EC8 Webinars

Second Generation of Eurocode 8

Webinar 2.1

Bridge classification & structural analysis

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FIRST vs SECOND-GENERATION EUROCODE 8 PART 2

EN1998-2:2005 113+33=146 pages

1 Introduction (**1.1.1** Scope of EN 1998-2)
2 Basic requirements & compliance criteria
~~**3** Seismic action~~
4 Analysis
5 Strength verification
6 Detailing
7 Bridges with seismic isolation

Annexes:

A Probabilities related to the reference seismic action…
B Relationship between displ. ductility & curv. ductility
C Estimation of effective stiffness of RC ductile members
D Spatial variability of ground motion: models & analysis
E Probable mat. prop.'s & PH def. cap for NL analysis
F Added mass of entrained water for immersed piers
G Calculation of capacity design effects
H Static NL analysis (pushover)
J Variation of design prop.'s for isolator units
JJ λ-factors for common isolator types
K Tests for validation of design prop.'s of isolator units

FprEN1998-2:2024 (CEN/TC250/SC8 N1307)

70+15=85 pages

1 Scope (**1.1**Scope of EN 1998-2)
2 Normative references
3 Terms, definitions and symbols
4 Basis of design
5 Modelling and structural analysis
6 Verifications of structural members to limit states
7 Detailing for ductility
8 Specific rules: bridges with antiseismic devices
9 Specific rules: cable-stayed & extradosed bridges
10 Specific rules: integral abutment bridges

Annexes:

A Characteristics of earthquake resistant bridges
B Added mass of entrained water for immersed piers
C Additional information on timber bridges
D Displacement-based approach for integral abutment bridges

FprEN1998-1-1:2023 (CEN/TC250/SC8 N1283)
EN15129
FprEN1990-1:2023,A2 (CEN/TC250/SC10 N667)

OUTLINE OF WEBINAR 2.1

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4.2 Seismic actions
- 4.2.1 General
- 4.2.2 Spatial variability of the seismic action

4.3 Characteristics of earthquake resistant bridges
- 4.3.1 Conceptual design
- 4.3.2 Primary and secondary seismic members
- 4.3.3 Resistance & ductility conditions – Capacity design
- 4.3.4 Connections
- 4.3.5 Control of displacements – Ancillary elements
- 4.3.6 Choice of DC – Seismic action limits for DC1 to DC3
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National Annex
for low seismic action class

5. MODELLING & STRUCTURAL ANALYSIS

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- 5.1.1 General
- 5.1.2 Torsional effects about a vertical axis
- 5.1.3 Second-order effects

5.2 Methods of analysis
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- 5.2.2 Force-based approach
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5.3 Methods of analysis accounting for spatial variability of ground motion
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- 5.3.4 Long bridges on non-uniform soil

5.4 Combination of the seismic action with other actions

Webinar 2.2

Clause 4 - Basis of design

4. BASIS OF DESIGN

4.1 Basic requirements

- Importance Classes (IC) replaced by Consequence Classes (CC) in 2nd gen. EN1998
- CCs for bridges are not redefined here. CCs from **EN1990 A.2** are adopted

(1) Consequence class CC3 should be divided into CC3-a and CC3-b according to FprEN 1998-1-1:2024, 4.2(3).

NOTE The definition of consequence classes for bridges is given in EN 1990:2023, Table A.2.1(NDP).

Table A.2.1 (NDP) — Examples of bridges in different consequence classes

Consequence class [a]	Description of consequence	Examples
CC4b	Highest	
CC3b	High (upper class)	Where an increased level of reliability is required, when specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties
CC3a	High (lower class)	Railway bridges on main railway lines, bridges over main railway lines, bridges over and under major roads
CC2	Normal	Bridges not in other consequence classes
CC1	Low	Short span bridges on local roads with little traffic (provided they do not span over main railway lines or major roads)
CC0b	Lowest	Elements other than structural, see 3.1.1.7.

[a] CC3b corresponds to an increased level of reliability compared to CC3a.
[b] For provisions concerning CC0 and CC4, see 4.3.

4. BASIS OF DESIGN

4.2 Seismic action (1/3)

- $T_{\mathrm{LS,CC}}$ are the same as in Part 1-2 for buildings for the same LS & CC
- $T_{\mathrm{LS,CC}}$ depend on target reliability index (NDP) suggested in Part 1-1 (*):

$$T_{\mathrm{LS,CC}} = -\frac{t_{\mathrm{L}}}{\ln \Phi\left(0{,}8\beta_{\mathrm{t,LS,CC}}\right)}$$

(*) Not to be confused with the well-known formula $T_{\mathrm{LS,CC}} = -\frac{t_{\mathrm{L}}}{\ln(1-p)}$ since p is the probability of the design seismic action being exceeded, while $\beta_{\mathrm{t,LS,CC}}$ is related to the probability $p_f < p$ that the limit state is exceed

- Performance factor $\gamma_{\mathrm{LS,CC}}$ (formerly, importance factor γ_{I}) depends on $T_{\mathrm{LS,CC}}$

$$\gamma_{\mathrm{LS,CC}} = \left(\frac{T_{\mathrm{LS,CC}}}{T_{\mathrm{ref}}}\right)^{\frac{1}{k}}$$

4.2.1 General

(1) FprEN 1998-1-1:2024, 4.3(3) to (5), should be applied.

NOTE 1 The values of return period $T_{\mathrm{LS,CC}}$, according to FprEN 1998-1-1:2024, 4.3(3), are those given in Table 4.2 (NDP). The values in Table 4.2 (NDP) are computed with the values of $\beta_{\mathrm{t,LS,CC}}$ suggested in FprEN 1998-1-1:2024, F.3. If the National Annex gives different values of $\beta_{\mathrm{t,LS,CC}}$ for use in a country, values of $T_{\mathrm{LS,CC}}$ should be updated according to the note in FprEN 1998-1-1:2024, 4.3(3).

NOTE 2 When performance factors are used, according to FprEN 1998-1-1:2024, 4.3(5), the values of $\gamma_{\mathrm{LS,CC}}$ are those given in Table 4.3 (NDP). The values in Table 4.3 (NDP) are consistent with those in Table 4.2(NDP). If the National Annex gives different values of $\beta_{\mathrm{t,LS,CC}}$ for use in a country, values of $\gamma_{\mathrm{LS,CC}}$ should be updated according to the note in FprEN 1998-1-1:2024, 4.3(5).

Table 4.2 (NDP) — Return period $T_{\mathrm{LS,CC}}$ values, in years, for bridges

Limit state	Consequence class			
	CC1	CC2	CC3-a	CC3-b
NC	600	1600	2500	5000
SD	275	475	600	900
DL	100	115	125	140

Table 4.3 (NDP) — Performance factor $\gamma_{\mathrm{LS,CC}}$ values for bridges

Limit state	Consequence class			
	CC1	CC2	CC3a	CC3b
NC	1,10	1,50	1,75	2,20
SD	0,80	1,00	1,10	1,25
DL	0,60	0,60	0,65	0,65

$\gamma_{\mathrm{I}} = 0{,}85$ $\gamma_{\mathrm{I}} = 1{,}30$

4. BASIS OF DESIGN

4.2 Seismic action (2/3)

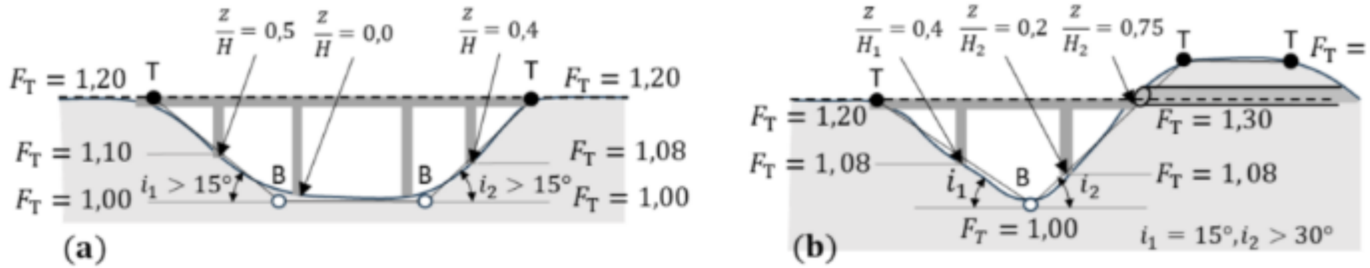
- Guidance for evaluation of F_T in bridges
- A set of values is obtained, one per support!
  - Note: if deck level is the free flat ground surface, deamplification would occur, but this is not considered
- Unless spatial variability is considered in the analysis, $\bar{F}_T$ is used to amplify the input motion
- This applies when the model does not include the soil domain with its geometry

4.2.1 General

(2) The seismic action should be taken as given in FprEN 1998-1-1:2024, 5.2.

(3) In application of (2), a distinct value of the topographic amplification factor F_T should be calculated according to FprEN 1998-1-1:2024, Table 5.5, at each support, as illustrated in Figure 4.1.

NOTE For narrow valleys, the seismic motion at B, to which the amplification factors F_T are applied to calculate the motions at other supports, is conventionally taken as the motion on a free flat ground surface as given in FprEN 1998-1-1:2024, Clause 5.



Key

H slope height

z height of support with respect to slope base point B

i inclination angle of slope

Figure 4.1 — Calculation of topographic amplification factors at supports in the case of (a) wide valleys and (b) narrow valleys

(7) When included in the model, soil-structure interaction (SSI) should conform to FprEN 1998-5:2024, Clause 8.

NOTE If inertial and kinematic interaction are modelled simultaneously by means of response-history analysis of the whole structure-foundation-soil system, according to FprEN 1998-5:2024, 8.5, topographic amplification as well as spatial variability of the seismic action (4.2.2) are implicitly included in the model.

4. BASIS OF DESIGN

4.2 Seismic action (3/3)

- Site considerations:
  - co-seismic displacement
    This is one place where the code gives a recommandation but few indications (topic not yet amenable of simplified treatment).
  - Some indication on displacements in Part 4 (pipelines share the vulnerability with bridges due to their extended nature)
  - slope stability
  - liquefaction susceptibility

Webinar 5

4.2.1 General

(6) If expected to be relevant, ground permanent displacements should be evaluated through specific studies. Their consequences should be minimized by appropriate measures, such as selecting a suitable structural system.

NOTE 1 Ground permanent displacements are expected to be important in close vicinity to active and shallow faults.

NOTE 2 The seismic action in FprEN 1998-1-1:2024, 5.2, accounts only for ground shaking or transient displacement, not for permanent displacements. The latter, arising from ground failure or fault rupture, can result in imposed deformations with severe consequences for bridges.

4.3 Characteristics of earthquake resistant bridges

4.3.1 Conceptual design

(7) If a bridge crosses a potentially active tectonic fault, the discontinuity of the ground displacement should be accommodated either by adequate flexibility of the structure or by provision of suitable movement joints.

NOTE The total differential displacement at a fault crossing consists of the sum of the differential displacement in the seismic design situation (transient part of seismic motion), calculated consistently with the return period of the design seismic action, and of a quasi-static differential displacement due to slow movement on the fault developed over the design life of the bridge. Information on the seismic part of the differential displacement at fault crossing can be found in prEN 1998-4:2023, Annex E. This standard does not give information on the quasi-static component of the differential displacement.

(8) Slope stability should be verified and the effect of potential instability on the bridge assessed according to FprEN 1998-5:2024, 7.2.

(9) The liquefaction potential of the foundation soil should be investigated in accordance with FprEN 1998-5:2024, 7.3.

4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (1/5)

- Energy dissipation in members, devices or both, but…
- …not in the foundation!
- Torsion

4.3.1 Conceptual design

(1) A bridge structure shall be able to resist the seismic action in any direction.

(2) Seismic performance of a bridge should be considered since the early stage of conceptual design, achieving a structural system that, with acceptable costs, satisfies the performance requirements specified in FprEN 1998-1-1:2024, 4.1.

NOTE 1 (2) applies to all seismic action classes.

NOTE 2 Guidance for good practice is given in informative Annex A.

(3) Satisfaction of performance requirements in the seismic design situation should be achieved by means of either a), b) or their combination:

a) resistance through structural members, possibly involving energy dissipation in clearly identified critical zones (design to ductility classes DC1, DC2 or DC3);

b) use of antiseismic devices.

(4) The seismic performance of structural members should be verified according to Clause 6.

NOTE Specific rules for "bridges equipped with antiseismic devices", "cable-stayed and extradosed bridges" and "integral abutment bridges" are given in Clauses 8, 9 and 10, respectively.

(10) Bridge foundations should not be intentionally used as sources of hysteretic energy dissipation and therefore should, as far as practicable, be designed to remain elastic under the design seismic action.

(5) The torsional resistance of a bridge structure around the vertical axis should not rely on the torsional rigidity of a single pier.

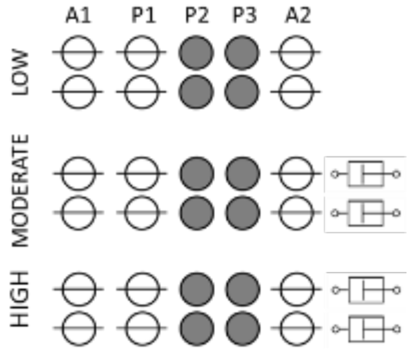
(6) In single span bridges, the bearings should be designed to resist the effects of global torsion around the vertical axis.

4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (2/5)

- Primary & secondary members
- Sacrificial elements

All supports primary in the transverse direction
P2, P3 and A2 primary in the longitudinal one



4.3.2 Primary and secondary seismic members

(1) Supporting members (piers and abutments) resisting the seismic forces in the longitudinal and transverse directions should be designated as primary. The number of primary members may be less than the total number of supporting members, by using sliding or flexible bearings between the deck and some piers. Supporting members other than primary should be designated as secondary.

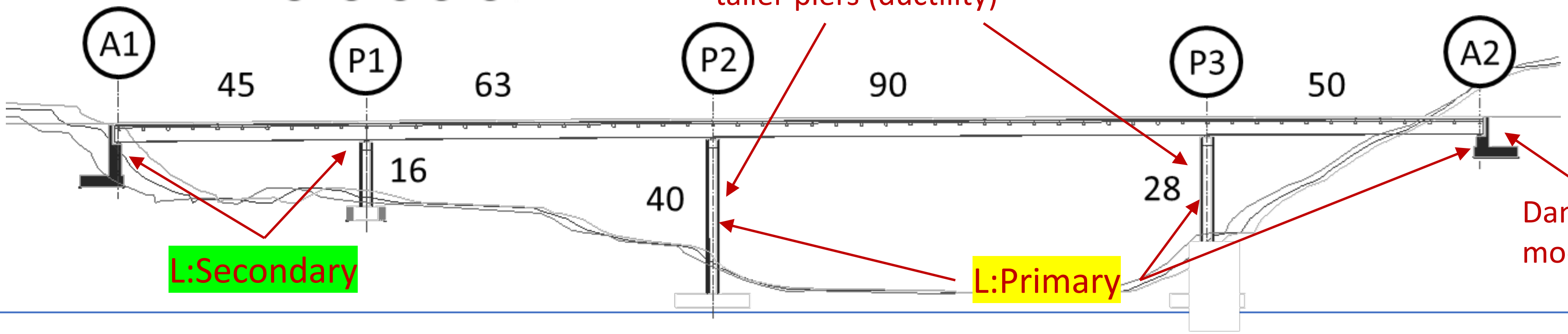
NOTE For example, disconnection in the longitudinal direction can be used to reduce the stresses arising from imposed deck deformations due to thermal actions, shrinkage, and other non-seismic actions. Disconnection in the transverse direction can lead to a better distribution of forces among supporting members, as shown in A.5.

(4) When an abutment-deck connection is rigid, either because it is monolithic or through fixed bearings or seismic links, and the corresponding abutment contributes significantly to the seismic resistance both in the longitudinal and transverse direction, it should be designated as primary member. A rigid connection may be exploited for seismic resistance, especially with shorter and medium length bridges (see specific provisions in Clause 10).

(5) Portions of structural members designed to fail to protect the remaining parts should be designated as sacrificial. Sacrificial elements should be designed to withstand the action effects in the non-seismic design situations without damage. They should cater for a predictable mode of damage in the seismic design situation, minimise risks to persons in case of failure and provide for the possibility of repair.

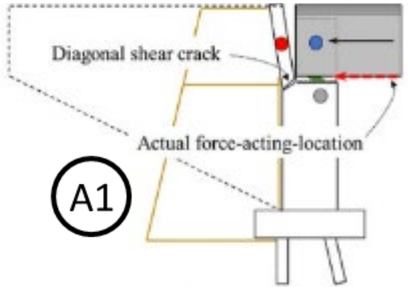
NOTE Abutments' back-walls can represent such locations, if designed as sacrificial elements.

Deck used to restrain displacements in taller piers (ductility)



L:Secondary

L:Primary

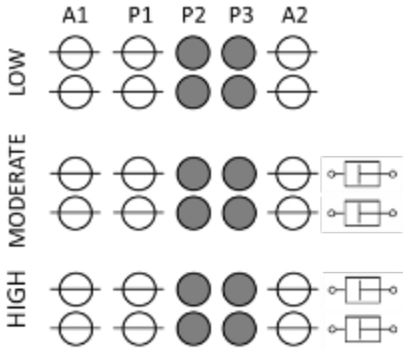


Dampers to reduce displacement for moderate and high seismic action class

4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (3/5)

- 4.3.6 applies to bridges exploiting ductility
- All supports primary in transverse dir.
  A1 and P2 secondary in longitudinal one



- Unique ductility class ≠ same q factor
- Not all DC's can be used irrespective of seismic action class (DC determines detailing, not just value of q)

4.3.6 Choice of ductility class – Limits of seismic action for design to DC1, DC2 and DC3

(1) 4.3.6 should be applied to reinforced concrete, steel and composite steel-concrete bridges with one or more support (pier or abutment) rigidly connected to the deck (either monolithically or through fixed bearings or links) and exploiting ductility for seismic protection.

NOTE Bridges equipped with antiseismic devices, cable-stayed and extradosed bridges, integral abutment bridges and timber bridges are covered in Clauses 8, 9, 10 and Annex C respectively.

(2) The primary structure should be assigned a ductility class according to FprEN 1998-1-1:2024, 4.5.2(3).

NOTE Bridge supports can be classified as primary seismic members in one local direction and secondary in the orthogonal one, depending on the type of bearings used (e.g. unidirectional sliders allow relative displacement with minimum friction in the longitudinal direction, while restraining the movement in the transverse one).

(3) The ductility class should be unique for the bridge (i.e. the same for all members and in all directions).

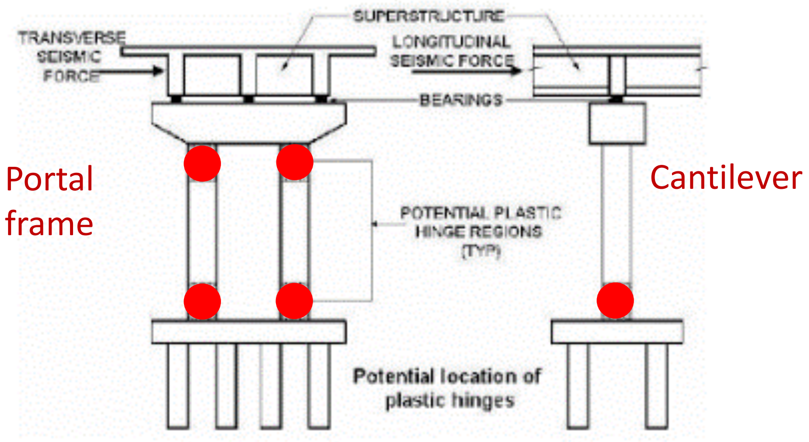
(4) In high seismic action class, bridges of CC2 and higher should be designed for DC3.

(5) Seismic design for DC1 should not be adopted in moderate and high seismic action classes.

	DC1	DC2	DC3
Low	Ok	Ok	Ok
Moderate	-	Ok	Ok
High	-	Ok (CC1)	Ok

4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (4/5)



4.3.3 Resistance and ductility conditions – Capacity design rules

(1) The locations of critical zones should be chosen to ensure accessibility for inspection and repair. Such locations should be clearly indicated in the appropriate design documents.

(2) For DC2 and DC3 structures (see 4.3.6), a dependably stable partial or full mechanism should develop in the structure through the formation of flexural plastic hinges.

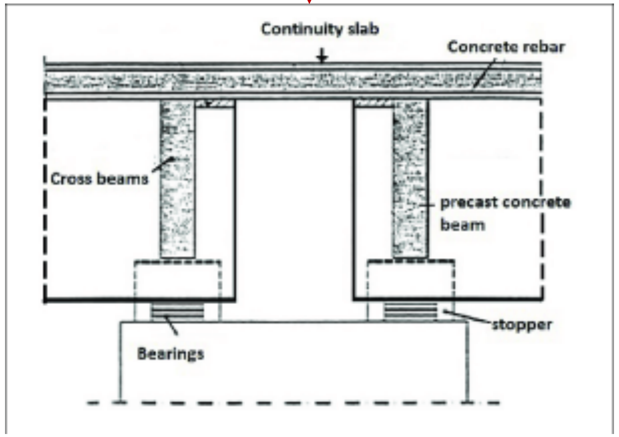
NOTE 1 These hinges normally form in the piers and act as the primary energy dissipating components.

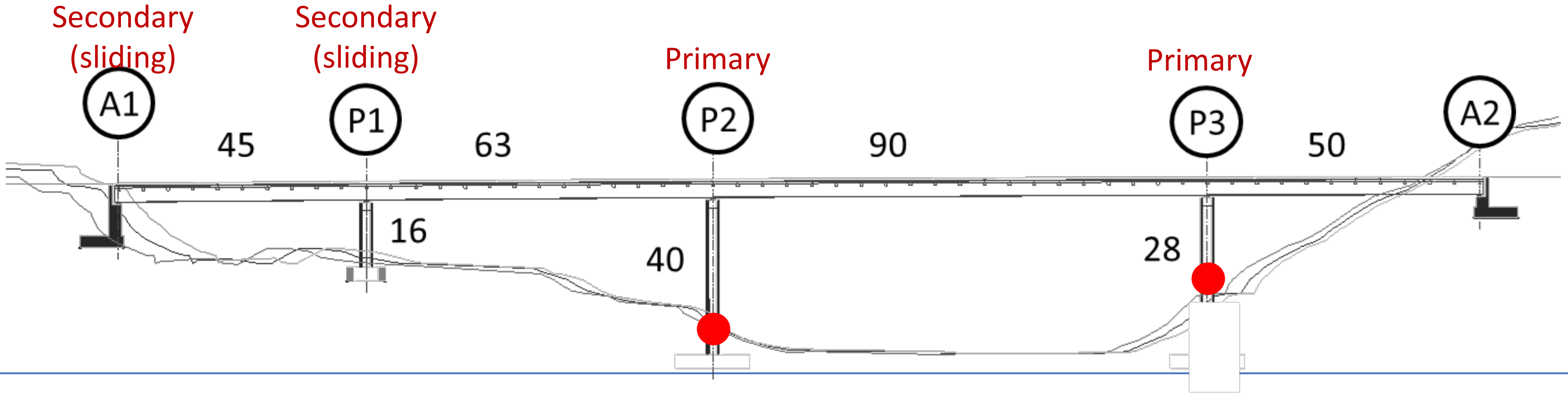
NOTE 2 Flexural plastic hinges do not necessarily form in all piers, according to 4.3.2(1) (partial plastic mechanism).

(5) The bridge deck should remain within the elastic range under the design seismic action, except as given in 5.1.1(8).

(6) Plastic hinges (in bending about the transverse axis) may form in continuity slabs.

NOTE Continuity slabs are cast-in-place slabs commonly employed to provide top slab continuity between adjacent simply supported spans formed of precast concrete girders completed by a top reinforced concrete slab.





4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (5/5)

- Avoid UNSEATING! (principle)

4.3.4 Connections

(1) Connections between supporting and supported members shall be designed in order to ensure structural integrity and avoid unseating under the total design displacement in the seismic design situation with increased reliability.

(2) (1) should be ensured by designing connections used for securing structural integrity according to Clause 8.

(3) Appropriate overlap lengths should be provided between supporting and supported members at moveable connections, to avoid unseating (see 6.3.6 and 8.5).

4.3.5 Control of displacements – Detailing of ancillary elements

(1) Detailing of structural components and ancillary elements shall be provided to accommodate the displacements in the seismic design situation.

(2) Clearances between adjacent members should be provided for protection of deck extremities. Such clearances should accommodate the design value of the total relative displacement in the seismic design situation, d_{Ed}, determined as given by Formula (4.1).

$$d_{Ed} = d_G \text{"+"} d_E \text{"+"} \psi_2 d_T \tag{4.1}$$

(3) If abutment displacements towards the deck are larger or equal than the smaller of the displacement components d_G, d_E and $\psi_2 d_T$ in (2), they should be added to d_E in Formula (4.1).

NOTE Tall reinforced earth abutments can exhibit larger displacements than reinforced concrete ones.

(4) Second-order effects according to 5.1.3 should be taken into account in the calculation of the design value of the total relative displacement in the seismic design situation.

(6) Large shock forces on sensitive components such as prestressing anchorages, caused by unpredictable impact between deck extremities, should be prevented by means of ductile/resilient members or special energy absorbing devices (buffers). Such members should possess a slack at least equal to the design value of the total relative displacement in the seismic design situation, d_{Ed}.

4. BASIS OF DESIGN

4.3 Characteristics of earthquake resistant bridges (5/5)

- Avoid UNSEATING! (principle)
- Avoid pounding (exception: sacrificial backwall, but deck must be tolerant, i.e., exclude prestressed decks)
  - $d_G = \max(short\ term, long\ term)$ creep, shrinkage, prestressing and losses
  - d_G =SRSS of displ. of adjacent portions
  - Annex C, for timber bridges Formula (4.1) should include also d_ω the displacement due to average moisture content variation $\Delta\omega$
- Second order effects accounted for amplifying displacements when

$$0{,}1 \leq \theta = \frac{M_{II}}{M_I} = \frac{qN_{Ed}d}{q_s q_R V_{Ed} h} < 0{,}2$$

or by nonlinear analysis when

$$0{,}2 \leq \theta < 0{,}3$$

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(1) Connections between supporting and supported members shall be designed in order to ensure structural integrity and avoid unseating under the total design displacement in the seismic design situation with increased reliability.

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$$d_{Ed} = d_G \text{ "+" } d_E \text{ "+" } \psi_2 d_T \qquad (4.1)$$

(3) If abutment displacements towards the deck are larger or equal than the smaller of the displacement components d_G, d_E and $\psi_2 d_T$ in (2), they should be added to d_E in Formula (4.1).

NOTE Tall reinforced earth abutments can exhibit larger displacements than reinforced concrete ones.

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(6) Large shock forces on sensitive components such as prestressing anchorages, caused by unpredictable impact between deck extremities, should be prevented by means of ductile/resilient members or special energy absorbing devices (buffers). Such members should possess a slack at least equal to the design value of the total relative displacement in the seismic design situation, d_{Ed}.

Clause 5 - Modelling & structural analysis

Mass, stiffness and damping
Torsion and second-order effects
Force-based approach (linear analysis)
Displacement-based approach (nonlinear analysis)

5. MODELLING & ANALYSIS

5.1 Modelling (1/5)

- **Masses** additional to structural ones:
  - Traffic: only the uniform distributed load (UDL)of the corresponding load model in EN1991 is considered (LM1 for roads and LM71 for railways)
  - Water: Annex B is the same as in EN1998-2:2005

5.1.1 General

(1) The model of the bridge should comply with prEN 1998-1-1:2022, 6.2.

(2) The values of combination coefficients $\psi_{E,i}$ defined in prEN 1998-1-1:2022, 6.2.1(3), Formula (6.1), for the masses associated to variable actions should account for the severity of traffic conditions. In the absence of more accurate values based on traffic analysis, values of $\psi_{E,i}$ may be taken as given in Table 5.1.

NOTE 1 Road bridges with severe traffic conditions can be considered as applying to motorways and other roads of national importance. Railway bridges with severe traffic conditions can be considered as applying to inter-city rail links and high-speed railways.

NOTE 2 In applying prEN 1998-1-1:2022, 6.2.1(3), Formula (6.1), $Q_{k,i}$ is the UDL system of load model LM1 for road and of load model LM71 for railway bridges, respectively.

Table 5.1 — Values of $\psi_{E,i}$

Type of variable action	$\psi_{E,i}$
Traffic variable action (normal traffic and footbridges)	0,0
Road traffic action (severe traffic conditions)	0,2
Railway traffic action (severe traffic conditions)	0,3
Other variable actions	0,0

(3) When the piers are immersed in water, and unless a more accurate assessment of the hydrodynamic interaction is made, this effect may be estimated by taking into account a spread added mass of entrained water acting in the horizontal directions on the immersed pier. The hydrodynamic influence on the vertical seismic action may be neglected.

NOTE Informative Annex B gives a procedure for the calculation of the added mass of entrained water, in the horizontal directions, for immersed piers.

5. MODELLING & ANALYSIS

5.1 Modelling (2/5)

- **Stiffness**: secant to elastic limit
  - Piers: secant to yield $E_c I_y$ (50% FBA)
  - Deck, flexural:
    - gross stiffness (=100%)
    - continuity slabs 25%
  - Deck, torsional:
    - Open section or slabs **0%**
    - Prestressed box section 50%
    - RC box sections 30%

5.1.1 General

(4) In application of prEN 1998-1-1:2022, 6.2.2(1), the elastic stiffness of each member should correspond to its secant effective stiffness at the elastic limit.

(5) For reinforced concrete members, the secant effective stiffness may be estimated as given in a) and b):

a) the stiffness of the cracked section at the initiation of yield of the reinforcement, for piers;

b) the stiffness of the uncracked section, for prestressed or reinforced concrete decks, except as given in (8).

(6) For the force-based approach, unless a more accurate analysis of the cracked members is performed according to (5)a), the elastic flexural and shear stiffness properties of piers designed to develop plastic hinges may be taken equal to 50 % of the corresponding stiffness of the uncracked members.

(7) In concrete decks consisting of precast concrete beams and cast *in situ* slabs, continuity slabs (see 4.3.3(6)) should be included in the model of seismic analysis, taking into account their eccentricity relative to the deck axis and a reduced value of their flexural stiffness. Unless this stiffness is estimated based on the rotation of the relevant plastic hinges, a value of 25 % of the flexural stiffness of the uncracked gross concrete section may be used for the continuity slab.

(8) If the deck is modelled as a single beam or equivalent grid model for the purpose of seismic design, the significant reduction of the torsional stiffness of concrete members, in relation to the uncracked torsional stiffness, should be accounted for. Unless a more accurate calculation is made, the fractions of the torsional stiffness of the uncracked gross section given in a) to c) may be used:

a) for open sections or slabs, the torsional stiffness may be ignored;

b) for prestressed box sections, 50 % of the uncracked gross section torsional stiffness;

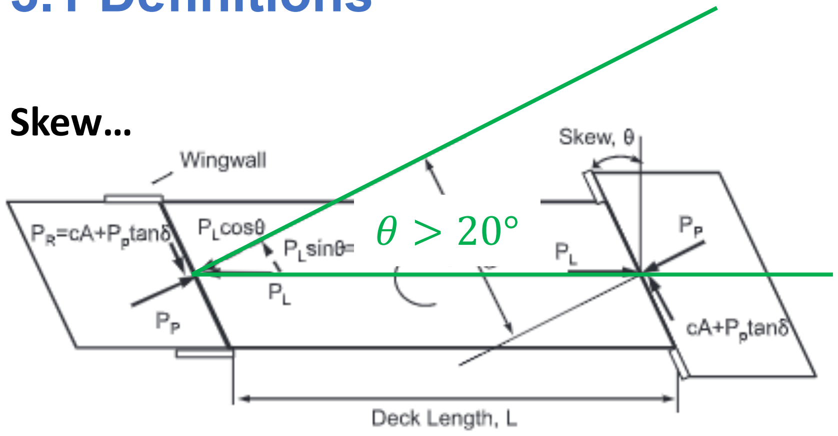
c) for reinforced concrete box sections, 30 % of the uncracked gross section torsional stiffness.

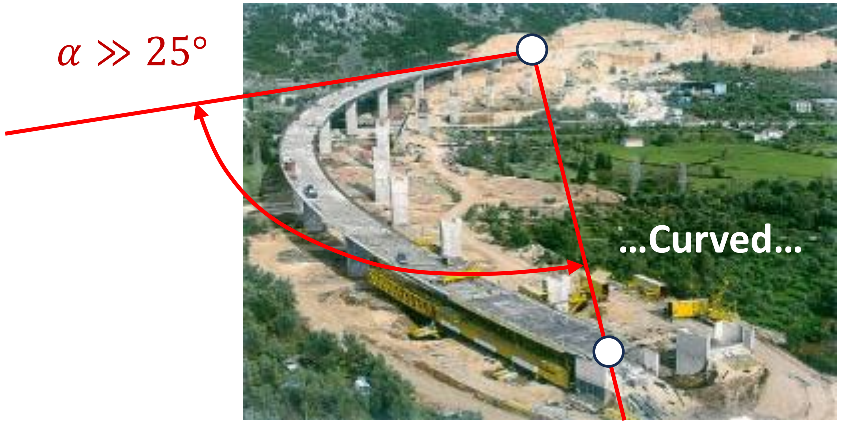
NOTE More accurate calculations are needed when torsion contributes to static equilibrium, as in curved bridges.

3. TERMS AND DEFINITIONS

3.1 Definitions

Skew…

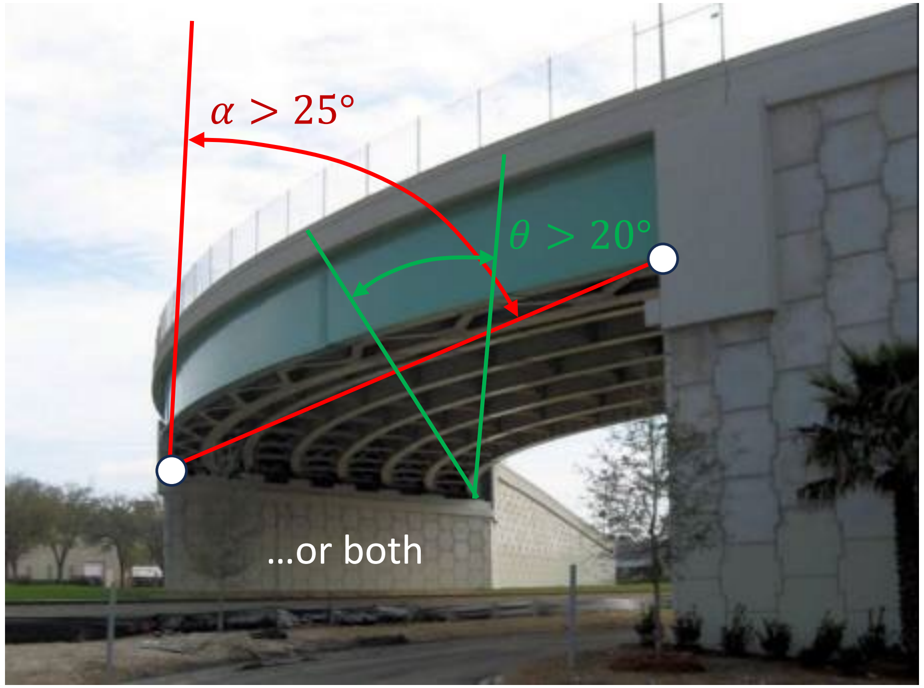




3.1.9
skew bridge
bridge whose spans are not perpendicular to the axis of the supports, with an angle of skew (3.2.2.2) larger than 20°

3.1.10
curved bridge
bridge with an angle between the initial and final tangents to the curved longitudinal axis larger than 25°

Note 1 to entry: All other bridges are considered straight.



5. MODELLING & ANALYSIS

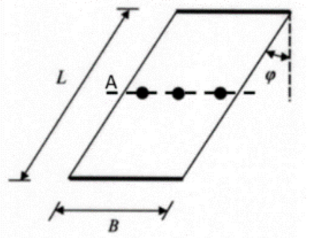
5.1 Modelling (3/5)

- **Torsion**: dynamic phenomenon
- Equivalent static moment
  $$M_t = \pm F_b e_{td}$$
  $e_{td} = L(0{,}03 + 0{,}1\sin\varphi)$ or $B(0{,}03 + 0{,}1\sin\varphi)$
- Can be used also with RSA

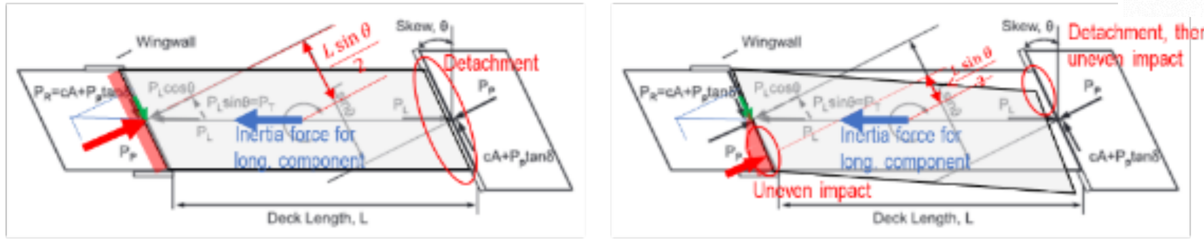
5.1.2 Torsional effects about a vertical axis

(1) Torsional motions of the bridge about a vertical axis should be considered in the analysis of skew bridges (skew angle $\varphi > 20°$) or bridges with a ratio $B/L > 0{,}5$ (Figure 5.1).

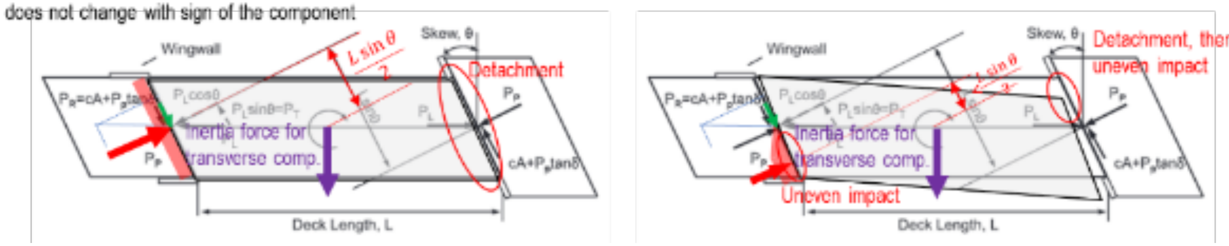
NOTE Such bridges tend to rotate about the vertical axis, even when the centre of mass theoretically coincides with the centre of stiffness.



Dynamic response, one-sided reaction takes place. After rotation initiates, the lever arm reduces



In the transverse direction the tendency to rotation has the same sign (clockwise for the case in the figure). For both components (L and T), the sign of rotation does not change with sign of the component



(4) For bridges with large skew angle ($\varphi > 45°$) supported on the abutments through bearings, the dependence of horizontal stiffness of the bearings on axial force should be accurately modelled, considering the concentration of vertical reactions near the obtuse angles.

NOTE The uneven distribution of vertical reactions amongst bearings in skew bridges cannot be captured with a single beam model. See note to 3.1.4.

5. MODELLING & ANALYSIS

5.1 Modelling (4/5)

- **Damping**:
    - Note: values are given for use in elastic RSA (to calculate η) or elastic RHA, NOT for q-factor approach
    - Weighted damping useful only for elastic RSA, for RHA individual damping assigned to each component

5.1.1 General

(9) When elastic response spectrum analysis or response-history analysis are used, the following values of equivalent viscous damping ratio ξ may be assumed, based on the material of the members where the larger part of the deformation energy is dissipated during the seismic response, for the evaluation of η according to prEN 1998-1-1:2022, 5.2.2.2(12).

— Welded steel 0,02

— Bolted steel 0,04

— Reinforced concrete 0,05

— Prestressed concrete 0,02

— Timber 0,03

NOTE 1 In general, the larger part of deformation energy is dissipated in piers.

NOTE 2 When the q-factor approach is used, there is no correction of damping in the reduced spectrum defined in prEN 1998-1-1:2022, 6.4.1.

(10) When the structure comprises several components i with different viscous damping ratios, ξ_i, the effective viscous damping of the structure ξ_{eff} may be estimated by Formula (5.1).

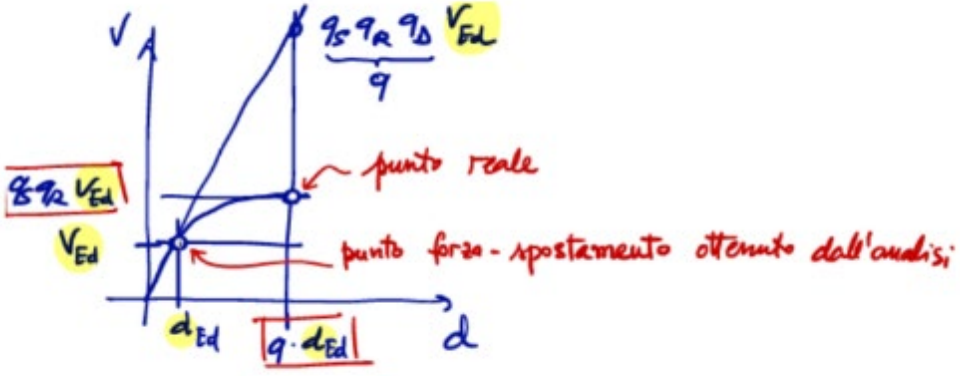
$$\xi_{\text{eff}} = \frac{\sum \xi_i E_{\text{di}}}{\sum E_{\text{di}}} \tag{5.1}$$

where E_{di} is the deformation energy induced in component i by the seismic action. Effective damping ratios may be conveniently estimated separately for each eigenmode, based on the relevant value of E_{di}.

5. MODELLING & ANALYSIS

5.1 Modelling (5/5)

- **Second-order effects**
- Sensitivity coefficient is computed differently, depending on whether analysis is linear (FBA) or NL (DBA)



5.1.3 Second-order effects

(1) Second-order effects (P-Δ effects) may be neglected if the condition given by Formula (5.3) is fulfilled in all piers.

$$\theta \leq 0{,}1 \tag{5.3}$$

where θ is the pier top displacement sensitivity coefficient, given by Formula (5.4) for the force-based approach and Formula (5.5) for the displacement-based approach, respectively.

$$\theta = \frac{P_{\mathrm{tot}} d_{\mathrm{E,p}}}{q_{\mathrm{R}} q_{\mathrm{S}} V_{\mathrm{p}} h_{\mathrm{p}}} \tag{5.4}$$

$d_{\mathrm{E,p}} = q_{\mathrm{disp}} d_{\mathrm{r}}$

$$\theta = \frac{P_{\mathrm{tot}} d_{\mathrm{E,p}}}{V_{\mathrm{p}} h_{\mathrm{p}}} \tag{5.5}$$

$d_{\mathrm{E,p}}$ from (NL) analysis

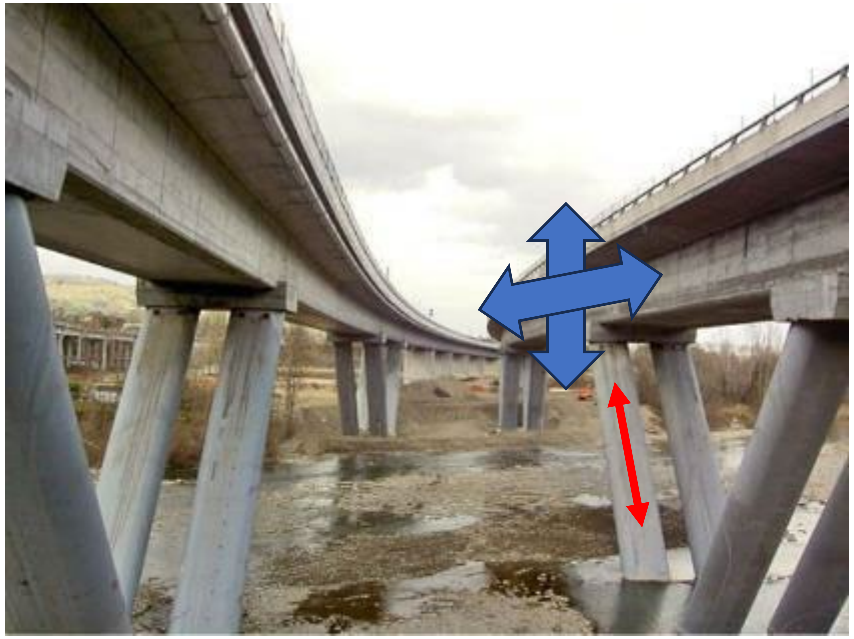
where

P_{tot}	is the total vertical force acting at the top of the pier (including the pier's upper half self-weight), due to the masses considered in the seismic analysis of the structure, in accordance with 5.4 and prEN 1998-1-1:2022, 6.2.1(3);
$d_{\mathrm{E,p}}$	is the design pier top displacement under the design seismic action, calculated in accordance with prEN 1998-1-1:2022, Formula (6.9), for the force-based approach, and that corresponding to the target displacement of the equivalent single-degree of freedom oscillator, calculated according to prEN 1998-1-1:2022, Formula (6.28) or (6.29), for the displacement-based approach;
V_{p}	is the shear force acting on the pier in the seismic design situation, as obtained in the analysis;
h_{p}	is the pier height;

5. MODELLING & ANALYSIS

5.2 Analysis (1/5)

- Vertical component
  - Consider
  - Neglect



Viaduct over the Reno river, Italy

5.2.1 General

(1) Depending on the selected method, the corresponding provisions of prEN 1998-1-1:2022, 6.4 to 6.6, should be applied.

(2) The effects of the vertical component of the seismic action should be accounted for according to 4.2.1~~(3)~~(4)

NOTE Cases where they can be neglected are given in 4.2.1~~(4)~~(5)

4.2.1 General

(4) The vertical component of the seismic action should be considered for the verification of a) to d):

a) structural members in prestressed concrete decks;

b) structural members in cable-stayed bridges;

c) antiseismic devices;

d) piers, in case of high seismic action class, if subjected to bending stresses due to vertical permanent actions of the deck, or if the bridge is located within 5 km of an active seismo-tectonic fault.

NOTE Case d) refers to inclined piers or vertical piers with monolithic connection to the deck.

(5) The effects of the vertical component may be omitted for piers in cases of low and moderate seismic action classes.

5. MODELLING & ANALYSIS

5.2 Analysis (2/5)

- Behaviour factor
- DC1: overstrength only
- DC2/DC3: also redundancy & ductility

5.2.2 Force-based approach

5.2.2.1 Behaviour factors

(1) For DC1, a behaviour factor q equal to 1,5 may be used for horizontal seismic actions, regardless of the structure.

(2) For DC2 and DC3, values of the behaviour factor components q_R and q_D, and of the behaviour factor q for horizontal seismic actions, not larger than those specified in Table 5.2 may be used, depending on the type of ductile member. The final maximum value of q should not be lower than q_S = 1,5, irrespective of all reductions in (3), (7) and (9).

Type of Ductile Members	q_R	q_D		$q = q_S\ q_R\ q_D$	
		DC2	DC3	DC2	DC3
Reinforced concrete piers:					
Multiple double-bending vertical piers *(i.e. more than one monolithically connected pier in longitudinal direction or multi-column piers in transverse direction)*	1,2	$1{,}3\lambda(\alpha_s)$	$2{,}0\lambda(\alpha_s)$	$2{,}3\lambda(\alpha_s)$	$3{,}6\lambda(\alpha_s)$
Multiple single-bending vertical piers *(i.e. more than one pin-connected pier in longitudinal direction or single-column piers in transverse direction)*	1,0	$1{,}3\lambda(\alpha_s)$	$2{,}0\lambda(\alpha_s)$	$2{,}0\lambda(\alpha_s)$	$3{,}0\lambda(\alpha_s)$
Inclined struts in bending	1,1	$1{,}0\lambda(\alpha_s)$	$1{,}3\lambda(\alpha_s)$	$1{,}6\lambda(\alpha_s)$	$2{,}1\lambda(\alpha_s)$
Steel Piers:					
Vertical piers in bending	1,1	1,3	2,2	2,1	3,6
Inclined struts in bending	1,1	1,0	1,2	1,6	2,0
Piers with normal bracing	1,1	1,1	1,5	1,8	2,5
Piers with eccentric bracing	1,3	1,3	2,2	2,1	3,6
Abutments rigidly connected to the deck:					
In general	1,1	1,0	1,1	1,6	1,8
Integral abutment bridges (see 10)	1,0	1,0	1,0	1,5	1,5
Arches	1,1	1,0	1,2	1,6	2,0

5. MODELLING & ANALYSIS

5.2 Analysis (3/5)

- Behaviour factor ≠ DC
- Reductions:
  - Foundation flexibility
  - High axial force
  - Non-accessibility of critical zones
  - Irregular inelastic demand

(15) For the application of the force-based approach, a bridge should be considered to have regular seismic behaviour in the considered horizontal direction, when the condition given in Formula (5.10) is satisfied.

$$\rho = \frac{r_{max}}{r_{min}} \le \rho_o \tag{5.10}$$

(17) Bridges that do not conform to Formula (5.10), should be considered to have irregular seismic behaviour, in the considered horizontal direction. Such bridges should either be designed using a reduced q-value given by Formula (5.12) or should be designed with the displacement-based approach in accordance with 5.2.3.

$$q' = q\frac{\rho_0}{\rho} \ge q_s \tag{5.12}$$

5.2.2 Force-based approach

5.2.2.1 Behaviour factors

(5) While the ductility class for the bridge is unique, according to 4.3.6(3), in neither curved nor skew bridges, different values of the behaviour factor q may be used in each of the two horizontal directions.

NOTE 1 4.3.6(2) implies that, in the general case of a curved or skew bridge, when different ductility is available in different directions for each supporting pier, the lower q of the primary members governs the DC. When the bridge is neither curved nor skew, an exception can be made using the higher value of q in the direction of higher available ductility, while using a lower value of q in the orthogonal, less ductile, direction.

(7) If soil-structure interaction is considered according to 5.1.1(13), a value $q_{D,SSI}$, should be calculated according to prEN 1998-1-1:2022, 6.4.1(5).

NOTE Deformation of the soil-foundation system absorbs a portion of the overall deformation reducing the inelastic part in the structure and, thus, the dissipated energy corresponding to q_D.

(8) For reinforced concrete members, the maximum values of q-factors specified in Table 5.2 should be used only if the normalized axial force η_k defined in Formula (5.7) does not exceed 0,30.

$$\eta_k = N_{Ed} / (A_c f_{ck}) \tag{5.7}$$

(9) If $0{,}30 < \eta_k \le 0{,}60$, even in a single ductile member, the value of q_D should be reduced to:

$$q_{D,N} = q_D - \frac{\eta_k - 0{,}3}{0{,}3}(q_D - 1) \tag{5.8}$$

(11) Reductions of q_D due to axial force, according to (9) or foundation flexibility, according to (7), should be cumulated, when both are required. In this case, the reduced value q'_D should be obtained as the product of q_D and the ratios $q_{D,SSI}/q_D$ and $q_{D,N}/q_D$ calculated independently according to (7) and (9).

(12) The maximum values of the q-factor for DC2 and DC3 specified in Table 5.2 may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, the values should be reduced as given by Formula (5.9).

$$q' = 0{,}6q \ge q_S \tag{5.9}$$

5. MODELLING & ANALYSIS

5.2 Analysis (4/5)

- Pushover
- This is the traditional single-mode invariant N2 version of the method
- Applicable only to straight bridges with two modes having effective modal mass larger than 60% in the two plan directions

5.2.3 Displacement-based approach

5.2.3.1 Non-linear static analysis

(1) Non-linear static analysis should be carried out according to FprEN 1998-1-1:2024, 6.5.

(2) Except as given in (3), the non-linear static analysis should not be used in the cases given in a) and b):

a) if the fundamental mode in the considered direction has effective modal mass lower than 60 %;

b) for curved bridges in compliance to 3.1.10.

NOTE Single-mode non-linear static (pushover) analysis leads to realistic results when the response of the structure to the horizontal seismic action can be reasonably approximated by a generalized single-degree of freedom system. Assuming the influence of the pier masses to be minor, the above condition is always met in the longitudinal direction of approximately straight bridges. The condition is also met in the transverse direction when the distribution of the stiffness of piers along the bridge provides an approximately uniform lateral support to a relatively stiff deck. This is the most common case for bridges where the height of the piers decreases towards the abutments or does not present intense variations. When, however, the bridge has one stiffer and unyielding pier, located between groups of regular piers, the system cannot be approximated in the transverse direction by a single-degree of freedom and pushover analysis can lead to unrealistic results. A similar exception holds for long bridges, when very stiff piers are located between groups of regular ones, or in bridges in which the mass of some piers has a significant effect on the dynamic behaviour, in either of the two directions. When possible and expedient, such irregular arrangements can be avoided, e.g. by providing sliding connection between the deck and the pier(s) that cause the irregularity.

5. MODELLING & ANALYSIS

5.2 Analysis (5/5)

- Pushover
- Permission to use alternative methods (e.g., MPA) but without guidance and with warning about limitations = when N2 is not applicable, better to resort to RHA
- N2 is simple and should stay simple: independent analyses and verifications in the tLong. And Transversal directions

5.2.3 Displacement-based approach

5.2.3.1 Non-linear static analysis

(3) As an alternative to (2), non-linear static analysis methods accounting for the response of higher modes may be used.

NOTE Modal pushover methods consist of a repetition for higher mode patterns of the standard non-linear static analysis in FprEN 1998-1-1:2024, 6.5. Those methods are not covered in this standard. Note that methods accounting for the response of higher modes have limitations and are not of general applicability.

(4) Non-linear static analysis should be carried out in the longitudinal and transverse directions of the bridge.

NOTE Verifications are carried out independently for the two analyses.

(5) The control node (reference point) should be selected as the one with maximum modal ordinate for the mode under consideration (i.e. the one with largest effective modal mass in the considered direction), according to FprEN 1998-1-1:2024, 6.5.2(4).

(6) Combination of the components of the seismic action should comply with FprEN 1998-1-1:2024, 6.5.4(6).

NOTE Formula (6.30) in FprEN 1998-1-1:2024, 6.5.4(6), considers the effect of orthogonal components of the seismic action on the single mode used in pushover.

Questions & answers

Webinar 2.1 Bridge classification & structural analysis