DRAFT FOR DEVELOPMENT

DD ENV 1998-2 : 1996

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

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Association of Consulting Engineers
British Geological Survey
Department of the Environment (Building Research Establishment)
Department of the Environment (Property and Building Directorate)
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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.

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

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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

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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 mulitiplying 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)

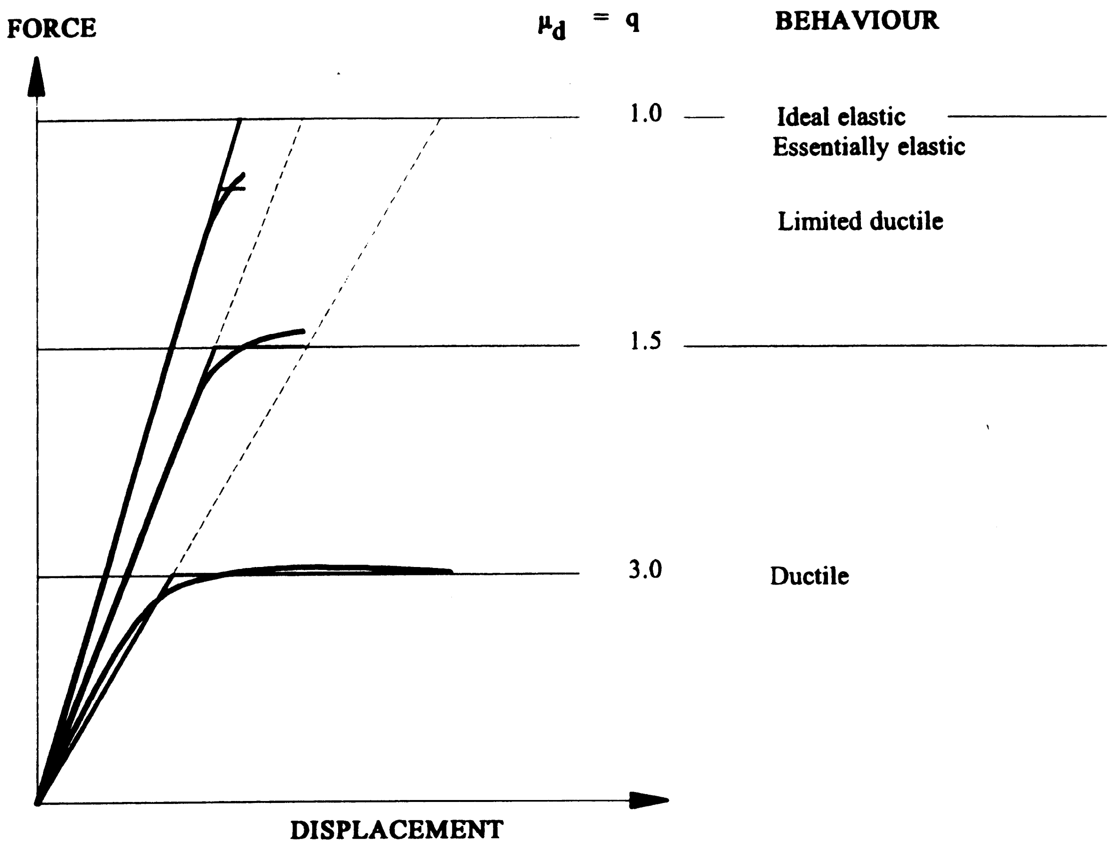


Figure 2.1
Seismic behaviour

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).

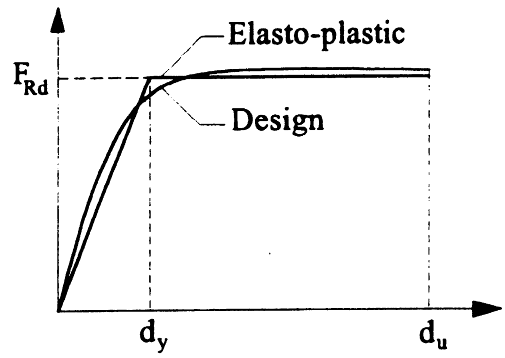


Figure 2.2
Global force-displacement diagram
(Skeleton curve)

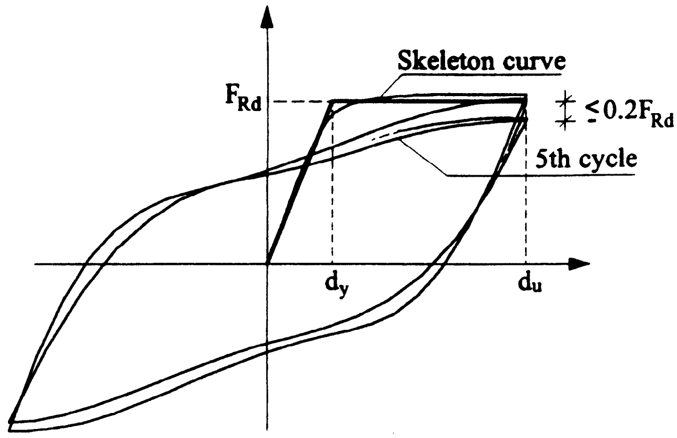


Figure 2.3
Force-displacement cycles

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:

$$\mu_c = C_u/C_y \tag{2.2}$$

or as rotation ductility of the hinge:

$$\mu_\varphi = \varphi_u/\varphi_y = 1+(\varphi_u-\varphi_y)/\varphi_y = 1+R \tag{2.3}$$

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).

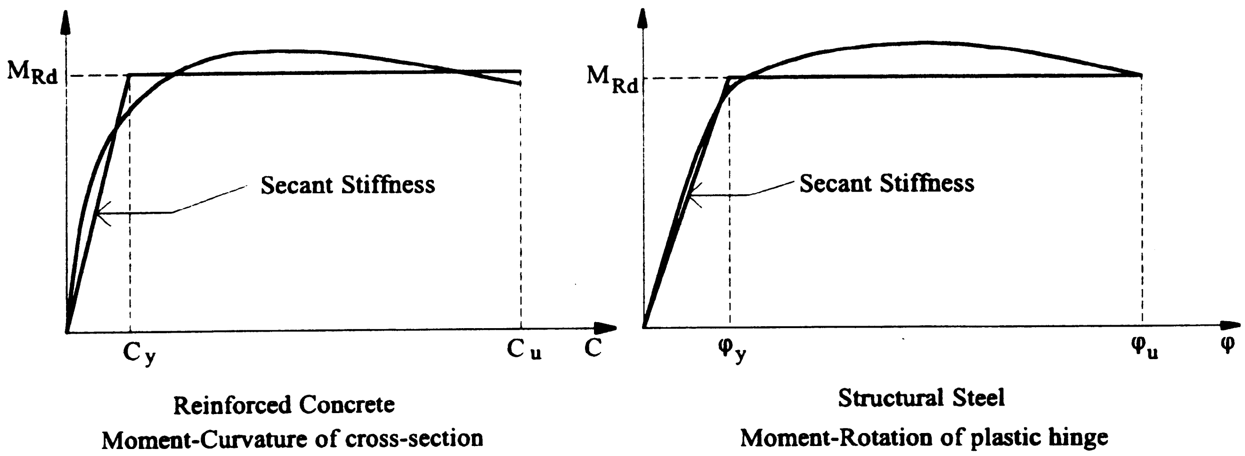


Figure 2.4
Moment - deformation diagrams at plastic hinges

(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.

$$d_E = \pm q d_{Ee} \tag{2.4}$$

(5)P When the fundamental period of the bridge T is less than T_o=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:

$$d_E = \pm \mu_d d_{Ee} \tag{2.5}$$

where the displacement ductility μ_d shall be estimated as

$$\mu_d = (q-1)\frac{T_o}{T} + 1 \tag{2.6}$$

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:

$$d_{Ed} = d_E + d_G \pm d_{Ts} \tag{2.7}$$

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 strinkage 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.

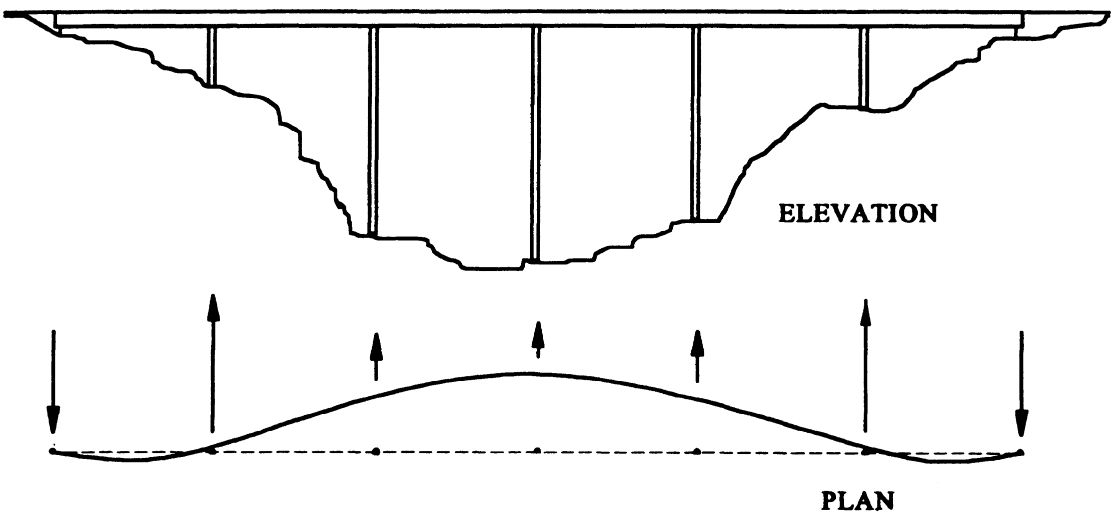


Figure 2.5
Unfavourable distribution of transverse seismic action

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 rèsponse 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

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

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.

Note
The average shall be computed separately for each of the two horizontal components and for the vertical component.

(3) The site averaged response spectrum may be substituted by an envelope spectrum obtained by considering, for each period, the highest value of the site dependent response spectra corresponding to the different soil conditions at the foundations of the bridge.

3.2.2.3 Site dependent power spectrum

(1)P The earthquake action can be described by a stochastic stationary gaussian process defined by a power spectrum and considered with a duration limited to a given time interval. This description of the motion shall be consistent with the site dependent response spectrum. Consistency between power spectrum and response spectrum shall be defined as equality between the response spectrum value and the mean value of the probability distribution of the largest extreme value (for the duration considered) of the response of a one degree of freedom oscillator with a corresponding natural frequency and viscous damping.

Note
The term extreme value refers to the absolute value of a maximum or a minimum value. It should be noted that in some cases (local) maximum values may have negative values and (local) minimum values may have positive values.

3.2.2.4 Time history representation

(1)P The earthquake action can be described by an ensemble of artificially generated or real accelerograms. This ensemble shall contain a large enough number of accelerograms so that reliable estimates of the earthquake action effects are obtained and shall be consistent with the corresponding site dependent response spectra. Specific provisions are given in Annex E.

(2)P Consistency between the accelerogram ensemble and the response spectrum shall be defined as equality, within an appropriate confidence interval, between the response spectrum and the sample average of the largest extreme value of the response of a one degree of freedom oscillator with a corresponding natural frequency and viscous damping.

3.2.2.5 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 Clause 4.2.4 of Part1.1.

3.2.3 Six component model

3.2.3.1 General

(1)P The six component model of the earthquake motion at a point shall be developed from the probable contribution of the P, S, Rayleigh and Love waves to the total earthquake vibration. However, the simplified models referred in Annex D may be used if geological discontinuities are not present.

3.2.3.2 Separation of the components of the seismic action

(1)P For the separation of the components of the seismic action the relevant provisions of 3.1.3 are applicable. However, the vertical component may, in general, be disregarded if the bridge is not particularly sensitive to vibrations in this direction; furthermore, the rotational components are usually not important and can also be disregarded.

3.3 Characterization of the spatial variability

(1)P The spatial variability shall be considered when:

- Geological discontinuities (e.g. soft soil contiguous to crystaline rock) or marked topographical features are present;
- The length of the bridge is greater than [600 m], even if there are no geological discontinuities or marked topographical features.

(2) In annex D models to take into account the spatial variability of the earthquake motion are presented.

(3)P The spatial variability dealt with in this subclause concerns the continuous deformation of the ground, in the elastic or in the post-elastic range. However, in the case of strong earthquakes, discontinuous deformations, due to surface faulting or soil ruptures, may be induced. Measures to prevent the risks related to this type of hazard, such as the adoption of structural systems which minimize its effects, shall be taken. (See also Clause 2.4 (9)).

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 adequately represent the distribution of stiffness and mass so that all significant deformation modes and inertia forces are properly activated under the design seismic excitation.

(2) It is sufficient , in most cases , to use two separate models in the analysis , one for modelling the behaviour in the longitudinal direction ,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 Clause 4.1.7.

4.1.2 Masses

(1)P For the calculation of masses 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 according to the selected degrees of freedom.

(3)P For design purposes the mean values of the permanent actions are identified by their characteristic values ; the quasi -permanent values of variable actions are given by $\psi_{21} Q_{1k}$ where Q_{1k} is the characteristic value of traffic load (see Clause 4.4 of Part 1.1.). In general and in accordance with EC 1, Part 3 the value of $\psi_{21}=0$ shall be used for bridges with normal traffic and foot bridges.

(4)P For bridges with intense traffic the following values of ψ_{21} are recommended:

- Road bridges $\psi_{21} = [0.2]$
- Railway bridges $\psi_{21} = [0.3]$

The above values shall be applied to the uniform load of Model 1 (LM 1) according to EC1, Part 3.

(5) When the piers are immersed in water, in the absence of a rigorous assessment of the hydrodynamic interaction, this effect may be estimated by taking into account an added mass of entrained water per unit length of the immersed pier, as described in Annex F.

4.1.3 Element stiffness

(1) For the estimation of element stiffnesses refer to Clause 2.3.6.1. In general, when using stiffness values corresponding to uncracked cross-sections, higher earthquake effects are obtained.

(2) For second order effects refer to Clauses 2.4 (5) and 5.4 (1). High second order effects may occur in bridges with slender piers and in special bridges like arch and cable - stayed bridges (see also Annex H).

4.1.4 Modelling of the soil

(1)P In general the supporting elements which transmit the seismic action from the soil to the bridge shall be assumed as fixed to the foundation soil (see 3.2.1 (2)a). However the soil-structure interaction effects may be considered according to Part 5, using appropriate impendances or appropriately defined rotational soil springs.

(2) Soil-structure interaction effects should be used when the displacement due to soil flexibility is greater than 30% of the total displacement at the centre of mass of the deck.

(3) In those cases in which it is difficult to estimate reliable values for the mechanical properties of the soil , the analysis should be carried out using the estimated probable highest and lowest values. High estimates of soil stiffness should be used for computing the internal forces and low estimates for computing displacements.

4.1.5 Torsional effects

(1)P Skew bridges (angle of intersection $\varphi<70^{o}$) and bridges with a ratio L/B < 2.0 tend to rotate about the vertical axis, despite a theoretical coincidence of the centre of mass with the centre of stiffness. (L is the total length of the continuous deck and B is the width of the deck).

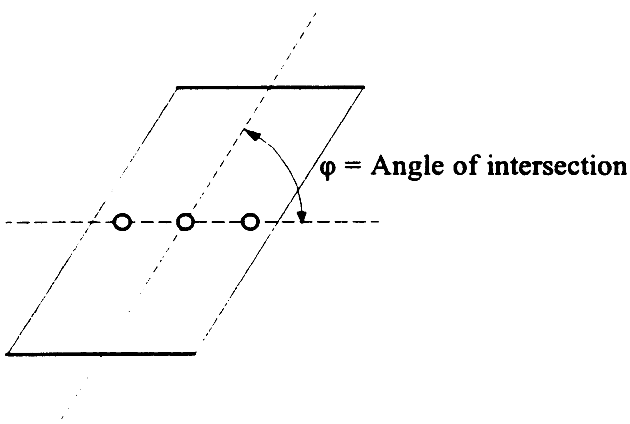


Figure 4.1
Skew bridge

(2) Highly skewed bridges ($\varphi<45^{o}$) should in general be avoided in high seismicity regions. If this is not possible, adequate modelling of the actual horizontal stiffness of the bearings must be effected, taking into account the concentration of the vertical reactions near the acute angles. Alternatively an increased accidental eccentricity may be used. A similar approach should also be adopted in cases of bridges with large horizontal curvature.

(3)P When using the Fundamental Mode Method (see Clause 4.2.2) for the design of such bridges, the following static torsional moment shall be considered to act about the vertical axis at the center of gravity of the deck:

$$M_t = \pm F\ e \tag{4.1}$$

where:
F is the horizontal force determined according to equ. (4.8)
$e = e_a + e_d$
e_a=0.03L or 0.03B is the accidental mass eccentricity, and
e_d=0.05L or 0.05B is an additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration; for the calculation of e_a and e_d the dimension L or B transverse to the direction of excitation shall be used.

(4) When using a Full Dynamic Model (space model) the dynamic part of the torsional excitation is taken into account if the centre of mass is displaced by the accidental eccentricity e_a in the most unfavorable direction and sense. However, the torsional effects may also be estimated using the static torsional moment of equ. (4.1).

(5)P The torsional resistance of a bridge structure shall not rely on the torsional rigidity of a single pier. In single span bridges the bearings shall be designed to resist the torsional effects.

4.1.6 Behaviour factors for linear analysis

(1)P The standard procedure of the present code is an equivalent linear dynamic analysis. Behaviour factors reflect the ductility of the structure i.e. the capability of the bridge piers to withstand, without failure, seismic actions in the post-elastic range.The available levels of ductility are defined in Clause 2.3.2.

Note
The linear analysis method, using an appropriate behaviour factor, is generally considered to be a reasonable compromise between the uncertainties intrinsic to the seismic problem and the relevant admissible errors on the one hand and the required effort for the analysis and design on the other.

(2)P The occurence of flexural plastic hinges in the ductile elements is an essential requirement for the application of the values of the q-factor defined in Table 4.1 for ductile behaviour.

This possibility is available only if the tensile yield strain is reached or exceeded at the critical sections of the ductile elements when determining the required strength under the bending moments and normal force of the design seismic load combination (see also Clause 2.3.2.1). In the case of reinforced concrete sections the yield strain of the tensile reinforcement must be reached or exceeded when determining the required reinforcement according to Clause 5.6.3.1.

(3)P Behaviour factors greater than 1.2 must not be used without special consideration, if the development of plastic hinges in the piers is not probable, because the piers do not reach yielding under the design seismic action. This is likely to occur when practically the entire seismic action is carried by one or two very stiff and strong elements (abutments or piers) which remain within the elastic range.

(4) In high seismicity areas when dissipation is concentrated at the abutments, the use of energy absorbing devices at the abutments is recommended.

(5) No plastic hinges will in general develop in piers which are flexibly connected to the deck in the direction under consideration .A similar situation will occur in individual piers having very low stiffness in comparison to the other piers (see Clauses 2.3.2.1 (6) and (7)).

(6)P The maximum values of the behaviour factor q which may be used for the two horizontal seismic components are given in Table 4.1, depending on the post-elastic behaviour of the ductile elements, where the main energy dissipation takes place. If a bridge has various types of ductile elements (which is not advisable) the lowest relevant q-factor shall be used. However, different q factors may be used in each horizontal direction.

Note

Use of behaviour factor values less than the maximum allowable given in Table 4.1 shall normally lead to reduced ductility demands, which in general implies a reduction of potential damage. Such a use is therefore at the discretion of the designer.

Table 4.1: Maximum values of the behaviour factor q

Ductile Elements	**Seismic Behaviour**	
	Limited Ductile	**Ductile**
Reinforced concrete piers		
Vertical piers in bending (a_s*≥ 3.5)	1.5	3.5
Squat piers ($a_s = 1.0$)	1.0	1.0
Inclined struts in bending	1.2	2.0
Steel Piers		
Vertical piers in bending	1.5	3.5
Inclined struts in bending	1.2	2.0
Piers with normal bracing	1.5	2.5
Piers with eccentric bracing	-	3.5
Abutments	1.0	1.0
Arches	1.2	2.0

* $a_s = H/L$ is the aspect ratio of the pier: For $1.0 < a_s < 3.5$ the q-factors may be obtained by linear interpolation

(7)P For reinforced concrete elements the values of q-factors given in Table 4.1 are valid when the normalized axial force η_K defined in Clause 5.3 (3) does not exceed 0.30. Bridges in which $0.30 < \eta_K < 0.6$ are considered as special and the relevant q-factors shall be reduced according to Annex H (Clause H.1).

(8)P The values of the q-factor for Ductile Behaviour given in Table 4.1 may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, the values of Table 4.1 shall be divided by 1.4; however final q-values less than 1.0 need not be used. When energy dissipation is intended to occur at plastic hinges located in piles, which are designed for ductile behaviour, and at points which are not accessible, a final q-value of 2.5 may be used for vertical piles and 1.5 for inclined piles.

(9)P The behaviour factor for the analysis in the vertical direction shall always be taken equal to 1.0.

(10)P When the entire design seismic action is resisted by elastomeric bearings the flexibility of the bearings imposes a practically elastic behaviour of the system, i.e. $q \cong 1.0$ (see Annex B, Note). Such bridges shall be designed according to Section 7.

(11)P For the estimation of the behaviour factor of special bridges refer also to Section 8.

4.1.7 Vertical component of the seismic action

(1) The effects of the vertical seismic component on the piers may, as a rule, be omitted in zones of low and moderate seismicity. In zones of high seismicity these effects need only be investigated in the exceptional cases when the piers are subjected to high bending stresses due to the permanent actions of the deck.

(2)P The effects of the vertical seismic component in the upward direction in prestressed concrete decks, shall be always investigated.

(3)P The effects of the vertical seismic component on bearings and links shall be assessed in all cases.

(4) The estimation of the effects of the vertical component may be carried out using the Fundamental Mode Method and the Flexible Deck Model (Clause 4.2.2.4).

4.2 Methods of analysis

4.2.1 Linear dynamic analysis - Response spectrum method

4.2.1.1 Definition, field of application

(1)P The Response Spectrum Analysis is an elastic analysis of the peak dynamic responses of all significant modes of the structure , using the ordinates of the site-dependent design spectrum (Clause 3.2.2.5). The overall response is obtained by statistical combination of the maximum modal contributions. Such an analysis may be applied in all cases in which a linear analysis is allowed.

(2)P The earthquake action effects shall be determined from an appropriate discrete linear model (Full Dynamic Model), idealized in accordance with the laws of mechanics and the principles of structural analysis, and compatible with an adequate idealization of the earthquake action.

(3)P For the following types of bridges, which are defined as special bridges, the recommendations of Section 8 shall also be considered:
cable-stayed bridges, arch bridges, bridges with inclined supporting struts, bridges with extreme geometry, and bridges with markedly differing yielding of piers, etc (see also Clause 1.1).

4.2.1.2 Significant modes

(1)P All modes having significant contribution to the total structural response shall be considered.

(2) For bridges in which the total mass can be considered as a sum of "effective modal masses", the above criterion is considered to be satisfied if the sum of the effective modal masses, for the modes considered, amounts at least to 90% of the total mass of the bridge.

4.2.1.3 Combination of modal responses

(1)P The probable maximum value E of an action effect (force, displacement etc.) , shall be taken in general equal to the square root of the sum of squares of the modal responses E_i (SRSS-rule)

$$E = \sqrt{\Sigma E_i^2} \qquad (4.2)$$

This action effect shall be assumed to act in both senses.

(2)P When two modes have closely spaced natural periods $T_j \leq T_i$, with the ratio $\rho = T_j/T_i$ exceeding the value $0.1/(0.1 + \xi')$, where ξ' is the viscous damping ratio (see (3) below), the SRSS rule becomes unconservative and more accurate rules must be applied.

(3) For the above case the method of the Complete Quadratic Combination (CQC - method) may be used:

$$E = \sqrt{\Sigma_i \Sigma_j E_i r_{ij} E_j} \tag{4.3}$$

with $i = 1 \ldots n$, $j = 1 \ldots n$

with the correlation factor

$$r_{ij} = \frac{8\xi'^2 (1+\rho)\rho^{2/3}}{(1-\rho^2)^2 + 4\xi'^2 \rho(1+\rho)^2} \tag{4.4}$$

where:
$\rho = T_j/T_i$ and
ξ' is the viscous damping ratio (not in %) = $\xi/100$

(4) When the differential displacement along the base of the bridge can induce substantial stresses in the structure, the value of the earthquake action effects can be determined in the case of application of the SRSS-rule as

$$E = \sqrt{\Sigma_i E_i^2 + \Sigma_m (k_m d_m)^2} \tag{4.5}$$

and in the case of aplication of the CQC-method as

$$E = \sqrt{\Sigma_i \Sigma_j E_i r_{ij} E_j + \Sigma_m (k_m d_m)^2} \tag{4.6}$$

where k_m is the effect of the m-th independent motion and d_m is the assymtotic value of the spectrum for the m-th motion for large periods, expressed in displacements.

4.2.1.4 Combination of the components of seismic action

(1)P The probable maximum action effect E, due to the simultaneous occurrence of seismic actions along the horizontal axes X, Y and the vertical axis Z, may be estimated from the maximum action effects E_x, E_y and E_z due to independent seismic action along each axis, as follows:

$$E = \sqrt{E_x^2 + E_y^2 + E_z^2} \tag{4.7}$$

(2) Alternatively it is sufficient to use as design seismic action A_{Ed} the most adverse of the following combinations:

$$\begin{aligned} &A_{Ex} \text{ "+" } 0.30A_{Ey} \text{ "+" } 0.30\,A_{Ez} \\ &0.30A_{Ex} \text{ "+" } A_{Ey} \text{ "+" } 0.30A_{Ez} \\ &0.30A_{Ex} \text{ "+" } 0.30A_{Ey} \text{ "+" } A_{Ez} \end{aligned} \tag{4.8}$$

where A_{Ex} , A_{Ey} and A_{Ez} are the seismic actions in each direction X, Y and Z respectively. A_{Ez} should be considered according to the requirements of Clause 4.1.7.

4.2.2 Fundamental mode method

4.2.2.1 Definition

(1)P Equivalent static seismic forces are derived from the inertia forces corresponding to the fundamental natural period of the structure in the direction under consideration, using the relevant ordinate of the design response spectrum. The method includes also simplifications regarding the shape of the first mode and the estimation of the fundamental period.

(2)P Depending on the particular characteristics of the bridge, this method can be applied using three different approaches for the model, namely:

- the Rigid Deck Model
- the Flexible Deck Model
- the Individual Pier Model

(3)P The rules of Clause 4.2.1.4 for the combination of the components of seismic action shall be applied.

4.2.2.2 Field of application

(1)P The method may be applied in all cases in which the dynamic behaviour of the structure can be sufficiently approximated by a single dynamic degree of freedom model. This condition is considered to be satisfied in the following cases:

(a) In the longitudinal direction of approximately straight bridges with continuous deck, when the seismic forces are carried by piers whose total effective mass is less than 1/5 of the mass of the deck.

(b) In the transverse direction of case (a) when the structural system is approximately symmetrical about the centre of the deck, i.e. when the theoretical eccentricity e_o between the centre of stiffness of the supporting elements and the centre of mass of the deck does not exceed 5% of the length of the deck (L).

(c) In the case of piers carrying simply supported spans when no significant interaction between piers is expected and the total effective mass of each pier is less than 1/5 of the mass of the part of the deck carried by the pier.

4.2.2.3 Rigid deck model

(1)P This model can be applied only when - under the earthquake action - the deformation of the deck in a horizontal plane is negligible compared to the displacements of the pier tops. This is always valid in the longitudinal direction of approximately straight bridges with continuous deck. In the transverse direction the deck may be assumed rigid if $L/B \leq 4.0$ or, in general, if the following condition is satisfied:

$$\frac{\Delta d}{d_a} \leq 0.20 \qquad (4.9)$$

where:

L is the total length of the continuous deck

B is the width of the deck and

Δd and d_a are respectively the maximum difference and the 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 distibution.

(2)P The earthquake effects shall be determined by applying at the deck a horizontal equivalent static force F given by the expression:

$$F = MS_d(T) \qquad (4.10)$$

where:

M is the total effective mass of the structure, equal to the mass of the deck plus the mass of the upper half of the piers.

$S_d(T)$ is the spectral acceleration of the design spectrum (Clause 3.2.2.5) corresponding to the fundamental period T of the bridge, estimated as:

$$T = 2\pi\sqrt{\frac{M}{K}} \qquad (4.11)$$

where $K=\Sigma K_i$ is the stiffness of the system, equal to the sum of the stiffnesses of the resisting elements.

(3)P In the transverse direction the force F shall be distributed along the deck proportionally to the distribution of the effective masses.

4.2.2.4 Flexible deck model

(1)P The Flexible Deck Model shall be used when the condition (4.7) is not satisfied.

(2) In the absence of a more rigorous calculation, the fundamental period of the structure, in the direction under examination, may be estimated by the Rayleigh method, using a generalized single degree of freedom system, as follows:

$$T = 2\pi\sqrt{\frac{\Sigma m_i d_i^2}{\Sigma f_i d_i}} \qquad (4.12)$$

where:

m_i is the mass concentrated at the i-th nodal point

d_i is the displacement in the direction under examination when the structure is acted upon by forces $f_i = m_i g$ acting at all nodal points in the same direction.

(3)P The earthquake effects shall be determined by applying at all nodal points horizontal forces F_i given by :

$$F_i = \frac{4\pi^2}{T^2}\,\frac{S_d(T)}{g}\,d_i m_i \tag{4.13}$$

where :

T is the period of the fundamental mode of vibration for the direction under consideration,

m_i is the mass concentrated at the i-th point,

d_i is the displacement of the i-th nodal point, approximating the shape of the first mode (it may be taken equal to the values determined in (2) above),

$S_d(T)$ is the spectral acceleration of the design spectrum (Clause 3.2.2.5), and

g is the acceleration of gravity.

4.2.2.5 Torsional effects in the transverse direction

(1)P When the Rigid or the Flexible Deck Model is used in the transverse direction of a bridge, torsional effects shall be estimated by applying a static torsional moment M_t according to equ (4.1) Clause 4.1.5 (3). The relevant eccentricity shall be estimated as follows:

$$e = e_o + e_a + e_d \tag{4.14}$$

where:

e_o is the theoretical eccentricity (see case (b) of Clause 4.2.2.2)

$e_a = 0.03L$ is the accidental eccentricity and

$e_d = 0.03L\sqrt{1 + e_o / e_a}$ is an approximation to the dynamic amplification.

(2)P The force F shall be determined either according to equation (4.8) or as ΣF_i according to equation (4.3). The moment M_t may be distributed to the supporting elements using the Rigid Deck Model.

4.2.2.6 Individual pier model

(1) In many cases, earthquake action in the transverse direction of bridges is resisted mainly by the piers, and no strong interaction between adjacent piers develops. In such cases the earthquake effects acting on the i-th pier may be approximated by considering the action of an equivalent static force.

$$F_i = M_i\, S_d(T_i) \tag{4.15}$$

where

M_i is the effective mass attributed to pier i and

$T_i = 2\pi\sqrt{\frac{M_i}{K_i}}$ is the fundamental period of the same pier.

(2) This simplification may be applied as a first approximation for preliminary analyses, when the following condition is met for all adjacent piers i and i+1

$$0.95 < T_i/T_{i+1} \leq 1.05 \qquad (4.16)$$

Otherwise a redistribution of the effective masses attributed to each pier, leading to the satisfaction of the above condition, is required.

4.2.3 Alternative linear methods

4.2.3.1 Power spectrum analysis

(1)P A linear stochastic analysis of the structure shall be performed , either by applying modal analysis or frequency dependent response matrices , using as input the acceleration power density spectrum (see Clause 3.2.2.3).

(2)P The elastic action effects shall be defined as the mean value of the probability distribution of the largest extreme value of the response during the duration considered in the earthquake model.

(3)P The design values shall be determined by dividing the elastic effects by the appropriate behaviour factor q.

(4)P The method has the same field of application as the Response Spectrum Analysis.

4.2.3.2 Time series analysis

(1)P In a time series analysis the effects of the earthquake action shall be identified with the sample average of the extreme response computed for each accelerogram in the sample. For the definition of time histories see Clause 3.2.2.4 and for details see Annex E.

4.2.3.3 Criteria for the applicability of the results

(1)P The criteria specified in Part 1.2, Clause 3.3.4.1 are applicable.

4.2.4 Non - linear time domain analysis

(1)P The structure's time dependent response can be obtained through direct numerical integration of its non-linear differential equations of motions. The input shall consist of ground motion time histories (accelerograms , see also Clause 3.2.2.4) developed for the specific site and representing preferably actual earthquakes.

(2)P Unless otherwise specified in this Part, this method can be used only in combination with a standard response spectrum analysis to provide insight in the post - elastic response and comparison between required and available local ductilities. With the exception of bridges on isolating devices (Section 7) and special bridges (Section 8), the results of the nonlinear analysis shall not be used to relax requirements resulting from the response spectrum analysis. The rules for the strength verification of sections (Clauses 5.6.3 and 5.7) and for the design of foundations (Clause 5.8), as well as the specific detailing rules aiming at a minimum local ductility must be adhered to.

5. STRENGTH VERIFICATION

5.1 General

(1)P The criteria of this clause are applicable to the earthquake resisting system of bridges. For bridges provided with Base isolating devices, Section 7 shall be applied.

5.2 Design strength

(1)P The material safety factors γ_M as defined in EC2, EC3 and EC4 for the fundamental load combinations shall also be used for strength verifications under seismic load combinations and capacity design effects.

Note
The material safety factors are repeated here for ease of reference

Reinforced Concrete	
Concrete	γ_C = [1.5]
Reinforcing steel	γ_S = [1.15]
Structural Steel	
	γ_{M0} = [1.1]
Plastic resistance of gross section	γ_{M1} = [1.1]
Resistance of net section at bolt holes	γ_{M2} = [1.25]
Bolts, rivets, pins, welds, slip	γ_M = [1.25]

5.3 Capacity design effects

(1)P For structures of ductile behaviour, capacity design effects (F_C) shall be calculated by analysing the intended plastic mechanism under the permanent actions and a level of seismic action at which all intended flexural hinges have developed bending moments equal to an appropriate upper fractile of their flexural resistance, called moment overstrength M_o.

(2)P The moment overstrength of a section shall be calculated as:

$$M_o = \gamma_o M_{Rd} \tag{5.1}$$

where:

γ_o is the overstrength factor

M_{Rd} is the design flexural strength of the section, in the selected direction and sense, based on the actual section geometry and reinforcement configuration and quantity (with γ_M values for fundamental load combinations). In determining M_{Rd} ,the interaction with the axial force and eventually with the bending moment in the other direction, both resulting from the combination of the permanent actions (gravity loads and prestressing) and the design seismic action in the same direction and sense, shall be considered.

(3)P The value of the overstrength factor shall be taken in general as:

$$\gamma_o = 0.7 + 0.2q \qquad (5.2)$$

where q is the relevant behaviour factor

In the case of reinforced concrete sections, with special confining reinforcement according to Clause 6.2.1, in which the value of the normalized axial force

$$\eta_k = N_{Ed}/(A_c f_{ck}) \qquad (5.3)$$

exceeds 0.1, the value of the overstrength factor shall be increased to:

$$\gamma_o = [1+2(\eta_k-0.1)^2]\,(0.7+0.2q) \qquad (5.4)$$

where:
N_{Ed} is the value of the axial force at the plastic hinge corresponding to the design seismic combination, positive if compressive
A_c is the area of the section and
f_{ck} is the characteristic concrete strength.

(4)P Within members containing plastic hinge(s), the capacity design bending moment M_C at the vicinity of the hinge (see Figure 5.1) shall not be assumed greater than the relevant design flexural resistance M_{Rd} of the hinge assessed according to Clause 5.6.3.1.

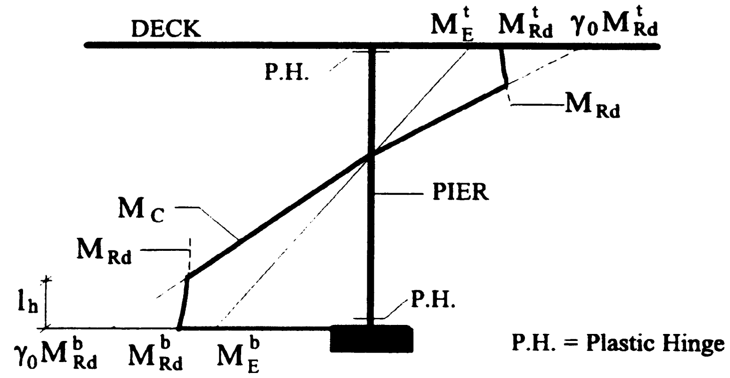


Figure 5.1
Capacity design moments within member containing plastic hinge(s)

Note:
The M_{Rd}-curves shown in Fig. 5.1 correspond to a pier with variable cross section (increasing downwards). In case of constant cross section M_{Rd} is also constant.

(5)P Capacity design effects shall be calculated in general for each sense of the seismic action in both the longitudinal and the transverse directions. A relevant procedure and simplifications are given in Annex G.

(6)P When sliding bearings participate in the plastic mechanism, their capacity shall be assumed equal to $\gamma_{of}R_{df}$,where

γ_{of} = 1.30 is a magnification factor for friction due to ageing effects and
R_{df} is the maximum design friction force of the bearing.

(7)P In the case of members where no plastic hinges are intended to form and which resist shear forces from elastomeric bearings, the capacity design effects shall be calculated on the basis of the maximum deformation of the elastomeric bearings corresponding to the design displacement of the deck. A 30% increase of the bearing stiffness shall be used in these cases. If plastic hinge formation is however intended in such a member the capacity design effects shall be governed by the overstrength flexural capacity of the hinge.

5.4 Second order effects

(1) In case of linear analysis, second order effects may be estimated using displacements

$$d_E = 0.5\,(1+q)d_{Ee} \qquad (5.5)$$

where q is the behaviour factor and d_{Ee} are the seismic displacements obtained from the first order elastic analysis.

5.5 Design seismic combination

(1)P The design value of action effects E_d, in the seismic design situation shall be derived from the following combination of actions:

$$G_k \text{ "+" } P_k \text{ "+" } A_{Ed} \text{ "+" } \psi_{21}Q_{1k} \qquad (5.6)$$

where:
G_k are the permanent loads with their characteristic values,
P_k is the characteristic value of prestressing after all losses,
A_{Ed} is the most unfavorable combination of the components of the earthquake action according to Clause 4.2.1.4,
Q_{1k} is the characteristic value of the traffic load, and
ψ_{21} is the combination factor according to Clause 4.1.2 (3).

(2)P Seismic action effects need not be combined with action effects due to imposed deformations (temperature variation, shrinkage, settlements of supports, ground residual movements due to seismic faulting).

(3)P An exception to the above rule is the case of bridges in which the seismic action is resisted by elastomeric laminated bearings (see also Clause 6.6.2.3(3)).
In this case elastic behaviour of the system must be assumed and the action effects due to imposed deformations must be accounted for. To this end the supporting elements (piers and/or abutments) carrying the elastomeric bearings shall be designed with capacity design effects determined according to Clause 5.3 (7). The design displacement of the deck shall be determined according to Clause 2.3.6.3 (2).

Note
It is noted that in the above case the displacement due to the creep does not normally induce additional stresses to the system and can therefore be omitted. Creep also reduces the effective value of long-term imposed deformations (e.g. shrinkage).

(4)P For wind and snow actions the value $\psi_{21}=0$ shall be assumed.

5.6 Resistance verification of concrete sections

5.6.1 Design effects

(1) When the resistance of a section depends significantly on the interaction of more than one action effects (e.g. bending moments and axial force) it is sufficient that the Ultimate Limit State conditions, given in the following clauses, are satisfied separately by the extreme (max. or min.) value of each action, taking into account the interaction with the coincidental accompanying values of the other actions.

5.6.2 Structures of limited ductile behaviour

(1)P

$$E_d \leq R_d \tag{5.7}$$

where:

E_d is the design action effect under the seismic load combination including second order effects, and

R_d is the design resistance of the section.

(2)P In regions of moderate to high seismicity ($a_g \geq [0.10g]$) the shear resistance of potential plastic hinges shall be verified according to 5.6.3.4.

5.6.3 Structures of ductile behaviour

5.6.3.1 Flexural resistance of sections of plastic hinges

(1)P

$$M_{Ed} \leq M_{Rd} \tag{5.8}$$

where:

M_{Ed} is the design moment under the seismic load combination, including second order effects, and

M_{Rd} is the design flexural resistance of the section, taking into account the interaction of the accompanying design effects (axial force and eventually the bending moment in the other direction).

(2)P The longitudinal reinforcement of the member containing the hinge shall remain constant and fully active at least over the length l_h indicated in Figure 5.1

5.6.3.2 Flexural resistance of sections outside the region of plastic hinges

(1)P $$M_C \leq M_{Rd} \qquad (5.9)$$

where :

M_C is the capacity design moment as defined in Clause 5.3, and

M_{Rd} is the design resistance of the section, taking into account the interaction of the corresponding design effects (axial force and eventually the bending moment in the other direction).

5.6.3.3 Shear resistance of elements outside the region of plastic hinges

(1)P Verification of web diagonal compression

$$V_C \leq V_{Rd2} \qquad (5.10)$$

(2)P Verification of shear reinforcement

$$V_C \leq V_{cd}+V_{wd} \qquad (5.11)$$

where:

V_C is the shear force resulting from capacity design as per Clause 5.3.

Design shear resistances shall be calculated according to EC2, Part 1 as follows:

V_{Rd2}	from formula 4.19
$V_{cd} = V_{Rd1}$	from formula 4.18
V_{wd}	from formula 4.23

Note

Formulae (4.19), (4.18) and (4.23) are repeated here, in concise form, to facilitate cross reference.

$$V_{Rd2} = 0.5 v f_{cd} b_w 0.9d \quad \text{with } v = 0.7\text{-}f_{cK}/200 \geq 0.5$$

$$V_{cd} = V_{Rd1} = [\tau_{Rd} k(1.2+40\rho_1)+0.15\sigma_{cp}]b_w d$$

where :

$\tau_{Rd} = 0.035 f_{cK}^{2/3}$

$k = 1.6\text{-}d \geq 1$

$\rho_1 = A_{sl}/(b_w d)>0.02$ is the ratio of longitudinal tension reinforcement,

$\sigma_{cp} = N_{Ed}/A_c$ is the average normal stress under the design seismic effects, and

d is the depth of the section in m

$$V_{wd} = (Asw/s)0.9d\ f_{ywd}$$

where:

Asw and s are the area and spacing of the stirrups respectively,

f_{ywd} is the design yield strength of the shear reinforcement, and

b_w is the width of the web of the section

5.6.3.4 Shear resistance of plastic hinges

(1)P Verification of diagonal compression

$$V_C < V_{Rde} \qquad (5.12)$$

where:
V_{Rde} is the shear resistance corresponding to the compressive concrete strength after degradation
$= 0.275 v f_{ck} b_{wc} d_c$
where:
$v = 0.7 - f_{ck}/200 \geq 0.5$ and
b_{wc}, d_c are the confined web width and depth of the section respectively

(2)P Verification of shear reinforcement

$$V_C \leq V_{cde} + V_{wd} \qquad (5.13)$$

where:
V_{cde} is the contribution of the concrete after degradation and is equal to:

$V_{cde} = 0.$ if $n_k \leq 0.1$
$V_{cde} = 2.5 \tau_{Rd} b_{wc} d_c$ for $\eta_k > 0.1$
with $\eta_k = N_{Ed}/(A_{cc} f_{ck})$
V_{wd} is the contribution of reinforcement calculated according to 5.6.3.3 (2),
N_C is the design axial force positive if compressive, and
A_{cc} is the confined (core) concrete area of the section.

(3) In circular sections the effective shear area $b_{wc} d_c$ may be assumed equal to the confined concrete area $\pi D_{sp}^2/4$, and d_c may be assumed equal to D_{sp}, where D_{sp} is the spiral diameter

(4)P Verification of sliding shear

$$V_C \leq A_v f_{yd} + \min N_{Ed} \qquad (5.14)$$

where
A_v is the total distributed longitudinal reinforcement with a design strength f_{yd}

(5) The above verification (5.14) is not applicable in squat wall type elements with a shear ratio $\alpha_s = M/Vd < 2.0$. For such cases, which are quite rare in bridges, the relevant provisions of EC8/Part 1.3 shall be applied.

5.7 Resistance verification for steel and composite elements

5.7.1 Piers

(1)P The resistance and ductility verification of steel and composite elements of piers shall be performed as specified in EC 8, Part 1.3, Clauses 3.2, 3.5, 3.6 and 3.7. For ductile structures cross sectional classes 1 or 2 shall be used. For limited ductile structures cross sectional classes 1, 2 or 3 may be used.

5.7.2 Bridge deck

(1)P For the resistance verification of bridge decks refer to Clause 6.1.

5.8 Foundations

5.8.1 General

(1)P Bridge foundation systems shall be designed to comply with the basic requirements set forth in Clause 5.1 of Part 5. More specifically, bridge foundations shall not be intentionally used as sources of hysteretic energy dissipation and therefore shall, as far as practicable, be designed to remain undamaged under the design seismic action.

(2)P The possible effect of soil structure interaction shall be assessed on the basis of the relevant provisions of Section 6 of Part 5. For special bridges non-linear analysis may be used for the assessment of the relevant non-linear effects.

5.8.2 Design action effects

(1)P For the purpose of resistance verification, the design action effects on the foundations shall be evaluated as follows:

(2)P Bridges of limited ductile behaviour
The design action effects shall be those obtained from the linear analysis of the structure, under the design seismic action defined in Clause 3.2.2.5, multiplied by the q-factor used.

(3)P Bridges of ductile behaviour
The design action effects shall be obtained by applying the capacity design procedure to the piers according to Clause 5.3.

5.8.3 Resistance verification

(1)P The resistance verification of the foundations elements shall be carried out in accordance with Clause 5.4.1 (Direct foundations) and Clause 5.4.2 (Piles and piers) of Part 5.

6. SPECIFIC DETAILING RULES

6.1 Scope

(1)P The rules of this clause are applicable to structures of ductile behaviour and aim at securing a minimum level of curvature/rotation ductility at the plastic hinges. Regarding materials, the provisions of Part 1.3 - Clause 2.2 for ductility class "H" for concrete elements and Clause 3.2 for structural steel elements - are applicable.

(2)P For structures of limited ductile behaviour recommendations for the detailing of critical sections are given in Clause 6.5. Materials for concrete elements shall conform to the provisions of Clause 2.2 of Part 1.3 for ductility class "L".

(3)P In general no formation of plastic hinges is allowed in the deck. Therefore there is no need for the application of specific detailing rules other than those valid for bridges under non-seismic permanent and variable actions (see Parts 2 of relevant ECs).

6.2 Concrete piers

6.2.1 Confinement

(1)P Ductile behaviour of the compression concrete zone shall be secured along potential plastic hinge regions.

(2)P In potential hinge regions where the normalized axial force (see 5.3.(3)) exceeds the following limit:

$$\eta_k = N_{Ed}/A_c f_{ck} > 0.08 \tag{6.1}$$

confinement of the compression zone according to 6.2.1.3 is in general necessary.

(3)P No confinement is required in piers with flanged sections (box- or I-Section) if, under ultimate seismic load conditions, a curvature ductility

$$\mu_c = [13] \tag{6.2}$$

is attainable with the maximum compressive strain in the concrete not exceeding the value of

$$\varepsilon_{cu} = 0.35\% \tag{6.3}$$

(4)P In cases of deep compression zones, the confinement may be limited to that depth in which the compressive strain exceeds $0.5\varepsilon_{cu}$

(5)P The quantity of confining reinforcement is defined by the mechanical reinforcement ratio:

$$\omega_{wd} = \rho_w \cdot f_{yd}/f_{cd} \tag{6.4}$$

where:
ρ_w is the transverse reinforcement ratio defined in 6.2.1.1 or 6.2.1.2.

6.2.1.1 Rectangular sections

(1)P The transverse reinforcement ratio is defined as:

$$\rho_w = A_{sw}/s.b \qquad (6.5)$$

where:
A_{sw} is the total area of hoops or ties in the one direction of confinement.
s is the spacing of hoops or ties in the longitudinal direction, subject to the following restrictions:

$s \leq 6$ longitudinal bar diameters
$s \leq 1/5$ of the smallest dimension of the concrete core.

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

(2)P The distance C between hoop legs or supplementary cross ties shall not exceed 1/3 of the smallest dimension b_{min} of the concrete core, nor 350mm. It need not be less than 200mm (see figure 6.1).

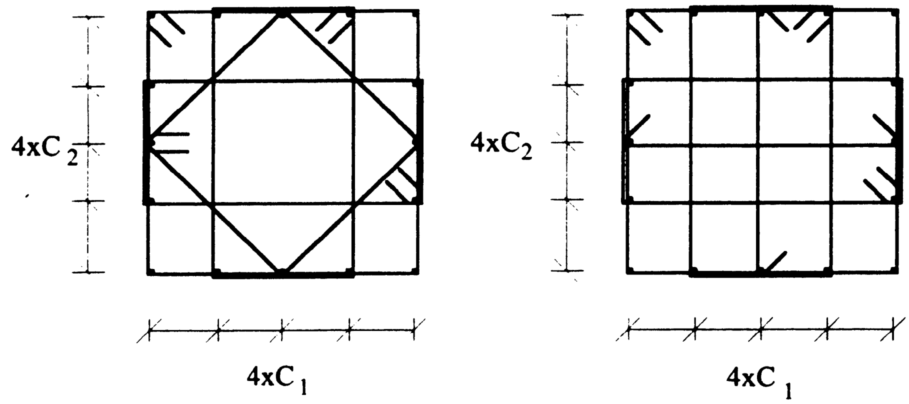
(3)P Bars inclined at an angle $\alpha > 0$ to the direction of confinement shall be assumed to contribute to the total area A_{sw} of equation (6.5) by their area multiplied by $\cos\alpha$.

6.2.1.2 Circular sections

(1)P The volumetric ratio ρ_w of the spiral reinforcement relative to the concrete core is used

$$\rho_w = 4A_{sp}/D_{sp}.s \qquad (6.6)$$

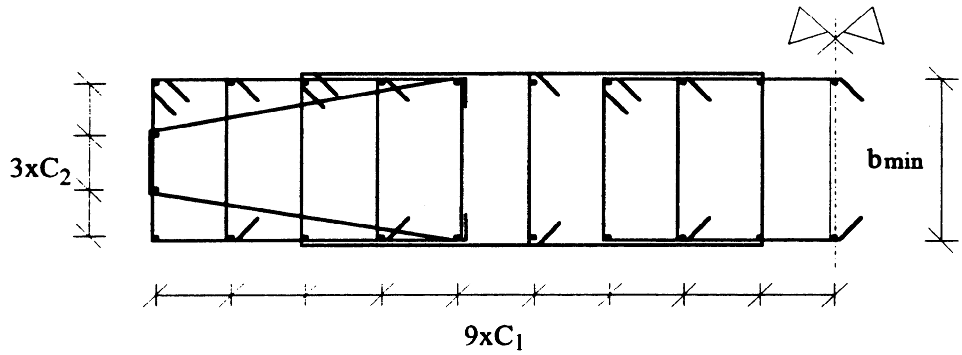
where:
A_{sp} is the area of the circumferential or spiral bar
D_{sp} is the diameter of the spiral of the circumferential or spiral bar
s is the spacing of these bars, subject to the following restrictions
$s \leq 6$ longitudinal bar diameters
$s \leq 1/5$ of the diameter of the concrete core.



4 overlapping hooks

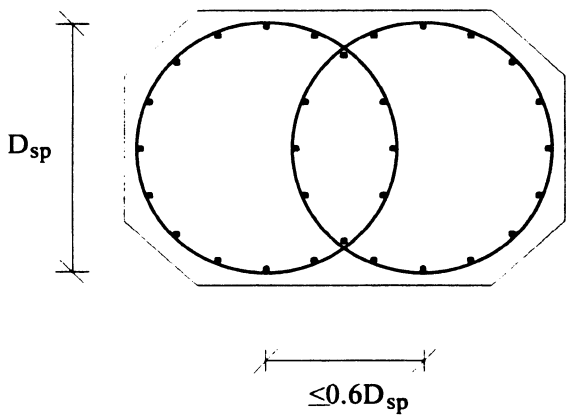
3 overlapping hooks +crossties

C_1, C_2 $\leq$ min(b/3 , 350mm)
need not be <200mm



Overlaping hooks + crossties (wall-type element)

Typical details using overlapping hooks and crossties



Typical detail using interlocking spirals

Figure 6.1
Typical details of confining reinforcement

6.2.1.3 Required confining reinforcement

1)P The minimum amount of confining reinforcement shall be determined as follows:

- for rectangular hoops and crossties

$$\omega_{wd,r} \geq 1.74 \frac{A_c}{A_{cc}} (0.009\mu_c + 0.17)\eta_k - 0.07 \geq \omega_{w,min} \qquad (6.7)$$

where:
A_c is the gross concrete area of the section,
A_{cc} is the confined (core) concrete area of the section, and
μ_c is the required curvature ductility

Depending on the intended seismic behaviour of the bridge the minimum values given in Table 6.1 are applicable

Table 6.1: Minimum values of μ_c and $\omega_{w,min}$

Seismic Behaviour	μ_c	$\omega_{w,min}$
Ductile	[13]	[0.12]
Limited ductile	[7]	[0.08]

- for circular hoops (spirals)

$$\omega_{wd,c} = 1.40\omega_{wd,r} \qquad (6.8)$$

(2)P When rectangular hoops and crossties are used, the minimum reinforcement condition shall be satisfied in both, transverse directions.

(3)P Interlocking spirals are quite efficient for confining approximately rectangular sections. The distance between the centres of interlocking spirals shall not exceed $0.6D_{sp}$, where D_{sp} is the diameter of the spiral (see figure 6.1).

6.2.1.4 Extent of confinement - Length of potential plastic hinges

(1)P When $\eta_k = N_{Ed}/A_c f_{ck} \leq 0.3$ the design length L_h of potential plastic hinges shall be estimated as the largest of the following:

- depth of pier section perpendicular to the axis of the hinge
- distance from the point of max. moment to the point where the moment is reduced by 20%.

(2)P When $0.6 \geq \eta_k > 0.3$ the design length of the potential plastic hinges as determined above shall be increased by 50%.

(3) The design length of plastic hinges (L_h) defined above is intended to be used exclusively for detailing the reinforcement of the hinge. It must not be used for estimating the rotation of the hinge.

(4)P When confining reinforcement is required the amount specified in Clause 6.2.1.3 shall be provided over the entire length of the plastic hinge. Outside the length of the hinge the transverse reinforcement shall be gradually reduced to the amount required by other criteria. The amount of transverse reinforcement provided over an additional length L_h adjacent to the theoretical end of the plastic hinge shall not be less than 50% of the amount of the confining reinforcement.

6.2.2 Buckling of longitudinal compression reinforcement

(1)P Buckling of longitudinal reinforcement must be avoided along potential hinge areas even after several cycles into the plastic region. Therefore all main longitudinal bars shall be restrained against outward buckling by transverse reinforcement (hoops or crossties) perpendicular to the longitudinal bars at a max. spacing of $6d_{sl}$, where d_{sl} is the diameter of the longitudinal bar.

(2)P Along straight section boundaries the restraining transverse ties shall be anchored at distances not exceeding 20cm, by legs extending to the interior of the element.

(3)P The minimum amount of transverse ties shall be determined as follows:

$$A_t / s = \Sigma A_s f_{ys} / 1.6 f_{yt} \quad (\mathrm{mm^2/m}) \qquad (6.9)$$

where:
A_t is the area of one tie leg, in mm^2,
s is the distance between tie legs, in m,
ΣA_s is the sum of the areas of the longitudinal bars restrained by the tie in mm^2,
f_{yt} is the yield strength of the tie, and
f_{ys} is the yield strength of the longitudinal reinforcement.

6.2.3 Other rules

(1)P Due to the potential loss of concrete cover in the plastic hinge region, the anchorage of the confining reinforcement shall be effected through 135^o hooks surrounding a longitudinal bar plus adequate extension (min. 10 diameters) into the core concrete.

(2)P Similar anchoring or full strength lap weld is required for the lapping of spirals within potential plastic hinge regions.

(3)P No splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region.

6.3 Steel piers

(1)P For ductile structures the detailing rules of Clause 3.5.3 of Part 1.3 shall be applied.

6.4 Foundations

6.4.1 Direct foundation structures

(1)P All direct foundation structures such as footings, rafts, box-type caissons, piers etc., are not allowed to enter the plastic range under the design seismic action, and hence they do not require special detailing reinforcement.

6.4.2 Pile foundations

(1)P In the case of pile foundations, it is sometimes difficult, if not altogether impossible, to avoid localized hinging in the piles. Pile integrity and ductile behaviour must be secured in such cases.

(2)P The potential hinge locations are:

a. near the pile heads, at the junction with the foundation mat, when the upper soil layers have poor mechanical characteristics;
b. at the interfaces of soil layers having markedly different shear deformability.

(3)P For the location (a) confining reinforcement of the amount specified in Clause 6.2.1.3 along a vertical length equal to 5 pile diameters, shall be provided.

(4)P For the locations (b), unless a more rigorous analysis is performed, longitudinal as well as confining reinforcement of the same amount as that required at the pile head shall be provided for a length of two pile diameters on each side of the interface.

6.5 Structures of limited ductile behaviour

(1)P For structures of limited ductile behaviour designed with $q \leq 1.5$ and located in areas of moderate to high seismicity ($a_g > [0.10g]$), the following rules are applicable to the critical sections, aiming at securing a minimum of limited ductility.

(2) The application of the same rules for structures located in low seismicity areas is recommended.

(3)P A section is considered to be critical, i.e. location of a potential plastic hinge when:

$$M_{Rd} / M_{Ed} < 1.30 \qquad (6.10)$$

where:
M_{Ed} is the maximum design moment under the seismic action combinations and
M_{Rd} is the minimum flexural resistance of the section under the same combination.

(4) As far as possible the location of potential plastic hinges should be accessible for inspection.

(5)P Where according to 6.2.1 (3), confinement is necessary for attaining a minimum curvature ductility μ_c= [7], confining reinforcement as required by Clause 6.2.1.3 for μ_c = [7], shall be provided. In such a case it is also required to secure the longitudinal reinforcement against buckling, according to the rules of Clause 6.2.2.

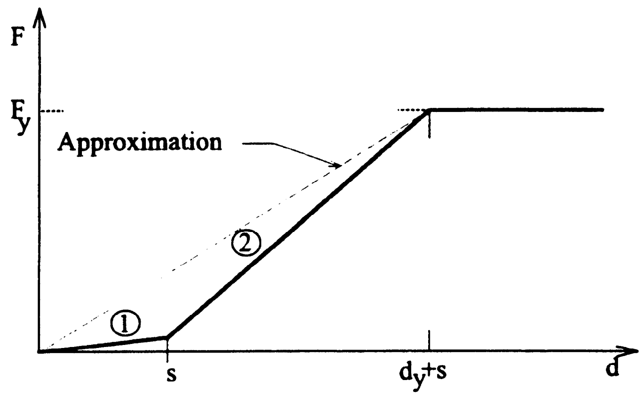
6.6 Bearings and seismic links

6.6.1 General requirements

(1)P Non-seismic horizontal actions on the deck shall be transmitted to the supporting elements (abutments or piers) through the structural connections which may be monolithic, or through bearings. For non-seismic actions the bearings shall be checked according to the relevant codes and standards (Parts 2 of relevant ECs and EN 1337).

(2)P The design seismic action shall in general be transmitted through the bearings. Seismic links, provided with proper slack so as to allow the non-seismic displacements of the bridge, may be used to transmit the entire design seismic action provided that potential shock effects are adequately prevented. In the latter case the analysis for the seismic action shall be based on an appropriate model taking into account a linear approximation of the force-displacement relationship of the linked structure (see Fig. 6.2).

Note
This function of seismic links may not be applicable to bridges subject to high horizontal non-seismic actions or to special displacement limitations as for instance in railway bridges.



s : Slack of the link
dy : Yield deflection of supporting element
① : Stiffness of bearing
② : Stiffness of supporting element

Figure 6.2
Force-displacement relationship for linked structure

(3)P The structural integrity of the bridge must be secured under extreme seismic displacements. This requirement shall be implemented at fixed supports through capacity design of the normal bearings or through provision of additional links as a second line of defense. At moveable connections either adequate overlap (seat) lengths according to Clause 6.6.4 shall be provided or seismic links shall be used.

(4)P All types of bearings shall be replaceable without major difficulty.

6.6.2 Bearings

6.6.2.1 Fixed bearings

(1)P The design seismic actions on fixed bearings shall be determined as capacity design effects; however they need not exceed those corresponding to $q=1$.

(2) Fixed bearings may be designed solely for the effects of the design seismic combinations, provided that they can be replaced without major difficulties and that seismic links are provided as a second line of defense.

6.6.2.2 Moveable bearings

(1)P Moveable bearings shall accomodate without damage the total design seismic displacement determined according to Clause 2.3.6.3 (2).

6.6.2.3 Elastomeric bearings

(1)P Elastomeric bearings may be used in the following arrangements:

a. On individual supports, to accommodate imposed deformations and resist only non-seismic horizontal actions while the resistance to the design seismic action is provided by structural connections (monolithic or through fixed bearings) of the deck to other supporting elements (piers or abutments).
b. On all or on individual supports, with the same function as in (a) above, combined with seismic links which are designed to resist the seismic action.
c. On all supports, to resist both the non-seismic and the seismic actions.

(2)P Elastomeric bearings used in arrangements (a) and (b) shall be designed to resist the maximum shear deformation corresponding to the design seismic action.

(3)P The seismic behaviour of bridges, in which the seismic action is resisted entirely by elastomeric bearings on all supports (arrangement (c) above), is governed by the large flexibility of the bearings. Such bridges and the bearings shall be designed in accordance with Section 7.

6.6.3 Seismic links

(1)P Seismic links may consist of shear key arrangements, buffers, dampers and/or linkage bolts or cables. Friction connections are not considered as positive linkage.

(2)P Seismic links are required in the following cases:

a. In combination with elastomeric bearings, if necessary, in order to carry the design seismic action.
b. In combination with fixed bearings which are not designed with capacity design effects.
c. In combination with normal elastomeric laminated bearings which are designed to carry the design seismic action (see Clause 7.6.3).
d. Between the deck and abutment or pier, at moveable end-supports, in the longitudinal direction, when the requirements for minimum overlap length according to Clause 6.6.4, are not satisfied.
e. Between adjacent parts of the deck at intermediate separation joints (located within the span).

(3)P The design actions for the seismic links of the previous paragraph shall be determined as follows:

- In cases (a) and (b) as capacity design effects (the horizontal resistance of the bearings shall be assumed zero).

- In cases (c), (d) and (e), in the absence of a rational analysis taking into account the dynamic interaction of the deck(s) and the supporting elements, the linkage elements may be designed for an action equal to αQ where $\alpha = a_g/g$, with a_g the design ground acceleration, and Q the weight of the section of the deck linked to a pier or abutment, or in case of two deck sections linked together, the least or the two weights.

(4)P The links shall be provided with adequate slack or margins so as to remain inactive:

- under the design seismic action in cases (b), (c), (d) and (e)
- under non-seismic actions in case (a).

(5) When using seismic links, means for reducing shock effects should be provided.

(6)P Holding down devices shall be provided at all supports where the total vertical design seismic reaction opposes and exceeds the following percentages of the permanent load compressive reactions:

- 80% in structures of ductile behavior where the vertical design seismic reaction is determined as a capacity design effect with all plastic hinges having developed their overstrength capacities.

- 50% in structures of non-ductile behavior where the vertical design seismic reaction is determined from the analysis under the design seismic action only (including the contribution of the vertical seismic component).

(7)P The above requirements (6) refer to the total vertical reaction of the deck on a support and are not applicable to individual bearings. However, no lift-off of individual bearings shall take place under the design seismic action.

6.6.4 Minimum overlap lengths

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

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

(3) In the absence of a more accurate estimation the minimum overlap length l_{ov} may be estimated as follows:

$$l_{ov} = l_m + d_{eg} + d_{es} \tag{6.11}$$

$$d_{eg} = Lv_g/c_P \leq 2d_g \tag{6.12}$$

where:

l_m is the minimum support length securing the safe transmission of the vertical reaction, but no less than [40cm],

d_{eg} is the effective displacement of the two parts due to differential seismic ground displacement,

d_g is the design value of the peak ground displacement as defined by Clause 4.2.3 of EC8:Part 1.1 and given in Table 6.2,

v_g is the peak ground velocity estimated from the design ground acceleration $a_g = \alpha g$, using the values of the ratio v_g/a_g given in Table 6.2, and

c_P is a low estimate of the compression-wave propagation velocity in the soil. In the absense of more accurate data the typical values given in Table 6.3 may be used.

Table 6.2: Parameters of ground motion

Soil Class	A	B	C
Ratio v_g/a_g (sec.)	[0.090]	[0.135]	[0.160]
Ratio d_g/a_g (sec^2)	[0.06]	[0.09]	[0.108]

Table 6.3: Low estimates of compression-wave propagation velocity

Soil Type	c_P(m/sec)
Rock	[1000]
Dense granular soil or stiff overconsolidated clays	[500]
Medium density (silty) sands	[300]
Medium to soft clays	[150]

L is the effective length of the deck (or deck section) estimated as follows:

- For decks connected to piers either fully or through seismic links, L is the distance between the support and the next pier linked to the deck. If a group of piers is linked to the deck, L is the distance between the support and the centre of the group.
- For decks (or deck sections) connected to abutments at the other end through seismic links, L is the total length of the deck (or deck section).

d_{es} is the effective seismic displacement of the support due to the deformation of the structure, estimated as follows:

- For decks connected to piers either monolithically or through fixed bearings, acting as full seismic links, $d_{es} = d_{Ed}$, where d_{Ed} is the total longitudinal design seismic displacement determined according to equation (2.7), Clause 2.3.6.3.
- For decks connected to piers or to an abutment through seismic links with slack equal to s:

$$d_{es} = d_{Ed} + s \qquad (6.13)$$

(4) In the case of an intermediate separation joint between two sections of the deck or of an end-support of a deck section on an intermediate pier, l_{ov} should be estimated by taking the square root of the sum of the squares of the values calculated for each of the two sections in the first case, or the section and the pier in the second.

6.7 Concrete abutments and retaining walls

6.7.1 General requirements

(1)P All structural components of the abutments shall be designed to remain elastic under the design seismic action. The design of the foundation shall be in accordance with Clause 5.8. Depending on the structural function of the horizontal connection between abutment and deck the following rules are applicable.

6.7.2 Abutments flexibly connected to the deck

(1)P In this type of abutment the deck is supported through moveable or elastomeric bearings. Neither the elastomeric bearings nor the eventual seismic links are designed to carry the design seismic action. Such abutments have a negligible contribution in the horizontal support of the deck.

(2)P The following actions, assumed to act in phase, shall be considered for the seismic design of these abutments:

a. Earth pressures including seismic effects determined according to Section 7 of EC8 Part 5.
b. Inertia forces acting on the mass of the abutment and on the mass of earthfill eventually carried by the abutment. In general these effects may be determined using the design ground acceleration a_g.

c. Actions from the bearings determined as capacity design effects according to Clauses 5.3 (5) and 5.3 (6).

(3)P When the earth pressures assumed in (a) above are determined according to EC8 Part 5, on the basis of an acceptable displacement of the abutment, adequate provision for this displacement shall be made in determining the gap between the deck and the abutment backwall. In this case it must also be insured that the displacement assumed in determining actions (a) can actually take place before an eventual failure of the abutment itself occurs. For this reason the design of the body of the abutment shall be effected considering the seismic part of actions (a) increased by 30%.

6.7.3 Abutments rigidly connected to the deck

(1)P The connection of the abutment to the deck is either monolithic or through fixed bearings or through links designed to carry the seismic action. Such abutments have a major contribution in the seismic resistance of the deck, both in the longitudinal and in the transverse direction.

(2)P The model to be used for the analysis of the bridge must incorporate, in an appropriate way, the soil interaction at the abutments using realistic values for the relevant stiffness parameters.

(3) When the seismic resistance of the bridge is secured by the contribution of other supporting elements as well (piers), the use of high and low estimates for the soil parameters is recommended in order to arrive at results which are on the safe side both for the abutments and for the piers.

(4)P A behaviour factor q=1.0 shall be used, in the analysis of the bridge.

(5)P The following actions shall be taken into account in the longitudinal direction:

a. Inertia forces acting on the mass of the structure which may be estimated using the Fundamental Mode Method (Clause 4.2.2.)
b. Static earth pressures acting on both abutments (E_o)
c. The additional seismic earth pressures

$$\Delta E_d = E_d - E_o \qquad (6.14)$$

where:
E_d is the total earth pressure acting on the abutment under seismic conditions. The pressures ΔE_d are assumed to act in the same direction on both abutments.

(6)P The deck to abutment connection (including eventual fixed bearings or links) shall be designed, in this case, for the action effects resulting from the above paragraphs.

Note
No capacity design is required in this case because, as a rule, the strength of the abutments is intrinsically too high for that. On the other hand the assumption of q=1.0 is conservative enough.

(7) In order that damage of the soil or the embankment behind the abutments is kept within acceptable limits the design seismic displacement should not exceed [6]cm.

(8) The soil reaction activated by the movement of the abutment and of wingwalls monolithically connected to the abutment is assumed to act on the following surfaces:

- In the logitudinal direction, on the external face of the backwall of that abutment which moves against the soil or fill.
- In the transverse direction, on the internal face of those wingwalls which move against the fill.

These abutment elements should be designed to resist this soil reaction, in addition to the static earth pressures.

6.7.4 Retaining walls

(1)P Free standing retaining walls shall be designed according to the rules of Clauses 6.7.2 (2) and (3).

7. BRIDGES WITH ISOLATING DEVICES

7.1 Scope

Bridge structures provided with isolating devices intended to reduce the seismic response shall be designed to comply with the provisions of this Section.

7.2 Design seismic action

7.2.1 Design spectra

(1)P Properly substantiated site-specific spectra with 5% damping are required for the design of bridges with a fundamental period of the isolated structure (effective period) T_I greater than 3.0 seconds , or located on a soil type C, or located within 15 km of an active fault. Such spectra shall not be taken less than the normal elastic site dependent spectra defined by Clause 3.2.2, which may be used for the design of all other bridges.

7.2.2 Time-history representation

(1)P Pairs of horizontal ground motion time-history components shall be selected from at least three recorded events. For each scaled pair of horizontal components the square root of the sum of squares (SRSS) of the 5%-damped spectrum of the components shall not fall bellow 1.3 times the 5%-damped spectrum of the design earthquake by more than 10% in the period range from T_I-1.0 to T_I+2.0 seconds.

(2)P The duration of time-histories shall be consistent with the magnitude and source characteristics of the design earthquake. Time-histories developed for sites within 15km of a major active fault shall incorporate near-fault phenomena.

7.3 Methods of analysis

7.3.1 Full or partial isolation

(1)P Full isolation of a bridge is achieved if under the design seismic action the structure of the bridge - with the possible exception of the isolation system itself - remains within the elastic range. In the opposite case the bridge is considered as partially isolated. In this case the deck shall remain within the elastic range and the post-elastic response shall be restricted to the piers.

(2)P In the case of full isolation equivalent linear response spectrum analysis (Fundamental or Multimode Analysis), using the effective secant stiffness of the isolation system at the design displacement may be applied under the additional conditions mentioned in Clauses 7.3.2 and 7.3.3 below. In the case of partial isolation non-linear time-history analysis shall be used.

7.3.2 Fundamental mode method

(1)P This method may be used when designing for full isolation if all the following criteria are met :

a. General criteria

a1. The distance of the bridge site from the nearest active fault is greater than 15 km.
a2. The soil category is A or B.
a3. The effective period T_I does not exceed 3.0 seconds and is at least three times the elastic fixed base period of the structure.
a4. The criteria of Clause 4.2.2.2 are met.

b. Criteria regarding the isolation system

b1. The effective stiffness of the isolating system (see Clause J.7) at the design displacement (d_{Ed}) is at least 50% of the effective stiffness at $0.2d_{Ed}$.
b2. The effective damping of the isolation system (see Clause J.9.2) does not exceed 10%.
b3. The force-displacement characteristics of the isolation system are not dependent on the rate of loading and on the vertical and bidirectional loads.
b4. The isolation system produces a restoring force such that its increase between $0.5d_{Ed}$ and d_{Ed} is at least 0.025 of the total gravity load above the isolators.

7.3.3 Response spectrum analysis

This method may be used when designing for full isolation if criterion (a2) and all criteria (b) of the previous clause are met.

7.3.4 Procedure of the response spectrum analysis

(1)P Multimode and fundamental mode response spectrum analysis shall be performed using the effective damping ξ_I (in %) of the isolation system, at an amplitude equal to the design displacement, in the period range of $T \geq 0.8T_I$. In the period range $T < 0.8T_I$, 5% damping shall be used.

(2)P The modification factor η_I for multiplication of the spectral values for $\xi_I \neq 5\%$ shall be taken as:

$$\eta_I = (7.0/(2+\xi_I))^{0.35} \qquad (7.1)$$

(3)P The combination of the two horizontal components of the seismic action shall follow the rules of Clause 4.2.1.4.

(4)P Torsional effects according to Clauses 4.1.5 and 4.2.2.5 shall be considered.

7.3.5 Time-history analysis

(1)P Time-history analysis may be used in all cases.

(2)P Each pair of time-histories shall be applied simultaneously. The maximum response resulting from all analyses for the parameter of interest shall be used for design.

7.4 Modelling

7.4.1 Isolation system

(1)P When the fundamenal mode method or the response spectrum analysis is used the isolation system shall be modeled using the minimum effective secant stiffness (K_{min}) at the maximum design displacement. This stiffness will be derived using the deformation characteristics developed and verified by tests according to Annex J.

(2)P When the response spectrum analysis or time-history analysis is used, the model of the isolation system must be able to appropriately:

- Account for the spatial distribution of isolator units
- Activate displacement in both horizontal directions and rotation about the vertical axis.
- Assess overturning forces on individual isolator units.
- Account for the effects of vertical load, bilateral load and/or the rate of loading if the force deflection properties of the isolation system are strongly dependent on one or more of these attributes.

7.4.2 Bridge structure

(1)P When the fundamental mode method or the response spectrum analysis is used the linear model of the structure above and below the isolation interface shall reflect the actual distribution of the stiffness. For reinforced concrete piers and abutments the stiffness of the uncracked sections may be used.

(2)P When non-linear time history analysis of a partially isolated structure is performed, the deformation characteristics of the yielding elements (piers) shall adequately approximate their actual post-elastic behavior.

7.5 Verification

7.5.1 Bridge structure

7.5.1.1 Full isolation

(1)P The design seismic action effects (E_{Id}) on the isolated structure, above and below the isolation interface, shall be determined as:

$$E_{Id} = \gamma_{oI} E_I \tag{7.2}$$

where:
E_I are the action effects resulting from the analysis of the isolated structure and
$\gamma_{oI} = 1.10\, K_{max}/K_{min}$ is an overstrength factor depending on the ratio of the maximum to the minimum effective stiffness of the isolation system at the design displacement.

7.5.1.2 Partial isolation

(1)P The flexural strength of the sections of the intended plastic hinges shall be verified using design action effects determined from equation (7.2) with $\gamma_{oI} = K_{max}/K_{min}$

(2)P All other design of sections shall be effected according to Clauses 5.6.3.2, 5.6.3.3 and 5.6.3.4, using capacity design effects based on the overstrength of the intended plastic hinges, according to Clause 5.3.

(3)P The detailing rules for ductility in the regions of plastic hinges, according to Clause 6.2, shall be applied.

(4)P Ductility demands in terms of curvature or rotational ductility exceeding the value of 13 shall be specifically checked against the available ductilities.

7.5.2 Isolation system

7.5.2.1 General requirements

(1)P All mechanical devices used to reduce the seismic response, with the exception of elastomeric bearings used in combination with seismic links according to Clause 7.6.3, shall comply with the following requirements and shall be tested and validated according to the provisions of Annex J.

(2)P Increased reliability is required for the isolating devices. This shall be effected by applying a magnification factor γ_x on the seismic displacement, as defined in Clause 7.5.2.2.

(3)P Access for inspection and replacement of all components of the isolation system must be provided. A regular inspection and maintenance program must be established and applied. This program shall be approved by the competent National Authorities.

(4)P Criterion (b4) of Clause 7.3.2 must be met by all isolation systems.

7.5.2.2 Design displacement for isolating devices

(1)P The design displacement at the isolation interface (d_{Ex}) shall be estimated from the maximum displacement resulting from the seismic analysis (d_E) as follows:

$$d_{Ex} = \gamma_x \, d_E \tag{7.3}$$

with the factor γ_x calculated as follows:

$$\gamma_x = \frac{0.8}{\sqrt{\alpha}} \tag{7.4}$$

however $$1.25 \leq \gamma_x \leq 3.00 \tag{7.5}$$

where $\alpha = a_g/g$ and a_g is the design ground acceleration.

7.5.2.3 Total design displacement

(1)P The total design displacement (d_{Exd}) under seismic conditions shall be estimated as follows:

$$d_{Exd} = d_{Ex} + d_G + d_{Ts} \qquad (7.6)$$

where:
d_G is the displacement due to permanent or quasi permanent actions (e.g. post-tensioning, shrinkage and creep for concrete decks), and
d_{Ts} is the combination value of the thermal displacement according to Clause 2.3.6.3

7.5.2.4 Vertical load stability

(1)P Each isolator unit shall be verified to be stable, with a safety factor of at least one, at the total design displacement d_{td} , under the following load combination:

$$G_k \text{"}\pm\text{"} \gamma_x A_{Ed} \qquad (7.7)$$

where:
G_k is the dead load of the bridge
A_{Ed} is the most unfavorable combination of the components of the design seismic action according to Clause 4.2.1.4 (2).

7.5.2.5 Overturning

(1)P The factor of safety against overturning at the isolation interface of each support under the load combination of Clause 7.5.2.4 shall not be less than 1.0. Local lift-off of individual elements is permitted provided that it does not cause instability of the isolator units or overstressing of other elements.

7.5.2.6 Clearances

(1)P Adequate clearances shall be provided to accommodate the total design displacement according to Clause 7.5.2.3.

7.6 Elastomeric bearings

7.6.1 General requirements

(1)P Elastomeric bearings utilized in implementing seismic isolation of bridges shall be designed for seismic action, according to the Clauses that follow. For non-seismic actions the bearings shall be checked according to the relevant codes and standards (Parts 2 of ECs).

(2)P The bearings must be laminated i.e. reinforced with integrally bonded steel plates.

(3)P Special elastomeric bearings
Elastomeric bearings conforming to the prototype tests of Annex J for seismic isolation systems are designated in this code as Special Elastomeric Bearings. For such bearings the effective damping ξ_I and the secant shear modulus G at the design seismic displacement shall be determined from these tests.

(4) Normal elastomeric bearings
Normal laminated bearings with Shore A hardness 60 ± 5, may be used for the seismic isolation of bridges, if, in the absence of special tests, the following values are used in the design:

$\xi_I = 5.0\%$
$G = 1.2\ N/mm^2$ for $\varepsilon_s \leq 1.2$ (see Clause 7.6.2.3)
$G = 1.6\ N/mm^2$ for $\varepsilon_s = 2.0$
$\varepsilon_{bu} = 4.0$ (see Clause 7.6.3.1)

<u>Note</u>
It is pointed out that bridges isolated through normal elastomeric bearings have a practically elastic response to earthquakes. Therefore a reduction of the design forces can be obtained only through the shift of the fundamental period caused by the high flexibility of the bearings. The following important consequences must be taken into account:

- The reduction of seismic forces is accompanied by a very significant increase of seismic displacements.
- The isolation is less effective when the predominant period of the design seismic motion is high (period T_C of the design response spectrum, see 3.2.2.2.1 (5)) i.e. for soil types B and C.

(5)P Adequate clearances shall be provided in accordance with Clause 2.3.6.3.

7.6.2 Total design shear strain

7.6.2.1 Components of shear strain

(1)P The total design shear strain (ε_{td}) shall be determined as the sum of the following components:

$$\varepsilon_{td} = \varepsilon_c + \varepsilon_s + \varepsilon_\alpha \qquad (7.8)$$

where:
ε_c is the shear strain due to compression
ε_s is the shear strain due to the total design seismic displacement according to Clause 2.3.6.3 and
ε_α is the shear strain due to angular rotation

7.6.2.2 Shear strain due to compression

(1)P The shear strain due to compression shall be determined as follows:

$$\varepsilon_c = \frac{1.5}{S}\,\frac{\sigma_e}{G} \tag{7.9}$$

where:
G is the shear modulus of the elastomer
σ_e is the maximum effective normal stress of the bearing calculated as:

$$\sigma_e = \frac{N_{Sd}}{A_r} \tag{7.10}$$

where:
N_{Sd} is the maximum axial force on the bearings resulting from the design seismic load combination
A_r is the minimum reduced effective area of the bearing calculated as follows:

- for rectangular bearings with steel plate dimensions b_x and b_y (without holes)

$$A_r = (b_x - d_{Edx})\,(b_y - d_{Edy}) \tag{7.11}$$

- for circular bearings with steel plate of diameter D

$$A_r = (\delta - \sin\delta)\,\frac{D^2}{4} \tag{7.12}$$

with

$$\delta = 2\arccos(d_{Ed}\,/\,D) \quad \text{and} \quad d_{Ed} = \sqrt{d_{Edx}^2 + d_{Edy}^2}$$

In the above equations d_{Edx} and d_{Edy} are the total relative displacements under seismic conditions, in the x- and y- directions respectively, of the two bearing faces, including the design seismic displacements (with torsional effects) as well as the displacements due to the imposed deformations of the deck (i.e. shrinkage and creep where applicable and 50% of the design thermal effects).

S is the shape factor of the relevant elastomer layer, defined as the ratio of the effective compressed area divided by the side area free to bulge i.e.

- for rectangular bearings: $S = \dfrac{b_x\,b_y}{2(b_x+b_y)t_i}$ (7.13)

- for circular bearings: $S = \dfrac{D}{4t_i}$ (7.14)

In the above equations t_i is the thickness of the elastomer layers.

7.6.2.3 Shear strain due to the shear displacement

(1)P The shear strain due to the total design seismic displacement $d_{Ed} = \sqrt{d_{Edx}^2 + d_{Edy}^2}$, including torsional effects, shall be determined as follows:

$$\varepsilon_s = \frac{d_{Ed}}{t_t} \tag{7.15}$$

where:
$t_t = \Sigma t_i$ is the total thickness of the elastomer.

7.6.2.4 Shear strain due to angular rotations

(1)P The shear strain due to angular rotations shall be determined as follows:

- For rectangular bearings:

$$\varepsilon_\alpha = (b_x{}^2\alpha_x + b_y{}^2\alpha_y) \;/\; 2t_i\, t_t \tag{7.16}$$

where:
α_x and α_y are the angular rotations across the b_x and b_y dimensions of the bearings respectively.

- For circular bearings of diameter D:

$$\varepsilon_\alpha = D^2\alpha \;/\; 2t_i\, t_t \tag{7.17}$$

where $$\alpha = \sqrt{\alpha_x^2 + \alpha_y^2}$$

(2) Normally in bridges the influence of ε_α is negligible for the seismic verification.

7.6.3 Design criteria for normal elastomeric bearings

(1)P Adequate seismic links shall be provided where it is necessary to secure the structural integrity of the bridge in accordance with Clause 6.6.1 (4). The links shall be designed in accordance with Clause 6.6.3.

(2)P The bearings shall comply with the following seismic design criteria

7.6.3.1 Maximum shear strains

- Total shear strain

$$\varepsilon_{td} \leq 0.75\ \varepsilon_{bu} \tag{7.18}$$

where ε_{bu} = 5 for bearings complying with the tests of Annex J
= 4 for normal laminated bearings according to 7.6.1 (3).

- Design seismic strain

$$\varepsilon_s \leq 2.0 \tag{7.19}$$

7.6.3.2 Stability

(1)P Either of the following criteria must be satisfied

$$b_{min} / t_t \geq 4 \tag{7.20}$$

or

$$\frac{\sigma_e}{G} \leq \frac{2b_{min}}{3t_t} S_{min} \tag{7.21}$$

where:
b_{min} is the minimum dimension of the bearing
S_{min} is the minimum shape factor of the bearing layers
t_t is the total elastomer thickness and
σ_e is the maximum effective normal stress of the bearing according to Clause 7.6.2.2.

7.6.3.3 Fixing of bearings

(1)P Friction may be considered to insure that sliding of the bearing does not occur, if both the following criteria are satisfied under the most adverse seismic design condition.

$$\frac{V_{Ed}}{N_{Ed}} \leq 0.1 + \frac{k_f}{\sigma_e} \tag{7.22}$$

$$\sigma_e \geq 3.0 \text{ N/mm}^2 \tag{7.23}$$

where:
k_f is 0.6 for concrete and 0.2 for all other surfaces,
V_{Ed} and N_{Ed} are respectively the shear and the axial force transmitted simultaneously through the bearing according to the design seismic combinations, and
$\sigma_e = N_{Ed}/A_r$ is the effective normal stress in N/mm^2

(2)P In the opposite case positive means of fixing shall be provided to resist the entire maximum design shear force V_{Ed}.

7.6.3.4 Clearances

(1)P The clearance for the protection of main or critical structural elements and the slack of the seismic links shall accommodate a total design seismic displacement calculated according to Clause 2.3.6.3, with the value of the seismic displacement d_E increased by 20%.

7.6.4 Design criteria for special elastomeric bearings

(1)P Such bearings must comply with the prototype tests given in Annex J.

(2)P The bearings shall satisfy the criteria defined in Clauses 7.5.2.4 and 7.5.2.5.

(3)P The criteria given in Clause 7.6.3.1 and 7.6.3.2 shall be met.

(4)P Positive means of fixing capable of resisting the entire maximum design shear force V_{Ed} shall be provided.

(5) The use of seismic links is not compulsory.

8. SPECIAL BRIDGES

8.1 Introduction

(1)P The following bridge types are considered special in the sense that the rules of the present code may not cover all aspects of their seismic design:

- Arch bridges
- Bridges with inclined supporting struts or V-type piers
- Cable stayed bridges
- Bridges with extreme geometry (highly skewed bridges or bridges with large horizontal curvature)
- Bridges with markedly differing yielding of the piers.

(2) The provisions of this clause are intended as general guidelines and not as conclusive design rules for each particular case.

8.2 Selection of the intended seismic behaviour

8.2.1 Elastic behaviour

(1) In many cases it is preferable to select an elastic seismic behavior ($q=1$) i.e. to design the bridge so as to remain elastic under the design seismic action. Such cases are:

- Arch bridges or bridges with inclined struts in which the design and detailing for ductility of the plastic hinges may be unreliable due to the presence of high axial compression.

- Cable stayed bridges in which the seismic response may be dominated by higher mode effects.

(2) In such cases multimode response spectrum analysis should be performed and second order effects should be estimated with adequate accuracy.

(3) The critical sections resulting from the elastic analysis should be located at members with inherent ductility and adequate detailing aiming at the improvement of this ductility is recommended. Avoidance of brittle failure modes should be secured according to Clause 8.3.

8.2.2 Ductile behaviour

8.2.2.1 General

(1)P The assumption of a ductile behaviour for special bridges is allowed if the selected behaviour factors can be justified by an appropriate accompanying non-linear time history analysis according to Clause 4.2.4.

8.2.2.2 High axial forces

(1)P When high axial forces ($\eta_k > 0.30$) prevail at the plastic hinges (f.i. in arch bridges or bridges with inclined supporting struts) an equivalent linear multimode analysis, using reduced values, according to Annex H, of the q-factors defined in Clause 4.1.6 for ductile behaviour, is allowed under the following conditions:

a. The deck remains elastic
b. The seismic displacement is caused mainly by the deformation of the supporting elements of the structure (i.e. not by the deformation of the bearings or of the soil).
c. The influence of the higher modes on the seismic response is not high. This condition is considered to be satisfied if the modal mass of the fundamental mode in the direction considered is at least 70% of the mass of the deck.

8.2.2.3 Sequential yielding of the piers

(1) In the longitudinal direction a ductile behaviour may in general be assumed for rectilinear bridges, subject to the conditions (a), (b) and (c) of the previous clause.

(2) In the transverse direction the influence of sequential yielding of the piers on the transverse bending of the deck and on the distribution of the seismic action to the piers must be taken into account. A more detailed presentation of this subject is given in Annex H.

8.3 Avoidance of brittle failure modes

(1)P Structural components such as fixed bearings, sockets and anchorages for cables and stays and other non-ductile connections shall be designed using capacity design effects determined from the strength of the cables and an overstrength factor of at least 1.4.

(2)P This verification may be omitted if it can be proven that the integrity of the structure is not affected by the failure of such connections. This proof must also cover the risk of sequential failure, such as may occur in stays of cable-stayed bridges.

(3)P Similar conservative assumptions as in (1) above must be used to avoid other types of brittle failure modes such as:

- Torsional failure of the deck in bridges with large horizontal curvature.
- Combined torsional and bending failure of piers in highly skewed bridges.

ANNEX A (INFORMATIVE)

DESIGN SEISMIC EVENT FOR BRIDGES AND RECOMMENDATIONS FOR ITS SELECTION DURING THE CONSTRUCTION PHASE

A.1 Design seismic event

(1) The design seismic event can be defined by selecting an acceptably low probability (p) of it being exceeded within the design life (t_d) of the structure. Then the return period of the event (t_r) is given by the expression:

$$t_r = 1/(1-(1-p)^{1/td}) \qquad \text{(A.1)}$$

(2) The design seismic action corresponding to γ_I=1.0 usually reflects a design seismic event with a return period of approximately [475] years. Such an event has a probability of exceedence ranging between [0.10] and [0.19] for a design life ranging between [50] and [100] years respectively. This level of design action is applicable to the majority of the bridges, which are considered to be of average importance.

A.2 Design seismic event for the construction phase

(1) Assuming that t_c is the duration of the construction phase of a bridge and p is the acceptable probability of exceedence of the design seismic event during this phase, the return period t_{rc} is given by equation (A.1), using t_c instead of t_d. For the relatively small values usually associated with t_c ($t_c \leq$ 5years), equation (A.1) may be approximated by the following simpler relation:

$$t_{rc} \cong \frac{t_c}{p} \qquad \text{(A.2)}$$

It is recommended that the value of p does not exceed 0.05.

(2) The value of the design ground acceleration a_{gc} corresponding to a return period t_{rc}, depends on the seismicity of the region. In many cases the following relation offers an acceptable approximation

$$a_{gc} / a_g = (t_{rc}/t_{ro})^k \qquad \text{(A.3)}$$

where:

a_g is the standard design ground acceleration corresponding to the reference return period t_{ro} (= [475] years)

The value of the exponent $k \cong$ [0.30 ÷ 0.45] must be estimated taking into account a reliable statistical evaluation of available seismic data.

(3) Independently from the design seismic actions, the robustness of all partial bridge structures should be secured during the construction phases.

ANNEX B (INFORMATIVE)

RELATIONSHIP BETWEEN DISPLACEMENT DUCTILITY AND CURVATURE DUCTILITY OF PLASTIC HINGES IN CONCRETE PIERS

(1) Assuming that:

- the horizontal displacement at the center of mass of the deck is due only to the deformation of a fully fixed cantilever pier of length L and that
- L_h is the length of the plastic hinge developing at the base of the pier

the required curvature ductility μ_c of the hinge corresponding to a structure displacement ductility μ_d, as defined in clause 2.3.5.2, is:

$$\mu_c = \frac{C_u}{C_y} = 1 + \frac{\mu_d - 1}{3\lambda(1-0.5\lambda)} \qquad \text{(B.1)}$$

where: $\lambda = L_h/L$

(2) In reinforced concrete sections (where the curvature ductility is used as a measure of the ductility of the plastic hinge), the value of the ratio λ is influenced by such effects as the reinforcement tensile strain penetration in the adjoining element, the inclined cracking due to shear-flexure interaction etc. The following expressions are given only as indicative estimates, since no generally accepted method is presently available.

$$L_h = 0.08L + 0.022\, D_s f_y \qquad \text{(B.2)}$$

or

$$L_h = (0.4 - 0.6)h \qquad \text{(B.3)}$$

where:
D_s and f_y are the diameter and the yield stress of the longitudinal reinforcement in m and N/mm^2 respectively and,
h is the section depth.

(3) When a considerable part of the deck displacement is due to the deformation of other components which remain elastic after the formation of the plastic hinge, the required curvature ductility μ_{cf} is given by the expression

$$\mu_{cf} = 1 + f\,(\mu_c - 1) \qquad \text{(B.4)}$$

where $f = d_{tot}/d_p$, is the ratio of the total deck displacement to the displacement due to the deformation of the pier only and μ_c is calculated from equation (B.1)

Note

If the seismic action is transmitted between deck and pier through flexible elastomeric bearings inducing f.i. a value of f=5 and assuming that a certain value of μ_c, fi. $\mu_c=15$, would be required in the case of rigid connection between the deck and pier, the required value of μ_{cf} according to equation (B.4) amounts to 71, which is certainly not available. It is therefore evident that the high flexibility of the elastomeric bearings, used in the same force path with the stiff pier, imposes a practically elastic behavior to the whole system.

ANNEX C (INFORMATIVE)

ESTIMATION OF THE EFFECTIVE STIFFNESS OF R. CONCRETE DUCTILE MEMBERS

C.1 General

(1) The effective stiffness of ductile concrete components used in linear seismic analysis should be equal to the secant stiffness at the theoretical yield point. In the absence of a more accurate method one of the following approximate methods may be used:

C.2 Method 1

(1) The effective moment of inertia J_{eff} of a pier of constant cross section is estimated as follows:

$$J_{eff} = 0.08 \,.\, J_{un} + J_{cr} \qquad \text{(C.1)}$$

where:
J_{un} is the moment of inertia of the cross-section of the uncracked pier
J_{cr} is the moment of inertia of the cracked section at the yield point of the tensile reinforcement. This is estimated from the expression:

$$J_{cr} = M_y/(E_c.C_y) \qquad \text{(C.2)}$$

in which M_y and C_y are the yield monent and curvature of the section respectively and E_c is the elastic modulus of concrete.

(2) These expressions have been derived from a parametric analysis of a simplified non-linear model of cantilever pier with hollow rectangular and hollow and solid circular cross-sections.

C.3 Method 2

(1) The effective stiffness is estimated from the design ultimate moment M_{Rd} and the yield curvature C_y of the plastic hinge section as follows:

$$E_cJ_{eff} = \nu M_{Rd}/C_y \qquad \text{(C.3)}$$

where:
$\nu = 1.20$ is a correction coefficient reflecting the stiffening effect of the uncracked part of the pier

$$C_y = (\varepsilon_{sy} - \varepsilon_{cy})/d \qquad \text{(C.4)}$$

and

d is the effective depth of the section,

ε_{sy} is the yield strain of the reinforcement,

ε_{cy} is the compressive strain of the concrete at the yield of the tensile reinforcement.

(2) It should be noted that an overestimation of the effective stiffness leads to results which are on the safe side regarding the seismic actions while displacements can be corrected after the analysis, on the basis of the actual stiffness.

(3) As an example, the assumption of the values $\varepsilon_{cy} = -2.10^{-3}$ $\varepsilon_{sy} = 2.10^{-3}$ leads to:

$$E_c J_{eff} = 300 M_{Rd}\, d \qquad (C.5)$$

which should, in general, be on the high side.

(4) The analysis performed on the basis of a value of E_cJ_{eff} based on an assumed value of M_{Rd} needs to be corrected only if the required value $M_{Rd,req}$ is significantly higher that the assumed value M_{Rd}. When $M_{Rd,req} < M_{Rd}$ the displacements should be multiplied by the ratio $M_{Rd}/M_{Rd,req}$.

ANNEX D INFORMATIVE

SPATIAL VARIABILITY AND ROTATIONAL COMPONENTS OF EARTHQUAKE MOTION

D.1 General

(1) The characterization of the spatial variability and rotational components of the earthquake action motion shall be carried out by considering the probable contribution of the P, S, Love and Rayleigh waves to the total earthquake vibration and the variability of ground conditions. However, simplified models can be used. These models must obey the condition that the response spectra of the motion at every point shall not be smaller than the corresponding site dependent response spectra multiplied by [0.75].

D.2 Variability of earthquake motion

D.2.1 Introduction

(1) In general, a spatial variability model of the earthquake motion must be used only if there exist certain geological discontinuities or marked topographical features capable to introduce significant variations in the characteristics of the ground motion, or if the length of the bridge is greater than [600m].

(2) Spatial variability means that the motion at different points is different; the difference may be measured by the correlation function. If the correlation function is near unity, the motions are very similar; if the correlation function is near zero, the motions are very different and are considered independent. Spatial variability is very dependent on the frequency band considered; for distances of about 1000 m there is small variability in the low frequency band ($f < 1$ Hz) and a large variability in the high frequency band ($f > 5$ Hz). In general, the spatial variability model shall be such that two points separated by a given distance shall have independent motions in a certain frequency band. If two motions are considered independent, their contributions to the response may be combined by the "square root of the sum of the squares" rule.

(3) Under usual conditions the influence of the spatial variability of the earthquake motion upon the maximum values of the structural response is small. Thus, the spatial variability may be simply disregarded or represented by a very idealized model. A possible model for the spatial variability is presented in this annex; however, other models may be used if they respect the condition stated in D.1.

D.2.2 Wave propagation

(1) The propagation with velocity c of an earthquake vibration $u_i(t)$ (i=1,2,3) between a station a (taken as reference) and a station b is given by

$$u_i^b(t) = r_i u_i^a(t+d/c) \qquad \text{(D.1)}$$

where $u_i^b(t)$ and $u_i^{\alpha}(t)$ are the time histories of vibration at points b and a, r_i is the ratio of the wave amplitudes at b and at a and d is the distance between a and b measured along the ray of the wave. The ratio r_i is a measure of the dissipation of the vibration (by geometrical spreading or by frictional attenuation) along the distance ($d \rightarrow \infty$, $r \rightarrow 0$).

(2) Let $S_{pq}^{aa}(\omega)$ be the spectral functions matrix at station a; then the spectral density functions matrix at point b and the joint spectral density functions matrices between point a and b are given by

$$S_{pq}^{bb}(\omega) = r_p r_q S_{pq}^{aa}(\omega) \tag{D.2}$$

$$S_{pq}^{ab}(\omega) = r_p \exp(i\omega d / c) S_{pq}^{aa}(\omega) \tag{D.3}$$

$$S_{pq}^{ba}(\omega) = r_p \exp(-i\omega d / c) S_{pq}^{aa}(\omega) \tag{D.4}$$

In the case of Rayleigh waves, particles at the surface of the ground describe elliptical trajectories. Let x_1 and x_2 be the horizontal and vertical axis and $S_{11}^{aa}(\omega)$ be the power spectral density of the horizontal accelerations. Then the matrix of the spectral density functions for the vector of horizontal and vertical acceleration $[\ddot{\upsilon}_1, \ddot{\upsilon}_2]^T$ is given by

$$S^{aa}(\omega) = \begin{bmatrix} S_{11}^{aa}(\omega) & S_{12}^{aa}(\omega) \\ S_{21}^{aa}(\omega) & S_{22}^{aa}(\omega) \end{bmatrix} \tag{D.5}$$

where

$$S_{22}^{bb}(\omega) = \rho^2 S_{11}^{aa}(\omega) \tag{D.6}$$

$$S_{12}^{aa}(\omega) = \rho \exp(i\omega) S_{11}^{aa}(\omega) \tag{D.7}$$

$$S_{21}^{aa}(\omega) = \rho \exp(-i\omega) S_{11}^{aa}(\omega) \tag{D.8}$$

with ρ representing the ratio of the vertical to the horizontal component (about 1.5 for an elastic half space).

(3) To set up a wave propagation model it is necessary to decompose the earthquake vibration into adequate wave trains of P, S, Love and Rayleigh waves with appropriate attenuation and dispersive characteristics;
This decomposition depends at least on epicentral distance and focal depth and information on this subject is still not sufficient. Then the matrix of the spectral density functions for all points of the base is obtained by using equations (D.1) to (D.4) for every pair of points of the base.

D.2.3 Simplified model

D.2.3.1 Fundamentals

(1) The simplified model for the spatial variability (without wave propagation) should be based on an ensemble of independent motions which have nonzero values in only a limited zone of the earth surface. Those independent motions must respect the following rules:

- The spatial variability is the same for all components of the motion;
- Each independent motion is band limited. The highest frequency in each band shall not exceed 3 times the lowest frequency;
- For each frequency band, a square mesh is defined. The size of the sides of the square is taken equal to the wavelength corresponding to the lowest frequency of the band. This wavelength should be computed from the average S-wave velocity for the zone within one or two times the length of the bridge. When the size of the sides of the square is greater than five times the length of the bridge, the motion is idealized as a rigid base motion;
- Every node is allocated an independent motion, with the characteristics corresponding to the soil profile at the node;
- The motions on the sides and inside the squares are obtained by a weighted average of the independent motions at the four nodes of the square. The total motion u is given by

$$u(y_1,y_2,t) = \sum_{i=1}^{4} \alpha_i u_i^e(t) \qquad \text{(D.9)}$$

where:
α_i is the i-th weighting factor;
u_i^e is the i-th independent motion.
The weighting factors are given by

$$\alpha_i = \cos\left(\frac{\pi y_1}{l}\right) \cos\left(\frac{\pi y_2}{l}\right) \qquad \text{(D.10)}$$

where l is the length of the side, and y_1 and y_2 are the coordinates of the point under consideration, for a coordinate system whose axes have the same directions as the mesh and whose origin coincides with the i-th node.

(2) When the sites of the nodes have the same soil profile, the characteristics of the earthquake motion are the same everywhere and correspond to the soil profile characteristics.

D.2.3.2 Response spectra

(1) The quantification of the independent motions by response spectra is carried out through partial response spectra obtained from the site dependent response spectrum and the period interval of the independent motion. Let T_{1i} and T_{2i} be the lowest and highest period of the i-th frequency band. Then the partial response spectra are defined by the following rules (expressed in terms of a trilogarithmic representation):

- The partial response spectra for the period interval $[T_{1i}, T_{2i}]$ coincide with the site dependent response spectrum;
- The partial response spectra for periods smaller than $T_{1i}/2$ coincide with the line representing the peak value of ground acceleration a_{ig} for that interval, which can be determined by

$$a_{ig} = \sqrt{I_a(T_{2i}) - I_a(T_{1i})} \qquad \text{(D.11)}$$

- The partial response spectra for periods greater than $2T_{2i}$ coincide with the line representing the peak value of ground displacement d_{ig} for that band, which can be determined by

$$d_{ig} = \sqrt{I_d(T_{2i}) - I_d(T_{1i})} \qquad \text{(D.12)}$$

- In the period interval $[T_{1i}/2, T_{1i}]$ the partial response spectra are defined by a straight line passing through the value of the partial spectra at $T_{1i}/2$ and the geometric mean computed from the value a_{ig} of peak ground acceleration for the interval $[T_{1i}, T_{2i}]$ and the ordinate value of the site dependent response spectrum at T_{1i};
- In the period interval $[T_{2i}, 2T_{2i}]$ the partial response spectrum is defined by a straight line passing through the value of the partial spectra at $2T_{2i}$ and the geometric mean computed from the value d_{ig} of peak ground displacement for the interval $[T_{1i}, T_{2i}]$ and the ordinate value of the site dependent response spectrum at T_{2i}.

(2) The functions I_a (T) and I_d (T) correspond to the indefinite integrals of the acceleration and displacement power spectrum corresponding to the site dependent response spectrum. Those power spectra may be computed by the following approximate relations:

$$S_a = 0.2\,\xi'\,A^2T^{1.4} \quad \text{for } T < T_B \qquad \text{(D.13)}$$
$$S_a = 6\,\xi'\,V^2\,T^{-0.74} \quad \text{for } T_B < T < T_C \qquad \text{(D.14)}$$
$$S_a = 300\,\xi'\,D^2\,T^{-3.1} \quad \text{for } T_C < T \qquad \text{(D.15)}$$

where S_a is the acceleration power spectrum, ξ' is the value of the damping ratio (not in percent) and A, V and D are the values of spectral acceleration, velocity and displacement. T_B and T_C are the response spectrum parameters defined by Table 4.1 of EC8: Part 1.1. Note that the displacement power spectrum is given by $S_d(T) = (T/2\pi)^4 S_a(T)$.

(3) Where the differential displacements of the bridge foundations can induce substantial stresses in the structure, the values of the earthquake action effects shall be determined by

$$E = \sqrt{\sum_m \sum_n \rho_{mn} l_m l_n + \sum_j (k_j d_j)^2} \qquad (D.16)$$

$$\rho_{mn} = \frac{8 r^{3/2} (\xi'_m + r\xi'_n) \sqrt{\xi'_m \xi'_n}}{(1-r^2)^2 + 4\xi'_m \xi'_n r (1+r^2) + 4r^2 (\xi'^2_m + \xi'^2_n)} \qquad (D.17)$$

where E is the value of the earthquake action effect, l_m is the effect due to the m-th mode of vibration, k_j is the effect due to the j-th independent motion, d_j is the assymptotic value of the spectrum for the j-th motion for large periods expressed in displacements, $r = \omega_m / \omega_n$ and ξ'_m is the value of the viscous damping ratio for the m-th mode of vibration.

D.2.3.3 Power spectra

(1) The quantification by power spectra of the independent motions in the frequency band [f_{1i} , f_{2i}] should be carried out by considering partial power spectra with zero values for frequencies lower than f_{1i} and higher than f_{2i} , and coinciding with the site dependent power spectrum in the frequency band [f_{1i} , f_{2i}].

D.2.3.4 Time history representation

(1) The quantification of the independent motions by artificially generated accelerograms should be carried out in accordance with the consistency criteria stated in Clause 3.2.2.4, interpreted as to applied between the ensemble of accelerograms representing the independent motion and the partial response spectrum referred to in the response spectra section.

D.2.4 Relative static displacements model

D.2.4.1 General

(1) In normal bridges and in the absence of a more rigorous assessment , the effects of spatial seismic motion variability may be approximated by applying transient relative static displacements between a reference support point (r) and all other support points (i) of the bridge on the soil.

D.2.4.2 Relative static displacements

(1) The relative horizontal displacement d_{ri} in the longitudinal direction between supports (r) and (i) may be estimated - on a basis similar to that given in Clause 6.6.4 for d_{eg} - as follows:

$$d_{ri} = X_{ri} v_g / c_P \leq \sqrt{2} d_g \qquad (D.18)$$

where:
X_{ri} is the horizontal distance of support (i) from the reference support (r) measured in the longitudinal direction of the bridge,
d_g and v_g are respectively the design values of the peak ground displacement and peak ground velocity as defined by Clause 6.6.4 (3), and
c_P is the compression wave velocity in the soil layer under the supports. In the absence of more accurate data, the values given by Clause 6.6.4 (3) increased by 30% may be used.

(2) When the soil layers under the supports r and i are markedly different, the most unfavorable but mutually consistent values of v_g and d_g shall be used. For c_P a weighted average of the relevant values shall be estimated, using as weighting factors the proportions of the length of each layer to the total distance X_{ri}.

D.2.4.3 Design action effects

(1) The design seismic action effects may be estimated as follows:

$$E_d = \sqrt{E_{do}^2 + E_{rx}^2} \qquad \text{(D.19)}$$

where:
E_{do} are the design action effects assessed ignoring the spatial variability of the seismic motion, and
E_{rx} are the design action effects caused by the displacement vector $\{d_{ri}\}$ according to (D.18), in which all displacements are in the longitudinal direction.

(2) The approximation defined in Clause D.2.4.2 assumes that the relative displacements of the supports in the transverse and vertical directions do not induce significant action effects. This assumption is valid as a rule, because the relevant displacement vectors are -to a large degree- compatible with rigid body motion of the deck. Otherwise a similar approximation may be used for these displacements as well.

D.3 Rotational components

D.3.1 Introduction

(1) The rotational vibrations originate from the spatial derivatives of the transitional components; in consequence, whenever a spatially variable model is used, their inclusion is necessary for the coherency of the model. Moreover, the introduction of the rotational components does not increase significantly the amount of computations needed to perform the analysis.

(2) The consideration of the rotational components about the horizontal axis may be important for structures that are both high and stiff; the consideration of the rotational component about the vertical axis may be important for introducing torsion effects in symmetrical structures. In both cases, however, the contribution of the rotational components to the total response is in most cases small, i.e. do not increase the response more than 10%. Thus, even if the rotational components are not quantified very accurately, the resulting errors in the total response are acceptable.

(3) Due to the generally small contribution of the rotational components to the total value of the response, it is generally preferable to use a very conservative but simple model, as adopted in this subclause, than a very sophisticated model, whose parameters cannot usually be quantified without assumptions which are not, to a large extent, susceptible to rigorous justification. It should be pointed out, however, that the adopted model is a purely kinematical model, where it is assumed that displacements are orthogonal to the propagation direction; furthermore the values of the propagation velocity must also be assumed. Thus, it is also aplicable to Love waves.

D.3.2 Wave propagation

(1) In a direct and orthogonal coordinate system x_1, x_2 and x_3, with axis x_3 vertical, the rotations Θ_i due to a field of displacements u_j are given by

$$\Theta_i = \frac{1}{2}\left(\frac{\theta u_j}{\theta x_k} - \frac{\theta u_k}{\theta x_j}\right) \tag{D.20}$$

where (i, j, k) is an even permutation of (1,2,3). Consider a wave represented by a displacement field $u_j(x_k)$ travelling along x_k with velocity *c* without changes in its profile, which is represented by the equation

$$u_j(t) = f_j(x_k\text{-}ct) \tag{D.21}$$

where f_j is a shape function. The time and space derivatives of u_j are:

$$\frac{\theta u_j}{\theta t} = -c\,\frac{\theta f_j(x_k\text{-}ct)}{\theta(x_k\text{-}ct)} \tag{D.22}$$

$$\frac{\theta u_j}{\theta x_k} = \frac{\theta f_j(x_k\text{-}ct)}{\theta\,(x_k\text{-}ct)} \tag{D.23}$$

From those two expressions it follows that:

$$\frac{\theta u_j}{\theta x_k} = -\frac{1}{c}\,\frac{\theta u_j}{\theta t} \tag{D.24}$$

which shows how to transform time derivatives into space derivatives which may be used with equation (18) to obtain the final result:

$$\Theta_i = -\frac{1}{2c}\frac{\theta u_j}{\theta t} \quad \text{(D.25)}$$

$$\Theta_j = -\frac{1}{2c}\frac{\theta u_i}{\theta t} \quad \text{(D.26)}$$

$$\Theta_k = 0 \quad \text{(D.27)}$$

(2) The quantification of the rotational spectra presented in D.3.3 and D.3.4 assumes that the total earthquake motion is due to S-waves although S-waves cannot exist on the boundaries of elastic solids. In some cases the value to be considered for c is not the propagation velocity of the S-waves in the foundation soil; it is, however, always conservative to take a small value for c, because the amplitude of rotations are proportional to the inverse of **c**. It is generally conservative to attribute to **c** the value of the S-wave propagation on the top soil layer because this layer is generally the softest layer and the S-wave velocity is, in general, sufficiently near the lower bound of the phase velocity of Rayleigh and Love waves; although the group velocity may present significantly lower values, this only occurs for limited frequency bands, usually on the low frequency end of the spectrum, where the frequency content of the rotational spectra is smaller.

D.3.3 Response spectra

(1) The response spectra description of the six components of the earthquake action motion should be constituted by six mutually independent response spectra. Three of those spectra are the site dependent response spectra for the two horizontal components (axes x and y) and the vertical component (axis z) referred to in subclause 3.2.2.2. The rotational response spectra are defined by
where:

$$S^{\theta}_{ex} = \frac{2.0\pi S_e(T)}{cT} \quad \text{(D.28)}$$

$$S^{\theta}_{ey} = \frac{2.0\pi S_e(T)}{cT} \quad \text{(D.29)}$$

$$S^{\theta}_{ez} = \frac{2.0\pi S_e(T)}{cT} \quad \text{(D.30)}$$

where:

S^{θ}_{ex}, S^{θ}_{ey} and S^{θ}_{ez} are the rotational response spectra about axes x, y and z,

S_e is the site dependent response spectrum for the horizontal components,

c is the S-wave velocity, and
T is the period being considered.

Note:
Mutually independent response spectra indicates that the combination rule of "square root of the sum of the squares" may be applicable.

D.3.4 Power spectra

(1) The power spectra description of the earthquake action motion should be constituted by six mutually independent power spectra. Three of those spectra are the site dependent power spectra for the two horizontal components (axes x and y) and the vertical component (axis z). The power spectra for the rotational accelerations are given by

$$S_x^\theta(\omega) = \frac{0.98\omega^2}{4c^2} S(\omega) \tag{D.31}$$

$$S_y^\theta(\omega) = \frac{0.98\omega^2}{4c^2} S(\omega) \tag{D.32}$$

$$S_z^\theta(\omega) = \frac{\omega^2}{2c^2} S(\omega) \tag{D.33}$$

where:

S_x^θ, S_y^θ and S_z^θ are the rotational power spectra about axes x, y and z;
S is the site dependent power spectrum for the horizontal components;
c is the S wave velocity; and
ω is the frequency being considered.

(2) The rotational power spectra defined in this subclause are consistent with the rotational response spectra defined in the previous subclause.

D.3.5 Time history representation

(1) The time history representation of the earthquake action motion should be consistent, in terms of the criteria stated in Clause 3.2.2.4 with the response spectra representation defined in Clause 3.2.2.2.

ANNEX E (NORMATIVE)

ENSEMBLE OF ACCELEROGRAMS FOR STRUCTURAL ANALYSIS

E.1 Introduction

(1) The time history representation of the seismic action involves the set-up of an ensemble of accelerograms to serve as input in dynamic analyses. The results from those analyses should be appropriately evaluated, namely by statistical techniques. Because of this evaluation, an ensemble of artificial accelerograms is in general preferable to an ensemble of real accelerograms, since it can be considered to be a sample of independent and identically distributed accelerograms, which allows the use of well known statistical techniques.

E.2 Artificial accelerograms

(1) The artificially generated accelerograms may be either stationary or non-stationary. Stationary accelerograms may be generated conveniently as realizations of the stationary gaussian stochastic process quantified by the site dependent power spectrum. Non-stationary accelerograms may be generated directly:

(a) as realizations of appropriate nonstationary stochastic processes;
(b) by multiplying a stationary accelerogram by a time modulating function m(t).

(2) The shape of the modulating function depends on the seismo-tectonic characteristics of the region; a possible shape, appropriate for usual conditions, is presented below.

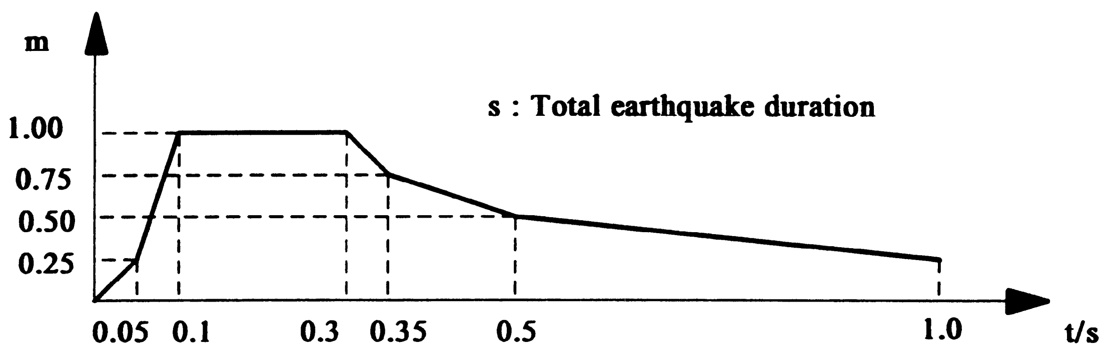


Figure E.1
Shape of modulating function

(3) In case (b) the frequency content is stable. Variable frequency content is obtained by case (a); in this case, the variable frequency content may represent not the characteristics of a single earthquake but the contribution to the total hazard of the possible earthquakes with different magnitudes and focal distances; this can be achieved by modelling the high frequency content to reflect the characteristics of nearby earthquakes and by modelling the low frequency content to reflect the characteristics of longer distance and higher magnitude earthquakes.

E.3 Real accelerograms

(1)P The real accelerograms to be used as input in a dynamic analysis should be recorded in a region or regions with simillar seismo-tectonic characteristics as the site of the structure. Those real accelerograms may be scaled by a factor neither smaller than 0.5 nor greater than 2.

E.4 Validation of ensembles of accelerograms

(1)P Ensemble of accelerograms shall be validated by computing the average response spectra S for a damping value of $\xi' = 0.05$ and for 100 oscilators with natural periods T_i in a geometric progression with common ratio 1.064786 i.e. with periods 0.04 s; 0.0426 s; 0.0454 s...20.0 s. The ensemble is acceptable if, at least for 50 different periods, the average response spectrum is greater or equal than the site dependent response spectrum $S_e(T_i)$. Furthermore the geometric average of the ratios $S_e(T_i)/S_e(T_i)$ for the 13 periods T_i nearest to the fundamental natural period of the structure should be greater than 1.

E.5 Number of accelerograms to be used in the analysis

(1) The results of the analysis performed with an ensemble of accelerograms depend on the number of accelerograms and on the characteristics of the individual accelerograms. It is generally appropriate to consider those results as random variables and use well known statistical techniques to relate the number of accelerograms in the ensemble and the uncertainties in the results.

(2) The effects of the earthquake action should in principle be equal to the average of the extreme response computed for a sample with an infinite number of accelerograms. When only a finite number is used the uncertainties in the results must be taken into account.

(3)P In general the results of the analysis must be corrected through the use of expressions, based on the relevant theory of statistics, that provide a value for the earthquake effects which is not smaller, with 80% probability, than the exact value.

(4) If in the analysis more than 10 accelerograms are used, the average value may be used without any corrections. If a smaller number is used the following correction should be applied:

$$E = (1 + \frac{0.352}{\sqrt{N}})E_a$$

where E is the effect of the earthquake action and E_a is the average of the extreme responses computed for the N accelerograms.

ANNEX F (NORMATIVE)

ADDED MASS OF ENTRAINED WATER FOR IMMERSED PIERS

(1)P The total effective mass of an immersed pier shall be assumed equal to the sum of:

- the actual mass of the pier (without allowance for buoyancy)
- the mass of water eventually enclosed within the pier (for hollow piers)
- the added mass m_a of externally entrained water per unit length of immersed pier.

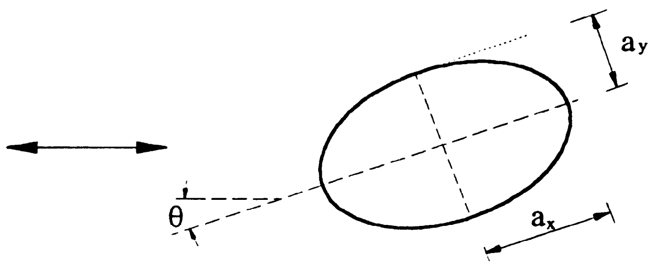
(2) For piers of circular cross section of radius R, m_a may be estimated as:

$$m_a = \rho \pi R^2 \qquad \text{(F.1)}$$

where ρ is the water density.

(3) For piers of elliptical section with axes $2a_x$ and $2a_y$ and earthquake action at an angle θ to the x-axis of the section, m_a may be estimated as:

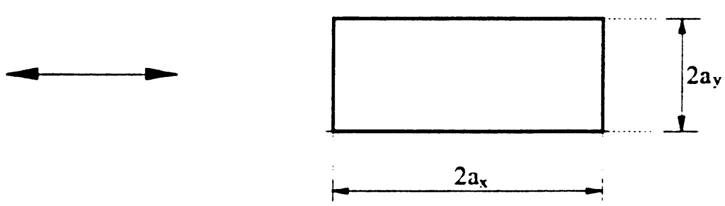
$$m_a = \rho\pi\,(a_y^2 \cos^2\theta + a_x^2 \sin^2\theta) \qquad \text{(F.2)}$$



(4) For piers of rectangular section with dimensions $2a_x.2a_y$ and for earthquake action in the x-direction, m_a may be estimated as:

$$m_a = k\rho\pi a_y^2 \qquad \text{(F.3)}$$

where the value of k is taken from the following table (linear interpolation is permitted)



a_y/a_x	k
0.1	2.23
0.2	1.98
0.5	1.70
1.0	1.51
2.0	1.36
5.0	1.21
10.0	1.14
∞	1.00

ANNEX G (NORMATIVE)

CALCULATION OF CAPACITY DESIGN EFFECTS

G.1 General procedure

(1)P The following procedure shall be applied in general for each sense and for each of the two directions of the design seismic action:

(2)P Step 1:
Calculation of the design flexural strengths $M_{Rd,h}$ of the sections of the intended plastic hinges, corresponding to the selected sense and direction of the seismic action (A_E). The strengths shall be based on the actual dimensions of the cross-sections and the final amount of longitudinal reinforcement. The calculation shall consider the interaction with the axial force and eventually with the bending moment in the other direction, both resulting from the combination G"+"A_E where G is the sum of the permanent actions (gravity loads and post-tensioning) and A_E is the design seismic action.

(3)P Step 2:
Calculation of the variation of action effects ΔF_C of the plastic mechanism, caused by the increase of the moments of the plastic hinges (ΔM_h), from the values due to the permanent actions ($M_{G,h}$) to the moment overstrength of the sections.

$$\Delta M_h = \gamma_o M_{Rd,h} - M_{G,h} \qquad \text{(G.1)}$$

where γ_o is the overstrength factor (see Clause 5.3)
The effects ΔF_C may in general be estimated from equilibrium conditions while reasonable approximations regarding the compatibility of deformations are acceptable.

(4)P Step3:
The final capacity design effects F_C shall be obtained by superimposing the variation ΔF_C to the permanent action effects F_G

$$F_C = F_G + \Delta F_C \qquad \text{(G.2)}$$

G.2 Simplifications

(1) When the bending moment due to the permanent actions at the plastic hinge is negligible compared to the moment overstrength of the section ($M_{G,h} << \gamma_o M_{Rd,h}$), Step 2 above may be replaced by a direct estimation of the effects ΔF_C from the effects A_E of the design earthquake action. This is usually the case in the transverse direction of the piers or in both directions when the piers are hinged to the deck. In such cases the capacity design shear of pier "i" may be estimated as follows:

$$V_{C,i} = \Delta V_i = \frac{\gamma_o M_{Rd,h,i}}{M_{E,i}} V_{E,i} \qquad \text{(G.3)}$$

and the capacity design effects on the deck and abutments may be estimated from the relation:

$$\Delta F_C \cong \frac{\Sigma V_{C,i}}{\Sigma V_{E,i}} E \qquad \text{(G.4)}$$

ANNEX H (NORMATIVE)

DUCTILITY OF SPECIAL BRIDGES

H.1 Influence of high axial forces (for reinforced concrete piers)

(1)P The q-factors given in Table 4.1 of Clause 4.1.6 (designated as q_o in this clause) are valid when the normalized axial force

$$\eta_k = \frac{N_{Ed}}{A_c f_{ck}} \qquad \text{(H.1)}$$

does not exceed 0.3. When η_k exceeds 0.6, plastic hinges are not allowed. Within the interval $0.3 < \eta_k \leq 0.6$ a linear reduction from the value of $q = q_o$ as defined by Table 4.1 to $q = 1.0$ shall be used i.e.

$$\text{for } \eta_k \leq 0.3 \qquad q = q_o \qquad \text{(H.2)}$$

$$\text{for } 0.3 < \eta_k \leq 0.6 \qquad q = q_o - (\eta_k/0.3 - 1)(q_o - 1) \qquad \text{(H.3)}$$

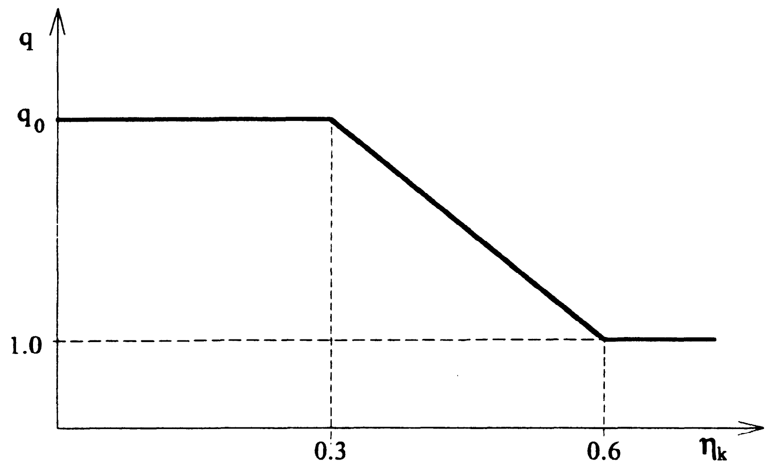


Figure H.1
Influence of η_k on the value of q

H.2 Sequential yielding of the piers

H.2.1 Simultaneous and sequential yielding

(1) A simultaneous yielding of the earthquake resisting piers gives the closest approximation to a theoretical elasto-plastic behaviour. This actually happens when the reinforcement of each pier is equal to that required by the action effects of the design seismic combination. In the case of earthquake resisting piers of substantially different heights (as may be inevitable in steep sided valleys) the seismic action -especially in the longitudinal direction- is concentrated on the shorter and stiffer pier(s), in which the relevant reinforcement requirements are governing. In the more flexible piers however, the minimum reinforcement requirements may be substantially higher than the seismic requirements. In this case sequential yielding is inevitable, with the implications outlined in the following Clauses.

H.2.2 Sequential yielding in the longitudinal direction

(1) The maximum post-elastic strains are induced in the pier which yields first, where the highest local ductility demand shall also probably appear. The increased reinforcement causing some piers to yield later does not cause, in general, a reduction of the dissipated energy, despite the reduced plastic deformations, because of the increased energy dissipation in these piers due to their higher strength. Very flexible piers, which may not enter yielding bếfore the ultimate displacement of the system is reached, have in any case a negligible contribution to the seismic resistance. Therefore, in the longitudinal direction of rectilinear bridges a ductile behaviour may be assumed even under conditions of sequential yielding of the piers.

(2) In the case of sequential yielding, careful estimation of the stiffness of the piers is necessary so that an underestimation of the total stiffness and of the dynamic action effects be avoided. It is recommended that uncracked stiffness be used for all piers, as a conservative assumption, for estimating the seismic action effects. A correction of the relevant displacement should be made, after the analysis, on the basis of the calculated level of strains in the piers.

H.2.3 Sequential yielding in the transverse direction

(1) Sequential yielding of piers in the transverse direction may cause quite substantial deviations between the results of an "equivalent" linear analysis and the actual non-linear response of the bridge. Some of the causes of such deviations are the following:

a. An eventual progressive yielding of the piers from one end of the bridge to the other may induce additional horizontal eccentricity resulting in a disproportionate increase of the ductility demands on those piers that yield first.
b. The transverse bending of the deck predicted by the linear analysis may be invalidated after yielding of the piers.

(2) A reliable estimation of the effects of case (a) above is only possible, in general, by performing an appropriate non-linear time-history analysis. The undesirable effects can however be avoided if the structural system of the bridge is - or can be designed to be - symmetrical in elevation.

(3) In the absence of a non-linear dynamic analysis the effects of case (b) on the deck may be approximated by designing the deck to resist also the action effects due to the equivalent static transverse seismic load, using for each pier (i) a corrected stiffness equal to its secant siffness calculated for a displacement of its top equal to:

$$d_i = \delta d_{Ei} \qquad \text{(H.4)}$$

where:

d_{Ei} is the displacement resulting at the top of pier i from the linear analysis and

δ is the maximum of the ratios:

$$\delta_j = \frac{d_{yj}}{d_{Ej}} = \frac{M_{Rdj}}{M_{Ej}} \qquad \text{(H.5)}$$

where:

d_{yj} is the displacement of the top of pier j at yield and

M_{Rdj} and M_{Ej} are respectively the design flexural strength and the design seismic moment at the base (plastic hinge) of pier j.

ANNEX J (NORMATIVE)

PROTOTYPE TESTS OF SEISMIC ISOLATION SYSTEMS

J.1 Scope

(1)P The tests specified in this Annex aim at establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests.

(2)P The deformation characteristics and damping values of the isolation system used in the design and analysis of seismic-isolated bridges shall be based on the tests, described in this Annex performed on two full-size specimens of each type and size of isolator unit of the isolation system.

Note
In due course the requirements of this Annex will eventually be covered by pertinent EN Standards, in which case this Annex will be reviewed.

J.2 Records

(1)P For each cycle of tests the force-deflection and hysteretic behaviour of the test specimen shall be recorded .

J.3 Sequence and cycles

(1)P The following sequence of tests shall be performed for the prescribed number of cycles, at a vertical load equal to the average dead load, on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the maximum non-seismic design force.

2. Three fully reversed cycles of loading at each of the following increments of the design seismic displacement: 0.25, 0.50, 0.75 and 1.0.

3. Twenty fully reversed cycles of loading at 1.0 times the total design displacement.

(2)P If an isolator unit is also a vertical load-carrying element, then Item 2 of the sequence of cyclic tests specified above shall be performed for the following two additional vertical loads:

$$1.2\ G_k + |F_{Ed}|$$
$$0.8\ G_k - |F_{Ed}|$$

where
G_k is the dead load and
F_{Ed} is the vertical load due to earthquake, based on peak response due to the design earthquake.

J.4 Units dependent on loading rate

(1)P If the force-deflection properties of the isolator units are dependent on the rate of loading, each set of tests specified in Clause J.3 shall be performed at a frequency, f, in the range of 0.1 to 1.0 times the inverse of the effective period T_I. The frequency, f, shall be the minimum frequency of testing at which the effective stiffness and the effective damping at the design displacement are at least 85 percent of the corresponding values when the isolator unit is tested at a frequency equal to the inverse of period T_I.

(2)P If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that represents full-scale prototype loading rates.

(3)P The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 15 percent difference in the effective stiffness at the design displacement, when tested at a frequency equal to the inverse of the effective period of the isolated bridge and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period of the isolated bridge.

J.5 Units dependent on bilateral load

(1)P If the force-deflection properties of the isolator units are dependent on bilateral load, then the tests specified in Clauses J.3 and J.4 shall be augmented to include bilateral load at increments of the total maximum displacement 0.25 and 1.0, 0.50 and 1.0, and 1.0 and 1.0 in each direction respectively.

(2)P If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, then such scale specimens shall be of the same type and material, and manufactured with the same processes and quality as full-scale prototypes.

(3)P The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a 15 percent difference in effective stiffness at the design displacement.

J.6 Static test under the total design displacement

(1)P Isolator units that carry vertical load shall be statically tested for maximum and minimum downward vertical load, combined with the constant horizontal load which induces the total design displacement, under zero vertical load.

(2)P For isolation systems consisting exclusively of elastomeric bearings used in combination with seismic links the total design displacement shall be determined in accordance with Clause 2.3.6.3 and the vertical loads according to Clause J.3 (2).

(3)P For all other isolation systems the total design displacement shall be determined in accordance with Clauses 7.5.2.2 and 7.5.2.3 and the vertical load shall be the most severe value determined according to Clauses J.3 (2) and 7.5.2.4.

J.7 Determination of force-deflection characteristics

(1)P The force-deflection characteristics of the isolation system shall be based on the cyclic load test results of each fully reversed cycle of loading.

(2)P The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$K_{eff} = \frac{F_p - F_n}{d_p - d_n} \qquad \text{(J.1)}$$

where:
F_p and F_n are the maximum positive and maximum negative forces, respectively
d_p and d_n are the maximum positive and maximum negative test displacement, respectively.
If the minimum effective stiffness is to be determined, then $F_{p,min}$ and $F_{n,max}$ shall be used in the equation.

J.8 System adequacy

(1)P The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

a. The force-deflection plots of all tests specified in Clause J.3 have a positive incremental force-carrying capacity.

b. For each increment of the test displacement specified in Item 2 of Clause J.3 and for each vertical load case specified in the same clause there is no greater than a 10 percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.

c. For each increment of the test displacement specified in Item 2 of Clause J.3 and for each vertical load case specified in the same clause, there is no greater than a 10 percent difference in the average value of the effective stiffness of the two test specimens of a common type and size of isolator unit over the required three cycles of test.

d. For each specimen there is no greater than a 20 percent change in the initial effective stiffness of each test specimen over the 20 cycles, in the test specified in Item 3 of Clause J.3.

e. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over the 20 cycles in the test specified in Item 3 of Clause J.3.

f. All specimens of vertical load-carrying elements of the isolation system remain stable without reduction of their load-carrying capacity for a period of 60 seconds when tested for static load as prescribed in Clause J.6.

J.9 Design properties of the isolation system

J.9.1 Effective stiffness

(1)P The minimum and maximum effective stiffness of the isolation system shall be determined as follows:

a. The value of K_{min} shall be based on the minimum effective stiffness of the individual isolator units as established by the cyclic tests of Item 2 of Clause J.3 at a displacement amplitude equal to the design displacement.

b. The value of K_{max} shall be based on the maximum effective stiffness of the individual isolator units as established by the cyclic tests of Item 2 of Clause J.3 at a displacement amplitude equal to the design displacement.

c. For isolator units that are found by the tests of Clauses J.3, J.4 and J.5 to have force-deflection characteristics which vary with vertical load, rate of loading or bilateral load, respectively, the value of K_{max} shall be increased and the value of K_{min} shall be decreased, as necessary, to bound the effects of the measured variation in effective stiffness.

J.9.2 Effective damping

(1)P The effective damping (ξ'_I) of the isolation system shall be calculated as:

$$\xi'_I = \frac{\text{Total Area}}{2\pi k_{max}\, d_{Ed}^2} \qquad \text{(J.2)}$$

where the Total Area shall be taken as the sum of the areas of the hysteresis loops of all isolator units and the hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of Item 2 Clause J.3, at a displacement amplitude equal to the design displacement d_{Ed}.

J.10 Influence of other factors

J.10.1 Temperature

(1)P If the stiffness and/or the effective damping of the isolators depend on their temperature, within the range of [-5°C] to [35°C], separate tests shall be performed according to Clause J.3 to assess the effect of temperature on K_{max}, K_{min} and ξ'_I.

J.10.2 Ageing

(1)P The effects of ageing on the stiffness of the units shall be tested by submitting the specimens to suitable accelerated ageing procedures. Appropriate extrapolation of the results shall be used for the assessment of the effective maximum stiffness.

J.10.3 Load history

(1)P When the initial (virgin) stiffness of the isolators is substantially reduced after a number of cycles of loading (e.g. in the cases of high damping rubber or improved damping rubber isolators) the effective stiffness shall be taken as the average stiffness of the first 4 cycles at the design displacement measured in special tests performed on two virgin (unscragged) specimens according to Item 3 of paragraph (1) of Clause J.3.

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