

Eurocode 8 — Design of structures for earthquake resistance —

Part 2: Bridges

ICS 91.120.25; 93.040

National foreword

This British Standard is the UK implementation of EN 1998-2:2005+A2:2011, incorporating corrigendum February 2010. It supersedes BS EN 1998-2:2005+A1:2009, which is withdrawn.

The start and finish of text introduced or altered by amendment is indicated in the text by tags. Tags indicating changes to CEN text carry the number of the CEN amendment. For example, text altered by CEN amendment A1 is indicated by A1 A1.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum February 2010 is indicated in the text by AC1 AC1.

The structural Eurocodes are divided into packages by grouping Eurocodes for each of the main materials: concrete, steel, composite concrete and steel, timber, masonry and aluminium.

The UK participation in its preparation was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/8, Structures in seismic regions.

A list of organizations represented on this subcommittee can be obtained on request to its secretary.

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN.

To enable EN 1998-2 to be used in the UK, the NDPs have been published in a National Annex, which has been made available by BSI.

There are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998-2 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions. Background information on the circumstances in which this might apply in the UK can be found in PD 6698:2009.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

Amendments/corrigenda issued since publication

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31 August 2009	Implementation of CEN amendment A1:2009
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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 European Standard was approved by CEN on 7 July 2005.

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. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists 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 Central Secretariat has the same status as the official versions.

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Foreword

This European Standard EN 1998-2, Eurocode 8: Design of structures for earthquake resistance: Bridges, has been prepared by Technical Committee CEN/TC250 «Structural Eurocodes», the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

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 May 2006, and conflicting national standards shall be withdrawn at latest by March 2010.

This document supersedes ENV 1998-2:1994.

According to the CEN-CENELEC Internal Regulations, the National Standard Organisations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

¹ Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode:	Basis of structural 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

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs).

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by

² In accordance with Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

³ In accordance with Art. 12 of the CPD the interpretative documents shall:

- a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc.;
- c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, de facto, play a similar role in the field of the ER 1 and a part of ER 2.

CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.*:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

- decisions on the use of informative annexes, and
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1998-2

The scope of this Part of EN 1998 is defined in **1.1**.

Except where otherwise specified in this Part, the seismic actions are as defined in EN 1998-1:2004, Section 3.

⁴ see Art.3.3 and Art.12 of the CPD, as well as 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Due to the peculiarities of the bridge seismic resisting systems, in comparison to those of buildings and other structures, all other sections of this Part are in general not directly related to those of EN 1998-1:2004. However several provisions of EN 1998-1:2004 are used by direct reference.

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.

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.

National annex for EN 1998-2

This standard gives alternative procedures, values and recommendations for classes, with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1998-2 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1998-2:2005 through clauses:

Reference	Item
1.1.1(8)	Informative Annexes A, B, C, D, E, F, AC1 H, JJ and K AC1
2.1(3)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement of the bridge (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
2.1(4)P	Importance classes for bridges
2.1(6)	Importance factors for bridges
2.2.2(5)	Conditions under which the seismic action may be considered as accidental action, and the requirements of 2.2.2(3) and 2.2.2 (4) may be relaxed.
2.3.5.3(1)	Expression for the length of plastic hinges
2.3.6.3(5)	Fractions of design displacements for non-critical structural elements
2.3.7(1)	Cases of low seismicity
2.3.7(1)	Simplified criteria for the design of bridges in cases of low seismicity
3.2.2.3	Definition of active fault
3.3(1)P	Length of continuous deck beyond which the spatial variability of seismic action may have to be taken into account
3.3(6)	Distance beyond which the seismic ground motions may be considered as completely uncorrelated
3.3(6)	factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports
4.1.2(4)P	ψ_{21} values for traffic loads assumed concurrent with the design seismic action
4.1.8(2)	Upper limit for the value in the left-hand-side of expression (4.4) for the seismic behaviour of a bridge to be considered irregular

5.3(4)	Value of overstrength factor γ_0
5.4(1)	Simplified methods for second order effects in linear analysis
5.6.2(2)P b	Value of additional safety factor γ_{BdI} on shear resistance
5.6.3.3(1)P b	Alternatives for determination of additional safety factor γ_{Bd} on shear resistance of ductile members outside plastic hinges
6.2.1.4(1)P	Type of confinement reinforcement
6.5.1(1)P	Simplified verification rules for bridges of limited ductile behaviour in low seismicity cases
6.6.2.3(3)	Allowable extent of damage of elastomeric bearings in bridges where the seismic action is considered as accidental action, but is not resisted entirely by elastomeric bearings
6.6.3.2(1)P	Percentage of the compressive (downward) reaction due to the permanent load that is exceeded by the total vertical reaction on a support due to the design seismic action, for holding-down devices to be required.
6.7.3(7)	Upper value of design seismic displacement to limit damage of the soil or embankment behind abutments rigidly connected to the deck.
7.4.1(1)P	Value of control period T_D for the design spectrum of bridges with seismic isolation
7.6.2(1)P	Value of amplification factor γ_{Is} on design displacement of isolator units
7.6.2(5)	Value of γ_m for elastomeric bearings
7.7.1(2)	 Values of the ratio δ for the evaluation of the lateral restoring capability
7.7.1(4)	Value of γ_{du} reflecting uncertainties in the estimation of design displacements 
J.1(2)	Values of minimum isolator temperature in the seismic design situation
J.2(1)	Values of λ -factors for commonly used isolators

Foreword to amendment A1

This document (EN 1998-2:2005/A1:2009) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This Amendment to the European Standard EN 1998-2:2005 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by September 2009, and conflicting national standards shall be withdrawn at the latest by March 2010.

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

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Foreword to amendment A2

This document (EN 1998-2:2005/A2:2011) has been prepared by Technical Committee CEN/TC “Structural Eurocodes”, the secretariat of which is held by BSI.

This Amendment to the European Standard EN 1998-2:2005 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by September 2012, and conflicting national standards shall be withdrawn at the latest by September 2012.

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According to the CEN/CENELEC Internal Regulations, the national standards organizations 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, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and the United Kingdom.

1 INTRODUCTION

1.1 Scope

1.1.1 Scope of EN 1998-2

(1) The scope of Eurocode 8 is defined in EN 1998-1:2004, **1.1.1** and the scope of this Standard is defined in **1.1.1**. Additional parts of Eurocode 8 are indicated in EN 1998-1:2004, **1.1.3**.

(2) Within the framework of the scope set forth in EN 1998-1:2004, this part of the Standard contains the particular Performance Requirements, Compliance Criteria and Application Rules applicable to the design of earthquake resistant bridges.

(3) This Part primarily covers the seismic design of 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. It is also applicable to the seismic design of cable-stayed and arched bridges, although its provisions should not be considered as fully covering these cases.

(4) Suspension bridges, timber and masonry bridges, moveable bridges and floating bridges are not included in the scope of this Part.

(5) This Part contains only those provisions that, in addition to other relevant Eurocodes or relevant Parts of EN 1998, should be observed for the design of bridges in seismic regions. In cases of low seismicity, simplified design criteria may be established (see **2.3.7(1)**).

(6) The following topics are dealt with in the text of this Part:

- Basic requirements and Compliance Criteria,
- Seismic Action,
- Analysis,
- Strength Verification,
- Detailing.

This Part also includes a special section on seismic isolation with provisions covering the application of this method of seismic protection to bridges.

(7) Annex G contains rules for the calculation of capacity design effects.

(8) Annex J contains rules regarding the variation of design properties of seismic isolator units and how such variation may be taken into account in design.

NOTE 1 Informative Annex A provides information for the probabilities of the reference seismic event and recommendations for the selection of the design seismic action during the construction phase.

NOTE 2 Informative Annex B provides information on the relationship between the displacement ductility and the curvature ductility of plastic hinges in concrete piers.

NOTE 3 Informative Annex C provides information for the estimation of the effective stiffness of reinforced concrete ductile members.

NOTE 4 Informative Annex D provides information for modelling and analysis for the spatial variability of earthquake ground motion.

NOTE 5 Informative Annex E gives information on probable material properties and plastic hinge deformation capacities for non-linear analyses.

NOTE 6 Informative Annex F gives information and guidance for the added mass of entrained water in immersed piers.

NOTE 7 Informative Annex H provides guidance and information for static non-linear analysis (pushover).

NOTE 8 Informative Annex JJ provides information on λ -factors for common isolator types.

NOTE 9 Informative Annex K contains tests requirements for validation of design properties of seismic isolator units.

1.1.2 Further parts of EN 1998

See EN 1998-1:2004.

1.2 Normative References

1.2.1 Use

(1)P The following normative documents contain provisions, which through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies (including amendments).

1.2.2 General reference standards

EN 1998-1:2004, **1.2.1** applies.

1.2.3 Reference Codes and Standards

EN 1998-1:2004, **1.2.2** applies.

1.2.4 Additional general and other reference standards for bridges

EN 1990: Annex A2 Basis of structural design: Application for bridges

EN 1991-2:2003 Actions on structures: Traffic loads on bridges

EN 1992-2:2005	Design of concrete structures. Part 2 – Bridges
EN 1993-2:2005	Design of steel structures. Part 2 – Bridges
EN 1994-2:2005	Design of composite (steel-concrete) structures. Part 2 – Bridges
EN 1998-1:2004	Design of structures for earthquake resistance. General rules, seismic actions and rules for buildings
EN 1998-5:2004	Design of structures for earthquake resistance. Foundations, retaining structures and geotechnical aspects.
EN 1337-2:2000	Structural bearings – Part 2: Sliding elements
EN 1337-3:2005	Structural bearings – Part 3: Elastomeric bearings
prEN 15129:200X	Antiseismic Devices

1.3 Assumptions

(1) In addition to the general assumptions of EN 1990:2002, **1.3** the following assumption applies.

(2)P It is assumed that no change of the structure will take place during the construction phase or during the subsequent life of the structure, unless proper justification and verification is provided. Due to the specific nature of the seismic response this applies even in the case of changes that lead to an increase of the structural resistance of members.

1.4 Distinction between principles and application rules

(1) The rules of EN 1990:2002, **1.4** apply.

1.5 Definitions

1.5.1 General

(1) For the purposes of this standard the following definitions are applicable.

1.5.2 Terms common to all Eurocodes

(1) The terms and definitions of EN 1990:2002, **1.5** apply.

1.5.3 Further terms used in EN 1998-2

capacity design

design procedure used when designing structures of ductile behaviour to ensure the hierarchy of strengths of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes

ductile members

members able to dissipate energy through the formation of plastic hinges

ductile structure

structure that under strong seismic motions can dissipate significant amounts of input energy through the formation of an intended configuration of plastic hinges or by other mechanisms

limited ductile behaviour

seismic behaviour of bridges, without significant dissipation of energy in plastic hinges under the design seismic action

positive linkage

connection implemented by seismic links

seismic isolation

provision of bridge structures with special isolating devices for the purpose of reducing the seismic response (forces and/or displacements)

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

seismic behaviour

behaviour of the bridge under the design seismic event which, depending on the characteristics of the global force-displacement relationship of the structure, can be ductile or limited ductile/essentially elastic

seismic links

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

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

design seismic displacement

displacement induced by the design seismic actions.

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.

1.6 Symbols

1.6.1 General

(1) The symbols indicated in EN 1990:2002, **1.6** apply. For the material-dependent symbols, as well as for symbols not specifically related to earthquakes, the provisions of the relevant Eurocodes apply.

(2) Further symbols, used in connection with the seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols in EN 1998-2 are listed and defined in the following subsections.

1.6.2 Further symbols used in Sections 2 and 3 of EN 1998-2

d_E	design seismic displacement (due only to the design seismic action)
d_{Ee}	seismic displacement determined from linear analysis
d_G	long term displacement due to the permanent and quasi-permanent actions
d_g	design ground displacement in accordance with EN 1998-1:2004, 3.2.2.4
d_i	ground displacement of set B at support i
d_{ri}	ground displacement at support i relative to reference support 0
d_T	displacement due to thermal movements
d_u	ultimate displacement
d_y	yield displacement
A_{Ed}	design seismic action
F_{Rd}	design value of resisting force to the earthquake action
L_g	distance beyond which the ground motion may be considered completely uncorrelated
L_i	distance of support i from reference support 0
$L_{i-1,i}$	distance between consecutive supports $i-1$ and i
R_i	reaction force at the base of pier i
S_a	site-averaged response spectrum
S_i	site-dependent response spectrum
T_{eff}	effective period of the isolation system
γ	importance factor
Δd_i	ground displacement of intermediate support i relative to adjacent supports $i-1$ and $i+1$
μ_d	displacement ductility factor
ψ_2	combination factor for the quasi-permanent value of thermal action

1.6.3 Further symbols used in Section 4 of EN 1998-2

d_a	average of the displacements in the transverse direction of all pier tops under the transverse seismic action, or under the action of a transverse load of similar distribution
d_i	displacement of the i -th nodal point
d_m	asymptotic value of the spectrum for the m -th motion for long periods, expressed in terms of displacements
e	$e_a + e_d$
e_a	accidental mass eccentricity ($= 0,03L$, or $0,03B$)
e_d	additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration ($= 0,05L$ or $0,05B$)
e_o	theoretical eccentricity
g	acceleration of gravity
h	depth of the cross-section in the direction of flexure of the plastic hinge
k_m	effect of the m -th independent motion
r_i	required local force reduction factor at ductile member i
$r_{\min}$	minimum value of r_i
$r_{\max}$	maximum value of r_i
A_{Ed}	design seismic action
A_{Ex}	seismic action in direction x
A_{Ey}	seismic action in direction y
A_{Ez}	seismic action in direction z
B	width of the deck
E	probable maximum value of an action effect
E_i	response in mode i
F	horizontal force determined in accordance with the fundamental mode method
G	total effective weight of the structure, equal to the weight of the deck plus the weight of the top half of the piers
G_i	weight concentrated at the i -th nodal point
K	stiffness of the system
L	total length of the continuous deck
L_s	distance from the plastic hinge to the point of zero moment
M	total mass
$M_{Ed,i}$	maximum value of design moment in the seismic design situation at the intended location of plastic hinge of ductile member i
$M_{Rd,i}$	design flexural resistance of the plastic hinge section of ductile member i
M_t	equivalent static moment about the vertical axis through the centre of mass of the deck

$Q_{k,1}$	characteristic value of traffic load
R_d	design value of resistance
$S_d(T)$	spectral acceleration of the design spectrum
T	period of the fundamental mode of vibration for the direction under consideration
X	horizontal longitudinal axis of the bridge
Y	horizontal transverse axis of the bridge
Z	vertical axis
α_s	shear span ratio of the pier
Δ_d	maximum difference of the displacements in the transverse direction of all pier tops under the transverse seismic action, or under the action of a transverse load of similar distribution
η_k	normalized axial force ($= N_{Ed}/(A_c f_{ck})$)
$\theta_{p,d}$	design value of plastic rotation capacity
$\theta_{p,E}$	plastic hinge rotation demand
ξ	viscous damping ratio
$\psi_{2,i}$	factor for quasi-permanent value of variable action i

1.6.4 Further symbols used in Section 5 of EN 1998-2

d_{Ed}	relative transverse displacement of the ends of the ductile member under consideration
f_{ck}	characteristic value of concrete strength
f_{ctd}	design value of tensile strength of concrete
f_{sd}	reduced stress of reinforcement, for limitation of cracking
f_{sy}	design value of yield strength of the joint reinforcement
z_b	internal lever arm of the beam end sections
z_c	internal lever arm of the plastic hinge section of the column
$A_C (V_C, M_C, N_C)$	capacity design effects
A_c	area of the concrete section
A_{Ed}	design seismic action (seismic action alone)
A_{Sd}	action in the seismic design situation
A_{sx}	area of horizontal joint reinforcement
A_{sz}	area of vertical joint reinforcement
E_d	design value of action effect of in the seismic design situation
G_k	characteristic value of permanent load
M_o	overstrength moment

M_{Ed}	design moment in the seismic design situation
M_{Rd}	design value of flexural strength of the section
N_{Ed}	axial force in the seismic design situation
N_{cG}	axial force in the column under the permanent and the quasi-permanent actions in the seismic design situation
N_{jz}	vertical axial force in a joint
Q_{1k}	characteristic value of the traffic load
Q_2	quasi-permanent value of actions of long duration
P_k	characteristic value of prestressing after all losses
R_d	design value of the resistance of the section
R_{df}	design value of the maximum friction force of sliding bearing
T_{Rc}	resultant force of the tensile reinforcement of the column
$V_{E,d}$	design value of shear force
V_{jx}	design value of horizontal shear of the joint
V_{jz}	design value of vertical shear of the joint
V_{lbc}	shear force of the beam adjacent to the tensile face of the column
γ_M	material partial factor
γ_o	overstrength factor
γ_{of}	magnification factor for friction due to ageing effects
$\gamma_{Bd}, \gamma_{Bd1}$	additional safety factor against brittle failure modes
ρ_x	ratio of horizontal reinforcement in joint
ρ_y	reinforcement ratio of closed stirrups in the transverse direction of the joint panel (orthogonal to the plane of action)
ρ_z	ratio of vertical reinforcement in joint
ψ_{21}	combination factor
ΔA_{sx}	area of horizontal joint reinforcement placed outside joint body
ΔA_{sz}	area of vertical joint reinforcement placed outside joint body

1.6.5 Further symbols used in Section 6 of EN 1998-2

a_g	design ground acceleration on type A ground (see EN 1998-1:2004, 3.2.2.2).
b	cross-sectional dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the centre line of the perimeter hoop
b_{min}	smallest dimension of the concrete core
d_{bL}	diameter of longitudinal bar
d_{eg}	effective displacement due to the spatial variation of the seismic ground displacement

d_{es}	effective seismic displacement of the support due to the deformation of the structure
d_g	design peak ground displacement as specified by EN 1998-1:2004, 3.2.2.4
f_t	tensile strength
f_y	yield strength
f_{ys}	yield strength of the longitudinal reinforcement
f_{yt}	yield strength of the tie
l_m	minimum support length securing the safe transmission of the vertical reaction
l_{ov}	minimum overlap length
s	spacing of tie legs on centres
s_L	maximum (longitudinal) spacing
s_T	spacing of between hoop legs or supplementary cross ties on centres
s_t	transverse spacing
v_g	design ground velocity
v_s	shear wave velocity in the soil at small shear strains
A_c	area of the gross concrete section
A_{cc}	cross-sectional area of the confined concrete core of the section
A_{sp}	cross-sectional area of the spiral or hoop bar
A_{sw}	total cross-sectional area of hoops or ties in the one transverse direction of confinement
A_t	cross-sectional area of one tie leg
D_i	inside diameter
D_{sp}	diameter of the spiral or hoop bar
E_d	total earth pressure acting on the abutment under seismic conditions as per EN 1998-5: 2004
F_{Rd}	design resistance
L_h	design length of plastic hinges
L_{eff}	effective length of deck
Q_d	weight of the section of the deck linked to a pier or abutment, or the least of the weights of the two deck sections on either side of an intermediate separation joint
S	soil factor specified in EN 1998-1:2004, 3.2.2.2
T_C	corner period of elastic spectrum as specified in EN 1998-1:2004, 3.2.2.2
α_g	design ground acceleration on type A ground
γ_I	importance factor
γ_s	free-field seismic shear deformation of the soil

δ	parameter depending on the ratio f_t/f_y
μ_Φ	required curvature ductility factor
$\sum A_s$	sum of the cross-sectional areas of the longitudinal bars restrained by the tie
ρ_L	ratio of the longitudinal reinforcement
ρ_w	transverse reinforcement ratio
ω_{wd}	mechanical ratio of confinement reinforcement

1.6.6 Further symbols used in Section 7 and Annexes J, JJ and K of EN 1998-2

a_g	design ground acceleration on type A ground
$a_{g,R}$	reference peak ground acceleration on type A ground reference
d	design displacement
d_b	displacement of isolator
d_{bd}	design displacement of isolator corresponding to the design displacement of the isolating system d_{cd}
d_{bi}	displacement of isolator i
$d_{bi,a}$	increased design displacement of isolator i
$d_{bi,d}$	design displacement of isolator i
d_{cd}	design displacement of the isolating system
d_{cf}	design displacement of the isolating system resulting from the fundamental mode method
$d_{d,m}$	displacement of the stiffness centre derived from the analysis
$\langle A_1 \rangle d_{G,i}$	offset displacement of isolator i $\langle A_1 \rangle$
d_{id}	displacement of the superstructure at the location of substructure and isolator i
d_m	displacement capacity of the isolating system
d_{max}	maximum displacement
$\langle A_1 \rangle d_{m,i}$	maximum total displacement of each isolator unit i $\langle A_1 \rangle$
d_n, d_p	minimum negative and positive displacement in test respectively
d_{rm}	residual displacement of the isolating system
d_y	yield displacement
e_x	eccentricity in the longitudinal bridge direction
r	radius of gyration of the deck mass about vertical axis through its centre of mass
$\text{sign}(\dot{d}_b)$	sign of the velocity vector $\dot{d}_b$
t_e	total elastomer thickness
v	velocity of motion of a viscous isolator
v_{max}	maximum velocity of motion of a viscous isolator
x_i, y_i	co-ordinates of pier i in plan

A_b	effective cross-sectional area of elastomeric bearing
E_D	dissipated energy per cycle at the design displacement of isolating system d_{cd}
E_{Di}	dissipated energy per cycle of isolator unit i , at the design displacement of isolating system d_{cd}
E_E	design seismic forces
E_{EA}	seismic internal forces derived from the analysis
F_{max}	max force corresponding to the design displacement
F_n, F_p	minimum negative and maximum positive forces of test, respectively, for units with hysteretic or frictional behaviour, or negative and positive forces of test respectively corresponding to d_n and d_p , respectively, for units with viscoelastic behaviour
F_y	yield force under monotonic loading
F_0	force at zero displacement under cyclic loading
G_b	shear modulus of elastomeric bearing
G_g	apparent conventional shear modulus of elastomeric bearing in accordance with EN 1337-3:2005

HDRB High Damping Rubber Bearing

H_i	height of pier i
K_{bi}	effective stiffness of isolator unit i
K_e	elastic stiffness of bilinear hysteretic isolator under monotonic loading
K_L	stiffness of lead core of lead-rubber bearing
K_p	post elastic stiffness of bilinear hysteretic isolator
K_{eff}	effective stiffness of the isolation system in the principal horizontal direction under consideration, at a displacement equal to the design displacement d_{cd}
$K_{eff,i}$	composite stiffness of isolator units and the corresponding pier i
K_{fi}	rotation stiffness of foundation of pier i
K_R	stiffness of rubber of lead-rubber bearing
K_{ri}	rotation stiffness of foundation of pier i
K_{si}	displacement stiffness of shaft of pier i
K_{ti}	translation stiffness of foundation of pier i
K_{xi}, K_{yi}	effective composite stiffness of isolator unit and pier i

LRB Lead Rubber Bearing

M_d	mass of the superstructure
N_{Sd}	axial force through the isolator
PTFE	polytetrafluorethylene
Q_G	permanent axial load of isolator
R_b	radius of spherical sliding surface
S	soil factor of elastic spectrum in accordance with EN 1998-1:2004, 3.2.2.2

T_C, T_D	corner periods of the elastic spectrum in accordance with 7.4.1(1)P and EN 1998-1:2004, 3.2.2.2
T_{eff}	effective period of the isolating system
$T_{\text{min,b}}$	minimum bearing temperature for seismic design
V_d	maximum shear force transferred through the isolation interface
V_f	maximum shear force estimated through the fundamental mode method
UBDP	Upper bound design properties of isolators
LBDP	Lower bound design properties of isolators
α_b	exponent of velocity of viscous damper
γ_I	importance factor of the bridge
ΔF_{Ed}	additional vertical load due to seismic overturning effects
ΔF_m	force increase between displacements $d_m/2$ and d_m
μ_d	dynamic friction coefficient
ξ	equivalent viscous damping ratio
ξ_b	contribution of isolators to effective damping
ξ_{eff}	effective damping of the isolation system
ψ_{fi}	combination factor

2 BASIC REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Design seismic action

(1)P The design philosophy of this Standard is to achieve with appropriate reliability the non-collapse requirement of **2.2.2** and of EN 1998-1:2004, **2.1(1)P**, for the design seismic action (A_{Ed}).

(2)P Unless otherwise specified in this part, the elastic spectrum of the design seismic action in accordance with EN 1998-1:2004, **3.2.2.2**, **3.2.2.3** and **3.2.2.4** applies. For application of the equivalent linear method of **4.1.6** (using the behaviour factor q) the spectrum shall be the design spectrum in accordance with EN 1998-1:2004, **3.2.2.5**.

(3)P The design seismic action, A_{Ed} , is expressed in terms of: (a) the reference seismic action, A_{Ek} , associated with a reference probability of exceedance, P_{NCR} , in 50 years or a reference return period, T_{NCR} , (see EN 1998-1:2004, **2.1(1)P** and **3.2.1(3)**) and (b) the importance factor γ_I (see EN 1990: 2002 and EN 1998-1:2004, **2.1(2)P**, **2.1(3)P** and **(4)**) to take into account reliability differentiation:

$$A_{Ed} = \gamma_I A_{Ek} \quad (2.1)$$

NOTE 1 The value to be ascribed to the reference return period, T_{NCR} , associated with the reference seismic action for use in a country, may be found in its National Annex. The recommended value is: $T_{NCR} = 475$ years.

NOTE 2 Informative Annex A gives information on the reference seismic action and on the selection of the design seismic action during the construction phase.

(4)P Bridges shall be classified in importance classes, depending on the consequences of their failure for human life, on their importance for maintaining communications, especially in the immediate post-earthquake period, and on the economic consequences of collapse.

NOTE The definitions of the importance classes for bridges in a country may be found in its National Annex. The recommended classification is in three importance classes, as follows:

In general road and railway bridges are considered to belong to importance class II (average importance), with the exceptions noted below.

Importance class III comprises bridges of critical importance for maintaining communications, especially in the immediate post-earthquake period, bridges the failure of which is associated with a large number of probable fatalities and major bridges where a design life greater than normal is required.

A bridge may be classified to importance class I (less than average importance) when both of the following conditions are met.

- the bridge is not critical for communications, and
- the adoption of either the reference probability of exceedance, P_{NCR} , in 50 years for the design seismic action, or of the standard bridge design life of 50 years is not economically justified.

Importance classes I, II and III correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, **B3.1**.

(5)P The importance classes are characterised by different importance factors γ_i as described in **2.1(3)P** and in EN 1998-1:2004, **2.1(3)P**.

(6) The importance factor $\gamma_i = 1,0$ is associated with a seismic action having the reference return period indicated in **2.1(3)P** and in EN 1998-1:2004, **3.2.1(3)**.

NOTE The values to be ascribed to γ_i for use in a country may be found in its National Annex. The values of γ_i may be different for the various seismic zones of the country, depending on the seismic hazard conditions and on public safety considerations (see NOTE to EN 1998-1:2004, **2.1(4)**). The recommended values of γ_i for importance classes I, and III are equal to 0,85, and 1,3, respectively.

2.2 Basic requirements

2.2.1 General

(1)P The design shall aim at fulfilling the following two basic requirements.

2.2.2 No-collapse (ultimate limit state)

(1)P After occurrence of the design seismic action, the bridge shall retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur.

(2) Flexural yielding of specific sections (i.e. the formation of plastic hinges) is allowed to occur in the piers. When no seismic isolation is provided, such flexural yielding is in general necessary in regions of high seismicity, in order to reduce the design seismic action to a level corresponding to a reasonable increase of the additional construction cost, compared to a bridge not designed for earthquake resistance.

(3) The bridge deck should in general be designed to avoid damage, other than locally to secondary components such as expansion joints, continuity slabs (see **2.3.2.2(4)**) or parapets.

(4) When the design seismic action has a substantial probability of exceedance within the design life of the bridge, the design should aim at a damage tolerant structure. Parts of the bridge susceptible to damage by their contribution to energy dissipation under the design seismic action should be designed to enable the bridge to be used by emergency traffic, following the design seismic action, and to be easily repairable.

(5) When the design seismic action has a low probability of being exceeded within the design life of the bridge, the seismic action may be considered as an accidental action, in accordance with EN 1990:2002, **1.5.3.5** and **4.1.1(2)**. In such a case the requirements of **(3)** and **(4)** may be relaxed.

NOTE The National Annex may specify the conditions under which **(5)** will be applied, as well as the extent of the relevant relaxations of **(3)** and **(4)**. It is recommended that **(3)** and **(4)** are applicable when the reference return period T_{NCR} is approximately equal to 475 years.

2.2.3 Minimisation of damage (serviceability limit state)

(1)P A seismic action with a high probability of occurrence may cause only minor damage to secondary components and to those parts of the bridge intended to contribute to energy dissipation. All other parts of the bridge should remain undamaged.

2.3 Compliance criteria

2.3.1 General

(1)P To conform to the basic requirements set forth in **2.2**, the design shall comply with the criteria outlined in the following Clauses. In general the criteria, while aiming explicitly at satisfying the no-collapse requirement (**2.2.2**), implicitly cover the damage minimisation requirement (**2.2.3**) as well.

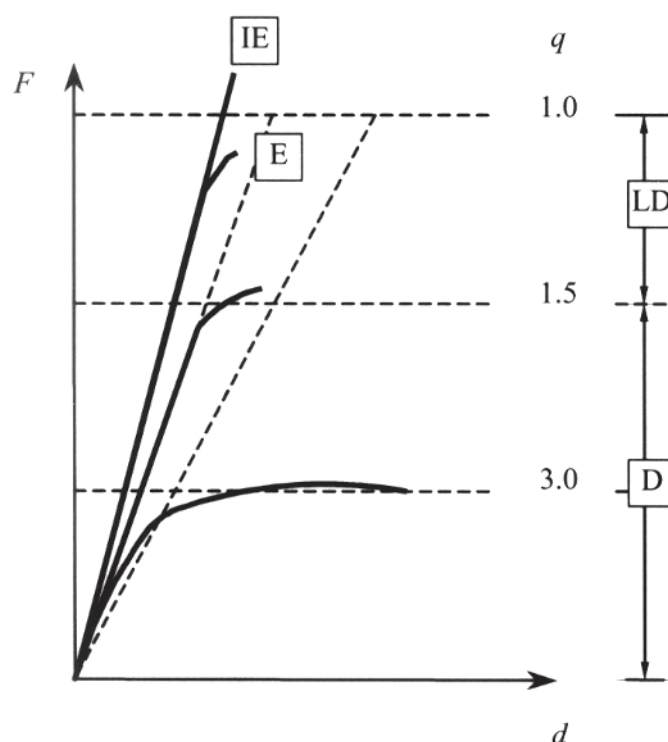
(2) Compliance with the criteria set forth in this standard is deemed to satisfy all basic requirements of **2.2**.

(3)P The compliance criteria depend on the behaviour which is intended for the bridge under the design seismic action. This behaviour may be selected in accordance with **2.3.2**.

2.3.2 Intended seismic behaviour

2.3.2.1 General

(1)P The bridge shall be designed so that its behaviour under the design seismic action is either ductile, or limited ductile/essentially elastic, depending on the seismicity of the site, on whether seismic isolation is adopted for its design, or any other constraints which may prevail. This behaviour (ductile or limited ductile) is characterised by the global force-displacement relationship of the structure, shown schematically in Figure 2.1 (see also Table 4.1).



Key

q – Behaviour factor
IE - Ideal elastic
E - Essentially elastic
LD - Limited ductile
D - Ductile

Figure 2.1: Seismic behaviour

2.3.2.2 Ductile behaviour

(1) In regions of moderate to high seismicity it is usually preferable, both for economic and safety reasons, to design a bridge for ductile behaviour, i.e. to provide it with reliable means to dissipate a significant amount of the input energy under severe earthquakes. This is accomplished by providing for the formation of an intended configuration of flexural plastic hinges or by using isolating devices in accordance with Section 7. The part of this sub-clause that follows refers to ductile behaviour achieved by flexural plastic hinges.

(2)P Bridges of ductile behaviour shall be designed so that a dependably stable partial or full mechanism can develop in the structure through the formation of flexural plastic hinges. These hinges normally form in the piers and act as the primary energy dissipating components.

(3) As far as is reasonably practicable, the location of plastic hinges should be selected at points accessible for inspection and repair.

(4)P The bridge deck shall remain within the elastic range. However, formation of plastic hinges (in bending about the transverse axis) is allowed in flexible ductile concrete slabs providing top slab continuity between adjacent simply-supported precast concrete girder spans.

(5)P Plastic hinges shall not be formed in reinforced concrete sections where the normalised axial force η_k defined in 5.3(4) exceeds 0,6.

(6)P This standard does not contain rules for provision of ductility in prestressed or post-tensioned members. Consequently such members should be protected from formation of plastic hinges under the design seismic action.

(7) Flexural plastic hinges need not necessarily form in all piers. However the optimum post-elastic seismic behaviour of a bridge is achieved if plastic hinges develop approximately simultaneously in as many piers as possible.

(8) The capability of the structure to form flexural hinges is necessary, in order to ensure energy dissipation and consequently ductile behaviour (see 4.1.6(2)).

NOTE The deformation of bridges supported exclusively by simple low damping elastomeric bearings is predominantly elastic and does not lead in general to ductile behaviour (see 4.1.6(11)P).

(9) The global force-displacement relationship should exhibit a significant force plateau at yield and should ensure hysteretic energy dissipation over at least five inelastic deformation cycles (see Figures 2.1, 2.2 and 2.3).

NOTE Elastomeric bearings used over some supports in combination with monolithic support on other piers, may cause the resisting force to increase with increasing displacements, after plastic hinges have formed in the other supporting members. However, the rate of increase of the resisting force should be appreciably reduced after the formation of plastic hinges.

(10) Supporting members (piers or abutments) connected to the deck through sliding or flexible mountings (sliding bearings or flexible elastomeric bearings) should, in general, remain within the elastic range.

2.3.2.3 Limited ductile behaviour

(1) In structures with limited ductile behaviour, a yielding region with significant reduction in secant stiffness need not appear under the design seismic action. In terms of force-displacement characteristics, the formation of a force plateau is not required, while deviation from the ideal elastic behaviour provides some hysteretic energy dissipation. Such behaviour corresponds to a value of the behaviour factor $q \leq 1,5$ and shall be referred to, in this Standard, as "limited ductile".

NOTE Values of q in the range $1 \leq q \leq 1,5$ are mainly attributed to the inherent margin between design and probable strength in the seismic design situation.

(2) For bridges where the seismic response may be dominated by higher mode effects (e.g cable-stayed bridges), or where the detailing of plastic hinges for ductility may not be reliable (e.g. due to a high axial force or a low shear-span ratio), a behaviour factor of $q = 1$ is recommended, corresponding to elastic behaviour.

2.3.3 Resistance verifications

(1)P In bridges designed for ductile behaviour the regions of plastic hinges shall be verified to have adequate flexural strength to resist the design seismic action effects as specified in 5.5. The shear resistance of the plastic hinges, as well as both the shear and flexural resistances of all other regions, shall be designed to resist the "capacity design effects" specified in 2.3.4 (see also 5.3).

(2) In bridges designed for limited ductile behaviour, all sections should be verified to have adequate strength to resist the design seismic action effects of 5.5 (see 5.6.2).

2.3.4 Capacity design

(1)P For bridges of ductile behaviour, capacity design shall be used to ensure that an appropriate hierarchy of resistance exists within the various structural components. This is to ensure that the intended configuration of plastic hinges will form and that brittle failure modes are avoided.

(2)P Fulfilment of (1)P shall be achieved by designing all members intended to remain elastic against all brittle modes of failure, using "capacity design effects". Such effects result from equilibrium conditions at the intended plastic mechanism, when all flexural hinges have developed an upper fractile of their flexural resistance (overstrength), as specified in 5.3.

(3) For bridges of limited ductile behaviour the application of the capacity design procedure is not required.

2.3.5 Provisions for ductility

2.3.5.1 General requirement

(1)P The intended plastic hinges shall be provided with adequate ductility, to ensure the required overall global ductility of the structure.

NOTE The definitions of global and local ductilities, given in 2.3.5.2 and 2.3.5.3, are intended to provide the theoretical basis of ductile behaviour. In general they are not required for practical verification of ductility, which is effected in accordance with 2.3.5.4.

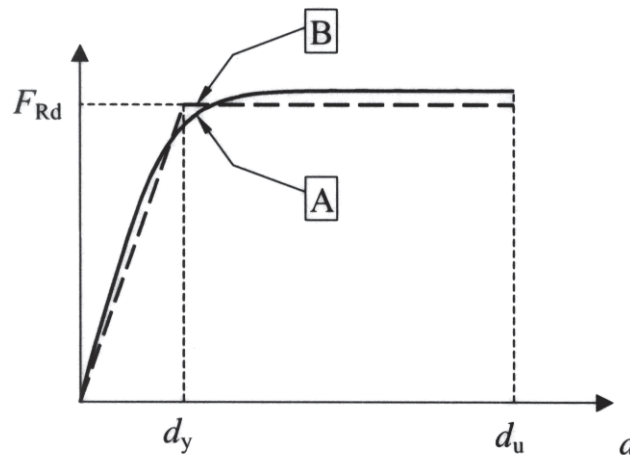
2.3.5.2 Global ductility

(1) Referring to an equivalent one-degree-of-freedom system with an idealised elastic-perfectly plastic force-displacement relationship, as shown in Figure 2.2, the design value of the ductility factor of the structure (available displacement ductility factor) is defined as the ratio of the ultimate limit state displacement (d_u) to the yield displacement (d_y), both measured at the centre of mass: i.e. $\mu_d = d_u/d_y$.

(2) When an equivalent linear analysis is performed, the yield force of the global elastic-perfectly plastic force-displacement is assumed equal to the design value of the resisting force, F_{Rd} . The yield displacement defining the elastic branch is selected so as to best approximate the design force-displacement curve (for monotonic loading).

(3) The ultimate displacement d_u is defined as the maximum displacement satisfying the following condition. The structure should be capable of sustaining at least 5 full cycles of deformation to the ultimate displacement:

- without initiation of failure of the confining reinforcement for reinforced concrete sections, or local buckling effects for steel sections; and
- without a drop of the resisting force for steel ductile members or without a drop exceeding 20% of the ultimate resisting force for reinforced concrete ductile members (see Figure 2.3).

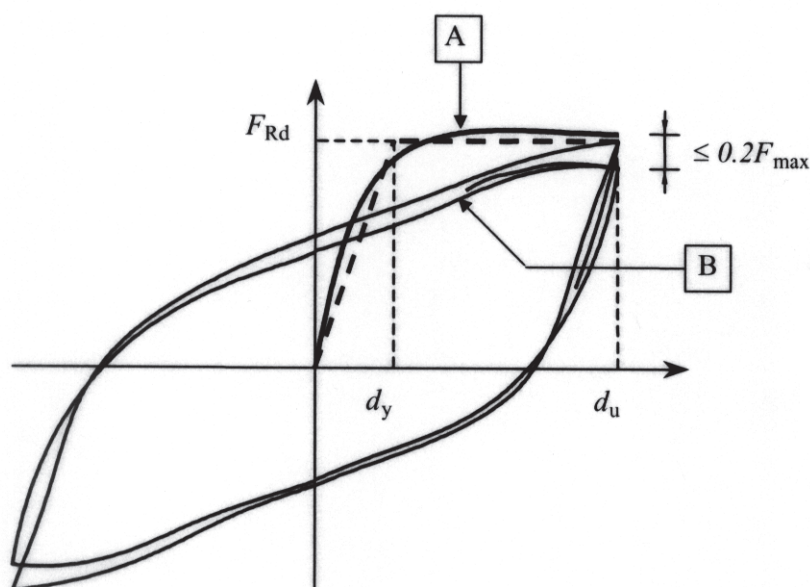


Key

A - Design

B - Elastoplastic

Figure 2.2: Global force-displacement diagram (Monotonic loading)



Key

A - Monotonic loading

B - 5th cycle

Figure 2.3: Force-displacement cycles (Reinforced concrete)

2.3.5.3 Local ductility at the plastic hinges

(1) The global ductility of the structure depends on the available local ductility at the plastic hinges (see Figure 2.4). This can be expressed in terms of the curvature ductility factor of the cross-section:

$$\mu_{\Phi} = \Phi_u / \Phi_y \quad (2.2)$$

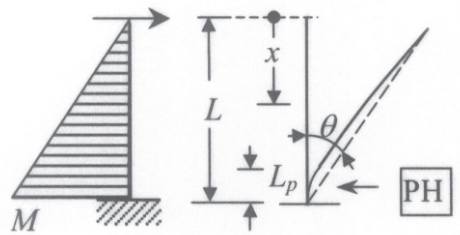
or, in terms of the chord rotation ductility factor at the end where the plastic hinge forms, that depends on the plastic rotation capacity, $\theta_{p,u} = \theta_u - \theta_y$, of the plastic hinge:

$$\mu_{\theta} = \theta_u / \theta_y = 1 + (\theta_u - \theta_y) / \theta_y = 1 + \theta_{p,u} / \theta_y \quad (2.3)$$

The chord rotation is measured over the length L , between the end section of the plastic hinge and the section of zero moment, as shown in Figure 2.4.

NOTE 1 For concrete members the relationship between θ_p , Φ_u , Φ_y , L and L_p is given by equation (E16b) in E.3.2 of Informative Annex E.

NOTE 2 The length of plastic hinges L_p for concrete members may be specified in the National Annex, as a function of the geometry and other characteristics of the member. The recommended expression is that given in Annex E.



Key
PH - Plastic hinge

Figure 2.4: Chord rotation $\theta = \frac{1}{L} \int_0^L \phi x dx$

(2) In the above expressions the ultimate deformations should conform to the definitions in **2.3.5.2(3)**.

NOTE The relationship between curvature ductility of a plastic hinge and the global displacement ductility factor for a simple case is given in **Annex B**. That relationship is not intended for ductility verification.

2.3.5.4 Ductility verification

(1)P Conformance to the Specific Rules specified in **Section 6** is deemed to ensure the availability of adequate local and global ductility.

(2)P When non-linear static or dynamic analysis is performed, chord rotation demands shall be checked against available rotation capacities of the plastic hinges (see **4.2.4.4**).

(3) For bridges of limited ductile behaviour the provisions of **6.5** should be applied.

2.3.6 Connections - Control of displacements - Detailing

2.3.6.1 Effective stiffness - Design seismic displacement

(1)P When equivalent linear analysis methods are used, the stiffness of each member shall be chosen corresponding to its secant stiffness under the maximum calculated stresses under the design seismic action. For members containing plastic hinges this corresponds to the secant stiffness at the theoretical yield point (See Figure 2.5).

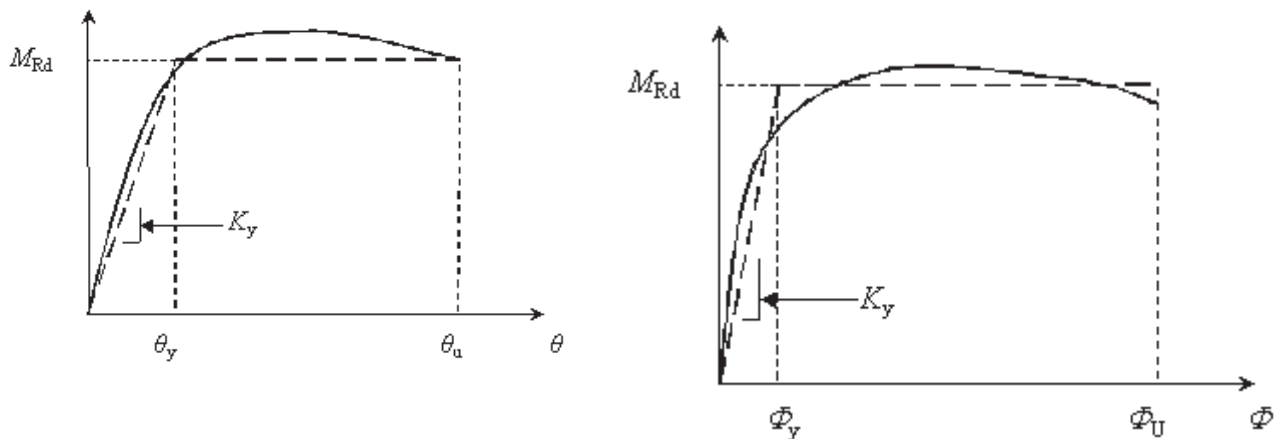


Figure 2.5: Moment - deformation diagrams at plastic hinges

Left: Moment-rotation relationship of plastic hinge for structural steel;

Right: Moment-curvature relationship of cross-section for reinforced concrete.

(2) For reinforced concrete members in bridges designed for ductile behaviour, and unless a more accurate method is used for its estimation, the effective flexural stiffness to be used in linear analysis (static or dynamic) for the design seismic action may be estimated as follows.

- For reinforced concrete piers, a value calculated on the basis of the secant stiffness at the theoretical yield point.
- For prestressed or reinforced concrete decks, the stiffness of the uncracked gross concrete sections.

NOTE **Annex C** gives guidance for the estimation of the effective stiffness of reinforced concrete members.

(3) In bridges designed for limited ductile behaviour, either the rules of (2) may be applied or the flexural stiffness of the uncracked gross concrete sections may be used for the entire structure.

(4) For both ductile and limited ductile bridges, the significant reduction of the torsional stiffness of concrete decks, in relation to the torsional stiffness of the uncracked deck, should be accounted for. Unless a more accurate calculation is made, the following fractions of the torsional stiffness of the uncracked gross section may be used:

- for open sections or slabs, the torsional stiffness may be ignored;
- for prestressed box sections, 50% of the uncracked gross section stiffness;
- for reinforced concrete box sections, 30% of the uncracked gross section stiffness.

(5) For both ductile and limited ductile bridges, displacements obtained from an analysis in accordance with (2) and (3) should be multiplied by the ratio of (a) the flexural stiffness of the member used in the analysis to (b) the value of flexural stiffness that corresponds to the level of stresses resulting from the analysis.

NOTE It is noted that in the case of equivalent linear analysis (see **4.1.6(1)P**) an overestimation of the effective stiffness leads to results which are on the safe side regarding the seismic action effects. In such a case, only the displacements need be corrected after the analysis, on the basis of the flexural stiffness that corresponds to the resulting level of moments. On the other hand, if the effective stiffness initially assumed is significantly lower than that corresponding to the stresses from the analysis, the analysis should be repeated using a better approximation of the effective stiffness.

(6)P If linear seismic analysis based on the design spectrum in accordance with EN 1998-1:2004, **3.2.2.5** is used, the design seismic displacements, d_E , shall be derived from the displacements, d_{Ee} , determined from such an analysis as follows:

$$d_E = \pm \eta \mu_d d_{Ee} \quad (2.4)$$

where

η is the damping correction factor specified in EN 1998-1:2004, **3.2.2.2(3)** determined with the ξ values specified for damping in **4.1.3(1)**.

(7) When the displacements d_{Ee} are derived from a linear elastic analysis based on the elastic spectrum in accordance with EN 1998-1:2004, **3.2.2.2** ($q = 1.0$), the design displacement, d_E , shall be taken as equal to d_{Ee} .

(8)P The displacement ductility factor shall be assumed as follows:

- when the fundamental period T in the considered horizontal direction is $T \geq T_o = 1,25T_C$, where T_C is the corner period defined in accordance with EN 1998-1:2004, **3.2.2.2**, then

$$\mu_d = q \quad (2.5)$$

- if $T < T_o$, then

$$\mu_d = (q - 1) \frac{T_o}{T} + 1 \leq 5q - 4 \quad (2.6)$$

where q is the value of the behaviour factor assumed in the analysis that results in the value of d_{Ee} .

NOTE Expression (2.6) provides a smooth transition between the “equal displacement” rule that is applicable for $T \geq T_o$, and the short period range (not typical to bridges) where the assumption of a low q -value is expedient. For very small periods ($T < 0,033$ sec), $q = 1$ should be assumed (see also **4.1.6(9)**), giving: $\mu_d = 1$.

(9)P When non-linear time-history analysis is used, the deformation characteristics of the yielding members shall approximate their actual post-elastic behaviour, both as far as the loading and unloading branches of the hysteresis loops are concerned, as well as potential degradation effects (see **4.2.4.4**).

2.3.6.2 Connections

(1)P Connections between supporting and supported members shall be designed in order to ensure structural integrity and avoid unseating under extreme seismic displacements.

(2) Unless otherwise specified in this Part, bearings, links and holding-down devices used for securing structural integrity, should be designed using capacity design effects (see 5.3, 6.6.2.1, 6.6.3.1 and 6.6.3.2).

(3) In new bridges appropriate overlap lengths should be provided between supporting and supported members at moveable connections, in order to avoid unseating (see 6.6.4).

(4) In retrofitting existing bridges as an alternative to the provision of overlap length, positive linkage between supporting and supported members may be used (see 6.6.1(3)P and 6.6.3.1(1)).

2.3.6.3 Control of displacements - Detailing

(1)P In addition to ensuring the required overall ductility, structural and non-structural detailing of the bridge and its components shall be provided to accommodate the displacements in the seismic design situation.

(2)P Clearances shall be provided for protection of critical or major structural members. Such clearances shall accommodate the total design value of the displacement in the seismic design situation, d_{Ed} , determined as follows:

$$d_{Ed} = d_E + d_G + \psi_2 d_T \quad (2.7)$$

where the following displacements shall be combined with the most unfavourable sign:

- d_E is the design seismic displacement in accordance with 2.3.6.1;
- d_G is the long term displacement due to the permanent and quasi-permanent actions (e.g. post-tensioning, shrinkage and creep for concrete decks);
- d_T is the displacement due to thermal movements;
- ψ_2 is the combination factor for the quasi-permanent value of thermal action, in accordance with EN 1990:2002, Tables A2.1, A2.2 or A2.3.

Second order effects shall be taken into account in the calculation of the total design value of the displacement in the seismic design situation, when such effects are significant.

(3) The relative design seismic displacement, d_E , between two independent sections of a bridge may be estimated as the square root of the sum of squares of the values of the design seismic displacement calculated for each section in accordance with 2.3.6.1.

(4)P Large shock forces, caused by unpredictable impact between major structural members, shall be prevented by means of ductile/resilient members or special energy absorbing devices (buffers). Such members shall possess a slack at least equal to the total design value of the displacement in the seismic design situation, d_{Ed} .

(5) The detailing of non-critical structural components (e.g. deck movement joints and abutment back-walls), expected to be damaged due to the design seismic action, should cater for a predictable mode of damage, and provide for the possibility of permanent repair. Clearances should accommodate appropriate fractions of the design seismic displacement and of the thermal movement, p_E and p_T , respectively, after allowing for any long term creep and shrinkage effects, so that damage under frequent earthquakes is avoided. The appropriate values of such fractions may be chosen, based on a judgement of the cost-effectiveness of the measures taken to prevent damage.

NOTE 1 The value ascribed to p_E and p_T for use in a country in the absence of an explicit optimisation may be found in its National Annex. The recommended values are as follows: $p_E = 0,4$ (for the design seismic displacement); $p_T = 0,5$ (for the thermal movement).

NOTE 2 At joints of railway bridges, transverse differential displacement may have to be either avoided or limited to values appropriate for preventing derailment.

2.3.7 Simplified criteria

(1) In cases of low seismicity, simplified design criteria may be established.

NOTE 1: The selection of the categories of bridge, ground type and seismic zone in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended that cases of low seismicity (and by consequence those of moderate to high seismicity) should be defined as recommended in the Note in EN 1998-1:2004, **3.2.1(4)**.

NOTE 2: Classification of bridges and simplified criteria for the seismic design pertaining to individual bridge classes in cases of low seismicity may be established by the National Annex. It is recommended that these simplified criteria are based on a limited ductile/essentially elastic seismic behaviour of the bridge, for which no special ductility requirements are necessary.

2.4 Conceptual design

(1) Consideration of the implications of the seismic action at the conceptual stage of the design of bridges is important, even in cases of low to moderate seismicity.

(2) In cases of low seismicity the type of intended seismic behaviour of the bridge (see **2.3.2**) should be decided. If a limited ductile (or essentially elastic) behaviour is selected, simplified criteria, in accordance with **2.3.7** may be applied.

(3) In cases of moderate or high seismicity, the selection of ductile behaviour is generally expedient. Its implementation, either by providing for the formation of a dependable plastic mechanism or by using seismic isolation and energy dissipation devices, should be decided. When a ductile behaviour is selected, **(4)** to **(8)** should be observed.

(4) The number of supporting members (piers and abutments) that will be used to resist the seismic forces in the longitudinal and transverse directions should be decided. In general bridges with continuous deck behave better under seismic conditions than those with many movement joints. The optimum post-elastic seismic behaviour is achieved if plastic hinges develop approximately simultaneously in as many piers as possible. However, the number of the piers that resist the seismic action may have to be less than the total number of piers, by using sliding or flexible mountings between the

deck and some piers in the longitudinal direction, to reduce the stresses arising from imposed deck deformations due to thermal actions, shrinkage and other non-seismic actions.

(5) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the magnitude of lateral forces induced by the design seismic action but increases the movement at the joints and moveable bearings and may lead to high second order effects.

(6) In the case of bridges with a continuous deck and with transverse stiffness of the abutments and of the adjacent piers which is very high compared to that of the other piers (as may occur in steep-sided valleys), it may be preferable to use transversally sliding or elastomeric bearings over the short piers or the abutments to avoid unfavourable distribution of the transverse seismic action among the piers and the abutments such as that exemplified in Figure 2.6.

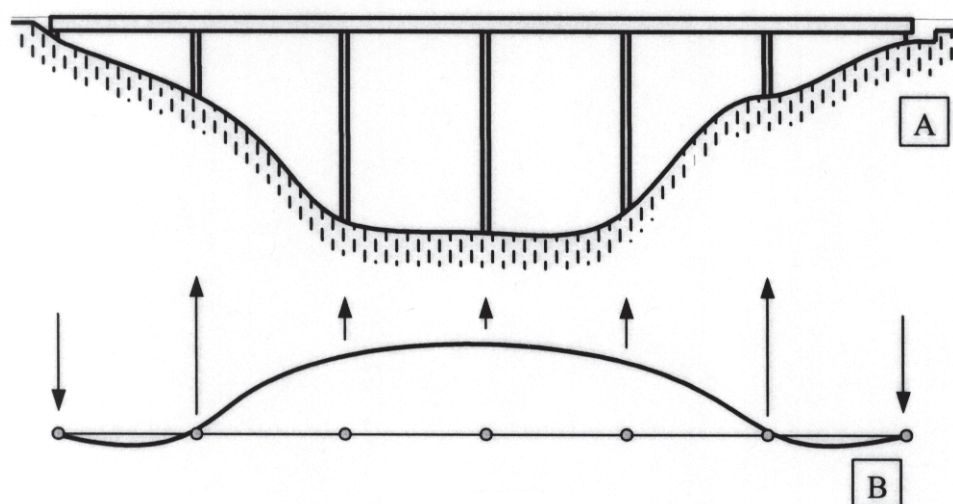
(7) The locations selected for energy dissipation should be chosen so as to ensure accessibility for inspection and repair. Such locations should be clearly indicated in the appropriate design documents.

(8) The location of areas of potential or expected seismic damage other than those in (7) should be identified and the difficulty of repairs should be minimised.

(9) In exceptionally long bridges, or in bridges crossing non-homogeneous soil formations, the number and location of intermediate movement joints should be decided.

(10) In bridges crossing potentially active tectonic faults, the probable discontinuity of the ground displacement should be estimated and accommodated either by adequate flexibility of the structure or by provision of suitable movement joints.

(11) The liquefaction potential of the foundation soil should be investigated in accordance with the relevant provisions of EN 1998-5:2004.



Key
A – Elevation
B - Plan

Figure 2.6: Unfavourable distribution of transverse seismic action

3 SEISMIC ACTION

3.1 Definition of the seismic action

3.1.1 General

(1)P The complexity of the model selected to describe the seismic action shall be appropriate to the relevant earthquake motion to be described and the importance of the structure and commensurate with the sophistication of the model used in the analysis of the bridge.

(2)P In this Section only the shaking transmitted by the ground to the structure is considered in the quantification of the seismic action. However, earthquakes can induce permanent displacements in soils arising from ground failure or fault rupture. These displacements may result in imposed deformations with severe consequences for bridges. This type of hazard shall be evaluated through specific studies. Its consequences shall be minimised by appropriate measures, such as selecting a suitable structural system. Tsunami effects are not treated in this Standard.

3.1.2 Application of the components of the motion

(1)P In general only the three translational components of the seismic action need to be taken into account for the design of bridges. When the response spectrum method is applied, the bridge may be analysed separately for the translational components of the seismic action in the longitudinal, transverse and vertical directions. In this case the seismic action is represented by three one-component actions, one for each direction, quantified in accordance with 3.2. The action effects shall be combined in accordance with 4.2.1.4.

(2)P When non-linear time-history analysis is performed, the bridge shall be analysed under the simultaneous action of the different components.

(3) The seismic action is applied at the interface between the structure and the ground. If springs are used to represent the soil stiffness either in connection with spread footings or with deep foundations, such as piles, shafts (caissons), etc. (see EN 1998-5:2004), the motion is applied at the soil end of the springs.

3.2 Quantification of the components

3.2.1 General

(1)P Each component of the earthquake motion shall be quantified in terms of a response spectrum, or a time-history representation (mutually consistent) as set out in EN 1998-1:2004, Section 3, which also provides the basic definitions.

3.2.2 Site dependent elastic response spectrum

3.2.2.1 Horizontal component

(1)P The horizontal component shall be in accordance with EN 1998-1:2004, **3.2.2.2**, depending on the ground type at the foundation of the supports of the bridge. When more than one ground types correspond to these supports, then **3.3** applies.

3.2.2.2 Vertical component

(1)P When the vertical component of the seismic motion needs to be taken into account (see **4.1.7**), the site-dependent response spectrum of this component shall be taken in accordance with EN 1998-1:2004, **3.2.2.3**.

3.2.2.3 Near source effects

(1)P Site-specific spectra considering near source effects shall be used, when the site is located within 10 km horizontally of a known active seismotectonic fault that may produce an event of Moment Magnitude higher than 6,5.

NOTE Unless the National Annex defines otherwise, it is recommended that a seismotectonic fault be considered to be active for the purposes of this requirement when there is an average historic slip rate of at least 1 mm/year and topographic evidence of seismic activity within the Holocene times (past 11000 years).

3.2.3 Time-history representation

(1)P When a non-linear time-history analysis is carried-out, at least three pairs of horizontal ground motion time-history components shall be used. The pairs should be selected from recorded events with magnitudes, source distances, and mechanisms consistent with those that define the design seismic action.

(2) When the required number of pairs of appropriate recorded ground motions is not available, appropriate modified recordings or simulated accelerograms may replace the missing recorded motions.

(3)P Consistency to the relevant 5% damped elastic response spectrum of the design seismic action shall be established by scaling the amplitude of motions as follows.

- a. For each earthquake consisting of a pair of horizontal motions, the SRSS spectrum shall be established by taking the square root of the sum of squares of the 5%-damped spectra of each component.
- b. The spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step.
- c. The ensemble spectrum shall be scaled so that it is not lower than 1,3 times the 5%-damped elastic response spectrum of the design seismic action, in the period range between $0,2T_1$ and $1,5T_1$, where T_1 is the natural period of the fundamental mode of the structure in the case of a ductile bridge, or the effective period (T_{eff}) of the isolation system in the case of a bridge with seismic isolation (see **7.2**).

d. The scaling factor derived from the previous step shall be applied to all individual seismic motion components.

(4) When the SRSS spectrum of the components of a recorded accelerogram gives accelerations the ratio of which to the corresponding values of the elastic response spectrum of the design seismic action shows large variation in the period range in (3)Pc, modification of the recorded accelerogram may be carried out, so that the SRSS spectrum of the modified components is in closer agreement with the elastic response spectrum of the design seismic action.

(5)P The components of each pair of time-histories shall be applied simultaneously.

(6) When three component ground motion time-history recordings are used for non-linear time-history analysis, scaling of the horizontal pairs of components may be carried out in accordance with (3)P, independently from the scaling of the vertical components. The latter shall be effected so that the average of the relevant spectra of the ensemble is not lower by more than 10% of the 5% damped elastic response spectrum of the vertical design seismic action in the period range between $0,2T_v$ and $1,5T_v$, where T_v is the period of the lowest mode where the response to the vertical component prevails over the response to the horizontal components (e.g, in terms of participating mass).

(7) The use of pairs of horizontal ground motion recordings in combination with vertical recordings of different seismic motions, consistent with the requirements of (1)P above, is also allowed. The independent scaling of the pairs of horizontal recordings and of the vertical recordings shall be carried out as in (6).

(8) Modification of the recorded vertical component in (6) and (7) is permitted using the method specified in (4).

3.2.4 Site dependent design spectrum for linear analysis

(1)P Both ductile and limited ductile structures shall be designed by performing linear analysis using a reduced response spectrum, called design spectrum, as specified by EN 1998-1:2004, 3.2.2.5.

3.3 Spatial variability of the seismic action

(1)P For bridge sections with a continuous deck the spatial variability shall be considered when one or both of the following two conditions hold.

- Soil properties along the bridge vary to the extent that more than one ground types (as specified in EN 1998-1:2004, 3.1.1) correspond to the supports of the bridge deck.
- Soil properties along the bridge are approximately uniform, but the length of the continuous deck exceeds an appropriate limiting length, L_{lim} .

NOTE The value ascribed to L_{lim} for use in a country may be found in its National Annex. The recommended value is: $L_{lim} = L_g/1.5$ where the length L_g is defined in (6) below.

(2)P The model describing spatial variability should account, even if only in a simplified way, for the propagative character of the seismic waves, 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 and refractions of the waves. The model should also account, even if only in a simplified way, for the further increase in loss of correlation due to differences in the mechanical properties of the soil along the bridge, which also modify the frequency content from one support to the other.

NOTE Models of the spatial variability of the earthquake motions and appropriate methods of analysis are presented in informative Annex D.

(3) Unless a more accurate evaluation is made, the simplified method specified in the paragraphs (4) to (7) may be used.

(4) The inertia response should be accounted for by one of the methods specified in Section 4 (see 4.2.1, 4.2.3 and 4.2.4) using a single input seismic action for the entire structure (e.g. a single response spectrum or corresponding accelerogram sets), corresponding to the most severe ground type underneath the bridge supports.

(5) The spatial variation of the seismic action may be estimated by pseudo-static effects of appropriate displacement sets, imposed at the foundation of the supports of the bridge deck. These sets should reflect probable configurations of the spatial variability of the seismic motion at free field and should be selected so as to induce maximum values of the seismic action effect under investigation.

(6) The requirements in (5) are deemed to be satisfied, by imposing each of the following two sets of horizontal displacements, applied separately, in each horizontal direction of the analysis, on the relevant support foundations or on the soil end of the relevant spring representing the soil stiffness. The effects of the two sets need not be combined.

a. Set A

Set A consists of relative displacements:

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

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

applied simultaneously with the same sign (+ or –) to all supports of the bridge (1 to n) in the horizontal direction considered (see Figure 3.1).

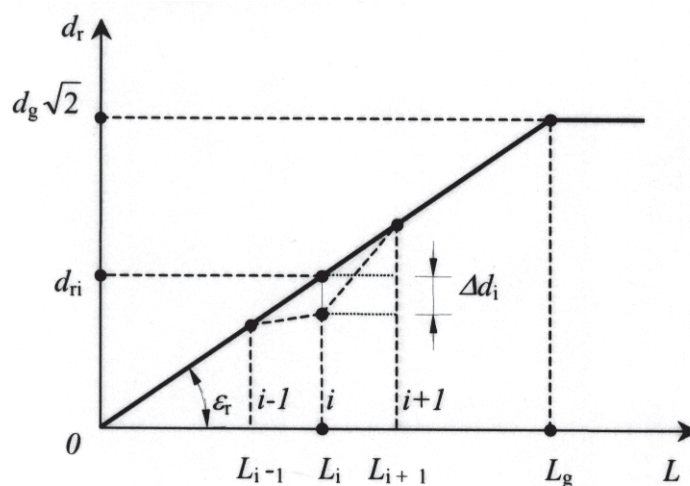


Figure 3.1 : Displacement Set A

where:

- d_g is the design ground displacement corresponding to the ground type of support i , in accordance with EN 1998-1:2004, 3.2.2.4;
- L_i is the distance (projection on the horizontal plane) of support i from a reference support $i = 0$, that may be conveniently selected at one of the end supports;
- L_g is the distance beyond which the ground motions may be considered as completely uncorrelated.

NOTE 1: The value ascribed to L_g for use in a country may be found in its National Annex. The recommended value is given in Table 3.1N, depending on the ground type:

Table 3.1N: distance beyond which ground motions may be considered uncorrelated

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

b. Set B

Set B covers the influence of ground displacements occurring in opposite directions at adjacent piers. This is accounted for by assuming displacements Δd_i of any intermediate support i (>1) relative to its adjacent supports $i-1$ and $i+1$ considered undisplaced (see Figure 3.1).

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

where:

- $L_{av,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ of intermediate support i to its adjacent supports $i-1$ and $i+1$ respectively. For the end supports (0 and n) $L_{av,0} = L_{01}$ and $L_{av,n} = L_{n-1,n}$;
- β_r is a factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports.

NOTE 2: The value ascribed to β_r for use in a country may be found in its National Annex. The recommended value is:

$\beta_r = 0.5$ when all three supports have the same ground type

$\beta_r = 1.0$ when the ground type at one of the supports is different than at the other two.

ε_r is as defined for set A above. If a change of ground type appears between two supports, the maximum value of ε_r should be used.

Set B consists of the following configuration of imposed absolute displacements with opposed sign at adjacent supports i and $i+1$, for $i = 0$ to $n-1$ (see Figure 3.2).

$$d_i = \pm \Delta d_i / 2$$

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

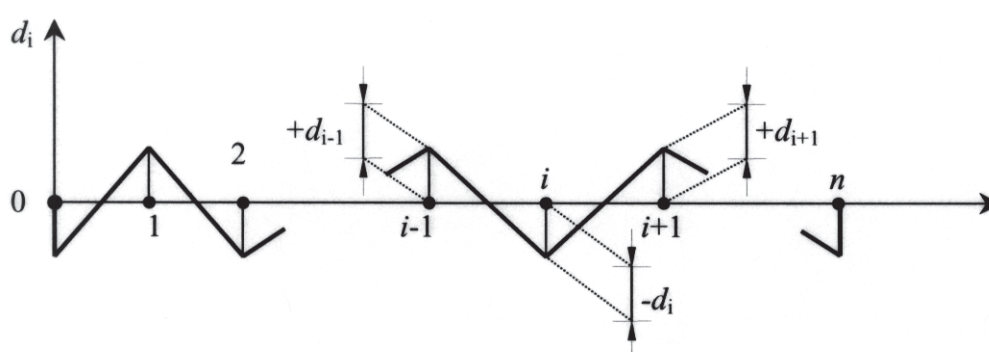


Figure 3.2 : Displacement Set B

(7)P In each horizontal direction the most severe effects resulting from the pseudo static analyses of (5) and (6) shall be combined with the relevant effects of the inertia response of (4), by using the SSRS rule (square root of the sum of squares). The result of this combination constitutes the effects of the analysis in the direction considered. For the combination of the effects of the different components of seismic action, the rules of 4.2.1.4 are applicable.

(8) When time-history analysis is performed the seismic motions applied at each support should reflect with sufficient reliability the probable spatial variability of the seismic action.

NOTE Guidance for generating samples of seismic motion reflecting the probable spatial variability is given in D.2 of Informative Annex D.

4 ANALYSIS

4.1 Modelling

4.1.1 Dynamic degrees of freedom

(1)P The model of the bridge and the selection of the dynamic degrees of freedom shall represent the distribution of stiffness and mass so that all significant deformation modes and inertia forces are activated under the design seismic excitation.

(2) It is sufficient, in certain cases, to use two separate models in the analysis, one for modelling the response in the longitudinal direction of the bridge, and the other for the transverse direction. The cases when it is necessary to consider the vertical component of the seismic action are defined in 4.1.7.

4.1.2 Masses

(1)P The mean values of the permanent masses and the quasi-permanent values of the masses corresponding to the variable actions shall be considered.

(2) Distributed masses may be lumped at nodes in accordance with the selected degrees of freedom.

(3)P For design purposes the mean values of the permanent actions shall be taken equal to their characteristic values.

(4)P The quasi-permanent values of variable actions shall be taken as equal to $\psi_{2,1}Q_{k,1}$ where $Q_{k,1}$ is the characteristic value of traffic load.

NOTE The value ascribed to: $\psi_{2,1}$ for use in a country may be found in its National Annex. The recommended values are:

Bridges with normal traffic and footbridges. In general and in accordance with the recommendation of EN 1990:2002, **Annex A2**, $\psi_{2,1}=0$.

Bridges with severe traffic and for the UDL system of Model 1 (LM1)

Road bridges $\psi_{2,1} = 0,2$;

Railway bridges $\psi_{2,1} = 0,3$.

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

When using $Q_{k,1}$, the adjustment factors α_Q and α_q should be applied in accordance with EN 1991-2:2003

(5) 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 an 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 omitted.

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