



BSI Standards Publication

Eurocode 8 — Design of structures for earthquake resistance

Part 2: Bridges

National foreword

This British Standard is the UK implementation of EN 1998-2:2025. It supersedes BS EN 1998-2:2005+A2:2011, which will be withdrawn on 30 March 2028.

The UK participation in its preparation was entrusted to Technical Committee B/525/8, Structures in seismic regions.

A list of organizations represented on this committee can be obtained on request to its committee manager.

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National choice is allowed in this standard where explicitly stated within notes. The National Annex to this standard contains the national choices to be used for buildings and civil engineering works constructed in the UK.

The first generation of EN Eurocodes was published between 2002 and 2007, with conflicting British Standards withdrawn in 2010. This document forms part of the second generation of EN Eurocodes.

The second generation of EN Eurocodes is expected to be published between 2023 and 2026. These documents are being published as soon as they are available. This is being done to enable users to prepare for the transition from the first generation to second generation of EN Eurocodes.

UK adoptions of the first generation of EN Eurocodes will be withdrawn by BSI on 30 March 2028. Until that date, the first generation documents should be considered as the applicable standards for buildings and civil engineering works constructed in the UK unless otherwise specified by the relevant authority or in the specification for a particular project.

This standard is intended to be used with its National Annex and other referenced documents, including other second generation Eurocodes, as an interdependent suite of documents.

While the use of provisions in this standard in conjunction with first generation Eurocodes is not precluded, it should be undertaken with care and should only be done when users are satisfied that it will not result in a lower level of reliability than the minimum level set in the first generation Eurocodes and associated UK National Annexes.

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

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European foreword

This document (EN 1998-2:2025) 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 European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by September 2027, and conflicting national standards shall be withdrawn at the latest by March 2028.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN shall not be held responsible for identifying any or all such patent rights.

This document supersedes 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.

The main changes compared to the previous edition are listed below:

- Alignment with the general rules given in EN 1998-1-1 and removal of redundancies
- Development of displacement-based design for bridges
- New approach for considering the spatial variability of the seismic action
- Provisions for long bridges on non-uniform soil conditions
- Further development of the verification rules
- Specific rules for cable-stayed and extradosed bridges
- Specific rules for integral abutment bridges
- New informative annex on timber bridges

Any feedback and questions on this document should be directed to the users’ national standards body.

A complete listing of these bodies can be found on the CEN website. According to the CEN-CENELEC Internal Regulations, the national standards organisations of the following countries are bound to implement this European Standard: 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 the United Kingdom.

0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of several 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 (all parts)

EN 1998 (all parts) defines the rules for the seismic design of new buildings and other structures, as well as temporary ones, including geotechnical aspects. EN 1998 also defines the rules for the seismic assessment and retrofit of existing buildings and other structures.

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

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 EN 1990:2023, 4.3. Structures of consequence class CC4 are not fully covered by the Eurocodes but may be required by the relevant authorities to follow EN 1998, or parts of it.

Given that seismic hazard is characterized by a significant uncertainty, a null seismic risk is not achievable in practice. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquake ground motions 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 is the seismic action associated with significant uncertainty, but, in addition, it should be recognized that engineering methods are associated with assumptions that may not be verified 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.

EN 1998 is subdivided in various parts:

- EN 1998-1-1 Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action
- EN 1998-1-2 Eurocode 8 — Design of structures for earthquake resistance — Part 1-2: Buildings
- EN 1998-2 Eurocode 8 — Design of structures for earthquake resistance — Part 2: Bridges
- EN 1998-3 Eurocode 8 — Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings and bridges
- EN 1998-4 Eurocode 8 — Design of structures for earthquake resistance — Part 4: Silos, tanks, pipelines, towers, masts and chimneys
- EN 1998-5 Eurocode 8 — Design of structures for earthquake resistance — Part 5: Geotechnical aspects, foundations, retaining and underground structures

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 EN 1998-1-1:2024, Clause 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 type of 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 D are informative.

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 clauses:

4.1(2)	4.2.1(1)	4.3.5(8)	4.3.7(2)
6.3.2(2)	10.4.2.1(1)		

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

Annex A	Annex B	Annex C	Annex D
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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 EN 1990:2023, 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, EN 1998-1-1 and EN 1998-5 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 EN 1998-1-1:2024, 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.

EN 1990:2023, *Eurocode - Basis of structural and geotechnical design*

EN 1998-1-1:2024, *Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action*

EN 1998-5:2024, *Eurocode 8 — Design of structures for earthquake resistance — Part 5: Geotechnical aspects, foundations, retaining and underground structures*

ISO 80000, *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, EN 1998-1-1:2024 and the following apply.

3.1.1

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

3.1.2

longitudinal direction of the bridge

direction x defined by the line connecting the centres of the two end-sections of the deck

3.1.3

transverse direction of the bridge

plan direction y orthogonal to the longitudinal direction x

Note 1 to entry: In skew bridges, the above defined horizontal directions generally do not coincide with principal axes of inertia of the bearings, 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 link

restrainer through which part or all of the seismic action may be transmitted

Note 1 to entry: 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

Note 1 to entry: 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

Note 1 to entry: 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/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 EN 1998-1-1:2024, 3.1.9. The length of the critical region is defined in 7.2.2.1.

3.1.9**skew bridge**

bridge whose spans are not perpendicular to the axis of the supports, with an angle of skew (3.2.1.2) larger than 20°

3.1.10**curved bridge**

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

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

3.1.11**ductile member**

primary seismic member where a plastic hinge can form

3.1.12**sacrificial element**

portion of a structural member designed as a fuse, to fail and protect the remaining parts of the member

3.2 Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990:2023, 3.2, and in EN 1998-1-1:2024, 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 apply.

In addition, further symbols and abbreviations, used in connection with the seismic design situation, are defined in the present standard where they occur, for ease of use. However, the most frequently occurring symbols used in EN 1998-2 are listed and defined in 3.2.1 and additional abbreviations are given in 3.2.2.

3.2.1 Symbols

3.2.1.1 Symbols used in Clause 4

3.2.1.1.1 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

3.2.1.1.2 Lower case Greek symbols

η_k	Normalised axial force
ψ_2	Combination factor for the quasi-permanent value of the thermal action

3.2.1.2 Symbols used in Clause 5

3.2.1.2.1 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
$\mathbf{F}_i$	Vector of 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}	Smallest characteristic length value L_g among all bridge supports as given in EN 1998-1-1:2024, 5.2.3.2(3)
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 in the 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

3.2.1.2.2 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_{td}	Total design 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_p	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 $r_i = q \frac{M_{Ed,i}}{M_{Rd,i}}$
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

3.2.1.2.3 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

3.2.1.2.4 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	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 (vector) from modal analysis
$\psi_{E,i}$	Combination coefficients

3.2.1.2.5 Symbols used in Clause 6

3.2.1.2.6 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

3.2.1.2.7 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

3.2.1.2.8 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

3.2.1.3 Symbols used in Clause 7

3.2.1.3.1 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

3.2.1.3.2 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_{yd}	Design value of 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

3.2.1.3.3 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

3.2.1.3.4 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

3.2.1.4 Symbols used in Clause 8

3.2.1.4.1 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)

3.2.1.4.2 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

3.2.1.4.3 Lower case Greek symbols

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

3.2.1.5 Symbols used in Clause 10

3.2.1.5.1 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

3.2.1.5.2 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	Dimensionless soil-dependent parameter
d_{lim}	Design seismic displacement limit

3.2.1.5.3 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

3.2.1.6 Further symbols used in Annex B

3.2.1.6.1 Upper case Latin symbols

R	Radius of the immersed pier (circular cross section)
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3.2.1.6.2 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)

3.2.1.6.3 Lower case Greek symbols

θ	Angle of horizontal seismic action
ρ_w	Mass density of water

3.2.1.7 Further symbols used in Annex D

3.2.1.7.1 Upper case Latin symbols

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

3.2.1.7.2 Lower case Latin symbols

k_a	Stiffness for displacement towards the active limit pressure
k_p	Stiffness for displacement towards the passive limit pressure

3.2.1.7.3 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
ϕ	First-mode shape vector of the embankment in the longitudinal direction of the bridge
ψ_2	Combination factor for the quasi-permanent value of thermal action

3.2.2 Abbreviations

LBDP	Lower bound design properties of device
MSRS	Multiple Support Response Spectrum
SSI	Soil-structure interaction
UBDP	Upper bound design properties of device

3.3 S.I. Units

For the purposes of this document, the S.I. Units given in ISO 80000 apply.

For calculations, the following units are used when applicable:

— mass:	kg
— mass density:	kg/m ³
— forces and loads:	kN, kN/m, kN/m ²
— weight density:	N/m ³ , kN/m ³
— stresses and strengths:	Pa (= N/m ²), kPa (= kN/m ²), MPa (= MN/m ²)

- moments (bending, etc): kNm
- time: s
- displacement and length: m
- velocity: m/s
- acceleration: m/s²

4 Basis of design

4.1 Basic requirements

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

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

(2) For the application of EN 1998-1-1:2024, 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	CC3-a	CC3-b
δ	0,60	1,00	1,25	1,60

4.2 Seismic actions

4.2.1 General

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

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

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

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

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

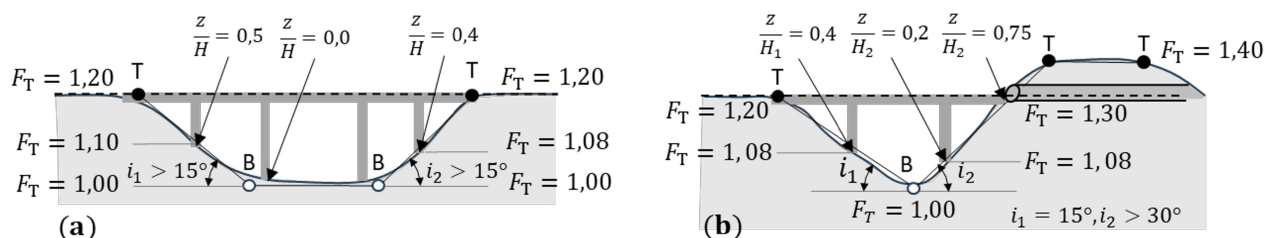
Table 4.3 (NDP) — Performance factor $\gamma_{LS,CC}$ values for bridges

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

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

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

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



Key

H slope height

z height of support with respect to slope base point B

i inclination angle of slope

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

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

- structural members in prestressed concrete decks;
- structural members in cable-stayed bridges;
- antiseismic devices;
- piers, in case of high seismic action class, if subjected to bending stresses due to vertical permanent actions of the deck, or if the bridge is located within 5 km of an active seismo-tectonic fault.

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

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

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

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

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

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

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

4.2.2 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) according to EN 1998-1-1:2024, 5.1.2(7), differ by more than 200 m/s;
- b) The total length of the bridge exceeds L_{lim} ;
- c) The maximum span length between two successive supports exceeds 60 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 60 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 EN 1998-1-1:2024, 4.1.

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

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

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

- a) resistance through structural members, possibly involving energy dissipation in clearly identified critical zones (design to ductility classes DC1, DC2 or DC3);
- b) use of antiseismic devices.

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

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

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

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

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

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

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

(9) The liquefaction potential of the foundation soil should be investigated in accordance with EN 1998-5:2024, 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 seismic members should be modelled in the structural analysis, designed and detailed for earthquake resistance in accordance with Clauses 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 (all parts), EN 1993 (all parts), EN 1994 (all parts), EN 1995 (all parts), EN 1996 (all parts) and EN 1999 (all parts), 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 Clause 10).

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

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

4.3.3 Resistance and ductility conditions – Capacity design rules

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

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

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

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

(3) 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 (3) is to ensure that the intended configuration of plastic hinges as stated in (2) will form and that brittle failure modes are avoided.

(4) 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.

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

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

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

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

NOTE 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 total design displacement in the seismic design situation with increased reliability.

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

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

4.3.5 Control of displacements – Detailing of ancillary elements

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

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

$$d_{Ed} = d_G + d_E + \psi_2 d_T \quad (4.1)$$

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 relative displacement due to seismic action;

d_T is the design relative displacement due to thermal movements;

ψ_2 is the combination factor for the quasi-permanent value of thermal action, given in EN 1990:2023, 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 displacement components d_G , d_E and $\psi_2 d_T$ in (2), they should be added to d_E in Formula (4.1).

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

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

(5) The design value of the relative displacement due to seismic action, 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 design value of the total 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 line), expected not to be damaged in the seismic design situation at SD limit state, should be designed and detailed accordingly.

NOTE Such ancillary elements can be 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, and directional portal frames) 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.

(9) Anchorage of ancillary elements to structural members should be designed according to EN 1998-1-1:2024, Annex G.