNEX A (informative)

Topographic amplification factors (ST)

Type of topographic profile	Sketch	Average slope angle, α	ST
Isolated cliff and slope		> 15°	1.2
Ridge with crest width significantly less than base width		15° to 30°	1.2
		> 30°	1.4

EXAMPLE OF SLOPE INSTABILITY

Loma Prieta 1989



General requirements and considerations

Permanent displacements/tilting may be acceptable, provided functional or aesthetic requirements are not violated

Build-up of significant PWP in backfill or supported soil is to be absolutely avoided

Methods of analysis should account for:

- inertial and interaction effects between structure and soil
- hydrodynamic effects in the presence of water
- compatibility of deformations of soil, wall, and free tendons

EXAMPLE OF BACKFILL LIQUEFACTION

Port Island 1995

PSEUDO STATIC ANALYSIS

Seismic coefficient

$$k_h = \left(a_g / g\right) S / r \qquad k_v = \pm 0.33 \text{ to } 0.50\, k_h$$

k depends on allowable displacement

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

SEISMIC ACTION

Includes the contribution of :

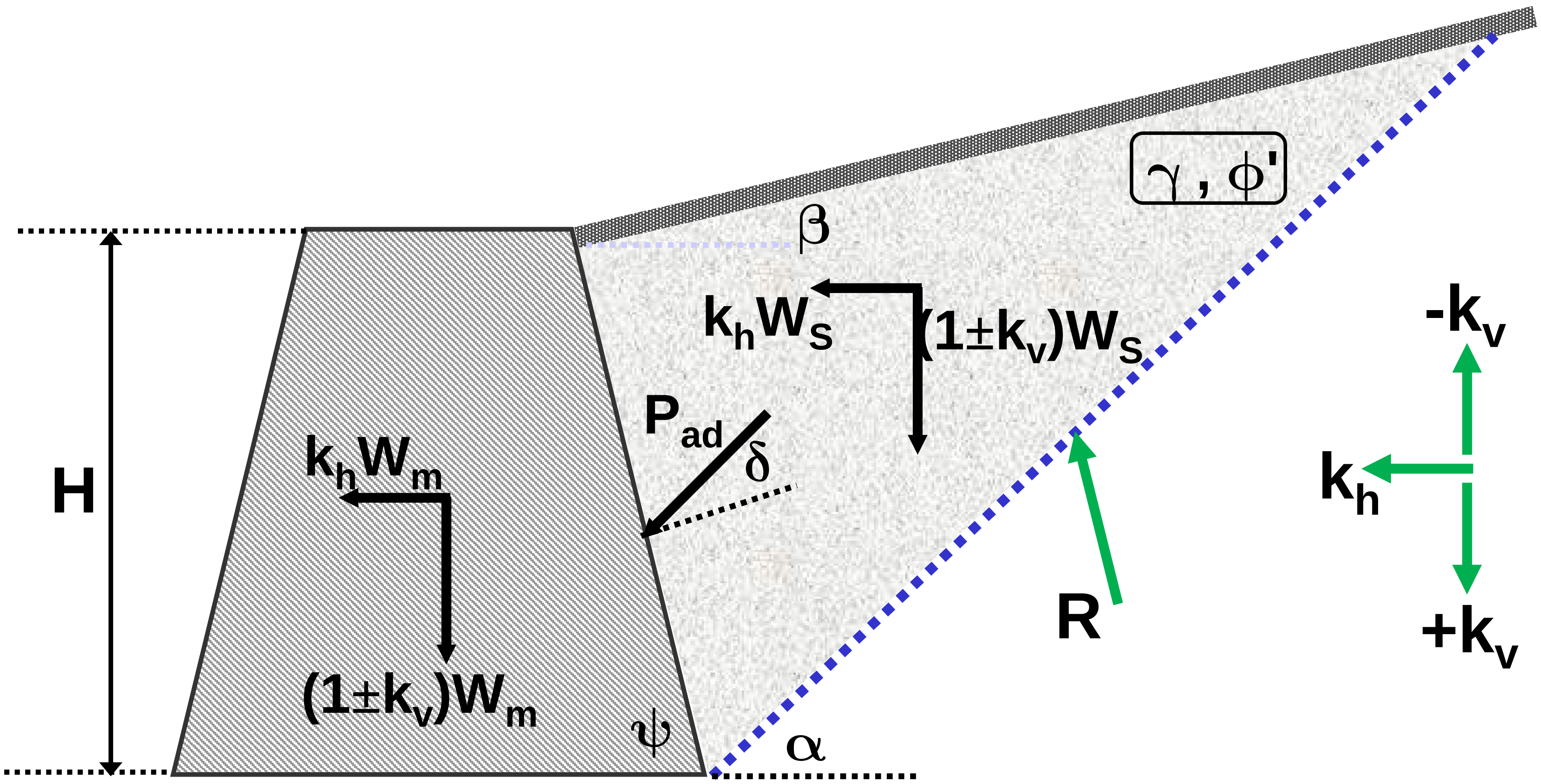
- Static and Dynamic earth pressures
- Hydrostatic and hydrodynamic water pressures
- Inertia forces in the wall

Annex E (normative) describes the Mononobe – Okabe formula

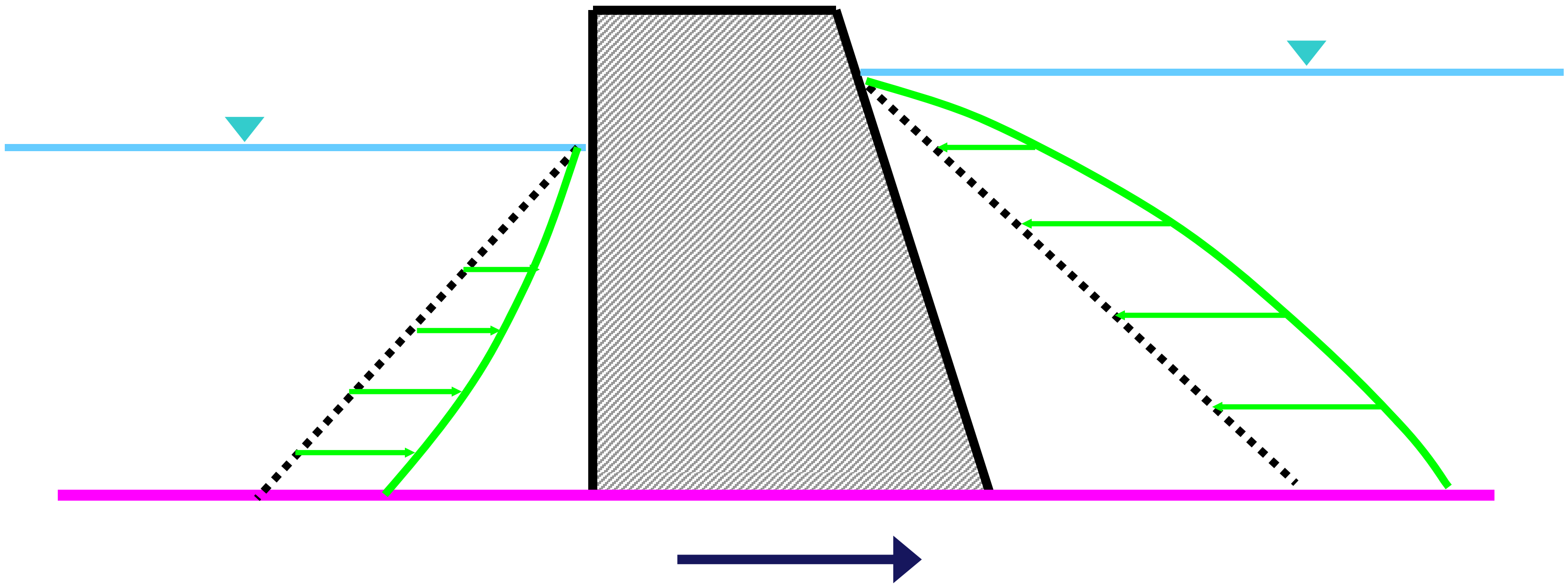
$$E_d = \frac{1}{2}\gamma^{*}(1 \mp k_v)K H^2 + E_{ws} + E_{wd}$$

K and γ^* depend on soil permeability

CALCULATION MODEL



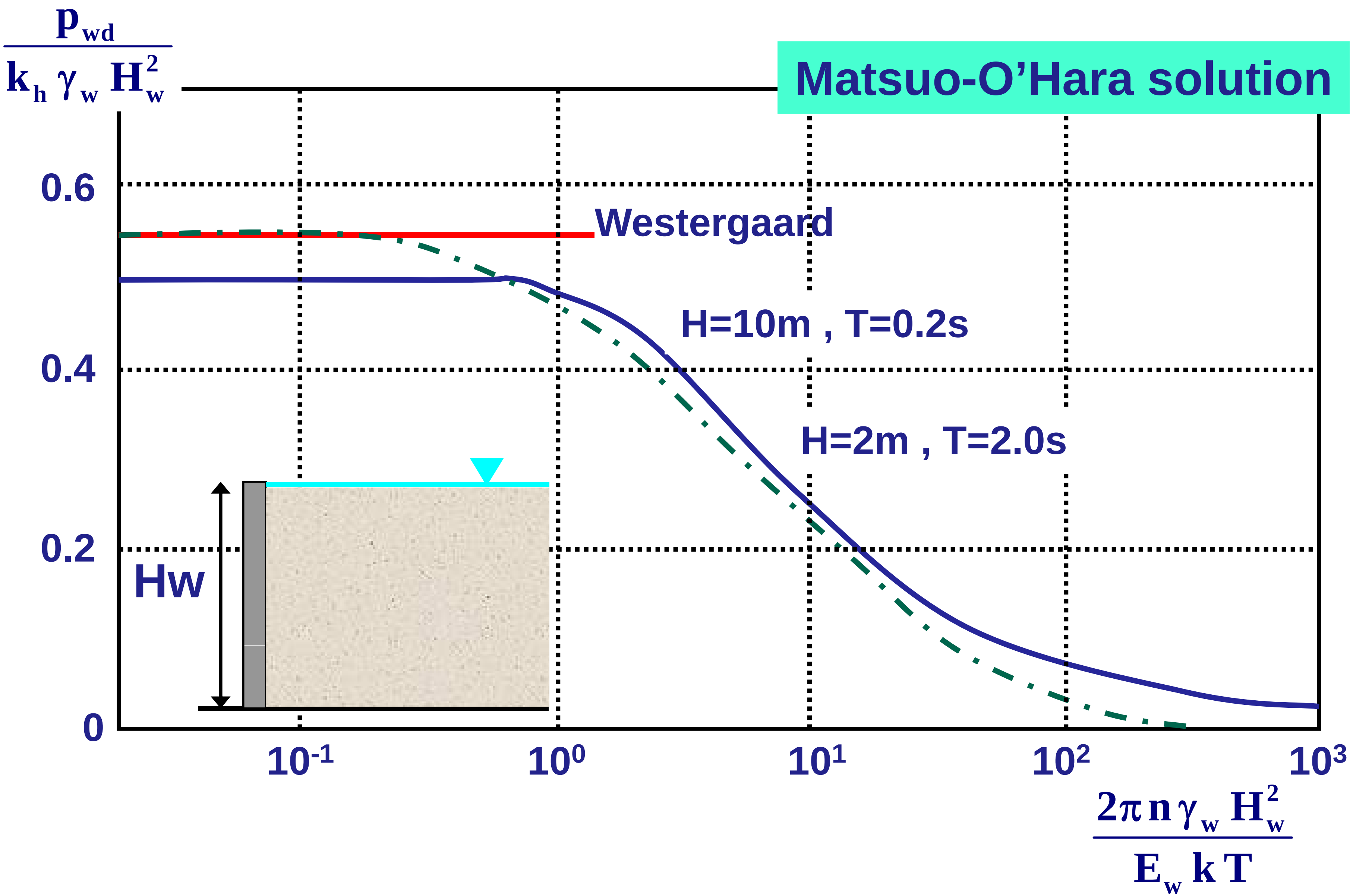
HYDRODYNAMIC WATER PRESSURES



Westergaard formula

$$q(z) = \pm \frac{7}{8} k_h \gamma_w \sqrt{Hz}$$

HYDRODYNAMIC WATER PRESSURES



CHOICE OF PARAMETERS

Dry soil above water

$\gamma^* = \gamma \quad , \quad E_{ws} = E_{wd} = 0 \quad , \quad \tan(\theta) = k_h / (1 \pm k_v)$

Saturated pervious soil below water

$\gamma^* = \gamma - \gamma_w \quad , \quad E_{ws} \ \& \ E_{wd} \neq 0 \ , \quad \tan(\theta) = (\gamma_d / \gamma') k_h / (1 \pm k_v)$

Saturated impervious soil below water

$\gamma^* = \gamma - \gamma_w \quad , \quad E_{ws} \neq 0 \ , \ E_{wd} = 0 \quad , \ \tan(\theta) = (\gamma / \gamma') k_h / (1 \pm k_v)$

RESISTANCE AND STRUCTURAL VERIFICATIONS

Foundation soil

- Stability of slope
- Stability w. r. to failure by sliding and loss of bearing capacity, for shallow foundation.

Anchorages

- 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 :

$$L = L_S\left[1 + 1.5\left(a_g / g\right)S\right]$$

RESISTANCE AND STRUCTURAL VERIFICATIONS

Backfill material must be immune from liquefaction

$FS \geq 2.0$

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:

$\mathbf{R_d > S_d}$

R_d : design resistance of the element,

S_d : design value of the action effect,

FOUNDATIONS

Foundations shall ensure transfer of forces to the soil without significant deformations

Foundation system must be homogeneous

Unless dynamically independent entities

Design action effects

evaluated according to capacity design considerations for *dissipative structures*

for non-dissipative structures, action effects obtained from the analysis

HOMOGENEOUS FOUNDATION SYSTEM

HOMOGENEOUS FOUNDATION SYSTEM

Design verifications

Sliding capacity

$$V_{SD} \leq F_{H1} + F_{H2} + 0.3\ F_B$$

- ✓ F_{H1} : Friction along the base $N_{SD} \tan\delta$
- ✓ F_{H1} : Friction along lateral sides
- ✓ F_B : Ultimate passive resistance

Bearing capacity (annex F – informative)

- ✓ Inclination and eccentricity of structural loads
- ✓ Inertia forces in soil
- ✓ F_B : Ultimate passive resistance

SPECIAL PROVISIONS

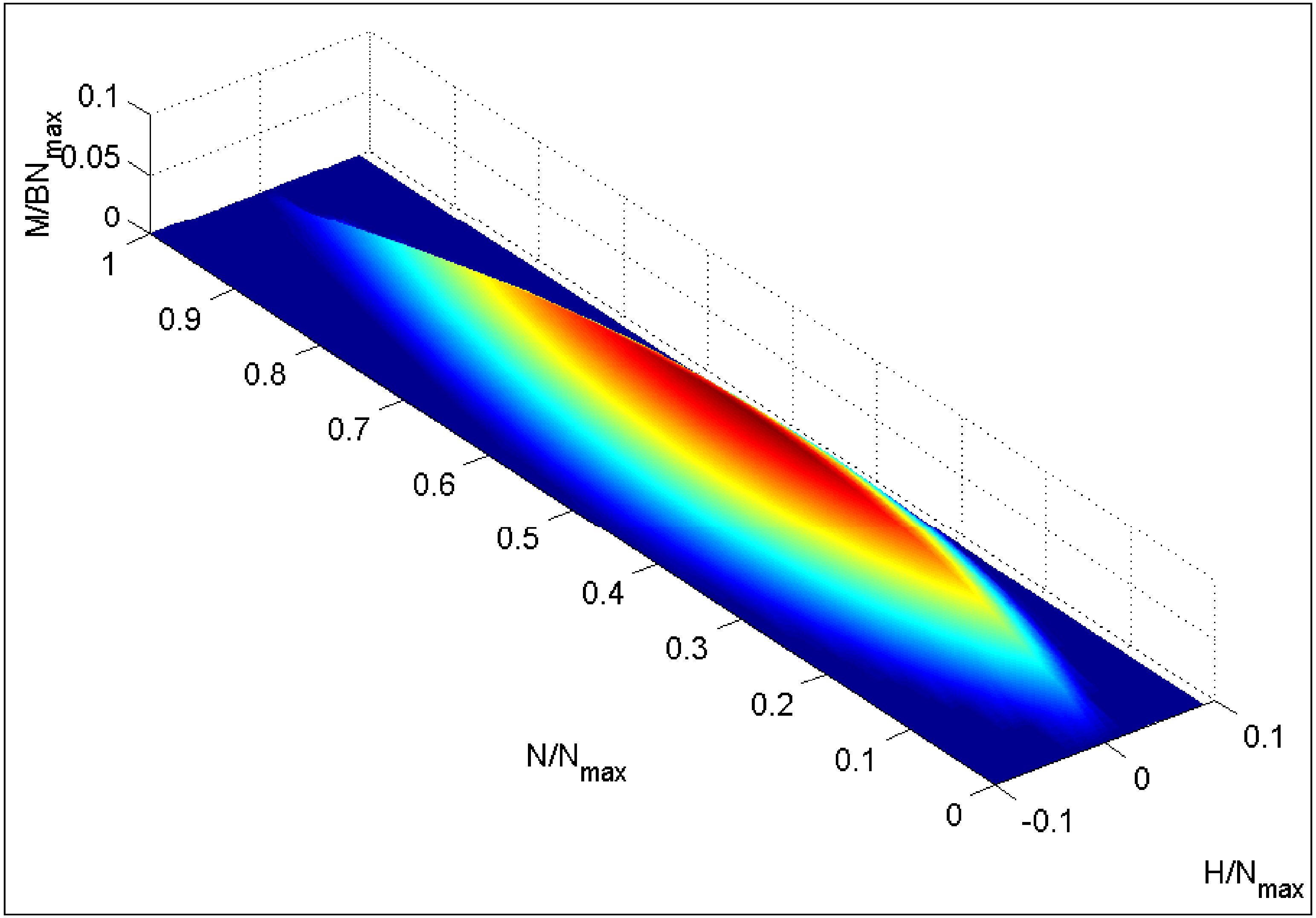
Sliding allowed provided

- Ground characteristics remain unaltered
- Sliding does not affect functionality of lifelines

Tie beams are mandatory except

- ground type A (rock)
- Low seismicity and ground type B (stiff soil)
- Beams of lower level can be used if h < 1m

BEARING CAPACITY



SURFACE OF ULTIMATE LOADS

$$\frac{\left(1-e\overline{F}\right)^{c_T}\left(\beta\overline{V}\right)^{c_T}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}}+\frac{\left(1-f\,\overline{F}\right)^{c'_M}\left(\gamma\overline{M}\right)^{c_M}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}}-1\leq 0$$

$$\overline{N}=\frac{\gamma_{RD}N_{sd}}{N_{max}}$$

$$\overline{V}=\frac{\gamma_{RD}V_{sd}}{N_{max}}$$

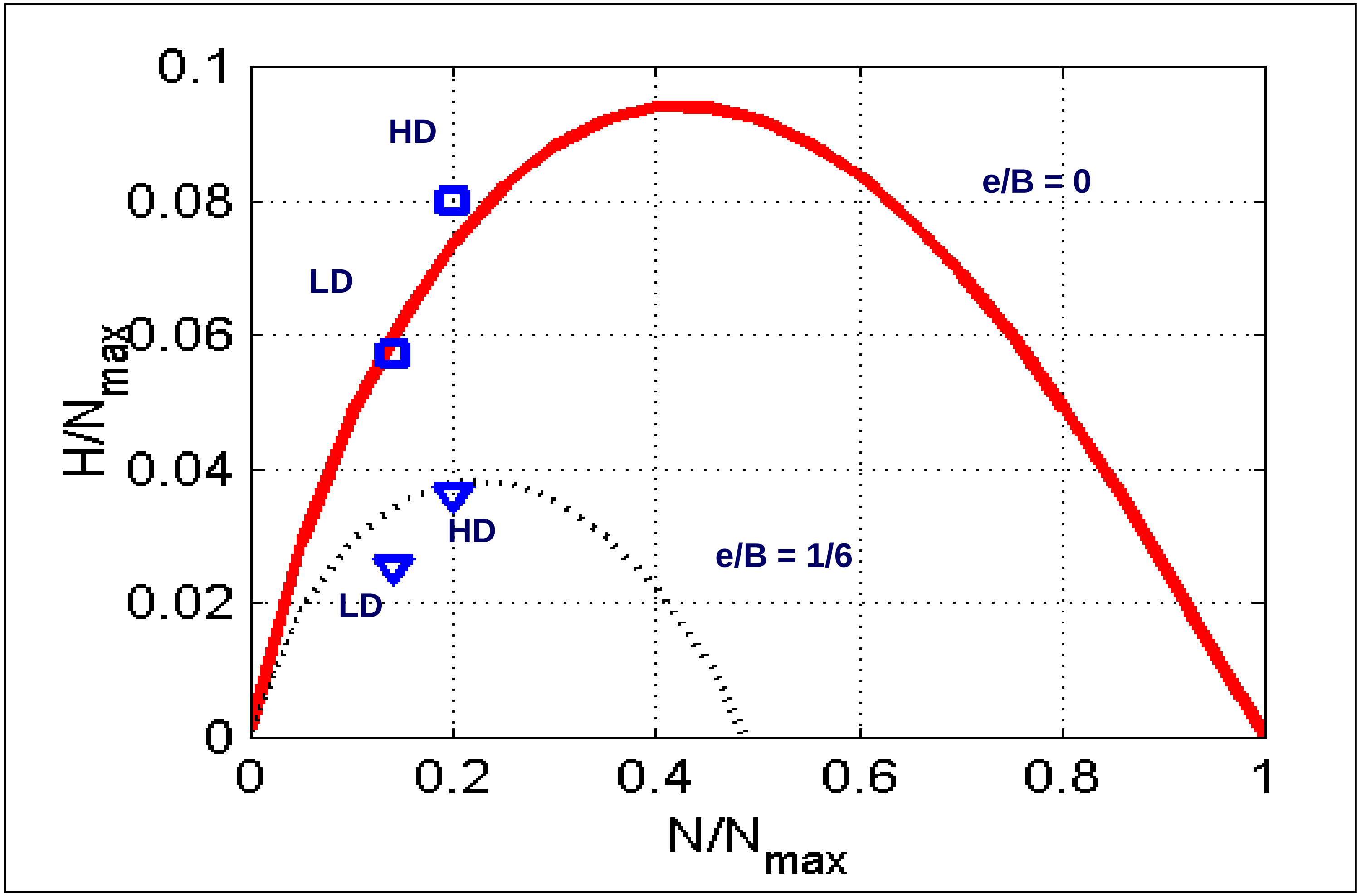
$$\overline{M}=\frac{\gamma_{RD}M_{sd}}{B\,N_{max}}$$

$$\overline{F}=\left\{\begin{matrix}\dfrac{\gamma_{RD}\,\rho\,a\,B}{C_u}\\[2ex]\dfrac{\gamma_{RD}\,a}{g\tan\phi}\end{matrix}\right\}$$

SURFACE OF ULTIMATE LOADS

	Purely cohesive soil	Purely cohesionless soil
a	0,70	0,92
b	1,29	1,25
c	2,14	0,92
d	1,81	1,25
e	0,21	0,41
f	0,44	0,32
m	0,21	0,96
k	1,22	1,00
K'	1,00	0,39
c_T	2,00	1,14
c_M	2,00	1,01
c'_M	1,00	1,01
β	2,57	2,90
γ	1,85	2,80

CROSS SECTION OF SURFACE OF ULTIMATE LOADS



MODEL FACTOR γ_{rd}

Medium dense sand	Loose dry sand	Loose saturated sand	Non sensitive clay	Sensitive clay
1.0	1.15	1.50	1.0	1.15

γ_{rd} reflects

- Approximation of theoretical model
- Allowance for permanent moderate displacements

Building design according to capacity design

⇒ **Clause 5.3.1 of EN 1998-5 for dissipative structures applies**

“The action effect for the foundations shall be based on capacity design considerations accounting for the development of possible overstrength”

4.4.2.6 of EN 1998-1 gives the design values of the action effects on foundation

DESIGN VALUES OF ACTION EFFECT

$$E_{\mathrm{Fd}} = E_{\mathrm{F,G}} + \gamma_{\mathrm{Rd}}\,\Omega E_{\mathrm{F,E}}$$

γ_{Rd} : overstrength factor = 1 for $q \leq 3$, 1.2 otherwise

$\Omega = R_{\mathrm{di}} / E_{\mathrm{di}} \leq q$

R_{di} design resistance

E_{di} design value of action effect in seismic situation

✓ Following table gives the values of E_{Fd}

$\gamma_{\mathrm{Rd}}\,\Omega = q = 3$

EXAMPLE OF BEARING CAPACITY CALCULATIONS

Column 7 of example building

	N	My	Vy	Mz	Vz	V	M
	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN)	(kN.m)
+X/+Y/max	2861	21	9	27	11	14	34
-X/+Y/max	2861	26	12	27	11	16	37
+X/-Y/max	2861	21	9	28	11	14	35
-X/-Y/max	2861	26	12	28	11	16	38
+X/+Y/min	2744	21	9	27	11	14	34
-X/+Y/min	2744	26	12	27	11	16	37
+X/-Y/min	2744	21	9	28	11	14	35
-X/-Y/min	2744	26	12	28	11	16	38

BEARING CAPACITY CALCULATIONS (I)

- **Footing dimensions 2m x 2m :**

  results from bearing capacity under permanent loads

- **Soil conditions : ground type B – stiff clay**

  - Assume C_u = 300 kPa for static conditions
  - For seismic conditions 10% reduction for cyclic degradation $C_u = 270$ kPa
  - Material factor $\gamma_M = 1.4 \Rightarrow C_{ud}$ y 195 kPa
  - According to annex F of EN 1998-5 $\gamma_{RD} = 1$

BEARING CAPACITY CALCULATIONS (II)

Although Annex F is for strip footing :

can be used for circular footing with appropriate value of N_{max}

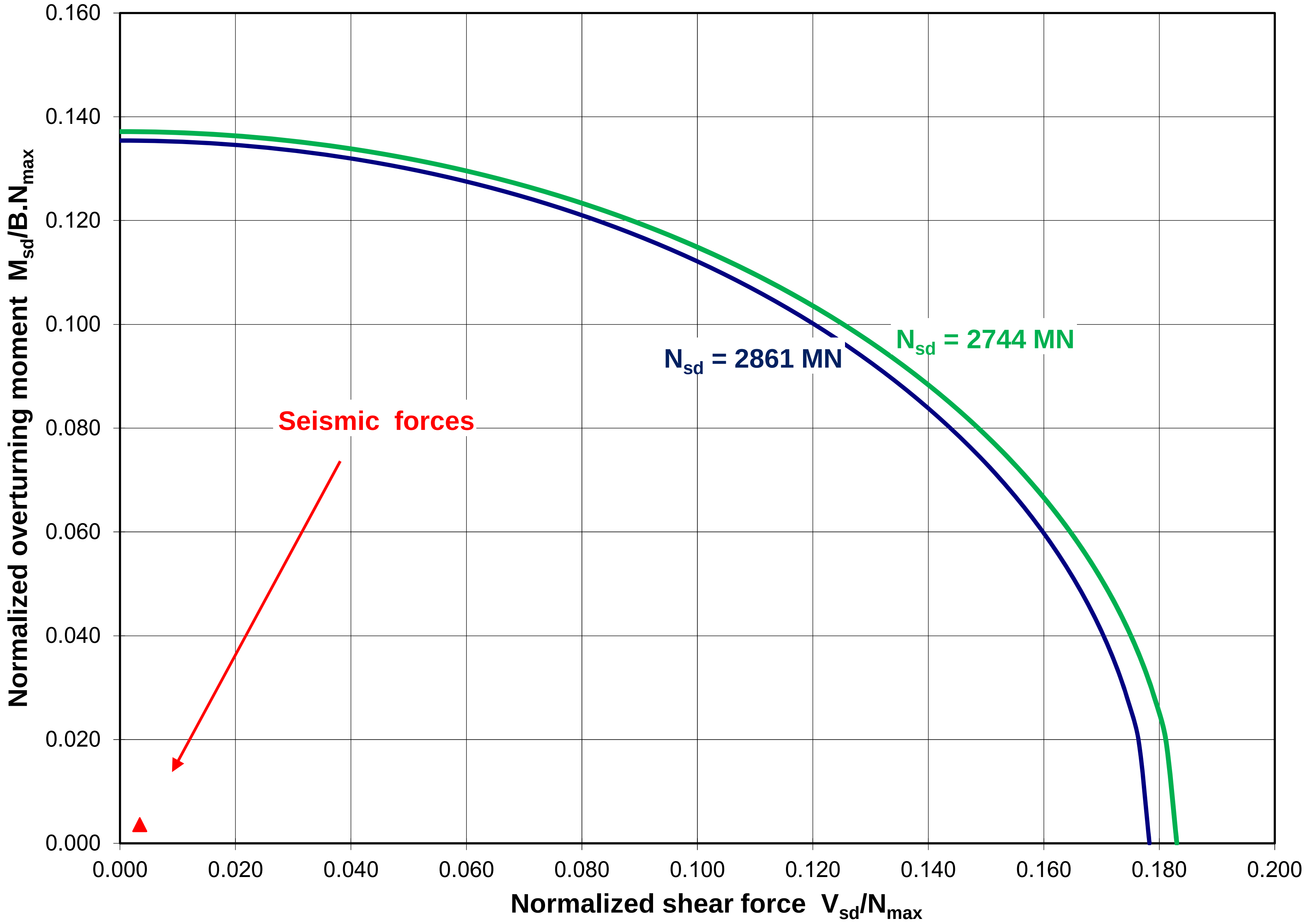
$N_{max} = \pi\, r^2\, N_c\, C_{ud} = 3.14 \times 1.13^2 \times 6.0 \times 195 = 4680$ kPa

$$\overline{N} = \frac{\gamma_{RD} N_{sd}}{N_{max}} = \frac{2861 \text{ or } 2744}{4680} = 0.61 \text{ or } 0.59$$

$$\overline{V} = \frac{\gamma_{RD} V_{sd}}{N_{max}} = \frac{16}{4680} = 0.0035 \qquad \overline{M} = \frac{\gamma_{RD} M_{sd}}{B N_{max}} = \frac{38}{2 \times 4680} = 0.0041$$

$$\overline{F} = \frac{\gamma_{RD}\, \rho\, a\, B}{C_{ud}} = \frac{2 \times 2.5 \times 2}{195} = 0.05$$

VERIFICATIONS



PILES AND PIERS

Should be designed to resist both:

Inertia forces from the superstructure

Kinematic forces due to the earthquake-induced soil deformations.

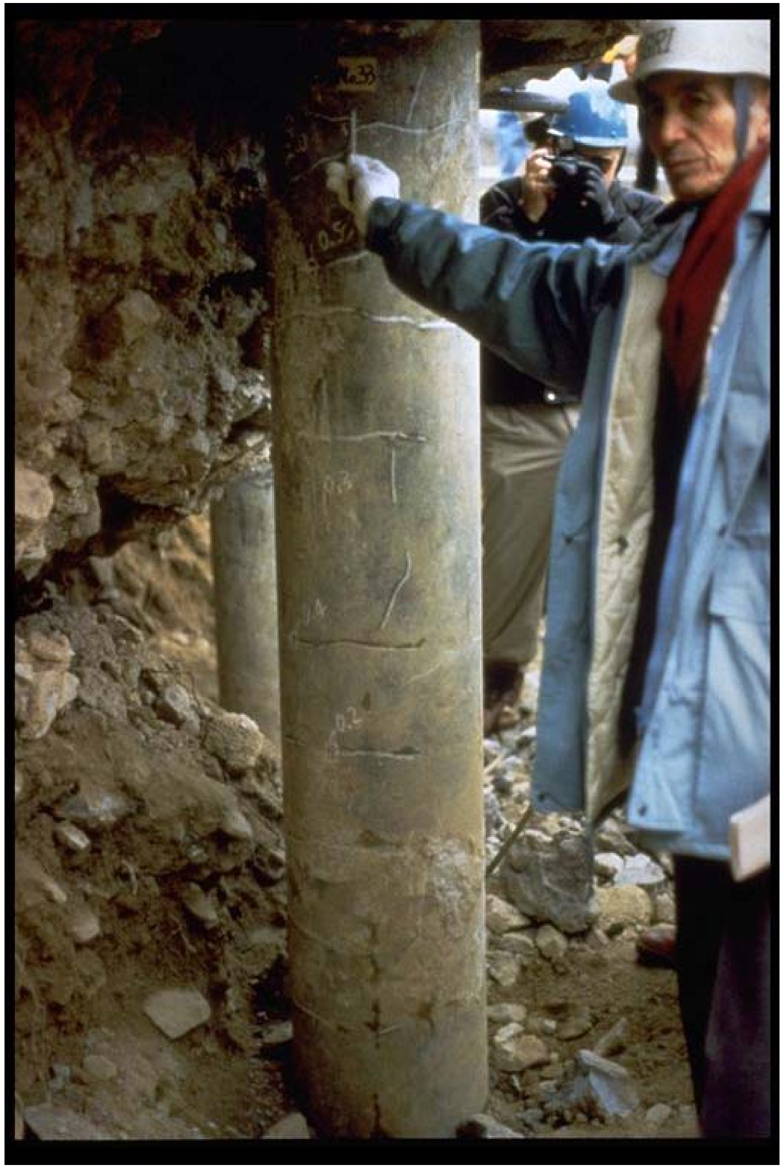
Kinematic interaction only required

Ground type D, S_1 or S_2 with consecutive layers of sharply contrasting stiffness

Design ground acceleration > 0.10 g, and

The supported structure is of importance category III or IV

EFFECT OF KINEMATIC INTERACTION ON PILES



PILE CAP CONNECTION

SPECIAL PROVISIONS

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

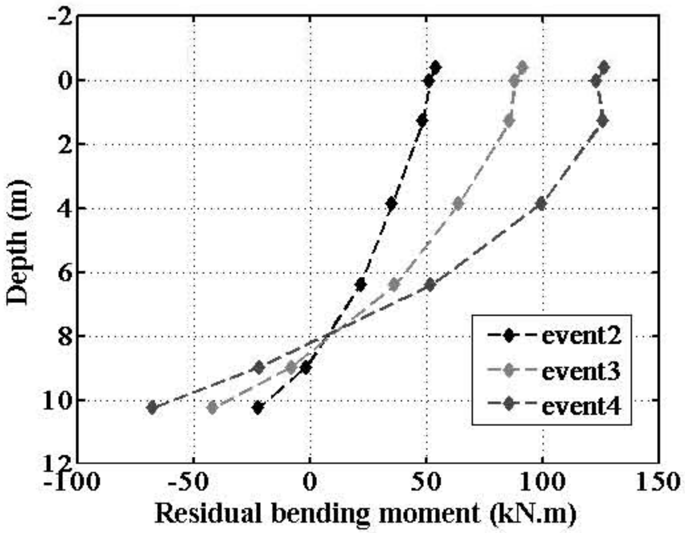
Inclined piles not recommended

- Although they carry out large horizontal forces
- Poor observed behaviour during earthquake but there exists counter examples
- Highly sensitive to soil settlement
- Less ductile behaviour than flexural piles

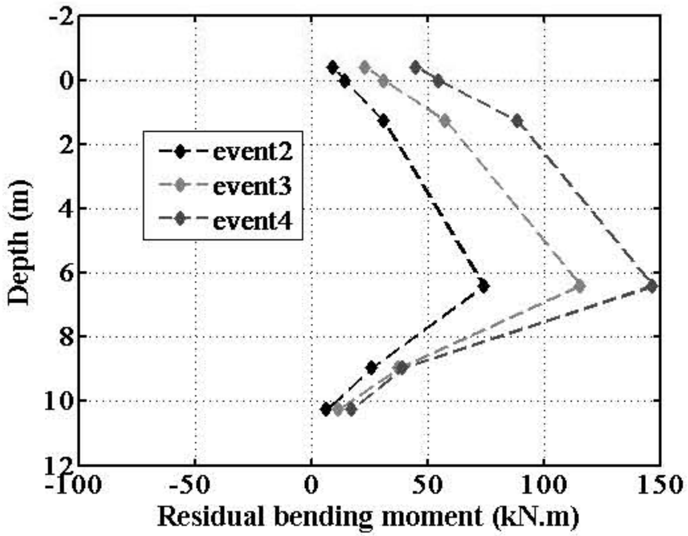
RESIDUAL BENDING MOMENTS IN PILES : CENTRIFUGE TESTS

LCPC, 2010

Inclined piles

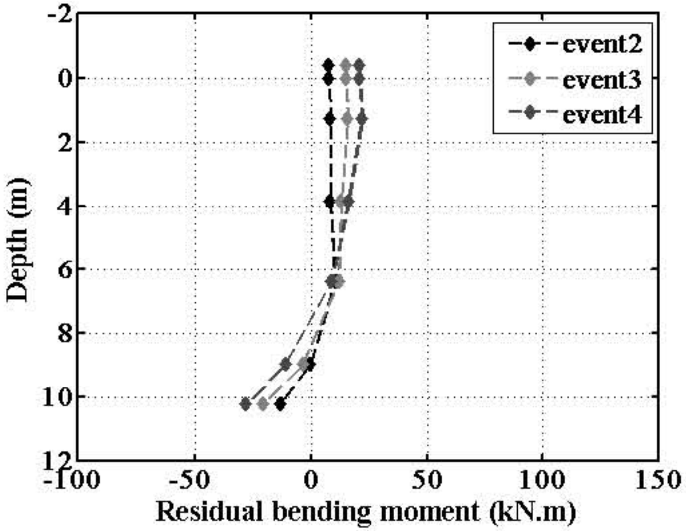


(a)

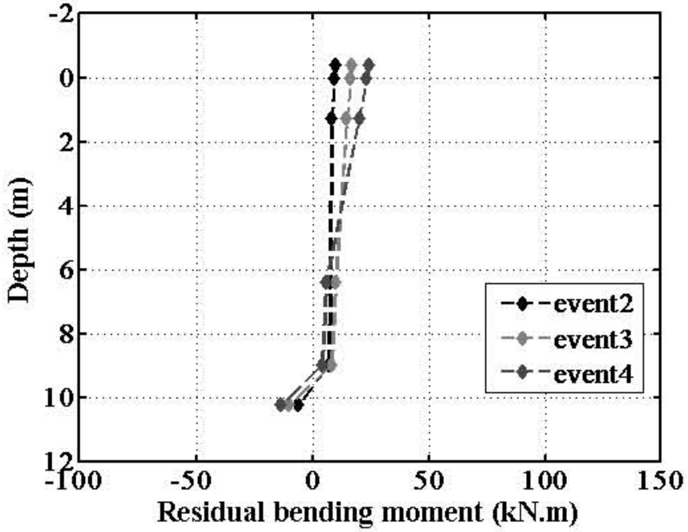


(c)

Vertical piles



(b)



(d)

SOIL STRUCTURE INTERACTION (annex D)

Mandatory for

Structures sensitive to p-δ effects

Massive or deeply embedded foundations

Slender structures (tower, mast…)

Structures founded on soft soil deposits $V_S < 100$ m/s

Piled foundations (see annex E for pile head stiffness)

EFFECT OF SOIL STRUCTURE INTERACTION

Mexico 1985