

Eurocode 8: Design of structures for earthquake resistance —

Part 1: General rules, seismic actions and rules for buildings

ICS 91.120.25

National foreword

This British Standard is the UK implementation of EN 1998-1:2004+A1:2013, incorporating corrigendum July 2009. It supersedes BS EN 1998-1:2004, which is withdrawn.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum July 2009 is indicated in the text by AC1 AC1.

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

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

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

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN. To enable BS EN 1998-1:2004+A1:2013 to be used in the UK, the latest version of the NA to this Standard containing these NDPs should also be used. At the time of publication, it is NA to BS EN 1998-1:2004.

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

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

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

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English version

Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

Eurocode 8: Calcul des structures pour leur résistance aux
séismes - Partie 1: Règles générales, actions sismiques et
règles pour les bâtiments

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben -
Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für
Hochbauten

This European Standard was approved by CEN on 23 April 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

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



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Contents	Page
FOREWORD.....	8
1 GENERAL.....	15
1.1 SCOPE	15
1.1.1 Scope of EN 1998.....	15
1.1.2 Scope of EN 1998-1	15
1.1.3 Further Parts of EN 1998.....	16
1.2 NORMATIVE REFERENCES.....	16
1.2.1 General reference standards.....	16
1.2.2 Reference Codes and Standards.....	17
1.3 ASSUMPTIONS	17
1.4 DISTINCTION BETWEEN PRINCIPLES AND APPLICATION RULES	17
1.5 TERMS AND DEFINITIONS	17
1.5.1 Terms common to all Eurocodes	17
1.5.2 Further terms used in EN 1998.....	17
1.6 SYMBOLS	19
1.6.1 General	19
1.6.2 Further symbols used in Sections 2 and 3 of EN 1998-1.....	19
1.6.3 Further symbols used in Section 4 of EN 1998-1	20
1.6.4 Further symbols used in Section 5 of EN 1998-1	21
1.6.5 Further symbols used in Section 6 of EN 1998-1	24
1.6.6 Further symbols used in Section 7 of EN 1998-1	25
1.6.7 Further symbols used in Section 8 of EN 1998-1	27
1.6.8 Further symbols used in Section 9 of EN 1998-1	27
1.6.9 Further symbols used in Section 10 of EN 1998-1	28
1.7 S.I. UNITS	28
2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA	29
2.1 FUNDAMENTAL REQUIREMENTS.....	29
2.2 COMPLIANCE CRITERIA.....	30
2.2.1 General	30
2.2.2 Ultimate limit state	30
2.2.3 Damage limitation state.....	31
2.2.4 Specific measures	32
2.2.4.1 Design	32
2.2.4.2 Foundations.....	32
2.2.4.3 Quality system plan.....	32
3 GROUND CONDITIONS AND SEISMIC ACTION.....	33
3.1 GROUND CONDITIONS	33
3.1.2 Identification of ground types.....	33
3.2 SEISMIC ACTION	35
3.2.1 Seismic zones	35
3.2.2 Basic representation of the seismic action.....	36
3.2.2.1 General.....	36
3.2.2.2 Horizontal elastic response spectrum	37
3.2.2.3 Vertical elastic response spectrum	40
3.2.2.4 Design ground displacement	41
3.2.2.5 Design spectrum for elastic analysis	41
3.2.3 Alternative representations of the seismic action	42
3.2.3.1 Time - history representation	42
3.2.3.2 Spatial model of the seismic action.....	43
3.2.4 Combinations of the seismic action with other actions.....	44
4 DESIGN OF BUILDINGS.....	45
4.1 GENERAL	45

4.1.1	Scope	45
4.2	CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS	45
4.2.1	Basic principles of conceptual design.....	45
4.2.1.1	Structural simplicity	45
4.2.1.2	Uniformity, symmetry and redundancy	45
4.2.1.3	Bi-directional resistance and stiffness	46
4.2.1.4	Torsional resistance and stiffness.....	46
4.2.1.5	Diaphragmatic behaviour at storey level	46
4.2.1.6	Adequate foundation	47
4.2.2	Primary and secondary seismic members	47
4.2.3	Criteria for structural regularity	48
4.2.3.1	General.....	48
4.2.3.2	Criteria for regularity in plan.....	49
4.2.3.3	Criteria for regularity in elevation	50
4.2.4	Combination coefficients for variable actions	52
4.2.5	Importance classes and importance factors	52
4.3	STRUCTURAL ANALYSIS	53
4.3.1	Modelling	53
4.3.2	Accidental torsional effects	54
4.3.3	Methods of analysis	54
4.3.3.1	General.....	54
4.3.3.2	Lateral force method of analysis	56
4.3.3.3	Modal response spectrum analysis	59
4.3.3.4	Non-linear methods.....	61
4.3.3.5	Combination of the effects of the components of the seismic action	64
4.3.4	Displacement calculation.....	66
4.3.5	Non-structural elements.....	66
4.3.5.1	General.....	66
4.3.5.2	Verification	67
4.3.5.3	Importance factors.....	68
4.3.5.4	Behaviour factors	68
4.3.6	Additional measures for masonry infilled frames.....	68
4.3.6.1	General.....	68
4.3.6.2	Requirements and criteria.....	69
4.3.6.3	Irregularities due to masonry infills	69
4.3.6.4	Damage limitation of infills	70
4.4	SAFETY VERIFICATIONS	71
4.4.1	General	71
4.4.2	Ultimate limit state	71
4.4.2.1	General.....	71
4.4.2.2	Resistance condition.....	71
4.4.2.3	Global and local ductility condition	72
4.4.2.4	Equilibrium condition	74
4.4.2.5	Resistance of horizontal diaphragms.....	74
4.4.2.6	Resistance of foundations.....	74
4.4.2.7	Seismic joint condition.....	75
4.4.3	Damage limitation	76
4.4.3.1	General.....	76
4.4.3.2	Limitation of interstorey drift.....	76
5	SPECIFIC RULES FOR CONCRETE BUILDINGS	78
5.1	GENERAL	78
5.1.1	Scope	78
5.1.2	Terms and definitions	78
5.2	DESIGN CONCEPTS	80
5.2.1	Energy dissipation capacity and ductility classes	80
5.2.2	Structural types and behaviour factors.....	81
5.2.2.1	Structural types	81
5.2.2.2	Behaviour factors for horizontal seismic actions.....	82
5.2.3	Design criteria	84
5.2.3.1	General.....	84
5.2.3.2	Local resistance condition.....	84
5.2.3.3	Capacity design rule	84
5.2.3.4	Local ductility condition	84

5.2.3.5	Structural redundancy	86
5.2.3.6	Secondary seismic members and resistances	86
5.2.3.7	Specific additional measures	86
5.2.4	Safety verifications	87
5.3	DESIGN TO EN 1992-1-1	87
5.3.1	General	87
5.3.2	Materials	88
5.3.3	Behaviour factor	88
5.4	DESIGN FOR DCM	88
5.4.1	Geometrical constraints and materials	88
5.4.1.1	Material requirements	88
5.4.1.2	Geometrical constraints	88
5.4.2	Design action effects	89
5.4.2.1	General	89
5.4.2.2	Beams	90
5.4.2.3	Columns	91
5.4.2.4	Special provisions for ductile walls	92
5.4.2.5	Special provisions for large lightly reinforced walls	94
5.4.3	ULS verifications and detailing	95
5.4.3.1	Beams	95
5.4.3.2	Columns	97
5.4.3.3	Beam-column joints	100
5.4.3.4	Ductile Walls	100
5.4.3.5	Large lightly reinforced walls	104
5.5	DESIGN FOR DCH	106
5.5.1	Geometrical constraints and materials	106
5.5.1.1	Material requirements	106
5.5.1.2	Geometrical constraints	106
5.5.2	Design action effects	107
5.5.2.1	Beams	107
5.5.2.2	Columns	107
5.5.2.3	Beam-column joints	107
5.5.2.4	Ductile Walls	108
5.5.3	ULS verifications and detailing	109
5.5.3.1	Beams	109
5.5.3.2	Columns	111
5.5.3.3	Beam-column joints	112
5.5.3.4	Ductile Walls	114
5.5.3.5	Coupling elements of coupled walls	119
5.6	PROVISIONS FOR ANCHORAGES AND SPLICES	120
5.6.1	General	120
5.6.2	Anchorage of reinforcement	120
5.6.2.1	Columns	120
5.6.2.2	Beams	120
5.6.3	Splicing of bars	122
5.7	DESIGN AND DETAILING OF SECONDARY SEISMIC ELEMENTS	123
5.8	CONCRETE FOUNDATION ELEMENTS	123
5.8.1	Scope	123
5.8.2	Tie-beams and foundation beams	124
5.8.3	Connections of vertical elements with foundation beams or walls	125
5.8.4	Cast-in-place concrete piles and pile caps	125
5.9	LOCAL EFFECTS DUE TO MASONRY OR CONCRETE INFILLS	126
5.10	PROVISIONS FOR CONCRETE DIAPHRAGMS	127
5.11	PRECAST CONCRETE STRUCTURES	127
5.11.1	General	127
5.11.1.1	Scope and structural types	127
5.11.1.2	Evaluation of precast structures	128
5.11.1.3	Design criteria	129
5.11.1.4	Behaviour factors	130
5.11.1.5	Analysis of transient situation	130
5.11.2	Connections of precast elements	131
5.11.2.1	General provisions	131
5.11.2.2	Evaluation of the resistance of connections	132
5.11.3	Elements	132

5.11.3.1	Beams.....	132
5.11.3.2	Columns.....	132
5.11.3.3	Beam-column joints.....	133
5.11.3.4	Precast large-panel walls.....	133
5.11.3.5	Diaphragms.....	135
6	SPECIFIC RULES FOR STEEL BUILDINGS.....	137
6.1	GENERAL.....	137
6.1.1	Scope.....	137
6.1.2	Design concepts.....	137
6.1.3	Safety verifications.....	138
6.2	MATERIALS.....	138
6.3	STRUCTURAL TYPES AND BEHAVIOUR FACTORS.....	140
6.3.1	Structural types.....	140
6.3.2	Behaviour factors.....	143
6.4	STRUCTURAL ANALYSIS.....	144
6.5	DESIGN CRITERIA AND DETAILING RULES FOR DISSIPATIVE STRUCTURAL BEHAVIOUR COMMON TO ALL STRUCTURAL TYPES.....	144
6.5.1	General.....	144
6.5.2	Design criteria for dissipative structures.....	144
6.5.3	Design rules for dissipative elements in compression or bending.....	145
6.5.4	Design rules for parts or elements in tension.....	145
6.5.5	Design rules for connections in dissipative zones.....	145
6.6	DESIGN AND DETAILING RULES FOR MOMENT RESISTING FRAMES.....	146
6.6.1	Design criteria.....	146
6.6.2	Beams.....	146
6.6.3	Columns.....	147
6.6.4	Beam to column connections.....	149
6.7	DESIGN AND DETAILING RULES FOR FRAMES WITH CONCENTRIC BRACINGS.....	150
6.7.1	Design criteria.....	150
6.7.2	Analysis.....	151
6.7.3	Diagonal members.....	152
6.7.4	Beams and columns.....	152
6.8	DESIGN AND DETAILING RULES FOR FRAMES WITH ECCENTRIC BRACINGS.....	153
6.8.1	Design criteria.....	153
6.8.2	Seismic links.....	154
6.8.3	Members not containing seismic links.....	157
6.8.4	Connections of the seismic links.....	158
6.9	DESIGN RULES FOR INVERTED PENDULUM STRUCTURES.....	158
6.10	DESIGN RULES FOR STEEL STRUCTURES WITH CONCRETE CORES OR CONCRETE WALLS AND FOR MOMENT RESISTING FRAMES COMBINED WITH CONCENTRIC BRACINGS OR INFILLS.....	159
6.10.1	Structures with concrete cores or concrete walls.....	159
6.10.2	Moment resisting frames combined with concentric bracings.....	159
6.10.3	Moment resisting frames combined with infills.....	159
6.11	CONTROL OF DESIGN AND CONSTRUCTION.....	159
7	SPECIFIC RULES FOR COMPOSITE STEEL – CONCRETE BUILDINGS.....	161
7.1	GENERAL.....	161
7.1.1	Scope.....	161
7.1.2	Design concepts.....	161
7.1.3	Safety verifications.....	162
7.2	MATERIALS.....	163
7.2.1	Concrete.....	163
7.2.2	Reinforcing steel.....	163
7.2.3	Structural steel.....	163
7.3	STRUCTURAL TYPES AND BEHAVIOUR FACTORS.....	163
7.3.1	Structural types.....	163
7.3.2	Behaviour factors.....	165
7.4	STRUCTURAL ANALYSIS.....	165
7.4.1	Scope.....	165
7.4.2	Stiffness of sections.....	166

7.5	DESIGN CRITERIA AND DETAILING RULES FOR DISSIPATIVE STRUCTURAL BEHAVIOUR COMMON TO ALL STRUCTURAL TYPES.....	166
7.5.1	General	166
7.5.2	Design criteria for dissipative structures	166
7.5.3	Plastic resistance of dissipative zones	167
7.5.4	Detailing rules for composite connections in dissipative zones.....	167
7.6	RULES FOR MEMBERS	170
7.6.1	General	170
7.6.2	Steel beams composite with slab	172
7.6.3	Effective width of slab.....	174
7.6.4	Fully encased composite columns	176
7.6.5	Partially-encased members	178
7.6.6	Filled Composite Columns	179
7.7	DESIGN AND DETAILING RULES FOR MOMENT FRAMES	180
7.7.1	Specific criteria.....	180
7.7.2	Analysis	180
7.7.3	Rules for beams and columns	180
7.7.4	Beam to column connections	181
7.7.5	Condition for disregarding the composite character of beams with slab.	181
7.8	DESIGN AND DETAILING RULES FOR COMPOSITE CONCENTRICALLY BRACED FRAMES.....	181
7.8.1	Specific criteria.....	181
7.8.2	Analysis	182
7.8.3	Diagonal members	182
7.8.4	Beams and columns	182
7.9	DESIGN AND DETAILING RULES FOR COMPOSITE ECCENTRICALLY BRACED FRAMES	182
7.9.1	Specific criteria.....	182
7.9.2	Analysis	182
7.9.3	Links	182
7.9.4	Members not containing seismic links.....	183
7.10	DESIGN AND DETAILING RULES FOR STRUCTURAL SYSTEMS MADE OF REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS	183
7.10.1	Specific criteria	183
7.10.2	Analysis	185
7.10.3	Detailing rules for composite walls of ductility class DCM	185
7.10.4	Detailing rules for coupling beams of ductility class DCM.....	186
7.10.5	Additional detailing rules for ductility class DCH.....	186
7.11	DESIGN AND DETAILING RULES FOR COMPOSITE STEEL PLATE SHEAR WALLS	186
7.11.1	Specific criteria	186
7.11.2	Analysis	187
7.11.3	Detailing rules.....	187
7.12	CONTROL OF DESIGN AND CONSTRUCTION	187
8	SPECIFIC RULES FOR TIMBER BUILDINGS.....	188
8.1	GENERAL	188
8.1.1	Scope	188
8.1.2	Definitions	188
8.1.3	Design concepts.....	188
8.2	MATERIALS AND PROPERTIES OF DISSIPATIVE ZONES	189
8.3	DUCTILITY CLASSES AND BEHAVIOUR FACTORS.....	190
8.4	STRUCTURAL ANALYSIS	191
8.5	DETAILING RULES	191
8.5.1	General	191
8.5.2	Detailing rules for connections	192
8.5.3	Detailing rules for horizontal diaphragms	192
8.6	SAFETY VERIFICATIONS	192
8.7	CONTROL OF DESIGN AND CONSTRUCTION	193
9	SPECIFIC RULES FOR MASONRY BUILDINGS	194
9.1	SCOPE	194
9.2	MATERIALS AND BONDING PATTERNS	194

9.2.1	Types of masonry units.....	194
9.2.2	Minimum strength of masonry units.....	194
9.2.3	Mortar	194
9.2.4	Masonry bond	194
9.3	TYPES OF CONSTRUCTION AND BEHAVIOUR FACTORS	195
9.4	STRUCTURAL ANALYSIS	196
9.5	DESIGN CRITERIA AND CONSTRUCTION RULES	197
9.5.1	General	197
9.5.2	Additional requirements for unreinforced masonry satisfying EN 1998-1	198
9.5.3	Additional requirements for confined masonry	198
9.5.4	Additional requirements for reinforced masonry.....	199
9.6	SAFETY VERIFICATION	200
9.7	RULES FOR “SIMPLE MASONRY BUILDINGS”	200
9.7.1	General	200
9.7.2	Rules	200
10	BASE ISOLATION	203
10.1	SCOPE	203
10.2	DEFINITIONS	203
10.3	FUNDAMENTAL REQUIREMENTS.....	204
10.4	COMPLIANCE CRITERIA	205
10.5	GENERAL DESIGN PROVISIONS	205
10.5.1	General provisions concerning the devices.....	205
10.5.2	Control of undesirable movements	206
10.5.3	Control of differential seismic ground motions	206
10.5.4	Control of displacements relative to surrounding ground and constructions	206
10.5.5	Conceptual design of base isolated buildings	206
10.6	SEISMIC ACTION	207
10.7	BEHAVIOUR FACTOR	207
10.8	PROPERTIES OF THE ISOLATION SYSTEM.....	207
10.9	STRUCTURAL ANALYSIS	208
10.9.1	General.....	208
10.9.2	Equivalent linear analysis	208
10.9.3	Simplified linear analysis.....	209
10.9.4	Modal simplified linear analysis	211
10.9.5	Time-history analysis.....	211
10.9.6	Non structural elements	211
10.10	SAFETY VERIFICATIONS AT ULTIMATE LIMIT STATE.....	211
	ANNEX A (INFORMATIVE) ELASTIC DISPLACEMENT RESPONSE SPECTRUM	213
	ANNEX B (INFORMATIVE) DETERMINATION OF THE TARGET DISPLACEMENT FOR NONLINEAR STATIC (PUSHOVER) ANALYSIS	215
	ANNEX C (NORMATIVE) DESIGN OF THE SLAB OF STEEL-CONCRETE COMPOSITE BEAMS AT BEAM-COLUMN JOINTS IN MOMENT RESISTING FRAMES	219

Foreword

This European Standard EN 1998-1, Eurocode 8: Design of structures for earthquake resistance: General rules, seismic actions and rules for buildings, has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by June 2005, and conflicting national standards shall be withdrawn at latest by March 2010.

This document supersedes ENV 1998-1-1:1994, ENV 1998-1-2:1994 and ENV 1998-1-3:1995.

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

Background of the Eurocode programme

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

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

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

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

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

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

- EN 1990 Eurocode: Basis of structural design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite steel and concrete structures
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design of structures for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures

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

Status and field of application of Eurocodes

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

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

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

² According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAGs/ETAs.

³ According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

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

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

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

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, **[AC1]** and may be followed by a National annex **[AC1]**.

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

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

It may also contain

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

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

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

Additional information specific to EN 1998-1

The scope of EN 1998 is defined in **1.1.1** and the scope of this Part of EN 1998 is defined in **1.1.2**. Additional Parts of EN 1998 are listed in **1.1.3**.

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

EN 1998-1 was developed from the merger of ENV 1998-1-1:1994, ENV 1998-1-2:1994 and ENV 1998-1-3:1995. As mentioned in **1.1.1**, attention must be paid to the fact that for the design of structures in seismic regions the provisions of EN 1998 are to be applied in addition to the provisions of the other relevant EN 1990 to EN 1997 and EN 1999.

One fundamental issue in EN 1998-1 is the definition of the seismic action. Given the wide difference of seismic hazard and seismo-genetic characteristics in the various member countries, the seismic action is herein defined in general terms. The definition allows various Nationally Determined Parameters (NDP) which should be confirmed or modified in the National Annexes.

It is however considered that, by the use of a common basic model for the representation of the seismic action, an important step is taken in EN 1998-1 in terms of Code harmonisation.

EN 1998-1 contains in its section related to masonry buildings specific provisions which simplify the design of "simple masonry buildings".

National annex for EN 1998-1

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may be made. Therefore the National Standard implementing EN 1998-1 should have a National Annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1998-1:2004 through clauses:

Reference	Item
1.1.2(7)	Informative Annexes A and B.
2.1(1)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
2.1(1)P	Reference return period T_{DLR} of seismic action for the damage limitation requirement. (or, equivalently, reference probability of exceedance in 10 years, P_{DLR}).
3.1.1(4)	Conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classification may be used.
3.1.2(1)	Ground classification scheme accounting for deep geology, including values of parameters S , T_B , T_C and T_D defining horizontal and vertical elastic response spectra in accordance with 3.2.2.2 and 3.2.2.3 .
3.2.1(1), (2),(3)	Seismic zone maps and reference ground accelerations therein.
3.2.1(4)	Governing parameter (identification and value) for threshold of low seismicity .

AC1 3.2.1(5)P AC1	Governing parameter (identification and value) for threshold of very low seismicity .
3.2.2.1(4), AC1 3.2.2.2(2)P AC1	Parameters S , T_B , T_C , T_D defining shape of horizontal elastic response spectra.
3.2.2.3(1)P	Parameters a_{vg} , T_B , T_C , T_D defining shape of vertical elastic response spectra.
3.2.2.5(4)P	Lower bound factor β on design spectral values.
4.2.3.2(8)	Reference to definitions of centre of stiffness and of torsional radius in multi-storey buildings meeting or not conditions (a) and (b) of 4.2.3.2(8)
4.2.4(2)P	Values of φ for buildings.
4.2.5(5)P	Importance factor γ_I for buildings.
4.3.3.1 (4)	Decision on whether nonlinear methods of analysis may be applied for the design of non-base-isolated buildings. Reference to information on member deformation capacities and the associated partial factors for the Ultimate Limit State for design or evaluation on the basis of nonlinear analysis methods.
4.3.3.1 (8)	Threshold value of importance factor, γ_I , relating to the permitted use of analysis with two planar models.
4.4.2.5 (2).	AC1 Overstrength factor γ_d for diaphragms. AC1
4.4.3.2 (2)	Reduction factor ν for displacements at damage limitation limit state
AC1 5.2.1(5)P AC1	Geographical limitations on use of ductility classes for concrete buildings.
5.2.2.2(10)	q_o -value for concrete buildings subjected to special Quality System Plan.
AC1 5.2.4(3) AC1	Material partial factors for concrete buildings in the seismic design situation.
5.4.3.5.2(1)	Minimum web reinforcement of large lightly reinforced concrete walls
5.8.2(3)	Minimum cross-sectional dimensions of concrete foundation beams.
5.8.2(4)	Minimum thickness and reinforcement ratio of concrete foundation slabs.
5.8.2(5)	Minimum reinforcement ratio of concrete foundation beams.
5.11.1.3.2(3)	Ductility class of precast wall panel systems.
5.11.1.4	AC1 Reduction factors k_p of behavior factors of precast systems. AC1
5.11.1.5(2)	Seismic action during erection of precast structures.
5.11.3.4(7)e	Minimum longitudinal steel in grouted connections of large panel

	walls.
AC1 6.1.2(1)P AC1	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concept; geographical limitations on use of ductility classes for steel buildings.
6.1.3(1)	Material partial factors for steel buildings in the seismic design situation.
6.2(3)	Overstrength factor for capacity design of steel buildings.
6.2 (7)	Information as to how AC1 EN 1993-1-10:2005 AC1 may be used in the seismic design situation
6.5.5(7)	Reference to complementary rules on acceptable connection design
6.7.4(2)	Residual post-buckling resistance of compression diagonals in steel frames with V -bracings.
AC1 7.1.2(1)P AC1	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concept; geographical limitations on use of ductility classes for composite steel-concrete buildings.
7.1.3(1), (3)	Material partial factors for composite steel-concrete buildings in the seismic design situation.
7.1.3(4)	Overstrength factor for capacity design of composite steel-concrete buildings
7.7.2(4)	Stiffness reduction factor for concrete part of a composite steel-concrete column section
AC1 8.3(1)P AC1	Ductility class for timber buildings.
9.2.1(1)	Type of masonry units with sufficient robustness.
9.2.2(1)	Minimum strength of masonry units.
9.2.3(1)	Minimum strength of mortar in masonry buildings.
9.2.4(1)	Alternative classes for perpend joints in masonry
9.3(2)	Conditions for use of unreinforced masonry satisfying provisions of EN 1996 alone.
9.3(2)	Minimum effective thickness of unreinforced masonry walls satisfying provisions of EN 1996 alone.
9.3(3)	Maximum value of ground acceleration for the use of unreinforced masonry satisfying provisions of EN. 1998-1
9.3(4), Table 9.1	q -factor values in masonry buildings.
9.3(4), Table 9.1	q -factors for buildings with masonry systems which provide enhanced ductility.
9.5.1(5)	Geometric requirements for masonry shear walls.
9.6(3)	Material partial factors in masonry buildings in the seismic design situation.

9.7.2(1)	Maximum number of storeys and minimum area of shear walls of “simple masonry building”.
9.7.2(2)b	Minimum aspect ratio in plan of “simple masonry buildings”.
9.7.2(2)c	Maximum floor area of recesses in plan for “simple masonry buildings”.
9.7.2(5)	Maximum difference in mass and wall area between adjacent storeys of “simple masonry buildings”.
10.3(2)P	Magnification factor on seismic displacements for isolation devices.

Foreword to amendment A1

This document (EN 1998-1:2004/A1:2013) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

This European Standard Amendment shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by February 2014, and conflicting national standards shall be withdrawn at the latest by February 2014.

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

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

1 GENERAL

1.1 Scope

1.1.1 Scope of EN 1998

(1)P EN 1998 applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes:

- human lives are protected;
- damage is limited; and
- structures important for civil protection remain operational.

NOTE The random nature of the seismic events and the limited resources available to counter their effects are such as to make the attainment of these goals only partially possible and only measurable in probabilistic terms. The extent of the protection that can be provided to different categories of buildings, which is only measurable in probabilistic terms, is a matter of optimal allocation of resources and is therefore expected to vary from country to country, depending on the relative importance of the seismic risk with respect to risks of other origin and on the global economic resources.

(2)P Special structures, such as nuclear power plants, offshore structures and large dams, are beyond the scope of EN 1998.

(3)P EN 1998 contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, must be observed for the design of structures in seismic regions. It complements in this respect the other Eurocodes.

(4) EN 1998 is subdivided into various separate Parts (see **1.1.2** and **1.1.3**).

1.1.2 Scope of EN 1998-1

(1) EN 1998-1 applies to the design of buildings and civil engineering works in seismic regions. It is subdivided in 10 Sections, some of which are specifically devoted to the design of buildings.

(2) Section **2** of EN 1998-1 contains the basic performance requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions.

(3) Section **3** of EN 1998-1 gives the rules for the representation of seismic actions and for their combination with other actions. Certain types of structures, dealt with in EN 1998-2 to EN 1998-6, need complementing rules which are given in those Parts.

(4) Section **4** of EN 1998-1 contains general design rules relevant specifically to buildings.

(5) Sections **5** to **9** of EN 1998-1 contain specific rules for various structural materials and elements, relevant specifically to buildings as follows:

- Section 5: Specific rules for concrete buildings;
- Section 6: Specific rules for steel buildings;
- Section 7: Specific rules for composite steel-concrete buildings;
- Section 8: Specific rules for timber buildings;
- Section 9: Specific rules for masonry buildings.

(6) Section 10 contains the fundamental requirements and other relevant aspects of design and safety related to base isolation of structures and specifically to base isolation of buildings.

NOTE Specific rules for isolation of bridges are developed in EN 1998-2.

(7) Annex C contains additional elements related to the design of slab reinforcement in steel-concrete composite beams at beam-column joints of moment frames.

NOTE Informative Annex A and informative Annex B contain additional elements related to the elastic displacement response spectrum and to target displacement for pushover analysis.

1.1.3 Further Parts of EN 1998

(1)P Further Parts of EN 1998 include, in addition to EN 1998-1, the following:

- EN 1998-2 contains specific provisions relevant to bridges;
- EN 1998-3 contains provisions for the seismic assessment and retrofitting of existing buildings;
- EN 1998-4 contains specific provisions relevant to silos, tanks and pipelines;
- EN 1998-5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects;
- EN 1998-6 contains specific provisions relevant to towers, masts and chimneys.

1.2 Normative References

(1)P This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

1.2.1 General reference standards

EN 1990 Eurocode - Basis of structural design

EN 1992-1-1 Eurocode 2 – Design of concrete structures – Part 1-1: General – Common rules for building and civil engineering structures

EN 1993-1-1 Eurocode 3 – Design of steel structures – Part 1-1: General – General rules

- EN 1994-1-1 Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General – Common rules and rules for buildings
- EN 1995-1-1 Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings
- EN 1996-1-1 Eurocode 6 – Design of masonry structures – Part 1-1: General – Rules for reinforced and unreinforced masonry
- EN 1997-1 Eurocode 7 - Geotechnical design – Part 1: General rules

1.2.2 Reference Codes and Standards

(1)P For the application of EN 1998, reference shall be made to **AC1** EN 1990 to EN 1997 and to EN 1999~~AC1~~.

(2) EN 1998 incorporates other normative references cited at the appropriate places in the text. They are listed below:

ISO 1000 The international system of units (SI) and its application;

AC1 EN 1090-2 Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures;

EN 1993-1-8 Eurocode 3: Design of steel structures – Part 1-8: Design of joints;

EN 1993-1-10 Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties; **AC1**

prEN 12512 Timber structures – Test methods – Cyclic testing of joints made with mechanical fasteners.

1.3 Assumptions

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

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

1.4 Distinction between principles and application rules

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

1.5 Terms and definitions

1.5.1 Terms common to all Eurocodes

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

1.5.2 Further terms used in EN 1998-1

(1) The following terms are used in EN 1998-1 with the following meanings:

behaviour factor

factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures

capacity design

design method in which elements of the structural system are chosen and suitably designed and detailed for energy dissipation under severe deformations while all other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained

dissipative structure

structure which is able to dissipate energy by means of ductile hysteretic behaviour and/or by other mechanisms

dissipative zones

predetermined parts of a dissipative structure where the dissipative capabilities are mainly located

NOTE 1 These are also called critical regions.

dynamically independent unit

structure or part of a structure which is directly subjected to the ground motion and whose response is not affected by the response of adjacent units or structures

importance factor

factor which relates to the consequences of a structural failure

non-dissipative structure

structure designed for a particular seismic design situation without taking into account the non-linear material behaviour

non-structural element

architectural, mechanical or electrical element, system and component which, whether due to lack of strength or to the way it is connected to the structure, is not considered in the seismic design as load carrying element

primary seismic members

members considered as part of the structural system that resists the seismic action, modelled in the analysis for the seismic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998

secondary seismic members

members which are not considered as part of the seismic action resisting system and whose strength and stiffness against seismic actions is neglected

NOTE 2 They are not required to comply with all the rules of EN 1998, but are designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the seismic design situation.

1.6 Symbols

1.6.1 General

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

(2) Further symbols, used in connection with seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols used in EN 1998-1 are listed and defined in 1.6.2 and 1.6.3.

1.6.2 Further symbols used in Sections 2 and 3 of EN 1998-1

A_{Ed}	design value of seismic action ($= \gamma_1 A_{Ek}$)
A_{Ek}	characteristic value of the seismic action for the reference return period
E_d	design value of action effects
N_{SPT}	Standard Penetration Test blow-count
P_{NCR}	reference probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement
Q	variable action
$S_e(T)$	elastic horizontal ground acceleration response spectrum also called "elastic response spectrum". At $T=0$, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S .
$S_{ve}(T)$	elastic vertical ground acceleration response spectrum
$S_{De}(T)$	elastic displacement response spectrum
$\boxed{AC1} S_d(T)$	design spectrum (for elastic analysis). $\boxed{AC1}$
S	soil factor
T	vibration period of a linear single degree of freedom system
T_s	duration of the stationary part of the seismic motion
T_{NCR}	reference return period of the reference seismic action for the no-collapse requirement
a_{gR}	reference peak ground acceleration on type A ground
a_g	design ground acceleration on type A ground
a_{vg}	design ground acceleration in the vertical direction
c_u	undrained shear strength of soil
d_g	design ground displacement
g	acceleration of gravity
q	behaviour factor

$v_{s,30}$	average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less
γ	importance factor
η	damping correction factor
ξ	viscous damping ratio (in percent)
$\psi_{2,i}$	combination coefficient for the quasi-permanent value of a variable action i
$\psi_{E,i}$	combination coefficient for a variable action i , to be used when determining the effects of the design seismic action

1.6.3 Further symbols used in Section 4 of EN 1998-1

E_E	effect of the seismic action
E_{Edx}, E_{Edy}	design values of the action effects due to the horizontal components (x and y) of the seismic action
E_{Edz}	design value of the action effects due to the vertical component of the seismic action
F_i	horizontal seismic force at storey i
F_a	horizontal seismic force acting on a non-structural element (appendage)
F_b	base shear force
H	building height from the foundation or from the top of a rigid basement
L_{max}, L_{min}	larger and smaller in plan dimension of the building measured in orthogonal directions
R_d	design value of resistance
S_a	seismic coefficient for non-structural elements
T_1	fundamental period of vibration of a building
T_a	fundamental period of vibration of a non-structural element (appendage)
W_a	weight of a non-structural element (appendage)
d	displacement
d_r	design interstorey drift
e_a	accidental eccentricity of the mass of one storey from its nominal location
h	interstorey height
m_i	mass of storey i
n	number of storeys above the foundation or the top of a rigid basement
q_a	behaviour factor of a non-structural element (appendage)
q_d	displacement behaviour factor
s_i	displacement of mass m_i in the fundamental mode shape of a building
z_i	height of mass m_i above the level of application of the seismic action
α	ratio of the design ground acceleration to the acceleration of gravity

γ_a	importance factor of a non-structural element (appendage)
γ_d	overstrength factor for diaphragms
θ	interstorey drift sensitivity coefficient

1.6.4 Further symbols used in Section 5 of EN 1998-1

A_c	Area of section of concrete member
A_{sh}	total area of horizontal hoops in a beam-column joint
A_{si}	total area of steel bars in each diagonal direction of a coupling beam
A_{st}	area of one leg of the transverse reinforcement
A_{sv}	total area of the vertical reinforcement in the web of the wall
$A_{sv,i}$	total area of column vertical bars between corner bars in one direction through a joint
A_w	total horizontal cross-sectional area of a wall
ΣA_{si}	sum of areas of all inclined bars in both directions, in wall reinforced with inclined bars against sliding shear
ΣA_{sj}	sum of areas of vertical bars of web in a wall, or of additional bars arranged in the wall boundary elements specifically for resistance against sliding shear
ΣM_{Rb}	sum of design values of moments of resistance of the beams framing into a joint in the direction of interest
ΣM_{Rc}	sum of design values of the moments of resistance of the columns framing into a joint in the direction of interest
D_o	diameter of confined core in a circular column
$M_{i,d}$	end moment of a beam or column for the calculation of its capacity design shear
$M_{Rb,i}$	design value of beam moment of resistance at end i
$M_{Rc,i}$	design value of column moment of resistance at end i
N_{Ed}	axial force from the analysis for the seismic design situation
T_1	fundamental period of the building in the horizontal direction of interest
T_C	corner period at the upper limit of the constant acceleration region of the elastic spectrum
V_{Ed}	shear force in a wall from the analysis for the seismic design situation
V_{dd}	dowel resistance of vertical bars in a wall
V_{Ed}	design shear force in a wall
$V_{Ed,max}$	maximum acting shear force at end section of a beam from capacity design calculation
$V_{Ed,min}$	minimum acting shear force at end section of a beam from capacity design calculation
V_{fd}	contribution of friction to resistance of a wall against sliding shear
V_{id}	contribution of inclined bars to resistance of a wall against sliding shear

$V_{Rd,c}$	design value of shear resistance for members without shear reinforcement in accordance with EN1992-1-1:2004
$V_{Rd,S}$	design value of shear resistance against sliding
b	width of bottom flange of beam
b_c	cross-sectional dimension of column
b_{eff}	effective flange width of beam in tension at the face of a supporting column
b_i	distance between consecutive bars engaged by a corner of a tie or by a cross-tie in a column
b_o	width of confined core in a column or in the boundary element of a wall (to centreline of hoops)
b_w	thickness of confined parts of a wall section, or width of the web of a beam
b_{wo}	thickness of web of a wall
d	effective depth of section
d_{bL}	longitudinal bar diameter
d_{bw}	diameter of hoop
f_{cd}	design value of concrete compressive strength
f_{ctm}	mean value of tensile strength of concrete
f_{yd}	design value of yield strength of steel
$f_{yd, h}$	design value of yield strength of the horizontal web reinforcement
$f_{yd, v}$	design value of yield strength of the vertical web reinforcement
f_{yld}	design value of yield strength of the longitudinal reinforcement
f_{ywd}	design value of yield strength of transverse reinforcement
h	cross-sectional depth
h_c	cross-sectional depth of column in the direction of interest
h_f	flange depth
h_{jc}	distance between extreme layers of column reinforcement in a beam-column joint
h_{jw}	distance between beam top and bottom reinforcement
h_o	depth of confined core in a column (to centreline of hoops)
h_s	clear storey height
h_w	height of wall or cross-sectional depth of beam
k_D	factor reflecting the ductility class in the calculation of the required column depth for anchorage of beam bars in a joint, equal to 1 for DCH and to 2/3 for DCM
k_w	factor reflecting the prevailing failure mode in structural systems with walls
l_{cl}	clear length of a beam or a column
l_{cr}	length of critical region

l_i	distance between centrelines of the two sets of inclined bars at the base section of walls with inclined bars against sliding shear
l_w	length of cross-section of wall
n	total number of longitudinal bars laterally engaged by hoops or cross ties on perimeter of column section
q_o	basic value of the behaviour factor
s	spacing of transverse reinforcement
x_u	neutral axis depth
z	internal lever arm
α	confinement effectiveness factor, angle between diagonal bars and axis of a coupling beam
α_o	prevailing aspect ratio of walls of the structural system
α_1	multiplier of horizontal design seismic action at formation of first plastic hinge in the system
α_u	multiplier of horizontal seismic design action at formation of global plastic mechanism
γ_c	partial factor for concrete
γ_{Rd}	model uncertainty factor on design value of resistances in the estimation of capacity design action effects, accounting for various sources of overstrength
γ_s	partial factor for steel
ε_{cu2}	ultimate strain of unconfined concrete
$\varepsilon_{cu2,c}$	ultimate strain of confined concrete
$\varepsilon_{su,k}$	characteristic value of ultimate elongation of reinforcing steel
$\varepsilon_{sy,d}$	design value of steel strain at yield
η	reduction factor on concrete compressive strength due to tensile strains in transverse direction
ζ	ratio, $V_{Ed,min}/V_{Ed,max}$, between the minimum and maximum acting shear forces at the end section of a beam
μ_f	concrete-to-concrete friction coefficient under cyclic actions
μ_ϕ	curvature ductility factor
μ_δ	displacement ductility factor
ν	axial force due in the seismic design situation, normalised to $A_c f_{cd}$
ξ	normalised neutral axis depth
ρ	tension reinforcement ratio
ρ'	compression steel ratio in beams
σ_{cm}	mean value of concrete normal stress

ρ_h	reinforcement ratio of horizontal web bars in a wall
ρ_l	total longitudinal reinforcement ratio
$\rho_{\max}$	maximum allowed tension steel ratio in the critical region of primary seismic beams
ρ_v	reinforcement ratio of vertical web bars in a wall
ρ_w	shear reinforcement ratio
ω_v	mechanical ratio of vertical web reinforcement
ω_{wd}	mechanical volumetric ratio of confining reinforcement

1.6.5 Further symbols used in Section 6 of EN 1998-1

L	beam span
M_{Ed}	design bending moment from the analysis for the seismic design situation
$M_{pl,RdA}$	design value of plastic moment resistance at end A of a member
$M_{pl,RdB}$	design value of plastic moment resistance at end B of a member
N_{Ed}	design axial force from the analysis for the seismic design situation
$N_{Ed,E}$	axial force from the analysis due to the design seismic action alone
$N_{Ed,G}$	axial force due to the non-seismic actions included in the combination of actions for the seismic design situation
$N_{pl,Rd}$	design value of yield resistance in tension of the gross cross-section of a member in accordance with $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$
$N_{Rd}(M_{Ed}, V_{Ed})$	design value of axial resistance of column or diagonal in accordance with $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} in the seismic situation
R_d	resistance of connection in accordance with $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$
R_{fy}	plastic resistance of connected dissipative member based on the design yield stress of material as defined in $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$.
V_{Ed}	design shear force from the analysis for the seismic design situation
$V_{Ed,G}$	shear force due to the non seismic actions included in the combination of actions for the seismic design situation
$V_{Ed,M}$	shear force due to the application of the plastic moments of resistance at the two ends of a beam
$V_{pl,Rd}$	design value of shear resistance of a member in accordance with $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$
$V_{wp,Ed}$	design shear force in web panel due to the design seismic action effects
$V_{wp,Rd}$	design shear resistance of the web panel in accordance with $\boxed{\text{AC1}}$ EN 1993-1-1:2005 $\langle \text{AC1} \rangle$
e	length of seismic link
f_y	nominal yield strength of steel
$\boxed{\text{AC1}} f_{y,\max}$	upper value of the yield strength of steel $\langle \text{AC1} \rangle$

q	behaviour factor
t_w	web thickness of a seismic link
t_f	flange thickness of a seismic link
Ω	multiplicative factor on axial force $N_{Ed,E}$ from the analysis due to the design seismic action, for the design of the non-dissipative members in concentric or eccentric braced frames per Cl. 6.7.4 and 6.8.3 respectively
α	ratio of the smaller design bending moment $M_{Ed,A}$ at one end of a seismic link to the greater bending moments $M_{Ed,B}$ at the end where plastic hinge forms, both moments taken in absolute value
α_1	multiplier of horizontal design seismic action at formation of first plastic hinge in the system
α_u	multiplier of horizontal seismic design action at formation of global plastic mechanism
γ_M	partial factor for material property
γ_{ov}	material overstrength factor
δ	beam deflection at midspan relative to tangent to beam axis at beam end (see Figure 6.11)
γ_{pb}	multiplicative factor on design value $N_{pl,Rd}$ of yield resistance in tension of compression brace in a V bracing, for the estimation of the unbalanced seismic action effect on the beam to which the bracing is connected
γ_s	partial factor for steel
θ_p	rotation capacity of the plastic hinge region
$\bar{\lambda}$	non-dimensional slenderness of a member as defined in AC1 EN 1993-1-1:2005 AC1

1.6.6 Further symbols used in Section 7 of EN 1998-1

A_{pl}	horizontal area of the plate
E_a	Modulus of Elasticity of steel
E_{cm}	mean value of Modulus of Elasticity of concrete in accordance with EN 1992-1-1:2004
I_a	second moment of area of the steel section part of a composite section, with respect to the centroid of the composite section
I_c	second moment of area of the concrete part of a composite section, with respect to the centroid of the composite section
I_{eq}	equivalent second moment of area of the composite section
I_s	second moment of area of the rebars in a composite section, with respect to the centroid of the composite section
$M_{pl,Rd,c}$	design value of plastic moment resistance of column, taken as lower bound and computed taking into account the concrete component of the section and only the steel components of the section classified as ductile

$M_{U,Rd,b}$	upper bound plastic resistance of beam, computed taking into account the concrete component of the section and all the steel components in the section, including those not classified as ductile
$V_{wp,Ed}$	design shear force in web panel, computed on the basis of the plastic resistance of the adjacent dissipative zones in beams or connections
$V_{wp,Rd}$	design shear resistance of the composite steel-concrete web panel in accordance with EN 1994-1-1:2004
b	width of the flange
b_b	width of composite beam (see Figure 7.3a) or bearing width of the concrete of the slab on the column (see Figure 7.7).
b_e	partial effective width of flange on each side of the steel web
b_{eff}	total effective width of concrete flange
b_o	width (minimum dimension) of confined concrete core (to centreline of hoops)
d_{bL}	diameter of longitudinal rebars
d_{bw}	diameter of hoops
f_{yd}	design yield strength of steel
f_{ydf}	design yield strength of steel in the flange
f_{ydw}	design strength of web reinforcement
h_b	depth of composite beam
h_c	depth of composite column section
k_r	rib shape efficiency factor of profiled steel sheeting
k_t	reduction factor of design shear resistance of connectors in accordance with EN 1994-1-1:2004
l_{cl}	clear length of column
l_{cr}	length of critical region
n	steel-to-concrete modular ratio for short term actions
q	behaviour factor
r	reduction factor on concrete rigidity for the calculation of the stiffness of composite columns
t_f	thickness of flange
γ_c	partial factor for concrete
γ_M	partial factor for material property
γ_{ov}	material overstrength factor
γ_s	partial factor for steel
ε_a	total strain of steel at Ultimate Limit State
ε_{cu2}	ultimate compressive strain of unconfined concrete
η	minimum degree of connection as defined in 6.6.1.2 of EN 1994-1-1:2004

1.6.7 Further symbols used in Section 8 of EN 1998-1

E_o	Modulus of Elasticity of timber for instantaneous loading
b	width of timber section
d	fastener-diameter
h	depth of timber beams
k_{mod}	modification factor for instantaneous loading on strength of timber in accordance with EN 1995-1-1:2004
q	behaviour factor
γ_M	partial factor for material properties

1.6.8 Further symbols used in Section 9 of EN 1998-1

$a_{g,urm}$	upper value of the design ground acceleration at the site for use of unreinforced masonry satisfying the provisions of Eurocode 8
A_{min}	total cross-section area of masonry walls required in each horizontal direction for the rules for “simple masonry buildings” to apply
$\langle AC_1 \rangle f_{b,min}$	normalised compressive strength of masonry units normal to the bed face $\langle AC_1 \rangle$
$\langle AC_1 \rangle f_{bh,min}$	normalised compressive strength of masonry units parallel to the bed face in the plane of the wall $\langle AC_1 \rangle$
$f_{m,min}$	minimum strength for mortar
h	greater clear height of the openings adjacent to the wall
h_{ef}	effective height of the wall
l	length of the wall
n	number of storeys above ground
$p_{A,min}$	Minimum sum of horizontal cross-sectional areas of shear walls in each direction, as percentage of the total floor area per storey
p_{max}	percentage of the total floor area above the level
q	behaviour factor
t_{ef}	effective thickness of the wall
$\Delta_{A,max}$	maximum difference in horizontal shear wall cross-sectional area between adjacent storeys of “simple masonry buildings”
$\Delta_{m,max}$	maximum difference in mass between adjacent storeys of “simple masonry buildings”
γ_m	partial factors for masonry properties
γ_s	partial factor for reinforcing steel
λ_{min}	ratio between the length of the small and the length of the long side in plan

1.6.9 Further symbols used in Section 10 of EN 1998-1

K_{eff}	effective stiffness of the isolation system in the principal horizontal direction under consideration, at a displacement equal to the design displacement d_{dc}
K_V	total stiffness of the isolation system in the vertical direction
K_{xi}	effective stiffness of a given unit i in the x direction
K_{yi}	effective stiffness of a given unit i in the y direction
T_{eff}	effective fundamental period of the superstructure corresponding to horizontal translation, the superstructure assumed as a rigid body
T_f	fundamental period of the superstructure assumed fixed at the base
T_V	fundamental period of the superstructure in the vertical direction, the superstructure assumed as a rigid body
M	mass of the superstructure
M_s	magnitude
d_{dc}	design displacement of the effective stiffness centre in the direction considered
d_{db}	total design displacement of an isolator unit
$e_{\text{tot},y}$	total eccentricity in the y direction
f_j	horizontal forces at each level j
r_y	torsional radius of the isolation system
(x_i, y_i)	co-ordinates of the isolator unit i relative to the effective stiffness centre
δ_i	amplification factor
ξ_{eff}	“effective damping”

1.7 S.I. Units

(1)P S.I. Units in accordance with ISO 1000 shall be used.

(2) For calculations, the following units are recommended:

- forces and loads: kN, kN/m, kN/m²
- unit mass: kg/m³, AC1tonne/m³AC1
- mass: kg, AC1tonne AC1
- unit weight: kN/m³
- stresses and strengths: N/mm² (= MN/m² or MPa), kN/m² (=kPa)
- moments (bending, etc): kNm
- acceleration: m/s², g (=9,81 m/s²)

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P Structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

- No-collapse requirement.

The structure shall be designed and constructed to withstand the design seismic action defined in Section 3 without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of: a) the reference seismic action associated with a reference probability of exceedance, P_{NCR} , in 50 years or a reference return period, T_{NCR} , and b) the importance factor γ_1 (see EN 1990:2002 and (2)P and (3)P of this clause) to take into account reliability differentiation.

NOTE 1 The values to be ascribed to P_{NCR} or to T_{NCR} for use in a country may be found in its National Annex of this document. The recommended values are $P_{\text{NCR}} = 10\%$ and $T_{\text{NCR}} = 475$ years.

NOTE 2 The value of the probability of exceedance, P_R , in T_L years of a specific level of the seismic action is related to the mean return period, T_R , of this level of the seismic action in accordance with the expression $T_R = -T_L / \ln(1 - P_R)$. So for a given T_L , the seismic action may equivalently be specified either via its mean return period, T_R , or its probability of exceedance, P_R in T_L years.

- Damage limitation requirement.

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the “damage limitation requirement” has a probability of exceedance, P_{DLR} , in 10 years and a return period, T_{DLR} . In the absence of more precise information, the reduction factor applied on the design seismic action in accordance with 4.4.3.2(2) may be used to obtain the seismic action for the verification of the damage limitation requirement.

NOTE 3 The values to be ascribed to P_{DLR} or to T_{DLR} for use in a country may be found in its National Annex of this document. The recommended values are $P_{\text{DLR}} = 10\%$ and $T_{\text{DLR}} = 95$ years.

(2)P Target reliabilities for the no-collapse requirement and for the damage limitation requirement are established by the National Authorities for different types of buildings or civil engineering works on the basis of the consequences of failure.

(3)P Reliability differentiation is implemented by classifying structures into different importance classes. An importance factor γ_1 is assigned to each importance class. Wherever feasible this factor should be derived so as to correspond to a higher or lower value of the return period of the seismic event (with regard to the reference return period) as appropriate for the design of the specific category of structures (see 3.2.1(3)).

(4) The different levels of reliability are obtained by multiplying the reference seismic action or, when using linear analysis, the corresponding action effects by this importance factor. Detailed guidance on the importance classes and the corresponding importance factors is given in the relevant Parts of EN 1998.

NOTE At most sites the annual rate of exceedance, $H(a_{gR})$, of the reference peak ground acceleration a_{gR} may be taken to vary with a_{gR} as: $H(a_{gR}) \sim k_0 a_{gR}^{-k}$, with the value of the exponent k depending on seismicity, but being generally of the order of 3. Then, if the seismic action is defined in terms of the reference peak ground acceleration a_{gR} , the value of the importance factor γ_I multiplying the reference seismic action to achieve the same probability of exceedance in T_L years as in the T_{LR} years for which the reference seismic action is defined, may be computed as $\gamma_I \sim (T_{LR}/T_L)^{-1/k}$. Alternatively, the value of the importance factor γ_I that needs to multiply the reference seismic action to achieve a value of the probability of exceeding the seismic action, P_L , in T_L years other than the reference probability of exceedance P_{LR} , over the same T_L years, may be estimated as $\gamma_I \sim (P_L/P_{LR})^{-1/k}$.

2.2 Compliance Criteria

2.2.1 General

(1)P In order to satisfy the fundamental requirements in **2.1** the following limit states shall be checked (see **2.2.2** and **2.2.3**):

- ultimate limit states;
- damage limitation states.

Ultimate limit states are those associated with collapse or with other forms of structural failure which might endanger the safety of people.

Damage limitation states are those associated with damage beyond which specified service requirements are no longer met.

(2)P In order to limit the uncertainties and to promote a good behaviour of structures under seismic actions more severe than the design seismic action, a number of pertinent specific measures shall also be taken (see **2.2.4**).

(3) For well defined categories of structures in cases of low seismicity (see **3.2.1(4)**), the fundamental requirements may be satisfied through the application of rules simpler than those given in the relevant Parts of EN 1998.

(4) In cases of very low seismicity, the provisions of EN 1998 need not be observed (see **3.2.1(5)** and the notes therein for the definition of cases of very low seismicity).

(5) Specific rules for "simple masonry buildings" are given in Section **9**. By conforming to these rules, such "simple masonry buildings" are deemed to satisfy the fundamental requirements of EN 1998-1 without analytical safety verifications.

2.2.2 Ultimate limit state

(1)P It shall be verified that the structural system has the resistance and energy-dissipation capacity specified in the relevant Parts of EN 1998.

(2) The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterised by the values of the behaviour factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. As a limiting case, for the design of structures classified as low-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may not be taken, in general, as being greater than the value of 1,5 considered to account for overstrengths. For steel or composite steel concrete buildings, this limiting value of the q factor may be taken as being between 1,5 and 2 (see Note 1 of Table 6.1 or Note 1 of Table 7.1, respectively). For dissipative structures the behaviour factor is taken as being greater than these limiting values accounting for the hysteretic energy dissipation that mainly occurs in specifically designed zones, called dissipative zones or critical regions.

NOTE The value of the behaviour factor q should be limited by the limit state of dynamic stability of the structure and by the damage due to low-cycle fatigue of structural details (especially connections). The most **AC1** unfavourable limiting condition should be applied **AC1** when the values of the q factor are determined. The values of the q factor given in the various Parts of EN 1998 are deemed to conform to this requirement.

(3)P The structure as a whole shall be checked to ensure that it is stable under the design seismic action. Both overturning and sliding stability shall be taken into account. Specific rules for checking the overturning of structures are given in the relevant Parts of EN 1998.

(4)P It shall be verified that both the foundation elements and the foundation soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations. In determining the reactions, due consideration shall be given to the actual resistance that can be developed by the structural element transmitting the actions.

(5)P In the analysis the possible influence of second order effects on the values of the action effects shall be taken into account.

(6)P It shall be verified that under the design seismic action the behaviour of non-structural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements. For buildings, specific rules are given in 4.3.5 and 4.3.6.

2.2.3 Damage limitation state

(1)P An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits defined in the relevant Parts of EN 1998.

(2)P In structures important for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.

2.2.4 Specific measures

2.2.4.1 Design

(1) To the extent possible, structures should have simple and regular forms both in plan and elevation, (see **4.2.3**). If necessary this may be realised by subdividing the structure by joints into dynamically independent units.

(2)P In order to ensure an overall dissipative and ductile behaviour, brittle failure or the premature formation of unstable mechanisms shall be avoided. To this end, where required in the relevant Parts of EN 1998, resort shall be made to the capacity design procedure, which is used to obtain the hierarchy of resistance of the various structural components and failure modes necessary for ensuring a suitable plastic mechanism and for avoiding brittle failure modes.

(3)P Since the seismic performance of a structure is largely dependent on the behaviour of its critical regions or elements, the detailing of the structure in general and of these regions or elements in particular, shall be such as to maintain the capacity to transmit the necessary forces and to dissipate energy under cyclic conditions. To this end, the detailing of connections between structural elements and of regions where non-linear behaviour is foreseeable should receive special care in design.

(4)P The analysis shall be based on an adequate structural model, which, when necessary, shall take into account the influence of soil deformability and of non-structural elements and other aspects, such as the presence of adjacent structures.

2.2.4.2 Foundations

(1)P The stiffness of the foundations shall be adequate for transmitting the actions received from the superstructure to the ground as uniformly as possible.

(2) With the exception of bridges, only one foundation type should in general be used for the same structure, unless the latter consists of dynamically independent units.

2.2.4.3 Quality system plan

(1)P The design documents shall indicate the sizes, the details and the characteristics of the materials of the structural elements. If appropriate, the design documents shall also include the characteristics of special devices to be used and the distances between structural and non-structural elements. The necessary quality control provisions shall also be given.

(2)P Elements of special structural importance requiring special checking during construction shall be identified on the design drawings. In this case the checking methods to be used shall also be specified.

(3) In regions of high seismicity and in structures of special importance, formal quality system plans, covering design, construction, and use, additional to the control procedures prescribed in the other relevant Eurocodes, should be used.

3 GROUND CONDITIONS AND SEISMIC ACTION

3.1 Ground conditions

3.1.1 General

(1)P Appropriate investigations shall be carried out in order to identify the ground conditions in accordance with the types given in **3.1.2**.

(2) Further guidance concerning ground investigation and classification is given in EN 1998-5:2004, **4.2**.

(3) The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake. The possibility of occurrence of such phenomena shall be investigated in accordance with EN 1998-5:2004, Section **4**.

(4) Depending on the importance class of the structure and the particular conditions of the project, ground investigations and/or geological studies should be performed to determine the seismic action.

NOTE The conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classification may be used may be specified in the National Annex.

3.1.2 Identification of ground types

(1) Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

NOTE The ground classification scheme accounting for deep geology for use in a country may be specified in its National Annex, including the values of the parameters S , T_B , T_C and T_D defining the horizontal and vertical elastic response spectra in accordance with **3.2.2.2** and **3.2.2.3**.

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	—	—
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s > 800$ m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content	< 100 (indicative)	—	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

(2) The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of N_{SPT} should be used.

(3) The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \quad (3.1)$$

where h_i and v_i denote the thickness (in metres) and shear-wave velocity (at a shear strain level of 10^{-5} or less) of the i -th formation or layer, in a total of N , existing in the top 30 m.

(4)P For sites with ground conditions matching either one of the two special ground types S_1 or S_2 , special studies for the definition of the seismic action are required. For these types, and particularly for S_2 , the possibility of soil failure under the seismic action shall be taken into account.

NOTE Special attention should be paid if the deposit is of ground type S_1 . Such soils typically have very low values of v_s , low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section 6). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response spectrum on the thickness and v_s value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

3.2 Seismic action

3.2.1 Seismic zones

(1)P For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant.

(2) For most of the applications of EN 1998, the hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR} . Additional parameters required for specific types of structures are given in the relevant Parts of EN 1998.

NOTE The reference peak ground acceleration on type A ground, a_{gR} , for use in a country or parts of the country, may be derived from zonation maps found in its National Annex.

(3) The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}) chosen by the National Authorities (see 2.1(1)P). An importance factor γ_I equal to 1,0 is assigned to this reference return period. For return periods other than the reference (see importance classes in 2.1(3)P and (4)), the design ground acceleration on type A ground a_g is equal to a_{gR} times the importance factor γ_I ($a_g = \gamma_I \cdot a_{gR}$). (See Note to 2.1(4)).

(4) In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

NOTE The selection of the categories of structures, ground types and seismic zones in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, a_g , is not greater than 0,08 g (0,78 m/s²), or those where the product $a_g \cdot S$ is not greater than 0,1 g (0,98 m/s²). The selection of whether the value of a_g , or that of the product $a_g \cdot S$ will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex.

(5)P In cases of very low seismicity, the provisions of EN 1998 need not be observed.

NOTE The selection of the categories of structures, ground types and seismic zones in a country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex. It is recommended to consider as very low seismicity cases either those in which the design ground acceleration on type A ground, a_g , is not greater than 0,04 g (0,39 m/s²), or those where the product $a_g S$ is not greater than 0,05 g (0,49 m/s²). The selection of whether the value of a_g , or that of the product $a_g S$ will be used in a country to define the threshold for very low seismicity cases, can be found in its National Annex.

3.2.2 Basic representation of the seismic action

3.2.2.1 General

(1)P Within the scope of EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an “elastic response spectrum”.

(2) The shape of the elastic response spectrum is taken as being the same for the two levels of seismic action introduced in **2.1(1)P** and **2.2.1(1)P** for the no-collapse requirement (ultimate limit state – design seismic action) and for the damage limitation requirement.

(3)P The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum.

(4) For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them.

NOTE 1 The selection of the shape of the elastic response spectrum to be used in a country or part of the country may be found in its National Annex.

NOTE 2 In selecting the appropriate shape of the spectrum, consideration should be given to the magnitude of earthquakes that contribute most to the seismic hazard defined for the purpose of probabilistic hazard assessment, rather than on conservative upper limits (e.g. the Maximum Credible Earthquake) defined for that purpose.

(5) When the earthquakes affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to enable the design seismic action to be adequately represented. In such circumstances, different values of a_g will normally be required for each type of spectrum and earthquake.

(6) For important structures ($\eta > 1,0$) topographic amplification effects should be taken into account.

NOTE Informative Annex A of EN 1998-5:2004 provides information for topographic amplification effects.

(7) Time-history representations of the earthquake motion may be used (see **3.2.3**).

(8) Allowance for the variation of ground motion in space as well as time may be required for specific types of structures (see EN 1998-2, EN 1998-4 and EN 1998-6).

3.2.2.2 Horizontal elastic response spectrum

(1)P For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2,5 - 1) \right] \quad (3.2)$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \quad (3.3)$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \left[\frac{T_C}{T} \right] \quad (3.4)$$

$$T_D \leq T \leq 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \left[\frac{T_C T_D}{T^2} \right] \quad (3.5)$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_1 \cdot a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see (3) of this subclause.

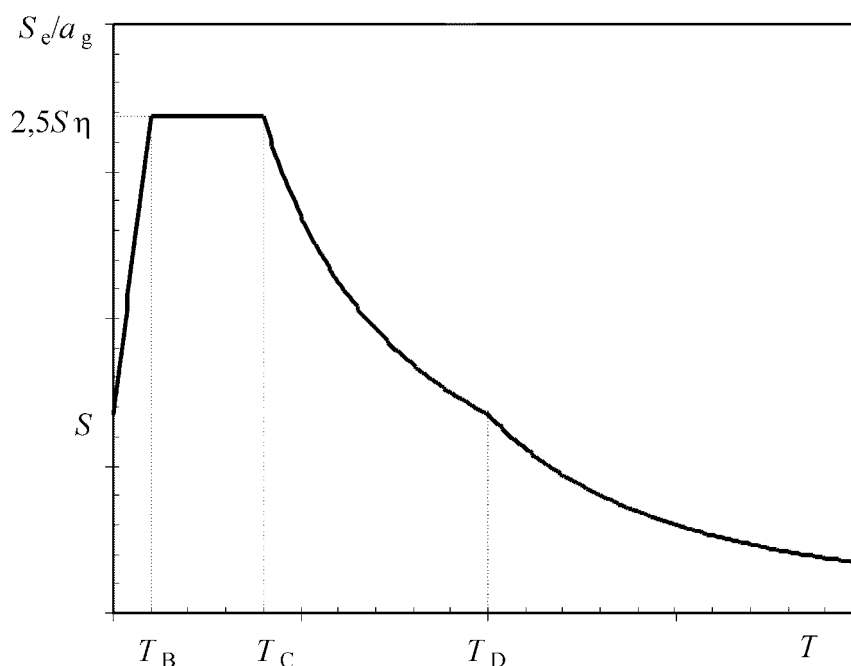


Figure 3.1: Shape of the elastic response spectrum

(2)P The values of the periods T_B , T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

NOTE 1 The values to be ascribed to T_B , T_C , T_D and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see 3.1.2(1)), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S , T_B , T_C and T_D are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalised by a_g , for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

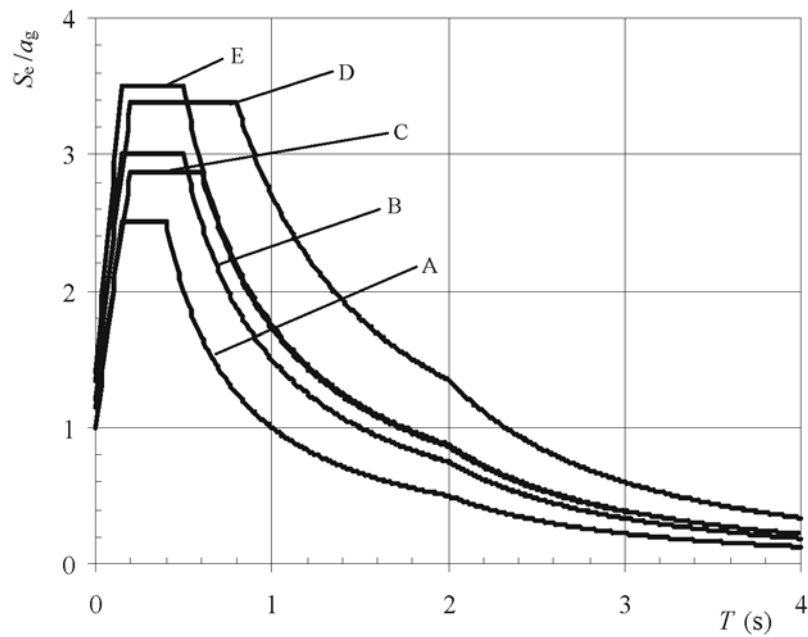


Figure 3.2: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

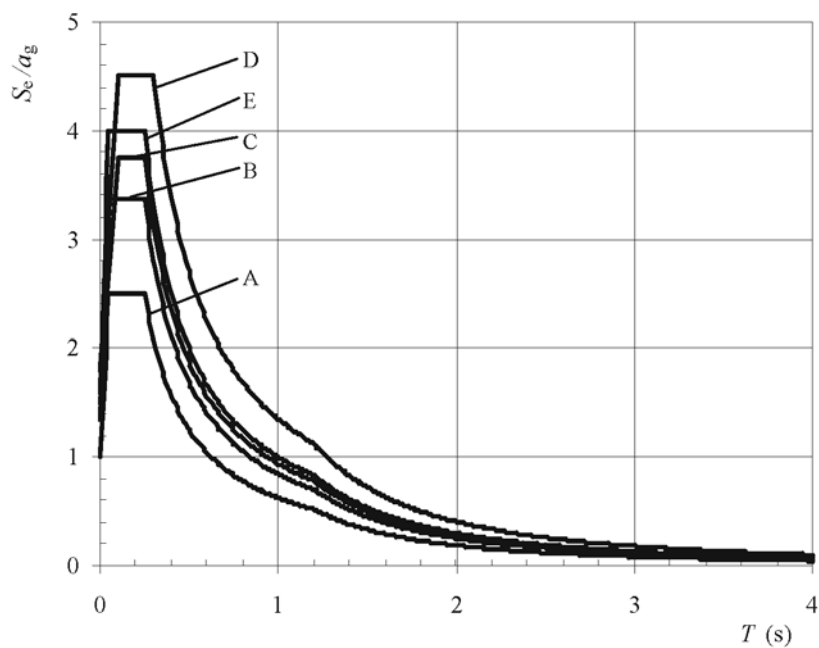


Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

Note 2 For ground types S_1 and S_2 , special studies should provide the corresponding values of S , T_B , T_C and T_D .

(3) The value of the damping correction factor η may be determined by the expression:

$$\eta = \sqrt{10/(5 + \xi)} \geq 0,55 \quad (3.6)$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage.

(4) If for special cases a viscous damping ratio different from 5% is to be used, this value is given in the relevant Part of EN 1998.

(5)P The elastic displacement response spectrum, $S_{De}(T)$, shall be obtained by direct transformation of the elastic acceleration response spectrum, $S_e(T)$, using the following expression:

$$S_{De}(T) = S_e(T) \left[\frac{T}{2\pi} \right]^2 \quad (3.7)$$

(6) Expression (3.7) should normally be applied for vibration periods not exceeding 4,0 s. For structures with vibration periods longer than 4,0 s, a more complete definition of the elastic displacement spectrum is possible.

NOTE For the Type 1 elastic response spectrum referred to in Note 1 to **3.2.2.2(2)P**, such a definition is presented in Informative Annex A in terms of the displacement response spectrum. For periods longer than 4,0 s, the elastic acceleration response spectrum may be derived from the elastic displacement response spectrum by inverting expression (3.7).

3.2.2.3 Vertical elastic response spectrum

(1)P The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using expressions (3.8)-(3.11).

$$0 \leq T \leq T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3,0 - 1) \right] \quad (3.8)$$

$$T_B \leq T \leq T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \quad (3.9)$$

$$T_C \leq T \leq T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \left[\frac{T_C}{T} \right] \quad (3.10)$$

$$T_D \leq T \leq 4s : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \left[\frac{T_C \cdot T_D}{T^2} \right] \quad (3.11)$$

NOTE The values to be ascribed to T_B , T_C , T_D and a_{vg} for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex. The recommended choice is the use of two types of vertical spectra: Type 1 and Type 2. As for the spectra defining the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave

magnitude, M_s , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table 3.4. These recommended values do not apply for special ground types S_1 and S_2 .

Table 3.4: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

3.2.2.4 Design ground displacement

(1) Unless special studies based on the available information indicate otherwise, the design ground displacement d_g , corresponding to the design ground acceleration, may be estimated by means of the following expression:

$$d_g = 0,025 \cdot a_g \cdot S \cdot T_C \cdot T_D \quad (3.12)$$

with a_g , S , T_C and T_D as defined in 3.2.2.2.

3.2.2.5 Design spectrum for elastic analysis

(1) The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

(2) To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behaviour factor q .

(3)P The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q , which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

(4)P For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \quad (3.13)$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \quad (3.14)$$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.15)$$

$$T_D \leq T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.16)$$

where

a_g , S , T_C and T_D are as defined in **3.2.2.2**;

$S_d(T)$ is the design spectrum;

q is the behaviour factor;

β is the lower bound factor for the horizontal design spectrum.

NOTE The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

(5) For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, a_{vg} replacing a_g , S taken as being equal to 1,0 and the other parameters as defined in **3.2.2.3**.

(6) For the vertical component of the seismic action a behaviour factor q up to to 1,5 should generally be adopted for all materials and structural systems.

(7) The adoption of values for q greater than 1,5 in the vertical direction should be justified through an appropriate analysis.

(8)P The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems.

3.2.3 Alternative representations of the seismic action

3.2.3.1 Time - history representation

3.2.3.1.1 General

(1)P The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement).

(2)P When a spatial model of the structure is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible in accordance with the relevant Parts of EN 1998.

(3) Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2) and recorded or simulated accelerograms (see 3.2.3.1.3).

3.2.3.1.2 Artificial accelerograms

(1)P Artificial accelerograms shall be generated so as to match the elastic response spectra given in 3.2.2.2 and 3.2.2.3 for 5% viscous damping ($\xi = 5\%$).

(2)P The duration of the accelerograms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishment of a_g .

(3) When site-specific data are not available, the minimum duration T_s of the stationary part of the accelerograms should be equal to 10 s.

(4) The suite of artificial accelerograms should observe the following rules:

a) a minimum of 3 accelerograms should be used;

b) the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of $a_g S$ for the site in question.

c) in the range of periods between $0,2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.3.1.3 Recorded or simulated accelerograms

(1)P Recorded accelerograms, or accelerograms generated $\overline{AC_1}$ through a numerical simulation $\overline{AC_1}$ of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of $a_g S$ for the zone under consideration.

(2)P For soil amplification analyses and for dynamic slope stability verifications see EN 1998-5:2004, 2.2.

(3) The suite of recorded or simulated accelerograms to be used should satisfy 3.2.3.1.2(4).

3.2.3.2 Spatial model of the seismic action

(1)P For structures with special characteristics such that the assumption of the same excitation at all support points cannot reasonably be made, spatial models of the seismic action shall be used (see 3.2.2.1(8)).

(2)P Such spatial models shall be consistent with the elastic response spectra used for the basic definition of the seismic action in accordance with 3.2.2.2 and 3.2.2.3.

3.2.4 Combinations of the seismic action with other actions

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002, 6.4.3.4.

(2)P The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\Sigma G_{k,j} + \Sigma \psi_{E,i} \cdot Q_{k,i} \quad (3.17)$$

where

$\psi_{E,i}$ is the combination coefficient for variable action i (see 4.2.4).

(3) The combination coefficients $\psi_{E,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

(4) Values of $\psi_{2,i}$ are given in EN 1990:2002 and values of $\psi_{E,i}$ for buildings or other types of structures are given in the relevant parts of EN 1998.

4 DESIGN OF BUILDINGS

4.1 General

4.1.1 Scope

(1)P Section 4 contains general rules for the earthquake-resistant design of buildings and shall be used in conjunction with Sections 2, 3 and 5 to 9.

(2) Sections 5 to 9 are concerned with specific rules for various materials and elements used in buildings.

(3) Guidance on base-isolated buildings is given in Section 10.

4.2 Characteristics of earthquake resistant buildings

4.2.1 Basic principles of conceptual design

(1)P In seismic regions the aspect of seismic hazard shall be taken into account in the early stages of the conceptual design of a building, thus enabling the achievement of a structural system which, within acceptable costs, satisfies the fundamental requirements specified in 2.1.

(2) The guiding principles governing this conceptual design are:

- structural simplicity;
- uniformity, symmetry and redundancy;
- bi-directional resistance and stiffness;
- torsional resistance and stiffness;
- diaphragmatic behaviour at storey level;
- adequate foundation.

These principles are further elaborated in the following subclauses.

4.2.1.1 Structural simplicity

(1) Structural simplicity, characterised by the existence of clear and direct paths for the transmission of the seismic forces, is an important objective to be pursued, since the modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behaviour is much more reliable.

4.2.1.2 Uniformity, symmetry and redundancy

(1) Uniformity in plan is characterised by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building. If necessary, uniformity may be realised by subdividing the entire building by seismic joints into dynamically independent units,

provided that these joints are designed against pounding of the individual units in accordance with **4.4.2.7**.

- (2) Uniformity in the development of the structure along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.
- (3) A close relationship between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness.
- (4) If the building configuration is symmetrical or quasi-symmetrical, a symmetrical layout of structural elements, which should be well-distributed in-plan, is appropriate for the achievement of uniformity.
- (5) The use of evenly distributed structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

4.2.1.3 Bi-directional resistance and stiffness

- (1)P Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction.
- (2) To satisfy **(1)P**, the structural elements should be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.
- (3) The choice of the stiffness characteristics of the structure, while attempting to minimise the effects of the seismic action (taking into account its specific features at the site) should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

4.2.1.4 Torsional resistance and stiffness

- (1) Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

4.2.1.5 Diaphragmatic behaviour at storey level

- (1) In buildings, floors (including the roof) play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).
- (2) Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular

care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

(3) Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis (e.g. rigidity of the diaphragm, see **4.3.1(4)**), particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm.

4.2.1.6 Adequate foundation

(1)P With regard to the seismic action, the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

(2) For structures composed of a discrete number of structural walls, likely to differ in width and stiffness, a rigid, box-type or cellular foundation, containing a foundation slab and a cover slab should generally be chosen.

(3) For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended, subject to the criteria and rules of EN 1998-5:2004, **5.4.1.2**.

4.2.2 Primary and secondary seismic members

(1)P A certain number of structural members (e.g. beams and/or columns) may be designated as “secondary” seismic members (or elements), not forming part of the seismic action resisting system of the building. The strength and stiffness of these elements against seismic actions shall be neglected. They do not need to conform to the requirements of Sections **5** to **9**. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition. Due allowance of 2nd order effects (P-Δ effects) should be made in the design of these members.

(2) Sections **5** to **9** give rules, in addition to those of EN 1992, EN 1993, EN 1994, EN 1995 and EN 1996, for the design and detailing of secondary seismic elements.

(3) All structural members not designated as being secondary seismic members are taken as being primary seismic members. They are taken as being part of the lateral force resisting system, should be modelled in the structural analysis in accordance with **4.3.1** and designed and detailed for earthquake resistance in accordance with the rules of Sections **5** to **9**.

(4) The total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members.

(5) The designation of some structural elements as secondary seismic members is not allowed to change the classification of the structure from non-regular to regular as described in **4.2.3**.

4.2.3 Criteria for structural regularity

4.2.3.1 General

(1)P For the purpose of seismic design, building structures are categorised into being regular or non-regular.

NOTE In building structures consisting of more than one dynamically independent units, the categorisation and the relevant criteria in **4.2.3** refer to the individual dynamically independent units. In such structures, “individual dynamically independent unit” is meant for “building” in **4.2.3**.

(2) This distinction has implications for the following aspects of the seismic design:

- the structural model, which can be either a simplified planar model or a spatial model ;
- the method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one;
- the value of the behaviour factor q , which shall be decreased for buildings non-regular in elevation (see **4.2.3.3**).

(3)P With regard to the implications of structural regularity on analysis and design, separate consideration is given to the regularity characteristics of the building in plan and in elevation (Table 4.1).

Table 4.1: Consequences of structural regularity on seismic analysis and design

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear-elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force ^a	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial ^b	Lateral force ^a	Reference value
No	No	Spatial	Modal	Decreased value

^a If the condition of **4.3.3.2.1(2)a** is also met.

^b Under the specific conditions given in **4.3.3.1(8)** a separate planar model may be used in each horizontal direction, in accordance with **4.3.3.1(8)**.

(4) Criteria describing regularity in plan and in elevation are given in **4.2.3.2** and **4.2.3.3**. Rules concerning modelling and analysis are given in **4.3**.

(5)P The regularity criteria given in **4.2.3.2** and **4.2.3.3** should be taken as necessary conditions. It shall be verified that the assumed regularity of the building structure is not impaired by other characteristics, not included in these criteria.

(6) The reference values of the behaviour factors are given in Sections **5** to **9**.