EUROCODE EN1998-2

SEISMIC DESIGN OF BRIDGES

- *Basil Kolias*

Basic Requirements

- **Non-Collapse**
  - Retain structural strength + residual resistance for emergency traffic.
  - Limit damage to areas of energy dissipation.

- **Damage Minimization**
  - Under probable seismic effects.

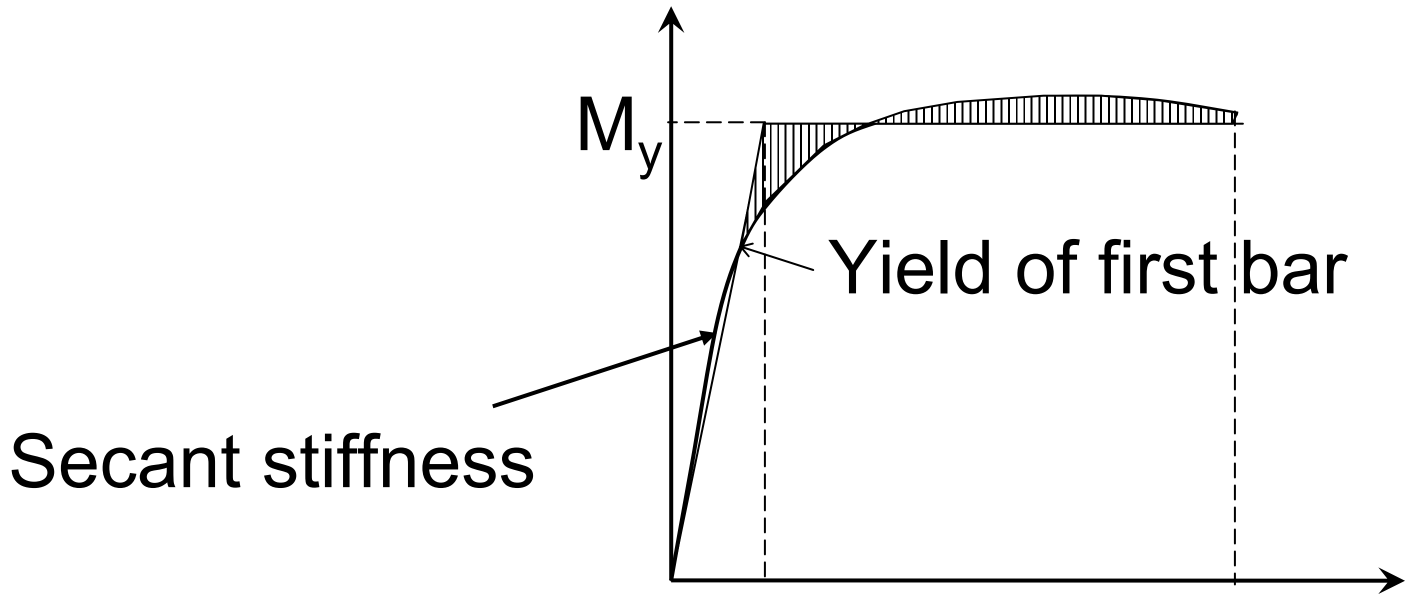
Analysis Methods

- **Equivalent Linear Analysis:**
  - Elastic force analysis (response spectrum) forces from unlimited elastic response divided by global behaviour factor = q.

design spectrum = elastic spectrum / q

Analysis Methods

- **Stiffness of Ductile Elements:**
  - secant stiffness at the theoretical yield



Analysis Methods

- **Non-linear Dynamic Time-History Analysis:**
  - In combination with response spectrum analysis without relaxation of demands.
  - For irregular bridges.
  - For bridges with seismic isolation.

- **Non-linear Static Analysis (Push-Over):**
  - For irregular bridges.

Ductility Classes

- **Limited Ductile Behaviour:**

  $$q \leq 1.50$$

- **Ductile Behaviour:**

  $$1.50 < q \leq 3.50$$

Compliance Criteria for Elastic Analysis

- **Limited Ductile Behaviour:**
  - Section verification with seismic design effects A_{Ed}
  - Verification of non-ductile failure modes (shear and soil) with elastic effects qA_{Ed} and reduction of resistance by $\gamma_{Bd} = 1.25$

Compliance Criteria for Elastic Analysis

- **Ductile Behaviour:**
  - Flexural resistance of plastic hinge regions with design seismic effects A_{Ed}.
  - All other regions and non-ductile failure modes (shear of elements & joints and soil) with capacity design effects A_C.
  - Local ductility ensured by special detailing rules (mainly confinement).

Compliance Criteria for Elastic Analysis

- **Control of Displacements:**
  - Assessment of seismic displacement d_E

$d_E = \eta \mu_d d_{Ee}$.

d_{Ee} = result of elastic analysis.

η = damping correction factor.

μ_d = displacement ductility as follows:

when $T \geq T_0 = 1.25 T_C$: $\boldsymbol{\mu_d = q}$

when $T < T_0$: $\boldsymbol{\mu_d = (q-1)T_0 / T + 1 \leq 5q - 4}$

Compliance Criteria for Elastic Analysis

- **Provision of adequate clearances for the total seismic design displacement:**

$$d_{Ed} = d_E + d_G + 0.5d_T$$

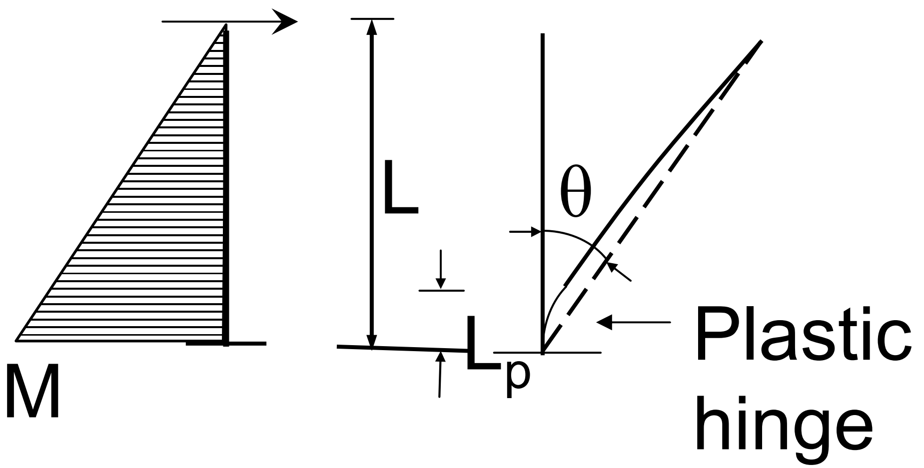
d_G due to permanent and quasi-permanent actions.

d_T due to thermal actions.

- **For roadway joints: 40% d_E and 50% d_T**

Compliance Criteria for Non-linear Analysis

Chord rotation: $\theta = \theta_y + \theta_p$



Compliance Criteria for Non-linear Analysis

- **Ductile Members:**
  Deformation verification

  - Plastic chord rotations of plastic hinges:
    demand ≤ design capacity

$$\theta_{p,E} \leq \theta_{p,d} \quad , \quad \theta_{p,d} = \theta_{p,u} / \gamma_{R,p} \quad , \quad \gamma_{R,p} = 1.40$$

$\theta_{p,u}$ = probable (mean) capacity from tests or derived from ultimate curvatures

Compliance Criteria for Non-linear Analysis

- **Non-ductile members:**
  - **Force verification** as in elastic analysis for regions outside plastic hinges and non-ductile failure modes, with capacity design effects replaced by:

$$\gamma_{R,Bd1} A_{Ed} \quad \text{with} \quad \gamma_{R,Bd1} = 1.25$$

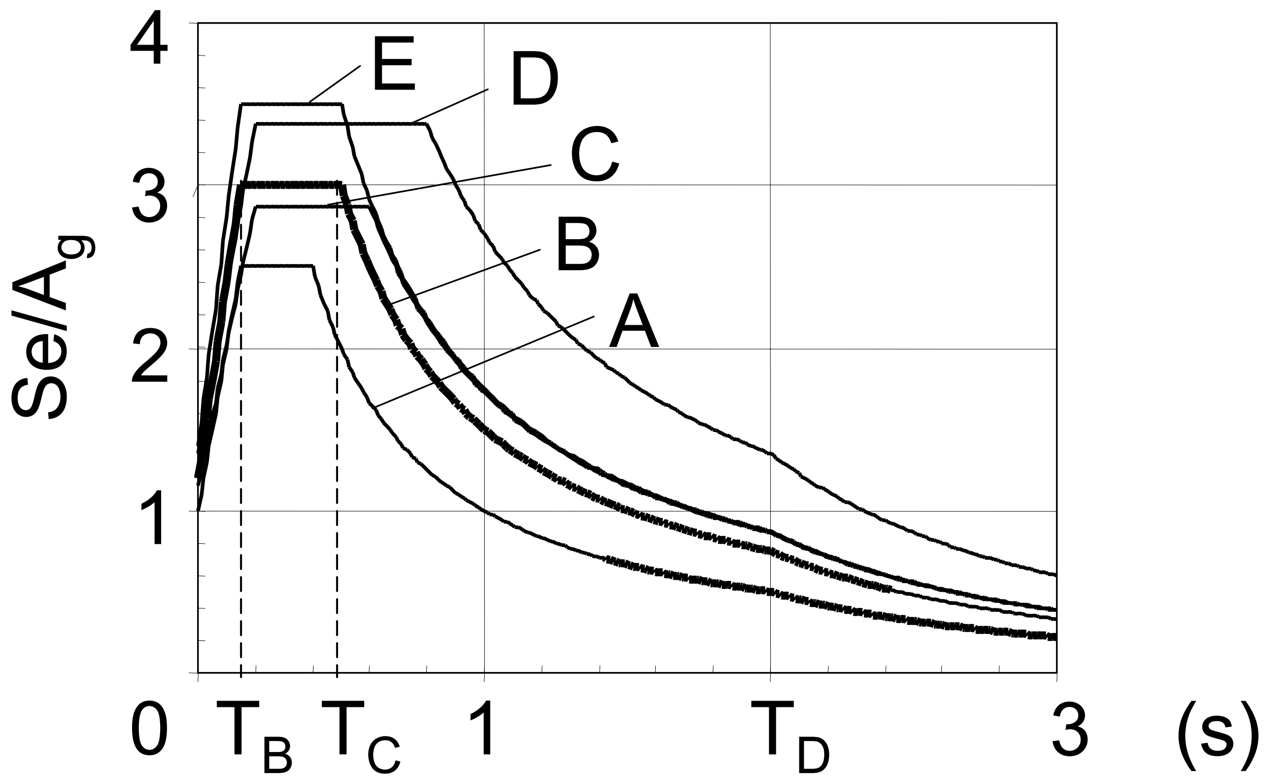
  - Design resistances:

$$R_d = R_k / \gamma_M$$

Seismic Action

- **Two types of elastic response spectra:**
  - Type 1 and 2.
- **5 types of soil:**
  - A, B, C, D, E.
- **4 period ranges:**
  - short, constant acceleration, velocity and displacement.
- **Design spectrum = elastic spectrum / *q*.**
- **3 importance classes:**
  - γ_I = 1.3, 1.0, 0.85.

Elastic Spectrum Type 1 ($\xi = 0.05$)



Seismic Action: Spatial Variability

Spatial variability model should account for:

- Propagation of seismic waves
- Loss of correlation due to reflections/refractions
- Modification of frequency content due to diff mechanical properties of foundation soil

Seismic Action: Spatial Variability

- **Rigorous model in Inf. Annex D:**
- **Simplified method:**
  - ⇒ Uniform support excitation + pseudostatic effects of two sets of displacement (A and B) imposed at supports.
  - ⇒ Sets A and B applied in the two principal horizontal directions but considered independently

Seismic Action: Spatial Variability

Displacement sets defined from:

- $d_g = 0.025\, a_g S T_C T_D$: max particle displ. corresponding to the ground type (EC8-1)
- L_g is the distance beyond which seismic motion is completely uncorrelated

Recommended Values of L_g(m)

Ground Type	A	B	C	D	E
L_g(m)	600	500	400	300	500

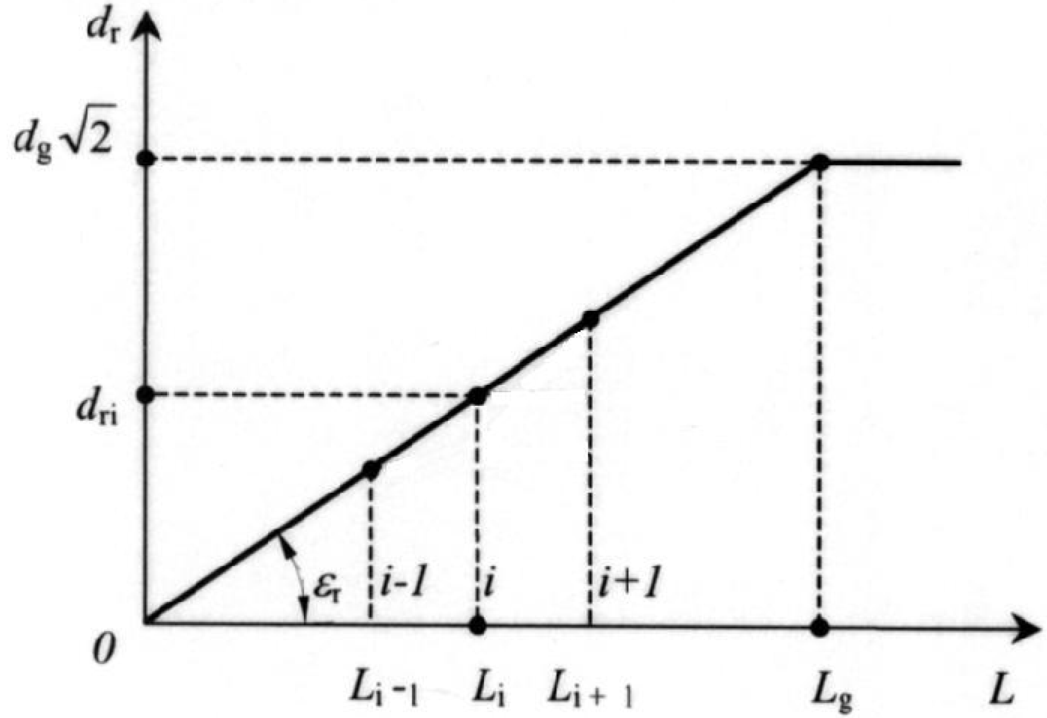
Seismic Action: Spatial Variability

Displacement set A
uniform expansion/contraction

- displacement of support **i** relative to support **0**

$$d_{ri} = \varepsilon_r \, L_i \le d_g \sqrt{2}$$

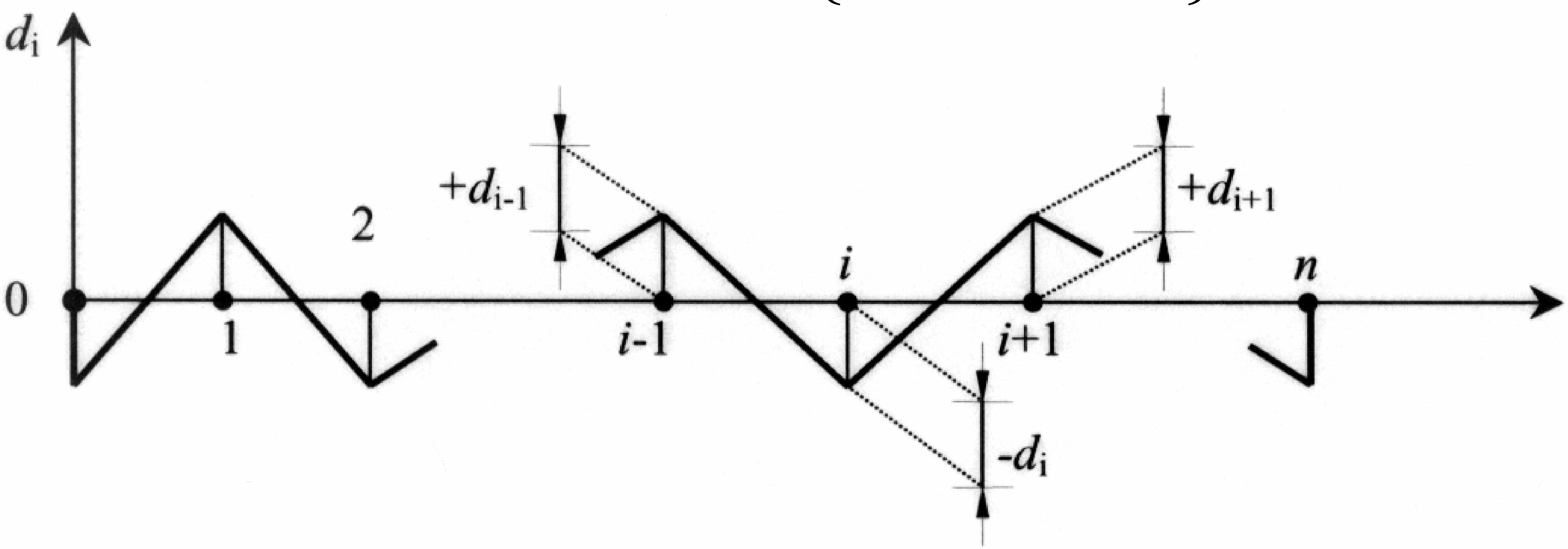
$$\varepsilon_r = \frac{d_g \sqrt{2}}{L_g}$$



Displacement set B with opposite directions at adjacent piers

$$\Delta d_i = \pm \beta_r \varepsilon_r \, L_{av,i}$$

$$d_i = \pm \Delta d_i / 2 \qquad \beta_r = \left. \begin{cases} 0.5 & \text{same} \\ 1.0 & \text{different} \end{cases} \right\} \text{ground type}$$



Regular / Irregular Bridges

- Criterion based on local required force reduction factors r_i of the ductile members i :

$$r_i = qM_{Ed,i}/M_{Rd,I} =$$

q x Seismic moment / Section resistance

- A bridge is considered regular when the “irregularity” index:

$$\rho_{ir} = max(r_i) / min(r_i) \leq \rho_0 = 2$$

- Piers contributing less than 20% of the average force are not considered

Regular / Irregular Bridges

- **For regular bridges:** equivalent elastic analysis is allowed with the ***q***-values specified, without checking of local ductility demands

- **Irregular bridges are:**
  - either designed with reduced behaviour factor:

$$q_r = q\,\rho_o / \rho_{ir} \geq 1.0$$

  - or verified by non-linear static (pushover) or dynamic analysis

Capacity Design Effects

- Correspond to the section forces under permanent loads and a seismic action creating the assumed pattern of plastic hinges, where the flexural overstrength:

$$M_o = \gamma_o M_{Rd}$$

  has developed with: $\gamma_o = 1.35$

- Simplifications satisfying the equilibrium conditions are allowed.

Detailing Rules

Confinement reinforcement

- Increasing with:
  - Normalised axial force: $\eta_k = N_{Ed} / (A_c f_{ck})$.
  - Axial reinforcement ratio ρ (for $\rho > 0.01$).
- Not required for hollow sections with:
  - $\eta_k \leq 0.20$ and restrained reinforcement.
- Rectangular hoops and crossties or Circular hoops or spirals or overlapping spirals

Detailing Rules

Restraining of axial reinforcement against buckling

- max support spacing:

$$s_L \leq \delta \text{Ø}_L$$

$$5 \leq \delta = 2{,}5\,(f_t / f_y) + 2{,}25 \leq 6$$

- minimum amount of transverse ties:

$$A_t / s_T = \Sigma A_s f_{ys} / 1{,}6 f_{yt} \quad (\mathrm{mm^2/m})$$

Detailing Rules

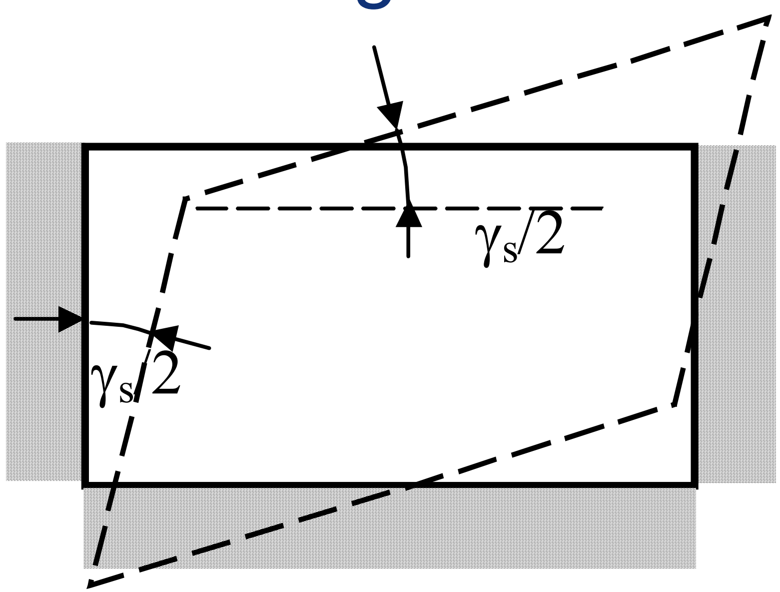
- **Hollow piers**
  - In the region of the plastic hinges
    Limitation of wall slenderness ratio:

$$b / t \quad \text{or} \quad D / t \leq 8$$

- **Pile foundations**
  - Rules for the location and required confinement of probable plastic hinges

Detailing Rules

- Bearings and seismic links.
- Holding down devices.
- Shock transmission units (STU).
- Min. overlap lengths at movable supports.
- Abutments and retaining walls.
- Culverts with large overburden.



$$\gamma_s = v_g/v_s$$

Bridges with Seismic Isolation

- **The isolating system arranged over the isolation interface reduces the seismic response by:**
  - either lengthening of the fundamental period.
  - or increasing of the damping.
  - or (preferably) by combination of both effects.

Design properties of the isolating system

- Nominal design properties (NDP) assessed by prototype tests, confirming the range accepted by the Designer.
- Design is required for:
  - Upper Bound design properties (UBDP).
  - Lower Bound design properties (LBDP).
- Bounds of Design Properties result either from tests or from modification factors.

Analysis methods

- Fundamental or multi mode spectrum analysis (subject to specific conditions).
- Non-linear time-history analysis.

Bridges with Seismic Isolation

Substructure

- Design for limited ductile behaviour: $\mathbf{q \leq} \; \boldsymbol{1.50}$

Bridges with Seismic Isolation

Compliance criteria

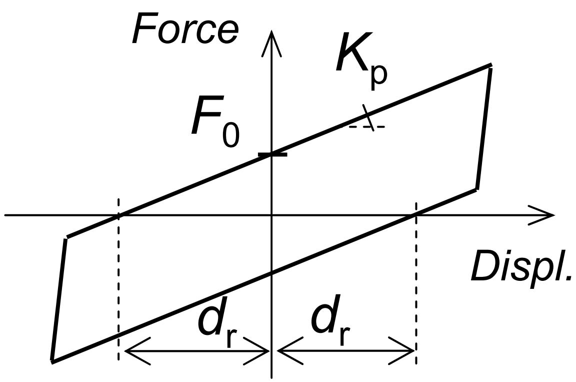
- **Isolating system**
  - Displacements increased by factor:

$$\gamma_{IS} = 1.50$$

  - Sufficient lateral rigidity under service conditions is required.

Lateral restoring capability (revision)

- **Governing parameter: d_{cd}/d_r**
  - d_{cd} = design displacement
  - $d_r = F_0/K_p$ = maximum residual displ.



- **Condition (1):** insignificant residual displ.: $\dfrac{d_{cd}}{d_r} \leq \delta$

  $\delta = 0.5$

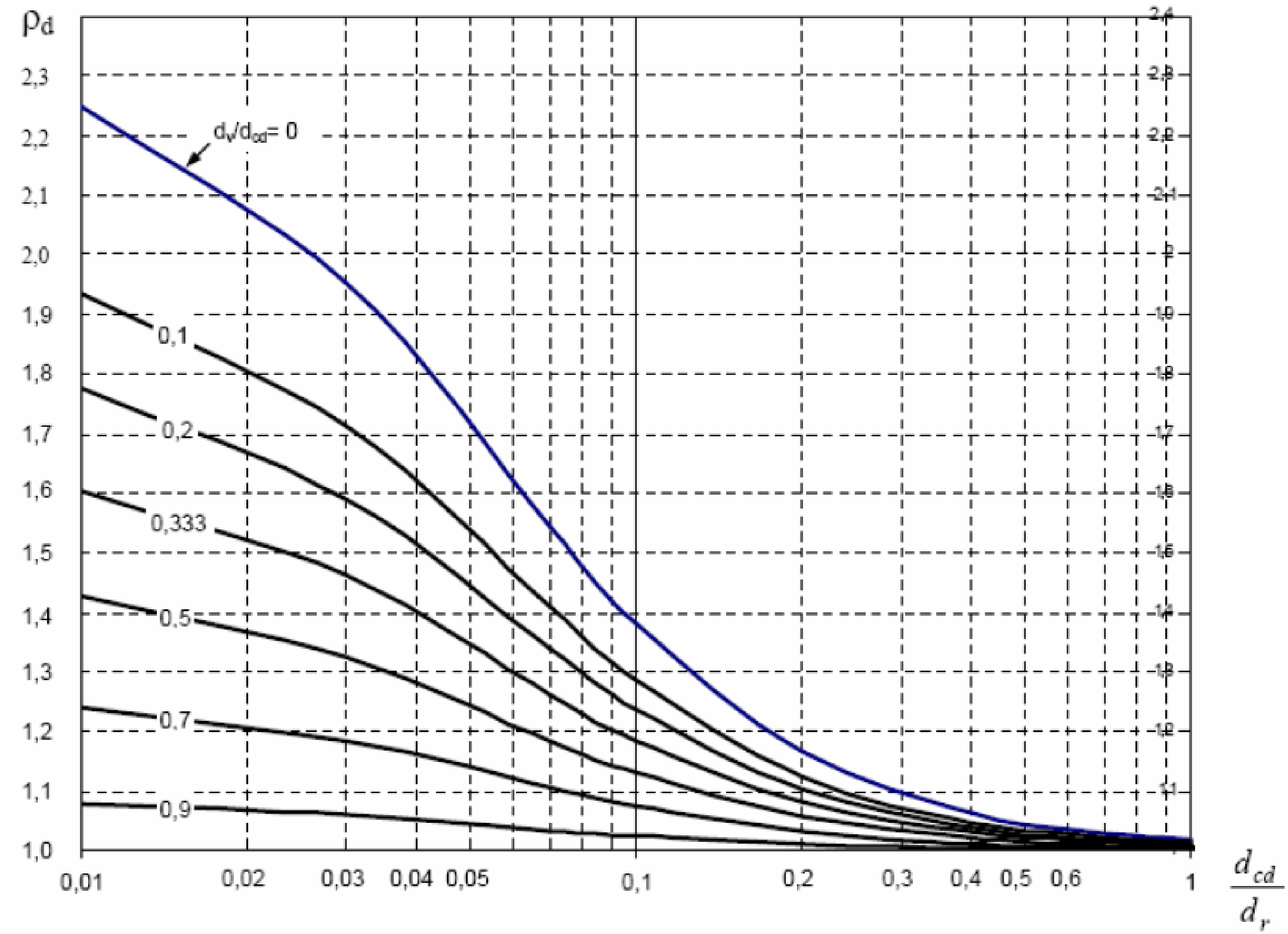
Or

- **Condition (2):** adequate capacity for accumulated residual displ.:

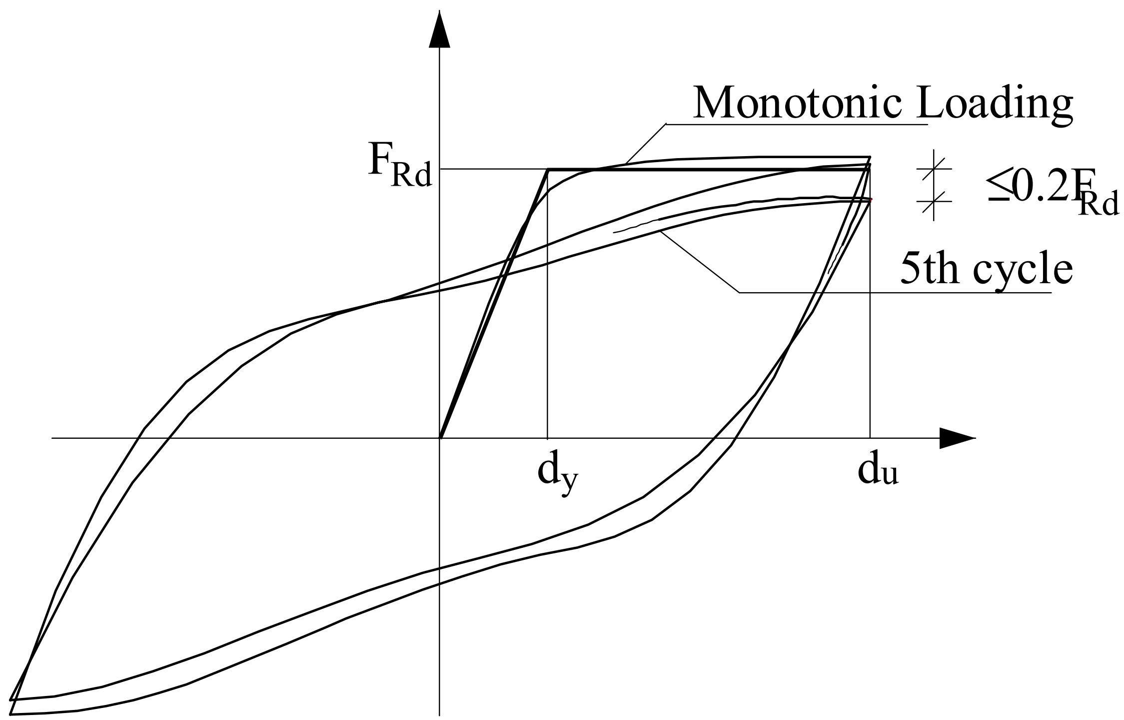
$$d_{mi} \geq d_{o,i} + \gamma_{du} d_{bi,d} \rho_d \qquad \rho_d = 1 + 1.35 \frac{1 - (d_y / d_{cd})^{0.6}}{1 + 80 (d_{cd} / d_r)^{1.5}} \qquad \gamma_{du} = 1.20$$

Bridges with Seismic Isolation

Lateral restoring capability



Ultimate Displacement



Deformation Capacity of Piers

Chord rotation $\boldsymbol{\theta_u = \theta_y + \theta_p}$

Plastic chord rotation $\boldsymbol{\theta_p}$ derived:

- **Directly from appropriate tests**
- **From the curvature, by integration**

Deformation Capacity of Piers

- **Ultimate curvature:** $\Phi_u = \dfrac{\varepsilon_{su} - \varepsilon_{cu}}{d}$

  - Reinforcement: $\boldsymbol{\varepsilon_{su} = 0.075}$ (EN1992-1-1)
  - Unconfined concrete: $\boldsymbol{\varepsilon_{cu} = -0.0035}$ (EN1992-1-1)
  - Confined concrete:

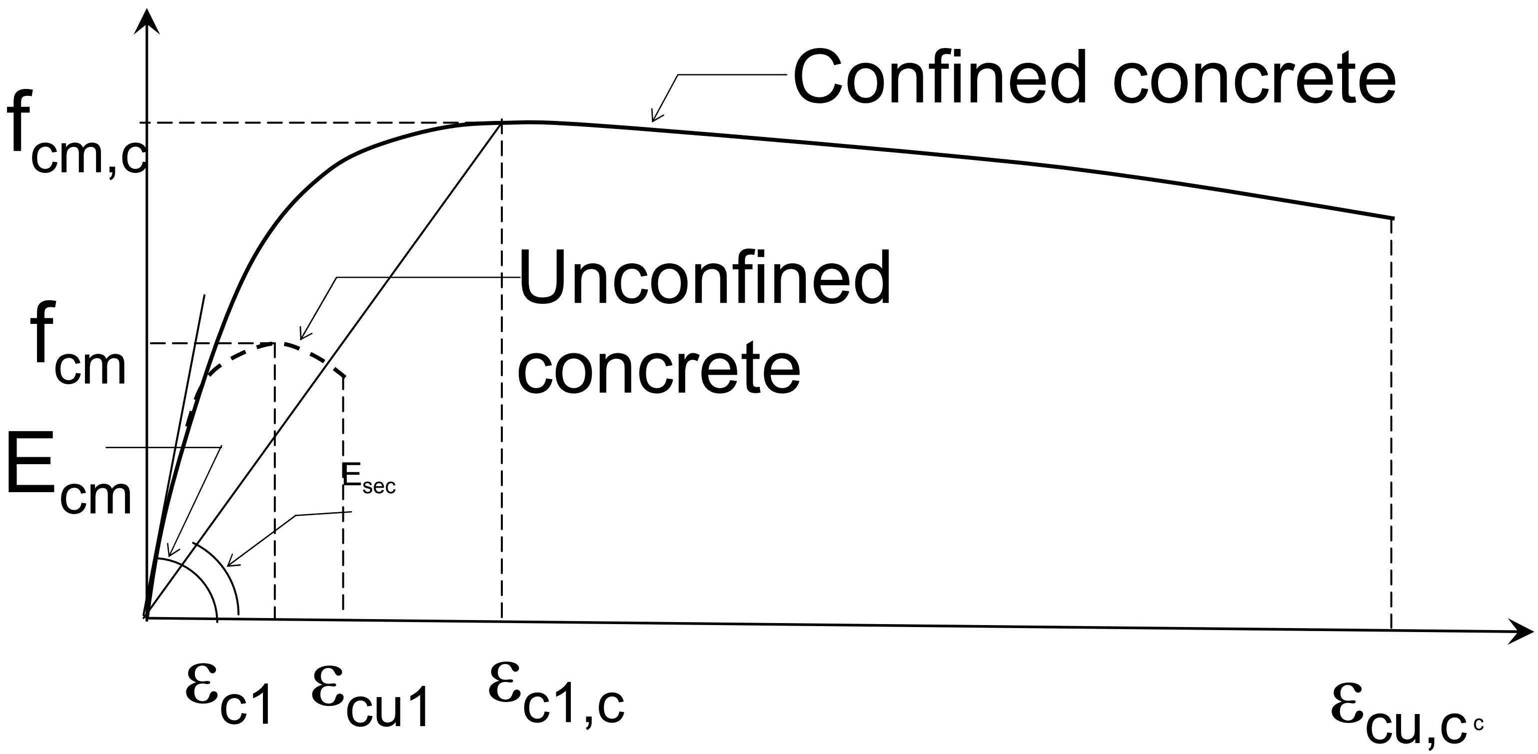
$$\varepsilon_{cu,c} = -0.004 - \frac{1.4 \rho_s f_{ym} \varepsilon_{su}}{f_{cm,c}}$$

Deformation Capacity of Piers

- **Mean material properties**
  - Reinforcement
    - $f_{ym}/f_{yk} = 1.15$, $f_{sm} / f_{sk} = 1.20$, $\varepsilon_{su} = \varepsilon_{uk}$
  - Concrete
    - $f_{cm} = f_{ck} + 8$ (MPa), $E_{cm} = 22(f_{cm}/10)^{0.3}$
  - Stress-strain diagram of concrete
    - Unconfined concrete: $\varepsilon_{c1} = -0.0007 f_{cm}^{0.31}$

Deformation Capacity of Piers

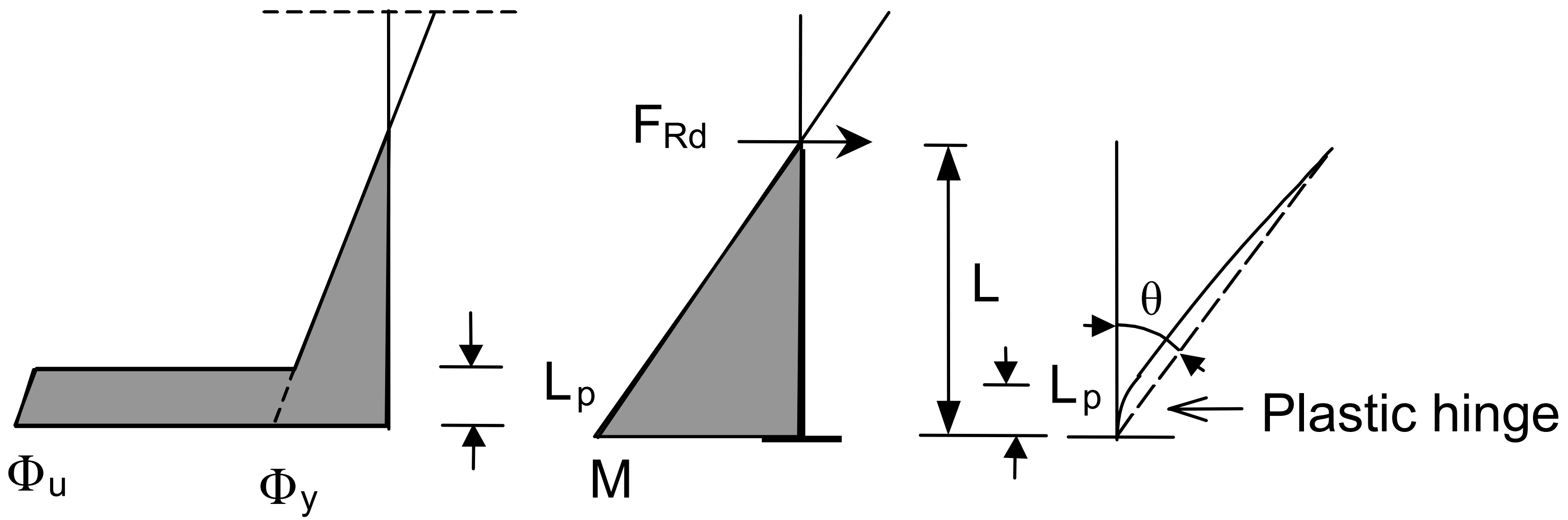
Confined concrete - Mander model



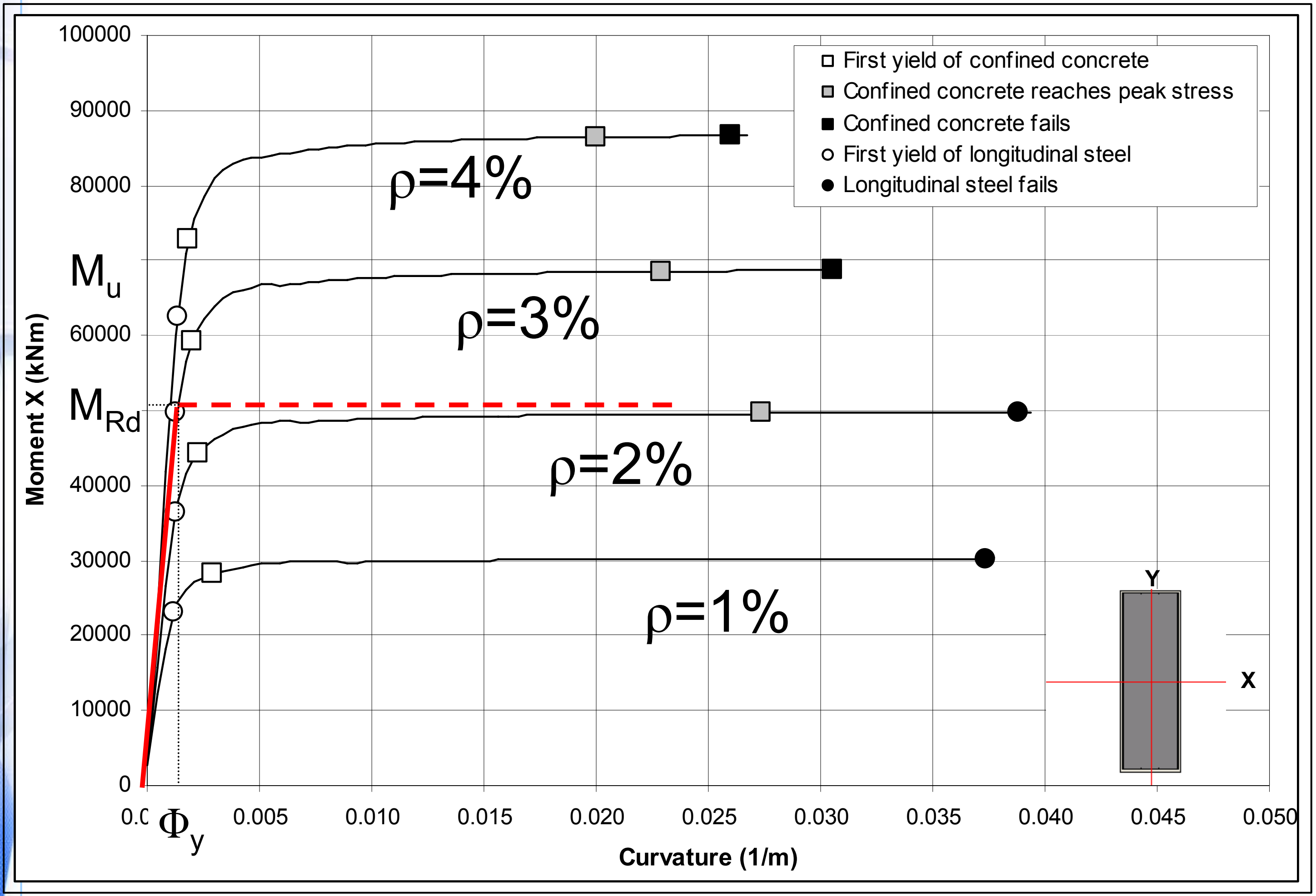
Deformation Capacity of Piers

Chord rotation: $\theta_u = \theta_y + \theta_{p,u}$

$$\theta_{p,u} = (\Phi_u - \Phi_y) L_p \left(1 - \frac{L_p}{2L}\right)$$



Deformation Capacity of Piers



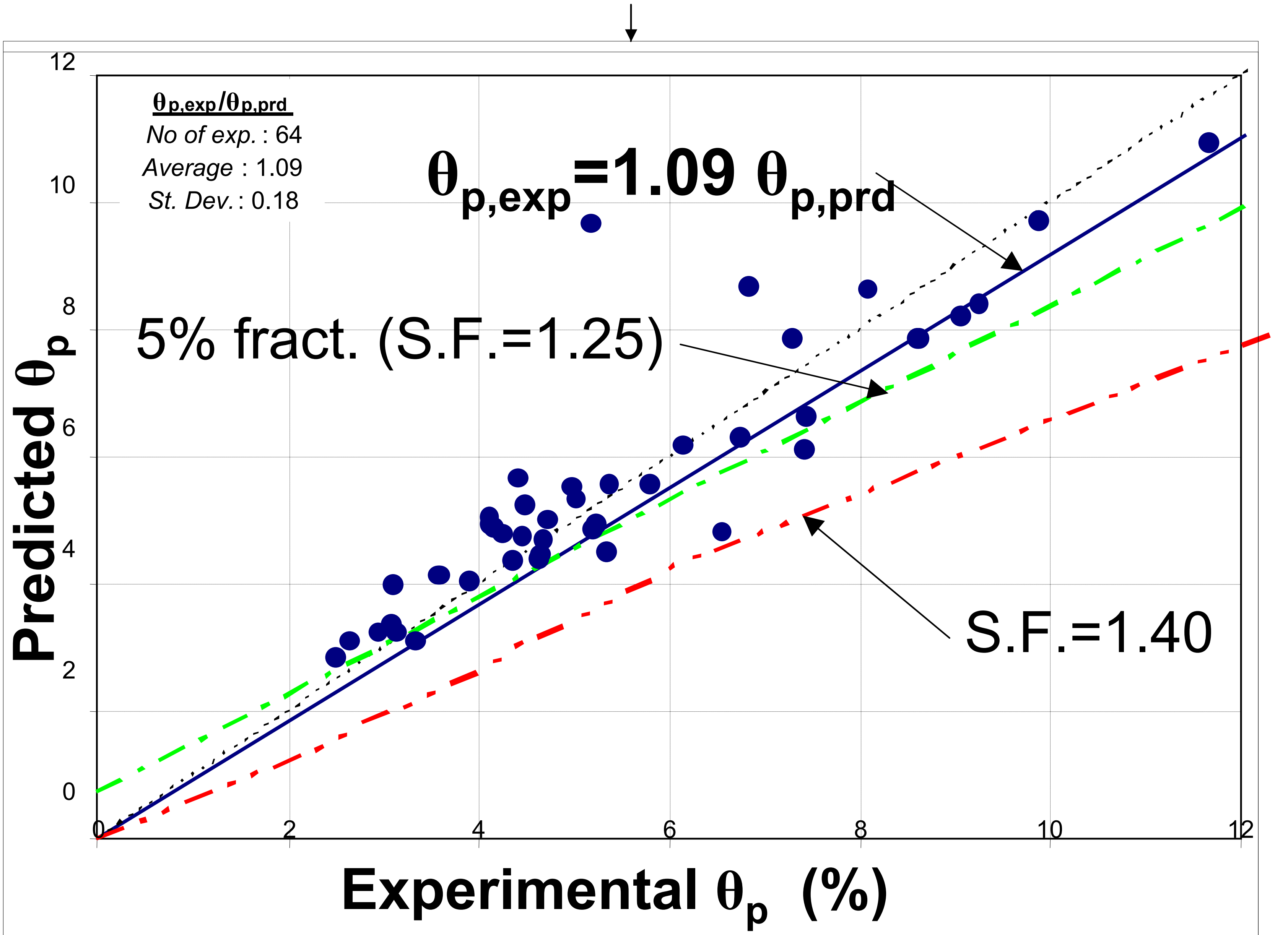
$$\Phi_y = 2.1\frac{\varepsilon_{sy}}{d}$$

Deformation Capacity of Piers

Calibration with test results

- Database:
  - 64 tests on R/C pier elements.
    - 31 circular, 25 rectangular, 8 box sections
- Curvature analysis for each test specimen
- Non-linear regression for the coefficients of:

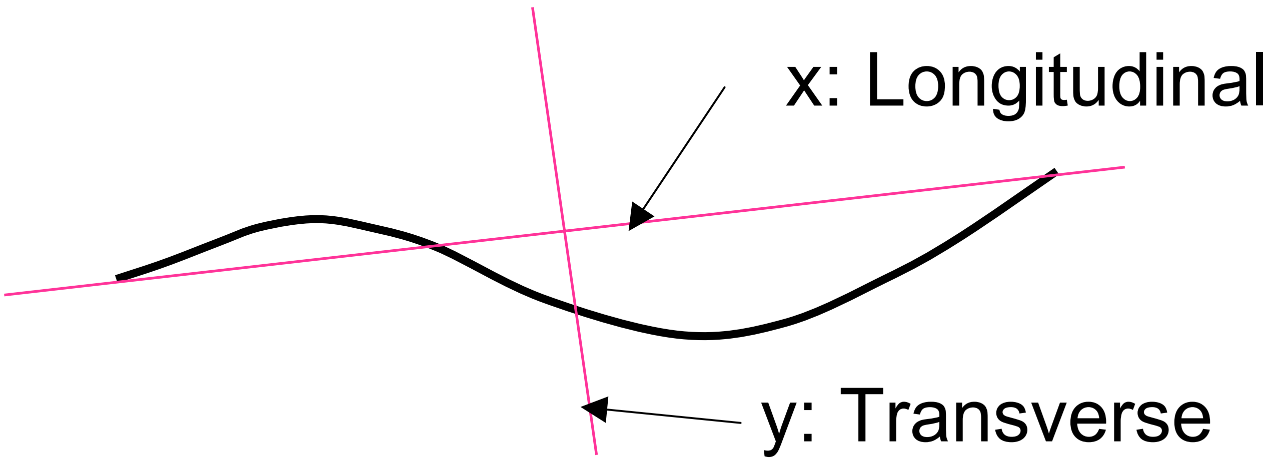
$$\mathbf{L_p = 0.10L + 0.015f_{yk}d_s}$$



θp,exp/θp,prd
No of exp. : 64
Average : 1.09
St. Dev. : 0.18
θp,exp=1.09 θp,prd
5% fract. (S.F.=1.25)
S.F.=1.40
Predicted θp
Experimental θp (%)
0
2
4
6
8
10
12

Non-linear Static Analysis

- **Based on the equal displacements rule**
- **Analysis directions:**



Non-linear Static Analysis

- Horizontal load increased until the displacement at the reference point reaches the design seismic displacement of elastic response spectrum analysis (q = 1), for

  Ex + 0.3Ey and Ey + 0.3Ex

- Reference point is the centre of mass of the **deformed** deck

Non-linear Static Analysis

- **Load distribution**

  Load increment at point ***i*** at step ***j***

$$\Delta F_{i,j} = \Delta\alpha_j G_i \zeta_i$$

  - distribution constant along the deck: $\zeta_i = 1$
  - distribution proportional to first mode shape

Thank you !!!