EUROCODE 8 – PART 2. SEISMIC DESIGN OF BRIDGES

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1. INTRODUCTION – SUMMARY

In this presentation, sections 2 to 7 give a comprehensive summary of the main provisions of the corresponding sections of EC8-2. Section 8 gives information on the criteria used by the standard for two key issues; the deformation capacity of ductile piers and the design of irregular bridges. Section 9 gives the revised provisions concerning the lateral restoring capability of seismic isolated bridges.
Certain general provisions contained in Part 1 of EC-8 (mainly concerning seismic actions) have been included in this presentation, so as to enhance readability by reducing cross-references.

2. BASIC REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Basic requirements

EC8-2 has following basic requirements

- Non – collapse requirement
  After the occurrence of the design seismic event, the bridge should retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur. The bridge should be damage-tolerant i.e. those parts of the bridges susceptible to damage, by their contribution to energy dissipation during the design seismic event, should be designed in such a manner as to ensure that, following the seismic event, the structure can sustain the actions from emergency traffic, and inspections and repair can be performed easily.
- Minimisation of damage
  Only secondary components and those parts of the bridge intended to contribute to energy dissipation during the design life of the bridge should incur minor damage during earthquakes with a high probability of occurrence.

The non-collapse requirement for bridges under the design seismic event is more stringent than the relevant requirement for buildings, as it contains the continuation of emergency traffic.

2.2 Seismic behaviour of structures

Two classes of intended seismic behaviour are foreseen for bridges.

- *ductile behaviour*, corresponding to values of the behaviour factor $1.50<q\leq 3.5$ (see 4.1)
- *limited ductile behaviour,* corresponding to q-values ≤ 1.50.

2.3 Compliance criteria for linear analysis

In general the compliance criteria given below aim explicitly at satisfying the non-collapse requirement. In conjunction with certain specific detailing rules, the same criteria are deemed to cover implicitly the damage minimization requirement as well.

- Resistance verifications
  - In bridges of ductile behaviour, the regions of plastic hinges are verified to have adequate flexural strength to resist the design seismic effects A_{Ed}. The shear resistance of the plastic hinges as well as both the shear and flexural resistances of all other regions, are designed to resist the "*capacity design effects*", determined in accordance with 5.1.
  - In bridges of limited ductile behaviour, all sections are verified to have adequate strength to resist the design seismic effects A_{Ed}. However, verification of non-ductile failure modes is carried out for action effects qA_{Ed} and resistances of numbers are divided by an additional safety factor γ_{Bd}, with recommended value ranging from 1.25 to 1.00
- Ductility verifications
  Conformance to special detailing values (see 6) is deemed to ensure adequate local and global ductility.

- Control of displacements
  The linear analysis and the resulting displacements are based on stiffness of the ductile members equal to their secant stiffness at the theoretical yield point.
  The design seismic displacements d_E are derived from the displacements d_{Ee}, determined from linear seismic analysis, as follows:
  $d_E = \eta\mu_d d_{Ee}$ (1)
  $\eta = \sqrt{0.10/(0.05+\xi)}$ (2)
  is the damping correction factor corresponding to the viscous damping ratio ξ (≠ 0.05) of the structure.
  The displacement ductility μ_d is assumed as follows:
  - when the fundamental period T in the direction under examination is $T \geq T_o = 1.25T_C$ where T_C is the spectrum period defining the limit between the constant acceleration and constant velocity regions of the acceleration spectrum (see 3.2)
    $\mu_d = q$ (3)
  - when $T<T_o$, then
    $$\mu_d = (q-1)\frac{T_o}{T} + 1 \leq 5q-4 \quad (4)$$
    q is the value of the behaviour factor assumed in the analysis for d_{Ee}

  Adequate clearances should be provided for protection of critical or major structural members. Such clearances shall accommodate the total design value of the displacement under seismic conditions d_{Ed} determined as follows:
  $d_{Ed} = d_E + d_G + 0.50d_T$ (5)
  where, following displacements are combined with the most onerous sign:
  d_E is the design seismic displacement in accordance with Eq. (1)
  d_G is the displacement due to the permanent and quasi-permanent actions measured in long term (including e.g. post-tensioning, shrinkage and creep for concrete decks).
  d_T is the displacement due to thermal movements.

The detailing of non-critical structural components (e.g. deck movement joints) and abutment back-walls, expected to be damaged during the design seismic event, should cater, for a predictable mode of damage and provide for the possibility of permanent repair. Clearances should accommodate appropriate fractions of the design seismic displacement and thermal movement, after allowing for any long term creep and shrinkage effects, so that damage under frequent earthquakes can be avoided. Recommended values are 40% of the design seismic d_E displacement and 50% of the thermal displacement d_T.

2.4 Compliance criteria for non-linear analysis

- Ductile members
  The verification that deformation demands are safely lower than the capacities of the plastic hinges, is performed in terms of plastic hinge rotation demands $\theta_{p,E}$, by comparison to relevant design rotation capacities $\theta_{p,d}$, as follows:
  $\theta_{p,E,} \leq \theta_{p,d}$ (6)
  Design plastic rotation capacity $\theta_{p,d}$, is derived from relevant test results or calculated from ultimate curvatures, by dividing the probable value $\theta_{p,u}$ by factor $\gamma_{R,p}$, (recommended value = 1.40) reflecting local defects of the structure, uncertainties of the model and/or the dispersion of relevant test results, as follows:
  $$\theta_{p,d} = \frac{\theta_{p,u}}{\gamma_{R,p}} \quad (7)$$
- Non-ductile members
  Verification of all members for non-ductile failure modes (shear of members and shear of joints adjacent to plastic hinges) as well as of foundation failure, is performed, in accordance with the relevant rules for linear analysis, assuming as design actions (in lieu of the capacity design effects), the maximum values of the responses of the ensemble of the analyses for the ground motions used

(maxA_{Ed}). These values should not exceed the design resistances R_d (= R_k/γ_M) of the corresponding sections.

3. SEISMIC ACTION

3.1 Soil classes (EC8-1)

Soil classes A, B, C and D in accordance with EC8-1, have the following correspondence to the relevant classes of NEHRP 2000 (FEMA-368).

EC8-1	A	B	C	D
NEHRP 2000	B (and A)	C	D	E

Soil class E consists of alluvium 5 to 20m thick underlain by stiffer material ($v_s \geq 800$ m/s)

3.2 Horizontal elastic response spectrum (EC8-1)

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Fig. 1):

$$0 \leq T \leq T_B : S_e(T) = \alpha_g S\left[1 + \frac{T}{T_B}(\eta 2.5 - 1)\right] \tag{8}$$

$$T_B \leq T \leq T_C : S_e(T) = \alpha_g \eta S 2.5 \tag{9}$$

$$T_C \leq T \leq T_D : S_e(T) = \alpha_g \eta S 2.5\left[\frac{T_C}{T}\right] \tag{10}$$

$$T_D \leq T \leq 4\text{sec} : S_e(T) = \alpha_g \eta S 2.5\left[\frac{T_C T_D}{T^2}\right] \tag{11}$$

S_e (T) ordinate of the elastic response spectrum,
T vibration period of a linear single-degree-of-freedom system,
α_g design ground acceleration
T_B, T_C limits of the constant spectral acceleration branch,
T_D value defining the beginning of the constant displacement response range of the spectrum,
S soil factor,
η damping correction factor (Eq. (2))

Table 1: Recommended values of the parameters for types 1 & 2 elastic response spectra

Case	*S*		T_B (s)		T_C (s)		T_D (s)		a_{vg}/a_g	
Spectrum type	1	2	1	2	1	2	1	2	1	2
Soil class A	1.0	1.00	0.15	0.05	0.4	0.25	2.0	1.2		
Soil class B	1.20	1.35	0.15	0.05	0.5	0.25	2.0	1.2		
Soil class C	1.15	1.50	0.20	0.10	0.6	0.25	2.0	1.2		
Soil class D	1.35	1.80	0.20	0.10	0.8	0.30	2.0	1.2		
Soil class E	1.40	1.60	0.15	0.05	0.5	0.25	2.0	1.2		
Vertical Spectrum	1.00	1.00	0.05	0.05	0.15	0.15	1.0	1.0	0.90	0.45

Two types of response spectra are defined. Type 2 spectrum is recommended only for regions where the design earthquake has a surface has a surface wave magnitude Ms ≤ 5.5.
The elastic displacement response spectrum, $DS_e(T)$, is obtained by direct transformation of the elastic acceleration spectrum, $S_e(T)$, using the following expression:

$$DS_e(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{12}$$

3.3 Vertical elastic spectrum (EC8-1)

Is defined by Eqs. (8) to (11) by replacing the numerical coefficient 2.5 by 3, the design ground acceleration α_g by α_{vrg} and using S=1.0 and the values of α_{rg}, T_B, T_C and T_D given in Table 1.

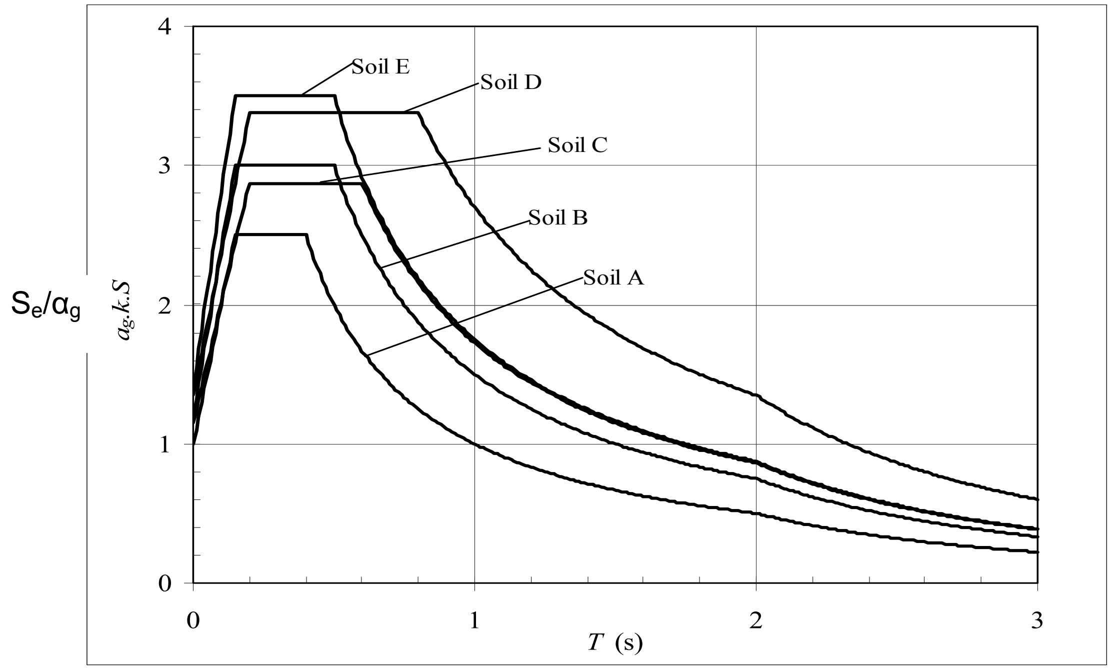

Fig. 1, Recommended elastic response spectrum, Type 1.

3.4 Design spectrum for elastic analysis (EC8-1)

The horizontal components are defined by Eqs. (9) to (11) by replacing the damping correction factor η by the inverse of the behaviour factor q (i.e. using η=1/q). For the very short period range following equation replaces Eq. (8).

$$0 \le T \le T_B : S_d(T) = \alpha_g S\left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{13}$$

3.5 Importance classes

Differentiation of target reliability may be effected by means of importance factors γ_I as $A_{Ed} = \gamma_I A_{Ek}$
A_{Ed} is the design seismic action and A_{Ek} is the characteristic seismic action (usually corresponding to a return period of 475 years). The recommended importance classes and corresponding factors as shown in Table 2.

Table 2: Bridge Importance Classes

Importance Class		γ_I
Greater than average	III	1.30
Average	II	1.00
Less than average	I	0.85

3.6 Spatial variability of seismic action

The model used for spatial variability should account for the propagation and the progressive loss of correlation of the seismic waves as well as to the eventual modification of the frequency content of the motions due to change of the mechanical properties of the soil. Informative Annex D of EC8-2 gives guidance for using a rigorous stochastic model of spatial variability.

The main text of the standard offers the possibility of using a simple approximative model based on pseudostatic effects of two sets of imposed displacements of the supports that are applied separately in each horizontal direction, and their effects need not be combined. These sets are defined on the basis of the maximum ground displacement d_g = 0.025 a_g ST_CT_D as defined by EC8-1 and a characteristic distance L_g, beyond which the ground motions may be considered as completely uncorrelated. Recommended values of L_g are given by the following Table 3.

Table 3: Distance beyond which ground motions may be considered uncorrelated

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

Accounting for spatial variability is required for bridges with continuous deck when either the bridge length exceeds $L_g/1.5$ or when more than one ground type correspond to the bridge supports.
Displacement set A (see Fig. 2a) consists of relative displacements d_{ri} applied simultaneously with the same sign to all supports of the bridge (i=1 to n) in the considered horizontal direction, as follows:

$$d_{ri} = \varepsilon_r L_i \le d_g \sqrt{2} \tag{14}$$

$$\varepsilon_r = \frac{d_g \sqrt{2}}{L_g} \tag{15}$$

L_i is the distance (projection on the horizontal plane) of support i from a reference support i = 0, that may conveniently selected at one of the end supports.
Displacement set B (see Fig. 2b) covers the influence of ground displacements occurring in opposite directions at adjacent piers. It consists of the following configuration of imposed absolute displacements d_i, d_{i+1} with opposed sign at adjacent supports i and i+1 respectively, for i=0 to n-1:

$$d_i = \pm \Delta d_i / 2 \tag{16}$$

$$d_{i+1} = \pm \Delta d_{i+1} / 2 \tag{17}$$

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

$L_{av,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ of intermediate support i to its adjacent supports i-1 and i+1, and β_r is a factor accounting for the magnitude of ground displacements occurring in opposite directions at adjacent supports. Recommended values of β_r are 0.5 when all three supports i-1, i, and i+1 have the same ground type, and 1.0 otherwise.

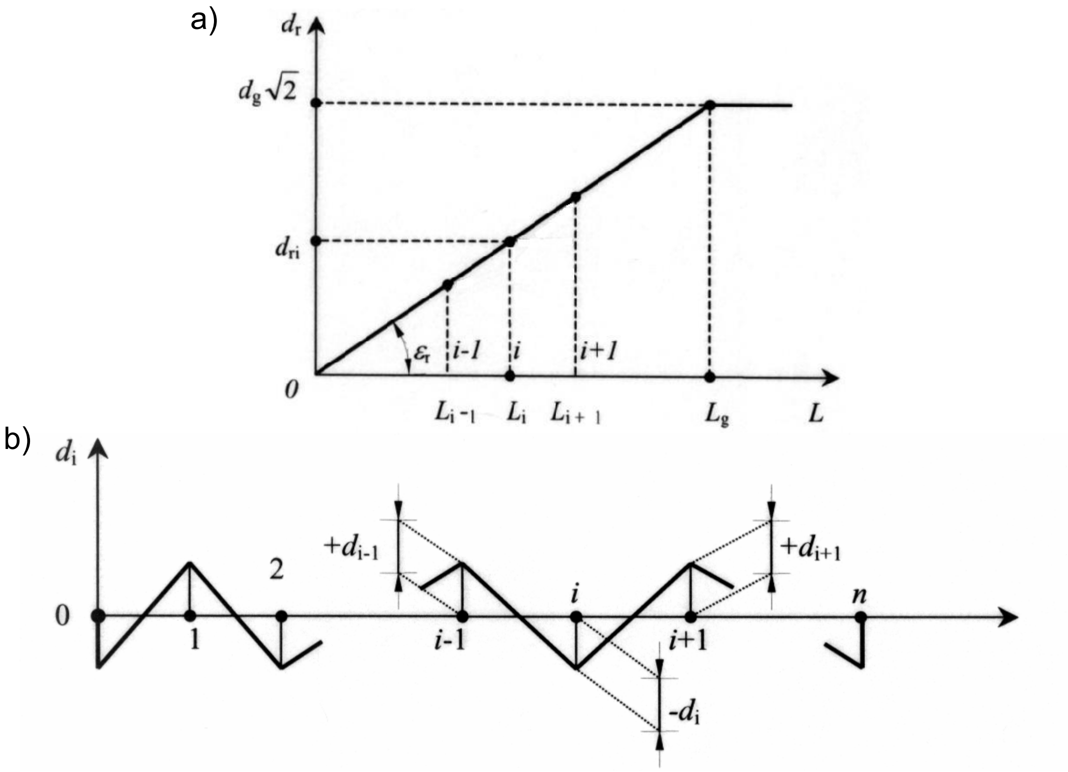

Fig 2, a) displacement set A, b) displacement set B

3.7 Time history representation
At least 3 pairs of horizontal ground motion time-histories should be used. Rules are given for scaling of the pairs of horizontal motions independent from the vertical component, so as to render them compatible to the elastic response spectrum.

4. ANALYSIS METHODS

4.1 Linear analysis with behaviour factor
The linear analysis using a global force reduction factor (behaviour factor q) is the normal analysis method. Response spectrum analysis may be applied in all cases, while equivalent static analysis with various simplifications is permitted under certain conditions. Table 4 gives the maximum values of the behaviour factor q.
For reinforced concrete ductile members the values of *q*-factors specified in Table 4 are applicable when the normalised axial force η_k does not exceed 0.30. When $0.30 < \eta_k \leq 0.60$, even in a single ductile member, the value of the behaviour factor should be reduced to:

$$q_r = q - \frac{\eta_k - 0.3}{0.3}(q-1) \geq 1.0 \tag{18}$$

$$\eta_k = N_{Ed}/(A_c f_{ck}) \tag{19}$$

N_{Ed} is the value of the axial force at the plastic hinge corresponding to the design seismic combination, positive if compressive, A_c is the area of the section and f_{ck} is the characteristic concrete strength.
The values of the q-factor for ductile behaviour specified in Table 4, may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, these values are multiplied by 0,6; however final q-values less than 1.0 need not be used.
When the main part of the design seismic action is resisted by elastomeric bearings the flexibility of the bearings imposes a practically elastic behaviour of the system. Such bridges are designed in accordance with the rules of seismic isolation (Section 7).
The inertial response of bridge structures whose mass follows essentially the horizontal seismic motion of the ground ("locked-in" structures), may be assessed using the design value of the seismic ground acceleration and q = 1. Abutments flexibly connected to the deck belong to this category.

Table 4: Maximum values of the behaviour factor q

Type of ductile members	Seismic Behaviour	
	Limited ductile	Ductile
Reinforced concrete piers:		
Vertical piers in bending ($\alpha_s^* \geq 3,0$)	1.5	3.5 $\lambda(\alpha_s)$
Inclined struts in bending	1.2	2.1 $\lambda(\alpha_s)$
Steel Piers:		
Vertical piers in bending	1.5	3.5
Inclined struts in bending	1.2	2.0
Piers with normal bracing	1.5	2.5
Piers with eccentric bracing	-	3.5
Abutments rigidly connected to the deck:		
In general	1.5	1.5
Locked in structures (par. (9), (10))	1.0	1.0
Arches	1.2	2.0

α_s = L/h is the shear ratio of the pier, where L is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross section in the direction of flexure of the plastic hinge.

For $\alpha_s \geq 3$ $\lambda(\alpha_s) = 1.0$, and for $3 > \alpha_s \geq 1.0$ $\lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}}$

4.2 Regular and irregular seismic behaviour of ductile bridges
Designating by $M_{Ed,i}$ the maximum value of design moment under the seismic load combinations at the intended location of plastic hinge of ductile member i, and by $M_{Rd,i}$ the design flexural resistance of the same section, with its actual reinforcement, under the concurrent action of the other action effects of the seismic load combination (Eq.(28)), then the required local force reduction factor r_i associated with member i, under the specific seismic action is:

$$r_i = qM_{Ed,i} / M_{Rd,I} \qquad (20)$$

A bridge is considered to have a regular seismic behaviour, in the direction under consideration, when following condition is satisfied

$$\rho_r = r_{max}/r_{min} \leq \rho_o \qquad (21)$$

r_{min} = minimum (r_i) and r_{max} = maximum (r_i), for all ductile members i
ρ_o = 2.0, is a limit value selected so as to ensure that sequential yielding of the ductile members shall not cause unacceptably high ductility demands on one member.
One or more ductile members (piers) may be exempted from the above calculation of r_{min} and r_{max}, if the sum of their shear contributions does not exceed 20% of the total seismic shear in the direction under consideration.
Bridges not meeting condition (21), shall be considered to have irregular seismic behaviour, in the direction under consideration. Such bridges should either be designed using a reduced q-value:

$$q_r = q\rho_o/\rho_r \geq 1.0 \qquad (22)$$

or should be designed based on results of non-linear analysis in accordance with 4.4 or 4.5.

4.3 Combination of modal responses and of the components of seismic action
Either the SRSS or the complete CQC modal combination rules are applicable.
The design seismic action effects A_{Ed} should be derived from the most adverse of the following combinations:

$$A_{Ex} \text{ "+" } 0{,}30A_{Ey} \text{ "+" } 0{,}30\,A_{Ez}$$
$$0{,}30A_{Ex} \text{ "+" } A_{Ey} \text{ "+" } 0{,}30A_{Ez} \qquad (23)$$
$$0{,}30A_{Ex} \text{ "+" } 0{,}30A_{Ey} \text{ "+" } A_{Ez}$$

A_{Ex}, A_{Ey} and A_{Ez} are the seismic actions in each direction X, Y and Z respectively and "+" implies "to be combined with".

4.4 Non - linear dynamic time-history analysis
In general, this method is used in combination with a normal response spectrum analysis to provide insight into the post - elastic response and comparison between required and available local ductilities. Generally, the results of the non-linear analysis are not intended to be used to relax requirements resulting from the response spectrum analysis. However, in the case of bridges with isolating devices and irregular bridges (4.2), lower results from a rigorous time-history analysis may be substituted for the results of the response spectrum analysis.

4.5 Static non-linear analysis (pushover analysis)
Pushover analysis is a static non-linear analysis of the structure under constant vertical (gravity) loads and monotonically increased horizontal loads, representing the effect of an horizontal seismic component. Second order effects should be accounted for. The horizontal loads are increased until the *target displacement* is reached at the *reference point*. This analysis should be used (alternatively to non - linear dynamic time-history analysis) in the case of irregular bridges.

- *Analysis directions, target displacements and reference point*
  The analysis should be carried out in the following two horizontal directions;the longitudinal direction x, as defined by the centres of the two end-sections of the deck and the transverse direction y, that should be assumed at right angles to the longitudinal direction.
  The *target displacement* is the maximum of the displacements in the relevant direction, at the centre of mass of the deformed deck, resulting from equivalent linear multi-mode spectrum analysis, assuming q = 1.0, for the following combinations of seismic components: E_x "+" $0.3E_y$ and E_y "+" $0.3E_x$. The spectrum analysis should be carried out using effective stiffness of ductile members as specified in 2.2
  The *reference point* should be the centre of mass of the deformed deck.

- *Load distribution*
  The horizontal load increments $\Delta F_{i,j}$ assumed acting on lumped mass G_i/g, in the direction investigated, at each loading step j, are taken equal to:
  $\Delta F_{i,j} = \Delta\alpha_j G_i \zeta_i$ (24)
  $\Delta\alpha_j$ is the horizontal load increment, normalized to the weight G_i, applied in step j, and
  ζ_i is a shape factor defining the load distribution along the structure, as follows
  a) *constant along the deck*, where
     for the deck
     $\zeta_i = 1$ (25)
     and for the piers connected to the deck
     $\zeta_i = z_i / z_P$ (26)
     z_i is the height of point i above the foundation of the individual pier and
     z_P is the height of the pier P (distance from the ground to the centreline of the deck)
  b) *proportional to the first mode shape*, where
     ζ_i is proportional to the component, in the direction investigated, of the modal displacement at point i, of the first mode, in the same direction. The mode having the largest participation factor in the direction under investigation should be considered as first mode in this direction.

5. DESIGN OF MEMBERS

5.1 Capacity design effects

For structures of ductile behaviour, capacity design effects F_C (V_C, M_C, N_C) are calculated by analysing the intended plastic mechanism under the permanent actions and the level of seismic action at which all intended flexural hinges have developed bending moments equal to an appropriate upper fractile of their flexural resistance, called the overstrength moment M_o. This calculation should be carried out on the basis of equilibrium conditions, while reasonable approximations regarding the compatibility of deformations are acceptable.
The capacity design effects need not be taken greater than those resulting from the design seismic combination (see 5.2) where the design effects A_{Ed} are multiplied by the q factor used.
The overstrength moment of a section is calculated as:
$M_o = \gamma_o M_{Rd}$ (27)
γ_o is the overstrength factor
M_{Rd} is the design flexural strength of the section, in the selected direction and sense, based on the actual section geometry, including reinforcement where relevant, and material properties (with γ_M values for fundamental load combinations). In determining M_{Rd}, biaxial bending under the permanent effects, and the seismic effects corresponding to the design seismic action in the selected direction and sense, shall be considered.
The value of the overstrength factor should reflect the probable deviation of material strength, and strain hardening. Recommended values are:
Concrete members: $\gamma_o = 1.35(1+2(\eta_k-0,1)^2)$ for confined sections with $\eta_k > 0.1$
$\gamma_o = 1.35$ for other concrete members
Steel members: $\gamma_o = 1.25$
Within members containing plastic hinge(s), the capacity design bending moment M_c at the vicinity of the hinge (Fig. 3) shall not be assumed greater than the relevant design flexural resistance M_{Rd} of the hinge assessed.

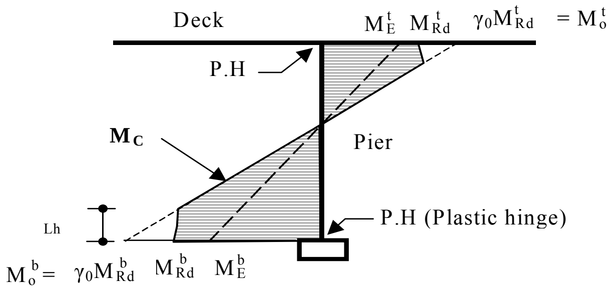

Fig 3, Capacity design moments M_C within member containing plastic hinges

5.2 Design seismic combination

The design value of action effects E_d, in the seismic design situation, are derived from the following combination of actions:

G_k "+" P_k "+" A_{Ed} "+" $\psi_{21}Q_{1k}$ "+" Q_2 (28)

"+" implies "to be combined with", G_k are the permanent loads with their characteristic values, P_k is the characteristic value of prestressing after all losses, A_{Ed} is the most unfavourable combination of the components of the earthquake action in accordance with Eq. 23, Q_{1k} is the characteristic value of the traffic load, and ψ_{21} is the combination factor with recommended values ψ_{21}=0 in general, ψ_{21} = 0.2 for road bridges with intense traffic and ψ_{21} = 0.3 for railway bridges.

Q_2 is the quasi permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.) Actions of long duration are considered to be concurrent with the design earthquake.

Seismic action effects need not be combined with action effects due to imposed deformations (temperature variation, shrinkage, settlements of supports, ground residual movements due to seismic faulting)

5.3 Member verification

The basic rules for member verification are given for linear analysis in 2.3 (Resistance verifications) and for non-linear analysis in 2.4

5.4 Verification of joints adjacent to plastic hinges

Joints between vertical ductile piers and deck or foundation members, adjacent to a plastic hinge, are designed in shear to resist the capacity design effects of the plastic joint in the relevant direction. Detailed rules and alternative reinforcement arrangements are given.

5.5 Deck verification

It should be verified that no significant yield occurs in the deck. This verification is carried out for bridges of limited ductile behaviour, under the most onerous design seismic combination in accordance with 5.2., and for bridges of ductile behaviour, under the capacity design effects determined in accordance with 5.1.

In analysis in the transverse direction, yielding of the deck about the vertical axis is considered to be significant when it reaches the reinforcement of the top slab of the deck at a distance from its edge equal to 1/10 of top slab width or at the junction with a web if it is closer to the edge. In this analysis, the significant reduction of the torsional stiffness of the deck with increasing torsional moments, should be accounted for.

5.6 Foundations

Bridge foundation systems are designed to comply with the general requirements set forth in EN 1998-5. In general it is not allowed that bridge foundations are intentionally used as sources of hysteretic energy dissipation and therefore should, as far as practicable, be designed to remain undamaged under the design seismic action.

6. DETAILING

6.1 Confinement

In potential hinge regions where the normalised axial force exceeds the limit: $\eta_k = N_{Ed}/A_c f_{ck} > 0{,}08$, confinement of the compression zone is in general necessary. No confinement is required in piers with flanged sections (box- or I-Section) if, under ultimate seismic load conditions, a curvature ductility $\mu_\Phi = 13$ for bridges of ductile behaviour, or $\mu_\Phi = 7$ for bridges of limited ductile behaviour, is attainable with the maximum compressive strain in the concrete not exceeding the value of $\varepsilon_{cu} = 0{,}35\%$. In cases of deep compression zones, the confinement may be limited to that depth in which the compressive strain exceeds $0{,}5\varepsilon_{cu}$ The quantity of confining reinforcement is defined by the mechanical reinforcement ratio:

$$\omega_{wd} = \rho_w . f_{yd}/f_{cd} \tag{29}$$

ρ_w is the transverse reinforcement ratio equal to $\rho_w = A_{sw}/s_L b$ for rectangular sections and $\rho_w = 4A_{sp}/D_{sp}.s_L$ for circular sections.

The minimum amount of confining reinforcement shall be determined as follows:

a) for rectangular hoops and cross-ties,in each direction

$$\omega_{wd,r} \geq \max\left(\omega_{w,req}, \frac{2}{3}\omega_{w,min}\right) \tag{30}$$

$$\omega_{w,req} = \frac{A_c}{A_{cc}}\lambda\eta_k + 0{,}13\frac{f_{yd}}{f_{cd}}(\rho_L - 0{,}01) \tag{31}$$

A_c is the gross concrete area of the section, A_{cc} is the confined (core) concrete area of the section, λ factor specified in Table 5 and ρ_L is the reinforcement ratio of the longitudinal reinforcement

Table 5: Minimum values of λ and $\omega_{w,min}$

Seismic Behaviour	λ	$\omega_{w,min}$
Ductile	0,37	0,18
Limited ductile	0,28	0,12

b) for circular hoops or spirals

$$\omega_{wd.c} \geq \max(1.4_{w,req}, \omega_{w,min}) \tag{32}$$

Interlocking spirals/hoops are quite efficient for confining approximately rectangular sections. The distance between the centres of interlocking spirals/hoops shall not exceed $0{,}6D_{sp}$, where D_{sp} is the diameter of the spiral/hoop.

6.2 Buckling of longitudinal compression reinforcement

Buckling of longitudinal reinforcement shall be avoided along potential hinge areas even after several cycles into the plastic region. Therefore all main longitudinal bars shall be restrained against outward buckling by transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars at a maximum (longitudinal) spacing $s_L = \delta\varphi_L$, where φ_L is the diameter of the longitudinal bars. Coefficient δ depends on the ratio f_t/f_y of the tensile strength f_t to the yield strength f_y of the transverse reinforcement, in terms of characteristic values, in accordance with the following relation:

$$5 \leq \delta = 2{,}5\,(f_t / f_y) + 2{,}25 \leq 6 \tag{33}$$

Along straight section boundaries, restraining of longitudinal bars should be effected in either of the following ways:

a) Through a perimeter tie engaged by intermediate cross-ties at alternate locations of longitudinal bars, at transverse (horizontal) spacing s_t not exceeding 200 mm. The cross- ties shall have 135^o-hooks at one end and 90^o-hook at the other. The hooks shall be alternated in both horizontal and vertical directions. In sections of large dimensions the perimeter tie may be spliced using appropriate overlap length combined with hooks. When $\eta_k > 0{,}30$, 90^o-hooks are not allowed for the cross-ties. In this case it is allowed to use lapped cross-ties with 135^o-hooks.

b) Through overlapping closed ties arranged so that every corner bar and at least every alternate internal longitudinal bar in engaged by a tie leg. The transverse (horizontal) spacing s_T of the tie legs should not exceed 200 mm.

The minimum amount of transverse ties shall be determined as follows:

$$A_t / s_T = \Sigma A_s f_{ys} / 1{,}6 f_{yt} \quad (mm^2/m), \tag{34}$$

A_t is the area of one tie leg, in mm^2, s_T is the transverse distance between tie legs, in m, ΣA_s is the sum of the areas of the longitudinal bars restrained by the tie in mm^2, f_{yt} is the yield strength of the tie, and f_{ys} is the yield strength of the longitudinal reinforcement.

6.3 Other rules for reinforcement detailing

Due to the potential loss of concrete cover in the plastic hinge region, the anchorage of the confining reinforcement shall be effected through 135^o hooks surrounding a longitudinal bar plus adequate extension (min. 10 diameters) into the core concrete.
Similar anchoring or full strength weld is required for the lapping of spirals or hoops within potential plastic hinge regions. In this case laps of successive spirals or hoops, when located along the perimeter of the member, should be displaced in accordance with 8.7.2 of EN1992-1.
No splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region. For mechanical couplers.6.3(2) of EC8-1 is applicable.

6.4 Hollow piers

Unless appropriate justification is provided, the ratio b/t of the clear width b to the thickness t of the walls, in the plastic region (length L_h of 6.2.1.4) of hollow piers having a single or multiple box cross section, should not exceed 8. For hollow cylindrical piers the above limitation is valid for the ratio D_i/t, where D_i is the inside diameter.
In simple or multiple box section piers and when the ratio $\eta_\kappa \leq 0{,}20$, there is no need for verification of confining reinforcement in accordance with 6.1, if the provisions of 6.2 are met.

6.5 Pile foundations

In the case of pile foundations, it is sometimes difficult, to avoid localized hinging in the piles. Pile integrity and ductile behaviour should be secured in such cases.
The potential hinge locations, where confinement is required, are the following

a) along 3 pile diameters, at the pile heads adjacent to the pile cap, when the rotation of pile cap about horizontal axis, transverse to the seismic action, is very small, due to large stiffness of pile group in this degree of freedom.
b) at the depth where maximum bending moments develop in the pile. This depth should be estimated by rational analysis, accounting for the effective pile flexural stiffness, the lateral soil stiffness and the rotational stiffness of the pile group at the pile cap.
c) at the interfaces of soil layers having markedly different shear deformability, due to kinematic pile-soil interaction (see 5.4.2 (1) of EN 1998-5.

For the locations b) and c), longitudinal as well as confining reinforcement of the same amount as that required at the pile head should be provided for a length of two pile diameters on each side of the point of maximum moment or interface.

6.6 Structures of limited ductile behaviour

For structures of limited ductile behaviour designed with $q \leq 1{,}5$ and located in areas of moderate to high seismicity, the following rules are applicable to the critical sections, aiming at securing a minimum of limited ductility.
A section is considered to be critical, i.e. location of a potential plastic hinge when:

$$M_{Rd} / M_{Ed} < 1{,}30 \quad (35)$$

M_{Ed} is the maximum design moment under the seismic action combinations and
M_{Rd} is the minimum flexural resistance of the section under the same combination.
As far as possible the location of potential plastic hinges should be accessible for inspection.
In concrete members, where in accordance with 6.1, confinement is necessary, confining reinforcement as required for limited ductility, shall be provided. In such a case it is also required to secure the longitudinal reinforcement against buckling, in accordance with 6.2.

6.7 Bearings and seismic links

In the absence of a monolithic deck-to-pier connection, the design seismic action shall in general be transmitted through the bearings. However, seismic links (consisting of shear keys, buffers or linkage bolts or cables) may be used to transmit the entire design seismic action provided that dynamic shock effects are mitigated and properly taken into account. These seismic links should generally allow the non-

seismic displacements of the bridge without transmitting significant loads. When seismic links are used the connection between the deck and the substructure should be properly modelled. As a minimum, a linear approximation of the force-displacement relationship of the linked structure shall be used.
The structural integrity of the bridge shall be secured under extreme seismic displacements. This requirement should be implemented at fixed supports through capacity design of the normal bearings or through provision of additional links as a second line of defence. At moveable connections either adequate overlap (seat) lengths in accordance with 6.10 shall be provided or seismic links shall be used.
All types of bearings and seismic links must be accessible for inspection and maintenance and shall be replaceable without major difficulty.

6.8 Holding-down devices
Holding down devices should be provided at all supports where the total vertical design seismic reaction opposes and exceeds a minimum percentage of the permanent load compressive reactions.

- 80% in structures of ductile behaviour where the vertical design seismic reaction is determined as a capacity design effect where the plastic hinges have developed their overstrength capacities.
- 50% in structures of limited ductile behaviour where the vertical design seismic reaction is determined from the analysis under the design seismic action only (including the contribution of the vertical seismic component).

The above requirement refers to the total vertical reaction of the deck on a support and is not applicable to individual bearings of the same support. However, no lift-off of individual bearings shall take place under the design seismic combination in accordance with 5.2.

6.9 Shock transmission units (STU)
Shock transmission units (STU) are devices that provide velocity-dependent restraint of the relative displacement between deck and supporting For low velocity movements ($v < v_1$), such as those due to temperature effects or creep and shrinkage of the deck, the movement is practically free (with very low reaction). For high velocity movements ($v > v_2$), such as those due to seismic or braking actions, the movement is blocked and the device acts practically as rigid connection. Shock transmission units may be provided with a force limiting function.
When STUs with force limiting function are used to resist seismic forces they shall have a design resistance F_{Rd} not less than:

- The reaction corresponding to the capacity design effects, in the case of ductile bridges
- The design seismic reaction multiplied by the q-factor used, in the case of limited ductile bridges.

The devices shall provide sufficient displacement capability for all slow velocity actions and full force capacity at their displaced status.
When STUs without force limiting function are used in bridges subject to seismic design situations, the devices shall provide sufficient displacement capability to accommodate the total design value of the relative displacement d_{Ed}
All STUs shall be accessible for inspection and maintenance/replacement.

6.10 Minimum overlap lengths
At supports where relative displacement between supported and supporting members is intended under seismic conditions, a minimum overlap length shall be provided. This overlap length shall be such as to ensure that the function of the support is maintained under extreme seismic displacements.
At an end support on an abutment and in the absence of a more accurate estimation the minimum overlap length l_{ov} may be estimated as follows:

$$l_{ov} = l_m + d_{eg} + d_{es} \quad (36)$$

$$d_{eg} = \varepsilon_s L_{eff} \leq 2d_g \quad (37)$$

$$\varepsilon_s = \frac{2d_g}{L_g} \quad (38)$$

l_m is the minimum support length securing the safe transmission of the vertical reaction $\geq$ 40cm,
d_{eg} is the effective displacement of the two parts due to differential seismic ground displacement,
d_g is the design value of the peak ground displacement = $0.025\alpha_g S T_C T_D$
L_g is the distance specified in 3.6
α_g, S ,T_C and $T_D$are in accordance with 3.2

When the bridge site is at a distance less than 5km from a known seismically active fault, capable to produce a seismic event of magnitude M $\geq$ 6.5, the value of d_{eg} estimated above, should be doubled.
L_{eff} is the effective length of deck, taken as the distance from the deck joint in question to the nearest full connection of the deck to the substructure. If the deck is fully connected to more than one pier, then L_{eff} shall be taken as the distance between the support and the centre of the group of piers. In this context "full connection" means a connection of the deck or deck section to a substructure member, either monolithically or through fixed bearing, seismic links, or STU.
d_{es} is the effective seismic displacement of the support due to the deformation of the structure, estimated as follows:

- For decks connected to piers either monolithically or through fixed bearings, acting as full seismic links, $d_{es} = d_{Ed}$, where d_{Ed} is the total longitudinal design seismic displacement, in accordance with Eq (5).
- For decks connected to piers or to an abutment through seismic links with slack equal to s:
  $d_{es} = d_{Ed} + s$ (39)

In the case of an intermediate separation joint between two sections of the deck l_{ov}, should be estimated by taking the square root of the sum of the squares of the values calculated for each of the two sections of the deck as above. In the case of an end support of a deck section on an intermediate pier, l_{ov} should be estimated as above and increased by the maximum seismic displacement of the top of the pier d_E.

6.11 Abutments and retaining walls

Detailed simplified rules are given, based on essentially elastic response of all main components.

6.12 Culverts with large overburden

When a culvert has a large depth of fill over the top slab (exceeding 1/2 of its span), the inertial seismic response may be neglected, and the response be estimated from the kinematic compatibility between the culvert structure and the free-field seismic deformation of the surrounding soil, corresponding to the design seismic action.
The free-field seismic soil deformation may be assumed as a uniform shear-strain field (see Fig. 4) with shear strain:

$\gamma_s = v_g / v_s$ (40)

v_g is the peak ground velocity estimated as $v_g = \dfrac{ST_C a_g}{2\pi}$ with S and T_C as defined in 3.2,

v_s is the shear wave velocity in the soil under the shear strain corresponding to the ground acceleration. This value may be estimated, from the value $v_{s,max}$ for small strains, using Table 4.1 of EN 1998-5.

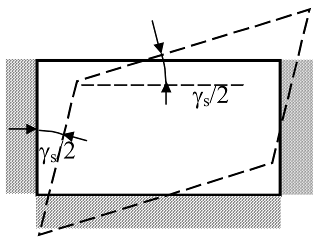

Fig. 4, Free-field soil deformation γ_s

7. BRIDGES WITH SEISMIC ISOLATION

This section covers the seismic design of bridges provided with isolating units (isolating system) arranged over the isolation interface and aiming at reducing the seismic response. The reduction may be achieved by either lengthening of the fundamental period, or by increasing of the damping or by combination of the two effects.

7.1 Basic requirements and compliance criteria

In addition to the basic requirements of 2.1, increased reliability is required for the isolating system. To this end the isolating system is designed for increased design displacements, with recommended value of increase factor γ_{IS}=1.50.

7.2 Design properties of the isolating systems

Following generic types of isolator units are considered:

- Units with hysteretic behaviour, including hysteretic steel devices and Lead Rubber Bearings (LRB).
- Elastomeric bearings, including normal low damping laminated bearings and special high damping elastomeric bearings.
- Units with viscous behaviour, including viscous fluid dampers
- Units with friction behaviour, including sliding devices with flat or spherical sliding surfaces.

The normal design properties of all isolator units should be assessed by means of special Prototype tests. Excepted from this rule are normal elastomeric bearings, for which normal design properties and design rules are defined by the code. Also excepted are flat siding bearings, as long as their contribution to the damping of the isolating system is ignored.

7.3 Variability of the design properties of the isolators

The analysis is in general carried out for two sets of design properties, reflecting the influence of external factors as ageing, temperature, loading history contamination and cumulative travel.

- Upper bound design properties (UBDP)
- Lower bound design properties (LBDP)

In the absence of special test results, rules are given for the estimation of the variation of the nominal design properties of common isolator types (elastomeric or LRB bearings and sliding devices).

7.4 Analysis methods

Fundamental mode spectrum analysis, multi-mode spectrum analysis and time history analysis, are the analysis methods foreseen by EC8-2, together with specific conditions for the applications of the first two.

7.5 Design of substructures

The design forces and the design rules for the substructures correspond to limited ductile behaviour ($q \leq 1.50$).

7.6 Special requirements for the isolating system

A minimum horizontal restoring force and a maximum value of static residual displacement of the isolating system are required at the design displacement.

Rules for provisions of sufficient lateral restraint at the isolation interface under serviceability criteria as well as for the eventual use of sacrificial bracings or STUs are given. Inspection and maintenance conditions are defined.

8. SEISMIC DEFORMATION CAPACITY OF PIERS

8.1 Ultimate displacement

The ultimate displacement d_u is defined as the maximum displacement satisfying the following condition. The structure shall be capable of sustaining at least 5 full cycles of deformation to the ultimate displacement, without initiation of failure of the confining reinforcement for reinforced concrete sections, or local buckling effects for steel sections, and without drop of the resisting force for steel ductile members or without a drop exceeding 20% of the maximum for reinforced concrete ductile members (see Fig. 5).

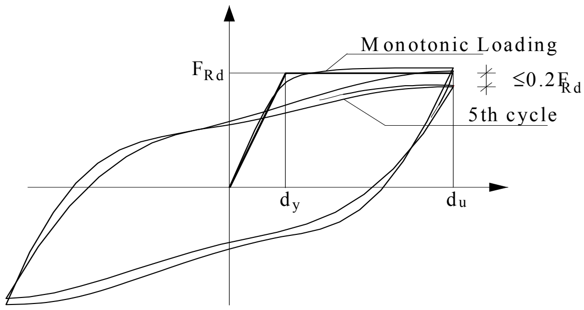

Fig. 5, Force-displacement cycles (Reinforced concrete)

8.2 Ultimate curvature

When no test results are available for the direct estimation of the ultimate displacement, this estimation may be based on curvature integration along the member, where the ultimate curvature Φ_u at the plastic hinge of the member is taken as:

$$\Phi_u = \frac{\varepsilon_s - \varepsilon_c}{d} \tag{41}$$

d is the effective section depth, ε_s and ε_c are the reinforcement and concrete strains respectively (compressive strains negative), derived from the condition that either of the two or both have reached the following ultimate values:

- compression strain of unconfined concrete ε_{cu1}= -0,0035 (EN 1992-1-1 for fck ≤ 50 MPa)
- compression strain of confined concrete, corresponding to the first fracture of confining hoop reinforcement (Mander model)

  $$\varepsilon_{cu,c} = 0{,}004 + \frac{1{,}4\rho_s f_{ym} \varepsilon_{su}}{f_{cm,c}} \tag{42}$$

  ρ_s = ρ_w for circular spirals or hoops, ρ_s = $2\rho_w$ for orthogonal hoops, and ε_{su} = ε_{uk}=0.075 is the characteristic value of the confining reinforcement steel elongation at maximum force (EN 1992-1-1 for Class C steel)
- ultimate tensile strain of reinforcement ε_{su} = ε_{uk} = 0.075 (EN 1992-1-1, Class C steel)

8.3 Chord rotation

The plastic rotation capacity $\theta_{p,u}$, and the total chord rotation θ_u of plastic hinges (see Fig. 6) may be estimated on the basis of the ultimate curvature Φ_u and the plastic hinge length L_p as follows:

$$\theta_u = \theta_y + \theta_{p,u} \tag{43}$$

$$\theta_{p,u} = (\Phi_u - \Phi_y) L_p \left(1 - \frac{L_p}{2L}\right) \tag{44}$$

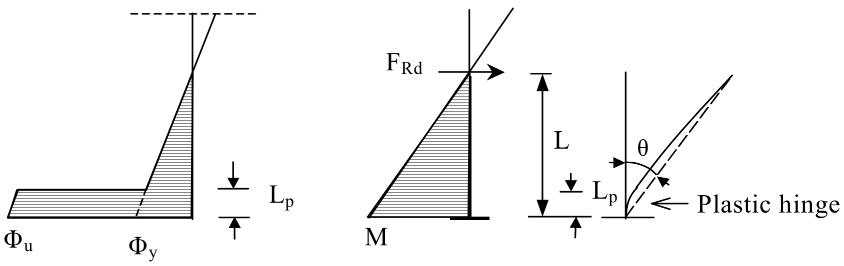

Fig. 6, Chord rotation $\theta = \frac{1}{L}\int_0^L \Phi x dx$

L is the distance from the plastic joint to the point of zero moment in the pier, Φ_y is the yield curvature
For linear variation of the bending moment, the yield rotation θ_y may be assumed:

$$\theta_y = \frac{\Phi_y L}{3} \qquad (45)$$

Both Φ_y and Φ_u should be assessed by means of a moment curvature analysis of the section under the axial load corresponding to the design seismic combination. When $\varepsilon_c \geq \varepsilon_{cu1}$, only the confined concrete core section should be taken into an account.

Φ_y should be evaluated by idealising the actual M-Φ diagram by a bilinear diagram of equal area beyond the first yield of reinforcement as shown in Fig. 7.

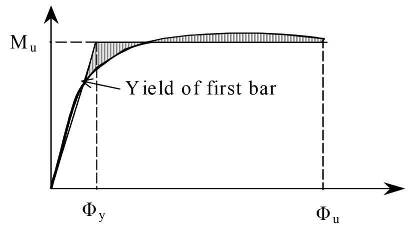

Fig. 7, Definition of Φ_y

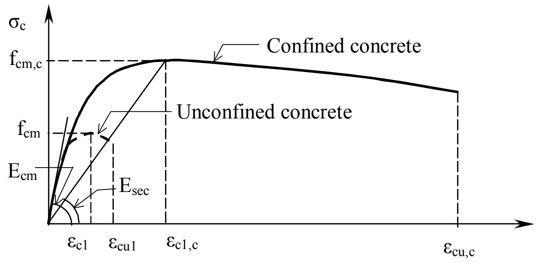

Fig. 8, Stress-strain relation for confined concrete (Mander model)

The above estimation of the plastic rotation capacity is valid for piers with shear ratio α_s=L/d ≥ 3.0
For 1,0 ≤ α_s < 3,0 the plastic rotation capacity should be multiplied by the reduction factor $\lambda(\alpha_s)$ (see Table 4)

8.4 Material parameters for moment-curvature analysis

- Concrete
  Mean values f_{cm}, E_{cm} (Table 3.1 of EN 1992-1-1) should be used
  ($f_{cm} = f_{ck} + 8$ (MPa), $E_{cm} = 22(f_{cm}/10)^{0.3}$)
  For unconfined concrete the stress-strain relation for non-linear analysis specified in 3.1.5 (1) of EN 1992-1-1, is used ($\varepsilon_{c1} = 0.0007 f_{cm}^{0.31}$).
  For confined concrete the Mander model is recommended (see Fig. 8)
- Reinforcement steel

Following values are recommended $f_{ym} / f_{yk} = 1.15$, $f_{tm} / f_{tk} = 1.20$ and $\varepsilon_{su} = \varepsilon_{uk}$

8.5 Plastic hinge length L_p

For a plastic hinge occurring at the top or the bottom junction of a pier with the deck or the foundation body (footing or pile cap), with longitudinal reinforcement of characteristic yield stress f_{yk} (in MPa) and bar diameter d_s, following relation for estimation of the plastic hinge length L_p is recommended

$$L_p = 0{,}10L + 0{,}015 f_{yk} d_s \qquad (46)$$

L is the distance from the plastic hinge section to the section of zero moment, under the seismic action.

9. LATERAL RESTORING CAPABILITY REVISION

The isolating system is required to present self-restoring capability in both principal horizontal directions, to avoid cumulative build-up of displacements. This capability is available when the system has small residual displacements in relation to its displacement capacity d_m.

The previous requirements are considered to be satisfied in a direction when either one of the following conditions is satisfied:

$$d_{cd} / d_r \leq \delta \qquad (47)$$

or

$$d_{mi} \geq d_{o,i} + \gamma_{du} d_{bi,d} \rho_d \,, \qquad \rho_d = 1 + 1.35 \frac{1 - \left(d_y / d_{cd}\right)^{0.6}}{1 + 80\left(d_{cd} / d_r\right)^{1.5}} \qquad (48)$$

where for the examined direction d_{cd} is the design displacement of the isolating system, d_r is the maximum residual displacement ($dr=F_0/K_p$ for bilinear systems), d_y is the yield displacement of the isolation system, d_{mi} is the displacement capacity of isolator i, $d_{bi,d}$ is the design displacement of isolator i corresponding to d_{cd}, and $d_{0,i}$ is the non-seismic offset displacement of isolator i. The recommended values for the numerical coefficients δ and γ_{du} are δ=0.5 and γ_{du}=1.20.
Condition (47) establishes the systems where the residual displacements are insignificant as compared to the design displacement d_{cd}. Condition (48) establishes the systems that process adequate displacement capacity d_m in order to accommodate the accumulation of seismic residual displacements during the lifetime of the structure. The factor ρ_d in relation (48) multiplies the design displacement to account for the possible accumulation of residual displacements. The factor ρ_d is plotted in Fig. 9.

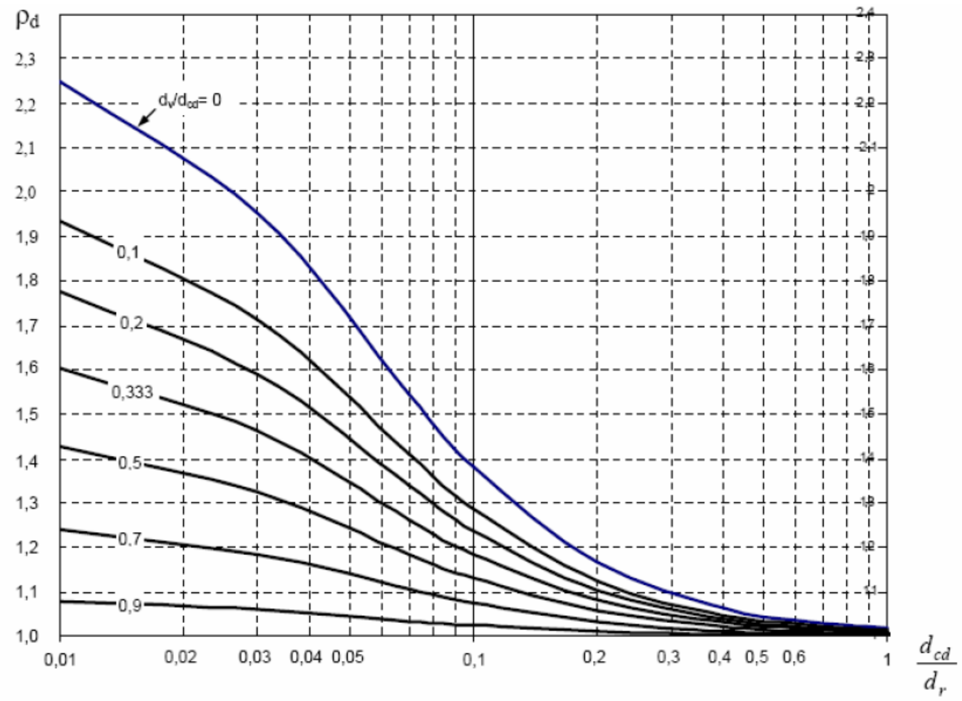

Fig. 9, Plot of factor ρ_d in relation (48).

REFERENCES

[1] CEN, prEN1998-2, EC8-2: Design provisions for earthquake resistance of structures. Part 2: Bridges, 2003.

[2] AASHTO, Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, USA, 1999.

[3] CALTRANS: Seismic Design Criteria – Version 1-2, December 2001

[4] FEMA-356, Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, 2000.

[5] ATC-32, Improved Seismic Design Criteria for California Bridges, 1996.

[6] ATC-18, Seismic Design Criteria for Bridges and other Highway Structures: Current and Future, 1997.

[7] Priestley, M.J.N, Seible, F., Clavi, G.M.: Seismic Design and Retrofit of Bridges, J. Wiley & Sons, 1996.

[8] Penzien, J.: Earthquake Engineering for Transportation Structures – Past, Present, Future, Earthquake Spectra, Volume 17, No 1, Feb. 2001.

[9] Panagiotakos, T.B., Fardis, M.N.: Deformations of RC members at Yielding and Ultimate Greek Conference of Earthquake Engineering, Thessaloniki, Greece, 2001.

[10] Kawashima, K.: Seismic Design and Retrofit of Bridges, 12th World Conference on Earthquake Engineering, 2000.

[11] Preumont, A.: The Generation of Spectrum Compatible Accelerogram for the Design of Nuclear Power Plants, Earthquake Engineering and Structural Dynamics, Vol.12, 481-497, 1984.

[12] Powell, G.H., Campbell, S.A.: DRAIN-3DX: Static and Dynamic Analysis of Inelastic 3D Structures, Berkeley 1994.

[13] Chadwell, C.: UCFyber: Cross Section Analysis of Reinforced Concrete, Berkeley 2001.

[14] Miranda, E.: Evaluation of Site-Dependent Inelastic Seismic Design Spectra, Journal of Structural Engineering, ASCE, 119(5), 1319–1338, 1993.

[15] Miranda, E.: Site-Dependent Strength Reduction Factors, Journal of Structural Engineering, ASCE, 119(12), 3505–3519, 1993.

[16] Lam, N., Wilson J. and Hutchinson, G.: The Ductility Factor in the Seismic Design of Buildings, Earthquake Engineering and Structural Dynamics,. 27, 749–769, 1998.

[17] Ang Beng Ghee, Priestley, M.J.N., and Park, R.: Ductility of Reinforced Concrete Bridge Piers Under Seismic Loading, Report 81-3, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1981.

[18] Chai, Y., Priestley, M.J. and Seible, F.: Seismic Retrofit of Circular Bridge Columns for Enhanced Flexural Performance, ACI Structural Journal, V88., No.5, September-October 1991, pp. 572-584.

[19] Cheok, G.S., and Stone, William C.: Behavior of 1/6-Scale Model Bridge Columns Subjected to Cycle Inelastic Loading, NBSIR 86-3494, Center for Building Technology, National Engineering Laboratory, National Institute of Standards and Technology, 1986.

[20] Davey, B.E.: Reinforced Concrete Bridge Piers Under Seismic Loading, Master of Engineering Report, Civil Engineering Department, University of Canterbury, New Zealand, 1975.

[21] Hose, Y.D., Seible, F. and Priestley, M.J.N.: Strategic Relocation of Plastic Hinges in Bridge Columns, Structural Systems Research Project, 97/05, University of California, San Diego, 1997.

[22] Kunnath, S.K., El-Bahy, A., Taylor, A., and Stone, W.: Cumulative Seismic Damage of Reinforced Concrete Bridge Piers, Technical Report NCEER-97-0006, National Center for Earthquake Engineering Research, 1997.

[23] Lehman, D.E., Moehle, J.P.: Seismic Performance of Well-Confined Concrete Bridge Columns, Pacific Earthquake Engineering Research Center, PEER 1998/01, 2000.

[24] Lim, K. Y., McLean, D., and Henley, E. H.: Moment-Reducing Hinge Details for the Bases of Bridge Columns, Transportation Research Record, No. 1275, Transportation Research Board, Washington, D.C., 1991.

[25] Lim, K.Y., and McLean, D.I.: Scale Model Studies of Moment-Reducing Hinge Details in Bridge Columns, ACI Structural Journal, Vol. 88, No. 4, July-August, 1991, pp. 465-474.

[26] McLean, D.I., and Lim, K.Y.: Moment-Reducing Hinge Details for the Bases of Bridge Columns, Report No.WA-RD 220.1, Washington State Department of Transportation, Planning, Research and Public Transportation Division, 1990.

[27] Munro, I.R.M., Park, R. and Priestley, M.J.N.,: Seismic Behaviour of Reinforced Concrete Bridge Piers, Report 76-9, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1976.

[28] Ng Kit Heng, Priestley, M.J.N.; and Park, R.: Seismic Behaviour of Circular Reinforced Concrete Bridge Piers, Report 78-14, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1978.

[29] Petrovski, J. and Ristic, D.: Reversed Cyclic Loading Test of Bridge Column Models, Report IZIIZ 84-164, Institute of Earthquake Engineering and Engineering Seismology, 1984.

[30] Pontangaroa, R.T., Priestley, M.J.N.; and Park, R.: Ductility of Spirally Reinforced Concrete Columns Under Seismic Loading, Report 79-8, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1979.

[31] Stone, W.C., and Cheok, G.S.: Inelastic Behavior of Full-Scale Bridge Columns Subjected to Cyclic Loading, NIST BSS 166, Building Science Series, Center for Building Technology, National Engineering Laboratory, National Institute of Standards and Technology, 1989.

[32] Zahn, F.A., Park, R., and Priestley, M.J.N.: Design of Reinforced Concrete Bridge Columns for Strength and Ductility, Report 86-7, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1986.

[33] Panagiotakos T.B., Kolias B., Plakas A. : Performance Based Seismic Design and Assessment of Irregular Bridges, Proceedings Fib-Symposium : Concrete Structures in Seismic Regions, Athens 6-9 May 2003.

[34] Katsaras C.P., Panagiotakos T.B., Kolias B. : Residual Displacements in Seismic Isolation Systems, Proceedings of ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2007), Rethymno, Crete, Greece, 13–16 June 2007.

[35] Katsaras C.P., Panagiotakos T.B. Kolias B. : Restoring capability of bilinear hysteretic seismic isolation systems, Journal of Earthquake Engineering and Structural Dynamics (published online), 2007.

RESEARCH WORK RELATED TO EC8-2

[1] Bardakis V., Fardis M.N. : Linear vs nonlinear seismic response analysis of concrete bridges, FIB symposium “Concrete Structures – Stimulators for Development”, Dubrovnik, Croatia, May, 2007.

[2] Bardakis V. : Displacement-based antiseismic bridge design, PhD thesis, University of Patras, Greece, 2007.

[3] Fardis M.N. (editor) : Guidelines for displacement-based design of buildings and bridges, LESSLOSS Risk Mitigation for Earthquakes and Landslides, Report No. 2007/05, July 2007.

[4] Katsaras C.P., Panagiotakos T.B., Kolias B. : Residual Displacements in Seismic Isolation Systems, Proceedings of ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2007), Rethymno, Crete, Greece, 13–16 June 2007.

[5] Katsaras C.P., Panagiotakos T.B. Kolias B. : Restoring capability of bilinear hysteretic seismic isolation systems, Earthquake Engineering and Structural Dynamics (published online), 2007.

[6] Katsaras C.P., Panagiotakos T.B. Kolias B. : Effect of torsional stiffness of prestressed concrete box girders and uplift of abutment bearings on seismic performance of bridges, Bulletin of Earthquake Engineering (under review), 2008.

[7] Lupoi A., Franchin P., Pinto E., Monti G. : Seismic design of bridges accounting for spatial variability of ground motion, Earthquake Engineering and Structural Dynamics, Vol. 34, No. 4-5, 2005, pp. 327 - 348.

[8] Nuti C., Vanzi I. : Influence of earthquake spatial variability on differential soil displacements and SDF system response, Earthquake Engineering and Structural Dynamics, Vol. 34, No. 11, 2005, pp. 1353 – 1374.

[9] Nuti C., Vanzi I. : Influence of earthquake spatial variability on the differential displacements of soil and single degree of freedom structures, Universita degli studi di Roma Tre, Technical Report N.1, 2004

[10] Panagiotakos T.B., Bardakis V., Fardis M.N. : Displacement-based seismic design procedure for concrete bridges with monolithic connection between deck and piers, 2nd FIB congress, Napoli, June 2006.

[11] Pinho R., Casarotti C., Antoniou S. : A comparison of single-run pushover analysis techniques for seismic assesment of bridges, Earthquake Engineering and Structural Dynamics, Vol. 36, 2007.

[12] Sextos A., Kappos A., Pitilakis K. : Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 2: Parametric Study, Earthquake Engineering and Structural Dynamics, V. 32, No. 4, 2003, pp. 629-652.

[13] Sextos A., Kappos A., Pitilakis K. : Seismic response of long concrete bridges: effect of coupled ground motion variability and soil-foundation interaction", FIB 2003 Symposium: Concrete Structures in Seismic Regions, Athens May 2003, CD ROM Proceedings, paper no. 206.

[14] Sextos A.G. Kappos, A.J. : Evaluation of the new Eurocode 8-Part 2 provisions regarding asynchronous excitation of irregular bridges, 4th European Workshop on the Seismic Behaviour of Irregular and Complex Structures, Thessaloniki, Aug. 2005, Paper no. 04.

[15] Sextos A., Kappos A., Kolias B. : Computing a 'reasonable' spatially variable earthquake input for extended bridge structures, Proceed. of the 13th ECEE, Geneva, Sep. 2006, paper no. 1601.