



DPC: 23/30479678 DC

BSI Group Headquarters

389 Chiswick High Road London W4 4AL

Tel: +44 (0)20 8996 9000

Fax: +44 (0)20 8996 7400

www.bsigroup.com

Date: 07 September 2023

Origin: European

Latest date for receipt of comments: 28 October 2023

Project No. 2022/01472

Responsible committee: B/525/8 Structures in seismic regions

Interested committees:

Title: Draft BS EN 1998-3 Eurocode 8 - Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings and bridges.

Please notify the secretary if you are aware of any keywords that might assist in classifying or identifying the standard or if the content of this standard

- i) has any issues related to 3rd party IPR, patent or copyright
- ii) affects other national standard(s)
- iii) requires additional national guidance or information

WARNING: THIS IS A DRAFT AND MUST NOT BE REGARDED OR USED AS A BRITISH STANDARD. THIS DRAFT IS NOT CURRENT BEYOND 28 October 2023

This draft is issued to allow comments from interested parties; all comments will be given consideration prior to publication. No acknowledgement will normally be sent. **See overleaf for information on the submission of comments.**

No copying is allowed, in any form, without prior written permission from BSI except as permitted under the Copyright, Designs and Patent Act 1988 or for circulation within a nominating organization for briefing purposes. Electronic circulation is limited to dissemination by e-mail within such an organization by committee members.

Further copies of this draft may be purchased from BSI Shop <http://shop.bsigroup.com> or from BSI Customer Services, Tel: +44(0) 20 8996 9001 or email cservices@bsigroup.com. British, International and foreign standards are also available from BSI Customer Services.

Information on the co-operating organizations represented on the committees referenced above may be obtained from <https://standardsdevelopment.bsigroup.com/>.

Responsible Committee Secretary: Ms Ashley Wagner (BSI)

Direct tel:

E-mail: ashley.wagner@bsigroup.com

Full document details: 23/30479678 DC

Introduction

This draft standard is based on national and international discussions. Your comments on this draft are invited and will assist in the preparation of the consequent standard.

For international standards, comments will be reviewed by the relevant UK national committee before sending the consensus UK vote and comments to the international committee, which will then decide appropriate action. If the international standard is approved, it is usual for the text to be published as a British Standard.

For national standards, comments will be reviewed by the relevant UK national committee and the resulting standards published as a British Standard.

UK Vote

Please indicate whether you consider the UK should submit a negative (with supporting technical reasons) or positive vote on this draft. Please indicate if you are aware of any reason why this draft standard should not be published as a British Standard.

Submission of Comments

- **Annotated drafts are not acceptable and will be rejected.**
- All comments should be submitted online at <https://standardsdevelopment.bsigroup.com/>. You will need to register in order to comment.

Template for comments and secretariat observations

Date: xx/xx/20xx	Document: ISO/DIS xxxx
------------------	------------------------

1	2	(3)	4	5	(6)	7
MB	Clause No./ Subclause No./Annex (e.g. 3.1)	Paragraph/ Figure/Table/Note	Type of comment	Comment (justification for change) by the MB	Proposed change by the MB	Secretariat observations on each comment submitted
	3.1	Definition 1	ed	Definition is ambiguous and needs clarifying.	Amend to read '...so that the mains connector to which no connection...'	
	6.4	Paragraph 2	te	The use of the UV photometer as an alternative cannot be supported as serious problems have been encountered in its use in the UK.	Delete reference to UV photometer.	

English Version

Eurocode 8 - Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings and bridges

Eurocode 8 - Calcul des structures pour leur résistance au séisme - Part 3: Evaluation et renforcement des bâtiments et des ponts

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben - Teil 3: Beurteilung und Ertüchtigung von Gebäuden und Brücken

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, 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.

This draft European Standard was established by CEN 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 CEN-CENELEC Management Centre has the same status as the official versions.

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

Recipients of this draft are invited to submit, with their comments, notification of any relevant patent rights of which they are aware and to provide supporting documentation.

Warning : This document is not a European Standard. It is distributed for review and comments. It is subject to change without notice and shall not be referred to as a European Standard.



EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

CEN-CENELEC Management Centre: Rue de la Science 23, B-1040 Brussels

Contents

Page

European foreword	7
Introduction	8
0.1 Introduction to the Eurocodes	8
0.2 Introduction to EN 1998 (all parts)	8
0.3 Introduction to prEN 1998-3	9
0.4 Verbal forms used in the Eurocodes	10
0.5 National annex for prEN 1998-3	10
1 Scope	12
1.1 Scope of prEN 1998-3	12
1.2 Assumptions	12
2 Normative references	13
3 Terms, definitions and symbols	13
3.1 Terms and definitions	13
3.2 Symbols and abbreviations	14
3.2.1 Symbols	14
3.2.2 Abbreviations	28
3.3 S.I. Units	29
4 Basis of design	30
4.1 Performance requirements	30
4.2 Compliance criteria for existing structures	30
4.2.1 Specificity of existing structures	30
4.2.2 Verification rules	31
4.2.3 Verification of Limit States	32
4.3 General procedure for the assessment and retrofitting design	33
4.3.1 Seismic assessment in the current state	33
4.3.2 Design of retrofitting	34
5 Information for structural assessment	34
5.1 General information and history	34
5.2 Required input data	35
5.3 Knowledge levels: Definitions	35
5.4 Knowledge levels: identification	36
5.4.1 Geometry	36
5.4.2 Preliminary analysis	37
5.4.3 Construction details	37
5.4.4 Materials	38
5.5 Representative values of material properties	38
6 Modelling, structural analysis and verification	40
6.1 General	40
6.2 Modelling	40
6.3 Analysis: Force-based approach	41
6.3.1 Reduced spectrum for the force-based approach	41
6.4 Analysis: Displacement-based approach	42
6.4.1 Linear elastic analysis	42
6.4.2 Non-linear static analysis	42

6.4.3	Non-linear response-history analysis	43
6.5	Safety verifications.....	43
6.5.1	General	43
6.5.2	Verifications to Near Collapse limit state	43
6.5.3	Verifications to additional limit states	44
7	Design of structural intervention	45
7.1	Criteria for a structural intervention	45
7.1.1	General	45
7.1.2	General technical criteria	45
7.1.3	Types of intervention	46
7.1.4	Ancillary elements	46
7.1.5	Justification of the selected intervention type	47
7.2	Retrofit design procedure	47
8	Specific rules for reinforced concrete structures	47
8.1	Scope	47
8.2	Identification of geometry, details and materials.....	48
8.2.1	General	48
8.2.2	Geometry	48
8.2.3	Details	48
8.2.4	Materials	49
8.3	Structural modelling	51
8.4	Resistance models for assessment	52
8.4.1	Introduction	52
8.4.2	Beams, columns and walls under flexure with or without axial force	52
8.4.3	Beams, columns and walls: verification of shear in critical zones.....	59
8.4.4	Beam-column joints.....	59
8.5	Verification of limit states	59
8.5.1	Beams, columns and walls under flexure with and without axial force	59
8.5.2	Beams, columns and walls: shear	60
8.5.3	Beam-column joints.....	61
8.6	Resistance models for retrofitting	62
8.6.1	General	62
8.6.2	Concrete jacketing.....	63
8.6.3	Steel jacketing.....	64
8.6.4	FRP plating and wrapping	65
9	Specific rules for steel and composite structures.....	70
9.1	Scope	70
9.2	Identification of geometry, details and materials.....	70
9.2.1	General	70
9.2.2	Geometry	71
9.2.3	Details	71
9.2.4	Materials	72
9.3	Structural modelling	77
9.4	Resistance models for assessment	80
9.4.1	General	80
9.4.2	Beams and columns under flexure with or without axial load	81
9.4.3	Steel bracings	91
9.4.4	Links in frames with eccentric bracings.....	91
9.4.5	Buckling restrained bracings.....	91
9.4.6	Steel column and beam splices	91
9.4.7	Beam-to-column web panel joint.....	92

9.4.8	Bracing-end connections.....	93
9.5	Verification of limit states	94
9.5.1	General.....	94
9.5.2	Beams and columns under flexure with or without axial load	95
9.5.3	Bracings.....	97
9.5.4	Links in frames with eccentric bracings.....	97
9.5.5	Steel column and beam splices	98
9.5.6	Beam-to-column web panel joint.....	98
9.5.7	Bracing-end connections.....	99
9.6	Resistance models for retrofitting.....	99
9.6.1	General.....	99
9.6.2	Weld retrofits	99
9.6.3	Retrofitting with stiffener or doubler plates	100
9.6.4	Beam-to-column joint retrofitting with haunched stiffeners	101
9.6.5	Retrofitting with encased composite columns.....	101
9.6.6	Retrofitting riveted or bolted connections and joints.....	101
10	Specific rules for timber buildings	102
10.1	Scope.....	102
10.2	Identification of geometry, details and materials.....	102
10.2.1	General.....	102
10.2.2	Geometry.....	103
10.2.3	Details	103
10.2.4	Materials	103
10.3	Classification of timber structural members.....	106
10.3.1	Timber diaphragms	106
10.3.2	Timber frames	108
10.4	Structural modelling.....	110
10.4.1	General.....	110
10.4.2	Diaphragms.....	110
10.4.3	Frames	112
10.5	Structural analysis.....	112
10.5.1	General.....	112
10.5.2	Local analysis of diaphragms with a force-based approach.....	112
10.6	Resistance models for assessment	114
10.6.1	General.....	114
10.6.2	Timber diaphragms	114
10.6.3	Carpentry connections.....	114
10.6.4	Dowel-type fastener connections.....	118
10.7	Verification to limit states	118
10.7.1	Timber diaphragms	118
10.7.2	Timber frames	119
10.7.3	Carpentry connections.....	119
10.7.4	Dowel-type connections	119
10.8	Resistance models for retrofitting.....	119
10.8.1	Material design resistance.....	119
10.8.2	Diaphragms.....	120
10.8.3	Timber frames	123
10.8.4	Carpentry connections.....	125
10.8.5	Dowel-type connections	126
11	Specific rules for masonry buildings	127
11.1	Scope.....	127
11.2	Identification of geometry, construction details and materials	127

11.2.1	General	127
11.2.2	Geometry	127
11.2.3	Construction details.....	128
11.2.4	Materials	128
11.3	Structural modelling and analysis.....	130
11.3.1	General	130
11.3.2	Modelling and analysis of global in-plane response of masonry walls.....	132
11.3.3	Modelling and analysis of partial out-of-plane mechanisms and limit analysis.....	135
11.3.4	Modelling of in-plane response of masonry infills in framed buildings.....	139
11.4	Resistance models for assessment	141
11.4.1	Resistance models for in-plane loaded masonry members	141
11.4.2	Resistance models for the assessment of partial out-of-plane mechanisms	151
11.5	Verification of limit states	152
11.5.1	Verification of global in-plane response of masonry walls	152
11.5.2	Verification of partial out-of-plane mechanisms	159
11.6	Analysis and resistance models for retrofitting.....	164
11.6.1	General	164
11.6.2	Structural modelling of strengthened buildings.....	165
11.6.3	Resistance models for strengthened masonry members.....	166
12	Specific rules for bridges	166
12.1	Scope	166
12.2	Performance requirements	166
12.3	Compliance criteria.....	167
12.3.1	Distinction between “ductile” and “brittle” mechanisms.....	167
12.3.2	Distinction between primary and secondary seismic members.....	167
12.4	Information for structural assessment.....	167
12.4.1	General	167
12.4.2	Procedure of investigations.....	167
12.4.3	Assessment of Knowledge Level.....	169
12.5	Assessment procedures.....	169
12.5.1	General	169
12.5.2	Bridges where inertial seismic action is dominant.....	169
12.5.3	Backfilled bridges where kinematic seismic action is dominant.....	170
12.6	Design of structural interventions	170
12.6.1	Intervention on piers	171
12.6.2	Intervention to foundations	171
12.6.3	Intervention on abutments and retaining structures	171
12.6.4	Intervention to bearings	172
12.6.5	Intervention to deck.....	172
Annex A (informative)	Preliminary analysis	173
A.1	Use of this annex	173
A.2	Scope and field of application	173
A.3	Reinforced concrete structures.....	173
A.4	Masonry structures.....	175
Annex B (informative)	Supplementary information for concrete structures.....	177
B.1	Use of this informative annex.....	177
B.2	Scope and field of application	177
B.3	Prediction of ultimate chord rotation at the end of a column with continuous or lap-spliced smooth (plain) bars, section consisting of rectangular parts and/or FRP ..	177
Annex C (informative)	Supplementary information for timber structures.....	180
C.1	Use of this annex	180

C.2	Scope and field of application	180
Annex D (informative)	Supplementary information for masonry buildings	183
D.1	Use of this annex	183
D.2	Scope and field of application	183
D.3	Classification of masonry types not conforming to EN 1996-1-1 and reference values for the material properties	183
D.4	Reference values for the equivalent in-plane stiffness of horizontal diaphragms of different types.....	187
D.5	Drift capacity of masonry members in the case of hybrid failure modes.....	188
D.6	Reference values for the material properties of strengthened masonry types	189
D.7	Repair and retrofitting techniques.....	190
D.7.1	Repair of cracks.....	190
D.7.2	Repair and retrofitting of wall intersections.....	191
D.7.3	Strengthening and stiffening of horizontal diaphragms	191
D.7.4	Tie beams.....	191
D.7.5	Retrofitting of buildings by means of steel ties	191
D.7.6	Retrofitting of rubble core masonry walls (multi-leaf walls).....	192
D.7.7	Retrofitting of walls by means of reinforced concrete jackets or steel profiles	192
D.7.8	Retrofitting of walls by means of polymer grids jackets	192
D.8	Floor spectral accelerations for masonry buildings	192
Annex E (informative)	Flowcharts for the application of this standard	194
E.1	Use of this annex	194
Bibliography		199

European foreword

This document (prEN 1998-3:2023) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1998-3:2005.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of parts:

- EN 1990 Eurocode — Basis of structural and geotechnical design
- EN 1991 Eurocode 1 — Actions on structures
- EN 1992 Eurocode 2 — Design of concrete structures
- EN 1993 Eurocode 3 — Design of steel structures
- EN 1994 Eurocode 4 — Design of composite steel and concrete structures
- EN 1995 Eurocode 5 — Design of timber structures
- EN 1996 Eurocode 6 — Design of masonry structures
- EN 1997 Eurocode 7 — Geotechnical design
- EN 1998 Eurocode 8 — Design of structures for earthquake resistance
- EN 1999 Eurocode 9 — Design of aluminium structures
- New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1998 (all parts)

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

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

Attention should be paid to the fact that, for the design of structures in seismic regions, the provisions of EN 1998 should be applied in addition to the relevant provisions of EN 1990 to EN 1997 and EN 1999. In particular, EN 1998 should be applied to structures of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3. Structures of consequence class CC4 are not fully covered by the Eurocodes but may be required to follow EN 1998, or parts of it, by the relevant Authorities.

By nature, perfect protection (a null seismic risk) against earthquakes is not feasible in practice, namely because the knowledge of the hazard itself is characterized by a significant uncertainty. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquakes likely to occur at a given location and representative of their frequency content. This representation is not the prediction of a particular seismic movement, and such a movement could give rise to more severe effects than those of the seismic action considered, inflicting damage greater than the one described by the Limit States contemplated in this Standard.

Not only the seismic action cannot be predicted, but in addition, it should be recognized that engineering methods are not perfectly predictive when considering the effects of this specific action, under which structures are assumed to respond in the non-linear regime. Such uncertainties are taken into account according to the general framework of EN 1990, with a residual risk of underestimation of their effects.

EN 1998 is subdivided in various parts:

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

EN 1998-1-2, Eurocode 8 — Design of structures for earthquake resistance – Part 1-2: Buildings;

EN 1998-2, Eurocode 8 — Design of structures for earthquake resistance – Part 2: Bridges;

EN 1998-3, Eurocode 8 — Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings and bridges:

EN 1998-4, Eurocode 8 — Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines, towers, masts and chimneys

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

0.3 Introduction to prEN 1998-3

prEN 1998-3 was developed because:

- For many existing structures, seismic resistance was not considered during the original construction, whereas non-seismic actions were catered for, at least by means of traditional construction rules;
- Seismic risk evaluations in accordance with present knowledge may indicate the need for retrofitting campaigns;
- Damage caused by earthquakes may create the need for major repairs, associated with large costs.

Seismic risk mitigation policies may differentiate between “active” and “passive” seismic assessment and retrofitting programmes.

- “Active” programmes may require owners of certain categories of structures to meet specific deadlines for the completion of the seismic assessment and – depending on its outcome – of the retrofitting. The categories of structures selected to be targeted may depend on the associated seismic risk, which depends on hazard, site conditions and vulnerability, and/or on the consequence class and occupancy, or, finally on the perceived vulnerability of the structure (as influenced by type of material and construction, size, age of the structure and contemporary design code, etc.);
- “Passive” programmes associate seismic assessment – possibly leading to retrofitting – with other events or activities related to the use of the structure and its continuity, such as a change in use that increases occupancy or consequence class, remodelling above certain limits (as a percentage of the area or of the total value of the structure), repair of damage after an earthquake, etc.

Therefore, this standard only provides technical clauses. The choice to apply it to a certain type of structure depends on the choice of the Authority concerned or the Project Owner, depending on the risk mitigation approach considered.

The choice of the Limit States to be verified, as well as the return periods of the seismic action ascribed to the various Limit States, may depend on the adopted programme for assessment and retrofitting. The relevant requirements may depend on the cost of the retrofitting works to be undertaken, in relation with the final accepted risks. In “passive” programmes triggered by remodelling, the relevant requirements may graduate with the extent and cost of the remodelling work undertaken.

Unlike new structures, where the mechanical and physical properties of the materials can be prescribed at the time of the project, existing structures can only be partially known, depending on the reconnaissance carried out and the methods of investigation applied. Therefore, the assumed properties for the analysis of these structures are tainted by uncertainties, all the more important as the knowledge resulting from the survey is limited. The conclusions of the assessment, and of the eventual retrofitting design, consequently suffer from inherent uncertainty and there remains a low probability of failure, even when the provisions of this standard have been met.

This standard addresses only the structural aspects of seismic assessment and retrofitting, which may correspond to a single component of a broader strategy for seismic risk mitigation. The conditions under which seismic assessment of individual buildings or bridges – possibly leading to retrofitting – may be required are beyond the scope of this standard. This standard will apply once the requirement to assess a particular structure has been established, in the situation where this structure is dynamically independent of the neighbouring ones. This standard may be applied also when the structure is connected to other structures not explicitly modelled, provided that the structural interaction may be neglected, or it is considered in the model through equivalent constraints and/or added masses.

In cases of low seismic action class (see prEN 1998-1-1:2022, 4.1(4)), this standard may be adapted to local conditions by appropriate National Annexes. The concept of risk-based assessment may be adopted in this context, in particular by countries in low seismicity areas.

In seismic retrofitting situations, qualitative verifications for the identification and elimination of major structural defects are very important and should not be discouraged by the quantitative analytical approach proper to this part of Eurocode 8. Preparation of documents of more qualitative nature is left to the initiative of the National Authorities.

0.4 Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National annex for prEN 1998-3

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1998-3 can have a National Annex containing all national choices to be used for the assessment and retrofitting of buildings and bridges to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in prEN 1998-3 through notes to the following clauses:

4.1(2)	4.1(3)	4.2.2(8)	5.4.4(1)
8.2.4.1(2)	8.6.4.3(10)	10.2.1(3)	A.3(2)

National choice is also allowed in prEN 1998-3 on the application of the following informative annexes:

Annex A

Annex B

Annex C

Annex D

Annex E

Annex F

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of prEN 1998-3

(1) This document is applicable to the assessment and retrofitting of buildings and bridges in seismic regions, namely as given in a) to c):

- a) To provide criteria for the assessment of the seismic performance of existing individual buildings and bridges;
- b) To describe the procedure to be followed in selecting necessary corrective measures;
- c) To set forth criteria for the design of retrofitting measures (i.e. design, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural members).

NOTE 1 For the purposes of this standard, retrofitting covers both the seismic upgrading (e.g. strengthening or adding a passive system) of undamaged structures and the repair and possible upgrading of earthquake-damaged structures.

NOTE 2 Annex E gives flowcharts for the application of this standard.

(2) Unless specifically stated, prEN 1998-1-1 and prEN 1998-5 apply.

(3) Reflecting the performance requirements of prEN 1998-1-1:2022, 4.1, this standard covers the seismic assessment and retrofitting of buildings and bridges made of the more commonly used structural materials: concrete, steel and composite, timber and masonry.

NOTE Annexes B, C and D contain additional guidance related to the assessment of reinforced concrete, timber and masonry structures, respectively, and to their retrofitting when necessary.

(4) This standard is intended for the assessment of individual structures, to decide on the need for structural intervention and to design the retrofitting measures that may be necessary. It is not intended for the vulnerability assessment of populations or groups of structures in seismic risk evaluations for various purposes (e.g. for determining insurance risk, for setting risk mitigation priorities, etc.).

(5) This standard provides (in its material-specific Clauses 8 to 11) criteria for the verification of the more common retrofitting techniques currently in use.

(6) This document gives specific rules for the assessment and retrofitting relevant to existing buildings and bridges of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3.

(7) Although the provisions of this standard are applicable to all common categories of buildings and bridges, the seismic assessment and retrofitting of monuments and heritage structures often requires different types of provisions and approaches, depending on the nature of the monuments.

1.2 Assumptions

(1) The assumptions of prEN 1998-1-1:2022, 1.2, are assumed to be applied.

(2) The provisions of this standard assume that the data collection and tests are performed by experienced personnel and that the engineer responsible for the assessment, the possible design of the retrofitting and the execution of work has appropriate experience of the type of structures being upgraded or repaired.

(3) Inspection procedures, checklists and other data-collection procedures should be documented and filed and should be referred to in the assessment/design documents.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

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

prEN 1998-1-1:2022, *Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action*

prEN 1998-1-2:2023, *Eurocode 8 — Design of structures for earthquake resistance — Part 1-2: Buildings*

prEN 1998-2:2023, *Eurocode 8 — Design of structures for earthquake resistance — Part 2: Bridges*

prEN 1998-5:2022, *Eurocode 8 — Design of structures for earthquake resistance — Part 5: Geotechnical aspects, foundations, retaining and underground structures*

ISO 80000 (all parts), *Quantities and units*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, prEN 1998-1-1:2022, 3.1, prEN 1998-1-2:2023, 3.1 (for buildings), prEN 1998-2:2023, 3.1 (for bridges), and the following, apply.

3.1.1

diaphragm

horizontal or nearly horizontal structural member, such as a floor or roof system, used to transfer inertial lateral forces to vertical members of the seismic action resisting system

3.1.2

diaphragm chord

boundary component perpendicular to the inertial force that is provided to resist tension or compression caused by the diaphragm moment coupling effect

3.1.3

diaphragm collector

component parallel to the inertial force that transfers lateral forces from the diaphragm of the structure to vertical members of the seismic action resisting system

3.1.4

diaphragm tie

diaphragm strut

component parallel to the inertial force that is provided to transfer wall anchorage or diaphragm inertial forces within the diaphragm

3.1.5

dominant knowledge level

knowledge level in the information category (geometry, construction details, material properties) to which a resistance model is most sensitive

3.1.6

knowledge level of construction details

KLD

knowledge level of detailing and connections and their amount, depending on the type of structure

3.1.7

knowledge level of geometry

KLG

knowledge level of geometric properties of the structural system and of the ancillary elements

3.1.8

knowledge level of material properties

KLM

knowledge level of mechanical properties of the constituent materials

3.1.9

simulated design

simulated design is a procedure resulting in the definition of sections and connection details and, in the case of reinforced concrete structures, of the amount and layout of reinforcement

3.2 Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990:2023, 3.2, prEN 1998-1-1:2022, 3.2, prEN 1998-1-2:2023, 3.2 (for buildings), and in prEN 1998-2:2023, 3.2 (for bridges), apply.

For the symbols related to materials, as well as for symbols not specifically related to the seismic design situation, the provisions of the relevant Eurocodes apply.

In addition, further symbols and abbreviations, used in connection with the seismic design situation, are defined in the present standard where they occur, for ease of use. However, the most frequently occurring symbols used in prEN 1998-3 are listed and defined in 3.2.2 and additional abbreviations are given in 3.2.3.

3.2.1 Symbols

3.2.1.1 Symbols used in Clause 8 and Annex B

Upper case Latin symbols

A_c	Column cross-section area
A_{compr}	Full area of compression zone (rectangular or not)
A_f	Horizontally projected cross-sectional area of FRP (fibre-reinforced polymer) strip/sheet with thickness t_f , width w_f and angle β between the fibre direction and the axis of the member ($= t_f w_f \sin \beta$)
A_s	Cross-sectional area of longitudinal steel reinforcement

A_{sh}	Total cross-sectional area of horizontal wall reinforcement
A_{sv}	Total cross-sectional area of vertical wall reinforcement
A_{sw}	Cross-sectional area of stirrup
$C_{Rd,c}$	Correction factor to be used in Formula (8.76)
D	Diameter of circular sections
D_o	Centreline diameter of circular hoop
E_f	FRP (fibre-reinforced polymer) elastic modulus
E_s	Modulus of elasticity of steel reinforcement
H_i	Total height of storey i
K	Strength enhancement factor due to confinement
L_e	Effective bond length
$L_V = M/V$	Shear span at member end
N	Axial force (positive for compression)
$N_{b,compr}$	Number of bars near the extreme compression fibres
R	Radius of FRP at chamfered corners of a rectangular section or around a circular one
R_c	Confining medium radius, equal to the bending radius of the steel tie
$V_{R,c}$	Shear resistance of member without web reinforcement
$V_{R,max}$	Shear resistance as determined by crushing in the diagonal compression strut
V_w	Contribution of transverse reinforcement to shear resistance

Lower case Latin symbols

a_c	Confining medium factor
a_n	Factor for confinement effectiveness within a section
a_s	Factor for confinement effectiveness along the length of the member or shear-span-ratio
a_f	Effectiveness factor for confinement by FRP of a rectangular section with corners chamfered by radius R
a_v	Factor equal to 1 if shear cracking is expected to precede flexural yielding at the end section, or equal to 0 otherwise
a_{vz}	Tension shift of the bending moment diagram
b	Width of steel straps in steel jacket
b_o and h_o	Confined core dimensions to the centreline of the perimeter bar
b_i	Centreline spacing of longitudinal bars (indexed by i) laterally restrained by a stirrup corner or hook along the perimeter of the cross-section
b_w	Width of the cross-section of one web, or width of a wall cross-section
b_x, b_y	Dimensions of a rectangular section with $b_x < b_y$
c	Concrete cover to reinforcement
c_f	Confinement coefficient which depends on the type of FRP

$c_{\min}$	Minimum concrete cover of lapped bars, or half the clear distance to the closest lap-spliced bar, whichever is smaller
d	Effective depth of section (depth to the tension reinforcement)
d'	Depth to the compression reinforcement
d_b	Bar diameter
d_{bL}	Diameter of longitudinal tension reinforcement
f_c	Concrete compressive strength
f_{cm}	Mean concrete compressive strength
f_{cc}	Confined concrete strength
f_{cd}	Design value of concrete strength
f_{ct}	Concrete mean tensile strength
f_{eff}	Effective stress for lap-spliced smooth bars
f_{idd}	Design value of FRP (fibre-reinforced polymer) debonding strength
$f_{idd,e}$	Design value of FRP (fibre-reinforced polymer) effective debonding strength
f_o	Stress of a smooth bar with a standard 180° hook at its anchorage
$f_{o,b}$	Maximum stress that a smooth bar can develop at its hook
$f_{o,b0}$	Maximum stress that a smooth starter bar can develop at the hook of its embedded length
$f_{o,t}$	Maximum stress that a smooth bar can develop at the hook of its embedded length
f_{sm}	Reduced steel yield strength due to lapping of ribbed longitudinal bars
$f_{u,f}$	Effective strength of an FRP
$f_{u,fd}$	Design value of the FRP ultimate strength
f_y	Estimated mean value of steel yield strength
f_{yd}	Design value of yield strength of (longitudinal) reinforcement
f_{yh}	Yield stress of horizontal web bars
$f_{yj,d}$	Design value of yield strength of jacket steel
f_{yL}	Yield stress of longitudinal steel bars
f_{yV}	Yield stress of vertical web bars
f_{yw}	Mean yield stress of transverse or confinement reinforcement
h	Depth of cross-section
$h_{b,i}$	Beam depth at the top of storey i
h_w	Depth of the web below the soffit of a slab
k_b	Geometrical correction factor
k_{eff}	FRP effectiveness factor depending on type of FRP material
k_G	Correction factor for design fracture energy (see Formula (8.73))
l_b	Embedment length of the bar into the uppermost member the column is connected to (normally a beam)
$l_{b,0}$	Length of the smooth starter bar connecting the column to the foundation

l_o	Lap length
$l_{o,i}$	Lap length of vertical bars at the base of a column in storey i
$l_{oy,min}$	Lap length beyond which the yield chord rotation is not reduced due to lap splicing
$l_{ou,min}$	Lap length beyond which the ultimate chord rotation is not reduced due to lap splicing
m_c	Number of concrete cores
n	Number of longitudinal bars laterally restrained by a stirrup corner or hook along the perimeter of the cross-section
p_c	Confining pressure on the lap splice; $A_{sw}f_{yw}/(s_w R_c)$ for confinement by steel ties; $t_f f_{u,f}/R$ for confinement by a FRP jacket of thickness t_f
s	Centreline spacing of stirrups
s_f	Centreline spacing of FRP (fibre-reinforced polymer) strips (= w_f for FRP sheets)
s_h	Spacing of horizontal wall reinforcement
s_v	Spacing of vertical wall reinforcement
s_w	Spacing of transverse reinforcement
t_f	Thickness of FRP (fibre-reinforced polymer) sheet
t_j	Thickness of steel jacket
x	Compression zone (neutral axis) depth
w_f	Width of FRP strip or sheet, measured orthogonally to the (strong) direction of the fibres
z	Length of section internal lever arm
z_i	Internal lever arm of a column in storey i

Upper case Greek symbols

Γ_{Fd}	Design fracture energy
$\Delta\theta_{u,slip}$	Post-yield fixed-end rotation due to yield penetration in the anchorage zone beyond the yielding end of the member

Lower case Greek symbols

α	Confinement effectiveness factor
β	Angle between the (strong) fibre direction in the FRP strip, sheet or fabric and the axis of the member
β_f	Confinement coefficient which depends on the type of material
γ_d	Partial factor for FRP (fibre-reinforced polymer) debonding
γ_{Rd}	Partial factor accounting for uncertainty in the ultimate deformation (resistance) which depends on different Knowledge Levels KLG, KLD, and KLM
δ	Angle between the diagonal and the axis of a column $\varepsilon_{co,c}$ concrete strain at f_c , which should be taken equal to 0,002
ε_{cu}	Unconfined concrete ultimate strain
$\varepsilon_{cu,c}$	Confined concrete ultimate strain for rectangular, circular or triangular compression zone
$\varepsilon_{f,ed}$	Maximum allowable strain in the FRP (may be taken equal to 0,005)

ε_{su}	Ultimate strain of tension reinforcement
$\varepsilon_{su,nom}$	Uniform elongation at tensile strength in a standard steel coupon test
$\varepsilon_{su,w}$	Ultimate strain of confinement reinforcement
$\varepsilon_{su,ribbed,laps}$	Reduced ultimate strain of extreme tension ribbed bars due to lap-splicing
$\varepsilon_{u,f}$	FRP (fibre-reinforced polymer) ultimate strain
θ	Chord rotation strut inclination angle in shear design
θ_{cr}	Angle of the diagonal crack on a wall
θ_y	Chord rotation at yielding of concrete member
$\theta_{y,slip}$	Chord rotation at the top and bottom end of a column in storey i at yielding of concrete member
θ_u	Ultimate chord rotation of concrete member
θ_u^{pl}	Plastic part of chord rotation
$\theta_{u,ribbed,laps}^{pl}$	Reduced plastic part of ultimate chord rotation due to lap-spliced ribbed reinforcement
θ_{u0}^{pl}	Basic value of plastic part of chord rotation
κ_{axial}	Correction factor of ultimate chord rotation of concrete member for an axial force other than 0
$\kappa_{concrete}$	Correction factor of ultimate chord rotation of concrete member for concrete strength different from 25 MPa
$\kappa_{confinement}$	Correction factor of ultimate chord rotation of concrete member for confinement of concrete due to transverse bars
$\kappa_{conform}$	Correction factor of ultimate chord rotation of concrete member for non-conformity with seismic design rules based on ductility: 1 if it conforms, 0,78 otherwise
κ_{reinf}	Correction factor of ultimate chord rotation of concrete member for asymmetrical reinforcement
$\kappa_{shearspan}$	Correction factor of ultimate chord rotation of concrete member for a shear span different from 2,5
$\lambda_{section}$	Correction factor of plastic hinge length for section width
$\lambda_{shearspan}$	Correction factor of plastic hinge length for shear span
λ_{axial}	Correction factor of plastic hinge length for axial load
μ_{Δ}	Displacement ductility factor
μ_{Δ}^{pl}	Plastic part of displacement ductility factor
ν	$= N/bhf_c$ (b width of compression zone)
ρ	Tensile reinforcement ratio in reinforced concrete members
ρ_d	Steel ratio of diagonal reinforcement
ρ_f	Volumetric ratio of FRP (fibre-reinforced polymer)
ρ_s	Geometric steel ratio
ρ_{tot}	Total longitudinal reinforcement ratio

ρ_{sv}	Volumetric ratio of transverse reinforcement defined as the volume of confining hoops over the volume of concrete core
ρ_{sw}	Ratio of transverse reinforcement in reinforced concrete members
ρ_v	Vertical web reinforcement ratio
ρ_h	Horizontal web reinforcement ratio
φ	Angle of inclined bars to the base section of a concrete wall
ϕ_u	Ultimate curvature calculated from section analysis at end section
ϕ_y	Yield curvature calculated from section analysis at end section
ω, ω'	Mechanical reinforcement ratio of tension and compression reinforcement

3.2.1.2 Symbols used in Clause 9

Upper case Latin symbols

A	Gross cross-sectional area of a structural steel member
A_b	Gross cross-sectional area of a structural steel bolt
A_c	Gross cross-sectional area of a steel column
A_e	Effective net cross-sectional area
A_g	Gross cross-sectional area
A_v	Effective cross-sectional shear area
E	Modulus of elasticity of structural steel material
$F\left(\frac{a_0}{t_{fu}}\right)$	Fracture parameter
G	Shear modulus of structural steel material
I_b	Moment of inertia of the bare steel cross section about the axis of bending
K_e	Elastic stiffness of a structural member for linear analysis
K_{Ic}	Fracture toughness parameter
K_θ	Elastic stiffness of the semi-rigid partial-strength beam-to-column joint
L	Centerline length of the steel beam between joints
L_b	Unbraced length of the steel beam
$L_{gp,b}$	Average unrestrained buckling length of the gusset plate of the bracing-end connection
L_v	Shear span
$M_{Ed,y}$	Bending moment in strong axis bending in the seismic design situation
$M_{Ed,z}$	Bending moment in weak axis bending in the seismic design situation
M_y^*	Effective flexural resistance at yield
M_y^{*+}	Effective flexural resistance at yield of a composite-steel beam under sagging
M_y^{*-}	Effective flexural resistance at yield of a composite-steel beam under hogging
M_u^*	Effective flexural resistance at ultimate
M_u^{*+}	Effective flexural resistance at ultimate of a composite-steel beam under sagging

M_u^{*-}	Effective flexural resistance at ultimate of a composite-steel beam under hogging
N_{Ed}	Axial load in the seismic design situation
$N_{b,e}$	Effective buckling resistance of a structural member
$N_{Ed,G}$	Axial load in the column due to non-seismic actions in the seismic design situation
N_{pl}	Tensile resistance of structural steel member
$N_{pl,e}$	Expected axial resistance at yield of structural steel member
Q_y^*	Effective resistance at yield
Q_r^*	Residual resistance
Q_u^*	Effective resistance at ultimate
Q_{NC}	Effective resistance at ultimate
Q_R^*	Effective resistance at yield or at ultimate, whichever is applicable
V_y^*	Effective shear resistance at yield
W_{el}	Elastic cross-sectional modulus
$W_{el,y}$	Elastic cross-sectional modulus in y-y axis bending
$W_{el,z}$	Elastic cross-sectional modulus in y-y axis bending
W_d	Effective width of gusset plate at the hinge zone

Lower case Latin symbols

a_0	Dimension of the smaller flange or web thickness that is not welded
a_h	Steel material post-yield hardening ratio
b_a	Distance of bolt centerline to bottom end of the T-stub flange
b_t	Distance between one row of fasteners in the T-stub flange (or in the split-tee flange) and the centreline of the stem
c	Width or depth of a part of a cross section
d	Depth of the edge of hollow structural steel cross section parallel to the direction of the seismic action
d_1	Distance from the centre of the split-tee stem to the edge of the split-tee flange fillet
f_y	Nominal yield strength of structural steel material
f_u	Lower-bound ultimate tensile strength of structural steel material
$f_{u,b}$	Nominal ultimate tensile strength of rivet or bolt
h	Full depth of steel cross section
i_z	Cross-sectional radius of gyration of a steel beam about its z-z axis
n_b	Least number of connecting bolts between two steel members
q	Behaviour factor
s_γ	Shape factor accounting for uncertainty in the deformation capacity at the limit state of Near-Collapse for links in eccentric bracings
t	Thickness
t_a	Thickness of a seat angle

t_f	Flange thickness
$t_{f,u}$	Thickness of the flange or web of the smaller member in a spliced connection
t_p	Thickness of gusset (or flange) plate
t_s	Thickness of the T-stub stem
t_w	Web thickness
w	Length of the flange angle or split tee

Lower case Greek symbols

γ_{Rd}	Partial factor accounting for uncertainty in the ultimate deformation (resistance) which depends on different Knowledge Levels KLG, KLD, and KLM
δ_y	Deformation at yield
δ_{DL}	Deformation capacity corresponding to the limit state Damage Limitation (DL)
δ_{NC}	Deformation capacity corresponding to the limit state Near-Collapse (NC)
δ_c^{pl}	Plastic deformation from ultimate to collapse
δ_u	Deformation at ultimate
δ_u^{pl}	Plastic deformation from yield to ultimate
δ_c	Deformation at collapse (loss of load carrying capacity of a member)
θ_y	Chord rotation over the member's length at yield
θ_c	Chord rotation over the member's length at collapse
θ_c^{pl}	Plastic rotation at post-ultimate for flexural hinges of a steel beam
θ_u^{pl}	Plastic rotation from yield to ultimate for flexural hinges of a steel beam
$\theta_u^{pl,+}$	Plastic rotation from yield to ultimate of a composite-steel beam under sagging
$\theta_u^{pl,-}$	Plastic rotation from yield to ultimate of a composite-steel beam under hogging
σ_{Ed}	Normal stress effect
σ_{cr}	Nominal tensile strength of splices made with partial joint penetration groove welds
ω_{rm}	Material randomness factor

3.2.1.3 Symbols used in Clause 10 and Annex C

Upper case Latin symbols

A	Diaphragm gross area
A_n	Diaphragm net area
B	Diaphragm width with respect to in-plane loads
E_m	Elastic modulus of uncracked masonry
F_a	Inertial force on diaphragm
G_{d0}	Diaphragm equivalent in-plane shear stiffness
$G_{d0,eff}$	Diaphragm effective equivalent in-plane shear stiffness accounting for condition assessment
L_a	Diaphragm span with respect to in-plane loads

$N_{c,I,d}$	Design normal compressive loading in the rafter in respect with the front heel
$N_{c,II,d}$	Design normal compressive loading in the rafter in respect with the rear heel
$N_{c,d}$	Design normal compressive loading in the rafter
$N_{c,i,d}$	Design normal compressive loading in the rafter according to each Double Step Joint heel
$N_{c,max,d}$	Maximal design normal compressive loading in the rafter of the Double Step Joint
$N_{c,tot,d}$	Total design normal compressive loading in the rafter of the Double Step Joint
T_{an}	Diaphragm in-plane natural period
<i>Lower case Latin symbols</i>	
b	Tie beam width
dr	Diaphragm in-plane drift
f_a	Inertial force per unit span length on diaphragm
$f_{c,0,k}$	Characteristic compressive strength parallel to the grain, in N/mm ²
$f_{c,90,k}$	Characteristic compressive strength perpendicular to the grain, in N/mm ²
$f_{c,\alpha,k}$	Characteristic compressive strength at an angle α to the grain, in N/mm ²
$f_{c,0,mean}$	Mean compressive strength parallel to the grain, in N/mm ²
$f_{c,90,mean}$	Mean compressive strength perpendicular to the grain, in N/mm ²
$f_{c,\alpha,mean}$	Mean compressive strength at an angle α to the grain, in N/mm ²
$f_{v,mean}$	Mean shear strength of timber parallel to the grain, in N/mm ²
g	Acceleration of gravity
h_i	Lower inter-story height
h_s	Upper inter-story height
l_v	Shear length
$l_{v,eff}$	Effective shear length taking into account the extent of the non-uniform shear stress distribution along the grain at the heel depth in the tie beam
$k_{c,90}$	Factor taking into account for the load configuration, possibility of splitting and degree of compressive deformation
k_{cr}	Reducer factor of the beam width by considering the impact of cracks on the shear strength
k_{mod}	Modification factor for duration of load and moisture content
$k_{v,red}$	Reduction coefficient taking into account the non-uniform shear stress distribution at the heel depth along the grain in the tie beam
m_{ap}	Diaphragm tributary seismic mass
q_{ap}	Diaphragm in-plane behaviour factor
$t_{ef,1}$	Effective length in the rafter side
$t_{ef,2}$	Effective length in the tie beam side
t_i	Out-of-plane loaded masonry wall thickness, for the level beneath the diaphragm
t_s	Out-of-plane loaded masonry wall thickness, for the level above the diaphragm
t_v	Heel depth

v_{Ed}	Design in-plane unit shear force on diaphragm
v_{Rd}	Design diaphragm in-plane unit shear strength
$v_{R,k}$	Diaphragm characteristic value of in-plane unit shear strength

Upper case Greek symbols

$\Delta_{max,i}$	Maximum diaphragm in-plane deflection
------------------	---------------------------------------

Lower case Greek symbols

α	Angle between a force and the direction of the grain
α_m	Modification factor accounting for the stiffness contribution of out-of-plane loaded walls on diaphragm equivalent in-plane shear stiffness
β	Rafter skew angle
γ	Inclination angle of the bottom-notch surface according to the grain
γ_M	Partial factor for material properties
θ	Inclination angle of the front-notch surface to the normal of the grain in the tie beam
μ_d	Diaphragm in-plane ductility factor
φ	Condition assessment factor

3.2.1.4 Symbols used in Clause 11 and Annex D

Upper case Latin symbols

D	In-plane horizontal dimension of the wall, length in the case of piers and depth for spandrels
D'	Depth of the compressed area at the end section of the pier
E	Elastic modulus of masonry
F^*	Total horizontal force of the equivalent SDOF system, for partial out-of-plane mechanisms
F_{DL}^*	Resistance at DL Limit State, in terms of force, in the equivalent SDOF system, for partial out-of-plane mechanisms
F_y	Horizontal resistance, in terms of force, of the equivalent SDOF system, for partial out-of-plane mechanisms
G	Shear modulus of masonry
G_k	Permanent actions (self-weights) carried by the k -th block, applied at the centre of gravity of the block
G_d	Equivalent shear modulus of the shell that models the horizontal diaphragm
H	Length of the masonry member, height of the pier and span of the spandrel
H_0	Distance between the section where the flexural resistance is attained and the contraflexure point
$H_{i(j)}$	Distance between the section i (j) and the contraflexure point
K_d	In-plane stiffness of floor system
L_i	Total work of internal forces in the connections between blocks, for the infinitesimal activation of the partial out-of-plane mechanism
N	Number of blocks in the kinematic chain of a partial out-of-plane mechanism

N_S	Minimum between the tensile resistance of the horizontal member attached to the spandrel and the compressive resistance of the diagonal equivalent masonry strut
P_u	Diagonal force at failure in diagonal compressive tests on masonry panels
$Q_{1,k}$	Variable actions $Q_{1,k}$ carried by the k -th block, applied at the centre of gravity of the block
$Q_{2,k}$	Sum of all variable actions not carried by the k -th block but generating on it a horizontal seismic force proportional to α
$Q_{3,k}$	Sum of all variable actions applied to the k -th block, which do not generate seismic forces
S_{De}	Elastic displacement response spectrum
S_e	Elastic acceleration response spectrum
S_{eZ}	Elastic acceleration floor response spectrum, at level z in the building
T_0	Initial period, for partial out-of-plane mechanisms
T_1	First period of vibration of a building, in the direction of verification of the partial out-of-plane mechanism
T_{SD}	Linear equivalent period for the evaluation of the displacement demand at SD limit state, of the equivalent SDOF system, for partial out-of-plane mechanisms
T_{NC}	Linear equivalent period for the evaluation of the displacement demand at NC limit state, of the equivalent SDOF system, for partial out-of-plane mechanisms
V_d	Shear resistance of a masonry member failing in diagonal cracking
$V_{d,lim}$	Shear resistance of masonry members due to failure of units
V_f	Shear resistance of a masonry member failing in flexure
V_s	Shear resistance of a masonry member failing in shear sliding
$V_{s,units}$	Limitation of V_s due to the failure of masonry units
V_R	Shear resistance of masonry members
Z	Parameter of the force-deformation relationship in the case of hybrid failure mode
Z_f	Parameter of the force-deformation relationship in the case of flexural failure
$Z_{s/d}$	Parameter of the force-deformation relationship in the case of shear sliding or diagonal cracking failure
X_1, X_2, X_3	Elastic moduli (E or G) of the two external wythes and of the inner core, in the case of three-leaf masonry

Lower case Latin symbols

a_z	Peak floor acceleration at level z , from acceleration floor spectra
b	Correction coefficient related to the shear stress distribution in the middle section of the panel, related to the aspect ratio of the panel
c	Coefficient of restitution of the partial out-of-plane mechanism
$c_{p,w}$	Correction factor for masonry wall w , which takes into account torsion effects
$c_{E,w,i}$	Correction factor for masonry wall w at level i , which takes into account torsion effects and higher mode effects in elevation
d	Target displacement evaluated by the equivalent bilinear SDOF system, result of the transformation of the pushover curve from non-linear static analysis

d_c	Displacement of the reference control point of the partial out-of-plane mechanism
d_{c0}	Displacement of the reference control point of the partial out-of-plane mechanism for which the horizontal multiplier is reduced to zero
d_u	Ultimate displacement capacity of the equivalent SDOF system, derived from the pushover curve obtained by the non-linear static analysis; Value of the displacement of the reference control point of the partial out-of-plane mechanism corresponding to a 40% drop of the horizontal seismic multiplier
d_{u2}	Value of the displacement of the reference control point of the partial out-of-plane mechanism corresponding to 60% drop of the horizontal seismic multiplier
d^*	Displacement of the equivalent SDOF system
d_{DL}	Displacement of the equivalent SDOF system at DL Limit State
d_{SD}	Displacement of the equivalent SDOF system at SD Limit State
d_{NC}	Displacement of the equivalent SDOF system at NC Limit State
d_y	Displacement at the yield point of the equivalent bilinear SDOF system, evaluated until the displacement corresponding to the Limit State of NC; Displacement of the equivalent SDOF system at maximum resistance, for partial out-of-plane mechanisms
d_{et}	Displacement of the control node obtained by linear analysis, with consideration of accidental torsion effects
$d_{et,w}$	Average displacement of the wall w at the level where the control displacement is defined, in order to consider the torsion effects
d_t	Target displacement of the control node obtained by non-linear static analysis
$d_{t,w}$	Displacement of the wall w at the level where the control displacement is defined, obtained by non-linear static analysis corresponding to the target displacement of the control node
f	Vertical compressive strength of masonry
f_b	Normalised compressive strength of masonry units
f_{bt}	Tensile strength of masonry units
f_h	Horizontal compressive strength of masonry
f_{ht}	Horizontal tensile strength of masonry at the end section of the spandrel member
f_t	Diagonal tensile strength of masonry
f_v	Shear strength of masonry
f_{v0}	Initial shear strength of modern masonry and regular pre-modern masonry, in the case of shear sliding failure; assumed as representative of the cohesion of the mortar joint
$\tilde{f}_{v0}$	Initial shear strength of pre-modern regular masonry, in the case of diagonal cracking failure
f_{LS}	Factor for amplifying the deformation demand in masonry members, in the case of linear analysis
g	Acceleration of gravity
h_i	Interstorey height at the storey below level i
l_d	Distance between two opposite walls of a floor area (diaphragm)
l_r	Rise of vault
l_v	Span of vault

m	Number of external forces, not related to the seismic action, applied to the blocks of the partial out-of-plane mechanism
m^*	Mass of equivalent SDOF system, for partial out-of-plane mechanisms
n	Number of direct tests carried out for the Bayesian updating of material properties
q_{LM}	Behaviour factor of partial out-of-plane mechanisms
r_i	Rotation at the end section i of the masonry member
$\bar{s}$	Standard deviation from experimental results of the material property (strength or stiffness), for the updating of the a-priori distribution by the Bayesian approach
t	Thickness of a masonry member (wall, pier, spandrel)
t_1, t_3	Thickness of the two external wythes, in the case of three-leaf masonry
t_2	Thickness of the inner core, in the case of three-leaf masonry
t_d	Conventional thickness of the shell that models the horizontal diaphragm
u_0	Lateral displacement at the contraflexure point of the masonry member
u_i	Lateral displacement at the end section i of the masonry member
u_{LS}	Limit value of the ratio between the spectral acceleration causing yielding of the equivalent SDOF model and the spectral acceleration seismic action effect, for the limit states of SD and NC
$u_{w,i}$	Average horizontal displacement of the wall w at level i
w	Unit weight of masonry
z	Vertical position (height) in the building, for acceleration floor spectra
$\bar{x}$	Mean value from the experimental results of the material property (strength or stiffness), for the updating of the a-priori distribution by the Bayesian approach

Upper case Greek symbols

Γ	Transformation factor, for partial out-of-plane mechanisms
----------	--

Lower case Greek symbols

α	Horizontal seismic multiplier for partial out-of-plane mechanisms, ratio of the applied horizontal inertial actions to the corresponding permanent dead loads
α_0	Horizontal seismic multiplier that activates partial out-of-plane mechanisms
$\delta_{GQ1y,k}$	Vertical displacement of the centre of gravity of the permanent actions G_k and variable actions $Q_{1,k}$, applied to the k -th block, for an infinitesimal activation of the mechanisms
$\delta_{GQ12x,k}$	Horizontal displacement of point of application of the resultant of horizontal forces $\alpha(G_k + Q_{1,k} + Q_{2,k})$ applied to the k -th block, in partial out-of-plane mechanism
$\delta_{Q3,k}$	Displacement of the point of application of the variable actions $Q_{3,k}$, in the direction of the force itself, evaluated for an infinitesimal activation of the mechanisms
ε_r	Normal deformation at compressive failure of the weakest of the two external wythes, in the case of three-leaf masonry
ε	Testing error related to the measurement of material properties
ϕ	Interlocking coefficient for a regular pattern masonry, defined as the ratio between the height of the masonry units and the length of overlapping (operationally, it is the tangent of the average inclination of the possible stair-stepped cracks)

κ	Coefficient taking into account testing error and variability in the building of material properties
κ'	Coefficient taking into account the accuracy of testing procedure (experimental test and correlation with the mechanical property)
κ''	Coefficient taking into account the variability of the mechanical property in the building, with respect to the dispersion within the whole masonry type
λ	Slenderness of the block, for partial out-of-plane mechanisms
μ	Local friction coefficient at the mortar joint
$\bar{\mu}$	Equivalent friction coefficient of pre-modern regular masonry, in the case of diagonal cracking failure
μ'	Mean value of the a-priori distribution of the material property (strength or stiffness) to be updated through experimental results using the Bayesian approach
ν	Normalised axial load
σ_0	Mean axial stress in masonry member: i) for piers, mean vertical stress in the transverse section of the panel; ii) for spandrels, the greater between the mean horizontal stress and the vertical stress, calculated by considering the distributed dead load possibly transferred from the horizontal diaphragms and/or the vertical stresses induced by the adjacent piers
σ_y	Mean vertical compressive stress acting on the horizontal joints at the end sections of the spandrel
σ'	Standard deviation of the a-priori distribution of the material property (strength or stiffness) to be updated through experimental results by the Bayesian approach
θ	Deformation seismic action effect (drift ratio) in a masonry member
$\theta_{d,i}$	In-plane angular deformation of a diaphragm, at level i
θ_e	Member drift ratio
$\theta_{(j)}$	Chord rotation at the end section i (j) of the masonry member
$\theta_{d,u}$	Ultimate chord rotation of the masonry member failing in diagonal cracking
$\theta_{d,u2}$	Second ultimate chord rotation of the masonry member failing in diagonal cracking
$\theta_{f,u}$	Ultimate chord rotation of the masonry member failing in flexure
$\theta_{f,u2}$	Second ultimate chord rotation of the masonry member failing in flexure
$\theta_{s,u}$	Ultimate chord rotation of the masonry member failing in shear sliding
$\theta_{s,u2}$	Second ultimate chord rotation of the masonry member failing in shear sliding
θ_u	Ultimate member drift, corresponding to a drop in the shear force with respect to the peak value
θ_{u2}	Second ultimate member drift, wherein the shear force is further reduced with respect to that at θ_u (by an amount that depends on the failure mechanism) with respect to the maximum shear resistance
$\theta_{w,i}$	Wall interstorey drift ratio between levels $i-1$ and i
θ_y	Member drift at yielding, corresponding to the attainment of the maximum shear strength
θ_{DL}	Member drift at the DL limit state

θ_{SD}	Member drift at the SD limit state
θ_{NC}	Member drift at the NC limit state
$\theta_{et,w,i}$	Interstorey drift ratio $\theta_{w,i}$ of wall w at the i -th storey, obtained by linear analysis, with consideration of accidental torsion effects
$\theta_{et,w,i}$	Interstorey drift ratio $\theta_{w,i}$ of wall w at the i -th storey, obtained by non-linear static analysis, corresponding to the target displacement
ξ_{SD}	Equivalent damping ratio at SD limit state, of the equivalent SDOF system, for partial out-of-plane mechanisms
ξ_{NC}	Equivalent damping ratio at NC limit state, of the equivalent SDOF system, for partial out-of-plane mechanisms
ζ	Coefficient for the interpolation of parameters of the masonry member force-deformation relationship in the case of hybrid mode of failure

3.2.2 Abbreviations

AFRP	Aramid Fibre Reinforced Polymer
c.o.v.	coefficient of variation
CFRP	Carbon Fibre Reinforced Polymer
CJP	Complete Joint Penetration groove welds
CLT	Cross-laminated timber
CVN	Charpy-V Notch impact testing
DL	Damage Limitation limit state
DSJ	Double Step Joint
FRP	Fibre Reinforced Polymer
GFRP	Glass Fibre Reinforced Polymer
KL	Knowledge level
KLD	Knowledge Level on Details
KLG	Knowledge Level on Geometry
KLM	Knowledge Level on Materials
LS	Limit State
LVL	Laminated veneer lumber
NC	Near Collapse limit state
NDT	Non-Destructive Technique
NS	NDT or SDT, depending on the testing methodology used
OSB	Orientated strand board
PAF	Polyacetal Fibre
PJP	Partial-Joint Penetration groove welds
RC	Reinforced concrete
SD	Significant Damage limit state

SDOF	Single Degree of Freedom system
SDT	Semi-Destructive Technique
SSJ	Single Step Joint
URM	Unreinforced Masonry

3.3 S.I. Units

S.I. Units in accordance with ISO 80000 shall be used.

For calculations, the following units should be used when applicable:

- forces and loads: kN, kN/m, MN/m
- length, displacement: m, mm
- unit mass: kg/m³, t/m³
- mass: kg, t
- weight density: kN/m³
- stresses and strengths: Pa (=N/m²), kPa (kN/m²), MPa (= MN/m²)
- moments (bending, etc.): kNm
- acceleration: m/s²

4 Basis of design

4.1 Performance requirements

(1) The performance requirements shall refer to the state of damage in the structure, herein described through the Limit States (LS) defined in prEN 1998-1-1:2022, 4.3(1).

(2) A seismic action should be associated with each Limit State to be verified. This seismic action should be characterised by its return period $T_{LS,CC}$ according to prEN 1998-1-1:2022, 4.3(3), or, alternatively, by a performance factor $\gamma_{LS,CC}$ according to prEN 1998-1-1:2022, 4.3(5).

NOTE The minimum values to be ascribed to $T_{LS,CC}$ or, alternatively, to $\gamma_{LS,CC}$ for each type of selected existing structure can be provided by the relevant authorities or can be found in the National Annex. They can be lower than those used for new structures, if lower values of $\beta_{t,LS,CC}$ are accepted for existing structures compared to those specified for new structures. Lower values of $\beta_{t,LS,CC}$ in the service life t_L can reflect a shorter residual service life of an existing structure.

(3) The seismic performance of the structure should be verified for the full set or a subset of the four Limit States; as a minimum, the Near Collapse LS should be verified.

NOTE 1 Since existing structures in general do not possess the adequate ductility ensured in new ones by means of capacity design and detailing for local ductility, verification of the LS of Significant Damage for a certain intensity does not necessarily imply verification of the LS of Near Collapse for a higher one.

NOTE 2 The types of structures to which this standard applies and the choice of the Limit States to be verified for each type of existing structure can be provided by the relevant authorities or can be found in the National Annex, and they can be different from those used for new structures.

(4) In application of prEN 1998-1-1:2022, 6.1(4), in the cases of low and moderate seismic action class (prEN 1998-1-1:2022, 4.1(4)), the seismic performance of structures may be verified by means of the force-based approach, using the q values in 6.3.1(1) and the seismic action for the Near Collapse LS.

NOTE The force-based approach relies on global ductile behaviour and redistribution capacity of the structure. Its application to existing structures is thus approximate and only reduced values of q , corresponding to member overstrength, can be used. As a result, the method is conservative. If the outcome of the verification is negative, this can be due to this conservatism, rather than to an actual deficit of the structure. The state of the structure can be re-assessed via a more accurate displacement-based approach to calibrate the retrofit design.

4.2 Compliance criteria for existing structures

4.2.1 Specificity of existing structures

(1) The structural model should be validated against observed damage (if present), before using it for the verification of seismic resistance.

NOTE Repairing and retrofitting or upgrading of seismically damaged structures has specific features both in terms of assessment procedures and intervention techniques.

(2) The effectiveness of retrofitting interventions should be verified in the case of application to masonry walls with severe cracks that affect the original interlocking among members.

(3) When designing a structural intervention for resistance against seismic actions, structural verifications should also be made with respect to non-seismic situations.

(4) Different sets of material and structural partial factors should be used, as well as different analysis procedures, depending on the completeness and reliability of the information available.

NOTE Since existing structures: (i) reflect the state of knowledge at the time of their construction, (ii) possibly contain hidden gross errors, (iii) have possibly been submitted to previous earthquakes or other accidental actions with possibly unknown effects, (iv) have possibly been modified during their service life in a way that can affect their structural performance, structural evaluation and possible structural intervention are typically subjected to a higher degree of uncertainty (reduced level of knowledge) than the design of new structures.

4.2.2 Verification rules

(1) Compliance with 4.1 shall be achieved by adoption of the seismic action, modelling, method of analysis, and verification procedures contained in this standard, in complement to prEN 1998-1-1 and prEN 1998-5, as appropriate for the different structural materials (i.e. concrete, steel, timber, masonry) and type of structures (buildings and bridges) within its scope.

(2) A certain number of the existing structural members may be designated as secondary seismic members, in accordance with the definitions in prEN 1998-1-2:2023, 4.4.2, for buildings and prEN 1998-2:2023, 4.3.2, for bridges.

(3) In bridges, a certain number of the secondary seismic members may be designated as “sacrificial”. These members may be neglected in modelling and verification and may be heavily damaged, so long as their failure mode does not endanger life safety and the integrity of other structural members and allows for the possibility of local repair.

NOTE See 12.3.2 for criteria for selecting and examples of such members.

(4) Unless higher level (reliability) methods are employed (prEN 1998-1-1:2022, Annex F), safety verifications should be carried out using the partial factor method, according to EN 1990:2023, 8.

(5) Design values of action effects (E_d) should be expressed according to EN 1990:2023, 8.3.2.1(1), including all relevant actions for the seismic combination (permanent, variable, seismic). For the application of this general expression in the present Eurocode part, the following definitions of symbols should be applied.

γ_{sd} is a partial factor considering uncertainty in modelling the action effects, depending on the state of the structure, and being equal to 1,0 for undamaged structures, and 1,15 otherwise;

A_{Ed} is the design value of the seismic action, appropriate for the Limit State to be verified (see 4.1(3)).

NOTE 1 γ_{sd} accounts for the lower reliability of analysis results obtained when modelling structures affected by aging, deterioration and possibly damaged by a previous seismic event (e.g. when assessment is part of retrofitting after an earthquake).

(6) Design values of the resistance (R_d) should be expressed according to the general expression given in EN 1990:2023, 8.3.5.1(1). For the application of this general expression in the present standard, the following definitions of symbols should be applied.

γ_{Rd} is a partial factor accounting for uncertainty in the resistance (strength or deformation) model, calculated according to note 2 of prEN 1998-1-1:2022, 6.7.2(1);

$X_{d,i}$ is the design value of the i -th material or product property. For existing materials X_d is obtained from tests of *in situ* properties and from additional sources of information, according to 5.5(1) to (3). For added materials, design values are obtained according to 5.5(4) and (5), respectively.

NOTE 2 The partial factor γ_{Rd} accounts for: a) target reliability; b) uncertainty in the relevant variables describing geometry, details and material properties entering the resistance model, including their statistical uncertainty of estimation based on limited sample size, plus the model error of the resistance model itself. The values of logarithmic standard deviation as a function of KL are given in this standard.

(7) The verifications of design values of action effects derived from (5) against the design value of corresponding resistances derived from (6) should be performed in terms of either generalised stresses or generalised deformations, depending on the type of approach (force-based or displacement-based) and failure mode.

NOTE 1 Appropriate criteria are given in Clause 6.

NOTE 2 While the outcome of safety verifications is binary (E_d is either higher, or equal/lower than R_d) reporting the ratio of the action effect to the resistance for each verification can be useful for the selection of the intervention scheme when needed.

(8) Verifications against brittle mechanisms and instabilities shall be performed with an appropriate reliability.

(9) (8) may be considered satisfied when the resistance to non-brittle mechanisms which limits the action effects to be considered (in capacity design) in calculating the resistance to brittle mechanisms duly accounts for overstrength. If hardening is accounted for in non-linear analysis (with values of the post-yield to pre-yield stiffness ratio not lower than 5%, unless properly justified), the overstrength factors given in prEN 1998-1-1:2022, 7.2.4 and 7.3.1, may be reduced.

NOTE 1 Relevant criteria are given in Clauses 6 and 8 to 12.

NOTE 2 Overstrength factor (see prEN 1998-1-1) accounts for unfavourable deviations of material properties (e.g. f_y) from mean values and for hardening.

(10) Design values of action effects (E_d) and of the corresponding resistances (R_d) may be compared in either local or global terms, depending on the modelling and method of analysis adopted, as specified in Clauses 6 and 8 to 12.

4.2.3 Verification of Limit States

4.2.3.1 General

(1) For each Limit State considered in the verification, action effects should be calculated using one of the methods given in Clause 6 for the seismic action defined in 4.1(2). Resistances should be based on requirements given in 4.2.3.2 to 4.2.3.5 and 5.5, using the relevant criteria given in Clauses 6 and 8 to 12.

4.2.3.2 Limit State of Near Collapse (NC)

(1) Resistances should be based on appropriately defined ultimate conditions in terms of strength or deformation parameters, depending on the type of verification to be carried out, accounting for the effect of strength degradation under cyclic loading.

(2) When this limit state is exceeded, it should be reported whether the loss of bearing capacity of the concerned member has the potential to escalate into a global collapse or it is deemed to remain confined in a partial localised collapse. When exceedance of this limit state produces a localised partial collapse, the analysis may also be repeated on the assumption that this exceedance is prevented by structural intervention, and both results should be reported.

4.2.3.3 Limit State of Significant Damage (SD)

(1) Resistances should be based on appropriately defined conditions of damage between the elastic limit and the ultimate state according to prEN 1998-1-1:2022, 6.7.2(1), in terms of strength or deformation parameters, depending on the verification to be carried out.

(2) If this limit state is verified in lieu of the Near Collapse one (see 4.1(3)), then the q -factor approach (see 6.4.2) may be used.

4.2.3.4 Limit State of Damage Limitation (DL)

(1) Resistances should be based on elastic limit (e.g. yield, for RC or steel structures) values for all structural members for both ductile and brittle failure mechanisms, as well as proper limits for ancillary components, including infills in buildings.

(2) Deformation limits for infills in buildings should be as given in prEN 1998-1-2:2023, 7.4.2.1, as appropriate.

4.2.3.5 Fully Operational Limit State (OP)

(1) Resistances should be based on damage of ancillary elements.

NOTE For a specific project, the relevant parties can specify all ancillary components of interest in the verification, together with a description of relevant damage states for each component and the associated requirements.

(2) The Fully Operational LS may be considered verified if the provisions in prEN 1998-1-1:2022, 6.7.3(7), and in prEN 1998-1-2:2023, 6.3.3(1) (for buildings) or in prEN 1998-2:2023, 6.4.3 (for bridges), are complied with.

4.3 General procedure for the assessment and retrofitting design

4.3.1 Seismic assessment in the current state

(1) Information for the structural assessment should be collected, according to 5.2 for input data and in 5.3 and 5.4 for Knowledge Levels. After surveys of geometry (see 5.4.1), a preliminary analysis, as described in 5.4.2 and Annex A, may be used for a) and b):

- a) to identify critical portions of the structure, wherein further investigations should be concentrated;
- b) to carry out a preliminary assessment of the seismic resistance, in order to decide whether to proceed to a detailed seismic assessment of the structure in its current state or to go directly to the design of retrofitting interventions.

(2) Investigations on details (see 5.4.3) and experimental tests on materials (see 5.4.4) should be carried out, consistently with different possible Knowledge Levels. Representative values for material properties should be as defined in 5.5.

(3) A structural model should be set up, in accordance with prEN 1998-1-1:2022, 6.2, prEN 1998-1-2:2023, 5.1, for buildings, prEN 1998-2:2023, 5.1, for bridges and the additional provisions in 6.3 and specific material-dependent features in 8.3, 9.3, 10.3 or 11.3.

(4) A method of analysis should be chosen (see 6.4), subject to specific conditions for application. In addition to methods of analysis given in prEN 1998-1-1:2022, 6, and in prEN 1998-1-2:2023, 5.3, for buildings and prEN 1998-2:2023, 5.2, for bridges, the distinctive features of existing buildings or bridges should be taken into account.

NOTE Old structures are not designed for a target seismic performance; hence analysis needs to adapt to the actual structural conditions. This applies, in particular, to non-engineered masonry buildings.

(5) Safety verifications should be carried out, as provided for linear (6.4.3) and non-linear (6.4.4) analyses, as well as for the simplified q -factor approach (in 6.4.2); the latter procedure should be used with low values of q .

NOTE The q -factor approach is a largely simplified method for existing structures, as the provided q values are relevant only to SD Limit State, and the basic concept behind the q -factor approach, i.e. that ductility requirements remain essentially constant in all parts of the structure, does not generally apply in the case of existing buildings or bridges.

(6) Resistance models should be used, as provided in the relevant material-dependent clauses, in terms of strength (generalised stresses in structural members, see – 8.4.3, 8.4.4, 9.4, ... 11.4.1.1) and/or deformation (chord rotations, drifts of masonry members, see 8.4.2, 9.4, ... 11.4.1.2).

EDITORIAL NOTE The list of reference will be finalised when Steel and Timber clauses are finalised

(7) Verification criteria for the different materials for the various Limit States should be used, as provided in 8.5, 9.5, 10.7 or 11.5.

4.3.2 Design of retrofitting

(1) Different alternative options may be considered for the retrofitting interventions, as given in a) to c):

- a) retrofitting existing structural members, without a significant change of the structural configuration: for the safety verifications, the same procedure used for the seismic assessment in the current state (see 4.3.1) should be adopted;
- b) implementation of new structural members that resist part of the seismic action effects: for the safety verifications, the same procedure used for the seismic assessment in the current state (see 4.3.1) should be adopted, with the exception that, for the verification of new members, the appropriate partial factors for displacement-based design, as given in EN 1998-1-2 for buildings and EN 1998-2 for bridges, should be used;
- c) design of a new structural system to resist all seismic action effects: the verification of the new primary structure should be made by referring either to prEN 1998-1-1, prEN 1998-1-2 or prEN 1998-2, or to prEN 1998-1-1 and prEN 1998-3; the members of the existing structure should be verified as secondary, with regard to compatibility of deformations induced by the primary structure; specific verifications should be done for the connections between primary and secondary systems.

NOTE In case c), the new structure needs to be stiff enough to limit the deformation of the existing structure.

5 Information for structural assessment

5.1 General information and history

(1) In assessing the earthquake resistance of existing structures, the input data should be collected from a variety of sources, including as given in a) to d):

- a) available documentation specific to the structure in question;
- b) relevant generic data sources (e.g. contemporary codes, standards and documented practice);
- c) field investigations and measurements;
- d) destructive, *in situ* and/or laboratory, and non-destructive tests (that may include *in situ* measurements of dynamic properties), as described in more detail in 5.2 and 5.4.

(2) Field investigations should also aim at identifying all possible threats to life safety posed by ancillary components, such as, e.g., chimneys, cornices, poorly braced equipment, or inadequate configurations (circulation or vulnerable access). These vulnerabilities should be considered in designing retrofitting interventions.

(3) The following features should be assured in order to ensure an appropriate inspection: accessibility, visibility, lighting, cleaning of surfaces.

(4) Cross-checks should be made between the data collected from different sources to minimise uncertainties. In case of conflicting information, *in situ* structure-specific information should be relied upon.

5.2 Required input data

- (1) The information for structural assessment should cover the items defined in a) to i):
- a) Identification of the structural system. The information should be collected either from on-site investigation or from original design and/or construction drawings, if available. In the latter case, information on possible structural changes since construction should also be collected.
 - b) Identification of the type of foundations.
 - c) Identification of the site conditions as defined in prEN 1998-1-1:2022, 5.1.
 - d) Information about the overall dimensions and cross-sectional properties of the structural members and the mechanical properties and condition of constituent materials.
 - e) Information about identifiable material defects and inadequate detailing.
 - f) Information on the seismic design criteria and the level of seismic action used for the initial design. In the case of non-engineered structures, information on the compliance with the rules of practice normally used in the area.
 - g) Description of the present and/or the planned use of the structure (with identification of its consequence class, as described in the relevant part of EN 1998).
 - h) Re-assessment of imposed actions considering the future use of the structure.
 - i) Information about the type and extent of previous and present structural damage, if any, including earlier repair and retrofitting measures.
- (2) Depending on the amount and quality of the information collected according to (1), quantitatively expressed through the corresponding Knowledge Levels defined in 5.3, different methods of analysis, as specified in 6, and values of the partial factors, defined in 4.2.2(6), should be adopted.

5.3 Knowledge levels: Definitions

- (1) The information collected should be classified into three categories defined in a) to c):
- a) *Geometry*: the geometric properties of the structural system and of such ancillary elements (e.g. masonry infill panels) that may affect the structural response;
 - b) *Construction details*: these include, as appropriate, the amount and detailing of reinforcement in reinforced concrete, connections between steel and/or timber members, connections between masonry walls and the nature of any reinforcing elements in masonry, the type of lintels and masonry spandrels, connections of floor diaphragms to the lateral-load resisting structure, etc.;
 - c) *Materials*: the mechanical properties of the constituent materials.

NOTE The investigation of geometric properties extends to all members that affect structural response to the seismic action. This means that some structural members can be excluded, if they can be classified at this stage as secondary, while some ancillary members can be included, like masonry infills, that in general affect stiffness and strength and, especially, when irregularly distributed infills can significantly alter the distribution of action effects.

(2) Amount and quality of the collected information in each category should be expressed through three distinct knowledge levels (KL), as defined in a) to c):

- a) KLG: knowledge level on Geometry, as detailed in 5.4.1;
- b) KLD: knowledge level on Construction Details, as detailed in 5.4.3;
- c) KLM: knowledge level on Material properties (one for each constituent material), as detailed in 5.4.4.

NOTE These KL are used differently.

(3) Each KL defined in (2) should take one of three distinct values, as defined in a) to c):

- a) Minimum.
- b) Average.
- c) High.

(4) Values of KLs may be different in different areas of the structure. Individual values for each category of information may be determined in different areas of the structure.

5.4 Knowledge levels: identification

5.4.1 Geometry

(1) For each type of structural member (beam, column, wall, diaphragm, etc.) and area of the structure, the achieved KL on geometry based on the collected information should be defined, based on Table 5.1.

Table 5.1 — KL on Geometry as a function of collected information

Original design documents (outline or detailed construction drawings)	Extent of survey*		
	L	E	C
Not available	KL G1	KL G2	KL G3
Incomplete set	KL G2	KL G3	
Complete set	KL G3		
* L: limited; E: extended; C: comprehensive (see 3.1.3)			
** For meaning of 1, 2, 3 see 5.4.4.			

(2) If discrepancies between the structural drawings and the survey results are significant, a more extensive dimensional survey should be performed (e.g., from limited to extended), or a lower KLG should be adopted.

(3) For each type of structural member (column, wall, beam, diaphragm, etc.), the minimum percentage of members (reinforced concrete or steel) that should be surveyed for dimensions is given by Formula (5.1), depending on the required extend of survey.

$$p = p_1 n^{-c} \leq 100 \quad (5.1)$$

where

n is the total number n of members of this type in the structure, determined according to (5);

p_1 and c are coefficients which should be taken from Table 5.2 for each level of survey.

Table 5.2 — Minimum requirements for different levels of survey (vertical members)

Level of survey	Limited (L)	Extended (E)	Comprehensive (C)
p_1	200	250	300
c	0,8	0,6	0,5

(4) The values of p_1 and c in Table 5.2 should be used for vertical members; for horizontal members, irrespective of the target KL, Limited survey may be undertaken.

NOTE The level of survey for horizontal members is not taken into account in determining the KL.

- (5) For the purpose of determining the number of members n , a) to c) should be applied:
- a) Floors should be subdivided into basic units through the intersection with basic axes of the structural system (i.e. a pair of perpendicular axes passing through the mass centre of the floor), while reinforced concrete walls should be considered as one segment per storey.
 - b) In the case of large walls, extending over more basic axes, wall segments may be further subdivided into smaller units determined through the intersection with basic axes.
 - c) In the case of masonry buildings, horizontal members (spandrels) and reinforced concrete ring beams should be surveyed if considered in the model.

(6) Conditions of symmetry and repetitiveness should be considered in planning surveys to avoid concentrating efforts on similar members.

NOTE Considering symmetry and repetitiveness means that the target percentage p of members to be inspected is not achieved by inspecting too many similar members.

(7) In buildings, members inspected should cover the entire height.

5.4.2 Preliminary analysis

(1) Once the geometry of the structure is known, a preliminary analysis may be carried out to identify critical portions (e.g. storeys) and members in the structure and inform the planning of inspection of construction details and tests on material properties.

NOTE The value of preliminary analysis in this context is not necessarily the seismic rating that it delivers but the opportunity it provides to gain a holistic view of potential structural weaknesses. The preliminary analysis, carried out with nominal properties, is in general linear elastic for reinforced concrete, steel frame structures and timber structures, and non-linear static for masonry structures. More detail can be found in Annex A.

(2) If a preliminary analysis is carried out according to (1), further investigations on construction details (5.4.3) and material properties (5.4.4) may be limited to, or focus mainly on, the identified critical portions. In the latter case the total number of members n should refer to the number of members in the identified critical portions.

5.4.3 Construction details

(1) Destructive inspection methods should in general be preferred. Reliable non-destructive methods may also be adopted in the inspections. Calibration against destructive methods should be carried out to an extent depending on the non-destructive method.

NOTE Cover removal (to inspect the reinforcement) is an example of destructive method.

(2) For each type of structural member (beam, column, wall, diaphragm, etc.) and area of the structure, the achieved KL on Construction Details based on the collected information should be defined, based on Table 5.3.

Table 5.3 — KL on Construction Details as a function of collected information

Original design documents (detailed structural drawings)	Inspections*		
	L	E	C
Not available	KLD1	KLD2	KLD3
Incomplete set	KLD2	KLD3	
Complete set	KLD3		
* L: limited; E: extended; C: comprehensive (see 3.1.5).			
** For meaning of 1, 2, 3 see 5.4.4.			

(3) 5.4.1(3) should be applied.

(4) In buildings, members inspected should cover the entire height, unless focusing on an identified critical portion is justified based on a preliminary analysis (5.4.2(2)).

5.4.4 Materials

(1) The knowledge levels concerning the properties of materials in the structure should be classified according to definitions a) to c):

- a) KLM1 (Minimum Knowledge) is attained when no direct information on the mechanical properties of the construction materials is available, either from original design specifications or from original test reports. Default values should be assumed in accordance with standards at the time of construction, accompanied by limited *in situ* testing in the most critical members. In the case of masonry structures, direct testing may be avoided, and reference values of predefined masonry types may be attributed after an extended visual survey of masonry features (according to Table 5.1). In the case of timber buildings and timber members, direct testing may be avoided provided that an accurate visual inspection is performed according to 10.2.4.1;
- b) KLM2 (Average Knowledge) is attained when information on the mechanical properties of the construction materials is available either (i) from extended *in situ* testing; or (ii) from original design specifications complemented by limited *in situ* testing. In the case of masonry structures, when original design documents are not available, direct testing may still be avoided, but, in addition to what is required for KLM1, the knowledge should be enhanced by extended non-destructive testing, as specified in Table 5.3 for inspections, which allows a more accurate classification of masonry types in the structure. In the case of pre-1940 timber buildings, when original design documents are not available, direct testing may be avoided, but, in addition to what is required for KLM1, the knowledge should be enhanced by non-destructive testing, as specified in Table 10.1;

NOTE 1 In case of masonry and pre-1940 timber structures, original design documents are rarely available.

- c) KLM3 (High Knowledge) is attained when information on the mechanical properties of the construction materials is available either (i) from comprehensive *in situ* testing; or (ii) from original test reports, complemented by limited *in situ* testing; or (iii) from original design specifications, complemented by extended *in situ* testing. In the case of masonry structures, in addition to what is required for KLM2, direct testing of material properties in the critical areas should be performed, in order to update the reference values of predefined masonry types; material properties should then be defined by using results of tests for updating the reference values for the masonry types. In the case of timber structures, in addition to what is required for KLM2, (semi) non-destructive testing, e.g. by resistance drilling, and/or density measurements on small samples in order to define the material properties in the critical zones should be performed (see Table 10.1).

NOTE 2 For use in a) to c), Annex D, Tables D.1 and D.2 provide reference values of predefined masonry types, unless the National Annex provides different reference values associated to specific masonry types. As concerns timber components, relevant information regarding material properties can be found at 10.2.1(3).

(2) For each type of structural member (beam, column, wall, diaphragm, etc.) and material, the achieved KL on Materials based on the collected information should be taken as defined in 8 to 12.

5.5 Representative values of material properties

(1) For existing materials, design values of material properties X_d for calculating resistances to be used in local verifications, should be taken as the mean. Mean values should be obtained from testing and additional sources of information, and different mean values may be considered in different areas of the structure, as appropriate, based on test results.

NOTE The variability of the material properties and the associated uncertainty of estimation based on a limited number of test values are accounted for through KL-dependent partial factors γ_{Rd} , specified for each resistance model, as a function of KLG, KLD and KLM in the relevant subclauses of 8 to 11.

(2) The standard deviation of the natural logarithm $s_{\ln X}$ of relevant test values (i.e. tests on the existing material and in the area of the structure under consideration) should always be reported, unless the properties are not derived from testing of *in situ* materials, as is the case for KLM1 for reinforcing steel, timber and masonry.

NOTE 1 Standard deviation of the natural logarithm of material strengths within the same structure are, indicatively, in the ranges:

- Infill walls: 0,20 to 0,40
- Concrete: 0,10 to 0,20
- Reinforcing steel: 0,05 to 0,10
- Structural steel: 0,05 to 0,10
- Timber: 0,15 to 0,25
- Masonry: 0,20 to 0,30 (specific values are provided in Annex D, Table D.1)

NOTE 2 Standard deviation of the natural logarithm of material properties determined from tests that lie above the upper bounds of the indicated ranges indicate poor construction quality and a lower confidence in the estimated mean values. In these cases, it is recommended to increase the level of testing of material properties.

(3) For existing materials, when the properties are not derived from testing of *in situ* materials, as is the case of KLM1 for reinforcing steel, timber and masonry, mean values should be obtained from standards in force at the time of construction (steel and timber), considering also a) and b):

- a) For concrete, the mean value may be obtained from the characteristic value as given by Formula (5.2).

$$f_{cm} = f_{ck} + 8 \text{ MPa} \quad (5.2)$$

- b) For steel and timber, characteristic values usually specified in codes should be converted to mean values accounting for the appropriate standard deviation for the identified steel or timber, assuming normal distribution.

NOTE 1 Appropriate standard deviation values are those given in NOTE 1 to (2).

NOTE 2 Additional information is given for masonry in Annex D, D.2.

(4) For added materials, design values of material properties X_d for calculating resistances to be used in local verifications, should be defined as the mean. The mean may be derived from the characteristic value as indicated in (3).

(5) For new materials, design values of material properties should be:

- a) calculated using characteristic properties (f_k) divided by the appropriate partial factor, in case a new structure is built to withstand all seismic action effects (see 4.3.2c)) and it is verified by the force-based approach according to prEN 1998-1-1:2022, 6.1(2).
- b) defined as the mean (f_m), if the new materials are used in new members designed to resist only part of the seismic action in parallel with existing ones (see 4.3.2b)), or they form a new structure that is verified with the displacement-based approach according to prEN 1998-1-1:2022, 6.1(6). The mean may be derived from the characteristic value as given in (3).

6 Modelling, structural analysis and verification

6.1 General

(1) A quantitative assessment procedure should be performed for checking whether an existing undamaged or damaged structure satisfies the required limit state, as specified in 4.1.

(2) Assessment should be carried out by means of the analysis methods specified in prEN 1998-1-1:2022, 6, complemented by prEN 1998-1-2:2023, 5.3, for buildings, and prEN 1998-2:2023, 5.2, for bridges, as modified in 6.3 and 6.4 to address the specific problems encountered in the assessment of existing structures.

NOTE Complementary information on procedures of analysis can be found in the relevant material- related clauses (8, 9, 10, 11), the clause related to bridges (12), as well as in the pertinent Annexes.

(3) When 4.1(4) does not apply, the force-based approach should not be used.

6.2 Modelling

(1) Based on information collected as prescribed in 5.2, a model of the structure should be set up. The model should be such that the action effects in all structural members can be determined under the combination of seismic action with other actions, given in EN 1990:2023, 8.4.3.5.

(2) In application of 4.2.1(1), whenever possible, the structural model used should incorporate information of the observed behaviour on the structure to be assessed, or on the same type of structures, during previous earthquakes.

(3) Unless more refined modelling approaches satisfying (2) are used, the structural model should comply with prEN 1998-1-1:2022, 6.2, complemented with either a) or b):

- a) all provisions of prEN 1998-1-2:2023, 5.1, for buildings,
- b) all provisions of prEN 1998-2:2023, 5.1, for bridges.

(4) In application of prEN 1998-1-1:2022, 6.2.1(4), mean values of all material properties, including strength and ultimate deformation properties in non-linear models, should be used in the structural model.

(5) Additional provisions specific to non-engineered masonry in 11.3 should be applied.

(6) For masonry infills in buildings prEN 1998-1-2:2023, 7.4, should be applied, according to prEN 1998-1-1:2022, 6.2.1(6).

(7) For secondary members of buildings prEN 1998-1-2:2023, 5.1.3, should be applied.

(8) For bridges, where spatial variability of the seismic action is accounted for according to prEN 1998-2:2023, 4.2.2, the corresponding model should be chosen depending on the analysis method according to prEN 1998-2:2023, 5.3.

(9) The non-linear model should satisfy the general rules of prEN 1998-1-1:2022, 6.2.3, the rules for buildings in prEN 1998-1-2:2023, 5.1, and prEN 1998-2:2023, 5.1, for bridges. The models given in a) to c) may be used:

- a) In the case of non-linear response-history analysis, the members may be modelled using appropriate hysteretic curves, the shape of which depends on the member's material properties, structural details, and the type of response.
- b) The envelopes of hysteretic curves should be defined based on the rules given in the relevant material-related clauses (8, 9, 10, 11).
- c) For non-linear static analysis, constitutive laws may be defined based on the envelopes of the hysteretic models.

NOTE Some appropriate models are shown in Figure 6.1. V and θ in Figure 6.1 indicate generalised stresses and deformations, respectively.

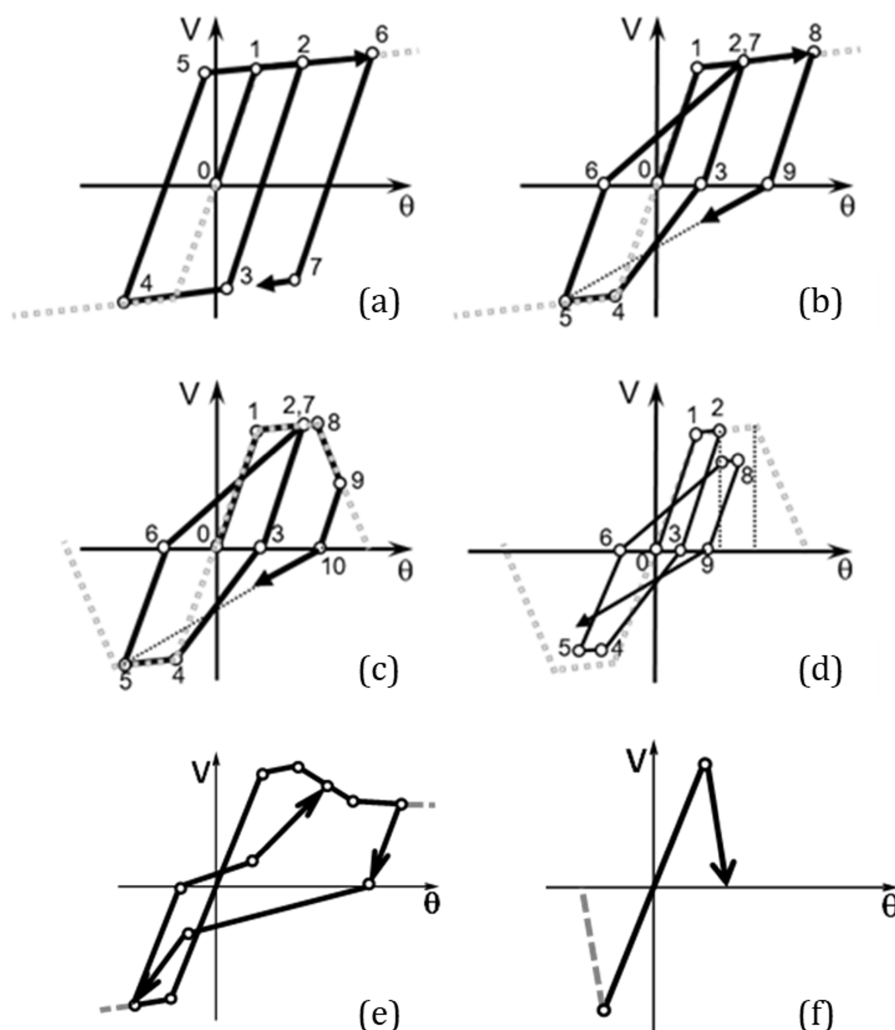


Figure 6.1 — Hysteretic models for non-linear response-history analysis: a) bilinear without strength and stiffness degradation, (b) without strength, but with stiffness degradation, (c) with in-cycle strength degradation, (d) with both in-cycle and cyclic strength degradation, e) with residual strength and pinching, f) brittle response

6.3 Analysis: Force-based approach

6.3.1 Reduced spectrum for the force-based approach

(1) Unless higher values of q are duly justified with reference to the local and global ductility, values of q factor not higher than those listed in Table 6.1 should be adopted for the horizontal components of seismic actions, regardless of the structural system and depending on the prevailing material.

Table 6.1 — Values of q -factors for the horizontal components of the seismic action

Prevailing material of the structure	q -factor
Reinforced concrete	1,5
Steel	2,0
Timber	1,5
Masonry	1,5

(2) A value of q_v equal to 1,5 for buildings and 1,0 for bridges should be adopted for the vertical component of the seismic action.

(3) When applicable according to prEN 1998-1-2:2023, 5.3.3, for buildings and prEN 1998-2:2023, 5.2.2.2, for bridges, the lateral force method may be used.

6.4 Analysis: Displacement-based approach

6.4.1 Linear elastic analysis

(1) If the action effects E_d and corresponding resistances R_d are expressed in terms of generalised stresses, where R_d are calculated according to the relevant clauses of prEN 1998-1-2 for buildings and prEN 1998-2 for bridges, using design values resulting from 4.2.2(6), the regions of primary structural members where $E_d > R_d$ are designated as critical zones. For each critical zone i a ratio ρ_i may be defined as given by Formula (6.1).

$$\rho_i = \frac{E_{d,i}}{R_{d,i}} \quad (\rho_i > 1) \quad (6.1)$$

(2) A linear elastic analysis may be performed using the elastic response spectrum given in prEN 1998-1-1:2022, 5.2.2, when $\rho_{\max}/\rho_{\min}$ does not exceed 2,5 for buildings and 2,0 bridges, where, considering all critical zones: $\rho_{\max} = \max(\rho_i)$; $\rho_{\min} = \min(\rho_i)$; and ρ_i is defined by Formula (6.1) for each critical zone.

NOTE When conditions occur for the similarity between the inelastic deformed shape and the elastic one, the displacement-based approach can be implemented in approximation via a linear elastic analysis. Deformations from the latter are used as seismic action effects for the verification of ductile mechanisms.

6.4.2 Non-linear static analysis

6.4.2.1 General

(1) When applied, a non-linear static analysis should be performed according to general rules of prEN 1998-1-1:2022, 6.5 and Annex D, with additional rules defined in prEN 1998-1-2:2023, 5.3.5, for buildings and in prEN 1998-2:2023, 5.2.3, for bridges. Additional rules for existing structures should be taken into account, as given in (2) to (5).

(2) If in a building a soft storey mechanism is expected or is predicted from non-linear static analysis with the “modal” pattern, a “uniform” pattern should be used in addition.

NOTE A “uniform” pattern is a mass-proportional distribution of forces. It is an appropriate distribution of inertia forces when floor displacement and hence accelerations are approximately equal due to development of a soft storey mechanism.

(3) In the case of masonry buildings with stiff diaphragms (see 11.3.2.2) or when the modal participating mass ratio of the predominant mode of vibration in the considered direction is lower than 70%, the “modal” pattern should be replaced by either a) or b).

- a) “triangular” pattern, based on lateral forces that are proportional to masses, considering an inverted triangular acceleration profile in elevation;
- b) modal combination of load patterns from relevant modes, defined as those in which the displacements have no change of sign in elevation; this option should be used in the presence of significant irregularity in elevation.

(4) In buildings without rigid diaphragms, lateral loads should be applied at the location of, and proportionally to, the masses of the model.

(5) In the case of masonry buildings with stiff diaphragms (see 11.3.2.2), the control displacement should be assumed as the average displacement among those of different walls, at the reference slab, weighed by the corresponding seismic masses.

NOTE In such buildings, the centre of mass of the slab at the top of the building (prEN 1998-1-2:2023, 5.3.5.2(5)), does not correspond to a physical point of the structural model. This definition of control displacement is coherent with the centre of mass, as they coincide in the case of rigid diaphragms.

6.4.3 Non-linear response-history analysis

(1) If non-linear response-history analysis is applied, it should be performed according to prEN 1998-1-1:2022, 6.6.

6.5 Safety verifications

6.5.1 General

(1) Verifications should be performed according to prEN 1998-1-1:2022, 6.7, as integrated or modified in 4.2.2, 4.2.3, 6.5.2 and 6.5.3.

(2) For all limit states, analysis methods and verification formats, action effects should be multiplied by γ_{Ed} .

6.5.2 Verifications to Near Collapse limit state

6.5.2.1 General

(1) With the exception of the case identified in 4.1(4), verifications to NC should be performed with the displacement-based approach. In application of prEN 1998-1-1:2022, 6.7.1(3), verifications should be carried out in local or global terms.

(2) prEN 1998-1-1:2022, 6.7.3(1), should be applied.

(3) Verifications in local terms shall be carried out when the analysis is linear elastic or non-linear response history.

(4) Verification in global terms may be carried out in conjunction with non-linear static analysis. For masonry structures or masonry-infilled frames, the ultimate displacement d_u may be established directly on the capacity curve (rather than mapped onto it from a local condition according to prEN 1998-1-1:2022, 6.5.2(7)), provided that the post-peak strength degradation is modelled at the member level (see Figure 6.1c to f and (11)).

6.5.2.2 Verifications in local terms using linear elastic analysis

(1) Verifications of ductile mechanisms should be performed in terms of generalised deformations according to prEN 1998-1-1:2022, 6.7.3(2). Deformation capacity R_d should be evaluated according to prEN 1998-1-1:2022, 7, complemented by Clauses 8 to 11 as appropriate.

(2) Verifications of brittle mechanisms should be performed in terms of generalised stresses according to prEN 1998-1-1:2022, 6.7.3(3). Seismic action effects corresponding to brittle mechanisms should be derived from equilibrium considering the resistance of non-brittle mechanisms (capacity-design-like approach) multiplied by γ_{sd} from 4.2.2(5) and by the overstrength factor according to 4.2.2(10). To this purpose, the resistances of non-brittle mechanisms should be derived considering the mean material properties and should not be divided by γ_{Rd} .

6.5.2.3 Verifications in local terms using non-linear analysis

(1) Verification in local terms (member level) should be performed when the post-peak strength degradation of structural members is not modelled (e.g. see Figure 6.1 a or b). It may be performed in all other cases.

(2) Verifications of ductile mechanisms should be performed in terms of generalised deformations according to prEN 1998-1-1:2022, 6.7.3(2). Deformation capacity R_d should be evaluated according to prEN 1998-1-1:2022, 7, complemented by Clauses 8 to 11, as relevant.

(3) Verifications of brittle mechanisms should be performed in terms of generalised stresses according to prEN 1998-1-1:2022, 6.7.3(3). Seismic action effects for the verifications of brittle mechanisms are those from the analysis, multiplied by γ_{sd} from 4.2.2(5) and for the overstrength factor according to 4.2.2(10).

(4) Failure modes that are not directly captured by the behaviour model in the analysis should be checked through verifications by partial models including the relevant part of the structure.

6.5.2.4 Verification in global terms

(1) Verification in global terms (structural system level) may be performed for masonry buildings and for buildings in which infills dominate their global capacity. In such buildings, the strength degradation after peak strength should be modelled at member level (see Clause 11).

(2) Verifications should be performed in terms of the equivalent SDOF displacement according to prEN 1998-1-1:2022, 6.7.3(4), where the resistance d^*_{NC} is defined according to 11.

(3) For buildings, the correction factors c_P or c_E should be evaluated according to prEN 1998-1-2:2023, 5.3.5.3, with the specific provisions for masonry buildings in 11.5.1.3.1 and 11.5.1.4.1. Local verifications should be made in members where factors c_P or c_E are greater than 1,2.

(4) The two components of the seismic action should be considered according to prEN 1998-1-1:2022, 6.5.4(6), for the control node displacement, which in the case of verification in global terms is included in the seismic action effect. For these combinations, forces in the secondary direction should be applied with the sign that produces an increase of the displacement in the main direction.

(5) The value of γ_{sd} for the calculation of E_d should be taken from 4.2.2(5).

(6) Failure modes that are not directly captured by the global model in the analysis should be checked through local verifications, possibly involving partial models.

NOTE In particular, this is the case of masonry buildings, for which the global model, which considers the “box-type” behaviour and the in-plane resistance of masonry walls, does not capture partial out-of-plane mechanisms (see 11.3.3).

6.5.3 Verifications to additional limit states

(1) Except when the verification is carried out in global terms (see 6.5.2.4), verifications of the SD limit state should be carried out according to prEN 1998-1-2:2023, 6.2, for buildings and prEN 1998-2:2023, 6.3, for bridges, with the values of the seismic action effects resulting from 6.4 to 6.6, combined with other action effects according to EN 1990:2023, 8.3.7.4, and the values of resistances resulting from 4.2.2(6) considering 4.2.3.

(2) Except when the verification is carried out in global terms (see 6.5.2.4), verifications of the DL limit state should be made according to prEN 1998-1-1:2022, 6.7.3, considering 4.2.3.4.

(3) Except when the verification is carried out in global terms (see 6.5.2.4), verifications of the OP limit state should be made according to prEN 1998-1-1:2022, 6.7.3, considering 4.2.3.5.

(4) Design effects corresponding to DL and OP should be defined taking into account design seismic actions relevant to these limit states.

NOTE For these limit states, the behaviour is linear elastic (i.e. corresponding to $q = 1$).

7 Design of structural intervention

7.1 Criteria for a structural intervention

7.1.1 General

(1) Based on the conclusions of the assessment of the structure and/or the nature and extent of damage, decisions should be taken for the intervention.

NOTE 1 This standard describes the technical aspects of the relevant criteria.

NOTE 2 As in the design of new structures, optimal decisions are pursued, taking into account social aspects, such as the disruption of use or occupancy during the intervention.

7.1.2 General technical criteria

(1) The selection of the type, technique, extent and urgency of the intervention should be based on the structural information collected during the assessment of the structure.

(2) The aspects described in a) to g) should be taken into account:

- a) All identified local gross errors (i. e. inducing early brittleness or instability) should be appropriately remedied;
- b) In case of highly irregular structures (both in terms of stiffness and overstrength distributions), structural regularity should be improved as much as possible, both in plan and (where feasible) in elevation;
- c) The required characteristics of regularity and resistance may be achieved by either modification of the strength and/or stiffness of an appropriate number of existing components, or by the introduction of new structural members;
- d) Increase of local ductility and deformation capacity should be done where required to satisfy the verifications;
- e) The increase in strength after the intervention should not substantially reduce the available global ductility;
- f) Specifically, for masonry structures: non-ductile lintels should be replaced, inadequate connections between floor and walls should be improved, out-of-plane horizontal thrusts against walls should be eliminated;
- g) The intervention may consist in reducing the action effects in the structure, rather than increasing its resistance. This reduction may be achieved by the use of seismic isolation and/or provision of supplemental damping to the structure.

7.1.3 Types of intervention

(1) An intervention may consist of one or more of the procedures described in a) to i):

- a) Local or overall modification of damaged or undamaged members (repair, upgrading or full replacement), considering the stiffness, strength and/or ductility of these members;

NOTE 1 This standard provides (in its material-specific Clauses 8 to 11) information on the retrofitting techniques currently in use. For reinforced concrete members, these include concrete jacketing, steel jacketing and FRP plating and wrapping. For steel members, these include weld retrofits, stiffening with steel plates and haunches, concrete encasing and bolt replacement as defined in 9.6. For timber members, these include several types of retrofitting methods defined in 10.8. For masonry members, these include techniques that improve material properties in terms of stiffness and strength and by addition of tension resisting members (see 11.6(4)).

- b) Addition of new structural members (e.g. bracings; steel, timber or reinforced concrete belts in masonry construction; etc.) or infill walls;
- c) Modification of the structural system (elimination of some structural joints; widening of joints; elimination of vulnerable members; modification into more regular and/or more ductile arrangements);

NOTE 2 This is, for instance, the case when vulnerable low shear-ratio columns or entire soft storeys are transformed into more ductile arrangements; similarly, when overstrength irregularities in elevation, or in-plan eccentricities are reduced by modifying the structural system.

- d) Addition of a new structural system to sustain some or all of the seismic action;
- e) Possible transformation of existing ancillary elements into structural members;
- f) Introduction of passive protection devices through dissipative bracing or other dissipative devices, or seismic isolation at an appropriate level of the structure;
- g) Mass reduction;
- h) Restriction or change of use of the structure;
- i) Partial demolition.

(2) The intervention may involve one or more procedures in combination. In all cases, the effects of structural modifications on the response of foundations should be taken into account.

(3) Techniques other than those covered by this standard may also be used, if they are appropriately documented and backed up by experimental evidence and they satisfy the general requirements of this standard.

(4) If seismic isolation is adopted, prEN 1998-1-1:2022, 6.8, and prEN 1998-1-2:2023, 8, for buildings or prEN 1998-2: 2022, 8, for bridges should be applied.

(5) If distributed energy dissipation systems are used, prEN 1998-1-1:2022, 6.8, and prEN 1998-1-2:2023, 9, for buildings or prEN 1998-2: 2022, 8, for bridges should be applied.

7.1.4 Ancillary elements

(1) In the case of buildings, decisions regarding repair or upgrading of ancillary elements should also be taken whenever, in addition to functional requirements, the seismic behaviour of these elements may endanger the life of occupants or users or affect the value or integrity of the contents.

(2) In cases considered in (1), full or partial collapse of these elements should be avoided by means of measures as given in a) to c):

- a) appropriate connections to structural members (see prEN 1998-1-2:2023, 7);
- b) increasing the resistance of ancillary elements (see prEN 1998-1-2:2023, 7);

- c) taking measures of anchorage to prevent possible falling out of parts of these elements.
- (3) The potential consequences of measures listed in (2) on the behaviour of structural members should be taken into account.

7.1.5 Justification of the selected intervention type

- (1) In all cases, the documents relating to retrofit design should include the justification of the type of intervention selected and the description of its expected effect on the structural response.
- (2) This justification should be made available to the relevant parties.

7.2 Retrofit design procedure

- (1) The retrofit design procedure should include steps a) to c):
- a) Conceptual design;
 - b) Analysis;
 - c) Verifications.
- (2) The conceptual design should cover a) to c):
- a) Selection of techniques and/or materials, as well as of the type and configuration of the intervention;
 - b) Prior to the above selections, possibly a preliminary analysis of the existing (as-built) structure to inform the selection;
 - c) Preliminary estimation of dimensions of additional structural parts;
 - d) Preliminary estimation of the modified stiffness of the retrofitted members.
- (3) The methods of analysis of the structure specified in 6.4 should be used, taking into account the modified characteristics of the structure.
- (4) Verifications should be carried out in accordance with 6.5 and 5.5, for both existing, modified and new structural members. As a minimum, the Limit State NC should be checked for the retrofitted structure.

NOTE Information on the resistance of existing and new structural members can be found in the relevant material-related Clauses 8, 9, 10, 11 and Annexes B, C and D.

8 Specific rules for reinforced concrete structures

8.1 Scope

- (1) This clause contains specific criteria for the assessment of reinforced concrete structures in their present state, and for their retrofitting, when necessary.
- (2) Rules complementary to 5 for knowledge levels of reinforced concrete structures are given in 8.2.
- (3) Rules complementary to 6.3 for structural modelling of reinforced concrete structures are given in 8.3 and 8.4.2.4.3.
- (4) Resistance models for assessment of existing members in terms of generalised deformations are given in 8.4.2.
- (5) Resistance models for assessment of existing members and joints in terms of shear are given in 8.4.3 and 8.4.4, respectively.
- (6) Safety verification rules complementary to 6.5 for verification of limit states are given in 8.5.

(7) Resistance models for retrofitting of existing members in cases of common techniques are given in 8.6.

8.2 Identification of geometry, details and materials

8.2.1 General

(1) 8.2 should be applied to the collection of information for buildings, in addition to Clause 5. For bridges, 12.4.2 should be applied.

(2) The aspects given in a) and b) should be carefully examined:

- a) Physical condition of reinforced concrete members and presence of any degradation, due to carbonation, steel corrosion, etc. Conditions of deterioration should be considered in establishing the value of γ_{sd} (4.2.2(5));
- b) Continuity of load paths between lateral load-resisting members.

8.2.2 Geometry

(1) The collected data should include all items a) to e):

- a) Identification of the lateral load-resisting systems in the two main horizontal directions;
- b) Orientation of one-way floor slabs;
- c) Depth and width of beams, columns and walls;
- d) Width of flanges in T-beams or L-beams (see prEN 1992-1-1:2021, 7.2.3(2), for the definition of the effective width of flanges);
- e) Possible eccentricities between axes of beams and columns at joints.

8.2.3 Details

(1) The collected data should include all items a) to f):

- a) Amount of longitudinal steel bars in beams, columns and walls;
- b) Amount and detailing of confining steel in critical regions and in beam-column joints;
- c) Amount of steel reinforcement in floor slabs contributing to the negative resisting bending moment of T- or L-beams;
- d) Seating lengths and support conditions of horizontal members;
- e) Depth of concrete cover;
- f) Lap-splices of longitudinal reinforcement.

(2) If lap-splice length is not reliably established for each structural member typology of the structure or its critical portion, when identified based on a preliminary analysis, short lap-splice should be assumed in the evaluation of deformation capacity according to 8.4.2, taking l_0 equal to $2/3$ of $l_{0y,min}$.

NOTE According to 5.4.3(1), destructive inspection methods can be preferred. In particular, lap-splice length is not a parameter that can be easily or reliably established through an indirect non-destructive method.