

Eurocode 8 : Design provisions for earthquake resistance of structures

Part 2. Bridges

ICS 91.120.20; 93.040

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/8, Structures in seismic regions, upon which the following bodies were represented:

- Association of Consulting Engineers
- British Geological Survey
- Department of the Environment (Building Research Establishment)
- Department of the Environment (Property and Building Directorate)
- Federation of Civil Engineering Contractors
- Institution of Civil Engineers
- Steel Construction Institute

This Draft for Development, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 May 1996

© BSI 1996

The following BSI reference relates to the work on this Draft for Development:
Committee reference B/525/8

ISBN 0 580 25714 2

Amendments issued since publication

Amd. No.	Date	Text affected

Contents

	Page
Committees responsible	Inside front cover
National foreword	ii
Foreword	10
Text of ENV 1998-2	13

National foreword

This Draft for Development has been prepared by Subcommittee B/525/8 and is the English language version of ENV 1998-2 : 1994 *Eurocode 8 : Design provisions for earthquake resistance of structures Part 2 : Bridges* published by the European Committee for Standardization (CEN).

ENV 1998-2 : 1994 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of buildings and civil engineering works. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel composite construction, timber, masonry and aluminium alloy, and also geotechnical and seismic design.

This publication is not to be regarded as a British Standard.

An ENV or European Prestandard (ENV) is made available for provisional application, but it does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard (EN).

There is no existing British Standard equivalent to ENV 1998-2 and there is no requirement in the Building Regulations to consider seismic actions on buildings and civil engineering works in the UK.

ENV 1998-1-1 states that the provisions of Eurocode 8 may be simplified or not observed in regions of low seismicity. Within the UK, the application of Eurocode 8 should not be necessary, unless the client or user of the works assesses that the associated seismic risk is such that it needs to be addressed.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies, both with respect to use in the UK and also in other more actively seismic areas. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted into an EN.

Comments should be sent in writing to the Secretary of B/525/8 at BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision, by 30 April 1997. Comments after this time will still be possible through corporate bodies, such as the engineering institutions.

ICS 91.120.20; 93.040

Descriptors: Civil engineering, buildings, structures, earthquake resistant structures, earthquake resistant design, bridges, computation

English version

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

Eurocode 8 — Conception et dimensionnement
des structures pour la résistance aux séismes —
Partie 2 : Ponts

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

This European Prestandard (ENV) was approved by CEN on 1993-06-04 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

© 1994 All rights of reproduction and communication in any form and by any means reserved in all countries to CEN and its members.

TABLE OF CONTENTS

	<i>Page</i>
FOREWORD	10
1. INTRODUCTION	13
1.1 Scope	13
1.2 Assumptions, units and symbols	13
1.3 Reference Codes	14
1.4 Distinction between Principles and Application Rules	15
1.5 Definitions	15
2. BASIC REQUIREMENTS AND COMPLIANCE CRITERIA	17
2.1 Design seismic event	17
2.2 Basic requirements	17
2.2.1 Non-collapse requirement (ultimate limit state)	17
2.2.2 Minimization of damage (serviceability limit state)	18
2.3 Compliance criteria	18
2.3.1 General	18
2.3.2 Intended seismic behaviour	18
2.3.2.1 Ductile behaviour	19
2.3.2.2 Limited ductile/essentially elastic behaviour	20
2.3.3 Resistance verifications	20
2.3.4 Capacity design	20
2.3.5 Provisions for ductility	21
2.3.5.1 General requirement	21
2.3.5.2 Structure ductility	21
2.3.5.3 Local ductility at the plastic hinges	22
2.3.5.4 Ductility verification	22
2.3.6 Connections-Control of displacements - Detailing	23
2.3.6.1 Design seismic displacement - Effective stiffness	23
2.3.6.2 Connections	24
2.3.6.3 Control of displacements - Detailing	24
2.3.7 Simplified criteria	25
2.4 Conceptual design	26

	<i>Page</i>
3. SEISMIC ACTION	28
3.1 Definition of the seismic action	28
3.1.1 General	28
3.1.2 Seismological aspects	28
3.1.3 Application of the components of the motion	28
3.2 Characterization of the motion at a point	28
3.2.1 General	28
3.2.2 Quantification of one component	29
3.2.2.1 Reference to EC8 - Part 1	29
3.2.2.2 Site dependent elastic response spectrum	29
3.2.2.3 Site dependent power spectrum	30
3.2.2.4 Time history representation	30
3.2.2.5 Site dependent design spectrum for linear analysis	30
3.2.3 Six component model	31
3.2.3.1 General	31
3.2.3.2 Separation of the components of the seismic action	31
3.3 Characterization of the spatial variability	31
4. ANALYSIS	32
4.1 Modelling	32
4.1.1 Dynamic degrees of freedom	32
4.1.2 Masses	32
4.1.3 Element stiffness	32
4.1.4 Modelling of the soil	33
4.1.5 Torsional effects	33
4.1.6 Behaviour factors for linear analysis	34
4.1.7 Vertical component of the seismic action	36
4.2 Methods of analysis	37
4.2.1 Linear dynamic analysis - Response spectrum methods	37
4.2.1.1 Definition, field of application	37
4.2.1.2 Significant modes	37
4.2.1.3 Combination of modal responses	37
4.2.1.4 Combination of the components of seismic action	38
4.2.2 Fundamental mode method	39
4.2.2.1 Definition	39
4.2.2.2 Field of application	39
4.2.2.3 Rigid deck model	39
4.2.2.4 Flexible deck model	40
4.2.2.5 Torsional effects in the transverse direction	41
4.2.2.6 Individual pier model	41

	<i>Page</i>
4.2.3 Alternative linear methods	42
4.2.3.1 Power spectrum analysis	42
4.2.3.2 Time series analysis	42
4.2.3.3 Criteria for the applicability of the results	42
4.2.4 Non-linear time domain analysis	42
 5. STRENGTH VERIFICATION	 43
5.1 General	43
5.2 Design strength	43
5.3 Capacity design effects	43
5.4 Second order effects	45
5.5 Design seismic combination	45
5.6 Resistance verification of concrete sections	46
5.6.1 Design effects	46
5.6.2 Structures of limited ductile behaviour	46
5.6.3 Structures of ductile behaviour	46
5.6.3.1 Flexural resistance of sections of plastic hinges	46
5.6.3.2 Flexural resistance of sections outside the region of plastic hinges	47
5.6.3.3 Shear resistance of elements outside the region of plastic hinges	47
5.6.3.4 Shear resistance of plastic hinges	48
5.7 Resistance verification for steel and composite elements	49
5.7.1 Piers	49
5.7.2 Bridge deck	49
5.8 Foundations	49
5.8.1 General	49
5.8.2 Design action effects	49
5.8.3 Resistance verification	49
 6. SPECIFIC DETAILING RULES	 50
6.1 Scope	50
6.2 Concrete piers	50
6.2.1 Confinement	50
6.2.1.1 Rectangular sections	51
6.2.1.2 Circular sections	51
6.2.1.3 Required confining reinforcement	53

	<i>Page</i>
6.2.1.4	53
6.2.2	54
6.2.3	54
6.3	54
6.4	55
6.4.1	55
6.4.2	55
6.5	55
6.6	56
6.6.1	56
6.6.2	57
6.6.2.1	57
6.6.2.2	57
6.6.2.3	57
6.6.3	57
6.6.4	59
6.7	60
6.7.1	60
6.7.2	60
6.7.3	61
6.7.4	62
7.	63
7.1	63
7.2	63
7.2.1	63
7.2.2	63
7.3	63
7.3.1	63
7.3.2	64
7.3.3	64
7.3.4	64
7.3.5	64
7.4	65
7.4.1	65
7.4.2	65

	<i>Page</i>
7.5	Verification 65
7.5.1	Bridge structure 65
7.5.1.1	Full isolation 65
7.5.1.2	Partial isolation 66
7.5.2	Isolation system 66
7.5.2.1	General requirements 66
7.5.2.2	Design displacement for isolating devices 66
7.5.2.3	Total design displacement 67
7.5.2.4	Vertical load stability 67
7.5.2.5	Overtopping 67
7.5.2.6	Clearances 67
7.6	Elastomeric bearings 67
7.6.1	General requirements 67
7.6.2	Total design shear strain 68
7.6.2.1	Components of shear strain 68
7.6.2.2	Shear strain due to compression 69
7.6.2.3	Shear strain due to the shear displacement 70
7.6.2.4	Shear strain due to angular rotations 70
7.6.3	Design criteria for normal elastomeric bearings 70
7.6.3.1	Maximum shear strains 70
7.6.3.2	Stability 71
7.6.3.3	Fixing of bearings 71
7.6.3.4	Clearances 72
7.6.4	Design criteria for special elastomeric bearings 72
8.	SPECIAL BRIDGES 73
8.1	Introduction 73
8.2	Selection of the intended seismic behaviour 73
8.2.1	Elastic behaviour 73
8.2.2	Ductile behaviour 73
8.2.2.1	General 73
8.2.2.2	High axial forces 74
8.2.2.3	Sequential yielding of the piers 74
8.3	Avoidance of brittle failure modes 74

		<i>Page</i>
ANNEXES		
ANNEX A	DESIGN SEISMIC EVENT FOR BRIDGES AND RECOMMENDATIONS FOR ITS SELECTION DURING THE CONSTRUCTION PERIOD (Informative)	75
A.1	Design seismic event	75
A.2	Design seismic event for the construction phase	75
ANNEX B	RELATIONSHIP BETWEEN DISPLACEMENT DUCTILITY AND CURVATURE DUCTILITY OF PLASTIC HINGES IN CONCRETE PIERS (Informative)	76
ANNEX C	ESTIMATION OF THE EFFECTIVE STIFFNESS OF REINFORCEMENT CONCRETE DUCTILE MEMBERS (Informative)	78
C.1	General	78
C.2	Method 1	78
C.3	Method 2	78
ANNEX D	SPATIAL VARIABILITY AND ROTATIONAL COMPONENTS OF EARTHQUAKE MOTION (Informative)	80
D.1	General	80
D.2	Variability of earthquake motion	80
D.2.1	Introduction	80
D.2.2	Wave propagation	80
D.2.3	Simplified model	82
D.2.3.1	Fundamentals	82
D.2.3.2	Response spectra	83
D.2.3.3	Power spectra	84
D.2.3.4	Time history representation	84
D.2.4	Relative static displacements model	84
D.2.4.1	General	84
D.2.4.2	Relative static displacements	84
D.2.4.3	Design action effects	85
D.3	Rotational components	85
D.3.1	Introduction	85
D.3.2	Wave protection	86

		<i>Page</i>
D.3.3	Response spectra	87
D.3.4	Power spectra	88
D.3.5	Time history representation	88
ANNEX E	ENSEMBLE OF ACCELEROGRAMS FOR STRUCTURAL ANALYSIS (Normative)	89
E.1	Introduction	89
E.2	Artificial accelerograms	89
E.3	Real accelerograms	90
E.4	Validation of ensembles of accelerograms	90
E.5	Number of Accelerograms to be used in the analysis	90
ANNEX F	ADDED MASS OF ENTRAINED WATER FOR IMMERSED PIERS (Normative)	92
ANNEX G	CALCULATION OF CAPACITY DESIGN EFFECTS (Normative)	93
G.1	General procedure	93
G.2	Simplifications	93
ANNEX H	DUCTILITY OF SPECIAL BRIDGES (Normative)	95
H.1	Influence of high axial forces	95
H.2	Sequential yielding of the piers	96
H.2.1	Simultaneous and sequential yielding	96
H.2.2	Sequential yielding in the longitudinal direction	96
H.2.3	Sequential yielding in the transverse direction	96
ANNEX J	PROTOTYPE TESTS OF SEISMIC ISOLATION SYSTEMS (Normative)	98
J.1	Scope	98
J.2	Records	98

	<i>Page</i>
J.3 Sequence and cycles	98
J.4 Units dependent on loading rate	99
J.5 Units dependent on bilateral load	99
J.6 Static test under the total design displacement	99
J.7 Determination of force-deflection characteristics	100
J.8 System adequacy	100
J.9 Design properties of the isolation system	101
J.9.1 Effective stiffness	101
J.9.2 Effective damping	101
J.10 Influence of other factors	102
J.10.1 Temperature	102
J.10.2 Ageing	102
J.10.3 Load history	103

FOREWORD

Objectives of the Eurocodes

- (1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.
- (2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.
- (3) Until the necessary set of harmonized technical specifications for products and for methods of testing their performance are available, some of the Structural Eurocodes cover some of these aspects in relevant Annexes.

Background to the Eurocode Programme

- (4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".
- (5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.
- (6) CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes.

Eurocode Programme

- (7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991 Eurocode 1 Basis of design and actions of 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 provisions for earthquake resistance of structures
EN 1999 Eurocode 9 Design of aluminium alloy structures

- (8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

- (9) This Part of ENV 1998 is being published as a European Pre-standard with an initial life of three years.
- (10) This Prestandard is intended for experimental application and for the submission of comments.
- (11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.
- (12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC8 at the following address:

IPQ c/o LNEC
Avenida do Brasil 101
P-1799 LISBOA Codex
PORTUGAL

or to the national standards organizations.

National Application Documents (NAD's)

- (13) In view of the responsibilities of authorities in member countries for the safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by []. The authorities in each member country are expected to assign definitive values to these safety elements.
- (14) Some of the harmonized supporting standards may not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving definitive values for safety elements, referencing compatible supporting standards and providing national guidance on the application of this Prestandard, will be issued by each member country or its Standards Organization.
- (15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

Matters specific to this Prestandard

- (16) The scope of Eurocode 8 is defined in Clause 1.1.1 of Part 1-1 of Eurocode 8 and the scope of this Part is herein defined in 1.1. Additional Parts of Eurocode 8 which are planned are indicated in Clause 1.1.3 of Part 1-1.
- (17) This Prestandard has been developed through the CEN Procedures from an earlier (1990) first draft entitled "EC8 Part 2 Bridges" that had been produced under EEC auspices.
- (18) In using the Prestandard in practice, particular regard should be paid to the underlying assumptions contained in Clause 1.3 of Part 1.1.

(19) Attention must be paid to the fact that this Part has to be used in conjunction with Part 1-1 and in addition to the provisions of the other relevant Eurocodes.

(20) Although the provisions for the seismic action presented in Part 1-1 have been basically followed some modifications and adjustments were found to be necessary in order to cope with the particularities of the bridges and especially the considerable distance between the supports of the bridge structure and its total length.

(21) In several cases this Prestandard allows alternative possibilities:

- The possibility of selecting a ductile or limited ductile seismic behaviour of the bridge is a choice offered by most seismic codes and is in fact inherent in all Parts of Eurocode 8.
- The different models which may be used for representing the seismic action are necessary in order to cover all possible cases encountered in the broad field of earthquake resistant design of bridges.
- The different methods of analysis reflect the necessary design sophistication depending on the specific bridge type.

Criteria and/or recommendations for selecting the most appropriate of the above possibilities are presented, to the extent possible, in the text of this Prestandard.

On the other hand the NAD's may be more specific on the applicability of each possibility and the Tendering Authorities (bridges are public works and thus subject to particular contractual aspects) may of course specify the one possibility considered most appropriate for the case in question.

(22) This Prestandard includes five normative and four informative Annexes.

1. INTRODUCTION

1.1 Scope

(1)P Within the framework of the general requirements set forth in Part 1.1, this part of the Code contains design Principles, Criteria and Application Rules applicable to the earthquake resistant design of bridges.

(2)P The Code covers primarily the seismic design of bridges in which the horizontal seismic actions are mainly resisted either at the abutments or through bending of the piers i.e. bridges composed of vertical or near vertical pier systems supporting the traffic deck superstructure. It can also be applied for the seismic design of other bridge types such as arch bridges, portal or tied bridges, and cable stayed bridges (see Chapter 8, Special Bridges).

(3)P The provisions contained in this Code cannot be considered as fully covering suspension bridges, moveable bridges or extreme cases of bridge configurations. (e.g. highly skewed bridges or bridges with large horizontal curvature). Timber and floating bridges are not included in the scope of this Part.

(4)P For such extreme cases of bridges, adequately conservative approaches, based mainly on capacity principles, must be adopted in order to cover the risks stemming from the peculiarities of each case and avoid the occurrence of brittle failure modes.

(5)P Finally, this Part of EC8 includes a special chapter on seismic isolation with provisions covering the application of this method of seismic protection.

1.2 Assumptions, units and symbols

(1)P The provisions given in Part 1.1 apply. Particular symbols however used frequently in Part 2 are presented in Clause 1.2.1 that follows.

1.2.1 Symbols particular to Part 2

γ_x	magnification factor for effects for the design of isolating devices
γ_o	overstrength factor (in connection with capacity design)
η	reduction factor due to damping
η_k	normalized axial force = $N_{Ed}/(A_c f_{ck})$
μ_c	curvature ductility
μ_d	displacement ductility
μ_φ	rotation ductility
ξ	viscous damping ratio in %
ξ'	viscous damping ratio (not in %) = $\xi/100$
ξ_I	effective damping of isolation systems
φ	angle of intersection (skew bridges)
ω_{wd}	mechanical reinforcement ratio (defining the confining reinforcement)
A_c	gross concrete area of a section
A_{cc}	confined concrete core area of a section

A_E	earthquake action
c_p	compression-wave propagation velocity
C_u	ultimate curvature
C_y	curvature at yield
d_E	design seismic displacement (due only to seismic action)
d_{Ed}	total design value of displacement under seismic conditions
d_{Ee}	seismic displacement determined from linear analysis
d_{eg}	effective displacement due to differential seismic ground displacement
d_{es}	effective seismic displacement of the support due to the deformation of the structure
d_{Ex}	design seismic displacement at the isolation interface
d_{Exd}	total design displacement at the isolation interface under seismic condition
d_g	peak ground displacement
d_G	displacement due to the long term effects of the permanent and quasi-permanent actions
E	seismic action effect
E_d	design value of an action effect under seismic conditions
F_C	capacity design effect
F_{Rd}	design resisting force to the earthquake action
J_{eff}	effective moment of inertia of a pier
k_{eff}	effective stiffness of an isolator
l_{ov}	minimum overlap length
m_a	added mass of externally entrained water per unit length of immersed pier
M_o	flexural overstrength of a plastic hinge section
M_t	torsional seismic action
N_{Ed}	axial force corresponding to the design seismic combination
S	site dependent elastic or design response spectrum (acceleration)
S_a	site averaged response spectrum
S_d	design spectral acceleration for linear analysis of ductile structures
S_e	site dependent elastic response spectrum (acceleration)
t_d	design life of structure
t_r	return period of seismic event
T_I	fundamental period of isolated structure

1.3 Reference Codes

(1)P Part 2 of Eurocode 8 is an integral part of that Eurocode and consequently its contents-albeit presented in a somewhat more direct and pragmatic way in order to transcend the general to the specific-abide by, are compatible with and reflect the principles contained in Part 1.1 "General Rules - Seismic Actions and General Requirements for Structures" of EC8.

(2)P Furthermore, it is also closely related and compatible with the other relevant Parts of EC 8 which are the materials' provisions of Part 1.3 and Part 5 dealing with foundations and geotechnical aspects. Unless otherwise specified in the present Part the relevant provisions of these Parts are applicable.

(3)P In accordance with the rules governing the entire EC 8, for the application of this Part reference shall be made to the other relevant Eurocodes and their supplements on bridges.

1.4 Distinction between Principles and Application Rules

(1)P Depending on the character of the individual clauses, distinction is made in this Eurocode between Principles and Application Rules.

(2)P The Principles comprise:

- general statements and definitions for which there is no alternative,
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3)P The Application Rules are generally recognised rules which follow the Principles and satisfy their requirements.

(4)P It is permissible to use alternative design rules which differ from the Application Rules given in Eurocode 8, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent, with regard to the safety and serviceability achieved, to the present Eurocode 8.

(5)P The Principles are identified by the letter P, following the paragraph number.

1.5 Definitions

(1)P The following terms are used in Part 2 with the following meaning:

Capacity design: The design procedure used in structures of ductile behaviour to secure 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.

Seismic Isolation: The provision of bridge structures with special isolating devices for the purpose of reducing the seismic response.

Spatial variability: Spatial variability of the seismic action means that the motion at different supports of the bridge is assumed to be different and, as a result, the definition of the seismic action cannot be based on the characterization of the motion at a single point, as is usually the case.

Seismic behaviour: The 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 are usually provided with appropriate slack so as to be activated only in the case when the design seismic displacement is exceeded.

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

Design seismic displacement: The displacement induced by the design seismic actions.

Total design displacement under seismic conditions: The displacement used to determine adequate clearances for the protection of critical or major structural elements. 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.

Special Elastomeric Bearings: Laminated elastomeric bearings intended for seismic isolation of bridges and conforming to the prototype tests of Annex J.

2. BASIC REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Design seismic event

(1)P The design philosophy of this Code, regarding the seismic resistance of bridges, is based on the general requirement that emergency communications shall be maintained, with appropriate reliability, after the design seismic event.

(2) Target reliabilites are selected on the basis of Clause 2.1 (2) of Part 1.1 (see also Annex A of this Part).

(3) Differentiation of target reliability may, in the absence of reliable statistical evaluation of seismological data, be obtained by multiplying the design seismic action with an importance factor γ_I having the following values:

Bridge Importance Category	Importance Factor γ_I
Greater than average	[1.30]
Average	1.00
Less than average	[0.70]

(4) To the category of "greater than average" importance belong bridges of critical importance for maintaining communications, especially after a disaster, bridges whose failure is associated with a large number of probable fatalities, and major bridges for which a design life greater than normal is required.

(5) To the category of "less than average" importance belong bridges which are not critical for communications and for which the adoption of either the standard probability of exceedence of the design seismic event or the normal bridge design life, is not economically justifiable.

(6) Recommendations for the selection of the design seismic event, appropriate for use during the construction period of bridges are given in Annex A.

2.2 Basic requirements

(1)P With reference to the probability of occurrence of an earthquake event in the design life of the bridge, the following two basic requirements are defined.

2.2.1 Non-collapse requirement (ultimate limit state)

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

(2)P The bridge shall be damage-tolerant, i.e. those parts of the bridge susceptible to damage by their contribution to energy dissipation during the design seismic event shall be designed in such a manner as to ensure that the structure can sustain the actions from emergency traffic, and inspections and repair can be performed easily.

(3)P To this end, flexural yielding of specific sections (i.e. the formation of plastic hinges) is allowed in the piers, and is in general necessary, in regions of high seismicity, in order to reduce the design seismic action to a level requiring reasonable additional construction costs.

(4)P The bridge deck however shall in general be protected from the formation of plastic hinges and from unseating under extreme seismic displacements.

2.2.2 Minimisation of damage (serviceability limit state)

(1)P After seismic actions with high probability of occurrence during the design life of the bridge, the parts of the bridge intended to contribute to energy dissipation during the design seismic event, shall only undergo minor damage without giving rise to any reduction of the traffic or the need of immediate repair.

2.3 Compliance criteria

2.3.1 General

(1)P In order to satisfy the basic requirements set forth in Clause 2.2, the design must comply with the criteria outlined in the following Clauses. In general the criteria while aiming explicitly at satisfying the non-collapse requirement (2.2.1), implicitly cover the damage minimization requirement (2.2.2) as well.

(2)P The compliance criteria depend on the behaviour which is intended for the bridge under the design seismic action. This behaviour may be selected according to the following Clause.

2.3.2 Intended seismic behaviour

(1)P The bridge shall be designed so that its behaviour under the design seismic action is ductile, or limited ductile/essentially elastic, depending on the characteristics of the global force-displacement relationship of the structure (see Fig. 2.1)

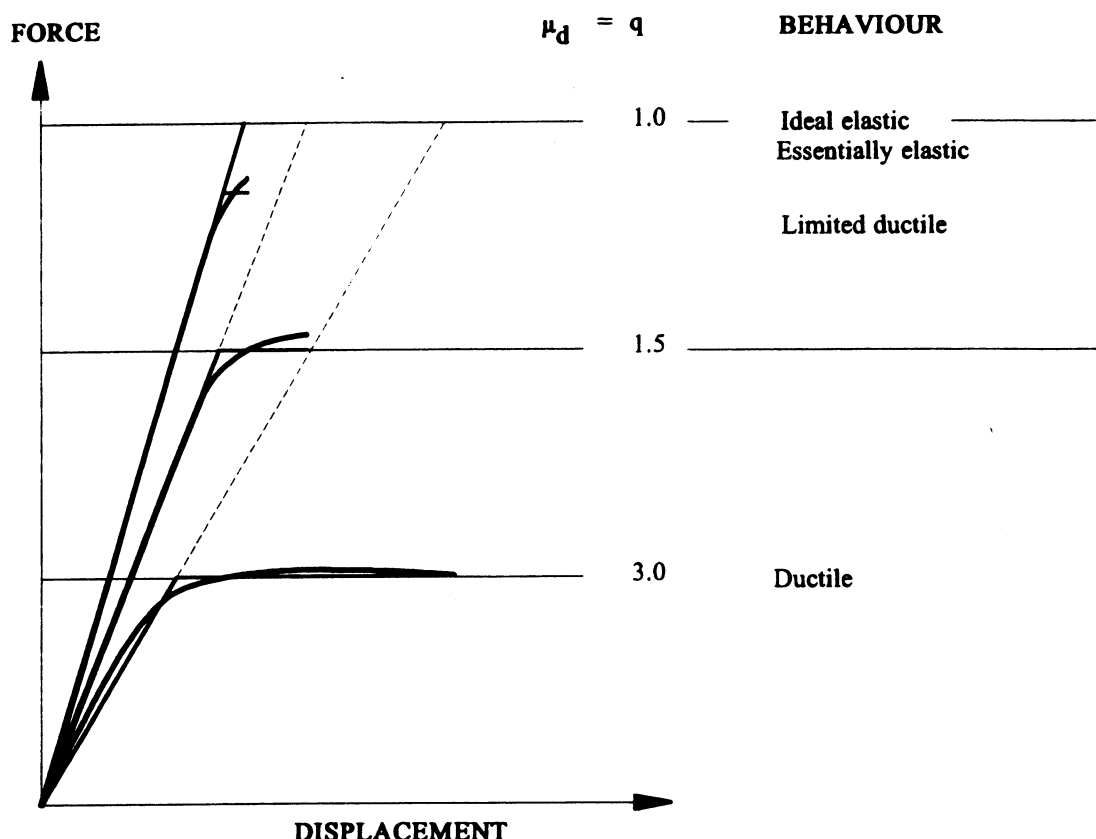


Figure 2.1
Seismic behaviour

2.3.2.1 Ductile behaviour

(1)P 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 according to Section 7. The part of this subclause that follows refers to ductile behaviour achieved by flexural plastic hinges.

(2)P The bridge shall be designed so that a dependably stable plastic mechanism can form in the structure through the formation of flexural plastic hinges, normally in the piers, which act as the primary energy dissipating components.

(3)P As far as possible the location of plastic hinges shall be selected at points accessible for inspection and repair. In general the bridge deck shall remain within the elastic range.

(4)P The formation of plastic hinges is not allowed in reinforced concrete sections where the normalized axial force η_k defined in 5.3.(3) exceeds 0.6.

(5)P The global force-displacement relationship shall show a significant force plateau at yield and shall be reversible in order to ensure hysteretic energy dissipation at least over 5 deformation cycles. (See Figures 2.1, 2.2 and 2.3).

Note

Elastomeric bearings used over some supports may cause some increase of the resisting force, with increasing displacements, after plastic hinges have formed in the other supporting elements. However the rate of increase of the resisting force should be appreciably reduced after the formation of plastic hinges.

(6) Flexural 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.

(7) Supporting elements (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.

(8) It is pointed out that the formation of flexural hinges is necessary in order to ensure energy dissipation and consequently ductile behaviour (See Clause 4.1.6 (2)). The deformation of the usual elastomeric bearings is mainly elastic and does not lead to ductile behaviour. (See Clause 4.1.6 (10)) When no plastic hinge develops, no ductile behaviour can be considered.

2.3.2.2 Limited ductile/essentially elastic behaviour

(1)P No significant yield appears under the design earthquake. 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 a behaviour corresponds to a behaviour factor $q \leq 1.5$ and shall be referred to, in this Code, as "limited ductile".

2.3.3 Resistance verifications

(1)P In bridges of ductile behaviour the regions of plastic hinges shall be verified to have adequate flexural strength to resist the design seismic effects as defined in Clause 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" defined in Clause 2.3.4 below (see also Clause 5.3).

(2)P In bridges of limited ductile behaviour all sections shall be verified to have adequate strength to resist the design seismic effects.

2.3.4 Capacity design

(1)P For bridges of ductile behaviour, capacity design shall be used to secure the hierarchy of resistances of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes.

(2)P This shall be effected by designing all elements intended to remain elastic for "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 defined in Clause 5.3.

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

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 structure ductility.

Note

The definitions of structure and local ductilities, given in subclauses 2.3.5.2 and 2.3.5.3 that follow, are intended to provide the theoretical basis of ductile behaviour. In general they are not required for practical verification of ductility, which is effected according to subclause 2.3.5.4.

2.3.5.2 Structure ductility

(1)P Referring to an equivalent one degree of freedom system, having an idealized elasto-plastic force-displacement relationship, as shown in Figure 2.2., the design value of the ductility of the structure (available displacement ductility) 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)P The constant maximum force of the global elasto-plastic diagram is assumed equal to the design resisting force F_{Rd} . The yield displacement defining the elastic branch is selected so as to best approximate the design curve (for repeated loading) up to F_{Rd} .

(3)P The ultimate displacement d_u is defined as the maximum displacement satisfying the following condition. The structure is capable to sustain at least 5 full cycles of deformation to the ultimate displacement,

- without initiation of failure of the confining reinforcement for R.C. sections, or local buckling effects for steel sections, and
- without drop of the resisting force for steel ductile elements or a drop exceeding 0.20 F_{Rd} for R.C. ductile elements (see Figure 2.3).

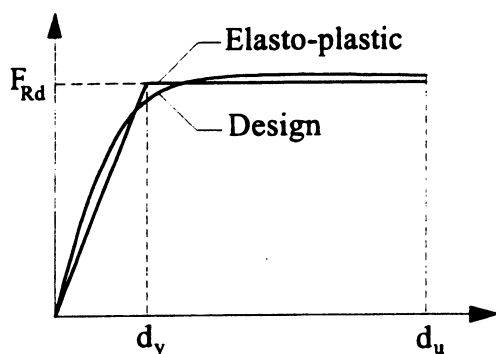


Figure 2.2
 Global force-displacement diagram
 (Skeleton curve)

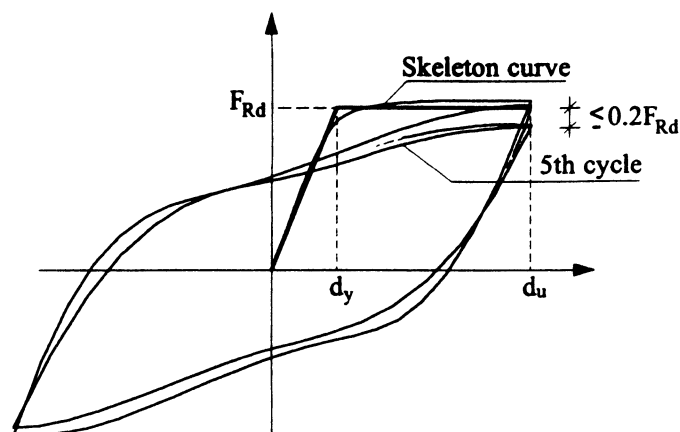


Figure 2.3
 Force-displacement cycles

2.3.5.3 Local ductility at the plastic hinges

(1)P The structure ductility depends on the available local ductility at the plastic hinges (See Figure 2.4) expressed as curvature ductility of the cross-section:

$$\mu_c = C_u / C_y \quad (2.2)$$

or as rotation ductility of the hinge:

$$\mu_\phi = \phi_u / \phi_y = 1 + (\phi_u - \phi_y) / \phi_y = 1 + R \quad (2.3)$$

R is the rotation capacity of the plastic hinge

(2)P In the above expressions the ultimate deformations are defined subject to the conditions of 2.3.5.2 (3).

2.3.5.4 Ductility verification

(1)P Compliance with the Specific Rules given in Section 6 shall be deemed, in general, to ensure the availability of adequate local and overall structure ductility.

(2)P In special cases the assumed ductility may be directly verified on the basis of the available curvature ductilities and the lengths of the plastic hinges, or the rotation ductilities (See Annex B).

(3)P When non-linear dynamic analysis is performed, ductility demands shall be checked against available local ductility capacities of the plastic hinges.

(4)P For bridges of limited ductile behaviour the provisions of Clause 6.5 must be applied.

2.3.6 Connections - Control of displacements - Detailing

2.3.6.1 Design seismic displacement - Effective stiffness

(1)P Within the framework of equivalent linear analysis methods allowed by the present code, the stiffness of each element shall adequately approximate its deformation under the maximum stresses induced by the design seismic action. For elements containing plastic hinges this corresponds to the secant stiffness at the theoretical yield point. (See Figure 2.4).

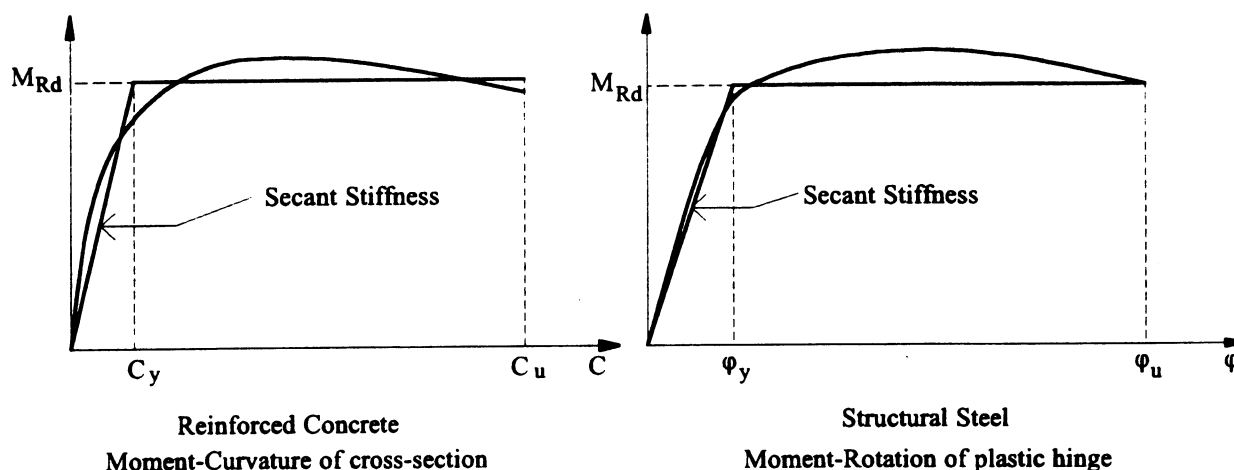


Figure 2.4
Moment - deformation diagrams at plastic hinges

(2) For reinforced concrete elements in bridges of ductile behaviour, in the absence of a more rigorous method, the effective stiffness may be estimated as follows:

- For ductile components (piers), a value calculated on the basis of the secant stiffness at the theoretical yield point of the plastic hinge (See Annex C).
- For reinforced and/or prestressed concrete components remaining within the elastic range, the stiffness of the uncracked sections.

(3)P In limited ductile bridges the stiffness of the uncracked sections shall be used globally.

Note

It is noted that an overestimation of the effective stiffness leads to results which are on the safe side regarding the seismic actions. In such a case, only the displacements need be corrected, after the analysis, on the basis of the resulting level of actual stresses (see Annex C). On the other hand, if the initial assumption of effective stiffness is significantly lower than that corresponding to the actual stresses, the analysis must be repeated using a better approximation of the effective stiffness.

(4)P The displacements d_{Ee} determined from linear seismic analysis, static or dynamic, shall be multiplied by the q-factor (behaviour factor) used in the analysis (see 4.1.6), to obtain the design seismic displacements d_E .

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

(5)P When the fundamental period of the bridge T is less than $T_0 = 1.5T_C$, where T_C has the values given by Table 4.1 of EC8: Part 1.1 (see also 3.2.2.2.1 (5)), the design seismic displacement shall be estimated as follows:

$$d_E = \pm \mu_d d_{Ee} \quad (2.5)$$

where the displacement ductility μ_d shall be estimated as

$$\mu_d = (q-1) \frac{T_0}{T} + 1 \quad (2.6)$$

and T is the fundamental period of the bridge in the direction under consideration.

(6)P When non-linear time domain analysis is used, the deformation characteristics of the yielding elements shall adequately approximate their actual post-elastic behaviour, both in the loading and unloading branches of the hysteresis loops, as well as the eventual degradation effects.

2.3.6.2 Connections

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

(2)P Bearings, links and holding-down devices used for securing structural integrity, shall be designed using capacity design effects, (see Clauses 5.3 and 6.6.2.1). Appropriate overlap lengths shall be provided between supporting and supported elements at moveable connections, in order to avoid unseating.

(3) Alternatively, positive linkage between connected elements may be used, (see Clause 6.6.3) as a second line of defense beyond the design seismic actions and displacements.

2.3.6.3 Control of displacements - Detailing

(1)P In addition to ensuring a satisfactory overall ductility, structural and non- structural detailing must ensure adequate behaviour of the bridge and its components under the design seismic displacements.

(2)P Adequate clearances shall be provided for protection of critical or major structural elements. Such clearances shall accommodate the total design value of the displacement under seismic conditions d_{Ed} determined as follows:

$$d_{Ed} = d_E + d_G \pm d_{Ts} \quad (2.7)$$

where:

- d_E is the design seismic displacement according to equation (2.4)
 d_G is the displacement due to the permanent and quasi-permanent actions measured in long term (e.g. post-tensioning, shrinkage and creep for concrete decks).
 d_{Ts} is the displacement due to thermal movements, corresponding to a representative value T_s of the temperature variation considered appropriate for combination with seismic effects, to be defined by the relevant Part of EC1. Until such a definition is available the following estimation may be used:

$$d_{Ts}=[0.4]d_T$$

where d_T is the design displacement due to thermal movements.

The total design seismic displacement shall be increased by the displacement due to second order effects when such effects have a significant contribution.

(3) The relative 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 calculated for each section.

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

(5)P At joints of railway bridges transverse differential displacement shall be either avoided or limited to values appropriate for preventing derailments.

(6) The detailing of non-critical structural elements (e.g. deck movement joints), expected to be damaged during the design seismic event, should cater, as far as possible, for a predictable mode of damage and provide for the possibility of permanent repair. Relevant clearances should accommodate appropriate fractions of the design seismic displacement and thermal movement, after allowing for any long term creep and shrinkage effects, so that damage under frequent earthquakes can be avoided.

(7) The appropriate values of such fractions depend on techno-economical considerations. In the absence of an explicit optimization the following values are recommended:

- [40%] of the design seismic displacement
- [50%] of the thermal movement

2.3.7 Simplified criteria

(1) In regions of low or moderate seismicity ($a_g \leq [0.10g]$), the National Application Documents may establish an appropriate classification of the bridges and specify simplified compliance criteria, pertaining to the individual classes. These simplified criteria may be 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 seismic effects at the conceptual stage of the design of bridges is important even in low to moderate seismicity regions.

(2) In areas of low to moderate seismicity ($a_g \leq [0.10g]$) the type of intended seismic behaviour of the bridge (see Clause 2.3.2) must be decided. If a limited ductile (or essentially elastic) behaviour is selected, the requirements of the following Clauses should be applied:

- Clause 6.5, regarding the accessibility of potential plastic hinges.
- Clause 6.6, regarding the design of bearings and links and the required seating lengths.

(3) In areas of moderate to 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 base isolation and energy dissipating devices, must be decided. When a ductile behaviour is selected the following main points should be considered:

(4) The number of supporting elements (piers and abutments) that will be used to resist the seismic forces in the longitudinal and the transverse direction must be decided. In general continuous structures behave better under earthquake conditions than bridges having 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 earthquake resisting piers may have to be reduced, by using flexible mountings between deck and piers in one or in both directions, to avoid either high reactions due to restrained deformations or an undesirable distribution of the seismic actions and/or of the capacity design effects. (See also (6) below)

(5) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the level of 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 continuous deck in which the transverse stiffness of the abutments and the adjacent piers is very high in relation to that of the other piers (as may occur in steep sided valleys), a very unfavourable distribution of the transverse seismic action on these elements may take place, as shown in figure 2.5. In such a case it may be preferable to use transversally moveable or flexible bearings over the abutments or the short piers.

(7) The location of the energy dissipating points must be chosen so as to ensure their accessibility for inspection and repair.

(8) The location of other potential or expected damage areas under severe motions must be identified and the difficulty of repairs must be minimised.

- (9) In exceptionally long bridges, or in bridges crossing non-homogeneous soil formations, the number and location of intermediate movement joints must be decided.
- (10) In the case of bridges going over 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 must be investigated according to the relevant provisions of EC8: Part 5.

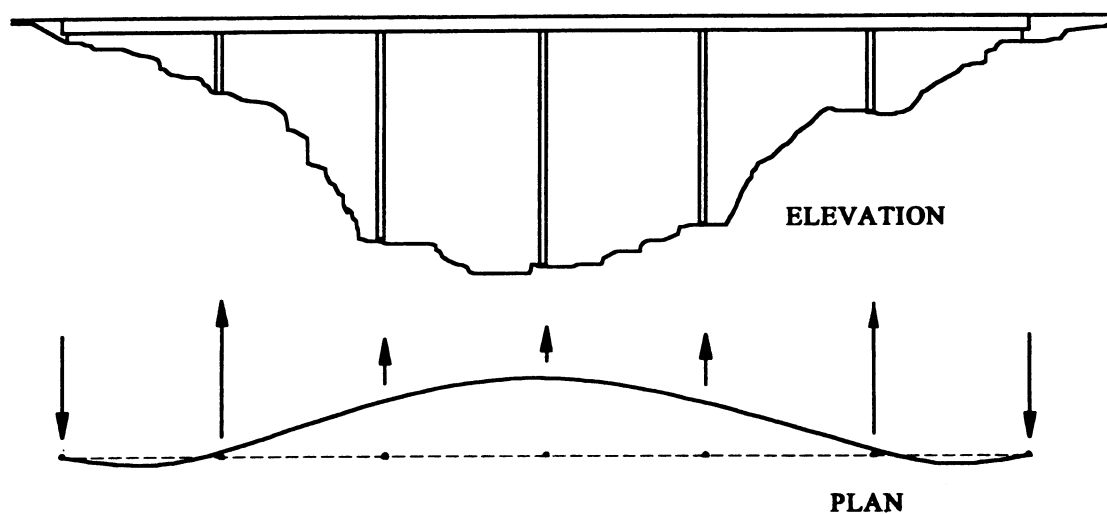


Figure 2.5
Unfavourable distribution of transverse seismic action

3. SEISMIC ACTION

3.1 Definition of the seismic action

3.1.1 General

(1)P The seismic action can be defined by means of different models, whose complexity shall be appropriate to the relevant earthquake motion to be described and commensurate with the model used for the idealization of the bridge.

(2)P In this clause only the vibratory excitation transmitted by the ground to the structure is considered in the quantification of the seismic action. However, earthquakes can induce permanent displacements in soils (ruptures, liquefaction of sandy layers and ground offset due to active faulting) that may result in imposed deformations with severe consequences to bridges. This type of hazard shall be evaluated through specific studies and its consequences shall be minimised by an appropriate selection of the structural system. Tsunami effects are not treated in this Code.

3.1.2 Seismological aspects

(1)P In the definition of the seismic action the following aspects shall be considered:

- the characterization of the motion at a point;
- the characterization of the spatial variability of the motion.

3.1.3 Application of the components of the motion

1(P) In general only the three translational components of the seismic action are taken into account. When the Response Spectrum method is applied the bridge may be analyzed separately for vibrations 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 according to 3.2.2 and 3.2.3.2. The action effects shall be combined according to 4.2.1.4;

2(P) When linear time domain analysis is performed or when the six component model or the spatial variability of the seismic motion is taken into account, the bridge shall be analyzed under the simultaneous action of the different components.

3.2 Characterization of the motion at a point

3.2.1 General

(1)P The characterization of the motion at a point shall be carried out in two phases:

- Quantification of each component of the motion;
- Construction of a three component model of the motion with three translational components, or of a six component model of the motion, with three translational components and three rotational components.

(2) The seismic action is generally applied at the interface between footings and ground. In case of pile supported footings or pile-piers it is necessary to consider the stiffness of the pile supports (according to Part 5). In general it is not necessary to use all six the components of the ground motion.

3.2.2 Quantification of one component

3.2.2.1 Reference to EC8 - Part 1

(1)P The quantification of one component of the earthquake motion shall be carried out in terms of a response spectrum, or a power spectrum, or a time history representation (mutually consistent) as presented in Section 4 of Part 1.1, General Rules and Requirements, which shall be referred to as containing the basic definitions.

3.2.2.2 Site dependent elastic response spectrum

3.2.2.2.1 Horizontal component

(1)P The horizontal component shall be taken in accordance with Clause 4.2.2 of Part 1.1.

(2) When the elastic spectrum is used as a design spectrum (see Clause 3.2.2.5) the values of the exponents k_1 and k_2 shall be taken equal to those of k_{d1} and k_{d2} of Clause 4.2.4 of Part 1.1.

3.2.2.2.2 Vertical component

(1)P When needed, the site dependent response spectrum for the vertical component of the earthquake motion shall be taken in accordance with Clause 4.2.1 (3) of Part 1.1

3.2.2.2.3 Site averaged response spectrum

(1)P In the case of bridges whose abutments and piers are supported on different soil conditions but which do not require the use of a spatial variability model for the seismic action, the site dependent response spectrum shall be replaced by a site averaged response spectrum.

(2)P The site averaged response spectrum S_a is defined as a weighted average of the appropriate site dependent response spectra and is determined by

$$S_a(T) = \sum_i \frac{r_i}{\sum_j r_j} S_i(T) \quad (3.1)$$

where r_i is the reaction force on the base of pier i when the deck is subjected to a unit displacement while the base is kept immobile; S_i is the site dependent response spectrum appropriate to the soil conditions at the foundation of pier i .