EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

DRAFT
prEN 1998-2

March 2023

ICS 91.120.25; 93.040

Will supersede EN 1998-2:2005

English Version

Eurocode 8 - Design of structures for earthquake resistance - Part 2: Bridges

Eurocode 8 - Calcul des structures pour leur résistance aux séismes - Partie 2: Ponts

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben - Teil 2: Brücken

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

This draft European Standard was established by CEN in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the CEN-CENELEC Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Republic of North Macedonia, Romania, Serbia, Slovakia, Slovenia, Spain, Sweden, Switzerland, Türkiye and United Kingdom.

Recipients of this draft are invited to submit, with their comments, notification of any relevant patent rights of which they are aware and to provide supporting documentation.

Warning : This document is not a European Standard. It is distributed for review and comments. It is subject to change without notice and shall not be referred to as a European Standard.

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

CEN-CENELEC Management Centre: Rue de la Science 23, B-1040 Brussels

Ref. No. prEN 1998-2:2023 E

Contents

Page

European foreword

This document (prEN 1998-2:2022) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document will supersede EN 1998-2:2005.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

- EN 1990 Eurocode: Basis of structural and geotechnical design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite steel and concrete structures
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design of structures for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures
- New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1998 Eurocode 8

EN 1998 defines the rules for the seismic design of new buildings and engineering works and the assessment and retrofit of existing ones, including geotechnical aspects, as well as temporary structures.

NOTE This standard also covers the verification of structures in the seismic situation during construction, when required.

Attention has to be paid to the fact that, for the design of structures in seismic regions, the provisions of EN 1998 should be applied in addition to the relevant provisions of EN 1990 to EN 1997 and EN 1999. In particular, EN 1998 should be applied to structures of consequence classes CC1, CC2 and CC3, as defined in prEN 1990:2021, 4.3. Structures of consequence class CC4 are not fully covered by the Eurocodes but may be required to follow EN 1998, or parts of it, by the relevant authorities.

By nature, perfect protection (a null seismic risk) against earthquakes is not feasible in practice, in particular because the knowledge of the hazard itself is characterized by a significant uncertainty. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquakes likely to occur at a given location and representative of their frequency content. This representation is not the prediction of a particular seismic movement, and such a movement could give rise to more severe effects than those of the seismic action considered, inflicting damage greater than the one described by the Limit States contemplated in this Standard.

Not only the seismic action cannot be predicted but, in addition, it should be recognized that engineering methods are not perfectly predictive when considering the effects of this specific action, under which structures are assumed to respond in the nonlinear regime. Such uncertainties are taken into account according to the general framework of EN 1990, with a residual risk of underestimation of their effects.

0.3 Introduction to EN 1998-2

EN 1998-2 provides general requirements for earthquake resistant design of new bridges. Except where otherwise specified in this Part, the seismic actions are as defined in prEN 1998-1-1:2022, 5. The scope of this Part of EN 1998 is defined in 1.1.

Since the seismic action is mainly resisted by the piers and the latter are usually constructed of reinforced concrete, a greater emphasis has been given to such piers. Additionally, bearings are in many cases important parts of the seismic resisting system of a bridge and are therefore treated accordingly. The same holds for seismic isolation devices.

EN 1998-2 is subdivided in ten clauses and includes four annexes, where Annexes A to C are informative and Annex D is normative.

0.4 Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National annex for EN 1998-2

National choice is allowed in this document where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing EN 1998-2 can have a National Annex containing all national choices to be used for the design of new bridges to be constructed in the relevant country.

When no national choice is given, the default choice given in this document is to be used.

When no national choice is made and no default is given in this document, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in EN 1998-2 through notes to the following:

4.1(4)	4.2.1(1)	4.3.5(8)	4.3.7(1)
6.3.2(2)			

National choice is allowed in EN 1998-2 on the application of the following informative annexes:

Annex A	Annex B	Annex C

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of EN 1998-2

(1) This document is applicable to the design and verification of new bridges in seismic regions. It gives general rules for the design and verification relevant to bridges of consequence classes CC1, CC2 and CC3, as defined in prEN 1990:2021, A.2.

NOTE 1 EN 1998-2 covers the design of reinforced concrete, steel and composite steel-concrete bridges, with the exception of prestressed piers. Guidance for design of timber bridges is given in Informative Annex C.

NOTE 2 The assessment of existing bridges is covered in EN 1998-3.

(2) Unless specifically stated, prEN 1998-1-1:2022 and prEN 1998-5:2022 apply.

(3) EN 1998-2 is applicable in complement to the other relevant Eurocodes.

NOTE EN 1998-2 contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, are used for the design of new bridges in seismic regions. EN 1998-2 complements in this respect the other Eurocodes.

(4) EN 1998-2 provides basic performance requirements and compliance criteria applicable to new bridges in seismic regions.

(5) EN 1998-2 is applicable to the seismic design of bridges exploiting ductility in structural members or through the use of antiseismic devices.

(6) EN 1998-2 gives detailing rules for ductility of the structural members in bridges designed to exploit ductility as a means of seismic protection. When ductility is exploited, EN 1998-2 primarily covers bridges in which the horizontal seismic actions are mainly resisted through bending of the piers or at the abutments, i.e. of bridges composed of vertical or nearly vertical pier systems supporting the traffic deck superstructure.

(7) EN 1998-2 gives specific rules for bridges equipped with antiseismic devices, for cable-stayed and extradosed bridges and for integral abutment bridges.

(8) EN 1998-2 is also applicable to the seismic design of arched bridges, although its provisions should not be considered as fully covering these cases.

NOTE Suspension bridges and masonry bridges, moveable bridges and floating bridges are not included in the scope of this Part.

1.2 Assumptions

(1) The assumptions of prEN 1998-1-1:2022, 1.2, are assumed to be applied.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

prEN 1998-1-1:2022 *Eurocode 8 – Design of structures for earthquake resistance – Part 1-1: General rules and seismic action*

prEN 1998-5:2022, *Eurocode 8 – Design of structures for earthquake resistance – Part 5: Geotechnical aspects, foundations, retaining and underground structures*

ISO 80000 (all parts), *Quantities and units*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, prEN 1998-1-1:2022, 3.1 and the following apply.

3.1.1
positive linkage
connection implemented by seismic links

3.1.2
spatial variability (of seismic action)
situation in which the ground motion at different supports of the bridge differs and, hence, the seismic action cannot be based on the characterisation of the motion at a single point

3.1.3
longitudinal and transverse directions of the bridge
the longitudinal direction *x* is defined by the line connecting the centres of the two end-sections of the deck. The transverse direction *y* is assumed to be orthogonal to the longitudinal direction

Note 1 to entry: In skew bridges, the above defined horizontal directions generally do not coincide with the bearings' principal axes of inertia, which can underestimate seismic effects if the two directions are considered independently. For this reason, it is important that the skew is properly accounted for in the numerical model and that the two horizontal directions of seismic action are properly combined.

3.1.4
seismic links
restrainers through which part or all of the seismic action may be transmitted. Used in combination with bearings, they can be provided with appropriate slack, so as to be activated only in the case when the design seismic displacement is exceeded

3.1.5
minimum overlap length
safety measure in the form of a minimum distance between the inner edge of the supported and the outer edge of the supporting member. The minimum overlap is intended to ensure that the function of the support is maintained under extreme seismic displacements

3.1.6
design seismic displacement
displacement induced by the design seismic actions

3.1.7
total design displacement in the seismic design situation
displacement used to determine adequate clearances for the protection of critical or major structural members. It includes the design seismic displacement, the displacement due to the long-term effect of the permanent and quasi-permanent actions and an appropriate fraction of the displacement due to thermal movements

3.1.8
critical region, critical zone
region/zone of a primary seismic member, where the most adverse combination of action effects (M, N, V, T) occurs and where plastic hinges can form

Note 1 to entry: In concrete bridge piers, critical regions are potential dissipative zones such as defined in prEN 1998-1-1:2022, 3.1.10. The length of the critical region is defined in 7.2.1.

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°. All other bridges are considered straight

3.1.11
ductile member
primary seismic member where a plastic hinge can form

3.2 Symbols and abbreviations

3.2.1 General

The symbols and abbreviations listed in prEN 1990:2021, 3.2 and in prEN 1998-1-1:2022, 3.2, apply.

For the symbols related to materials, as well as for symbols not specifically related to the seismic situation, the provisions of the relevant Eurocodes should be applied.

Further symbols and abbreviations, used in connection with seismic actions, are defined in the present standard where they occur, for ease of use. However, in addition, the most frequently occurring symbols used in EN 1998-2 are listed and defined in 3.2.2 and additional abbreviations are given in 3.2.3.

3.2.2 Symbols

3.2.2.1 Symbols used in 4

3.2.2.1.1 Lower case Latin symbols

d_E Design seismic displacement (due only to the design seismic action)

d_{Ed} Total design displacement in the seismic design situation

d_G Long-term relative displacement due to permanent and quasi-permanent actions

d_T Displacement due to thermal movements

k_u, k_d Stiffness of timber fasteners or connectors

3.2.2.1.2 Lower case Greek symbols

η_k Normalised axial force

ψ_2 Combination factor for the quasi-permanent value of the thermal action

3.2.2.2 Symbols used in 5

3.2.2.2.1 Upper case Latin symbols

A_c Concrete area of the cross-section

B Width of the deck

E_d Seismic action effects

E_{di} Deformation energy induced in component *i* by the seismic action

E_d^S Quasi-static part of the seismic action effect

E_d^D Dynamic part of the seismic action effect

E_{dk}^S Contribution of the *k*-th static mode under the peak ground displacement at support *k*.

$E_{d,i}$ Seismic action effects due to higher quasi-antisymmetric modes

$E_{d,u}$ Seismic action effects due to uniform excitation

E_{di}^D Contribution of the *i*-th mode under the design seismic action

E_{dik}^D *i*-th mode response to the seismic input (response spectrum) at the *k*-th support

F Horizontal force

F_i Static force on pier *i* in the lateral forces method

F_i Static forces due to the contribution of higher quasi-antisymmetric modes

F_b Seismic base shear force

L Total length of the continuous deck

L_{lim} Total length of the bridge

M Total bridge mass above the foundations

M_1 Equivalent modal mass

$M_{Ed,i}$ Maximum value of design moment at the intended plastic hinge location of ductile member *i* as derived from the analysis for the seismic design situation

$M_{Rd,i}$ Design flexural resistance of the same section with its actual reinforcement under the concurrent action of the non-seismic action effects in the seismic design situation

M_t Equivalent static moment

N_{Ed} Axial force at the plastic hinge seismic design situation

N_S, N_D Number of static and dynamic modes used in the analysis of long bridges on non-uniform soil

P_{tot} Total vertical force acting at the top of the pier

$Q_{k,i}$ Variable gravity loads as appearing in the seismic design situation

$\overline{SF}_i$ Mode amplification factor

T_1 Fundamental period in the considered direction

T_i Fundamental period of the *i*-th pier or *i*-th modal period from modal analysis

V_p Shear force acting on the pier in the seismic design situation

3.2.2.2.2 Lower case Latin symbols

a_s Shear span ratio ($=L_V/h$)

$d_{E,p}$ Design pier top displacement under the design seismic action

d_m Average of the piers top displacements under a transverse uniformly distributed load on the deck

e Total eccentricity ($e_a + e_d$)

e_o Theoretical eccentricity between the centre of stiffness of the supporting members and the centre of mass of the deck

e_a Accidental eccentricity

e_d Additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration

f_{ck} Characteristic concrete strength

h Depth of basin or pier height

q' Reduced value of q-factor

$q_{D,N}$ Reduced value of the ductility-related q-factor component due to axial force

$q_{D,SSI}$ Reduced value of the ductility-related q-factor component due to soil-structure interaction

m_i Mass over the *i*-th support

r_i Parameter defined as $r_i = q\dfrac{M_{Ed,i}}{M_{Rd,i}}$

r_{ij} Correlation coefficient between dynamic modes

r_k Vector collecting the *k*-th static mode

r_{min} Minimum value of r_i among all ductile members *i*

r_{max} Maximum value of r_i among all ductile members *i*

s_i Displacement over the *i*-th support in the horizontal direction when the structure is acted upon by the acceleration of gravity

3.2.2.2.3 Upper case Greek symbols

Γ_i *i*-th modal participation factor due to spatially variable excitation

Δd Maximum difference in displacement between any two pier tops under a transverse uniformly distributed load on the deck

3.2.2.2.4 Lower case Greek symbols

η Damping correction factor for the elastic response spectrum

η_k Normalised axial force

θ Pier top displacement sensitivity coefficient

λ Factor for the calculation of behaviour factor q

ξ Equivalent viscous damping ratio

ξ_i Equivalent viscous damping ratio of component i

ξ_{eff} Effective viscous damping of the structure

ρ, ρ_o Parameters for regular seismic behaviour

ρ_{kl} Correlation coefficient between seismic input motion at different supports

φ Parameter for calculating ψ_{Ei} or skew angle (angle between the longitudinal axis of the bridge and a line perpendicular to the alignment of intermediate or end supports)

φ_i *i*-th modal shape from modal analysis

$\psi_{E,i}$ Combination coefficients

3.2.2.3 Symbols used in 6

3.2.2.3.1 Upper case Latin symbols

A_{Ed} Design action effects in the seismic design situation

$A_{j,eff}$ Effective area of the joint

M_{Ed} Design moment in the seismic design situation

M_o Overstrength moment

M_{Rd} Design value of the flexural resistance of the section

N_{cG} Axial force of the pier under the non-seismic actions in the seismic design situation

N_{Ed} Axial force in the seismic design situation

N_{jz} Vertical axial joint force

N_{jx} Horizontal axial joint force

N_{jy} Horizontal axial joint force in the transverse direction

T_{Rc} Resultant force of the tensile reinforcement of the pier corresponding to the design flexural resistance, M_{Rd}, of the plastic hinge

V_{b1C} Shear force of the horizontal member adjacent to the tensile face of the pier, corresponding to the capacity design effects of the plastic hinge

$V_{Rdj,min}$ Minimum joint shear resistance

V_{jx} Design horizontal shear force of the joint

V_{jz} Design vertical shear force of the joint

3.2.2.3.2 Lower case Latin symbols

b_c, Cross-section width of the pier

b_j, Effective width of the joint

b_w Cross-section width of the web of the deck

d_c Diameter of circular pier

f_{ctd} Design value of the tensile strength of concrete

$f_{yd,h}$ Design value of the tensile strength of the horizontal reinforcement in the joint

h_b Cross-section depth of the “beam” (e.g. deck)

h_c Cross-section depth of the pier

l_{cr} Critical region length

3.2.2.3.3 Lower case Greek symbols

n_x, n_y, n_z Joint axial stresses

v_x, v_y, v_z Joint shear stresses

z_c, z_b Internal lever arms of pier and deck, respectively

β Angle between the vertical and the diagonal of the joint

γ_{Rd} Overstrength factor

η_k Normalised axial force

θ Chord rotation

σ_{sh} Stress in the horizontal reinforcement in the joint

ω_{rm} Material randomness factor

ω_{sh} Strain hardening factor

3.2.2.4 Further symbols used in 7

3.2.2.4.1 Upper case Latin symbols

A_c Area of the gross concrete section

A_{cc} Confined (core) concrete area of the section to the hoop centre line

A_{sp} Area of the spiral or hoop bar

A_{sx} Area of the horizontal joint reinforcement

A_{sz} Area of the vertical joint reinforcement

A_{sw} Total area of a layer of hoops or ties in the one direction of confinement

A_t Area of one tie leg

D_{sp} Diameter of the spiral or hoop bar

D_i Internal diameter of hollow circular piers

N_{Ed} Axial force in the seismic design situation

3.2.2.4.2 Lower case Latin symbols

b Dimension of the concrete core perpendicular to the direction of confinement under consideration, measured to the outside of the perimeter hoop

b_{min} Smallest dimension of the concrete core

d_{bL} Diameter of longitudinal bar

f_{ck} Characteristic concrete strength

f_{sy} Design value of yield strength of the joint reinforcement

f_{ys} Yield strength of the longitudinal reinforcement

f_{yt} Yield strength of the tie

h Thickness of walls making up the cross-section of hollow-core piers

l_{cr} Critical region length

s_L Spacing of hoops or ties in the longitudinal direction

s_T Transverse distance between hoop legs or supplementary cross-ties

3.2.2.4.3 Upper case Greek symbols

ΔA_{sx} Area of horizontal joint reinforcement placed outside the joint body

ΔA_{sz} Area of vertical joint reinforcement placed outside the joint body

3.2.2.4.4 Lower case Greek symbols

ε_{cu2} Maximum compressive strain in the concrete

η_k Normalised axial force

ρ_l Longitudinal reinforcement ratio

ρ_{lz} Vertical stirrups reinforcement ratio

ρ_{min} Minimum joint reinforcement ratio

ρ_x Ratio of horizontal reinforcement in the joint

ρ_y Reinforcement ratio of closed stirrups in the transverse direction of the joint panel (orthogonal to the plane of action)

ρ_w Transverse reinforcement volumetric ratio

ρ_z Ratio of vertical reinforcement in the joint

ϕ_y, ϕ_u Curvature at yield and ultimate

ω_{wd} Mechanical reinforcement ratio of confining reinforcement

$\omega_{w,req}$ Mechanical ratio of minimum confinement

3.2.2.5 Symbols used in 8

3.2.2.5.1 Upper case Latin symbols

H_i	Height of pier *i*
K_{bi}	Effective stiffness of isolator of pier *i*
K_{eff}	Effective stiffness of the isolation system in the principal horizontal direction under consideration
$K_{eff,i}$	Composite stiffness of isolator units and the corresponding pier *i*
K_{fi}	Rotational stiffness of the foundation of pier i
K_{si}	Displacement stiffness of pier *i*
K_{ti}	Translational stiffness of the foundation of pier *i*
K_{xi}, K_{yi}	Effective composite stiffnesses of isolator unit and pier *i* in the *x* and *y* directions, respectively
L_{eff}	Effective length of the deck
L_g	Distance parameter
M_d	Mass of the superstructure (deck)

3.2.2.5.2 Lower case Latin symbols

d_{cd}	Displacement at the centre of mass of the superstructure (deck)
d_{eg}	Effective displacement due to the spatial variation of the seismic ground displacement
d_g	Expected ground displacement under the design seismic action
d_{Ed}	Total seismic design displacement
d_{es}	Effective seismic displacement of the support due to the deformation of the structure
d_{id}	Superstructure displacement over pier *i*
e_x	Eccentricity in the longitudinal direction
l_m	Minimum support length
l_{ov}	Minimum overlap length
r	Radius of gyration of the deck mass about the vertical axis through its centre of mass
s	Slack of the seismic link
x_i, y_i	Coordinates of pier *i* relative to the effective stiffness centre

3.2.2.5.3 Lower case Greek symbols

δ_i	Amplification factor for the superstructure displacement over pier i
ξ_{eff}	Effective damping of the isolation system

3.2.2.6 Symbols used in 10

3.2.2.6.1 Upper case Latin symbols

E_d Total earth pressure acting on the abutment in the seismic design situation

K_o At-rest pressure coefficient

K_{PE} Passive pressure coefficient in the seismic design situation

$K_{PE,mob}$ Mobilised passive pressure coefficient at depth z from the top of the abutment

3.2.2.6.2 Lower case Latin symbols

i_u Function that interpolates the pressure coefficient between at-rest and passive values

u Abutment displacement at depth z from the top of the abutment

a Non-dimensional soil-dependent parameter

d_{lim} Design seismic displacement limit

3.2.2.6.3 Lower case Greek symbols

γ Weight density of soil or backfill material behind the abutment

$\sigma_{p,mob}$ Mobilised passive pressure at depth z from the top of the abutment

3.2.2.7 Symbols used in Annex B

3.2.2.7.1 Upper case Latin symbols

R Radius of the immersed pier (circular cross section)

3.2.2.7.2 Lower case Latin symbols

k Mass coefficient of rectangular immersed pier

m_a Total effective mass in a horizontal direction of immersed pier

a_x, a_y Dimensions of immersed pier (elliptical or rectangular cross section)

3.2.2.7.3 Lower case Greek symbols

θ Angle of horizontal seismic action

ρ Water density

3.2.2.8 Symbols used in Annex C

l Bridge span limit

3.2.2.9 Symbols used in Annex D

3.2.2.9.1 Upper case Latin symbols

$A_{co}(z)$ Contact area between structure and soil or backfill material at depth z

E_s Young's modulus of the soil

G_{emb} Embankment material shear modulus

H_{ab} Abutment height

H_{emb} Embankment height

L Horizontal characteristic length

L_a Horizontal characteristic length for active condition

L_p Horizontal characteristic length for passive condition

PGV_e Design peak value of horizontal ground velocity

S_{De} Spectral displacement at the fundamental period of the embankment vibrating in the longitudinal direction of the bridge

T_{emb} Fundamental period of the embankment vibrating in the longitudinal direction of the bridge

3.2.2.9.2 Lower case Latin symbols

k_a Stiffness for displacement towards the active limit pressure

k_p Stiffness for displacement towards the passive limit pressure

3.2.2.9.3 Lower case Greek symbols

ρ_{emb} Embankment material mass density

σ_o Initial pressure

σ_a Active pressure resistance limit

σ_p Passive pressure resistance limit

$\boldsymbol{\phi}$ First-mode shape of the embankment in the longitudinal direction of the bridge

ψ_2 Combination factor for the quasi-permanent value of thermal action

3.2.3 Abbreviations

MSRS Multiple Support Response Spectrum

UBDP Upper bound design properties of device

LBDP Lower bound design properties of device

3.3 S.I. Units

S.I. Units in accordance with ISO 80000 shall be used.

For calculations, the following units should be used when applicable:

- mass: kg, t
- mass density: kg/m³, t/m³
- forces and loads: kN, kN/m, kN/m²
- weight density: kN/m³
- stresses and strengths: Pa (= N/m²), kPa (= kN/m²), MPa (= MN/m²)
- moments (bending, etc.): kNm
- acceleration: m/s²

4 Basis of design

4.1 Basic requirements

(1) Consequence class CC3 should be divided into CC3a and CC3b according to prEN 1998-1-1:2022, 4.2(3).

NOTE The definition of consequence classes for bridges is given in prEN 1990:2021, Table A.2.1(NDP).

(2) For the application of prEN 1998-1-1:2022, 4.1(4), δ values should be determined.

NOTE The values of δ applicable to bridges are those in Table 4.1 (NDP), unless the relevant authorities or the National Annex give different values for use in a country.

Table 4.1 (NDP) — δ values for bridges

	Consequence class			
	CC1	CC2	CC3a	CC3b
δ	0,6	1,0	1,25	1,6

4.2 Seismic actions

4.2.1 General

(1) prEN 1998-1-1:2022, 4.3, should be applied.

NOTE Values of return period $T_{LS,CC}$ or performance factor $\gamma_{LS,CC}$ are those given in Table 4.2(NDP) and Table 4.3(NDP), respectively, unless the relevant authorities or the National Annex give different values for use in a country.

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

Limit state	Consequence class			
	CC1	CC2	CC3a	CC3b
NC	800	1600	2500	5000
SD	250	475	800	1600
DL	50	60	100	200

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

Limit state	Consequence class			
	CC1	CC2	CC3a	CC3b
NC	1,2	1,5	1,8	2,2
SD	0,8	1,0	1,25	1,5
DL	0,5	0,5	0,6	0,7

NOTE The values of the performance factor in Table 4.3 correspond to the ratios of the intensities characterized by the return periods in Table 4.2 to the intensity characterized by the reference return period for CC2 and SD equal to 475 years, calculated with a value of the hazard slope $k = 3$, appropriate for the seismicity corresponding to the Moderate seismic action class, using the expression $\gamma_{LS,CC} = \left(\frac{T_{LS,CC}}{T_{SD,2}}\right)^{\frac{1}{k}}$. See also prEN 1998-1-1:2022, 5.2.1(2).

(2) The seismic action should be taken as given in prEN 1998-1-1:2022, 5.2.

(3) The vertical component of the seismic action should be considered for the verification of the structural members in a) to d):

a) prestressed concrete decks;

b) 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.

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

(5) 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 prEN 1998-1-1:2022, 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.

(6) When included in the model, soil-structure interaction (SSI) should conform to prEN 1998-5:2022, 8.

NOTE The seismic action application depends on the adopted model of SSI.

4.2.2 Spatial variability of the seismic action

(1) Spatial variability of earthquake ground motion should be considered according to Table 5.3, when any of the conditions in a) to c) holds:

a) The maximum and minimum values of the average shear wave velocity $V_{s,H}$ calculated for the soil profiles under each bridge support (piers and abutments) differ by more than 200 m/s.

b) The total length of the bridge exceeds L_{lim}, equal to the smallest characteristic length value L_g among all bridge supports as given in prEN 1998-1-1:2022, 5.2.3.2(3).

c) The maximum span length between two successive supports exceeds 50 m (for bridges having two spans or more).

NOTE At abutments, the evaluation of $V_{s,H}$ involves only the foundation soil, not the embankment/backfill.

(2) When spatial variability of earthquake ground motion is not considered but the supports rest on soil belonging to different site categories, the seismic action for the most demanding category should be used.

NOTE Short to medium length bridges with span length not exceeding 50 m, can span over moderately varying soils with difference in $V_{s,H}$ at different supports lower than 200 m/s but leading to different site classification.

4.3 Characteristics of earthquake resistant bridges

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 prEN 1998-1-1:2022, 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 6.

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

(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 torsional effects.

(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, which should be 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 1998-41, 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 prEN 1998-5:2022, 7.2.

(9) The liquefaction potential of the foundation soil should be investigated in accordance with prEN 1998-5:2022, 7.3.

(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.

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.

(2) All primary structural members should be modelled in the structural analysis, designed and detailed for earthquake resistance in accordance with 5 to 10.

(3) In addition to compliance with their own seismic design requirements, the secondary seismic members and their connections should be designed and detailed to maintain support of non-seismic actions in the seismic design situation when subjected to the displacements caused by the most unfavourable seismic design condition.

NOTE In addition to those of EN 1992, EN 1993, EN 1994, EN 1995, EN 1996 and EN 1999, Clauses 5 to 10 give rules for the analysis, design and detailing of secondary seismic members.

(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 10).

NOTE 1 Culverts are a special case of this protection strategy.

NOTE 2 Eliminating joints reduces maintenance costs and increases durability.

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) The location of areas of potential or expected seismic damage in addition to critical zones, should be identified and the difficulty of repairs should be minimized.

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

(3) 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).

(4) An appropriate hierarchy of resistance should exist within the various structural components. This should be achieved by designing all members against all brittle modes of failure as well as all members or parts of members intended to remain elastic, using “capacity design effects” as specified in 6.3.2.

NOTE (4) is to ensure that the intended configuration of plastic hinges as stated in (3) will form and that brittle failure modes are avoided.

(5) Plastic hinges should not be formed in reinforced concrete members where the normalized axial force η_k defined in Formula (5).7) exceeds 0,6.

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

(7) 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.

(8) Prestressed members should be protected from formation of plastic hinges under the design seismic action.

NOTE 1 This standard does not contain rules for ductility of such members.

NOTE 2 This standard does not contain rules for post-tensioning of piers used as a seismic protection system (post-tensioning to provide a conventional cast-in-place pier or a precast pier rocking on the foundation with a recentering force).

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 design seismic displacements with increased reliability.

(2) (1) should be ensured by designing connections used for securing structural integrity according to 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 total design value of the relative displacement in the seismic design situation, d_{Ed}, determined as given by Formula (4).1).

$$d_{\mathrm{Ed}} = d_{\mathrm{G}} \text{ "+" } d_{\mathrm{E}} \text{ "+" } \psi_2 d_{\mathrm{T}} \tag{4.1}$$

where

d_G	is the governing value between the short-term (opening) and long-term (end of design service life) values of the relative displacement due to the permanent and quasi-permanent actions (e.g. post-tensioning, shrinkage and creep for concrete decks);
d_E	is the design seismic relative displacement;
d_T	is the displacement due to thermal movements;
ψ_2	is the combination factor for the quasi-permanent value of thermal action, given in prEN 1990:2021, Table A.2.7(NDP);
"+"	means combined with + or − sign to obtain the most unfavourable effect.

(3) If abutment displacements towards the deck are larger or equal than the smaller of the displacements 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 total design value of the displacement in the seismic design situation.

(5) The design seismic relative displacement, d_E, between two independent sections of a bridge, defined in (2), may be estimated as the square root of the sum of squares of the values of the design seismic displacement calculated for each section.

(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 total design value of the relative displacement in the seismic design situation, d_{Ed}.

(7) Ancillary elements of importance for crisis management (e.g. bridge equipment such as carried water pipes, electric or gas lead), expected not to be damaged in the seismic design situation at SD limit state, should be designed and detailed accordingly.

NOTE Such ancillary elements are defined by the relevant authorities. Continuity of service of essential utilities such as water and power is key to a faster recovery in the aftermath of an earthquake.

(8) The detailing of ancillary elements not addressed in (7) (e.g. deck movement joints, bridge equipment such as holding devices, noise barriers, lighting masts, directional portal frames, and abutment back-walls) should cater for a predictable mode of damage, minimize risks to persons in case of failure and provide for the possibility of repair. Clearances should accommodate the design seismic displacement at DL limit state and the movement due to temperature, creep and shrinkage.

NOTE At joints of railway bridges, transverse differential displacement can be either avoided or limited to values appropriate for preventing derailment. Limiting values can be found in the National Annex or be agreed with the railway operating authority.

(9) Anchorage of ancillary elements to structural members should be designed according to prEN 1998-1-1:2022, Annex F.

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 8, 9, 10 and Annex C respectively.

(2) The primary structure should be assigned a ductility class according to prEN 1998-1-1:2022, 4.4.2(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 and the same in all directions.

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

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

4.3.7 Simplified criteria

(1) Non-frame culverts, i.e. pipe culvert and single- or multi-cell box culvert, may be designed according to prEN 1998-1-1:2022, 4.1(7), with the further provisions given in 10.3.4(3).

(2) In cases of low seismic action class, simplified design criteria may be established for certain types of bridges.

NOTE 1 The selection of the categories of bridges and ground types for which the simplified criteria apply and the corresponding rules can be found in the National Annex for use in a Country.

NOTE 2 In some countries, this selection and the associated rules are given by the relevant authorities.

5 Modelling and structural analysis

5.1 Modelling

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	$\boldsymbol{\psi_{E,i}}$
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 per unit length of 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.

(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) For the force-based approach, the elastic flexural stiffness of members designed to DC1 may be taken equal to that of the uncracked section for the calculation of seismic action effects in terms of generalized stresses. (6) should be applied for the calculation of seismic action effects in terms of generalized deformations, including evaluation of second-order effects according to 5.1.3.

NOTE DC1 members do not attain the yield resistance in the seismic design situation. The cracking moment, however, even accounting for its increased value due to axial force in piers, can be attained and surpassed leading to cracking. For this reason, it is not conservative to assume uncracked stiffness for displacement calculations. One exception is the case of vertically prestressed piers (not covered in this standard, see note 1 to 1.1(1)).

(8) In concrete decks consisting of precast concrete beams and cast *in situ* slabs, continuity slabs (see 4.3.3(7)) 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 on the basis of 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.

(9) 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.

(10) When elastic response spectrum analysis or response-history analysis are used, the following values of equivalent viscous damping ratio ξ may be assumed, on the basis of 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.

(11) 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).

$$\xi_{eff} = \frac{\sum \xi_i E_{di}}{\sum E_{di}} \quad (5.1)$$

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, on the basis of the relevant value of E_{di}.

(12) For nonlinear analyses (see 5.2.3 and 5.2.3.2), the model should comply with prEN 1998-1-1:2022, 6.2.3.

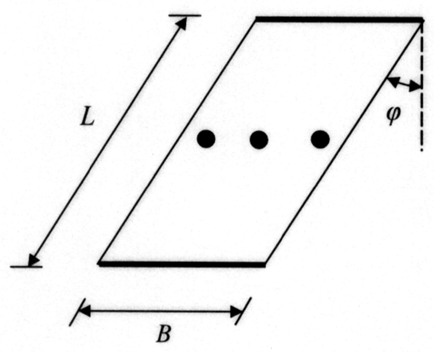
(13) Soil-structure interaction effects should be considered according to prEN 1998-5:2022, 8.1(5) and (6), using appropriate numerical models depending on the analysis method as prescribed in prEN 1998-5:2022, 8.

(14) In cases in which it is difficult to estimate reliably the mechanical properties of the soil, the analysis should be carried out using upper and lower bound estimates. High estimates of soil stiffness should be used for calculating the internal forces and low estimates for calculating the displacements of the bridge.

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 φ > 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.



Key

L total length of the continuous deck

B width of the deck

φ skew angle

Figure 5.1 — Skew bridge

(2) When using the lateral force method (see 5.2.2.2) for the design of skew bridges, the equivalent static moment given by Formula (5).2) should be considered to act about the vertical axis at the centre of gravity of the deck:

$$M_{t} = \pm F_{b}\, e \tag{5.2}$$

where

F_b	is the horizontal force determined in accordance with Formula (5).14);
e	$= e_a + e_d$;
e_a	$= 0{,}03L$ or $0{,}03B$, is the accidental eccentricity of the mass;
e_d	$= 0{,}1\ L \sin\varphi$ or $0{,}1\ B \sin\varphi$, is an additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration.

(3) For the calculation of e_a and e_d in (2), the dimension L or B transverse to the direction of excitation should be used.

(4) When using a dynamic method (response spectrum method or response-history analysis), the dynamic part of the torsional excitation may be taken into account by either a) or b):

a) displacing the centre of mass by the accidental eccentricity e_a in the most unfavourable direction and sense;

b) using the static torsional moment given by Formula (5).2).

(5) 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, taking into account 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.

5.1.3 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.

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

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

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);

$d_{E,p}$ 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 is the pier height;

q_S and q_R are the behaviour factor components according to prEN 1998-1-1:2022, 6.4.1(1), specified in 5.2.2.1.

NOTE The shear force V_p for the displacement-based approach includes the effect of overstrength and redistribution due to redundancy accounted for by q_S and q_R in the force-based approach.

(2) If $0,1 < \theta \leq 0,2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

(3) If $\theta > 0,2$ at any pier, the second-order effects should be taken into account directly by using established methods of second-order analysis which account for geometric nonlinearity, i.e. consider the equilibrium conditions on the deformed structure.

(4) The value of θ should not exceed 0,3.

NOTE Significant second-order effects can occur in bridges with slender piers and in special bridges, like arch and cable-stayed bridges.

5.2 Methods of analysis

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).

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

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 structural type and ductility class. The final maximum value of q should not be lower than $q_S = 1,5$, irrespective of all reductions in (3), (7) and (9).

NOTE 1 Use of behaviour factor components values less than the maximum allowable specified in Table 5.2 will lead to reduced ductility demands, implying a reduction of damage in the seismic design situation. Such a use can be decided for a specific project by the relevant parties.

NOTE 2 The values of q presented in Table 5.2 are obtained as the product of the default values of q_S, q_R and q_D, where q_S is taken equal to 1,5.

NOTE 3 Plastic hinges in continuity slabs, defined in 4.3.3(7), do not represent a dependable source of energy dissipation and do not concur to the value of q_D.

NOTE 4 Arches include bridges where the seismic action is mainly resisted by the arch, resulting in a total (gravity plus seismic) compression which is high and that limits ductility (due to high concrete compression in reinforced concrete arches or buckling in steel tubular arches).

Table 5.2 — Maximum values of the behaviour factor *q* for horizontal seismic actions (for DC2 and DC3)

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	$1{,}3\lambda(a_s)$	$2{,}0\lambda(a_s)$	$2{,}3\lambda(a_s)$	$3{,}6\lambda(a_s)$
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	$1{,}3\lambda(a_s)$	$2{,}0\lambda(a_s)$	$2{,}0\lambda(a_s)$	$3{,}0\lambda(a_s)$
Inclined struts in bending	1,1	$1{,}0\lambda(a_s)$	$1{,}3\lambda(a_s)$	$1{,}6\lambda(a_s)$	$2{,}1\lambda(a_s)$
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

(3) The factor $\lambda(a_s)$ in Table 5.2 should be as defined by Formula (5).6).

$$\lambda(a_s) = \begin{cases} \sqrt{\dfrac{1}{3}} & if\ a_s < 1 \\ \sqrt{\dfrac{a_s}{3}} & if\ 3 > a_s \geq 1 \\ 1 & if\ a_s \geq 3 \end{cases} \tag{5.6}$$

where $a_s = L_V/h$ is the shear span ratio, ratio between the distance from the plastic hinge to the point of zero moment and the cross-section height in the plane of deformation.

(4) In DC2 and DC3, for reinforced concrete piers with rectangular cross section when the compression zone has triangular shape, under the seismic action in the global direction under consideration, the minimum of the values of a_s, corresponding to the two sides of the section, should be used.

NOTE The compression zone has a triangular shape in rectangular sections when, for example, the pier's axes are rotated with respect to the considered direction, as it happens in skew or curved bridges.

(5) While the ductility class for the bridge is unique, according to 4.3.6(3), in straight bridges (neither curved or skew), 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 straight, 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.

NOTE 2 This can be the case for reinforced concrete wall piers or some type of timber piers, which have markedly different behaviour and ductility in the two directions.

(6) If a bridge has different types of ductile members, the behaviour factor q corresponding to the type (i.e. group) with the largest contribution to global horizontal base shear should be used.

(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.

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

where

N_{Ed}	is the value of the axial force at the plastic hinge seismic design situation, positive if compressive;
A_c	is the concrete area of the cross section;
f_{ck}	is the characteristic concrete strength.

(9) If 0,30 < η_k ≤ 0,60, even in a single ductile member, the value of q_D should be reduced to:

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

where η_k is the largest value of the normalized axial force amongst the primary seismic members.

(10) A value for $q_{D,N}$ = 1,0 (elastic behaviour) should be used for bridges in which the seismic force resisting system contains members with η_k > 0,6.

(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).

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

NOTE The term “accessible”, as used in the paragraph above, has the meaning of “accessible even with reasonable difficulty”. The foot of a pier shaft located in backfill, even at substantial depth, is considered to be “accessible”. On the contrary, the foot of a pier shaft immersed in deep water, or the heads of piles beneath a large pile cap, are in general not considered as “accessible”.

(13) Bridges where the main part of the design seismic action is resisted by elastomeric bearings, should be designed according to Clause 8.

NOTE In general, no plastic hinges will develop in piers which are flexibly connected to the deck in the direction considered. A similar situation will occur in individual piers with very low stiffness in comparison to the other piers (see 4.3.2(1)). Such members have negligible contribution in resisting the seismic actions and therefore do not affect the value of the q-factor (see (6)).

(14) The behaviour factor for the analysis in the vertical direction should be defined according to prEN 1998-1-1:2022, 6.4.1(6) and (7).

(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.

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

where

ρ_o = 2,0 is a limit value selected so as to ensure that sequential yielding of the ductile members will not cause unacceptably high ductility demands in one member;

r_{min} is the minimum value of r_i among all ductile members i;

r_{max} is the maximum value of r_i among all ductile members i;

r_i is given by Formula (5).11).

$$r_{i} = q \frac{M_{Ed,i}}{M_{Rd,i}} \tag{5.11}$$

where

$M_{Ed,i}$ is the maximum value of design moment at the intended plastic hinge location of ductile member i as derived from the analysis for the seismic design situation;

$M_{Rd,i}$ is the design flexural resistance of the same section with its actual reinforcement under the concurrent axial load in the seismic design situation.

NOTE 1 Since $M_{Ed,i} \le M_{Rd,i}$, it follows that $r_i \le q$.

NOTE 2 When, in a regular bridge, the maximum value of r_i among all ductile members, r_{max}, is substantially lower than q, the design cannot fully exploit the allowable maximum q-values. When r_{max} = 1,0 the bridge responds elastically to the design earthquake considered.

(16) One or more ductile members (piers) may be omitted in the calculation of r_{min} and r_{max} as defined in (15), if the sum of their shear contributions does not exceed 20 % of the total seismic shear in the considered horizontal direction.

(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 based on results of nonlinear analysis in accordance with 5.2.3.2 or, when applicable, 5.2.3.

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

NOTE In bridges of irregular seismic behaviour, the sequential yielding of the ductile members (piers) can cause substantial deviations of the results of the linear analysis performed for the force-based approach with a reduced spectrum from those of the nonlinear response of the bridge structure. The deviations are mainly due to: a) the plastic hinges which appear first usually develop the maximum post-elastic strains, which can lead to concentration of unacceptably high ductility demands in these hinges; b) following the formation of the first plastic hinges (normally in the stiffer members), the distribution of stiffness and hence of forces can change from that predicted by the equivalent linear analysis. This can lead to a substantial change in the assumed pattern of plastic hinges.

5.2.2.2 Lateral forces method

(1) The method, which consists in applying a system of static forces in both the longitudinal and transverse directions, may be applied to bridges whose response is not significantly affected by contributions from modes of vibration higher than the fundamental one in each considered direction. This condition may be considered satisfied in cases a) to d):

a) in the longitudinal direction of bridges with continuous straight deck, when the seismic forces are carried by piers, the total mass of which is less than 20 % of the mass of the deck;

b) in the transverse direction of case (a), if the theoretical eccentricity e_o between the centre of stiffness of the supporting members and the centre of mass of the deck does not exceed 5 % of the length of the deck (L);

c) in the case of piers carrying simply supported spans, if no significant interaction between piers is expected and the total mass of each pier is less than 20 % of the tributary mass of the deck;

d) in case the equivalent modal mass $M1$, determined by Formula (5).13), exceeds 70 % of the total bridge mass above the foundations.

$$M_1 = \frac{\left(\sum m_i s_i\right)^2}{\sum m_i s_i^2} \geq 0,7M \tag{5.13}$$

where

m_i	is the mass over the *i*-th support (including half of the mass of the pier);
s_i	is the displacement in meters over the *i*-th support in the considered direction when the structure is acted upon by a horizontal force corresponding to the acceleration of gravity, 9,81 m/s^2, in that direction;
M	is the total bridge mass above the foundations.

NOTE Decks that are not curved according to 3.1.11, are considered straight.

(2) The seismic base shear force F_b for each horizontal direction should be determined by Formula (5).14).

$$F_{\mathrm{b}} = \sum m_{\mathrm{i}} \, S_{\mathrm{r}} \left(T_1 \right) \tag{5.14}$$

where

$S_r(T_1)$ is the ordinate of the reduced spectrum (prEN 1998-1-1:2022, 6.4.1) at period T_1;

T_1 is the fundamental period in the considered direction.

(3) Unless a more accurate calculation is made, the fundamental period of the structure, in the horizontal direction considered, may be estimated via the Rayleigh quotient using a generalized single-degree of freedom system, as given by Formula (5).15).

$$T_1 = 2\sqrt{\frac{\sum m_{\mathrm{i}} s_{\mathrm{i}}^2}{\sum m_{\mathrm{i}} s_{\mathrm{i}}}} \tag{5.15}$$

(4) The seismic action effects should be determined by applying, in each horizontal direction, horizontal forces F_i given by Formula (5).16).

$$F_{\mathrm{i}} = F_{\mathrm{b}} \frac{m_{\mathrm{i}} s_{\mathrm{i}}}{\sum_j m_{\mathrm{j}} s_{\mathrm{j}}} \tag{5.16}$$

(5) In the seismic design situation, in the transverse direction, the deformation of the deck within a horizontal plane is negligible compared to the horizontal displacements of the pier tops, i.e. the deck may be assumed as rigid, if either a) or b) hold:

a) $L/B \leq 4{,}0$;

b) $\Delta d/d_m < 0{,}2$.

where

Δd = max(d_i-d_j) is the maximum difference in displacement between any two pier tops under a transverse uniformly distributed load on the deck;

d_m is the average of the pier top displacement under the same load.

NOTE The condition of rigid deck is always met in the longitudinal direction of approximately straight bridges with continuous deck.

(6) If the deck may be assumed as rigid according to (5), the fundamental period T_1 in each direction may be calculated by Formula (5).17).

$$T_1 = 2\pi \sqrt{\frac{\sum_i m_{\mathrm{i}}}{\sum_i k_{\mathrm{i}}}} \tag{5.17}$$

where k_i is the stiffness of the i-th support.

(7) If the deck may be assumed as rigid according to (5), the total base shear force F_b may be distributed along the deck proportionally to the distribution of the masses according to Formula (5).18).

$$F_{\mathrm{i}} = F_{\mathrm{b}} \frac{m_{\mathrm{i}}}{\sum_{j} m_{\mathrm{j}}} \tag{5.18}$$

(8) Irrespective of whether the deck is rigid or not, when torsional effects in the transverse direction (rotation about the vertical axis) are included according to 5.1.2, they may be estimated by applying a static torsional moment M_{t} in accordance with Formula (5).2), which may be distributed to supporting members as if the deck was rigid.

(9) The seismic action effects acting in the *i*-th pier may be calculated by applying on it a static force given by Formula (5).19) under the condition that Formula (5).20) is met for all adjacent piers *i* and *i*+1.

$$F_{\mathrm{i}} = m_{\mathrm{i}} S_{\mathrm{r}} \left(T_{\mathrm{i}} \right) \tag{5.19}$$

$$0,9 \leq \frac{T_{\mathrm{i}}}{T_{\mathrm{i+1}}} \leq 1,1 \tag{5.20}$$

where T_{i} is the fundamental period of the *i*-th pier, considered independently of the rest of the bridge, given by Formula (5).21).

$$T_{\mathrm{i}} = 2\pi \sqrt{\frac{m_{\mathrm{i}}}{k_{\mathrm{i}}}} \tag{5.21}$$

(10) Displacements induced by the reduced seismic action used to calculate the static forces, should be calculated according to prEN 1998-1-1:2022, 6.4.2.

(11) Combination of the components of the seismic action should comply with prEN 1998-1-1:2022, 6.4.4.

5.2.2.3 Response spectrum method

(1) Response spectrum analysis should comply with prEN 1998-1-1:2022, 6.4.3.

5.2.3 Displacement-based approach

5.2.3.1 Nonlinear static analysis

(1) Nonlinear static analysis should be carried out according to prEN 1998-1-1:2022, 6.5.

(2) Except as given in (3), the nonlinear 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 5.2.2.2(1).

NOTE Single-mode nonlinear 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.

(3) Alternatively to (2), nonlinear 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 nonlinear static analysis in prEN 1998-1-1:2022, 6.5. Those methods are not covered in this standard.

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

(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 model mass in the considered direction), according to prEN 1998-1-1:2022, 6.5.2(4).

5.2.3.2 Response-history analysis

(1) The choice of number and type of input motions, as well as calculation of seismic action effects, should comply with prEN 1998-1-1:2022, 6.6.

(2) The components of motion should be applied simultaneously, according to prEN 1998-1-1:2022, 6.3(1).

(3) Modelling should comply with 5.1 (in particular 5.1.1(12)).

NOTE 1 Additional modelling rules for cable-stayed and extradosed bridges and for integral abutment bridges are given in 9.3 and Annex D, respectively.

NOTE 2 Response-history analysis can also be carried out on a linear model to design bridges intended to respond in the elastic range.

5.3 Methods of analysis accounting for spatial variability of ground motion

5.3.1 General

(1) Analysis should account for spatial variability of earthquake ground motion according to Table 5 3.

NOTE 1 Spatial variability of the ground motion is due to: a) variation of the soil mechanical properties along the bridge, giving rise to differential site effects, which modify amplitude, phase and frequency content from one support to the other, including abutments; b) the propagation characteristics of the seismic waves (wave-passage), as well as for the progressive loss of correlation between motions at different locations due to the random non-homogeneity of the soil, involving complex reflections, refractions and superpositions of seismic waves.

NOTE 2 When the bridge and span lengths are short, wave-passage and loss of coherence are less important in relative terms. If the soil is uniform, spatial variability is not significant; otherwise, it is mainly due to differential site effects. For longer bridges and span lengths, wave passage and loss of coherence are important and local structural response amplification, in an overall beneficial reduction of the average response, occurs due to the excitation of higher modes otherwise not excited.

Table 5.3 — Analysis type for multiple support excitation

Soil conditions	**Bridge and span length**	
	Short-medium length ($L \leq L_{\text{lim}}$) and maximum span between adjacent piers $L_{\text{kl}} < 50$ m	Long bridge ($L > L_{\text{lim}}$) or maximum span between two successive piers $L_{\text{kl}} > 50$ m (for bridges having two spans or more)
The maximum and minimum shear wave velocity $V_{\text{s,H}}$ of the soil profiles under the supports (piers and abutments) do not vary by more than 200 m/s	Account for spatial variability is not required	Simplified higher mode excitation method (5.3.1) Alternatively, the simplified higher mode excitation method of analysis can be omitted, with an application of a 20 % increase in all seismic action effects* obtained from a regular uniform excitation analysis (e.g. response spectrum method)
The maximum and minimum shear wave velocity $V_{\text{s,H}}$ of the soil profiles under the supports (piers and abutments) vary by more than 200 m/s and the depth of valley along the bridge $h < 100$ m.	Simplified multiple-support response-history analysis, with ground motions at the supports obtained from a common input at the bedrock and separate 1D site response analyses at each support (5.3.2) Alternatively, 1D site response analysis can be omitted, with an application of a 20 % increase in all seismic action effects* obtained from a regular uniform excitation analysis (e.g. response spectrum method)	Multiple-support response-history analysis, with ground motions that comply with the spatial variability model (5.3.3(1)) or Multiple-support response spectrum method (5.3.3(2)) Alternatively, multi-support response analyses can be omitted, with an application of a 30 % increase in all seismic action effects* obtained from a regular uniform excitation analysis (e.g. response spectrum method)
The maximum and minimum shear wave velocity $V_{\text{s,H}}$ of the soil profiles under the supports (piers and abutments) vary by more than 200 m/s and the depth of valley along the bridge $h \geq 100$ m	Response-history analysis with spatially variable ground motions produced by means of 2D/3D site response analysis or Alternatively, 2D/3D site response analysis can be omitted, with an application of a 1D site response analysis per support and an additional 30 % increase in all seismic action effects* obtained from a regular uniform excitation analysis (e.g. response spectrum method)	
* Seismic action effects include generalised stresses as well as generalised deformations. The latter include relative displacements at deck joints and supports, which should be increased to avoid unseating failure due to spatially varying ground motions.		

NOTE It is always possible to carry out an analysis under a uniform excitation and to increase seismic action effects depending on the expected severity of local increase due to spatial variability of ground motion.

5.3.2 Long bridges on uniform soil

(1) For long bridges on uniform soil conditions according to Table 5 3, a simplified higher mode excitation method may be used.

NOTE The method accounts for the key impact of spatial variability on the response of long bridges on uniform soil, i.e. the excitation of higher modes with an odd number of inflection points in the deck (quasi-antisymmetric), leading to local increase of seismic action effects in some supports, due to wave-passage and loss of coherence. Modes with an odd number of inflection points in the deck are characterized by a reduced participation under a uniform excitation because they move approximately equal amounts of mass in opposite directions. They are excited by spatially varying ground motions. These modes would be antisymmetric only in an ideal perfectly symmetric bridge.

(2) If the simplified higher mode excitation method is used, the seismic action effects should be obtained according to Formula (5).22), as a SRSS combination of the contribution due to uniform excitation, $E_{\mathrm{d,u}}$, obtained by the response spectrum method (5.2.2.3) or response-history analysis (5.2.3.2) with uniform excitation (corresponding to the uniform site condition under the supports), and the contribution of the higher quasi-antisymmetric modes $E_{\mathrm{d,i}}$.

$$E_{\mathrm{d}} = \sqrt{E_{\mathrm{d,u}}^2 + \sum_i E_{\mathrm{d,i}}^2} \tag{5.22}$$

(3) The contribution of the higher quasi-antisymmetric modes $E_{\mathrm{d,i}}$ should be obtained by applying static forces according to Formula (5).23) for both the first and the second quasi-antisymmetric modes (i = 1, 2).

$$\boldsymbol{F}_{\mathrm{i}} = \left(\overline{SF}_{\mathrm{i}} - 1\right)\Gamma_{\mathrm{i}} S_{\mathrm{e}}\left(T_{\mathrm{i}}\right)\boldsymbol{M}\boldsymbol{\phi}_{\mathrm{i}} \tag{5.23}$$

where

F_{i}	is the vector of static forces for the i-th quasi-antisymmetric mode;
T_{i}	is the i-th modal period from modal analysis;
M	is the mass matrix;
ϕ_{i}	is the i-th modal shape from modal analysis;
$\Gamma_{\mathrm{i}} = \sum_{\mathrm{k}=1}^{N_{\mathrm{S}}} \Gamma_{\mathrm{ik}}$	is the i-th modal participation factor due to spatially variable excitation in N_{S} static modes, obtained as the sum of the participation factors of the i-th mode due to each individual static mode, given in (4);
$\overline{SF}_{\mathrm{i}}$	is a mode amplification factor assumed equal to 4,0 and 2,0, for the first and second quasi-antisymmetric mode, respectively.

NOTE The product $\Gamma_{\mathrm{i}}\, S_{\mathrm{e}}(T_{\mathrm{i}})\, M\phi_{\mathrm{i}}$ represents the modal forces due to the i-th mode. They do not coincide with those coming from a uniform excitation due to the different value of the participation factor Γ_{i}.

(4) The i-th modal participation factor in the k-th static mode may be evaluated according to Formula (5).24).

$$\Gamma_{\mathrm{ik}} = \frac{\boldsymbol{\phi}_{\mathrm{i}}^{T}\boldsymbol{M}\boldsymbol{r}_{\mathrm{k}}}{\boldsymbol{\phi}_{\mathrm{i}}^{T}\boldsymbol{M}\boldsymbol{\phi}_{\mathrm{i}}} \tag{5.24}$$

where r_{k} is the column vector collecting the k-th static mode, i.e. the displacements obtained by carrying out a static analysis under a unit displacement along the direction under consideration, at the k-th of N_{S} bridge supports.

NOTE The vector r_k has the same dimension of mode shapes ϕ_i. In models with many degrees of freedom (e.g. due to a refined description of the deck), it is convenient, for the sake of computing Γ_{ik}, to evaluate r_k with reference to just N_S unconstrained degrees of freedom, identified on the deck, including one per support and other characteristic locations needed to describe the deck deformed shape, and to consider a sub-vector of ϕ_i at the same N_S locations.

5.3.3 Short to medium length bridges on non-uniform soil

(1) For short to medium length bridges on non-uniform soil conditions, according to Table 5.3, a simplified response-history method may be used, where seismic input motions at supports are derived from the response of corresponding soil profiles to a common input at H_{800}, as defined in prEN 1998-1-1:2022, 5.1.2.

NOTE From a practical point of view, application of different motions at the supports cannot be done as for the case of uniform excitation, prescribing an acceleration time series at the base in each direction. The motion at each support is instead imposed as a displacement time series in each direction (like a time-varying settlement). Displacement response is thus in total terms, rather than relative (to the support).

(2) Ground motion selection at the bedrock should comply with prEN 1998-1-1:2022, assuming soil class A.

5.3.4 Long bridges on non-uniform soil

(1) For long bridges on non-uniform soil conditions, according to Table 5.3, a full response-history method with spatially variable ground motion time series should be used.

NOTE 1 This method is not covered in this standard.

NOTE 2 From an operational point of view, the method is not different from the simplified response-history one described in 5.3.2. The input ground motion time series are different, however, since they need to reflect both the modification of motion due to differential soil profiles and that due to wave-passage and loss of coherence effects. Motions of this type can be obtained, e.g. by: a) by selection of recorded ground motions at one support (e.g. one abutment), according to prEN 1998-1-1:2022, Annex C, and subsequent modification to obtain input motions at the other supports compatible with local frequency content at each support and accounting for the wave-passage and loss of coherence effects; b) by generation from a vector random process model. Modification of real motions or generation of artificial ones are not covered in this standard.

(2) Alternatively, the multiple-support response spectrum (MSRS) method may be used, according to (3) to (6).

(3) If the MSRS method is adopted, seismic action effects should be obtained as a suitable combination of their quasi-static part E_d^S and dynamic part E_d^D according to Formula (5).25).

$$E_d = \sqrt{\left(E_d^S\right)^2 + \left(E_d^D\right)^2} \tag{5.25}$$

(4) The quasi-static part of the seismic action effect may be calculated according to Formula (5).26).

$$E_{d}^{S} = \sqrt{\sum_{k=1}^{N_S} \sum_{l=1}^{N_S} \rho_{kl} E_{dk}^{S} E_{dl}^{S}} \tag{5.26}$$

where

E_{dk}^{S}	is the contribution of the k-th static mode, defined in 5.3.1(4), under the peak ground displacement at support k;
ρ_{kl}	is the correlation coefficient given in prEN 1998-1-1:2022, 5.2.3.2(3);
N_S	is the number of static modes, which coincides with the number of supports.

(5) The dynamic part of the seismic action effect may be calculated according to Formula (5).27).

$$E_{d}^{D} = \sqrt{\sum_{i=1}^{N_D} \sum_{j=1}^{N_D} r_{ij} E_{di}^{D} E_{dj}^{D}} \tag{5.27}$$

where

E_{di}^{D}	is the contribution of the i-th mode under the design seismic action;
r_{ij}	is the modal correlation coefficient given in prEN 1998-1-1:2022, 6.4.3.2;
N_D	is the number of dynamic modes considered.

(6) The value of E_{di}^{D} may be taken as given by Formula (5).28).

$$E_{di}^{D} = \sqrt{\sum_{k=1}^{N_S} \sum_{l=1}^{N_S} \rho_{kl} E_{dik}^{D} E_{dil}^{D}} \tag{5.28}$$

where

E_{dik}^{D}	is the i-th mode response to the seismic input (response spectrum) at the k-th support, calculated according to 5.2.2.3, but replacing the conventional modal participation factor with that given by Formula (5).24);
ρ_{kl}	is the correlation coefficient given in prEN 1998-1-1:2022, 5.2.3.2(3).

5.4 Combination of the seismic action with other actions

(1) The design value E_d of the effects of actions in the seismic design situation should be determined in accordance with prEN 1990:2021, 8.3.4.4.

NOTE For skew bridges see note to 3.1.3.

(2) In the case of bridges in which the seismic action is resisted by elastomeric laminated bearings, the action effects due to imposed deformations (caused by temperature, shrinkage, settlements of supports, residual ground movements due to seismic faulting) should be accounted for.

NOTE In this case, the displacement due to creep does not normally induce additional stresses to the system and can therefore be neglected. Creep also reduces the effective stresses induced in the structure by long-term imposed deformations (e.g. by shrinkage).

(3) In all other cases, action effects due to imposed deformations may be neglected.

6 Verifications of structural members to limit states

6.1 General

(1) Clause 6 should be applied to the earthquake resisting system of bridges designed for DC1, DC2 or DC3 (see 4.3.6). For bridges equipped with antiseismic devices, cable-stayed and extradosed bridges, and integral abutment bridges, 8, 9 and 10, respectively, should be applied.

(2) Verifications should ensure that resistance, stability and ductility conform to prEN 1998-1-1:2022, 6.7.1.

(3) Clause 6 should be applied for the design of structural members and for the detailing of the critical regions of each member type. Outside the critical regions, the detailing of structural members should satisfy relevant provisions in prEN 1992-1-1, prEN 1993-21 and prEN 1994-2[2].

6.2 Material requirements

6.2.1 General

(1) Concrete of a class lower than C25 should not be used in primary seismic members.

(2) Ribbed bars should be used as reinforcing steel in all regions of primary or secondary seismic members.

(3) Except as given in 6.2.2(1), in primary seismic members (see 3.1.9), reinforcing steel of ductility class B or C in prEN 1992-1-1:2021, Table 5.5, should be used.

(4) The steel categories for the base material and for the welds in steel and steel-concrete composite bridges should be taken as given in prEN 1993-1-10[2] for a stress level $\sigma_{\mathrm{Ed}} = 0{,}75\, f_{\mathrm{y}}(T)$ and the quasi-permanent value of service temperature.

NOTE 1 (4) defines steel with toughness and thickness adequate for yielded sections.

NOTE 2 The quasi-permanent value of the service temperature is defined in prEN 1990:2021, Annex A, A.1.5.3.

(5) The required toughness of steel and welds and the lowest service temperature adopted in combination with the seismic action should be defined.

(6) In bolted connections of primary seismic members, high strength preloaded bolts of grade 8.8 or 10.9 with controlled tightening in accordance with prEN 1993-1-8 should be used.

6.2.2 Design for DC2 and DC3

(1) In critical regions of primary seismic members (see 3.1.9) designed for DC3, reinforcing steel of ductility class C in prEN 1992-1-1:2021, Table 5.5, should be used.

(2) The material properties, such as yield strength and toughness, in the dissipative zones of steel and steel-concrete composite bridges shall be such that plastic deformations occur where they are intended to in the design.

(3) In the capacity design verifications of steel and steel-composite bridges, the possibility that the actual yield strength of steel is greater than the nominal one should be taken into account through the randomness material factor ω_{rm}, which is the ratio between the expected (i.e. mean) yield strength $f_{\mathrm{y,mean}}$ and the nominal yield strength f_{y} at the plastic hinge location and depends on steel grade, as specified in prEN 1998-1-1:2022, Table 7.1. Therefore, the resistance at yield of dissipative members should be calculated considering the randomness material factor ω_{rm}.

(4) For dissipative zones, the steel grade to be used should be specified and noted on the drawings. A higher grade should not be supplied for these zones (see prEN 1998-1-2[2], 11.19(3)).

6.3 Verification of Significant Damage (SD) limit state

6.3.1 General

(1) Verifications according to the force-based approach should be carried out in terms of local resistances, calculated as given in 6.3.3 to 6.3.7. Demand on non-ductile members should be obtained as capacity design effects, as specified in 6.3.2.

(2) Verifications according to the displacement-based approach should be carried out according to 6.3.8.

6.3.2 Capacity design effects

(1) In accordance with 4.3.3(4), brittle and other undesired failure mechanisms should be avoided by deriving design action effects of selected regions from equilibrium conditions, assuming that plastic hinges with their possible overstrength have formed in their adjacent areas. These capacity design effects should be taken equal to the minimum of those obtained in the seismic design situation with $q = 1$ and those obtained in the assumption that all flexural plastic hinges under the intended plastic mechanism have developed bending moments equal to an upper fractile of their flexural resistance, called the overstrength moment, M_{o}.

(2) In each considered direction, the overstrength moment of a section should be calculated as given in Formula (6).1).

$$M_{\mathrm{o}} = \gamma_{\mathrm{Rd}} \omega_{\mathrm{rm}} \omega_{\mathrm{sh}} M_{\mathrm{Rd}} \tag{6.1}$$

where

γ_{Rd} is the overstrength partial factor, reflecting the different importance of failure modes;

ω_{rm} is the material randomness factor, taken equal to 1,15 for reinforcement steel in reinforced concrete members and given in prEN 1998-1-1:2022, 7.3, Table 7.1, for structural steel and composite members;

ω_{sh} is the strain hardening factor, taken equal to 1,05 for reinforcement steel in reinforced concrete and given in prEN 1998-1-2[2], 11.8.6, Table 11.8, for structural steel and composite members;

M_{Rd} is the design flexural resistance of the section in accordance with prEN 1992-1-1:2021, 8.1, in the selected direction and sign, calculated for the concurrent axial load due to all actions in the seismic design situation.

NOTE The overstrength partial factor γ_{Rd} (NDP) is equal to 1,1 when M_{o} is used to calculate seismic action effects on the shear mechanism, and 1,0 otherwise, unless a relevant authority or the national annex give different values for use in a country.

(3) In the case of reinforced concrete sections with special confining reinforcement in accordance with 7.2.4.1, and with the value of the normalized axial force η_{k} given by Formula (5).7) not lower than 0,1, the value of the overstrength moment should be multiplied by $1+2(\eta_{\mathrm{k}} - 0,1)^2$.

(4) According to (1), within the length of members that develop plastic hinges, the capacity design bending moment M_{Ed} should be taken as the minimum between the bending moment obtained in the seismic design situation with $q = 1$ and the bending moment from analysis M'_{Ed} amplified due to development of the overstrength moment in the hinges. In this case, the overstrength moment M_{o} should be calculated neglecting ω_{rm} and γ_{Rd}. The capacity design bending moment M_{Ed} should not be greater than the relevant design flexural resistance M_{Rd} of the nearest hinge calculated in accordance with 6.3.3.2(1) for the entire length of the critical region l_{cr} (see 7.2.3).

NOTE The longitudinal reinforcement along the pier portion encompassing the critical zone and the zone adjacent to it is assumed to be from the same steel production.

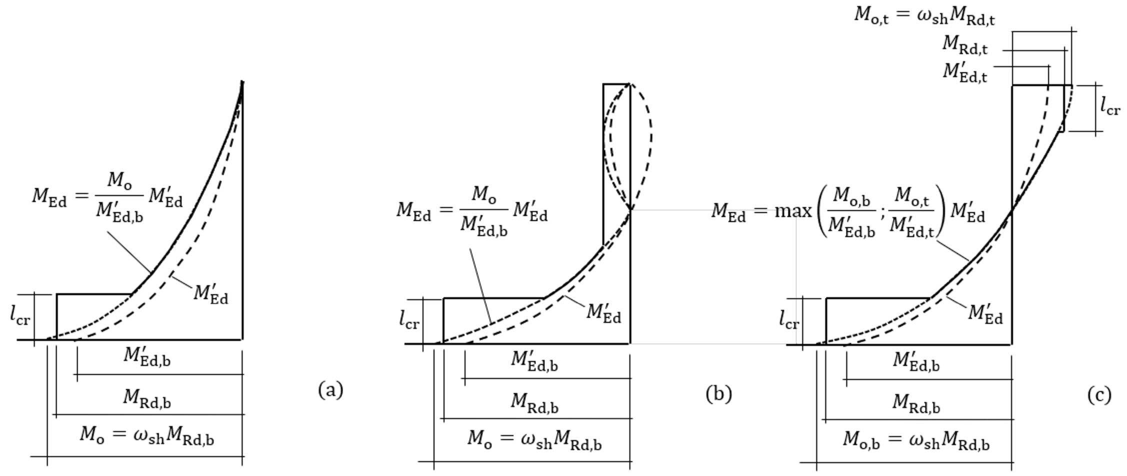


Figure 6.1 — Capacity design bending moment within the length of a member developing plastic hinges, with: (a) cantilever pier; (b) cantilever pier with significant higher modes effect; (c) pier that frames into the deck and is designed to form plastic hinges at both ends (subscripts "b" and "t" indicate "bottom" and "top", respectively)

(5) For the application of Formula (6).1) for timber bridges, the overstrength moment should be calculated starting from the flexural resistance of the dissipative connections. The overstrength partial factor γ_{Rd} should be taken equal to the value in prEN 1998-1-2[2], 13.4.3, Table 13.4, divided by the strength reduction factor k_{deg} = 0,8 defined in prEN 1998-1-2[2], 13.3.1(1), while ω_{rm} and ω_{sh} may both be taken equal to 1,0.

NOTE The resulting value of the overstrength partial factor for dissipative connections in timber construction accounts for material randomness, strain hardening and strength reduction due to cyclic loading.

6.3.3 Concrete members

6.3.3.1 General

(1) When the resistance of a section depends on multi-component action effects (e.g. uniaxial or biaxial bending moment and axial force), the SD limit state conditions specified in 6.3.3.2 and 6.3.3.3 may be satisfied by considering separately the extreme (maximum or minimum) value of each component of the action effect with the concurrent values of all other components of the action effect.

(2) For flexural resistance of sections in critical regions, the condition given by Formula (6).2) should be satisfied:

$$M_{Ed} \leq M_{Rd} \qquad (6.2)$$

where

M_{Ed} is the design action effect as derived from the analysis for the seismic design situation, including second-order effects if needed;

M_{Rd} is the design flexural resistance of the section in accordance with prEN 1992-1-1:2021, 8.1.

6.3.3.2 Structures of DC1

(1) 6.3.3.1(2) should be applied.

(2) Verifications of shear resistance of concrete members should be carried out in accordance with prEN 1998-1-1:2022, 7.2.3 (assimilating bridge piers to columns), with the additional provision that the design action effects should be calculated in accordance with 5.4(1), where the seismic action effect A_{Ed} should be multiplied by the behaviour factor q used in the linear analysis.

6.3.3.3 Structures of DC2 and DC3

6.3.3.3.1 Verification for flexure and shear

(1) 6.3.3.1(2) should be applied.

(2) M_{Rd} should be constant over the length l_{cr} of the critical region (defined in 7.2.3), as specified in 6.3.2(4) and shown in Figure 6.1. For the flexural resistance of sections outside critical regions, the condition given by Formula (6).3) should be satisfied.

$$M_{\mathrm{Ed}} \leq M_{\mathrm{Rd}} \tag{6.3}$$

where

M_{Ed}	is the design moment accounting for capacity design effects as specified in 6.3.2(4);
M_{Rd}	is the design resistance of the section in accordance with prEN 1992-1-1:2021, 8.1, taking into account the interaction of the other components of the design action effect (axial force and, when applicable, bending moment in the orthogonal direction).

(3) Verifications of shear resistance should be carried out in accordance with prEN 1998-1-1:2022, 7.2.3 (assimilating bridge piers to columns), with the additional rule that the design action effects should account for capacity design effects in accordance with 6.3.2.

6.3.3.3.2 Verification of joints adjacent to critical regions

(1) Any joint between a vertical ductile pier and the deck or a foundation member adjacent to a plastic hinge in the pier should be designed in shear to resist the capacity design effects of the plastic hinge in the relevant direction.

NOTE The pier is indexed in 6.3.3.3.2 with "c" (for "column"), while any other member (e.g. the deck) framing into the same joint is referred to as "beam" and indexed with "b" (see Figure 6.2).

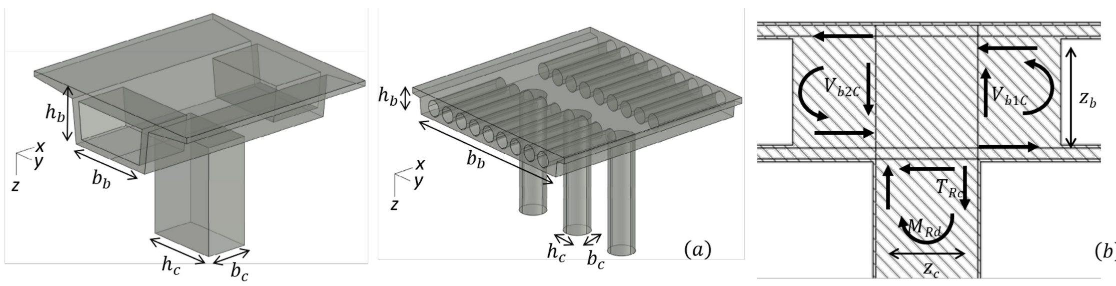


Figure 6.2 — Pier-deck joints: (a) definition of variables; (b) stress conditions

(2) For a vertical solid pier of depth h_c and of width b_c transverse to the direction of flexure of the plastic hinge, the effective width b_j of the joint should be assumed as given in a) to c):

a) when the pier frames into a slab or a transverse rib of a hollow slab, b_j is given by Formula (6).4);

$$b_j = b_c + 0,5h_c \tag{6.4}$$

b) when the pier frames directly into a longitudinal web of width b_w (b_w is parallel to b_c), b_j is given by Formula (6).5);

$$b_j = \min\left(b_w; b_c + 0,5h_c\right) \tag{6.5}$$

c) for circular piers of diameter d_c, the definitions given in a) or b), as appropriate, should be applied assuming $b_c = h_c = 0,9d_c$.

(3) The design vertical shear acting force in the joint, $V_{Edj,z}$, should be taken as given in Formula (6).6).

$$V_{Edj,z} = T_{Rc} - V_{b1C} \tag{6.6}$$

where

T_{Rc}	is the resultant force of the tensile reinforcement of the pier corresponding to the overstrength moment, M_o, of the plastic hinge in accordance with 6.3.2;
V_{b1C}	is the shear force of the horizontal member adjacent to the tensile face of the pier, corresponding to the capacity design effects of the plastic hinge.

(4) The design horizontal shear acting force in the joint $V_{Edj,x}$ may be calculated as given by Formula (6).7).

$$V_{Edj,x} = V_{Edj,z} \frac{z_c}{z_b} \tag{6.7}$$

where z_c and z_b are the internal lever arms of the plastic hinge and the horizontal member end sections, respectively; z_c and z_b may be assumed to be equal to 0,9 times the relevant effective section depths.

(5) The shear verification should be carried out at the centre of the joint, where, in addition to $V_{Edj,z}$ and $V_{Edj,x}$, the influence of the design acting axial forces given in a) to c) may be taken into account:

a) the vertical axial joint force $N_{Edj,z}$ given by Formula (6).8);

$$N_{Edj,z} = \frac{b_c}{2b_j} N_{cG} \tag{6.8}$$

where N_{cG} is the axial force of the pier under the non-seismic actions in the seismic design situation.

b) the horizontal joint force $N_{Edj,x}$, equal to the capacity design axial force effects in the horizontal member, including the effects of longitudinal prestressing after all losses (if such axial forces are actually effective throughout the width b_j of the joint);

c) the horizontal joint force $N_{Edj,y}$ in the transverse direction, equal to the effect of transverse prestressing after all losses, effective within the depth h_c, if such prestressing is provided.

(6) The capacity design, and therefore the relevant joint verification, should be carried out with both signs of the seismic action, + and –.

NOTE At knee-joints (e.g. over the end column of a multi-column bent in the transverse bridge direction), the sign of M_{Rd} and V_{b1C} can be opposite to that shown in Figure 6.2 and $N_{Edj,x}$ can be tensile.

(7) The design value of the shear stress in the joint, assumed unreinforced, at first cracking, $v_{Rdj,cr}$, may be taken as a lower limit to its design shear resistance, given by Formula (6).9).

$$v_{Edj} \leq v_{Rdj,cr} = f_{ctd}\sqrt{\left(1+\frac{n_x}{f_{ctd}}\right)\left(1+\frac{n_y}{f_{ctd}}\right)\left(1+\frac{n_z}{f_{ctd}}\right)} \tag{6.9}$$

where

f_{ctd}	is the design value of the tensile strength of concrete;
v_{Edj}	is the design acting shear stress, given by Formula (6).10);
n_x	is the joint axial stress in the horizontal direction x, given by Formula (6).11);
n_y	is the joint axial stress in the horizontal direction y, given by Formula (6).12);
n_z	is the joint axial stress in the vertical direction z, given by Formula (6).13).

$$v_{Edj} = v_x = v_z = \frac{V_{Edj,x}}{b_j z_c} = \frac{V_{Edj,z}}{b_j z_b} \tag{6.10}$$

$$n_x = \frac{N_{Edj,x}}{b_j h_b} \tag{6.11}$$

$$n_y = \frac{N_{Edj,y}}{h_b h_c} \tag{6.12}$$

$$n_z = \frac{N_{Edj,z}}{b_j h_c} \tag{6.13}$$

(8) The diagonal compression induced in the joint by the diagonal strut mechanism should not exceed the compressive strength of concrete in the presence of transverse tensile strains, taking into account also confining pressures and reinforcement.

(9) (8) may be considered satisfied if the design acting shear force on the concrete core of the joint does not exceed the design shear resistance calculated according to prEN 1998-1-1:2022, 7.2.4(3), with the modifications in a) to d):

a) the mean values of resistances are replaced by their design values: $V_{Rj,min}$ by $V_{Rdj,min}$, $V_{Rj,h}$ by $V_{Rdj,h}$, $V_{Rj,v}$ by $V_{Rdj,v}$, $V_{Rj,c}$ by $V_{Rdj,c}$;

b) the characteristic value of steel yield strength f_{yk} (or $f_{yk,h}$ or $f_{yk,v}$) is replaced by the corresponding design value f_{yd} (or $f_{yd,h}$ or $f_{yd,v}$);

c) the mean values of concrete compressive and tensile strengths f_{cm} and f_{ctm} are replaced by the corresponding design values f_{cd} and f_{ctd};

d) the horizontal acting axial forces are taken equal to $N_{jb} = N_{Edj,x}$ or $N_{jb} = N_{Edj,y}$, depending on the considered direction of verification.

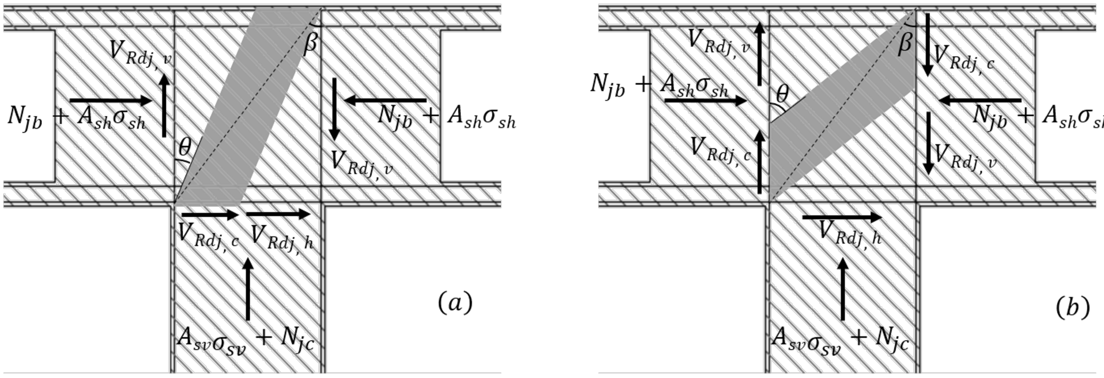


Figure 6.3 — Pier-deck joints: (a) stress conditions with $\theta < \beta$; (b) stress conditions with $\theta > \beta$

6.3.3.4 Deck verification

(1) It should be verified that no significant yielding occurs in the deck. This verification should be carried out according to a) or b), as appropriate:

a) for bridges of DC1, under the most adverse design action effect in accordance with 5.4;

b) for bridges of DC2 and DC3, under the capacity design effects determined in accordance with 6.3.2.

NOTE Yielding of the deck for flexure within a horizontal plane is considered to be significant if the reinforcement of the top slab of the deck yields up to a distance from its edge equal to 10 % of the top slab width, or up to the junction of the top slab with a web, whichever is closer to the edge of the top slab.

(2) When verifying the deck on the basis of capacity design effects for the seismic action acting in the transverse direction of the bridge, the significant reduction of the torsional stiffness of the deck with increasing torsional moments should be accounted for. Unless a more accurate calculation is made, the values specified in 5.1.1(9) may be assumed for bridges of DC1, or 70 % of these values for bridges of DC2 and DC3.

6.3.4 Steel and steel-concrete composite members

6.3.4.1 General

(1) Energy dissipation shall take place only in the piers and not in the deck.

(2) For members of DC2 and DC3 steel and steel-concrete composite bridges, prEN 1998-1-2[2], 11.8.1, 11.8.2 and 11.8.4, should be applied.

(3) Members of dissipative zones should be of cross-sectional class 1 in DC3 and 1 or 2 in DC2. Cross-sectional class 3 may be used when q = 1,5.

6.3.4.2 Steel piers

6.3.4.2.1 General

(1) For the verification of the pier under multi-component action effects, 6.3.1(1) should be applied.

(2) prEN 1998-1-2[2], 11.15, should be applied.

(3) For connections, prEN 1998-1-2[2], 11.8.6 and Annex E, should be applied.

6.3.4.2.2 Piers as moment resisting frames

(1) In DC2 and DC3 bridges, the capacity design action effects in piers consisting of moment resisting frames should be taken as in 6.3.2.

(2) The design of the sections of plastic hinges both in beams and columns of the pier should satisfy prEN 1998-1-2[2], 11.9.2, 11.9.3, 11.9.4 and 11.9.5, using the values of N_{Ed} and V_{Ed} as specified in (1).

6.3.4.2.3 Piers as frames with concentric bracings

(1) prEN 1998-1-2[2], 11.10, should be applied with the modifications in a) and b):

a) prEN 1998-1-2[2], 11.10.3 (9) and (10) should not be applied;

b) in case of multi-level concentric bracings, prEN 1998-1-2[2], 11.10.3(12), Formula (11).20), should be verified at all levels including the upper one.

6.3.4.2.4 Piers as frames with eccentric bracings

(1) prEN 1998-1-2[2], 11.11, should be applied.

6.3.4.2.5 Piers as frames with buckling-restrained bracings

(1) prEN 1998-1-2[2], 11.12, should be applied.

6.3.4.3 Steel or steel-concrete composite deck

(1) In DC2 and DC3 bridges, the deck should be verified for the capacity design effects in accordance with 6.3.1(1).

(2) In DC1 bridges, the verification of the deck should be carried out using the design action effects from the analysis.

(3) The resistance and stability should be verified in accordance with the relevant rules of prEN 1993-2[2] or prEN 1994-2[2] for steel or composite decks, respectively.

6.3.5 Foundations

6.3.5.1 General

(1) Bridge foundation systems should conform to prEN 1998-5:2022, 9.

(2) Soil-structure interaction should be assessed when it is necessary using prEN 1998-5:2022, 8.

6.3.5.2 Design action effects

(1) For the purpose of resistance verifications, for bridges designed on the basis of a forced-based approach, the design action effects on the foundations should be determined in accordance with prEN 1998-5:2022, 9.2.

6.3.5.3 Resistance verification

(1) The resistance verification of the foundations should be carried out in accordance with prEN 1998-5:2022, 9.3 and 9.4.

6.3.6 Connections

(1) Minimum overlap lengths at connections should be verified according to 8.5.

(2) Uplift of all bearings at the same support, before the target displacement is reached, should be avoided, unless it has no detrimental effect on the bearings.

(3) Uplift may be considered non-detrimental if the response of the bearing is unaltered when contact is re-established.

NOTE Uplift is typically non-detrimental for slider bearings if the lateral displacement does not exceed their capacity.

6.3.7 Concrete abutments

6.3.7.1 General requirements

(1) All main structural components of the abutments shall be designed to remain elastic under the design seismic action.

NOTE Abutment back-walls are structural components that can be designed as sacrificial elements and be considered ancillary, see 4.3.5(8).

(2) The design of the foundation should be in accordance with 6.3.5. Depending on the structural function of the horizontal connection between the abutment and the deck, the provisions of 6.3.7.2 and 6.3.7.3 should be applied.

6.3.7.2 Abutments flexibly connected to the deck

(1) If abutments are flexibly connected to the deck through elastomeric bearings, these (or the seismic links, if provided) may be designed to contribute to the seismic resistance of the deck, but not to that of the abutments.

(2) Seismic design of these abutments should be carried on according to prEN 1998-5:2022, 10.

6.3.7.3 Abutments rigidly connected to the deck

(1) When the connection of the abutment to the deck is considered as rigid, 10 should be applied.

6.3.8 Verification for the displacement-based approach

(1) Verification should be carried out in local terms, according to prEN 1998-1-1:2022, 6.7.2(2).

(2) Verification for bending, with or without axial force, within critical regions, should be carried out in terms of local deformations δ, e.g. chord rotation. Local deformation demand should be obtained from the analysis, according to 5.2.3, while local deformation capacity should be evaluated, depending on the material, according to prEN 1998-1-1:2022, 7. The value of $\alpha_{\mathrm{SD},\theta}$ should be 0,5.

(3) Verification outside critical regions for bending, with or without axial force, as well as shear, should be carried out in terms of forces. Resistances should be calculated according to 6.3.3 to 6.3.7. Design action effects should be those obtained from the analysis according to 5.2.3.

(4) The design of the foundation should be in accordance with 6.3.5.1. For the purpose of resistance verifications, for bridges designed on the basis of nonlinear analysis, the design action effects on the foundations should be those obtained from the analysis.

6.4 Verification to other limit states

6.4.1 Verification of Near Collapse (NC) limit state

(1) In case the NC limit state is used, verifications should be carried out with the displacement-based approach, via nonlinear static or response-history analysis, with the seismic action specified in 4.2.1(1).

NOTE The force-based approach relies on linear analysis. The seismic action for the NC limit state can drive the structure into the nonlinear range to an extent where results of a linear analysis are less reliable than they are at the SD limit state.

(2) Chord rotation capacity should be evaluated, depending on the material, according to prEN 1998-1-1:2022, 7.

6.4.2 Verification of Damage Limitation (DL) limit state

(1) In case the DL limit state is required, verification may be carried out with the force-based or the displacement-based approach.

(2) If the force-based approach is used, q = 1 should be used.

(3) Relevant criteria should be agreed with the relevant authority.

6.4.3 Verification of Operational (OP) limit state

(1) In case the OP limit state is required, verification may be carried out with the force-based or the displacement-based approach.

(2) If the force-based approach is used, q = 1 should be used.

(3) Relevant criteria should be agreed with the relevant authority.

7 Detailing for ductility

7.1 General

(1) Clause 7 should be applied to primary seismic members (piers and abutments) of bridges designed for DC2 and DC3 through plastic hinging and aims to ensure a minimum level of curvature/rotation ductility at the plastic hinges.

7.2 Concrete piers

7.2.1 General

(1) 7.2 should be applied to pier columns as well as to pier cap-beams when they are designed as dissipative in case of multi-column piers.

7.2.2 Longitudinal reinforcement

(1) The total longitudinal reinforcement ratio ρ_l should not be smaller than 0,5 % and should not be larger than 3 %.

(2) The diameter of the longitudinal bars should not be smaller than 16 mm.

7.2.3 Critical region

(1) When $\eta_k = N_{Ed}/A_c f_{ck} \leq 0,3$, the regions up to a distance l_{cr} from end sections where potential plastic hinges can form should be considered as being critical regions. l_{cr} should be estimated as the largest of the values given by a) and b):

a) the depth of the pier section within the plane of bending (perpendicular to the axis of rotation of the hinge);

b) the distance from the point of maximum moment to the point where the design moment is less than 80 % of the value of the maximum moment, but not larger than 1,5 times the depth of the pier section from a).

(2) When $0,3 < \eta_k \leq 0,6$, distance l_{cr} determined in (1) should be increased by 50 %.

(3) The length of critical regions (l_{cr}), defined in (1) or (2) as appropriate, should be used exclusively for detailing the reinforcement of the plastic hinge. It should not be used for estimating the plastic hinge rotation.

(4) The total longitudinal reinforcement ratio ρ_l in the critical region should not be smaller than 1 % and should not be greater than 3 %.

7.2.4 Confinement

7.2.4.1 General requirements

(1) Confinement should be ensured within the critical regions of the primary seismic members, using hoops and cross-ties of at least 10 mm in diameter, provided with a pattern such that the cross-section benefits from the triaxial stress conditions produced by the hoops and cross-ties.

(2) Confinement should be implemented through rectangular or circular hoops and/or cross-ties or through spirals.

NOTE If spirals are used, it is recommended to arrange them in two or more independent strands.

(3) Interlocking spirals/hoops may be used for confining approximately rectangular sections. The distance between the centres of interlocking spirals/hoops should not exceed 0,6 D_{sp}, where D_{sp} is the diameter of the spiral/hoop (see Figure 7.1).

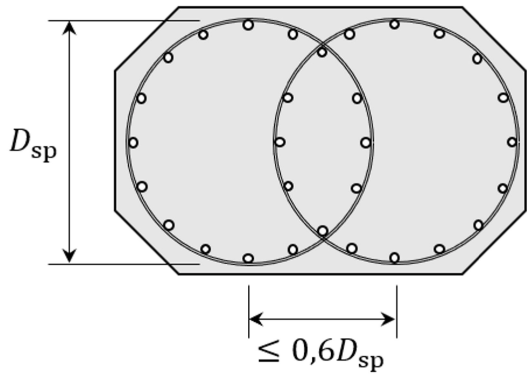


Figure 7.1 — Typical confinement detail in concrete piers using interlocking spirals/hoops

(4) The quantity of confining reinforcement should be defined through the mechanical reinforcement ratio given by Formula (7).1).

$$\omega_{wd} = \rho_w \frac{f_{yd}}{f_{cd}} \tag{7.1}$$

where ρ_w is the transverse reinforcement volumetric ratio, defined by a) or b), as appropriate:

a) in rectangular sections, ρ_w is defined by Formula (7).2).

$$\rho_w = \frac{A_{sw}}{s_L b} \tag{7.2}$$

where

A_{sw}	is the total area of a layer of hoops or ties in the one direction of confinement;
s_L	is the spacing of hoops or ties in the longitudinal direction;
b	is the dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the outside of the perimeter hoop.

b) in circular sections, ρ_w is the volumetric ratio of the spiral or hoop reinforcement relative to the concrete core given by Formula (7).3).

$$\rho_{w} = \frac{4A_{sp}}{D_{sp}s_{L}} \tag{7.3}$$

where

A_{sp} is the area of the spiral or hoop bar;

D_{sp} is the diameter of the spiral or hoop bar;

s_L is the spacing of these bars.

NOTE If different spacing s_L is used for different hoop patterns (e.g. internal vs. external), within the critical region, a value of the reinforcement volumetric ratio is calculated for each group of transverse reinforcement, and then values are added to get the total volumetric ratio.

(5) The amount of confining reinforcement should be larger than the minimum $\omega_{wd,min}$ determined as given in a) or b), as appropriate:

a) for rectangular hoops and cross-ties, $\omega_{wd,min}$ should be taken equal to 0,08 for DC2 and 0,12 for DC3;

b) for circular hoops or spirals, $\omega_{wd,min}$ should be taken equal to 0,12 for DC2 and 0,18 for DC3.

(6) When rectangular hoops and cross-ties are used, the minimum reinforcement condition should be satisfied in both transverse directions.

(7) In cases of deep compression zones, the confinement should extend at least up to the depth where the value of the compressive strain exceeds 0,5 ε_{cu2}.

(8) When confining reinforcement is required, a) to c) should be applied:

a) the amount specified in (5) should be provided over the entire length of the critical region;

b) outside the critical region, the transverse reinforcement may be gradually reduced to the amount required by other criteria;

c) the amount of transverse reinforcement provided over an additional length l_{cr} adjacent to the critical region should not be less than 50 % of the amount of the confining reinforcement required in the critical region.

7.2.4.2 Rectangular sections

(1) The spacing of hoops or ties in the longitudinal direction, s_L, should satisfy both conditions given by a) and b):

a) $s_L \leq 6$ times the longitudinal bar diameter, d_{bL};

b) $s_L \leq 1/5$ of the smallest dimension of the confined concrete core, to the hoop centre line.

(2) The transverse distance s_T between hoop legs or supplementary cross-ties should not exceed the smallest of the values given by a) and b):

a) 1/3 of the smallest dimension b_{min} of the concrete core to the hoop centre line;

b) 200 mm for $b_{min} \leq 1{,}0$ m, 300 mm for $b_{min} > 1{,}5$ m and linear in between (see Figure 7.2).

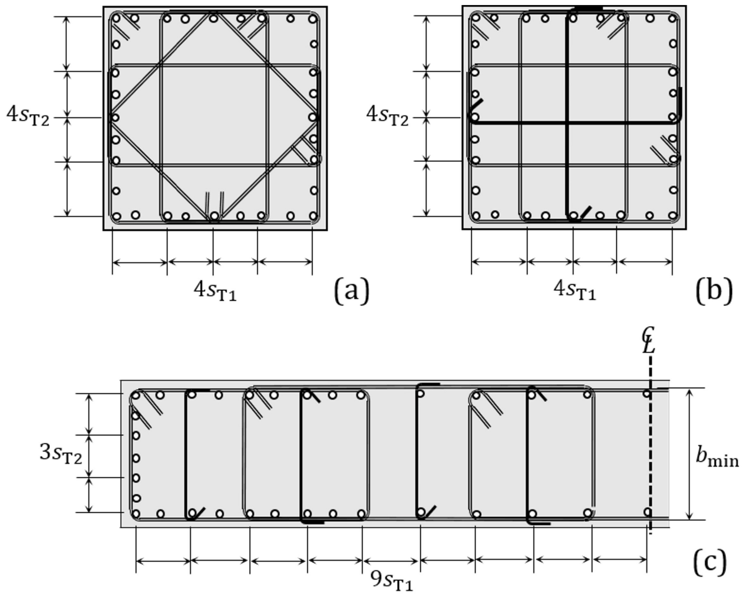


Figure 7.2 — Typical confinement details in concrete piers with rectangular section using overlapping rectangular hoops and cross-ties: (a) four closed overlapping hoops; (b) three closed overlapping hoops plus cross-ties; (c) closed overlapping hoops plus cross-ties

(3) Bars inclined at an angle $\alpha > 0$ to the transverse direction in which ρ_w refers to, should be assumed to contribute to the total area A_{sw} of expression (6.4) by their area multiplied by cos α.

(4) While reinforcement required to resist shear runs the entire width, in large rectangular wall-type sections, confinement reinforcement may be terminated when it enters the concrete core by 1000 mm plus the anchorage length.

7.2.4.3 Circular sections and sections confined with spiral or hoops

(1) The spacing of spiral or hoop bars, s_L, should satisfy both conditions given in a) and b):

a) $s_L \leq 6$ times the longitudinal bar diameter, d_{bL};

b) $s_L \leq 1/5$ of the diameter of the confined concrete core to the hoop centre line.

7.2.4.4 Hollow-core sections

(1) In hollow-core sections, confinement should be provided as for wall sections using closed overlapping hoops plus cross-ties, as in Figure 7.3.

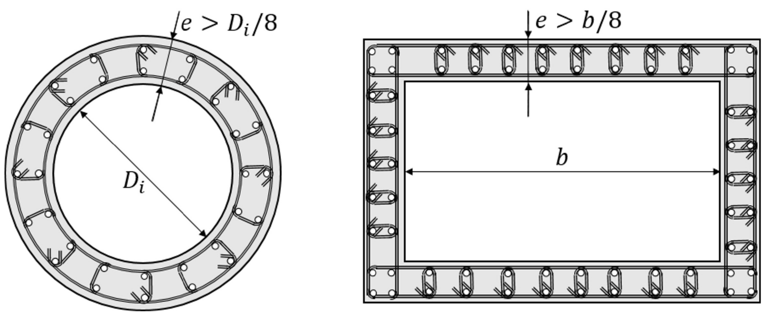


Figure 7.3 — Typical confinement detail in hollow piers

7.2.5 Buckling of longitudinal compression reinforcement

(1) Buckling of longitudinal reinforcement shall be avoided along potential hinge areas, even after several cycles into the post-yield region.

(2) To meet (1), all main longitudinal bars should be restrained against outward buckling by transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars at a (longitudinal) spacing s_L not exceeding $5d_{bL}$, where d_{bL} is the diameter of the longitudinal bars.

(3) Along straight section boundaries, restraining of longitudinal bars should be achieved in complying to either a) or b):

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 should have 135°-hooks at one end and 135°- or 90°-hooks at the other. Cross-ties with 135°-hooks at both ends may consist of two lapped spliced pieces. If $\eta_k > 0{,}30$, 90°-hooks should not be used for the cross-ties. If the cross-ties have dissimilar hooks at the two ends, these hooks should be alternated in adjacent cross-ties, both horizontally and vertically. In sections of large dimensions, the perimeter tie may be spliced using appropriate lapping length combined with hooks (Figure 7.4);

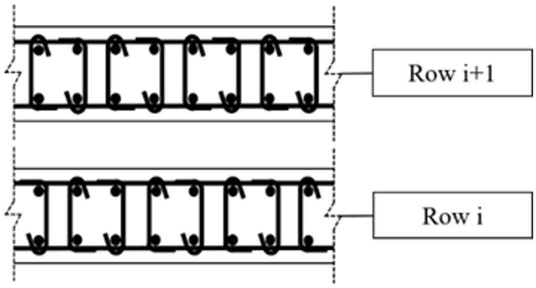


Figure 7.4 — Examples of cross-ties in critical regions

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

(4) The minimum amount of transverse ties should be as given by Formula (7).4).

$$\min\left(\frac{A_t}{s_L}\right) = \frac{\sum A_s f_{ys}}{1{,}6 f_{yt}} \left(mm^2 / m\right) \tag{7.4}$$

where

A_t	is the area of one tie leg, in mm²;
s_L	is the spacing of the legs along the axis of the member, in m;
ΣA_s	is the sum of the areas of the longitudinal bars restrained by the tie, in mm²;
f_{yt}	is the yield strength of the tie;
f_{ys}	is the yield strength of the longitudinal reinforcement.

7.2.6 Other rules

(1) The longitudinal reinforcement should remain constant and fully effective over the length of the critical region l_{cr} specified in 7.2.1.

(2) The distance of the first hoop to the end section of the member corresponding to the critical region should not be larger than 50 mm.

(3) Due to the potential loss of concrete cover in the plastic hinge region, the confining reinforcement should be anchored by 135°-hooks (unless a 90°-hook is used in accordance with 7.2.5(3)a) surrounding a longitudinal bar plus adequate extension (minimum of 10 diameters) into the core concrete.

(4) Similar anchoring or a full-strength weld may be implemented for the lapping of spirals or hoops within critical regions. In this case laps of successive spirals or hoops, when located along the perimeter of the member, should be staggered in accordance with prEN 1992-1-1:2021, 11.5.

(5) There should be no splicing by lapping or welding of longitudinal reinforcement within the critical region.

NOTE For mechanical couplers, see prEN 1998-1-2[2], 10.11.3.

7.2.7 Hollow piers

(1) Unless appropriate justification is provided, the ratio b/h of the clear width b to the thickness h of the walls, in the critical region (length l_{cr} in accordance with 7.2.1) of hollow piers with a single or multiple box cross-section, should not exceed 8.

(2) For hollow circular piers, the limitation in (1) should be applied to the ratio D_i/h, where D_i is the inside diameter.

7.2.8 Joints adjacent to critical regions

7.2.8.1 General

(1) Any joint between a vertical ductile pier and the deck or a foundation member adjacent to a critical region in the pier should satisfy 7.2.8.2.

NOTE The pier is indexed in 7.2.8 with "c" (for "column"), while any other member framing into the same joint is referred to as "beam" and indexed with "b".

7.2.8.2 Reinforcement minimum ratios and arrangement in the joints

(1) A minimum amount of shear reinforcement should be provided in the joint panel in both horizontal directions, in the form of closed links. The joint reinforcement ratio should not be less than ρ_{min} given by Formula (7).5).

$$\rho_{min} = \frac{f_{ctd}}{f_{sy}} \tag{7.5}$$

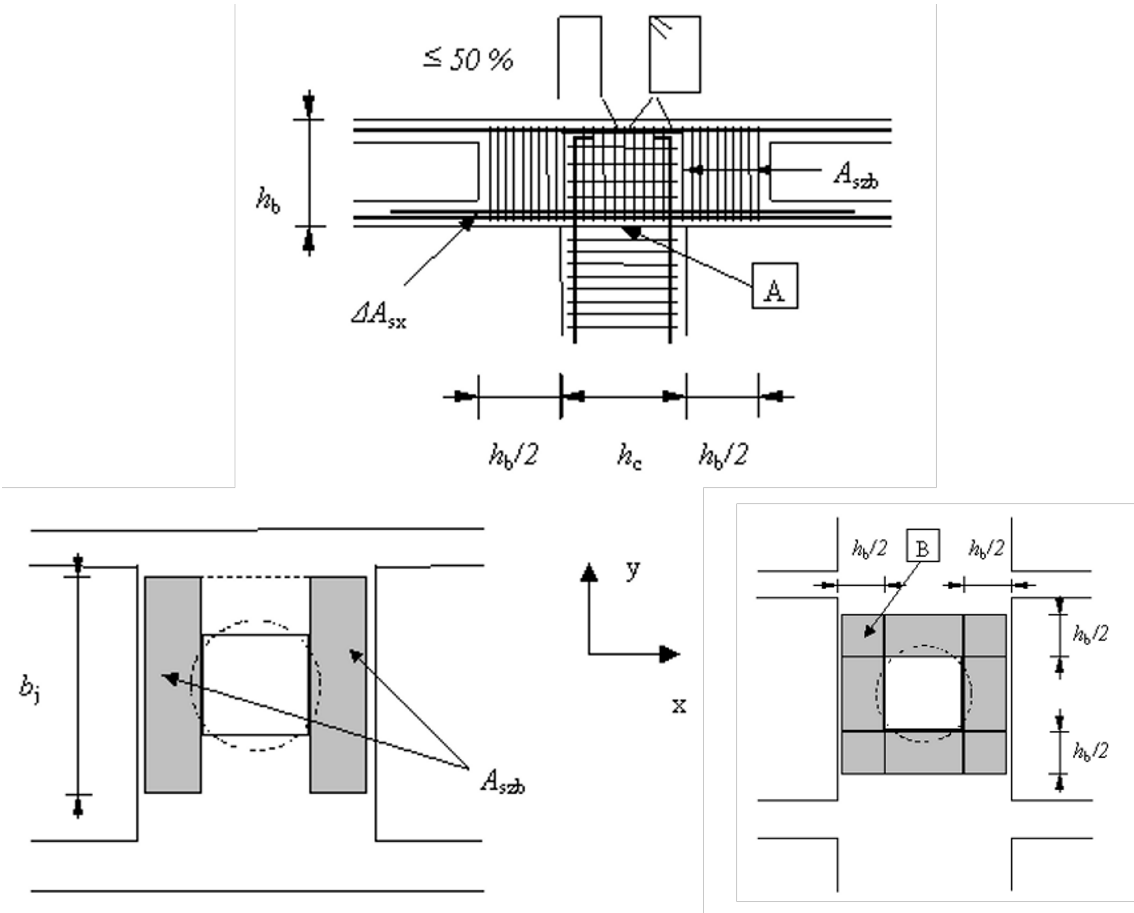
(2) Vertical stirrups should enclose the horizontal member (deck, cap-beam, foundation pile-cap or footing) longitudinal reinforcement at the face opposite to the pier. Horizontal stirrups should enclose the pier vertical reinforcement, as well as horizontal member horizontal bars anchored into the joint. Pier stirrups/hoops should be continued into the joint.

(3) Up to 50 % of the total amount of vertical stirrups required in the joint may be U-bars, enclosing the longitudinal reinforcement of the horizontal member at the face opposite to the pier (see Figure 7.5).

(4) 50 % of the bars of the top and bottom longitudinal reinforcement of the horizontal member, when continuous through the joint body and adequately anchored beyond it, may be taken into account for covering the required horizontal joint reinforcement area A_{sx}.

(5) The longitudinal (vertical) pier reinforcement should penetrate the horizontal member, up to its reinforcement layers at the face opposite to the pier-horizontal member interface. In the direction of flexure of the plastic hinge, the bars of both tensile regions of the pier should be anchored by a 90° hook directed towards the centre of the pier.

(6) In geometrical configurations where the amount of required reinforcement A_{sz} and/or A_{sx} impairs the feasible constructability of the joint, then the alternative arrangement, described in (7) and (8), may be applied (see Figure 7.5).



Key

A pier-horizontal member interface

B stirrups in common areas count in both directions

Figure 7.5 — Alternative arrangements of joint reinforcement, with: (a) vertical section within plane xz; (b) plan view for plastic hinges forming in the x-direction; (c) plan view for plastic hinges in the x- and y-directions

(7) Vertical stirrups of amount $\rho_{1z} \geq \rho_{min}$, should be preferentially placed within the joint body. The remaining area $\Delta A_{sz} = (\rho_z - \rho_{1z})\, b_j\, h_c$, should be placed on each side of the horizontal member, within the joint width b_j and not further than $0{,}5h_b$ from the corresponding pier face.

(8) The horizontal reinforcement within the joint body may be reduced by $\Delta A_{sx} \leq \Delta A_{sz}$, provided that the ratio of the horizontal reinforcement remaining within the joint body satisfies Formula (7).8).

(9) The tensile reinforcement of the horizontal member top and bottom fibres at the faces of the pier should then be increased by ΔA_{sx}, over the reinforcement required in the relevant "beam" sections for the verification in flexure under capacity design effects. Additional bars to cover this requirement should be placed within the joint width b_j; these bars should be adequately anchored, to be fully effective at a distance h_b from the pier face.

7.3 Steel piers

(1) For DC2 and DC3 bridges, the detailing rules of prEN 1998-1-2[2], 11.8, 11.9, 11.10, and 11.12, as modified in 6.3.4 of the present standard, should be applied.

(2) The detailing rules for connections in prEN 1998-1-2[2], Annex E, should be applied.

7.4 Foundations

7.4.1 Spread foundation

(1) Spread foundations such as footings, rafts, box-type caissons, piers, etc., shall not enter the plastic range under the design seismic action. Their design should comply with prEN 1998-5:2022, 9.4.

NOTE Hence, they do not require special reinforcement detailing.

7.4.2 Pile foundations

(1) Design of reinforced concrete pile foundations of bridges should comply with prEN 1998-5:2022, 9.5. In particular, when it is not feasible to avoid localized hinging in the piles using the capacity design procedure, pile integrity and ductile behaviour should be ensured.

(2) For design of wooden pier piles the species of prEN 1995-1-1[2], Figure D.1, should be applied.

8 Specific rules for bridges equipped with antiseismic devices

8.1 General

(1) Clause 8 should be used in addition to prEN 1998-1-1:2022, 6.8, for bridges equipped with antiseismic devices.

8.2 Seismic action, basic requirements and compliance criteria

(1) In fully isolated bridges, the superstructure (i.e. the deck) should remain within the elastic range under the capacity design effects given in prEN 1998-1-1:2022, 6.8.2.2(6).

(2) With the exception of (3), the substructure (i.e. supports) of fully isolated bridges and the secondary structural members (i.e. isolated supports) of partially isolated bridges should be designed as non-dissipative. Verifications as defined for DC1 in 6 should be used for them. In particular, global or local ductility conditions may be neglected.

(3) For tall heavy piers or pylons (i.e. where pier self-weight fundamental vibration mode contribution exceeds 50 % of the total design bending moment at the base), in moderate or high seismic action class, verifications and detailing as defined for DC2 in (6) and (7) should be adopted.

(4) For the application of prEN 1998-1-1:2022, 6.8.2.2(9), the action effects corresponding to the elastic range may be calculated with $q = q_S$.

(5) No uplift of seismic isolators carrying vertical force should occur in the seismic design situation. In case of uplift forces, these should be handled by choosing suitable isolators and/or restrainers used to prevent uplifting.

8.3 General provisions concerning antiseismic devices

(1) In fully isolated bridges, prEN 1998-1-1:2022, 6.8.2.3(11), may be considered satisfied if vertical deformations of the seismic isolators are less than 5 % of the horizontal deformations in the seismic design situation. This condition may be neglected if sliding or elastomeric bearings are used as seismic isolators.

8.4 Methods of analysis

8.4.1 General

(1) The basic requirements in prEN 1998-1-1:2022, 6.8.5.1, should be satisfied.

(2) The analysis methods in a) to c) may be used for bridges equipped with antiseismic devices:

a) equivalent linear lateral force method;

b) equivalent linear response spectrum method;

c) response-history analysis.

(3) In fully isolated bridges, multi-mode response spectrum analysis or response-history analysis (prEN 1998-1-1:2022, 6.8.5.4) may be performed on the basis of nominal essential properties, instead of on the basis of UBDPs and LBDPs, provided that:

a) the conditions of prEN 1998-1-1:2022, 6.8.5.3(2), are met;

b) the design seismic displacements d_E, resulting from a fundamental mode equivalent linear response spectrum analysis (prEN 1998-1-1:2022, 6.8.5.3), based on UBDPs and LBDPs, do not differ from that corresponding to the design properties by more than ± 15 %.

(4) In fully isolated bridges, the effects of the vertical component of the seismic action may be determined by linear response spectrum analysis, regardless of the method used for the determination of the response to the horizontal seismic action.

8.4.2 Equivalent linear lateral force method

(1) In this method, the deck should be considered as rigid according to 5.2.2.2(5).

(2) The shear force transferred through the isolating interface in each principal direction should be determined considering the superstructure as a single-degree of freedom system using a) to d):

a) the effective stiffness of the isolation system, K_{eff};

b) the effective damping of the isolation system, $\xi_{eff;}$

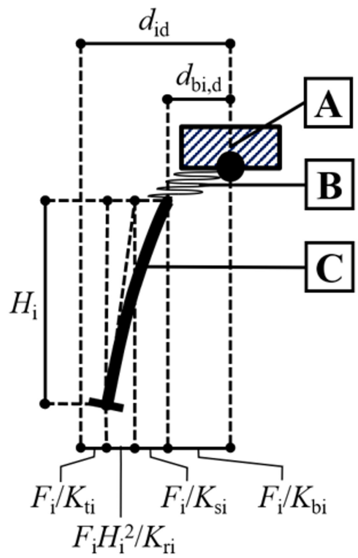
c) the mass of the superstructure, M_d;

d) the spectral acceleration $S_e(T_{eff},\eta_{eff})$ corresponding to the effective period, T_{eff}, and to the damping correction factor, $\eta_{eff} = \eta_{eff}(\xi_{eff})$.

(3) For a pier of height H_i with a displacement stiffness K_{si} (force/displacement), supported by a foundation with translational stiffness K_{ti} (force/displacement) and rotational stiffness K_{ri} (moment/rotation), and carrying isolator unit i with effective stiffness K_{bi} (force/displacement), the composite stiffness $K_{eff,i}$ may be calculated as given by Formula (8).1) (see Figure 8.1).

$$\frac{1}{K_{\mathrm{eff,i}}}=\frac{1}{K_{\mathrm{bi}}}+\frac{1}{K_{\mathrm{ti}}}+\frac{1}{K_{\mathrm{si}}}+\frac{H_{\mathrm{i}}^{2}}{K_{\mathrm{ri}}} \tag{8.1}$$

NOTE The flexibility of the isolator is such that the corresponding relative displacement $d_{bi} = F_i/K_{bi}$ is typically much larger than the other components of superstructure displacement. For this reason, the effective damping of the system depends on the sum of dissipated energies of the isolators and/or dampers (when present), ΣE_{Di}, and the relative displacement of the isolator is practically equal to the displacement of the superstructure at this point ($d_{bi}/d_{id} = K_{eff,i}/K_{bi} \approx 1$).



Key

A superstructure

B isolator *i*

C support *i*

Figure 8.1 — Composite stiffness of support and isolator *i*

(4) For the determination of the seismic action effects on the isolating system and the substructures in the principal transverse direction (i.e. direction *y*), the influence of plan eccentricity as required by prEN 1998-1-1:2022, 6.8.5.3(4), in the longitudinal direction e_x (between the effective stiffness centre and the centre of mass of the deck) on the superstructure displacement d_{id} over pier *i*, should be evaluated as given by Formula (8).2).

$$d_{\mathrm{id}}=\delta_{\mathrm{i}}d_{\mathrm{cd}} \tag{8.2}$$

with d_{cd} the displacement at the centre of mass of the superstructure (deck) and δ_i given by Formula (8).3).

$$\delta_{\mathrm{i}}=1+\frac{e_{\mathrm{x}}}{r\,r_{\mathrm{x}}}x_{\mathrm{i}} \tag{8.3}$$

with r_x given by Formula (8).4).

$$r_x^2 = \frac{\Sigma\left(x_i^2 K_{yi} + y_i^2 K_{xi}\right)}{\Sigma K_{yi}} \quad (8.4)$$

where

e_x	is the eccentricity in the longitudinal direction;
r	is the radius of gyration of the deck mass about the vertical axis through its centre of mass;
x_i and y_i	are the coordinates of pier i relative to the effective stiffness centre;
K_{yi} and K_{xi}	are the effective composite stiffnesses of isolator unit and pier i in the y and x directions, respectively.

NOTE In straight bridges usually $y_i << x_i$. In such cases, the term $y_i^2 K_{xi}$ in expression (8.4) may be omitted.

8.4.3 Equivalent linear response spectrum method

(1) The modelling of the substructures should reflect with sufficient accuracy the distribution of their stiffness properties and at least the rotational stiffness of the foundation. When the pier has significant mass and height, or if it is immersed in water, its mass distribution should also be properly modelled.

8.4.4 Response-history analysis

(1) prEN 1998-1-1:2022, 6.8.5.5, should be applied.

8.5 Minimum overlap length at connections

(1) At supports where relative displacement between supported and supporting members is intended under seismic conditions, a minimum overlap length should be provided.

(2) The overlap length should be such as to ensure that the function of the support is maintained under extreme seismic displacements.

(3) At an end support of an abutment, the minimum overlap length l_{ov} may be estimated as given by Formula (8).5).

$$l_{ov} = l_m + d_{eg} + d_{es} \quad (8.5)$$

where

l_m	is the minimum support length ensuring the safe transmission of the vertical reaction, but no less than 400 mm;
d_{eg}	is the effective displacement, given by Formula (8).6), of the two parts due to the spatial variation of the seismic ground displacement; when the bridge site is at a distance less than 5 km of a known seismically active fault, capable of producing a seismic event of magnitude $M \geq 6,5$, and unless a specific seismological investigation is available, the value of d_{eg} to be used should be taken as double that obtained from Formula (8).6).

$$d_{eg} = \varepsilon_e L_{eff} \leq 2d_g \quad (8.6)$$

where ε_e is given by Formula (8).7).

$$\varepsilon_e = \frac{2d_g}{L_g} \tag{8.7}$$

where

d_g is the expected ground displacement under the design seismic action according to prEN 1998-1-1:2022, 5.2.2.4;

L_g is the distance parameter specified in prEN 1998-1-1:2022, 5.2.3.2;

L_{eff} is the effective length of the 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 a group of more than one pier, then L_{eff} should be taken as the distance between the support and the centre of the group of piers;

d_{es} is the effective seismic displacement of the support due to the deformation of the structure, estimated as given in a) or b):

a) for decks connected to piers either monolithically or through fixed bearings acting as full seismic links, by Formula (8).8.

$$d_{\mathrm{es}} = d_{\mathrm{Ed}} \tag{8.8}$$

where d_{Ed} is the total design value of the longitudinal displacement in the seismic design situation determined in accordance with Formula (4.1).

b) for decks connected to piers or to an abutment through seismic links with slack equal to *s*, by Formula (8).9.

$$d_{\mathrm{es}} = d_{\mathrm{Ed}} + s \tag{8.9}$$

NOTE In this context, "full connection" means a connection of the deck or deck section to a substructure member, either monolithically or through fixed bearings, seismic links, or STUs without a force limiting function.

(4) 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 in accordance with (3). At an end support of a deck section on an intermediate pier, l_{ov} should be taken as the value estimated in accordance with (3) plus the maximum displacement of the top of the pier in the seismic design situation, d_E.

9 Specific rules for cable-stayed and extradosed bridges

9.1 General

(1) Clause 9 should be applied only to cable-stayed and extradosed bridges, in addition to prEN 1992-1-1:2021, Annex K, K.13.4, and prEN 1993-1-11[2]. Verifications of structural members not explicitly mentioned in Clause 9 should be carried out as in Clause 6.

(2) If antiseismic devices are used, they should fulfil Clause 8 (partially isolated structures).

9.2 Basis of design

(1) The calculation of the effect of the seismic action in the bridge should take into account the influence of the construction sequence on the effect of permanent actions.

9.3 Modelling and structural analysis

(1) Response-history analysis should be the preferred method of analysis for cable-stayed bridges. The dynamic analysis should start from the deformed configuration of the bridge under the permanent actions.

NOTE The seismic response of cable-stayed bridges can present significant material and/or geometric nonlinearities due to nonlinear response of the cables, second-order effects in the deck and the pylons, and large displacements.

(2) In low seismic action class, multi-mode equivalent linear response spectrum analysis may be used for cable-stayed bridges without antiseismic devices.

(3) The modelling of the bridge should reflect with sufficient accuracy the coupling between the transverse bending of the deck and its torsional response.

NOTE This coupling is governed by the distribution of mass and stiffness in the deck as well as the cable arrangement.

(4) Second-order effects should be taken into account in the calculation when they are relevant due to slenderness of the deck and/or the pylons, according to 5.1.3.

(5) A global three-dimensional model should be used to capture the flexural-torsional coupling as well as the geometric nonlinearity of the cable elements, pylons and deck.

(6) The stay cable internal damping coefficient should be consistent with the calculated cable displacement.

NOTE The total damping depends on the relative contribution of each member (pylons, cable-system and deck), and their interaction, and can be significantly lower than 5 % of the critical damping.

(7) Energy dissipation of antiseismic devices located at the deck-pylon interface or at the cables should be considered explicitly in the analysis by their nonlinear response.

(8) If abutments' and piers' foundations are not included in the model, the model should account for the effect of their flexibility through foundation impedances, according to prEN 1998-5:2022, 8.

NOTE prEN 1998-5:2022, informative Annex D, gives guidance for calculating foundation impedances.

9.4 Verifications

9.4.1 General

(1) In cable-stayed bridges, all the components except the antiseismic devices should remain within the elastic range in the seismic design situation.

(2) In any horizontal direction, the displacement of the deck should be limited to avoid impact between deck and pylon.

(3) Verification of displacement compatibility should take into account all potential aggravating effects such as second-order effects, contribution of higher modes or spatial variability of seismic demand (including active fault crossing).

(4) In multi-leg pylons, the additional axial load due to seismic response should be considered at each individual leg.

9.4.2 Avoidance of brittle failure of specific non-ductile components

(1) Non-ductile structural components, such as fixed bearings, sockets and anchorages for cables and stays, and other non-ductile connections, should be designed to resist capacity design effects. These capacity design effects should be taken equal to the minimum of those obtained in the seismic design

situation with $q = 1$ and those obtained in the assumption that the relevant ductile members (e.g. the cables) have developed their strength, multiplied by an overstrength factor $\gamma_{Rd} \geq 1{,}3$.

(2) The verification in (1) may be omitted if it can be demonstrated that the integrity of the structure is not affected by failure of such connections. This demonstration should also address the possibility of sequential failure, such as it can occur in stays of cable-stayed bridges.

9.5 Detailing

(1) In cable-stayed bridges, the deck should be continuous.

(2) Antiseismic devices may be used at the deck-pylon interface or at the deck-abutment interface in order to provide restraints and/or energy dissipation.

NOTE Other layouts are possible. For instance, special seismic cable damping devices can be used.

(3) Horizontal deck restraint in the transverse direction should be provided at the deck-pylon interface and/or at the abutments.

NOTE 1 In the transverse direction, the resistance is provided mainly by the deck-tower interface since the cables provide little restraint to deck movements. In the longitudinal direction, the resistance is provided by both the cable-pylon system and the deck-pylon interface, if any.

NOTE 2 The cables can be either connected to the pylon top (fan arrangement) or distributed over the height in a harp or semi-fan type of arrangement. Distributed type of arrangements provide a stiffer solution than a fan arrangement.

(4) Vertical restraint of the deck at the deck-to-pylon interface may be used.

10 Specific rules for integral abutment bridges

10.1 General

(1) Clause 10 should be used for the modelling, analysis and verification of integral abutment bridges.

NOTE Integral abutment bridges are continuous bridges where the connections between the deck and both the abutments are monolithic (Figure 10.1). Unless specific provisions are taken to avoid or minimize interaction, the vibration of the structure cannot happen independently of that of the surrounding medium (the approach embankments or the natural soil, depending on whether the bridge is above-ground or embedded up to the deck level).

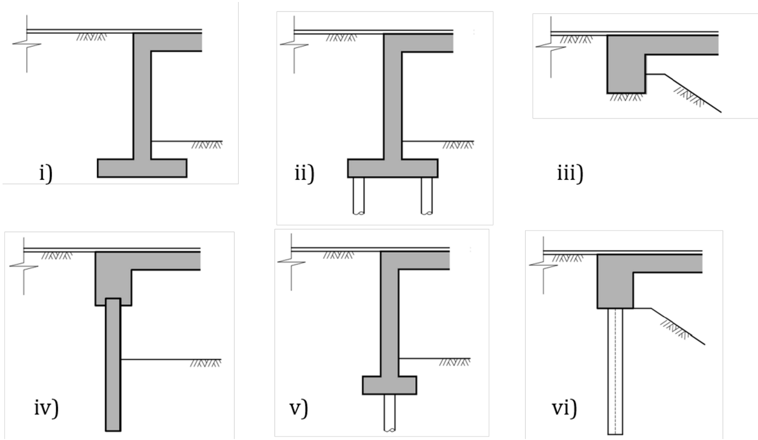


Figure 10.1 — Types of integral abutment bridges: i) full height integral abutment on pad footing; ii) full height integral abutment on piles; iii) bank pad; iv) embedded wall integral abutment; v) full height integral abutment on single row of piles; vi) bank pad on single row of piles. Other types are possible

(2) Clause 10 may be applied when bridges are semi-integral, i.e. the rigid connection does not include all degrees of freedom and is realized through fixed bearings or seismic links that restrain the relative movement between the deck and one or both abutments.

10.2 Basis of design

(1) The calculation of the effects of the seismic action should incorporate the effects of interaction between soil and abutments.

(2) Action effects should be calculated using both upper and lower bound estimates of soil properties.

NOTE The requirement in (2) is in order to arrive at results which are on the safe side both for the abutments and for the piers.

(3) The calculation of the effects of the seismic action may incorporate the effects on the soil pressures against the abutments of a) and b):

a) the construction sequence;

b) thermal cycling previous to the occurrence of an earthquake, if no special provisions are taken to prevent interaction and the material (soil or backfill) in contact with the abutments is coarse-grained.

NOTE 1 Interaction between soil and structure occurs at the foundation and through earth pressures on the vertical abutment wall. The initial pressure distribution resulting from the construction sequence is important in determining the dynamic pressure distribution during the earthquake.

NOTE 2 In coarse-grained soils and backfill, cyclic deformation induces particle realignment and progressive compaction that cause stiffening. This phenomenon, known as ratcheting, is associated, e.g. with repeated thermal cycling, and can lead to an increase in the initial at-rest pressures. Ratcheting is not present in fine-grained soils.

(4) Seismic response should be calculated based on kinematic compatibility between the bridge structure and the free-field seismic deformation of the soil and the embankment.

(5) Verification should be carried out considering, for each component of the seismic action, the most unfavourable effects resulting from the application of the actions as defined in 10.3 in one direction or the opposite.

(6) Integral abutment bridges and culverts may be considered to be embedded structures, if the abutments are embedded in stiff natural soil formations (prEN 1998-1-1:2022, Table B.2) over at least 80 % of their lateral area.

(7) Due to difficulties in repair, integral abutment bridges should be designed to DC1.

10.3 Modelling and structural analysis

10.3.1 General

(1) Structural members should be modelled as linear, accounting for cracking of concrete parts, according to 5.1.1(4) to (6).

NOTE Design according to DC1 implies linear response.

(2) The seismic analysis of integral abutment bridges should comply with either a) or b):

a) force-based approach according to 10.3.2;

b) displacement-based approach according to 10.3.3.

10.3.2 Force-based approach

(1) A behaviour factor q = 1,5 should be used, according to Table 5.2. The behaviour factor should be used to divide internal forces due to the seismic action, rather than the spectral acceleration acting on structural masses.

NOTE 1 Internal forces depend on pressures that, together with the foundation reaction, equilibrate the inertia forces on the structural mass. Reduction of spectral acceleration on the structural mass by q would alter the overall distribution of forces between foundation and abutment.

NOTE 2 The value of q coincides with q_S which accounts for the difference between expected and design strength, not for reduction in spectral acceleration due to ductility.

(2) The actions in a) and b) should be taken into account in the longitudinal direction (Figure 10.2):

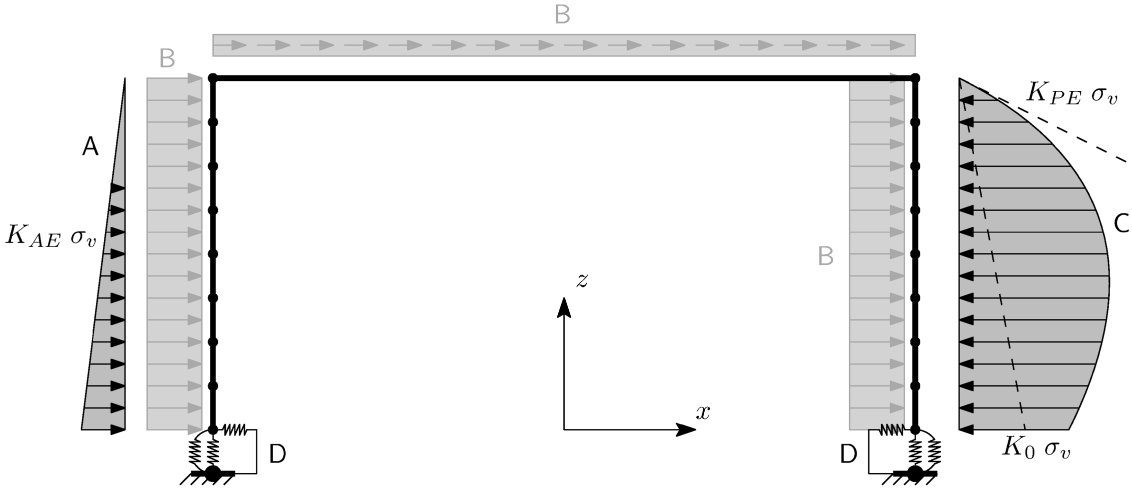
a) total (static plus seismic) earth pressures E_d acting on the abutments in the seismic design situation, calculated according to prEN 1998-5:2022, 10.3.2, duly accounting for the effect of friction between soil and abutment wall. The pressures E_d may be assumed to correspond to the active limit on one abutment (away from which the structure's mass is accelerated, denoted as 'upstream') and intermediate between the at-rest and the passive limit on the other abutment (towards which the structure's mass is accelerated, denoted as 'downstream');

NOTE 1 The motion of soil masses on both sides can be considered to be the same. As a result, when on one side the structure is moving towards the soil, pushing it while contact pressures increase towards the passive limit, on the other side it is moving away from it with displacements that are larger than those corresponding to attainment of the active limit pressures (prEN 1998-5:2022, Annex F, F.3). Depending on the seismic action intensity, the displacement can be not large enough to mobilize the full passive resistance over the entire abutment height, especially when friction is accounted for.

NOTE 2 Maximum internal forces occur when the structure moves towards the soil on the 'downstream' side.

b) inertia forces acting on the mass of the structure, evaluated as the product of structural masses and the maximum response spectral acceleration corresponding to the constant acceleration range of the elastic response spectrum S_{α}, as given in prEN 1998-1-1:2022, 5.2.2.2.

NOTE 3 The structure cannot oscillate with its natural vibration period as if it were not in contact with the surrounding medium. On the other hand, determination of the predominant period of vibration for the structural portion of the soil-embankment-structure system is not feasible within the context of the force-based approach. This period is in general short, larger than T_{A} and likely than T_{B}. Plateau acceleration is thus conservatively employed as an approximation.



Key

A total active earth pressures

B inertia effects on the structural masses

C total passive-side earth pressures

D foundation impedances

Figure 10.2 — Modelling in the force-based approach

(3) In the case of cemented backfill, the earth pressure on the 'upstream' side may be neglected.

(4) The total earth pressures on the 'downstream' side should be evaluated with Formula (10).1).

$$\sigma_{\mathrm{p,mob}}(z) = K_{\mathrm{PE,mob}}(z)\gamma z \tag{10.1}$$

where

$\sigma_{\mathrm{p,mob}}(z)$ is the mobilised passive pressure at depth z from the abutment top;

$K_{\mathrm{PE,mob}}(z)$ is the mobilised passive pressure coefficient in the seismic design situation at depth z;

γ is the weight density of soil or backfill material behind the abutment.

(5) The mobilised passive pressure coefficient may be calculated with Formula (10).2).

$$K_{\mathrm{PE,mob}}(z) = K_{\mathrm{o}} + \left(K_{\mathrm{PE}} - K_{\mathrm{o}}\right) i_{\mathrm{u}}(z) \tag{10.2}$$

where

K_o is the at-rest pressure coefficient;

K_{PE} is the passive pressure coefficient in the seismic design situation according to prEN 1998-5:2022, 10;

$i_u(z)$ is the interpolation function at depth z from the abutment top.

NOTE prEN 1998-5:2022, Annex F, F.3, gives guidance for evaluating the passive pressure coefficient in the seismic design situation.

(6) Function i_u may be evaluated with Formula (10).3).

$$i_u(z) = \frac{u(z)}{az + u(z)} \tag{10.3}$$

where

$u(z)$ is the abutment displacement at depth z from the abutment top;

a is a non-dimensional soil-dependent parameter, equal to 0,1 for loose soil and 0,01 for firm soil.

(7) Consistency between the abutment displacement profile $u(z)$ used to determine the pressures and the profile deriving from the analysis under these pressures should be verified. A linear abutment displacement profile may be used as a first approximation. An initial value for the displacement at the base of the abutment may be obtained dividing half of the seismic inertia force in (2)b) by the total horizontal foundation stiffness evaluated according to (8). The top displacement may be taken as a multiple of the bottom displacement.

NOTE The pressure distribution is intermediate between the at-rest and the passive one, as a function of displacements. These displacements are not known in advance; therefore, iteration is necessary. The suggested bottom displacement is obtained neglecting the foundation rotation, for the sake of simplicity. The top displacement can be anywhere between two and ten times the bottom one. Assuming the full passive resistance is mobilised to avoid iteration is not conservative because it reduces the forces on the foundations.

(8) The model should account for the effect of the flexibility of the abutment and piers foundation.

(9) The effect of foundation flexibility may be accounted for through static foundation impedances according to prEN 1998-5:2022, 8. Group effects may be taken equal to their static values.

NOTE prEN 1998-5:2022, Annex D, gives guidance for calculating foundation impedances of both shallow and deep foundations.

(10) If the bridge is skew (φ > 20°), response in the transverse direction should be obtained from the same spatial model used for the longitudinal response. For smaller skew angles and straight bridges, separate models may be used.

(11) In the transverse direction, analysis may be carried out with any of the methods in 5.2, with due consideration of the deck restraint at the abutments.

(12) For the purpose of determining the soil-abutment stiffness at the deck-abutment connection, the abutment wall may be considered rigid to the foundation level with flexibility contributed only by the foundation.

10.3.3 Displacement-based approach

(1) The displacement-based approach should be implemented by either a) or b):

a) nonlinear static analysis;

b) response-history analysis.

(2) For the purpose of the displacement-based approach, the soil should be modelled as a discretised inelastic continuum.

(3) If (2) is not applied, mutually independent inelastic springs may be used to model the soil in contact with the abutment walls.

NOTE Annex D provides guidance on this aspect.

(4) For response-history analysis, the model should include the entire soil-foundation-structure system. The analysis model should allow for the transmission of seismic waves across the lateral and bottom boundaries of the system, according to prEN 1998-5:2022, 8.5(2).

NOTE 1 In (4), soil means the natural soil deposit beneath the structure, as well as the backfill material and soil, natural or embankment, beside the abutments.

NOTE 2 Annex D also provides guidance on this aspect.

10.3.4 Culverts

(1) Culverts may be analysed using 10.3.2 or 10.3.3 if they do not carry large overburden and provided the bottom slab is included in the model and supported on appropriate foundation impedances. If they carry large overburden, they should be designed as underground structures according to prEN 1998-5:2022, 11.

(2) Overburden on culverts should be considered large if depth of fill over the top slab exceeds 50 % of its span.

(3) Non-frame culverts may be designed according to prEN 1998-1-1:2022, 4.1(7). According to prEN 1998-5,2022, 11.5, analysis should be carried out as specified in prEN 1998-5,2022, 11.3, as in a) or b) below:

a) for all shapes, applying imposed ground deformations as specified in prEN 1998-5:2022, 11.3.2.2;

b) Alternatively, for rectangular, single- or multi-cell box culverts, applying the earth pressures specified in prEN 1998-5:2022, 10.3.

For circular culverts, prEN 1998-5:2022, Annex H, provides an analytical solution for the increment of internal forces. For all other shapes, deformation shall be imposed to a numerical model to read its internal forces.

10.4 Verifications

10.4.1 Verification of Significant Damage limit state

(1) Verifications of structural members should be carried out according to Clause 6.

10.4.2 Verification to other limit states

10.4.2.1 Verification of Damage Limitation limit state

(1) In order that damage of the soil or the embankment behind an abutment rigidly connected to the deck is kept within acceptable limits, the design seismic displacement should not exceed a limit value, d_{lim}, depending on the consequence class of the bridge.

NOTE Values for d_{lim} can be given by the relevant authority or can be found in the national annex.

Annex A
(informative)

Characteristics of earthquake resistant bridges

A.1 Use of this annex

(1) This informative annex provides complementary/supplementary guidance to 4.4.

NOTE National choice on the application of this informative annex is given in the national annex. If the national annex contains no information on the application of this informative annex, it can be used.

A.2 Scope and field of application

(1) This annex gives good practice rules governing earthquake resistant design relative to:

- deck;
- skew angle;
- disconnection of horizontal degrees of freedom at selected supporting members;
- choice of ductility class.

(2) Good practice rules should not be considered as mandatory, as they cannot in practice be all satisfied.

NOTE Satisfying good practice rules allows for more economical design.

A.3 Deck

(1) Bridges with continuous deck should be preferred to those with many movement joints.

NOTE In general, the former behave better in seismic situations.

(2) In exceptionally long bridges, or in bridges crossing non-homogeneous soil formations, the deck should be separated into a number of segments by introducing intermediate movement joints.

A.4 Skew bridges

(1) Lateral restrainers should be used at the abutments to prevent rotation around the vertical axis.

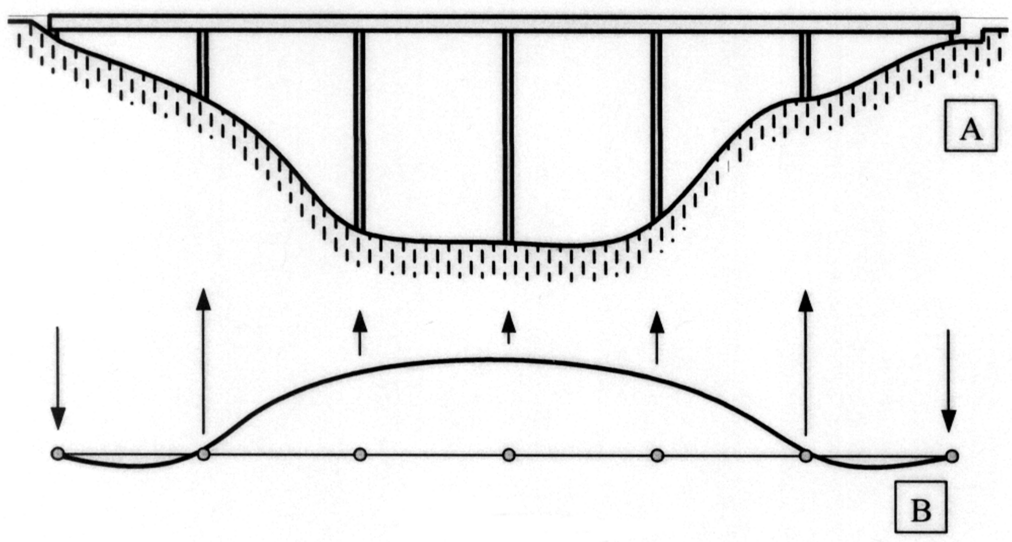
(2) Deck joints at abutments should be designed to accommodate calculated seismic displacements increased by 30 %.

NOTE In skew bridges, rotations around the vertical axis can increase displacements and shocks effects between deck and abutment can increase deck unseating risk.

(3) Highly skew bridges ($\varphi > 45^{\circ}$) should be avoided in cases of high seismic action class.

A.5 Choice of supporting members resisting the seismic action

(1) In the case of bridges with a continuous deck and with transverse stiffness of the abutments and of the adjacent piers which is very large compared to that of the other piers (as it can occur in steep-sided valleys), transversally sliding or elastomeric bearings may be used over the short piers or the abutments to avoid unfavourable distribution of the transverse seismic action among the piers and the abutments, as those exemplified in Figure A.1.



Key

A elevation

B plan

Figure A.1 — Unfavourable distribution of transverse seismic action

(2) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports.

NOTE Large flexibility reduces the magnitude of lateral forces induced by the design seismic action but increases the movement at the joints and moveable bearings and can lead to high second-order effects.

(3) Fixed deck-pier connection may be used as an effective means to limit top displacements of tall and heavy piers under the design seismic action.

NOTE The fixed connection may be provided only for dynamic forces (e.g. by use of velocity-dependent devices), if non-seismic displacements (e.g. thermal, shrinkage) are a concern.

A.6 Choice of ductility class

(1) In low seismic action class (prEN 1998-1-1:2022, 4.1(4)), the type of intended seismic behaviour of the bridge should be decided. If an elastic behaviour is selected, adoption of ductility class DC1 or simplified criteria, in accordance with 4.3.7, may be applied.

(2) In cases of moderate or high seismic action classes (prEN 1998-1-1:2022, 4.1(4)), the earthquake resistance of the bridge should be implemented either by providing for the formation of a dependable plastic mechanism (adoption of ductility class DC2 or DC3) or by using seismic isolation and energy dissipation devices. Depending on the selection of DC2 or DC3, specific design and detailing requirements according to Clauses 6 and 7 should be adopted.

NOTE For bridges with one or more piers rigidly connected to the deck (either monolithically or through fixed bearings or links) in moderate seismicity zones, choice between DC2 and DC3 generally depends on the value of seismic action index S_{δ}, the mass of the structure and economic considerations such as the construction cost and the expected repair cost in case of seismic event.

Annex B
(informative)

Added mass of entrained water for immersed piers

B.1 Use of this annex

(1) This informative annex provides complementary/supplementary guidance to 5.1.1.

NOTE National choice on the application of this informative annex is given in the national annex. If the national annex contains no information on the application of this informative annex, it can be used.

B.2 Scope and field of application

(1) This annex gives possible methods to calculate the effective mass of an immersed pier.

B.3 Effective mass of an immersed pier

(1) Unless otherwise substantiated by calculation, the total effective mass in a horizontal direction of an immersed pier should be assumed equal to the sum of:

— the actual mass of the pier (without allowance for buoyancy);

— the mass of water possibly enclosed within the pier (for hollow piers);

— the added mass m_a of externally entrained water per unit length of immersed pier.

(2) For piers of circular cross-section of radius R, m_a may be estimated using Formula (B.1).

$$m_{\mathrm{a}} = \rho \pi R^2 \tag{B.1}$$

where ρ is the water density.

(3) For piers of elliptical section (see Figure B.1) with axes $2a_x$ and $2a_y$ and horizontal seismic action at an angle θ to the x-axis of the section, m_a may be estimated using Formula (B.2).

$$m_{\mathrm{a}} = \rho \pi \left(a_{\mathrm{y}}^2 \cos^2 \theta + a_{\mathrm{x}}^2 \sin^2 \theta \right) \tag{B.2}$$

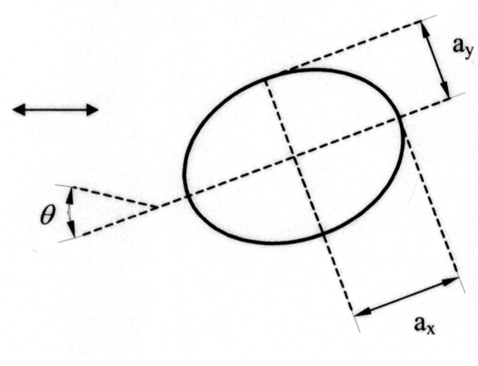


Figure B.1 — Definition of dimensions of elliptical pier section

(4) For piers of rectangular section with dimensions $2a_x$ by $2a_y$ and for earthquake action in the x-direction (see Figure B.2), m_a may be estimated using Formula (B.3).

$$m_a = k \rho \pi a_y^2 \tag{B.3}$$

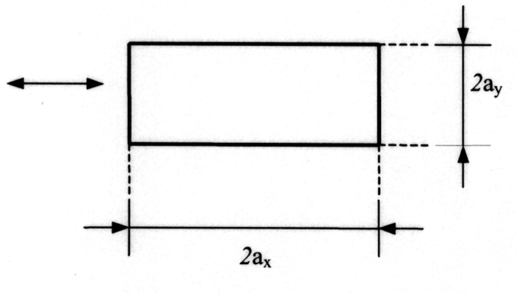


Figure B.2 — Definition of dimensions of rectangular pier section

Table B.1 — Dependence of added mass coefficient of rectangular piers on cross-sectional aspect ratio

a_y/a_x	k
0,1	2,23
0,2	1,98
0,5	1,70
1,0	1,51
2,0	1,36
5,0	1,21
10,0	1,14
∞	1,00

Annex C
(informative)

Additional information on timber bridges

C.1 Use of this annex

(1) This informative annex provides complementary/supplementary guidance to EN 1995-2 for timber bridges in seismic areas.

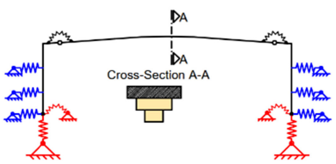
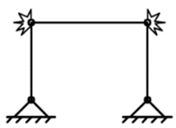
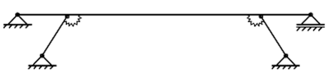
NOTE National choice on the application of this informative annex is given in the national annex. If the national annex contains no information on the application of this informative annex, it can be used.

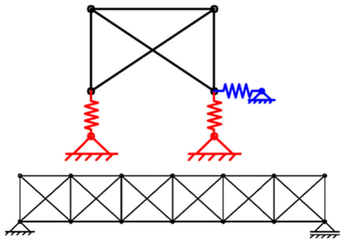
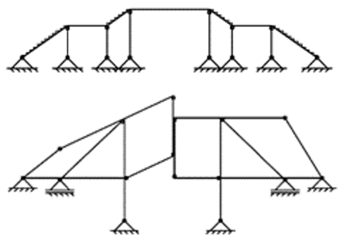
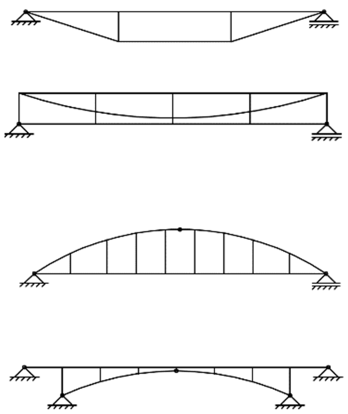
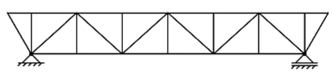
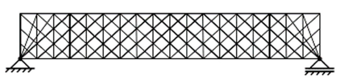
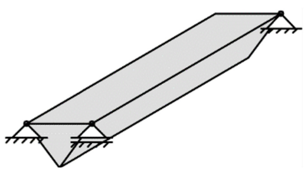
C.2 Scope and field of application

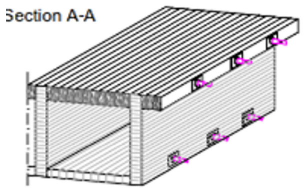
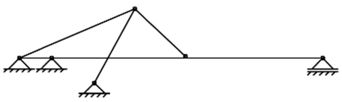
(1) This annex should be used for the design of timber bridges of the types indicated in Table C.1.

NOTE Timber bridges are manufactured mostly as glulam construction.

Table C.1 — Examples of Structural Types of Timber Bridges

Examples of structural types *	**Bridge type and used span *L***	**Similar to building type in prEN 1998-1-2[2], Table 13.1**	**Structural system and source of energy dissipation/ductility (if any)**
	d-b) Integral abutment bridges $L \leq 45$ m	d) Moment-resisting frame (MRF) structures (longitudinal direction of the bridge)	Frame with minimum two moment-transmitting joints with dowel-type fasteners / fastener plasticization
	d-b) Portal frames of bridges (part of the bridge; e.g. entrance) $L \leq 10$ m	d) Moment-resisting frame (MRF) structures (transverse direction of the bridge)	Frame with minimum two moment-transmitting joints with dowel-type fasteners / fastener plasticization
	e-b) Strutted (or truss) frame bridges with dowel-type connections $L \leq 50$ m	e) Braced frame structure with dowel-type connections (longitudinal direction of the bridge)	Multi-span girder; joints between girder and piers with dowel-type fastener connections / fastener plasticization

	e-b) Timber pier made of a truss system (part of the bridge or the abutment) $L \leq 10$ m Horizontal bracings of bridges (part of the bridge) $L \leq 90$ m	e) Braced frame structure with dowel-type connections (transverse direction of the bridge)	Timber frame with dowel-type fastener connections/fastener plasticization
	h-b) Crossings; Draw bridges $L \leq 40$ m	h) Braced frame structures with carpentry connections	Strut and tie model; Modal mass
	i-b) Tied-arch bridges $L \leq 40$ m; Suspension bridges $L \leq 50$ m; Arch bridges with or without hangers $L \leq 90$ m; Spandrel-braced bridges $L \leq 90$ m	i) Two-pin and three-pin arches, three-pin frames and dome structures	Single-span girder
	j-b) Large span truss bridges $L \leq 150$ m	Not applicable	Single-span girder
	j-b) Lattice-truss bridges (tunnel) with carpentry joints $L \leq 90$ m	Not applicable	Single-span girder
	k-b) Hollow-box-girder bridges $L \leq 80$ m	Not applicable	Single-span girder

	k-b) T-beam and box girder bridges with stress-laminated timber deck (materials see prEN 1995-2[1], Figure 3.3) $L \leq 25$ m [1] Under development.	Not applicable	Single-span or continuous girder
	k-b) Stressed ribbon bridges $L > 150$ m	j) Large-span timber truss portal frame structures	Continuous girder
	l-b) Cable-stayed bridges / Construction with pylons 50 m $\leq L \leq$ 200 m	Not applicable	Continuous girder

(2) This annex may also be used for the design of other types of timber bridges; however, validity of this application should be demonstrated via experimental and analytical support.

C.3 Basis of design

(1) For single-span girders with a span $L \leq 12$ m according to Table C.1, normally the seismic design may be omitted.

(2) Timber members of timber bridge superstructures should be designed for elastic response in the seismic design situation, with the exception of (9).

(3) Satisfaction of performance requirements in the seismic design situation should be ensured by either a) or b):

a) use of seismic isolation, according to Clause 8;

b) inelastic response in bearings and/or connections, as specified in this annex.

(4) When designed for inelastic response, bearings should conform to relevant requirements of Clause 8 (e.g. overlap length).

Uplift in bearings should be considered carefully, as it can be an issue for lightweight structures such as pedestrian timber bridges.

(5) When connections are designed for inelastic response, brittle failure should be avoided, e.g. by use of reinforcement. Reinforcement may be designed to resist a force equal to the strength perpendicular to grain in the minimum reserved area around the connector.

NOTE 1 Reinforcement is needed because, in timber bridges, fatigue problems can be caused by cyclic loading due to traffic and seismic action.

Reinforcement should be taken as equal to the strength perpendicular to grain in the area around the dowel. The dowel spacing parallel to the grain, a_1, should be increased by at least the diameter of the dowel or the screw.

NOTE 2 The area of the screw also needs to be taken into consideration in block shear failures.

(6) Unless differently stated, for timber bridges 4.3.4 and 4.3.5 should be applied, together with prEN 1998-1-2[2], 13.

(7) If overstrength of bearings, connections and timber members can be relied upon, the force-based approach may be used for design to DC1, with the values of q given in Table C.2.

NOTE The values of q given in the table for DC1 correspond to the conventional value of $q_S = 1{,}5$ related to overstrength. Bearings and connections do not necessarily possess the necessary overstrength, and for timber bridges they are the only element relied upon.

(8) If in addition to (7), inelastic response in the connections and/or the bearings is used, the force-based approach may be used for design to DC2, with the values of q given in Table C.2. Design for DC3 should not be used.

NOTE Ductility achievable through connections and bearings is limited.

(9) In low seismic action class, timber bridges with glulam and LVL members may be designed to DC2. In this case, the rules for cross laminated timber in prEN 1998-1-2[2], 13.3.2(2) and (3), may be used.

(10) Lateral stability may be ensured by the deck structure or lateral (wind) bracing and (if existing) by portal bracing and cross beams (see prEN 1995-2[2]) and not through connections resulting in uneven utility of dowels.

(11) The connection between timber deck and abutment should not be brittle. Screws should be avoided as a means to directly fastening the deck at the abutments.

NOTE Screws can be used to fasten steel parts of the deck-abutment connection. A fin from the cross beam into the deck can serve as a ductile connection.

C.4 Modelling

(1) Damping ratios for timber parts may be found in prEN 1995-2[2], 9.4.1.5.

(2) The analysis may be limited to the effects of the first two horizontal, vertical and torsional modes of vibration.

(3) Regarding the stiffness of timber fasteners or connectors, the values k_u and k_d should be taken from prEN 1995-1-1[2].

C.5 Force-based approach

(1) The behaviour factor for timber bridges defined in Table C.1 should be taken as given in Table C.2.

Table C.2 — Values of *q* for timber bridges

Type of ductile members	DC1	DC2		
	$q = q_S$	q_R	q_D	$q = q_S q_R q_D$
d-b) Integral abutment bridges, moment-resisting-frame structures including portal frames	1,5	1,1	1,3	2,2
e-b) Strutted (or truss) frame bridges with dowel-type connections, timber piers, horizontal bracings of bridges	1,5	1,0	1,3	2,0
f-b) Timber pier fixed on foundation	1,5	1,1	1,2	2,0
h-b) Crossings, draw bridges	1,5	n.a.	n.a.	n.a.
i-b) Tied-arch bridges, suspension bridges, arch bridges with or without hangers, spandrel-braced bridges	1,5	n.a.	n.a.	n.a.
j-b) Large-span truss bridges, lattice-truss bridges (tunnel) with carpentry joints	1,5	n.a.	n.a.	n.a.
k-b) Hollow-box-girder bridges, T-beam and box girder bridges with stress-laminated timber deck, stress ribbon bridges	1,5	n.a.	n.a.	n.a.
l-b) Cable-stayed bridges	1,5	n.a.	n.a.	n.a.

(2) If a timber bridge is designed for DC1, the material partial factor $\gamma_M = 1,3$ should be used, according to prEN 1998-1-2[2], 13.3(5).

NOTE In this case, all timber members and connectors conform to EN 1995-2.

(3) If a timber bridge is designed for DC2, structural detailing rules for timber in EN 1998-1-2 should be followed.

(4) For timber connections, prEN 1998-1-2[2], 13.7 (CLT or glulam), 13.10 (MRFs), 13.11 (Beam structures with dowel type connections) and 13.12 (Vertical cantilever structures made of CLT or glulam) should be used.

Annex D
(normative)

Displacement-based approach for integral abutment bridges

D.1 Use of this annex

(1) This normative annex provides complementary/supplementary guidance to 10.3.3 for the application of the displacement-based approach in seismic design of integral abutment bridges.

D.2 Scope and field of application

(1) This annex provides indications on modelling of soil in contact with the abutment walls through mutually independent inelastic springs and other aspects related to nonlinear static and response-history analysis for integral abutment bridges.

D.3 Modelling for nonlinear analysis

(1) If springs are used according to 10.3.3(3), they should describe a depth-dependent nonlinear pressure-deflection (σ $-\delta$) relation between the active σ_a and passive σ_p resistance limits (prEN 1997-1:2022, 9.5.4), in the seismic design situation, according to prEN 1998-5:2022, 10.

NOTE 1 Mutually independent springs can be used to represent vertical or horizontal reaction of the soil. The former case is of interest when they model soil reaction along the horizontal contact surface of a shallow foundation. In the context of integral abutment bridges, and more in general of retaining structures, springs represent soil reaction along vertical contact surfaces. In the latter case, if the soil is granular, its stiffness and strength vary with depth, along with vertical stress.

NOTE 2 prEN 1998-5:2022, Annex F, F.3, gives guidance for calculating active and passive earth pressures in the seismic design situation.

(2) The constitutive law of springs should be composed of at least four linear branches: one elastic, from the initial pressure σ_0 to passive resistance σ_p, one elastic from the initial pressure to the active resistance σ_a, and two horizontal branches at the active and passive resistance levels (Figure D.1a).

NOTE Continuous models, like e.g. the hyperbolic one (Figure D.1b), describe the evolution of stiffness over the entire range of deformation and are the most appropriate to capture initial stresses.

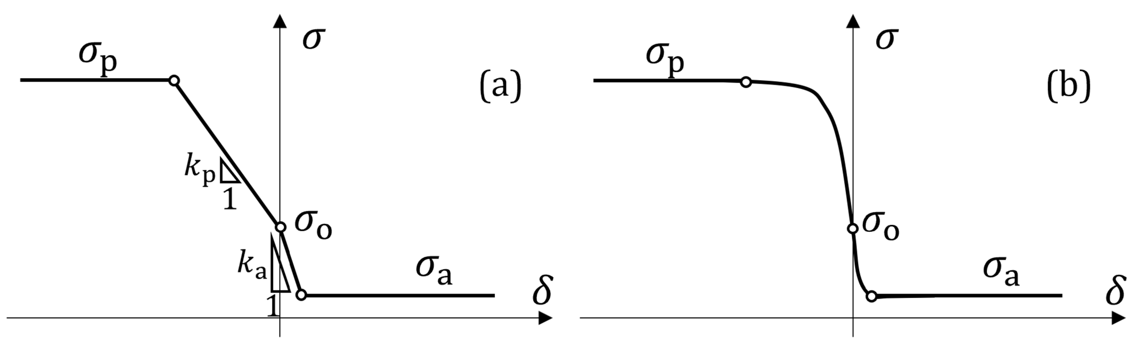


Figure D.1 — Inelastic soil spring model: (a) quadrilinear spring with different secant to active and passive stiffnesses; (b) hyperbolic model

(3) Initial pressures may be assigned values different from 'at-rest' pressures, due to preloading, according to 10.2(3).

(4) The secant stiffness for the active-side pressure may be calculated using Formula (D.1).

$$k_{\mathrm{a}}(z) = \frac{E_{\mathrm{s}}(z)A_{\mathrm{co}}(z)}{L_{\mathrm{a}}} \tag{D.1}$$

where

$E_{\mathrm{s}}(z)$ is the Young's modulus of natural soil or backfill material at depth z from the abutment top;

$A_{\mathrm{co}}(z)$ is the contact area between structure and soil or backfill material at depth z;

L_{a} is the characteristic length, measuring the volume of soil involved in the deformation behind the abutment, in active conditions, which may be calculated using Formula (D.2).

$$L_{\mathrm{a}} = \frac{2}{3}\min\left(H_{\mathrm{ab}} + D; 2H_{\mathrm{ab}}\right)\tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \tag{D.2}$$

where

H_{ab} is the abutment height;

D is the abutment foundation embedment length.

(5) The secant stiffness for the passive-side pressure may be evaluated by Formula (D.3).

$$k_{\mathrm{p}}(z) = \frac{E_{\mathrm{s}}(z)A_{\mathrm{co}}(z)}{L_{\mathrm{p}}} \tag{D.3}$$

where L_{p} is the characteristic length, measuring the volume of soil involved in the deformation behind the abutment, in passive conditions, which may be calculated using Formula (D.4).

$$L_{\mathrm{p}} = \frac{2}{3}\min\left(D; H_{\mathrm{ab}}\right)\tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \tag{D.4}$$

(6) The secant stiffness should be evaluated with soil properties compatible with its expected level of deformation. In the absence of more accurate determinations, prEN 1998-5:2022, Table 6.1, may be used for the ratio of secant to initial soil stiffness.

NOTE The ratios of G/G_0 in prEN 1998-5:2022, Table 6.1, apply also to E/E_0.

(7) As an approximation, trilinear springs with a single elastic branch of stiffness k_{p} may be used (Figure D.2a).

(8) As an approximation, non-symmetric tension-compression springs may be used if the 'at-rest' pressures are applied as a force distribution on the abutment back-walls (Figure D.2b).

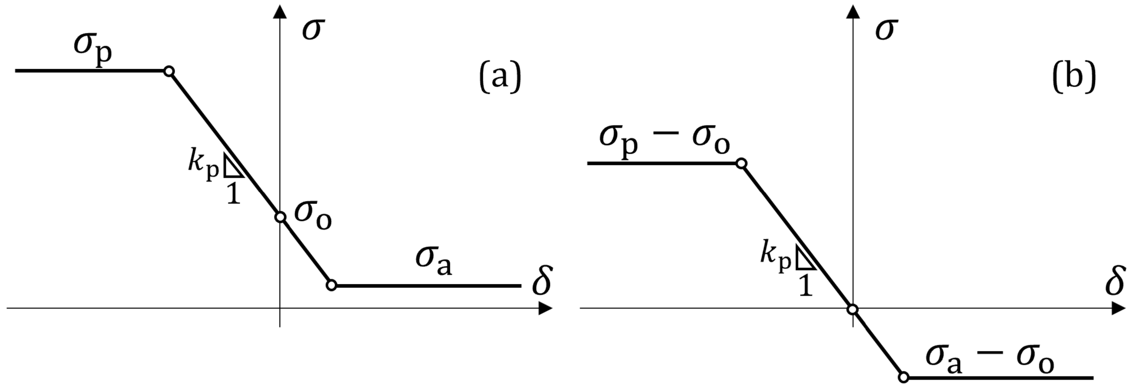


Figure D.2 — Simplified inelastic soil spring model: (a) trilinear; (b) trilinear in tension and compression

(9) For the abutments' and piers' foundations, 10.3.2(4) should be applied.

(10) If foundations are shallow, sliding should be modelled.

D.4 Nonlinear static analysis

(1) The nonlinear static analysis should be carried out by imposing a) and b) (see Figure D.3):

a) the free-field displacement profile at the soil-end of the springs on both abutments of the bridge;

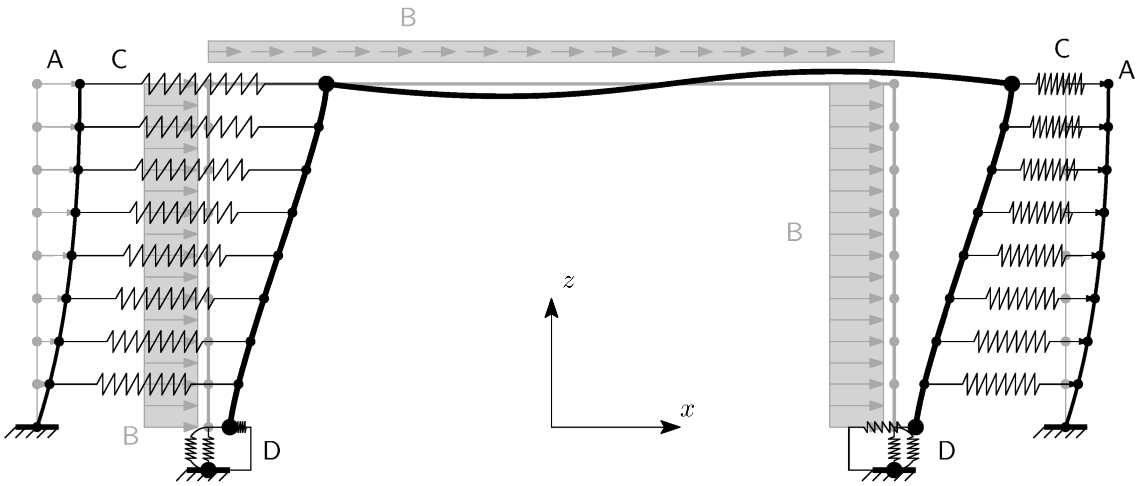
b) equivalent lateral forces on the structure according to 10.3.2(2)b).

(2) If the integral abutment bridge is above-ground and in contact with approach embankments, the free-field displacements δ_{ff} should be taken equal to as given in Formula (D.5).

$$\delta_{ff}(z) = S_{De}(T_{emb})\phi(z) \tag{D.5}$$

where

S_{De}	is the spectral displacement, at the fundamental period T_{emb} of the embankment vibrating in the bridge longitudinal direction, modelled as a (constant or variable-section) shear beam, for the limit state under consideration;
ϕ	is the corresponding embankment first-mode shape in the bridge longitudinal direction.



Key

A applied displacement profile

B inertia effects on the structural masses

C mutually independent Winkler springs

D foundation impedances

Figure D.3 — Modelling in the displacement-based approach

(3) If a more refined evaluation is not carried out, the embankment fundamental period in the bridge longitudinal direction may be evaluated by Formula (D.6), and a half-sine wave may be used as first-mode shape.

$$T_{emb} = 4H_{emb}\sqrt{\frac{\rho_{emb}}{G_{emb}}} \tag{D.6}$$

where

H_{emb} is the embankment height;

ρ_{emb} is the embankment material mass density;

G_{emb} is the embankment material shear modulus.

(4) Equivalent linear properties compatible with the embankment deformation should be used to calculate ϕ and T_{emb}.

(5) If the integral abutment bridge is embedded, the free-field displacements should be taken as a linear profile with maximum value given by Formula (D.7).

$$\delta_{\mathrm{ff}}\left(z=0\right) = \frac{PGV_e}{v_{s,H}} H_{ab} \tag{D.7}$$

where PGV_{e} is the design peak value of horizontal ground velocity, as given in prEN 1998-1-1:2022, 5.2.2.4, for the limit state under consideration.

(6) The nonlinear static analysis should be carried out at the end of the construction sequence.

(7) The nonlinear static analysis may be carried out after application of a thermal deformation of the deck given by Formula (D.8).

$$\psi_2 \epsilon_T = \psi_2 \alpha \Delta T \quad \text{(D.8)}$$

where ψ_2 is the combination factor for the quasi-permanent value of thermal action, as given in prEN 1990:2021, Table A.2.7 (NDP).

D.5 Nonlinear response-history analysis

(1) prEN 1998-1-1:2022, 5.2.3.1, 6.6 and D.3 should be applied. Spectral compatibility should be checked as for site-specific seismic soil amplification and geotechnical analyses (prEN 1998-1-1:2022, D.3(2)).

NOTE Given the dependence of deformations and internal forces on the soil response, response-history analysis of integral abutment bridges requires recorded motions.

(2) 10.3.3(4) should be applied.

(3) If mutually independent nonlinear springs are used, a) to c) should be applied:

a) for the soil-abutment interface, D.3 should be applied;

b) soil springs on the foundation members should comply with prEN 1998-5:2022, 8.3(2);

c) the seismic action should be applied by exciting a one-dimensional soil column connected to the soil-side of the above springs (Figure D.4), according to prEN 1998-5:2022, 8.3(5). As an alternative, if the soil is not included in the model, seismic action may be applied as displacement time-series at the soil-side of Winkler springs, calculated by one-dimensional soil response analysis according to prEN 1998-5:2022, 8.3(4).

NOTE A one-dimensional soil column is a discrete shear-type (multi-degree of freedom mass-spring) model of a soil deposit commonly used for one-dimensional site response analysis.

(4) The one-dimensional soil column, included in the model or used to perform a separate one-dimensional site-response analysis, should include the embankment, if present, above the natural soil deposit. The one-dimensional soil column model should allow for the transmission of seismic waves across its bottom boundary, according to prEN 1998-5:2022, 8.5(2).

(5) If the one-dimensional soil column is included in the model, a) to c) should be applied:

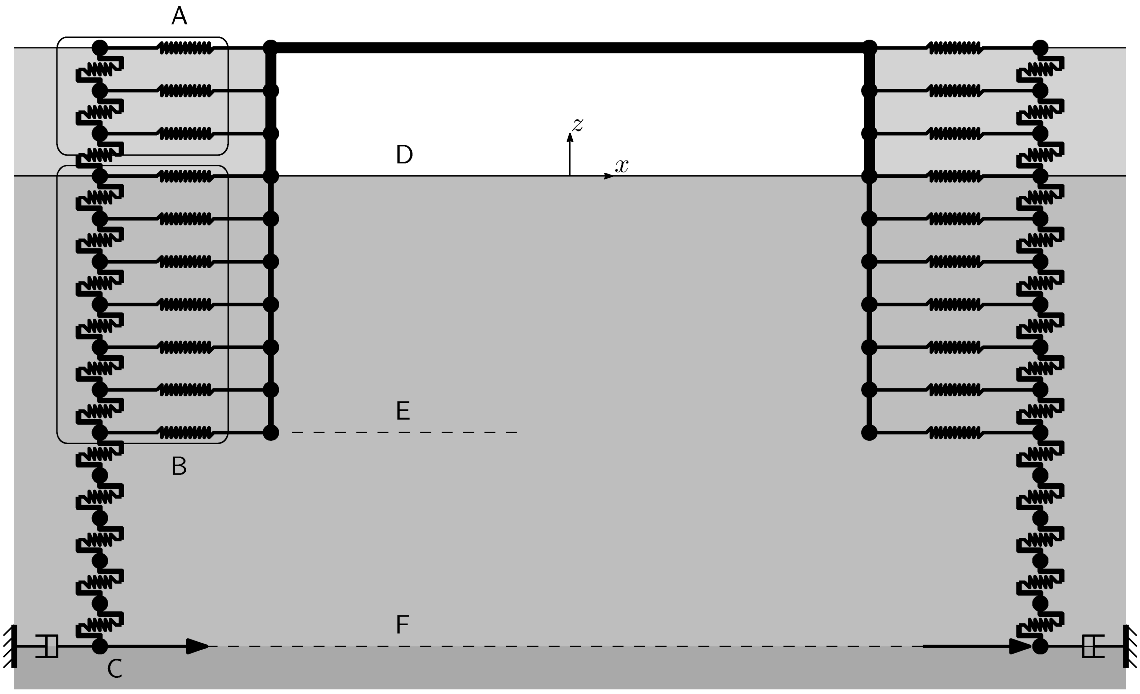
a) the area of the natural soil portion of the column should be large enough that its vibration is unaffected by the presence of the bridge structure;

b) an appropriate depth-dependent hysteretic constitutive law should be used for the soil column elements to avoid excessive amplification of the base motion at the surface;

c) the top portion corresponding to the embankment, if present, should retain its actual physical dimensions and may be modelled as a variable section shear beam.

(6) Response-history analysis should be carried out at the end of the construction sequence.

(7) Response-history analysis may be carried out after application of a thermal deformation of the deck as specified in D.4(7).



Key

A shear beam model of embankment and springs representing single-sided contact

B portion of shear beam model of foundation soil and springs representing double-sided contact

C base node of model where input motion is applied

D base of embankment

E base of foundation

F base of model

Figure D.4 — Modelling for the response-history analysis (vertical springs not shown for clarity)

Bibliography

References contained in recommendations (i.e. “should” clauses)

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative standards could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

prEN 1990:2021 *Basis of structural and geotechnical design*

prEN 1992-1-1:2021 *Eurocode 2: Design of concrete structures – Part 1-1: General rules – Rules for buildings, bridges and civil engineering structures*

prEN 1993-1-8:2021 *Eurocode 3: Design of steel structures – Part 1-8: Design of joints*

prEN 1993-1-10 *Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties (under development)*

prEN 1993-1-11 *Eurocode 3 – Design of steel structures – Part 1-11: Design of structures with tension components (under development)*

prEN 1993-2 *Eurocode 3 – Design of steel structures – Part 2: Steel bridges (under development)*

prEN 1994-2 *Eurocode 4 – Design of composite steel and concrete structures – Part 2: Bridges (under development)*

prEN 1995-1-1 *Eurocode 5 – Design of timber structures – Common rules and rules for buildings – Part 1-1: General (under development)*

prEN 1997-1:2022, *Eurocode 7: Geotechnical design – Part 1: General rules*

prEN 1998-1-2 *Eurocode 8 – Design of structures for earthquake resistance – Part 1-2: Buildings (under development)*

References contained in permissions (i.e. “may” clauses)

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

prEN 1995-2 *Eurocode 5 – Design of timber structures – Part 2: Bridges (under development)*

References contained in possibilities (i.e. “can” clauses) and notes

The following documents are cited informatively in the document, for example in “can” clauses and in notes.

EN 1992 (all parts), *Eurocode 2 – Design of concrete structures*

EN 1993 (all parts), *Eurocode 3 – Design of steel structures*

EN 1994 (all parts), *Eurocode 4 – Design of composite steel and concrete structures*

EN 1995 (all parts), *Eurocode 5* – Design of timber structures

EN 1996 (all parts), *Eurocode 6* – Design of masonry structures

EN 1999 (all parts), *Eurocode 9* – Design of aluminium structures

prEN 1998-4 *Eurocode 8 – Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines, towers, masts and chimneys (under development)*