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Eurocode 8 — Design of structures for earthquake resistance —   
Part 1-2: Buildings

Eurocode 8 – Auslegung von Bauwerken gegen Erdbeben – Teil 1-2: Hochbauten

Eurocode 8 – Calcul des structures pour leur résistance au séisme – Part 1-2: Bâtiments

ICS:

Contents Page

[European foreword 13](#_Toc134197739)

[0 Introduction 14](#_Toc134197740)

[1 Scope 17](#_Toc134197741)

[1.1 Scope of prEN 1998-1-2 17](#_Toc134197742)

[1.2 Assumptions 17](#_Toc134197743)

[2 Normative references 17](#_Toc134197744)

[3 Terms, definitions and symbols 17](#_Toc134197745)

[3.1 Terms and definitions 17](#_Toc134197746)

[3.2 Symbols and abbreviations 21](#_Toc134197783)

[3.2.1 Symbols 22](#_Toc134197784)

[3.2.2 Abbreviations 40](#_Toc134197785)

[3.3 S.I. Units 40](#_Toc134197786)

[4 Basis of design 42](#_Toc134197787)

[4.1 Performance requirements 42](#_Toc134197788)

[4.2 Seismic actions 42](#_Toc134197789)

[4.3 Compliance criteria 43](#_Toc134197790)

[4.4 Characteristics of earthquake resistant buildings 43](#_Toc134197791)

[4.4.1 Conceptual design 43](#_Toc134197792)

[4.4.2 Primary and secondary seismic members 44](#_Toc134197793)

[4.4.3 Torsionally flexible buildings 45](#_Toc134197794)

[4.4.4 Structural regularity 46](#_Toc134197795)

[5 Modelling and structural analysis 47](#_Toc134197796)

[5.1 Modelling 47](#_Toc134197797)

[5.1.1 General 47](#_Toc134197798)

[5.1.2 Masses 47](#_Toc134197799)

[5.1.3 Stiffness 47](#_Toc134197800)

[5.2 Minimum design eccentricity in buildings 48](#_Toc134197801)

[5.3 Methods of analysis 49](#_Toc134197802)

[5.3.1 General 49](#_Toc134197803)

[5.3.2 Force-based approach 49](#_Toc134197804)

[5.3.3 Lateral forces method of analysis 50](#_Toc134197805)

[5.3.4 Response spectrum analysis 50](#_Toc134197806)

[5.3.5 Non-linear static analysis 51](#_Toc134197807)

[5.3.6 Response-history analysis 53](#_Toc134197808)

[6 Verifications of structural members to limit states 53](#_Toc134197809)

[6.1 General 53](#_Toc134197810)

[6.2 Verification of Significant Damage (SD) limit state 54](#_Toc134197811)

[6.2.1 General 54](#_Toc134197812)

[6.2.2 Equilibrium condition 54](#_Toc134197813)

[6.2.3 Resistance conditions 54](#_Toc134197814)

[6.2.4 Control of second-order effects 55](#_Toc134197815)

[6.2.5 Limitation of interstorey drift 55](#_Toc134197816)

[6.2.6 Capacity design in DC2 56](#_Toc134197817)

[6.2.7 Capacity design in DC3 57](#_Toc134197818)

[6.2.8 Resistance of horizontal diaphragms and bracings 57](#_Toc134197819)

[6.2.9 Resistance of foundations 58](#_Toc134197820)

[6.2.10 Seismic joint condition 58](#_Toc134197821)

[6.2.11 Verification of transfer zones in DC2 and DC3 58](#_Toc134197822)

[6.2.12 Verification of underground basements 59](#_Toc134197823)

[6.3 Verification to other limit states 60](#_Toc134197824)

[6.3.1 Verification to Near Collapse (NC) limit state 60](#_Toc134197825)

[6.3.2 Verification of Damage Limitation (DL) limit state 60](#_Toc134197826)

[6.3.3 Verification of fully Operational (OP) limit state 61](#_Toc134197827)

[7 Ancillary elements 61](#_Toc134197828)

[7.1 General 61](#_Toc134197829)

[7.2 Verification at Significant Damage (SD) limit state 61](#_Toc134197830)

[7.2.1 Seismic action effects 61](#_Toc134197831)

[7.2.2 Performance factors 63](#_Toc134197832)

[7.3 Verification at Near Collapse (NC) limit state 63](#_Toc134197833)

[7.4 Masonry infilled frames 63](#_Toc134197834)

[7.4.1 General 63](#_Toc134197835)

[7.4.2 Design of frames with interacting infills 64](#_Toc134197836)

[7.4.3 Design of frames with non-interacting infills 71](#_Toc134197837)

[7.5 Structures with claddings 71](#_Toc134197838)

[7.5.1 Basis of design 71](#_Toc134197839)

[7.5.2 Analysis 72](#_Toc134197840)

[7.5.3 Cladding panels 72](#_Toc134197841)

[7.6 Partitions 74](#_Toc134197842)

[7.6.1 Basis of design 74](#_Toc134197843)

[7.6.2 Verification of partitions 74](#_Toc134197844)

[8 Base isolated buildings 74](#_Toc134197845)

[8.1 Field of application 74](#_Toc134197846)

[8.2 Basis of design 74](#_Toc134197847)

[8.2.1 Compliance criteria 74](#_Toc134197848)

[8.2.2 Control of undesirable movements 74](#_Toc134197849)

[8.2.3 Control of differential seismic ground motions 75](#_Toc134197850)

[8.2.4 Control of displacements relative to surrounding ground and constructions 75](#_Toc134197851)

[8.3 Structural analysis 75](#_Toc134197852)

[8.3.1 General 75](#_Toc134197853)

[8.3.2 Fundamental-mode equivalent linear response-spectrum analysis 75](#_Toc134197854)

[8.4 Verification of Significant Damage Limit State 76](#_Toc134197855)

[9 Buildings with energy dissipation systems 77](#_Toc134197856)

[9.1 General 77](#_Toc134197857)

[9.2 Basis of design 77](#_Toc134197858)

[9.2.1 Compliance criteria 77](#_Toc134197859)

[9.2.2 Main structural system 77](#_Toc134197860)

[9.2.3 Energy dissipation system 78](#_Toc134197861)

[9.2.4 Control of torsional effects 78](#_Toc134197862)

[9.3 Structural analysis 78](#_Toc134197863)

[9.3.1 General 78](#_Toc134197864)

[9.3.2 Non-linear response spectrum analysis 78](#_Toc134197865)

[9.3.3 Energy-balance based analysis 87](#_Toc134197866)

[9.3.4 Non-linear response history analysis 92](#_Toc134197867)

[9.3.5 Combination of the effects of the components of seismic action 92](#_Toc134197868)

[9.4 Verification to Limit States 92](#_Toc134197869)

[9.4.1 General 92](#_Toc134197870)

[9.4.2 Verification to Significant Damage (SD) limit state 92](#_Toc134197871)

[9.4.3 Verification to Near Collapse (NC) limit state 92](#_Toc134197872)

[9.4.4 Verification to Damage Limitation (DL) limit state 93](#_Toc134197873)

[9.4.5 Verification of fully Operational (OP) limit state 93](#_Toc134197874)

[10 Specific rules for concrete buildings 93](#_Toc134197875)

[10.1 General 93](#_Toc134197876)

[10.2 Basis of design and design criteria 93](#_Toc134197877)

[10.2.1 General rules on design action effects 93](#_Toc134197878)

[10.2.2 Local resistance condition 94](#_Toc134197879)

[10.2.3 Local ductility condition 94](#_Toc134197880)

[10.2.4 Capacity design rule for moment resisting frames 94](#_Toc134197881)

[10.3 Materials requirements 95](#_Toc134197882)

[10.3.1 General 95](#_Toc134197883)

[10.3.2 Design for DC1 95](#_Toc134197884)

[10.3.3 Design for DC2 and DC3 95](#_Toc134197885)

[10.3.4 Safety verifications 95](#_Toc134197886)

[10.4 Structural types, behaviour factors, limits of seismic action, limits of drift and partial factors for the displacement-based approach 95](#_Toc134197887)

[10.4.1 Structural types 95](#_Toc134197888)

[10.4.2 Behaviour factor for horizontal components of the seismic action in force-based analysis 97](#_Toc134197889)

[10.4.3 Limits of seismic action for design to DC1, DC2 and DC3 98](#_Toc134197890)

[10.4.4 Limits of drift 98](#_Toc134197891)

[10.4.5 Partial factors on resistances for the displacement-based approach 98](#_Toc134197892)

[10.5 Beams 99](#_Toc134197893)

[10.5.1 Geometrical and other provisions 99](#_Toc134197894)

[10.5.2 Specific rules for beams supporting discontinued vertical members 99](#_Toc134197895)

[10.5.3 Design action effects 100](#_Toc134197896)

[10.5.4 SD limit state verifications and detailing 101](#_Toc134197897)

[10.6 Columns 103](#_Toc134197898)

[10.6.1 Geometrical and other provisions 103](#_Toc134197899)

[10.6.2 Design action effects 103](#_Toc134197900)

[10.6.3 SD limit state verifications and detailing 104](#_Toc134197901)

[10.7 Beam-column joints 106](#_Toc134197902)

[10.8 Ductile walls 108](#_Toc134197903)

[10.8.1 Geometrical and other constraints 108](#_Toc134197904)

[10.8.2 Design action effects 109](#_Toc134197905)

[10.8.3 SD limit state verifications and detailing 111](#_Toc134197906)

[10.9 Large walls 116](#_Toc134197907)

[10.9.1 Geometrical provisions 116](#_Toc134197908)

[10.9.2 Design action effects 116](#_Toc134197909)

[10.9.3 SD limit state verifications and detailing 117](#_Toc134197910)

[10.10 Flat slabs 118](#_Toc134197911)

[10.10.1 Basis of design 118](#_Toc134197912)

[10.10.2 SD limit state verifications and detailing 119](#_Toc134197913)

[10.11 Provisions for anchorages and laps 124](#_Toc134197914)

[10.11.1 General 124](#_Toc134197915)

[10.11.2 Anchorage of reinforcement in beams 124](#_Toc134197916)

[10.11.3 Laps and mechanical couplers 126](#_Toc134197917)

[10.12 Provisions for concrete diaphragms 127](#_Toc134197918)

[10.12.1 Cast in place diaphragms 127](#_Toc134197919)

[10.12.2 Precast concrete diaphragms 128](#_Toc134197920)

[10.13 Prestressed concrete 128](#_Toc134197921)

[10.14 Precast concrete structures 128](#_Toc134197922)

[10.14.1 Structural types and behaviour factor *q* 128](#_Toc134197923)

[10.14.2 Rules applicable to all structural types and to DC1, DC2 and DC3 129](#_Toc134197924)

[10.14.3 Precast moment resisting frames 130](#_Toc134197925)

[10.14.4 Precast walls 132](#_Toc134197926)

[10.14.5 Precast floors and roof diaphragms. Rules for ductility classes DC1, DC2 and DC3 134](#_Toc134197927)

[10.15 Design and detailing of foundations 135](#_Toc134197928)

[11 Specific rules for steel buildings 135](#_Toc134197929)

[11.1 General 135](#_Toc134197930)

[11.2 Basis of Design 135](#_Toc134197931)

[11.2.1 Ductility classes 135](#_Toc134197932)

[11.2.2 Safety verifications 135](#_Toc134197933)

[11.3 Materials 135](#_Toc134197934)

[11.4 Structural types, behaviour factors and limits of seismic action 136](#_Toc134197935)

[11.4.1 Structural types 136](#_Toc134197936)

[11.4.2 Behaviour factors 140](#_Toc134197937)

[11.4.3 Limits of seismic action for design to DC1, DC2 and DC3 141](#_Toc134197938)

[11.5 Structural analysis 142](#_Toc134197939)

[11.6 Verification to Limit States 143](#_Toc134197940)

[11.6.1 General 143](#_Toc134197941)

[11.6.2 Verification at Significant Damage limit state in a force-based approach 143](#_Toc134197942)

[11.6.3 Verification at Significant Damage limit state in a displacement-based approach 143](#_Toc134197943)

[11.6.4 Limitation of interstorey drift at Significant Damage limit state 144](#_Toc134197944)

[11.7 Design rules for low-dissipative (DC1) and non-dissipative structural behaviour for all structural types 145](#_Toc134197945)

[11.7.1 General 145](#_Toc134197946)

[11.7.2 Design rules for low-dissipative structures 145](#_Toc134197947)

[11.7.3 Design rules for non-dissipative structures 145](#_Toc134197948)

[11.8 Design rules for dissipative (DC2 and DC3) structural behaviour common to all structural types 146](#_Toc134197949)

[11.8.1 General 146](#_Toc134197950)

[11.8.2 Design criteria for dissipative structures 146](#_Toc134197951)

[11.8.3 Verification for dissipative members in compression or bending 146](#_Toc134197952)

[11.8.4 Verification for dissipative parts of members in tension 147](#_Toc134197953)

[11.8.5 Verification of members 147](#_Toc134197954)

[11.8.6 Verification of connections in dissipative zones 149](#_Toc134197955)

[11.8.7 Verification of column-to-column splices 151](#_Toc134197956)

[11.9 Design rules for moment resisting frames 152](#_Toc134197957)

[11.9.1 Design criteria 152](#_Toc134197958)

[11.9.2 Verification of beams 152](#_Toc134197959)

[11.9.3 Verification of columns 153](#_Toc134197960)

[11.9.4 Verification of beam to column joints 155](#_Toc134197961)

[11.9.5 Verification of column base joints 157](#_Toc134197962)

[11.10 Design rules for frames with concentric bracings 158](#_Toc134197963)

[11.10.1 Design criteria for DC2 and DC3 158](#_Toc134197964)

[11.10.2 Analysis 159](#_Toc134197965)

[11.10.3 Verification of diagonal members 160](#_Toc134197966)

[11.10.4 Verification of beams and columns 162](#_Toc134197967)

[11.10.5 Verification of beam to column connections 163](#_Toc134197968)

[11.10.6 Verification of brace connections 163](#_Toc134197969)

[11.10.7 Verification of column base joints 164](#_Toc134197970)

[11.11 Design rules for frames with eccentric bracings 165](#_Toc134197971)

[11.11.1 Design criteria 165](#_Toc134197972)

[11.11.2 Verification of seismic links 165](#_Toc134197973)

[11.11.3 Verification of members and connections not containing seismic links 169](#_Toc134197974)

[11.11.4 Verification of connections of the seismic links 170](#_Toc134197975)

[11.11.5 Verification of beam to column connections 170](#_Toc134197976)

[11.12 Design rules for frames with buckling restrained bracings 170](#_Toc134197977)

[11.12.1 Design criteria 170](#_Toc134197978)

[11.12.2 Analysis 171](#_Toc134197979)

[11.12.3 Verification of buckling restrained bracings 172](#_Toc134197980)

[11.12.4 Conformity criteria 172](#_Toc134197981)

[11.12.5 Verification of beams and columns 173](#_Toc134197982)

[11.12.6 Verification of beam to column connections 173](#_Toc134197983)

[11.12.7 Verification of brace connections 173](#_Toc134197984)

[11.12.8 Verification of column base joints 174](#_Toc134197985)

[11.13 Design rules for dual frames - moment resisting frames combined with either concentric, eccentric or buckling restrained bracings 175](#_Toc134197986)

[11.14 Design rules for lightweight steel systems 175](#_Toc134197987)

[11.14.1 General 175](#_Toc134197988)

[11.14.2 General verification rules for low-dissipative (DC1) and dissipative (DC2 and DC3) structural behaviour common to all lightweight steel systems 175](#_Toc134197989)

[11.14.3 Additional verification rules for dissipative (DC2 and DC3) structural behaviour common to all lightweight steel systems. 176](#_Toc134197990)

[11.14.4 Specific verification for dissipative (DC2 and DC3) strap braced walls 177](#_Toc134197991)

[11.14.5 Specific verification for dissipative (DC2 and DC3) shear walls with steel sheet sheathing 177](#_Toc134197992)

[11.14.6 Specific verification for dissipative (DC2 and DC3) shear walls with wood sheathing 177](#_Toc134197993)

[11.14.7 Specific verification for dissipative (DC2 and DC3) shear walls with gypsum sheathing 178](#_Toc134197994)

[11.15 Verification of inverted pendulum structures 178](#_Toc134197995)

[11.16 Design rules for steel structures with concrete cores or concrete walls and for moment resisting frames combined with infills 178](#_Toc134197996)

[11.16.1 Structures with concrete cores or concrete walls 178](#_Toc134197997)

[11.16.2 Moment resisting frames combined with infills 178](#_Toc134197998)

[11.17 Steel diaphragms 178](#_Toc134197999)

[11.18 Transfer zones. Design for DC2 and DC3 179](#_Toc134198000)

[11.19 Requirements for supply of material and execution 179](#_Toc134198001)

[12 Specific rules for composite steel–concrete buildings 180](#_Toc134198002)

[12.1 General 180](#_Toc134198003)

[12.2 Basis of design 180](#_Toc134198004)

[12.2.1 Design concepts 180](#_Toc134198005)

[12.2.2 Safety verifications 180](#_Toc134198006)

[12.3 Materials 180](#_Toc134198007)

[12.3.1 Concrete 180](#_Toc134198008)

[12.3.2 Reinforcing steel 181](#_Toc134198009)

[12.3.3 Structural steel 181](#_Toc134198010)

[12.4 Structural types, behaviour factors, limits of seismic action and limits of drifts 181](#_Toc134198011)

[12.4.1 Structural types 181](#_Toc134198012)

[12.4.2 Behaviour factors 182](#_Toc134198013)

[12.4.3 Limits of seismic action for design to DC1, DC2 and DC3 183](#_Toc134198014)

[12.5 Structural analysis 184](#_Toc134198015)

[12.5.1 General 184](#_Toc134198016)

[12.5.2 Stiffness of sections 184](#_Toc134198017)

[12.6 Verification to limit states 185](#_Toc134198018)

[12.6.1 General 185](#_Toc134198019)

[12.6.2 Verifications at Significant Damage limit state in a force-based approach 185](#_Toc134198020)

[12.6.3 Verifications at Significant Damage limit state in a displacement-based approach 185](#_Toc134198021)

[12.6.4 Limitation of interstorey drift at Significant Damage limit state 185](#_Toc134198022)

[12.7 Design rules for low-dissipative (DC1) and non-dissipative structural behaviour for all structural types 186](#_Toc134198023)

[12.7.1 General 186](#_Toc134198024)

[12.7.2 Design rules for low-dissipative structures 186](#_Toc134198025)

[12.7.3 Design rules for non-dissipative structures 186](#_Toc134198026)

[12.8 Design rules for dissipative (DC2 and DC3) structural behaviour common to all structural types 186](#_Toc134198027)

[12.8.1 General 186](#_Toc134198028)

[12.8.2 Design criteria for dissipative structures 186](#_Toc134198029)

[12.8.3 Verification of dissipative members in compression or bending 187](#_Toc134198030)

[12.8.4 Verification of dissipative members in tension 188](#_Toc134198031)

[12.8.5 Verification of members in DC2 and DC3 188](#_Toc134198032)

[12.8.6 Verification of beams 189](#_Toc134198033)

[12.8.7 Verification of composite columns 192](#_Toc134198034)

[12.8.8 Verification of composite joints in dissipative zones 195](#_Toc134198035)

[12.8.9 Verification of column-to-column splices 196](#_Toc134198036)

[12.9 Design and detailing rules for composite moment resisting frames in DC2 and DC3 196](#_Toc134198037)

[12.9.1 Design criteria 196](#_Toc134198038)

[12.9.2 Analysis 197](#_Toc134198039)

[12.9.3 Verification of beams 197](#_Toc134198040)

[12.9.4 Verification of columns 198](#_Toc134198041)

[12.9.5 Verification of column diaphragm plates 198](#_Toc134198042)

[12.9.6 Verification of beam to column joints 199](#_Toc134198043)

[12.9.7 Verification of column base joints 199](#_Toc134198044)

[12.10 Design and detailing rules for composite frames with concentric bracings in DC2 and DC3 199](#_Toc134198045)

[12.10.1 Design criteria 199](#_Toc134198046)

[12.10.2 Analysis 200](#_Toc134198047)

[12.10.3 Verification of diagonal members 200](#_Toc134198048)

[12.10.4 12.10.4. Verification of beams and columns 200](#_Toc134198049)

[12.10.5 Verification of beam to column connections 200](#_Toc134198050)

[12.10.6 Verification of brace connections 200](#_Toc134198051)

[12.10.7 Verification of column base joints 201](#_Toc134198052)

[12.11 Design and detailing rules for composite frames with eccentric bracings in DC2 and DC3 201](#_Toc134198053)

[12.11.1 Design criteria 201](#_Toc134198054)

[12.11.2 Analysis 201](#_Toc134198055)

[12.11.3 Verification of seismic Links 201](#_Toc134198056)

[12.11.4 Verification of diagonal members 201](#_Toc134198057)

[12.11.5 Verification of beams and columns 201](#_Toc134198058)

[12.11.6 Verification of members and connections not containing seismic links 202](#_Toc134198059)

[12.11.7 Verification of connections of the seismic links 202](#_Toc134198060)

[12.11.8 Verification of beam to column connections 202](#_Toc134198061)

[12.11.9 Verification of column base joints 202](#_Toc134198062)

[12.12 Design and detailing rules for composite frames with buckling restrained bracings 202](#_Toc134198063)

[12.12.1 Design criteria 202](#_Toc134198064)

[12.12.2 Analysis 202](#_Toc134198065)

[12.12.3 Design rules of buckling restrained bracings 202](#_Toc134198066)

[12.12.4 Conformity criteria 202](#_Toc134198067)

[12.12.5 Verification of beams and columns 202](#_Toc134198068)

[12.12.6 Verification of beams to column connections 203](#_Toc134198069)

[12.12.7 Verification of brace connections 203](#_Toc134198070)

[12.12.8 Verification of column base joints 203](#_Toc134198071)

[12.13 Design and detailing rules for composite dual frames in DC2 and DC3 203](#_Toc134198072)

[12.13.1 Design criteria 203](#_Toc134198073)

[12.14 Design and detailing rules for structural wall systems made of reinforced concrete shear walls composite with structural steel elements in DC2 and DC3 203](#_Toc134198074)

[12.14.1 Design criteria 203](#_Toc134198075)

[12.14.2 Analysis 204](#_Toc134198076)

[12.14.3 Verification of composite walls in DC2 204](#_Toc134198077)

[12.14.4 Detailing and verification of coupling beams in DC2 205](#_Toc134198078)

[12.14.5 Additional detailing rules for DC3 205](#_Toc134198079)

[12.15 Composite diaphragms, chords and collectors 206](#_Toc134198080)

[12.16 Transfer zones: Design for DC2 and DC3 206](#_Toc134198081)

[12.17 Checking of design and construction 206](#_Toc134198082)

[13 Specific rules for timber buildings 207](#_Toc134198083)

[13.1 General 207](#_Toc134198084)

[13.2 Basis of design 207](#_Toc134198085)

[13.2.1 Design concepts 207](#_Toc134198086)

[13.2.2 Safety verifications 208](#_Toc134198087)

[13.3 Materials 208](#_Toc134198088)

[13.3.1 Mechanical properties of dissipative zones 208](#_Toc134198089)

[13.3.2 Material properties 209](#_Toc134198090)

[13.4 Structural types, behaviour factors, capacity design rules and limits of seismic action 210](#_Toc134198091)

[13.4.1 Structural types 210](#_Toc134198092)

[13.4.2 Behaviour factors 213](#_Toc134198093)

[13.4.3 Capacity design rules common to all dissipative structural types 217](#_Toc134198094)

[13.4.4 Limits of seismic action for design to DC1 218](#_Toc134198095)

[13.5 Structural analysis 219](#_Toc134198096)

[13.6 Verification of limit states 221](#_Toc134198097)

[13.6.1 General 221](#_Toc134198098)

[13.6.2 Limitation of interstorey drift at Significant Damage limit state 221](#_Toc134198099)

[13.6.3 Non-linear static analysis 221](#_Toc134198100)

[13.7 Rules for cross laminated timber (CLT) structures 222](#_Toc134198101)

[13.7.1 General rules 222](#_Toc134198102)

[13.7.2 Verification in DC2 223](#_Toc134198103)

[13.7.3 Verification in DC3 228](#_Toc134198104)

[13.7.4 Detailing rules 230](#_Toc134198105)

[13.8 Rules for framed wall structures 231](#_Toc134198106)

[13.8.1 General rules 231](#_Toc134198107)

[13.8.2 Verification in DC2 232](#_Toc134198108)

[13.8.3 Verification in DC3 233](#_Toc134198109)

[13.8.4 Detailing rules 234](#_Toc134198110)

[13.9 Rules for log structures 234](#_Toc134198111)

[13.9.1 General rules 234](#_Toc134198112)

[13.9.2 Verification in DC2 236](#_Toc134198113)

[13.9.3 Detailing rules 236](#_Toc134198114)

[13.10 Rules for moment-resisting frames 237](#_Toc134198115)

[13.10.1 General rules 237](#_Toc134198116)

[13.10.2 Verification in DC2 237](#_Toc134198117)

[13.10.3 Verification in DC3 238](#_Toc134198118)

[13.10.4 Detailing rules 238](#_Toc134198119)

[13.11 Rules for braced frame structures with dowel-type connections 239](#_Toc134198120)

[13.11.1 General rules 239](#_Toc134198121)

[13.11.2 Verification in DC2 240](#_Toc134198122)

[13.11.3 Detailing rules 240](#_Toc134198123)

[13.12 Rules for vertical cantilever structures 241](#_Toc134198124)

[13.12.1 General rules 241](#_Toc134198125)

[13.12.2 Verification in DC2 241](#_Toc134198126)

[13.12.3 Detailing rules 241](#_Toc134198127)

[13.13 Rules for braced frame structures with carpentry connections and interacting masonry infill 242](#_Toc134198128)

[13.13.1 General rules 242](#_Toc134198129)

[13.13.2 Verification in DC2 243](#_Toc134198130)

[13.13.3 Detailing rules 243](#_Toc134198131)

[13.14 Rules for braced frame structures with carpentry connections 243](#_Toc134198132)

[13.14.1 General rules 243](#_Toc134198133)

[13.14.2 Detailing rules 244](#_Toc134198134)

[13.15 Verification of floor and roof diaphragms 244](#_Toc134198135)

[13.15.1 General rules 244](#_Toc134198136)

[13.15.2 Cross laminated timber (CLT) floor and roof diaphragms 245](#_Toc134198137)

[13.15.3 Framed floor and roof diaphragms 245](#_Toc134198138)

[13.15.4 Timber-concrete composite floor and roof diaphragms 246](#_Toc134198139)

[13.16 Transfer zones. Design for DC2 and DC3 246](#_Toc134198140)

[13.17 Checking of design and construction data 246](#_Toc134198141)

[14 Specific rules for masonry buildings 247](#_Toc134198142)

[14.1 General 247](#_Toc134198143)

[14.2 Basis of design 247](#_Toc134198144)

[14.2.1 Design concepts 247](#_Toc134198145)

[14.2.2 Rules applicable to structures designed to DC1 or DC2 247](#_Toc134198146)

[14.3 Materials 248](#_Toc134198147)

[14.4 Behaviour factors 249](#_Toc134198148)

[14.4.1 Behaviour factors for in-plane analysis 249](#_Toc134198149)

[14.4.2 Behaviour factors for out-of-plane analysis 251](#_Toc134198150)

[14.5 Structural analysis 251](#_Toc134198151)

[14.5.1 Modelling rules for linear analyses 251](#_Toc134198152)

[14.5.2 Modelling rules for non-linear analyses 253](#_Toc134198153)

[14.5.3 Force based analysis 255](#_Toc134198154)

[14.5.4 Linear structural analysis for determining the out-of-plane bending moment demand on walls 257](#_Toc134198155)

[14.6 Verification of limit states 258](#_Toc134198156)

[14.6.1 General requirements 258](#_Toc134198157)

[14.6.2 Verification for in-plane actions 259](#_Toc134198158)

[14.6.3 Verification for out-of-plane actions at SD limit state 261](#_Toc134198159)

[14.7 Design rules for members 261](#_Toc134198160)

[14.7.1 Limitations of piers and walls dimensions in DC1 and DC2 261](#_Toc134198161)

[14.7.2 Design rules for unreinforced masonry in DC2 262](#_Toc134198162)

[14.7.3 Design rules for confined masonry in DC2 263](#_Toc134198163)

[14.7.4 Design rules for reinforced masonry in DC2 263](#_Toc134198164)

[14.8 Rules for simple masonry buildings 264](#_Toc134198165)

[14.8.1 General 264](#_Toc134198166)

[14.8.2 Design rules 264](#_Toc134198167)

[14.9 Ultimate deformations 266](#_Toc134198168)

[14.9.1 General 266](#_Toc134198169)

[14.9.2 Unreinforced masonry members 266](#_Toc134198170)

[14.9.3 Reinforced masonry members 267](#_Toc134198171)

[14.9.4 Confined masonry members 267](#_Toc134198172)

[15 Specific rules for aluminium buildings 268](#_Toc134198173)

[15.1 General 268](#_Toc134198174)

[15.2 Basis of Design 268](#_Toc134198175)

[15.2.1 Design concepts 268](#_Toc134198176)

[15.2.2 Safety verifications 268](#_Toc134198177)

[15.3 Materials 269](#_Toc134198178)

[15.4 Structural types, behaviour factors and limits of seismic action 270](#_Toc134198179)

[15.4.1 Structural types 270](#_Toc134198180)

[15.4.2 Behaviour factors 270](#_Toc134198181)

[15.4.3 Limits of seismic action for design to DC1 and DC2 271](#_Toc134198182)

[15.5 Structural analysis 271](#_Toc134198183)

[15.6 Verification to Limit States 271](#_Toc134198184)

[15.6.1 General 271](#_Toc134198185)

[15.6.2 Resistance conditions at Significant Damage limit state 271](#_Toc134198186)

[15.6.3 Limitation of interstorey drift at Significant Damage limit state 271](#_Toc134198187)

[15.7 Design rules for low-dissipative (DC1) and non-dissipative structural behaviour common to all structural types 272](#_Toc134198188)

[15.7.1 General 272](#_Toc134198189)

[15.7.2 Design rules for low-dissipative structures 272](#_Toc134198190)

[15.7.3 Design rules for non-dissipative structures 272](#_Toc134198191)

[15.8 Design rules for dissipative (DC2) structural behaviour common to all structural types 272](#_Toc134198192)

[15.8.1 General 272](#_Toc134198193)

[15.8.2 Design criteria for dissipative structures 272](#_Toc134198194)

[15.8.3 Design rules for dissipative members in compression or bending 273](#_Toc134198195)

[15.8.4 Design rules for dissipative parts of members in tension 273](#_Toc134198196)

[15.8.5 Design rules for non-dissipative members 273](#_Toc134198197)

[15.8.6 Design rules for connections in dissipative zones 274](#_Toc134198198)

[15.8.7 Design rules for column-to-column splices 274](#_Toc134198199)

[15.9 Design rules for moment resisting frames 274](#_Toc134198200)

[15.9.1 Design criteria 274](#_Toc134198201)

[15.9.2 Beams 275](#_Toc134198202)

[15.9.3 Columns 275](#_Toc134198203)

[15.9.4 Beam to column joints 275](#_Toc134198204)

[15.9.5 Column base joints 275](#_Toc134198205)

[15.10 Design rules for frames with concentric bracings 275](#_Toc134198206)

[15.10.1 Design criteria for DC2 275](#_Toc134198207)

[15.10.2 Analysis for DC2 276](#_Toc134198208)

[15.10.3 Diagonal members 276](#_Toc134198209)

[15.10.4 Beams and columns 276](#_Toc134198210)

[15.10.5 Beam to column connections 276](#_Toc134198211)

[15.10.6 Brace connections 276](#_Toc134198212)

[15.10.7 Column base joints 276](#_Toc134198213)

[15.11 Design rules for dual frames - moment resisting frames combined with concentric bracings 276](#_Toc134198214)

[15.11.1 Design criteria 276](#_Toc134198215)

[15.12 Design rules for inverted pendulum structures 277](#_Toc134198216)

[15.13 Aluminium diaphragms 277](#_Toc134198217)

[15.14 Transfer zones. Design for DC2 277](#_Toc134198218)

[15.15 Checking of design, supply of material and execution 277](#_Toc134198219)

[Annex A (informative) Characteristics of earthquake resistant buildings and in plan regularity 278](#_Toc134198220)

[A.1 Use of this annex 278](#_Toc134198221)

[A.2 Scope and field of application 278](#_Toc134198222)

[A.3 Structural simplicity 278](#_Toc134198223)

[A.4 Uniformity, symmetry and redundancy 278](#_Toc134198224)

[A.5 Bi-directional resistance and stiffness 279](#_Toc134198225)

[A.6 Torsional resistance and stiffness 279](#_Toc134198226)

[A.7 Diaphragmatic behaviour at storey level 279](#_Toc134198227)

[A.8 Adequate foundation 279](#_Toc134198228)

[A.9 Regularity in plan 280](#_Toc134198229)

[Annex B (informative) Natural eccentricity and torsional radius 281](#_Toc134198230)

[B.1 Use of this annex 281](#_Toc134198231)

[B.2 Scope and field of application 281](#_Toc134198232)

[B.3 General 281](#_Toc134198233)

[B.4 Uniform type of lateral load resisting system 281](#_Toc134198234)

[B.5 Calculation by a 3D model 283](#_Toc134198235)

[Annex C (normative) Floor accelerations for ancillary elements 285](#_Toc134198236)

[C.1 Use of this normative annex 285](#_Toc134198237)

[C.2 Scope and field of application 285](#_Toc134198238)

[C.3 Floor spectra 285](#_Toc134198239)

[C.4 Modelling 286](#_Toc134198240)

[Annex D (normative) Buildings with energy dissipation systems 288](#_Toc134198241)

[D.1 Use of this normative annex 288](#_Toc134198242)

[D.2 Displacement ductility ratio 288](#_Toc134198243)

[D.3 Complementary rules for structures with velocity-dependent energy dissipation devices 289](#_Toc134198244)

[D.4 Complementary rules for structures with displacement-dependent energy-dissipation devices 290](#_Toc134198247)

[Annex E (normative) Seismic design of connections for steel buildings 294](#_Toc134198252)

[E.1 Use of this normative annex 294](#_Toc134198253)

[E.2 Scope and field of application 294](#_Toc134198254)

[E.3 Pre-qualified moment resisting beam-to-column joints 294](#_Toc134198255)

[E.4 E.4 Beam-to-column connections allowing rotations in braced frames 324](#_Toc134198275)

[E.5 Gusset plate connections in concentric bracings 325](#_Toc134198276)

[E.6 Partial strength connections in concentric bracings 329](#_Toc134198281)

[E.7 Brace connections in eccentric bracings 332](#_Toc134198284)

[E.8. Gusset plate connections in buckling restrained bracings 334](#_Toc134198287)

[Annex F (normative) Steel light weight structures 335](#_Toc134198288)

[F.1 Use of this normative annex 335](#_Toc134198289)

[F.2 General 335](#_Toc134198290)

[F.3 Strap braced walls 336](#_Toc134198293)

[F.4 Shear walls with steel sheet sheathing 338](#_Toc134198294)

[F.5 Shear walls with wood sheathing 342](#_Toc134198298)

[F.6 Shear walls with gypsum sheathing 344](#_Toc134198301)

[Annex G (normative) Design of connections of concrete or composite columns for dissipative composite steel-concrete moment resisting frames 346](#_Toc134198304)

[G.1 Use of this normative annex 346](#_Toc134198305)

[G.2 Scope and field of application 346](#_Toc134198306)

[G.3 Materials 346](#_Toc134198307)

[G.4 Design provisions 346](#_Toc134198308)

[G.5 Joints between steel beams and reinforced concrete or composite columns 346](#_Toc134198309)

[G.6 Composite joints using diaphragm plates 355](#_Toc134198321)

[G.7 Full-strength composite joints with double-split tee connections in concrete filled tube columns 358](#_Toc134198327)

[Annex H (informative) Seismic design of exposed and embedded steel and composite column base connections 362](#_Toc134198336)

[H.1 Use of this annex 362](#_Toc134198337)

[H.2 Scope and field of application 362](#_Toc134198338)

[H.3 Materials 362](#_Toc134198339)

[H.4 Exposed column base connections 362](#_Toc134198340)

[H.5 Embedded column base connections 367](#_Toc134198350)

[Annex I (normative) Design of the slab of steel-concrete composite beams at beam-column joints in moment resisting frames 371](#_Toc134198359)

[I.1 Use of this normative annex 371](#_Toc134198360)

[I.2 Scope and field of application 371](#_Toc134198361)

[I.3 Design of joints at exterior columns under negative moment 371](#_Toc134198362)

[I.4 Design of joints at exterior columns under positive moment 373](#_Toc134198363)

[I.5 Interior columns 376](#_Toc134198364)

[Annex J (informative) Drift limits for eccentrically loaded unreinforced masonry piers 379](#_Toc134198365)

[J.1 Use of this annex 379](#_Toc134198366)

[J.2 Scope and field of application 379](#_Toc134198367)

[J.3 Verification for in-plane actions 379](#_Toc134198368)

[Annex K (informative) Simplified evaluation of drift demands on infilled frames 380](#_Toc134198369)

[K.1 Use of this annex 380](#_Toc134198370)

[K.2 Scope and field of application 380](#_Toc134198371)

[K.3 Analysis 380](#_Toc134198372)

[Annex L (normative) Load-deformation relationships of dissipative timber components and resistances of non-dissipative timber components for non-linear analyses 383](#_Toc134198373)

[L.1 Use of this normative annex 383](#_Toc134198374)

[L.2 Scope and field of application 383](#_Toc134198375)

[L.3 Force-deformation relationships of dissipative timber components for non-linear analysis 383](#_Toc134198376)

[L.4 Resistances of non-dissipative timber components for non-linear analysis 387](#_Toc134198377)

[Annex M (informative) Material or product properties in EN 1998-1-2 388](#_Toc134198378)

[M.1 Use of this annex 388](#_Toc134198379)

[M.2 Scope and field of application 388](#_Toc134198380)

[Bibliography 390](#_Toc134198381)

European foreword

This document (prEN 1998-1-2:2023) 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 part of EN 1998-1:2004.

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.

A list of the main changes compared to the previous edition will be added at Formal Vote.

0 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 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, 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 1998-1-2**

prEN 1998-1-2 provides specific requirements for earthquake resistant design of new buildings, including rules for structural materials, additional to the ones in other Eurocodes. This document contains, in its Clause 14 related to masonry buildings, specific provisions that simplify the design of "simple masonry buildings”. This document also contains provisions for the design of base-isolated buildings and buildings with energy dissipation systems.

prEN 1998-1-2 is subdivided in fifteen clauses and includes thirteen annexes, where Annexes A, B, H, J, K and M are informative and Annexes C, D, E, F, G, I and L are normative.

**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 1998-1-2**

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 1998-1-2 can have a National Annex containing all national choices to be used for the design of buildings 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 document, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by the relevant parties.

National choice is allowed in prEN 1998-1-2 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.1(2) | 4.1(3) | 4.2(1) NOTE 1 | 4.2(1) NOTE 2 |
| 4.2(1) NOTE 3 | 6.3.2.1(2) | 7.2.2(1) | 10.3.4(1) |
| 10.4.5(3) | 11.6.3(4) | 13.2.2(2) | 14.3(5) |
| 14.6.1.3(2) | 14.6.1.4(3) | 14.8.2(6) | 14.8.3(2) NOTE 1 |
| 14.8.3(2) NOTE 2 | L.3(2)e) | L.3(2)f) |  |

National choice is also allowed in prEN 1998-1-2 on the application of the following informative annexes:

|  |  |  |  |
| --- | --- | --- | --- |
| Annex A | Annex B | Annex H | Annex J |
| Annex K | Annex M |  |  |

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.

# Scope

## Scope of prEN 1998-1-2

1. This document is applicable to the design and verification of new buildings and temporary structures in seismic regions. It gives specific rules for the design and verification relevant to buildings of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3.

NOTE The assessment and retrofitting of existing buildings is covered in prEN 1998-3.

1. Unless specifically stated, prEN 1998-1-1 and prEN 1998-5 apply.
2. EN 1998-1-2 is applicable in complement to the other relevant Eurocodes.

NOTE This document contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, are used for the design of new buildings and temporary structures in seismic regions. prEN 1998-1-2 complements in this respect the other Eurocodes.

## Assumptions

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

# 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 1995-1-1:2023, Eurocode 5 — Design of timber structures — Part 1-1: General rules and rules for buildings

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

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

EN 12512, Timber structures — Test methods — Cyclic testing of joints made with mechanical fasteners

ISO 80000 (all parts), Quantities and units

# Terms, definitions and symbols

## Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, prEN 1998-1-1:2022, 3.1, and the following apply.

### 

balloon frame construction

construction type that uses long continuous framing members running from the foundation or base of the timber structure to the top of the building, with intermediate floor structures nailed to them. The walls are therefore continuous vertically

### 

bounding frame

columns and upper and lower beams that surround infill walls and provide structural support

### 

box behaviour

a structure behaves as a box if it is composed of stiff vertical and horizontal planes connected to each other along lines where there are no relative displacements between the planes

### 

carpentry connection

connection where loads are transferred primarily in compression by shaping the connecting timber elements. Fasteners can be present but only to provide integrity to the connection

Note 1 to entry: Skew notch, mortice and tenon, and halving connections are examples of carpentry connections.

### 

cladding

element that covers the surface of a building. Claddings are fixed by means of connections onto the frame and protects the building from e.g. humidity, light, temperature, wind. Claddings can be made of different products like precast concrete panels, lightweight steel panels, glazing

### 

coupling effect

refers to the increase of horizontal stiffness and strength of the building that results from the flexural stiffness of floors, ring beams or spandrels and the moment frame interaction of these elements with walls. The coupling effect leads to a change in internal forces (axial load, shear force and bending moment) in vertical elements that results from shear forces and bending moments transferred by the horizontal members

### 

critical region

region of a primary seismic element, where the most adverse combination of action effects (M, N, V, T) occurs and where plastic hinges can form

Note 1 to entry: In concrete buildings, critical regions are potential dissipative zones. The length of the critical region is defined for each type of primary seismic element in the relevant subclauses of Clause 10.

### 

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

### 

diaphragm chord

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

### 

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

### 

dowel-type connection

connection with dowel-type mechanical fasteners loaded perpendicularly to their axis

Note 1 to entry: Nails, staples, screws, dowels and bolts are examples of dowel-type mechanical fasteners.

### 

ductility

ratio between the ultimate deformation of a member or connection and its deformation at the elastic limit

### 

infill

masonry wall constructed within the plane of an internal or external structural frame and bounded by that frame

### 

integrated connection systems

couple structure and cladding panels response. If the supporting structure is a moment frame and if the cladding panels are elements of walls, the integrated structural system can be a moment frame-equivalent dual system or a wall-equivalent dual system, depending on the performance of the connections and of the relative stiffness of the claddings and of the supporting frame

### 

interacting cladding

cladding activated by deflection of the supporting frame and significantly affecting the structural response of a building

### 

interacting infill

infill activated by deflection of the bounding frame and significantly affecting the structural response of a building

### 

isolation interface

surface or zone which separates the substructure and the superstructure and where the isolation system is located

Note 1 to entry: Arrangement of the isolation interface at the base of the structure is usual in buildings, tanks and silos. In bridges, the isolation system is combined with the bearings and the isolation interface lies between the deck and the piers or abutments.

### 

isostatic connection system

connection system which allows a free relative displacement between cladding and supporting structure under seismic conditions

### 

non-interacting infill

infill designed not to significantly change the structural response of a building, e.g. infill separated from the frame by a construction joint

### 

partition

wall or division used to divide the internal space into separate rooms. Partitions can be made of different materials (e.g. bricks, glass or other). Partition walls are placed inside the building between the floor above and the floor under it and vertical elements like walls or columns. A partition is not meant to play any structural role other than supporting itself

### 

pier

part of a masonry wall between vertical limits, which can be a wall free edge, a window, a door or a stiffening wall. The length of a pier cross section is the distance between two such interruptions and its height is that of the smaller adjacent opening

### 

platform frame construction

timber construction type that involves floor structures bearing onto wall panels, thereby creating a ‘platform’ for construction of the next level of wall panels. The timber walls are therefore interrupted by the floor structures

### 

principal direction of a building

horizontal direction in which a main lateral force resisting system is designed

### 

primary seismic member

structural member which participates to the lateral load resisting system

### 

primary structure

set of primary seismic members over the building

### 

primary bracing

set of primary members installed in a vertical plane at a given level of the building

### 

rigid joints

joints with negligible flexibility

### 

ring beams and ring ties

horizontal elements that connect all masonry piers at a specific elevation of a building

### 

secondary seismic member

structural member which does not participate to the lateral load resisting system, but keeps its other functions, like carrying gravity loads, during the seismic event

### 

semi-rigid joints

joints with significant flexibility

### 

spandrel

masonry element with a horizontal axis spanning between two adjacent piers

### 

substructure

part of the structure, which is located under the isolation interface, including the foundation

Note 1 to entry: The lateral flexibility of the substructure(s) is generally negligible in comparison to that of the isolation system, but this is not always the case.

### 

superstructure

part of the structure that is located above the isolation interface

### 

**transfer zone**

zone of a building where vertical components of the primary structure are interrupted for architectural reasons (for open spaces, car parks or setbacks, etc.)

### 

### walls

vertical members with an elongated cross section; in reinforced concrete walls, this means a length to thickness ratio of the cross section greater than 4

## Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990:2023, 3.2 and in prEN 1998-1-1:2022, 3.2 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 1998-1-2 are listed and defined in 3.2.1 and additional abbreviations are given in 3.2.2.

### Symbols

#### Upper case Latin symbols

|  |  |
| --- | --- |
| *A*br | area of a brace section |
| *A*c | column cross-section area |
| *A*j,ef | effective cross-sectional area of the beam-column joint |
| *A*mp,i | area of the section of primary pier *i* |
| *A*mp,tot | total pier section area |
| *A*p | pier section area |
| *A*s,int | integrity reinforcement area in the flat slab |
| *A*s1 | area of the beam’s top longitudinal reinforcement used to calculate the design shear in the joint |
| *A*s2 | area of the beam’s bottom longitudinal reinforcement used to calculate the design shear in the joint |
| *A*sL | area of the longitudinal bars |
| *A*st | area of one leg of the transverse reinforcement within the lap zone |
| *A*stif | area of a stiffener |
| *A*sw | area of shear reinforcement |
| *A*+ (*A*-) | areas of the vertical projections of the cross sections of a tension diagonal |
|  | one dimension of a rectangular floor together with |
| *B*CLT**,**i,**j** | length of the *jth* CLT shear wall at the *ith* storey |
| *C*eff,j | effective damping coefficient of a velocity-dependent energy dissipation device |
| *CF*1 | weighting factor of the response at the stage of maximum displacement |
| *CF*2 | weighting factor of the response at the stage of maximum velocity |
| *C*j | damping coefficient of the velocity-dependent energy dissipation device *j* |
| *CVF* | correction factor for velocity |
| *D*lower | smallest sieve size of the coarsest fraction of the aggregates permitted by the specification of the concrete |
| *D*max | maximum sieve size of the coarsest fraction of the aggregates |
| *E*a | modulus of elasticity of structural steel |
| *E*cm | mean modulus of elasticity of concrete |
| *E*d | design value of the action effect in the seismic design situation, including possible influence of second-order effects |
| *E*des,*k* | elastic strain energy stored by the energy dissipation devices at *k-*th storey while the primary seismic elements of the main structural system at the *k*-th storey remain in the elastic domain |
| *E*dH,*k* | energy dissipated through plastic deformation by the energy dissipation devices at the *k-*th storey while the primary seismic elements of the main structural system at the *k*-th storey remain in the elastic domain |
| *E*dH,*k*,max | maximum energy dissipation demand at the *k*-th storey from the energy dissipation devices under the reference seismic action |
| *E*dH,*k,*SD | amount of energy that can be dissipated by the energy dissipation devices at the *k*-th storey under cyclic deformations when any energy dissipation device on the storey attains its SD limit state |
| *E*di | design value of the action effect on the zone or element *i* in the seismic design situation |
| *E*d,E | combination of action effects *E*F,E and *E*F,G |
| *Ee* | amount of energy that the structure can absorb while the primary seismic elements of the main structural system remain in the elastic domain |
| *E*F,E | action effect from the analysis of the design seismic action |
| *E*F,G | action effect due to the non‑seismic actions included in the combination of actions in the seismic design situation |
| *E*H,k | required amount of energy dissipation through plastic deformation at the *k*-th storey of the building |
| *E*oop | resultant horizontal force acting on a wall and which creates out-of-plane deformation of the wall |
| *E*pes,k | elastic strain energy stored by the primary seismic elements of the main structural system at the *k-*th storey while these elements remain in the elastic domain |
| *E*pH,k | energy dissipated by the primary seismic elements of the main system at the *k*-th storey of the building |
| *E*pH,k,max | maximum energy dissipation demand at the *k-*th storey in the primary seismic elements of the main structural system under the design earthquake |
| *E*pH,k,SD | amount of energy that can be dissipated by the primary seismic elements of the main structural system at the *k*-th storey under cyclic deformations before reaching the SD limit state |
| *E*s | design value of the modulus of elasticity of reinforcing steel |
| *E*sec | short-term secant modulus of elasticity of masonry |
| *F\** | force of the equivalent SDOF system |
| *F*1 | load at the first cycle and at a certain deformation *δ* in a cyclic test performed in accordance with EN 12512 |
| *F*3 | load at the third cycle and at a certain deformation *δ* in a cyclic test performed in accordance with EN 12512 |
| *F*ap | horizontal seismic force, acting at the centre of mass of the ancillary element in the most unfavourable direction |
| *F*b | seismic base shear force |
| *F*c | compression force |
| *F*b,Rd | design bearing resistance of the member-to-sheathing connection |
| *F*Ed | design value of the lateral force acting on a steel shear wall in the seismic design situation |
| *F*Ed,E | actioneffect (shear force, bending moment, or axial force) in the non-dissipative joint, connection, or member, due to the design seismic action |
| *F*Ed,E,s | action effect in the shear connections due to the design seismic action |
| *F*Ed,G | action effect (shear force, bending moment, or axial force) in the non-dissipative joint, connection, or member, due to the non-seismic actions in the seismic design situation |
| *F*H | hysteresis loop adjustment factor |
| *F*i | horizontal force acting on storey *i* |
| *F*i1 | design lateral force at *i*-th floor in the first mode |
| *F*im | design lateral force at *i*-th floor in the *m*-th mode |
| *F*M,Ed | design shear force in a fastener in the outer circle due to the bending moment transmitted by the joint |
| *F*max | maximum load attained on the 1st cyclic envelope load-deformation curve in a cyclic test performed in accordance with EN 12512 |
| *F*oop | resultant horizontal force acting on an out-of-plane loaded pier |
| *F*Rd,b | design value of the strength of the non-dissipative, brittle, components |
| *F*Rd,hd,i,j | tensile-vertical design strength of the anchoring connections against overturning of the *jth* CLT shear-wall at the *ith* storey |
| *F*Rk,b | characteristic value of the strength of the non-dissipative, brittle, components |
| *F*Rd,c | design strength in seismic conditions of the single timber-to-timber connection used in the vertical joint |
| *F*Rd,d | design value of the strength of the dissipative, ductile, zones |
| *F*Rk,d | characteristic value of the strength of the dissipative, ductile, zones |
| *F*Rd,hd | design strength in seismic conditions of the anchoring connection against overturning |
| *F*Rd,s | design strength of the shear connections |
| *F*Rm,d,M | mean maximum resistance of the dissipative connection under seismic loading |
| *F*Rm,d,U | mean ultimate resistance of the dissipative connection under seismic loading |
| *F*Rm,d,Y | mean yield resistance of the dissipative connection under seismic loading |
| *F*t,90,Ed | design tensile force in reinforcement |
| *F*u | ultimate load attained on the 1st cyclic envelope load-deformation curve corresponding to the ultimate deformation *δ*u in a cyclic test performed in accordance with EN 12512 |
| *F*v,Rd | design shear resistance of screws |
| *F*v,Rk,d | characteristic strength of the selected ductile failure mode providing energy dissipation, according to prEN 1995-1-1 |
| *F*v,Rk,nd | characteristic strength of the less ductile failure mode |
| *F*y | yield load of a dissipative timber connection |
| *H*b | height of the building (in m) above the foundation or the top of a rigid basement |
| *I*a | moment of inertia of the structural steel cross-section with respect to the axis through the centroid of the transformed section |
| *I*b | second moment of area of the beam section |
| *I*c | moment of inertia of the uncracked concrete section with respect to the axis through the centroid of the transformed section |
| *I*oop | moment of inertia of a pier about the out-of-plane bending axis |
| *I*s | moment of inertia of the steel reinforcement with respect to the axis through the centroid of the transformed section |
| , | mass moments of inertia of the floor around axis *x* and *y* respectively, going through the floor centre of mass *G*i |
| *K*eff | effective stiffness |
| *KEDD,sj* | horizontal elastic storage stiffness component of the viscoelastic energy dissipation device *j* installed at the *s*-th storey |
| *K*SLS,v,mean | slip modulus of a connection at serviceability limit state calculated according to prEN 1995‑1-1:2023, 5.10.1 |
| *K*SLS,v,mean,c | slip modulus of a connection measured in cyclic tests according to EN 12512 |
| *K*SLS,anc | stiffness of anchoring 2D- or 3D-connector against overturning |
| *K*SLS,con | stiffness of the single timber-to-timber connection used in the vertical joint |
| *L*br | brace length |
| *L* | distance between parallel walls |
| *L*e | distance between the two-outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered |
| *L*b | beam length |
| *L*h | distance between dissipative zones at ends of a beam span |
| *L*i | floor dimension perpendicular to the direction of the seismic action |
| *L*st | distance between lateral-torsional restraints |
| *M*b,Rd | design bending resistances of a steel beam cross-section |
| *M*dst,d,E | overturning moment due to design seismic action |
| *M’*Edw | design bending moment of the wall from the analysis for the seismic design situation |
| *M’*Edw,base | design bending moment at the base of the wall from the analysis for the seismic design situation |
| *M*Ed | design bending moment from the analysis for the seismic design situation |
| *M*Ed,E | design rocking moment of the shear wall due to the seismic action |
| *M*Ed,E,i,j | design rocking moment of the *jth* segmented shear wall at the *ith* storey due to the seismic action |
| *M*Edw | design bending moment of the wall in the seismic design situation (envelope) |
| *M*i,d | moment at beam or column end *i* |
| *M*T,j,Ed | design moment in a joint *j* |
| *M*j,Rd | resistance of continuous beam-to column connections |
| *M*oop,Ed | design bending moment due to horizontal forces generating out-of-plane deformation |
| *M*oop,Rd | design bending resistance against out-of-plane action effects |
| *M*pier | total mass of a pier |
| *M*p,link | design value of plastic moment resistance of a link in eccentrically braced frames |
| *M*pl,Rd | design value of plastic moment resistance of a steel component |
| *M*Rd,b | design moment of resistance of a beam |
| *M*Rd,b,i | design moment of resistance of a beam at end *i* |
| *M*Rd,c | design moment of resistance of a column |
| *M*Rd,c,i | design moment of resistance of a column at end *i* |
| *M*Rd,inf | design moment of resistance at the bottom of a column |
| *M*Rd,rock | design rocking strength of the shear wall |
| *M*Rd,rock,i,j | design rocking strength of the *jth* segmented shear wall at the *ith* storey including the stabilizing effect of the vertical load |
| *M*Rdw,base | design moment of resistance at the base of the wall |
| *M*stb,d,G | stabilizing moment due to gravity loads in the seismic design situation |
| *N* | axial force |
| *N*Rd,b | design axial resistance of a beam cross-section (squash load, buckling load) |
| *N*Rd,c | design resistance of a column cross-section |
| *N*C,j,Ed | design compression in a joint *j* |
| *N*CT,BRS | Eulerian critical load of the sleeve of a buckling restrained brace |
| *N*Ed | design axial force from the analysis for the seismic design situation |
| *M*Edcb | moment in the coupling beam from the analysis |
| *N*Ed,E | axial force due to the design seismic action |
| *N*Ed,G | axial force due to the non-seismic actions in the seismic design situation |
| *N*Ed,i,j | total compressive axial load acting on the *jth* CLT shear wall at the *ith* storey |
| *N*EDD,j1 | maximum axial force in energy dissipation device *j* in the first mode |
| *N*EDDd,j1 | axial viscous damping force component in energy dissipation device *j* in the first mode |
| *N*EDDe,1j | axial elastic force component in a viscoelastic-type of energy dissipation device *j* in the first mode |
| *N*f,Ed | axial force in a compressed flange |
| *N*i | number of shear walls parallel to the seismic action at the *ith* storey |
| *N*max | maximum number of storeys in simple masonry buildings |
| *N*pl,Rd | design value of yield resistance in tension of the gross cross-section of a member in accordance with EN 1993-1-1 |
| *N*s | number of storeys of a building |
| *N*T,j,Ed | design tension in a joint *j* |
| *N*u,link | upper design resistance of a link in an eccentric bracing |
| *P*tot | total gravity load at and above a storey in the seismic design situation |
| *Q*m | local forces |
| *R*c,Rd | in-plane lateral resistance corresponding to the resistance of the member to sheathing connection in a light weight steel structure |
| *R*d | design resistance of a connection as given in the reference Eurocode |
| *R*di | design resistance of the zone or element *i* |
| *R*fy | plastic resistance of a dissipative member |
| *R*fo | plastic resistance of an aluminium dissipative member |
| *S*a | value of the floor acceleration spectrum |
| *S*d(*T*1) | ordinate of the reduced spectrum at period *T*1 |
| *S*e | ordinate of the elastic response spectrum |
| *S*ea | value of *S*e at the ancillary element period |
| *S*ep,1 | value of *S*e at the period of the first (fundamental) mode of the primary structure |
| *S*ep,i | value of *S*eat the period of the *i*th mode of the primary structure |
| *S*δ | seismic action index |
| *T*1 | fundamental period of the structure with an energy dissipation system in elastic conditions or fundamental period of vibration of the building for lateral motion in the direction considered |
| *T*1,est | estimate of the fundamental period of vibration of the building |
| *T*A | short period cut-off of the zero-period spectral acceleration – see prEN 1998-1-1:2022, 5.2.2.2(1) |
| *T*an | natural period of the ancillary element |
| *T*C | upper corner period of vibration of the constant spectral acceleration range |
| *T*d1 | fundamental period of the structure with energy dissipation devices when the primary seismic elements of the main structural system reach the design displacement *dDs* at each storey *s* |
| *T*eff | effective period |
| *T*eff1 | effective period in the first mode |
| *T*fm | *m-*th mode vibration period of the main structural system in elastic conditions, exclusive of energy dissipation system |
| *T*m | *m*-th mode vibration period of the structure with energy dissipation system in elastic conditions |
| *T*n | period of the *n*th mode of the structure in the considered horizontal direction |
| *T*p,i | natural period of the *i*th mode of the structure; *T*p,1 corresponds to *T*1 (in s) |
| *T*p1 | fundamental period of the primary seismic elements of the main structural system in elastic conditions, exclusive of energy dissipation system |
| *T*s1 | period that gives the maximum input energy between *T*d1and 1,4*T*d1 |
| *T*v | fundamental vibration period in the vertical direction |
| *V* | shear force |
| *V*Rd,b | design shear resistance of a steel beam cross-section |
| *V*c | shear force applied to a column |
| *V*Rd,c | design shear resistance of a steel column cross-section |
| *V’*Edw | design shear force of the wall in the seismic design situation as obtained from the analysis |
| *V’*Edw,1 | design shear force of the wall due to the mode with the largest participating mass, in the direction of analysis of the structure, as obtained from the modal response spectrum analysis |
| *V*dys | *s-*th storey yield shear of the energy dissipation system |
| *V*Ed | design shear force from the analysis in the seismic design situation |
| *V*Edcb | shear in the coupling beam calculated from the analysis |
| *V*Ed,E,LLRS,i | design global shear of the *ith* storey due to the seismic action |
| *V*Ed,E,i,j | design global shear of the *jth* segmented timber shear wall at the *ith* storey, due to seismic action |
| *V*Ed,M | Design shear coherent with plastic hinges present at both ends of a member |
| *V*Edw | diagram of magnified shear force applied to the wall |
| *V*Edw,base | design shear at the base of the wall in the seismic design situation (envelope) |
| *V*Edw,env | design shear of the wall in the seismic design situation (envelope) |
| *V*Edw,top | design shear at the top of the wall in the seismic design situation (envelope) |
| *V*fD1 | maximum base shear that can be endured by the primary seismic elements of the main structural system within the linear elastic range |
| *V*fDs | maximum shear that can be endured by the primary seismic elements of the main structural system at the *s*-th storey within the linear elastic range |
| *V*fys | *s*-th storey yield shear corresponding to the formation of a full plastic mechanism in the primary seismic elements of the main structural system |
| *V*i | shear force in pier *i* |
| *V*i,d | design shear force at beam or column end *i* |
| *V*ap,Rd | in-plane design resistance in shear of an infill panel |
| *V*Ed,E,i | total design shear load acting on the shear-wall at the *i*th storey due to seismic action *V*p,Rd |
| *V*p,link | design value of shear resistance of a link in eccentrically braced frames |
| *V*pl,Rd | design value of shear resistance of a steel member |
| *V*Rd | design shear resistance |
| *V*Rd,a,i,j | design lateral strength related to shear connections of the *jth* segmented timber shear wall at the *ith* storey |
| *V*Rd,c | design shear resistance of concrete without shear reinforcement |
| *V*Rd,LLRS,i | design lateral strength of the primary structure at the *ith* storey |
| *V*Rd,int | resistance of the integrity reinforcement in the flat slab |
| *V*R,peak | peak shear strength of an unreinforced masonry spandrel |
| *V*R,lim | absolute maximum value of the peak shear strength *V*R,peak |
| *V*Rd,MRF | shear resistance of the moment resisting frames at the building base |
| *V*Rd,sh,i,j | design lateral strength related to connections between sheathing material and timber frame of the *jth* shear wall at the *ith* storey |
| *V*Rd,total | shear resistance of the whole structure at the building base |
| *V*Rd,walls | shear resistance of the walls at the building base |
| *V*s,SD | total storey shear at SD limit state calculated from the analysis for the displacement-based approach |
| *V*storey | storey shear action effect |
| *V*tot | total shear at a given storey in the seismic design situation |
| *V*wp,Ed | design shear force in the web panel due to the action effects |
| *V*wp,Rd | shear resistance of the web panel |
| *V*wb,Rd | shear buckling resistance of a web panel |
| *W*pl,y | plastic section modulus |

#### Lower case Latin symbols

|  |  |
| --- | --- |
| *an* | index, corresponding to the ancillary element |
| *a* | slab overhang |
| *a*CLT | smaller size (width) of a hole in a CLT floor panel |
| *a*pj | clear distance between the inner and outer gusset plates in INERD pin connection |
| *b* | cross-section width of a beam or column |
| *b*0 | length of the control perimeter for punching shear resistance verification |
| *b*0C | smallest dimension of the concrete core (to the centreline of the hoops) |
| *b*b | cross-section width of a beam |
| *b*c | cross-section width of a column |
| *b*CLT | length of each single panel making a shear wall; longer size (length) of a hole in a CLT floor panel |
| *b*ef | effective width |
| *b*eff,Rd | partial effective width of the slab for the calculation of plastic resistance of composite sections |
| *b*f | wall flange thickness or steel beam flange width |
| *b*int | control perimeter activated by the integrity reinforcement in a flat slab |
| *b*j,ef | effective joint width |
| *b*max | largest cross-sectional dimension of a column |
| *b*pin | width of the pin section in INERD pin connection |
| *b*sp | correction coefficient in the calculation of masonry spandrel strength |
| *b*w | wall thickness |
| *b*w0 | wall web thickness |
| *c* | outstand of a steel flange, also named the side width of a steel member |
| *c*E | correction factor in pushover analysis which primarily accounts for higher-mode effects in elevation |
| *c*E,i | correction factor *c*E for the i-*th* storey of the structural member |
| *c*f | factor accounting for the masonry typology |
| *c*hd | distance from the anchoring connections against overturning to the nearest vertical edge of a CLT panel |
| *c*P | correction factor in pushover analysis which primarily accounts for the torsional effects |
| cpj | distance between the outer gusset plates in INERD pin connection |
| *c*P,j | correction factor *cP* for the j-*th* location of a structural member in plan |
| *c*v1 | coefficient for the determination of the shear resistance at the interfaces |
| *d* | lateral elastic displacement of the top of the building (in m) due to the gravity loads applied in the corresponding horizontal direction |
| *d*b | depth of a beam section |
| *d*bolt | nominal diametre of a bolt |
| *d\** | displacement of the equivalent SDOF system |
| *d*bL | longitudinal bar diameter |
| *d*bL,min | minimum longitudinal bar diameter |
| *d*bw | hoops diameter or straight links diameter |
| *d*c | depth of a section of a column or of a composite beam |
| *d*dg | size parameter describing the crack and failure zone roughness, taking account of concrete type and its aggregate properties |
| *d*Ds | maximum interstorey drift that can be sustained by the primary seismic elements of the main structural system at the *s*-th storey within the linear elastic range |
| *d*dys | *s-*th storey drift at yielding of the energy dissipation devices of this storey |
| *d*e,jt | value of the displacement at the *j*-th location in plan, obtained from the linear elastic analysis associated with the design seismic action |
| *d*et | value of the control displacement obtained from the linear elastic analysis associated with the design seismic action |
| *d*fa | fastener diameter |
| *d*f,y | yielding displacement of a structure with velocity-dependent energy dissipation devices, relative to the ground, at roof level |
| *d*fys | *s*-th storey drift corresponding to the formation of a full plastic mechanism in the primary seismic elements of the main structural system |
| *d*i1 | maximum horizontal displacement at the *i*-th floor relative to the ground in the first mode |
| *d*n | total lateral displacement of the shear-wall at the top of the *n*th storey |
| *d*r | design interstorey drift, evaluated as the difference of the average lateral displacements *d*s at the top and bottom of the storey under consideration and calculated in accordance with 6.4.1 |
| *d*r,DL | design interstorey drift at DL limit state |
| *d*r,SD | design interstorey drift at SD limit state |
| *d*ret,i | interstorey drift at the centre of mass of the *i*-th storey, obtained from linear elastic analysis associated with the design seismic action |
| *d*roof,y | yielding displacement of a structure with displacement-dependent energy dissipation devices, relative to the ground, at roof level |
| *d*roof1 | maximum displacement relative to the ground at roof level for inelastic behaviour of the structure with energy dissipation system, including the elastic part, in the first mode |
| *d*roof1e | maximum displacement relative to the ground at roof level in the first mode for a structure with energy dissipation in the elastic range |
| *drs*1 | maximum interstorey drift at the *s-*th storey |
| *d*rt,i | interstorey drifts at the centre of mass of the *i*-th storey which corresponds to the target displacement associated with the considered limit state |
| *d*t,j | displacement at the *j*-th location in plan, if the control displacement is equal to the target displacement *d*t |
| *d*sp | spandrel depth |
| *d*SD, | displacement of a control node at which a primary pier first reaches its resistance |
| *d*v | shear-resisting effective depth of the slab or drop panel |
|  | displacement of a control node at which a primary pier first reaches its resistance |
| *e* | length of a link in an eccentric bracing |
| *e*bp | base plate eccentricity |
| *e*crit | critical eccentricity of an exposed column base |
| *e*ox | distance between the centre of stiffness and the centre of mass, measured along the *x* direction |
| *f*ap | design lateral load per unit area in a panel or an infill |
| *f*b | normalized compressive strength of masonry units |
| *f*bv | normalized compressive strength of masonry units normal to bed face |
| *f*bh | normalized compressive strength of masonry units parallel to bed face |
| *f*c,eff,CLT,i,j | effective compressive design strength of the *jth* CLT shear wall at the *ith* storey |
| *f* 'cd | increased joint concrete compressive strength due to confinement by joint reinforcement |
| *f*cd | design compressive strength of a concrete cylinder |
| *f*ck | characteristic compressive strength of a concrete cylinder |
| *f*ctm | mean tensile strength of concrete |
| *f*k | characteristic compressive strength of masonry |
| *f*kt | characteristic shear strength associated to diagonal cracking of a masonry spandrel |
| *f*u | ultimate strength of steel |
| *f*u,k | characteristic ultimate tensile strength of the metal fastener |
| *f*y | nominal yield strength of constructional steel |
| *f*yd | design yield strength of steel reinforcement |
| *f*yk | characteristic yield strength of steel reinforcement |
| *f*y,mean | mean yield strength of constructional steel |
| *f*y,max | specified upper value of yield strength of constructional steel |
| *f*y,dl | design yield stress of the longitudinal rebars in a composite section |
| *f*y,dw | design yield stress of the confining hoops in a composite section |
| *f*y,df | design yield stress of the flange of the steel cross-section in a composite section |
| *f*ywd | design yield strength of the transverse reinforcement |
| *f*ywd,j | design yield strength of the transverse reinforcement in the joint |
| *f*vk0 | characteristic initial shear strength of masonry under zero compressive stress |
| *f*0 | conventional elastic strength of aluminium |
| *h* | cross-section depth of a beam or column |
| *h*0 | largest dimension of the concrete core to the centreline of the hoops |
| *h*b | cross-section depth of a beam |
| *h*c | cross-section depth of a column |
| *h*cr | height of the wall critical region |
| *h*ef | effective height of a masonry as defined in EN 1996-1-1 |
| *h*j,ef | effective joint depth |
| *h*min | free height of the shortest pier |
| *h*op | clear height of an opening adjacent to a masonry pier |
| *h*ap | length of an infill wall |
| *h*pier | clear height of a pier as in EN 1996-1-1:2022, 7.5.1.3(9) |
| *h*pin | height of the pin section used in INERD pin connection |
| *h*RB | depth of reinforced concrete ring beams |
| *h*s | storey height (also named interstorey height) |
| *h*s,cl | clear storey height |
| *h*sp | length of a spandrel |
| *h*ss | distance between the upper flanges of steel profiles or the tops of floor slabs at each of the levels above and below a splice |
| *h*w | wall height |
| *i*z | radius of gyration referring to the weak axis of a steel cross-section |
| *k* | ratio of characteristic tensile strength to yield strength of reinforcement |
| *k*16 | ratio between the 16th and the 5th percentiles of the resistance distribution |
| *k*br | vertical stiffness of a steel bracing |
| *k*D | factor for the anchorage verification of beam reinforcement in joints, reflecting the ductility class |
| *k*deg | strength reduction factor due to degradation under cyclic loading |
| *k*IR | amplification factor that applies on seismic action for masonry infills in case of structural irregularity in elevation |
| *k*mean | ratio between the mean and the characteristic strength in static conditions |
| *k*mod | modification factor for duration of load and moisture content for the strength of timber |
| *k*stif | minimum value over all stories of the ratio between the horizontal stiffness of the CLT walls and the total horizontal stiffness of the hybrid primary structure in each main direction at the *i*th storey |
| *k*t | factor of reduction of the design shear resistance of connectors related to the shape of profiled steel sheeting supporting a concrete slab |
| *k*w | factor accounting for the prevailing failure mode in large walls structures |
| *k*y | ratio between the yield and maximum strength of dissipative connections |
| *l*1, *l*2 | slab spans |
| *l*0 | distance between the centrelines of two walls |
| *l*b | anchorage length |
| *l*c | length of the boundary elements in the wall |
| *l*cl | beam or column clear length |
| *l*cs | length of a column over which the diagonal strut force of an infill is applied |
| *l*CLT | width of a CLT floor panel |
| *l*cr | beam or column critical region length |
| *l*f | wall flange width |
| *l*infill,left,x,i | length of an infill parallel to axis *x* situated at left of the axis |
| *l*infill,right,x,i | length of an infill parallel to axis *x* situated at right of the axis |
| *l*mp | length of a masonry pier or wall |
| *l*op | length of an opening |
| *l*ap | height of an infill wall or panel |
| *ls* | length of an infill panel diagonal |
|  | radius of gyration of floor *i* |
| *l*w | wall length |
| *m* | total mass of the building above the foundation or above the top of a rigid basement |
| *m\** | mass of an equivalent SDOF system |
| *m*an | mass of an ancillary element |
| *m*eff,j | effective modal mass corresponding to the *j*-th horizontal direction of loading |
| *m*eff1 | first mode effective mass of the structure with energy dissipation system in the elastic range |
| *m*f,eff1 | first mode effective mass of the main structural system in the elastic conditions, exclusive of the energy dissipation system |
| *m*lp | number of CLT panels in a segmented wall |
| *m*i | mass of storey or floor or level *i* |
| *m*s | factor for the determination of the shear magnification factor in the wall |
|  | mass above the *s*-th storey normalized by the total mass of the building |
| *n*vj | number of fasteners used in the vertical joint |
| *n* | value representing the extent to which the required amount of energy absorption of the building is distributed to each storey according to the stiffness and strength of each storey |
| *n*0 | modular ratio equal to *E*a/*E*cm |
| *ne* | number of fasteners in the outer circle of a moment transmitting joint |
| *p* | index corresponding to the primary structure |
| *p*A | minimum ratio of the area of masonry walls in one direction |
| *p*A,min | absolute minimum ratio of the area of masonry walls in one direction |
| *p*A,ref | reference ratio of the area of masonry walls in one direction |
| *ps* | deviation of the *s*-th storey seismic shear from an optimum value that would provide an even distribution of damage among storeys |
| *P*tot | total gravity load at and above the storey considered in the seismic design situation |
| *pts* | coefficient that accounts for the increase in the required energy absorption capacity in a storey *s* resulting from torsional effects |
| *q* | behaviour factor |
| *q*a | behaviour factor of an ancillary element |
| *q*D | behaviour factor component accounting for the deformation capacity and energy dissipation capacity |
| *q*D,CLT | default value for the reference Ductility Class of the component *q*D of the behaviour factor *q* for a CLT primary structure |
| *q*D,Hyb | value of the component *q*D of the behaviour factor *q* for a hybrid framed wall-CLT primary structure |
| *q*D,FW | default value for the reference Ductility Class of the component *q*D of the behaviour factor *q* for the framed wall primary structure |
| *q*oop | behaviour factor for out-of-plane bending of masonry walls |
| *q*R | behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures |
| *q*S | behaviour factor component accounting for overstrength due to all other sources |
|  | minimum torsional radius of the *i*-th floor |
| *r*x | square root of the ratio of the torsional stiffness to the lateral stiffness in the *y* direction (“torsional radius”) |
| *r*x,I *, r*y,I | x component of the torsional radius (respectively y component) |
| *s* | spacing of transverse shear or punching shear reinforcement |
| *s*h | distance between the centre of a plastic hinge and the column axis |
| *s*i | displacement of storey *i* under horizontal unitary acceleration |
| *sint* | width of the integrity bars group in a flat slab |
| *s*lap | spacing of the transverse reinforcement in a lap zone |
| *ss* | ratio of the required amount of energy absorption on storey *s* to the required amount of energy absorption on the first storey |
| *t* | thickness |
| *t*b,f | thickness of a beam flange |
| *t*c,f | thickness of a column flange |
| *t*ef | effective thickness of a masonry wall as defined in EN 1996-1-1 |
| *t*eff,CLT,i,j | effective compressive thickness of the *j*th CLT shear-wall at the *i*th storey |
| *t*f | steel flange thickness |
| *t*gp | thickness of a gusset plate |
| *tp* | thickness of an infill panel |
| *t*w | anchorage depth of a smooth nail |
| *t*RB | width of reinforced concrete ring beams |
| *t*sp | masonry spandrel thickness |
| *t*w | masonry wall thickness / thickness of the web of a steel profile |
| *t*wsp | thickness of a supplementary web plate |
| *u*ult | interstorey displacement in direction *x* (respectively *y*) at the pier where the shear force is incremented, corresponding to the attainment of the ultimate rotation in any pier of the storey along direction *x* (respectively *y*) |
| *w*gp | effective width of a gusset plate |
| *ws* | width of the compression strut in an infill panel |
| *ww* | wall length in light weight steel structures |
| *ww,eff* | effective wall length in light weight steel structures |
| *x* | distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered |
| *x*c | length of the compression zone |
| *z* | distance along the wall height |
| *z*c | plastic neutral axis depth of a composite section |
| *z*i, *z*j | heights of the masses *m*i and *m*j above the level of application of the seismic action (foundation or top of a rigid basement) |

#### Upper case Greek symbols

|  |  |
| --- | --- |
| **1 | first mode participation factor of a structure with energy dissipation system in the elastic range |
| *Γ*i | modal participation factor for the *i*th mode |
| **f1 | first mode participation factor of the main structural system in elastic conditions, exclusive of the energy dissipation system |
| **m | *m*-th mode participation factor of a structure with energy dissipation system in the elastic range |
| *d*j1 | relative displacement between each end of the energy dissipation device measured along its axis in the first mode |
| *v*j1 | relative velocity between each end of the energy dissipation device *j* measured along its axis in the first mode |
| *ΔF*1-3 | impairment of strength, namely theload reduction *ΔF*1-3=*F*1-*F*3 when attaining a certain deformation *δ* from the first to the third cycle of the same amplitude in a cyclic test performed in accordance with EN 12512 |
| *Δφ* | fraction of the critical section with the maximum punching shear stress |
| *V*max,I | maximum value of the reduction or increase *V* of the absolute value of the shear force in a pier resulting from a redistribution of forces |
| *ΔV*Rw | difference between the shear resistance of two storeys, related to the quantity of infills in each storey |
| *ΔM*Ed | variation of the moment in a column adjacent over one storey height |
| *M*Rb | sum of the design values of the moments of resistance of the beams framing the joint |
| *M*Rc | sum of the design values of the moments of resistance of the columns framing the joint |
| Ʃ*V*Ed | sum of the calculated seismic shears in all vertical primary seismic members of a storey |
| *ϕ*ij | value of the *i*th mode shape at floor *j* |
| **1 | vector of first mode of vibration normalized so that its component at roof level is equal to 1 |
| **i1 | component of the first mode vector (normalized so that at roof level the value is equal to 1) at the *i*-th floor of the structure with energy dissipation system in the elastic range |
| **m | vector of *m*-th mode of vibration normalized so that its component at roof level is equal to 1 |
| *Ω* | seismic action magnification factor |
| *Ω*d,i | ratio of the design resistance *R*di to the design action effect *E*di in member *i* or in storey *i* |
| *Ω*d | particular value of *Ω*d,i among all *Ω*d,I and applied in the design of specific parts of a structure |
| *Ω*d,M | design overstrength of a pin connection yielding in bending |
| *Ω*d,V | design overstrength of a pin connection yielding in shear |

#### Lower case Greek symbols

|  |  |
| --- | --- |
| *α* | angle of a brace with the horizontal |
| *α*0 | prevailing aspect ratio, i.e. prevailing height-to-length ratio of walls |
| *α*1 | value by which the horizontal seismic design action is multiplied, in order to first reach the plastic resistance in any member of the structure, while all other design actions remain constant |
| *α*jo | joint aspect ratio |
| **SD | ratio between resistance at the SD limit state and the ultimate resistance in terms of generalized displacements |
| *α*u | value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural (plastic) instability, while all other design actions remain constant |
| *α*ult | angle between the integrity bars and the flat slab plan at failure |
| **ve | velocity exponent of a velocity-dependent energy dissipation device |
| **a | performance factor of ancillary element |
| *γ*c | partial safety factor for concrete |
| *γ*CT | compression strength adjustment factor in buckling restrained braces |
| **d | overstrength factor which is used for multiplication of the seismic action effects in the diaphragm obtained from the analysis |
| *γ*DL,CC | performance factor for DL limit state and a Consequence Class CC |
| *γ*M | partial safety factor for material properties in the seismic design situation |
| **M,SD | partial factors on resistance of masonry |
| **Rd | overstrength factor used for reinforced concrete and timber design |
| **Rd,NC | partial factors on resistance at NC limit state |
| **Rd,SD | partial factors on resistance at SD limit state |
| *γ*s | partial safety factors for reinforcing steel |
| *γ*SB | partial safety factor in the calculation of the required area of masonry walls |
| **s,SD | partial factors on resistance at SD limit state for reinforcing steel |
| **x | amplification factor for energy dissipation devices |
| **lag | phase lag angle |
| *δ* | generalized deformation (displacement, rotation) |
| *δ*Fmax | deformation at maximum load of a dissipative timber connection |
| *δ*NC | deformation at NC of a dissipative connection under seismic loading |
| *δ*SD | deformation at SD of a dissipative connection under seismic loading |
| *δ*u | ultimate deformation of a dissipative connection under seismic loading |
| *δ*y | yield deformation of a dissipative connection under seismic loading |
| *ε* | shear magnification factor for walls |
|  | total strain of the steel profile in a composite section |
| *ε*cu | unconfined concrete ultimate strain |
|  | ultimate compressive strain of concrete considering confinement |
| **loss | loss factor of a viscoelastic energy dissipation device |
| *θ* | interstorey drift sensitivity coefficient |
| **e | drift ratio or chord rotation of a masonry member |
| **i | angle between the member axis at end section *i* and the chord line that joins the centroids of the two end sections |
| **j | angle of inclination of energy dissipation device *j* to the horizontal |
| **p | plastic rotation |
| **SD | chord rotation capacity of a pier |
| *θ*tot | rotation capacity of strong and weak partial strength joints for moment resisting frames |
| **u2 | deformation capacity in terms of member drift at 20 % drop in strength |
| **y | chord rotation at yield |
| *θ\** | largest chord rotation in the shortest pier |
| ** | correction factor used in the lateral force method of analysis |
|  | coefficient accounting for sensitivity of ancillary elements to interstorey drift at DL |
|  | coefficient reflecting the limitation of drift at SD |
|  | non-dimensional slenderness of columns |
| *µ* | ductility of a dissipative connection under seismic loading |
| **f | displacement ductility ratio of a structure with velocity-dependent energy dissipation devices |
| *ν* | strength reduction factor for cracked concrete |
| *ν*d | normalized axial load |
| *ν*d,c | normalized axial load in composite walls |
| **E,k | axial load ratio for the design seismic action, calculated as **E,k = *N*Ed/(*A*p*f*k) |
| *ν*G,k | axial load ratio for the non-seismic actions in the seismic design situation, calculated as *ν*G,k = *N*Ed,G/(*A*p*f*k) |
| νG,k,av | average axial load ratio of a set of piers |
| *ξ*a | damping value of the ancillary element in % of critical damping |
| **eff | effective damping |
| **eff1 | effective damping in the first mode |
| **effm | effective damping in the *m*-th mode |
| **H | component of effective damping of the structure due to post-yield hysteretic behaviour of the primary seismic elements of the main structural system |
| **I | inherent damping ratio |
| *ξ*p,i | damping value of the primary structure for buildings in % of critical damping |
| **V1 | component of effective damping of the first mode of the structure due to viscous dissipation of energy by the damping system at or just below the yield displacement of the primary seismic elements of the main structure |
| *ρ’*l | longitudinal reinforcement ratio in the compression zone |
| *ρ*l | longitudinal reinforcement ratio |
| *ρ*l,max | maximum longitudinal reinforcement ratio |
| *ρ*l,min | minimum longitudinal reinforcement ratio |
| *ρ*n | factor related to the effective height as in EN 1996-1-1:2022, 7.5.1.3(10) |
| *ρ*v,min | minimum vertical reinforcement ratio in the vertical joint between precast panels relatively to the area of grout |
| *ρ*w | shear reinforcement ratio |
| *ρ*w,min | minimum shear reinforcement ratio |
| *ρ*wj | transverse reinforcement ratio of the joint |
| *σ*cm | mean value of the concrete stress in the compression zone of the section of the wall |
| *σ*Ed | stress level |
| *τ*Ed | design punching shear stress for the seismic design situation |
| *τ*Ed,grav | punching shear stress for the gravity loads design situation |
| *τ*Edj | joint shear stress |
| *τ*Rd,c | design resistance of concrete to punching shear stress for members without shear reinforcement |
| *τ*Rdj | joint shear resistance |
| *τ*Rd,max | maximum punching shear stress resistance of planar members with shear reinforcement |
| *φ*imp | strength impairment factor of a dissipative timber connection |
| *ψ*max | maximum slab rotation relatively to the slab support connection |
| *ωpb* | multiplicative factor on design value *N*pl,Rd of yield resistance in tension of compression brace, for the estimation of the residual post-buckling resistance of the brace. |
| *ω*rm | ratio between the expected average yield strength of steel products of a certain steel grade and the nominal yield strength of that steel grade; for aluminium, ratio of the expected (i.e. average) value of the conventional elastic limit to its design value from EN 1999-1-1 |
| *ω*sh | factor taking into account the strain-hardening effect of steel or aluminium |
| *ω*wd | mechanical volumetric ratio of confining hoops |

### Abbreviations

|  |  |
| --- | --- |
| AAC | Aerated autoclaved concrete |
| CLT | Cross-laminated timber |
| GF | Gypsum fibre board |
| GLVL | Wood based composite which is produced from several LVL panels by face gluing (glued laminated veneer lumber) |
| KLD | Knowledge level on details |
| LVL | Laminated veneer lumber |
| LVL-C | Symmetrically arranged structural laminated veneer lumber comprising at least two crossband veneers |
| MRF | Moment resisting frame |
| NC | Near Collapse limit state |
| OP | Fully operational |
| OSB | Oriented strand board |
| PFA | Peak floor acceleration |
| SD | Significant Damage limit state |
| SDOF | Single-degree-of-freedom |
| SWP | Solid wood panels |
| 1D | One-dimensional |
| 2D | Two-dimensional |
| 3D | Three-dimensional |

## S.I. Units

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

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

|  |  |  |
| --- | --- | --- |
| — | mass: | kg, t |
| — | mass density: | kg/m3, t/m3 |
| — | unit mass: | kg/m3, t/m3 |
| — | forces and loads: | kN, kN/m, kN/m2 |
| — | weight density: | kN/m3 |
| — | stresses and strengths: | Pa (= N/m2), kPa (= kN/m2), MPa (= MN/m2) |
| — | moments (bending, etc.): | kNm |
| — | acceleration: | m/s2 |
| — | length: | m, mm |
| — | modulus of elasticity: | MPa, GPa |
| — | angle: | degree, mrad |

# Basis of design

## 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. Consequence class CC3 should be divided into CC3-a and CC3-b according to prEN 1998-1-1:2022, 4.2(3).

NOTE 1 The definitions of buildings included in CC3-a and CC3-b are given in Table 4.1 (NDP), unless the relevant authorities or the National Annex give different definitions for use in a country.

Table 4.1 (NDP) — Definitions of consequence classes CC3-a and CC3-b for buildings

|  |  |
| --- | --- |
| CC3-a | Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse (e.g. schools, assembly halls, cultural institutions) |
| CC3-b | Buildings of installations of vital importance for civil protection (e.g. hospitals, fire stations, and their equipment), and buildings which should remain operational at all times (e.g. communication centres, data centres, and their equipment) |

NOTE 2 Buildings that house very dangerous installations or materials are generally classified as CC4, which is not fully covered by this standard.

1. For the application of prEN 1998-1-1:2022, 4.1(4), ** values shall be determined.

NOTE The values of ** applicable to buildings are those given in Table 4.2 (NDP), unless the relevant authorities or the National Annex give different values for use in a country.

Table 4.2 (NDP) — ** values for buildings

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | **Consequence class** | | | |
| CC1 | CC2 | CC3-a | CC3-b |
| ****** | 0,6 | 1,0 | 1,25 | 1,6 |

## Seismic actions

1. prEN 1998-1-1:2022, 4.3(3) and (4), should be applied.

NOTE 1 The values of return period *T*LS,CC, according to prEN 1998-1-1:2022, 4.3(3), are those given in Table 4.3 (NDP). The values in Table 4.3 (NDP) are computed with the values of **t,LS,CC suggested in prEN 1998-1-1:2022, F.3. If a relevant authority or the National Annex give different values of **t,LS,CC for use in a country, values of *T*LS,CC should be updated according to the note in prEN 1998-1-1:2022, 4.3(3).

NOTE 2 When performance factors are used, according to prEN 1998-1-1:2022, 4.3(4), the values of **LS,CC are those given in Table 4.4 (NDP). The values in Table 4.4 (NDP) are consistent with those in Table 4.3(NDP). If a relevant authority or the National Annex give different values of **t,LS,CC for use in a country, values of **LS,CC can be updated according to the note in prEN 1998-1-1:2022, 4.3(5).

NOTE 3 When the OP limit state is required for CC3 buildings and their equipment, and unless the relevant authorities or the National Annex give different values for use in a country, or, in the absence of such requirements, these values are agreed for a specific project by the relevant parties, then:

— the return periods and performance factors for the structure are the same as under the DL limit state,

— the return period and performance factor for equipment are 475 years and 1, respectively.

Table 4.3 (NDP) — Return period *T*LS,CC values, in years, for buildings

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Limit state** | **Consequence class** | | | |
| CC1 | CC2 | CC3-a | CC3-b |
| NC | 500 | 1350 | 2500 | 5000 |
| SD | 275 | 475 | 700 | 1000 |
| DL | 100 | 115 | 125 | 140 |

Table 4.4 (NDP) — Performance factor *γ*LS,CC values for buildings

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Limit state** | **Consequence class** | | | |
| CC1 | CC2 | CC3-a | CC3-b |
| NC | 1,00 | 1,40 | 1,70 | 2,20 |
| SD | 0,85 | 1,00 | 1,15 | 1,30 |
| DL | 0,60 | 0,60 | 0,65 | 0,65 |

1. If *S*v is greater than 6 m/s2, the vertical component of the seismic action, as defined in prEN 1998‑1‑1:2022, 5.2.2.3, should be considered for the verification of the following primary or secondary seismic members, and their directly associated supporting structural members, in a) to e):
2. horizontal or nearly horizontal structural members spanning longer than 20 m;
3. horizontal or nearly horizontal cantilever components longer than 5 m;
4. horizontal or nearly horizontal pre-stressed components;
5. beams supporting columns;
6. all structural members of the primary structure of transfer zones (see 6.2.11).
7. The value of ** introduced in prEN 1998-1-1:2022, 6.4.1(6), should be taken equal to 0,08 for buildings.

## Compliance criteria

1. Unless 14.8 is used for “simple masonry buildings”, the seismic performance of buildings should be verified according to Clause 6.

NOTE Specific rules for “simple masonry buildings” are given in 14.8, where they are defined. By conforming to those rules, such “simple masonry buildings” are deemed to satisfy the performance requirements of prEN 1998‑1‑2 without analytical safety verifications.

## Characteristics of earthquake resistant buildings

### Conceptual design

1. A building structure shall be able to resist horizontal actions in any direction.
2. Seismic performance of a building should be considered in the early stages of conceptual design, achieving a structural system which, with acceptable costs, satisfies the performance requirements specified in prEN 1998-1-1:2022, 4.1.

NOTE Guidance for good practice is given in informative Annex A.

1. The seismic action resisting system of a building may belong to one of the structural types in 10.4.1 for reinforced concrete buildings, in 11.4.1 for steel buildings, in 12.4.1 for composite steel-concrete buildings, in 13.4.1 for timber buildings, in 14.2.2 for masonry buildings, in 7.4 for masonry infilled frames and in 15.4.1 for aluminium buildings.
2. Seismic action resisting systems may be different, and made of different materials, in two main directions of a building.
3. Hybrid structures, combining in the same plane two or more of the types of structural systems as given in (3), may be used.
4. Seismic action resisting systems different from those in (3) may be used, if they satisfy 10.4.1(6) for reinforced concrete buildings, 11.4.1(3) for steel buildings, 12.4.1(6) for composite buildings, 13.4.1(4) for timber buildings and 15.4.1(3) for aluminium buildings.

### Primary and secondary seismic members

1. A certain number of structural members (e.g. beams and/or columns) may be designated as secondary seismic members, not forming part of the seismic action resisting system of the building. The strength and stiffness of these members against seismic actions should be neglected in the models for seismic analysis.
2. The secondary seismic members and their connections should be designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the most unfavourable seismic design situation. Due allowance of second-order effects (*P-* effects), in both the structure and these members, should be made in the design of these members.
3. Secondary columns which yield under the displacements caused by the seismic design situation in (2) should satisfy the requirements for local ductility of columns in DC2 given in 10.5.4.2 for reinforced concrete, in 11.8.3 for steel, in 12.8.3 for composite and in 15.8.3 for aluminium; connections in timber structures should satisfy 13.2.1(7) and (8).
4. The designation of some structural members as secondary seismic members should not change the classification of the structure from non-regular to regular as described in 4.4.3 and 4.4.4.
5. All structural members not designated as being secondary seismic members should be taken as primary. They should be modelled in the structural analysis in accordance with 5.1 and designed and detailed for earthquake resistance in accordance with 10 to 15.

NOTE The primary structure is composed of the set of primary seismic members over the building. A primary bracing is a set of primary members installed in a vertical plane at a given level of the building.

1. The contribution of lateral stiffness of secondary members to the total stiffness should not affect significantly the dynamic behaviour of the structure. To fulfil this requirement, either a) or b) should be satisfied:
2. The total contribution to lateral stiffness of all secondary seismic members does not exceed 15 % of that of all primary seismic members;
3. The total contribution to lateral stiffness of all secondary seismic members does not exceed 30 % of that of all primary seismic members and two analysis are performed: one with only the primary members in the model, the other one including all primary and secondary members in the model; the most unfavourable seismic action effects resulting from both analyses should be considered in the verification of the primary seismic members and in the capacity design of the members and connections adjacent to those primary seismic members.
4. The primary structure should be assigned a ductility class according to prEN 1998-1-1:2022, 4.5.2(3). The ductility class should be unique for the building.

### Torsionally flexible buildings

1. A building should be considered torsionally flexible if it satisfies a) or b):
2. in each main horizontal direction, the greatest effective modal mass is not that of the first or second mode, possible local modes and equipment modes not being considered in this classification;

NOTE This condition corresponds to the fact that the first mode (in at least one horizontal direction) is substantially influenced by torsion.

1. the period of the first predominant torsional mode of vibration is greater than the periods of the predominant translational modes in the two main horizontal directions, possible local modes and equipment modes not being considered in this classification.
2. Buildings verifying Formula (4.1) at every storey *i*, with the possible exception of the highest storey, may be considered as not being torsionally flexible.

(4.1)

where

|  |  |
| --- | --- |
|  | is the minimum torsional radius of the *i*-th floor, equal to the square root of the ratio of the torsional stiffness, with respect to the centre of lateral stiffness *C*i (Figure 4.1), to the largest of the floor lateral stiffnesses: ; |
|  | is the radius of gyration of floor *i*: ; in the case of approximately rectangular floor with a well distributed mass, may be taken as ; |
| and | are the mass moments of inertia of the floor around axis x and y respectively, going through the floor centre of mass *G*i; |
| *m*i | is the mass of floor *i*; |
| and | are the dimensions of the rectangular floor. |

NOTE Annex B gives procedures to calculate and .

Figure 4.1 — Relative position of each primary resisting member *k* with respect to the centre of lateral stiffness and relative position of the latter with respect to the centre of mass

### Structural regularity

#### General

1. Although not mandatory, achieving building regularity should be regarded as a good practice for the design of earthquake resistant buildings.

NOTE Guidance for in-plan regularity is given in informative Annex A, while criteria for regularity in elevation are given in 4.4.4.2.

#### Regularity in elevation

1. For a building to be categorized as being regular in elevation, it should satisfy all the conditions listed in a) to c), considering that criteria in b) and c) apply to the structure above the top of the foundation or the top of a rigid basement when it exists:
2. all primary members, such as cores, structural walls or frames and the diaphragms which connect them, provide a continuous resisting system without interruption from the top of the foundation or the top of a rigid basement to the top of the building or, if setbacks at different heights are present, to the top of the corresponding zone of the building.
3. both the lateral stiffness and the mass of the individual storeys remain constant or decrease gradually by no more than 20 % relative to the storey below, without abrupt changes, from the base to at least one storey below the top storey.
4. the ratio of the actual storey resistance to the resistance required by the analysis does not vary by more than 30 % between adjacent storeys.

NOTE Within this context, the special aspects of masonry infilled frames are treated in Clause 7.

# Modelling and structural analysis

## Modelling

### General

1. The model of the building should comply with prEN 1998-1-1:2022, 6.2.
2. Seismic analysis may be performed using two planar models, one for each main horizontal direction, if the height of the single or multi-storey building does not exceed, respectively, 15 m or 10 m, and if the floors may be considered as rigid in their planes according to 5.1.3(6). In that case, the structural (natural) mass eccentricity, multiplied by 1,1, should be taken into account in both directions for the calculation of the seismic action effects.

### Masses

1. The combination coefficients **E*i* in prEN 1998-1-1:2022, 6.2.1(3), for the masses associated to variable actions should be calculated with Formula (5.1) where ** values should not be smaller than those given in Table 5.1.

(5.1)

Table 5.1 — Minimum values of ** for calculating **E*i*

|  |  |
| --- | --- |
| **Type of variable action** | ** |
| Categories\* A and C | 0,5 |
| Other categories | 1,0 |
| \* Categories as defined in prEN 1991-1-1:2023, Table 8.1. | |

1. When the floor and roof diaphragms or bracings of the building may be considered as being rigid in their planes according to 5.1.3(6), the masses and the mass moments of inertia of each of those planes may be lumped at the centre of gravity.

### Stiffness

1. The stiffness of secondary seismic members against seismic actions should be neglected if the condition in 4.4.2(6) a) is satisfied; it should be taken into account according to 4.4.2(6) b) if the condition in 4.4.2(6) a) is not satisfied.
2. In concrete buildings, in composite steel-concrete buildings, in masonry buildings and in hybrid buildings as defined in 4.4.1(5), the stiffness of the concrete, composite and masonry primary seismic members should be evaluated considering the effects of cracking. For concrete members and composite steel-concrete members, this effect may be considered by adopting the stiffness corresponding to the initiation of yielding of the reinforcement, considering mean values of material properties and the presence of axial forces.
3. Unless a more accurate analysis of the cracked members is performed, the elastic flexural and shear stiffness properties of concrete, composite and masonry members according to (2) may be taken to be equal to one-half of the corresponding stiffness of the uncracked members or to other fractions, if any, of the corresponding stiffness of the uncracked members defined in Clauses 10, 11, 12, 13, 14 or 15.
4. According to prEN 1998-1-1:2022, 6.2.1(6), infill walls which contribute significantly to the lateral stiffness and resistance of the building should be considered.

NOTE See 7.4 for masonry infills of concrete, steel or composite frames.

1. For non-linear analyses, the model should comply with prEN 1998-1-1:2022, 6.2.1 to 6.2.3.
2. When appropriate for the application of the relevant provisions of this standard, a floor or a roof may be considered as being rigid in-plane in a given direction if it satisfies related rules per material and structural types in Clauses 10 to 15.

NOTE These rules are, in particular, detailed in 10.12, 10.14.2, 10.14.5, 11.17, 13.5(4), 13.5(5), 14.4, 14.5.2.3 and 15.13.

## Minimum design eccentricity in buildings

1. In application of prEN 1998-1-1:2022, 6.1(10), design effects of the seismic action should consider a minimal torsional effect about the vertical axis.
2. In order to comply with (1), a minimum eccentricity, measured at right angle to the considered direction *i* of the seismic action, should be calculated at every storey *j* using Formula (5.2) and should be taken into account if it exceeds the natural eccentricity *e*0,i,j.

(5.2)

where *L*i,j is the width of the floors at the considered level *j*, measured at right angle to the direction *i* of the seismic action considered.

NOTE 1 The purpose is to cover uncertainties in cases where the natural eccentricity is very low.

NOTE 2 Annex B gives procedures for the calculation of the natural eccentricity.

1. When the force-based approach is used, the effects at any storey *j* of mass eccentricity perpendicular to direction *i* may be calculated as resulting from a torsional moment *M*i,j, which acts around the building vertical axis, given by Formula (5.3).

(5.3)

where

|  |  |
| --- | --- |
|  | is the horizontal force acting on the considered storey *j* in direction *i*; |
| *e*i,j | is the maximum value of *e*min,*i,j* and *e*0,*i,j* when using planar models, or is equal to *e*min,i,j – *e*0,i,j when using 3D models that encompass natural eccentricityfor a building where *e*min,i,j is greater than *e*0,i,j. |

1. When the non-linear static analysis is used, (1) may be considered as satisfied if, in 5.3.5.3, calculations of *d*et and *d*et,j comply with (2) and (3).
2. When the response-history analysis is used, (1) may be considered as satisfied if the storey mass is modelled in such a way that the eccentricity of the centre of mass of the storey is at least equal to emin,i,j in both horizontal directions.

## Methods of analysis

### General

1. Depending on the selected method, corresponding provisions in prEN 1998-1-1:2022, 6.4 to 6.6, should be applied.
2. The vertical component of the seismic action may be neglected in structural analysis, except in the cases described in 4.2(2).

### Force-based approach

1. Tables of default values of the behaviour factor given in Clauses 10 to 15 may be used for a given structural type and a given ductility class DC. They correspond to upper bound values which should not be exceeded.

NOTE For the definition of *q*, see prEN 1998-1-1:2022, 6.4.1 (1).

1. For buildings which are non-regular in elevation, the ductility behaviour factor *q*D should not be greater than the default value multiplied by 0,8, but not smaller than 1.
2. If the primary structure or the regularity classification of the building in elevation is different in the two horizontal directions and if the building is not torsionally flexible, the value of the behaviour factor *q* may also be different in the two directions.
3. If primary structures of different types or materials are used to resist the seismic action in the same direction, the lowest value of the behaviour factor *q* of the different primary structures should be used.
4. For torsionally flexible buildings, the value of the behaviour factor *q* should be taken as the minimum of both horizontal directions where the primary bracings are different, multiplied by 0,8, but not smaller than *q*s.
5. In case the force-based approach is used, the *q*R factor given in prEN 1998-1-1:2022, 6.4.1, may be taken as given by Formula (5.4).

qRu/1 (5.4)

where

|  |  |
| --- | --- |
| **1 | is the value by which the horizontal seismic design action needs to be multiplied, in order to first reach the resistance of a dissipative zone in any member of the structure, while all other design actions remain constant; |
| **u | is the value by which the horizontal seismic design action needs to be multiplied, in order to form dissipative zones in a number of sections sufficient for the development of overall structural (plastic) instability, while all other design actions remain constant. Factor **u may be obtained from a nonlinear static (pushover) global analysis. |

1. For buildings which are torsionally flexible, *q*R should be taken equal to 1.
2. Except when (7) applies, default values of *q*R given in Clauses 10 to 15 may be used.

NOTE Default values of *q*R depend on the construction material and structural system.

### Lateral forces method of analysis

1. This type of analysis should comply with prEN 1998-1-1:2022, 6.4.1 and 6.4.2. It should not be used for buildings taller than 30 m and for buildings with *T*1 > min (4 *T*C; 1,5 s).
2. The seismic base shear force *F*b, for each horizontal direction in which the building is analysed, should be determined using Formula (5.5).

(5.5)

where

|  |  |
| --- | --- |
| *S*r(*T*1) | is the ordinate of the reduced spectrum (prEN 1998-1-1:2022, 6.4.1) at period *T*1; |
| *T*1 | is the fundamental period of vibration of the building for lateral motion in the direction considered; |
| *m* | is the total mass of the building, above the foundation or above the top of a rigid basement, in accordance with 5.1.2; |
| ** | = 0,85 if *T*1 < min (2 *T*C; 1,2 s) and the building has more than two storeys;  = 1,0 otherwise. |

NOTE The correction factor ** accounts for the fact that, in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller, on average by 15 %, than the total building mass.

1. The fundamental period of vibration *T*1 (in s) of the building and the associated mode shape may be calculated using approximate methods of structural dynamics or taken as given by Formula (5.6).

(5.6)

where

|  |  |
| --- | --- |
| *m*i | is the mass of the *i*-th storey, calculated in accordance with 5.1.2; |
| *s*i | is the displacement (in m) of the *i*-th storey under the acceleration of gravity, 9,81 m/s², applied to the structure in the considered horizontal direction. |

NOTE For specific timber structures, a simplified method is given in 13.5(6), Formula (13.6).

1. The seismic action effects should be determined by applying horizontal forces *F*i in each horizontal direction, according to Formula (5.7).

(5.7)

with the same notations as in (3).

### Response spectrum analysis

1. Response spectrum analysis should comply with prEN 1998-1-1:2022, 6.4.1 and 6.4.3. It may be applied to any building.

### Non-linear static analysis

#### General

1. Non-linear static analysis should conform to prEN 1998-1-1:2022, 6.5.
2. The seismic action should be applied with both positive and negative signs and the most unfavourable seismic action effects resulting from the two cases should be used.
3. In low-rise wall buildings in which the bending moment in walls is low in comparison to shear action effects, each storey may be analysed independently.

#### Construction of the capacity curve

1. prEN 1998-1-1:2022, 6.5.1 and 6.5.2, should be applied, with the additional provisions given in (2) to (5).
2. The value in prEN 1998-1-1:2022, Formula (6.20), should be defined at the *i*-th storey of the building.
3. If the building satisfies the condition for applying the lateral forces method of analysis given in 5.3.3(1), the values of may be based on the displacement corresponding to the horizontal forces defined in Formula (5.7).
4. The lateral force , given by prEN 1998-1-1:2022, Formula (6.20), should be applied at the location of the centre of masses at each storey.
5. The control displacement *d*n, as in prEN 1998-1-1:2022, 6.5.2(5), should be the displacement at the centre of mass of the slab at the top of the building (or at the slab one storey below if the top storey has less than 50 % of the mass of the storey below).

#### Seismic action effects on structure and structural members

1. prEN 1998-1-1:2022, 6.3(1) and 6.5.1(2), may be considered as satisfied if the effects obtained from the pushover analysis are corrected as given in a) and b):
2. displacements at different locations in the building are multiplied by the correction factor *c*P,k, defined in (6), where *k* is the index denoting the location of the structural member in plan;
3. other (generalized) deformations, such as storey drifts and rotations, and (generalized) stresses, are multiplied by the product of correction factors *c*P,k and *c*E,i, defined in (6) and (8). The corrected (generalized) stress should not be greater than that determined from the (generalized) stress – (generalized) deformation diagram.

NOTE The correction factors *c*P,k and *c*E,i primarily account, respectively, for the torsional effects and higher mode effects in elevation. The values of *c*P,k vary in plan, while the values of *c*E*,*i vary in elevation of the building.

1. Correction factor *c*P,k may be taken equal to 1 if, in both horizontal directions and at each storey, the natural eccentricity *e*0,i,j is not greater than , where is defined in 5.2(2).
2. Correction factor *c*E,i may be taken equal to 1 if conditions a) to c) are satisfied:
3. the height of the building is smaller than 25 m;
4. *T*1< min (4*T*C; 1,5 s);
5. the resistance of any storey is not less than 80 % of that required by the force-based approach, using the value of *q* applicable to the structure under consideration.
6. The values of the corrections factors *c*P,k and *c*E,i should be calculated as the ratio between normalized deformations obtained from the linear elastic analysis and the ones from the pushover analysis. The normalized deformations from linear elastic analysis should be calculated by the response spectrum method (see 5.3.3), with consideration of the effects of torsion due to minimum eccentricity given by Formula (5.2) and the effects of combination of horizontal components of the seismic action (prEN 1998‑1-1:2022, 6.4.4). If conditions for the use of the lateral forces method are satisfied, it may be used for the calculation of the correction factors.

NOTE It is assumed that correction factors *c*P,k and *c*E,i account both for the effects of torsion due to minimum eccentricity and those due to the combination of horizontal components of the seismic action, if they are calculated as defined in (4). In this case, *c*dt = 1 (prEN 1998-1-1:2022, 6.5.4(6)).

1. For each structural member, the correction factor *c*P,k should be calculated for each direction of lateral forces for pushover analysis.
2. Unless (7) is applied, the value of *c*P,k should be determined from Formula (5.8).

(5.8)

where

|  |  |
| --- | --- |
| *k* | is the index denoting the location of the structural member in plan; |
| *d*et, *d*et,k | are the values of the control displacement, from the linear elastic analysis for the design seismic action, and the corresponding displacement at location *k* in plan, respectively; |
| *d*t, *d*t,k | are the target displacements associated with the considered limit state and the corresponding displacement at location *k* in plan. |

1. If the structural system is not torsionally flexible (see 5.2) and if, in both horizontal directions *i* and at each storey *j*, the two components of the natural eccentricity, *e*0x,j and *e*0y,j are not greater than 0,10 *L*i,j, the correction factor *c*P,k may be taken from Formula (5.9).

(5.9)

where

|  |  |
| --- | --- |
| *L*i,j | is defined in 5.2(2); |
| *x* | is the distance of the member *k* under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered; |
| *L*e | is the distance between the two-outermost lateral load resisting members, measured perpendicularly to the direction of the seismic action considered. |

1. Correction factor *c*E,i should be calculated for each direction of lateral forces for pushover analysis from Formula (5.10).

(5.10)

where

|  |  |
| --- | --- |
| *i* | is the index denoting the storey of the structural member; |
| *d*t and *d*et | as in (6); |
| *d*ret,i*,* | is the interstorey drift at the centre of mass of the *i*-th storey, from the linear elastic analysis for the design seismic action, in the direction under consideration; |
| *d*rt,i | is the interstorey drift at the centre of mass of the *i*-th storey which corresponds to the target displacement due to the design seismic action, in the direction under consideration. |

### Response-history analysis

1. Response-history analysis should conform to prEN 1998-1-1:2022, 6.6.

# Verifications of structural members to limit states

## General

1. Provisions of prEN 1998-1-1:2022, 6.7, should be applied, taking into account the Limit State under consideration and the associated seismic action.
2. For buildings of consequence classes other than CC3-b (see Table 4.1) the verifications in 6.2 and 6.3 may be considered as satisfied if conditions a) and b) are satisfied:
3. prEN 1998-1-1:2022, 4.3, 4.5.2(7) and 6.7, are satisfied and, if its height is taller than 20 m, the building is regular in elevation;
4. The shear force over the entire structure at the base level of the building(foundation or top of a rigid basement) in the seismic design situation is less than the corresponding shear force in other design situations, based on a linear analysis.

NOTE For example, this can be the case in regions where the wind action is more demanding than the seismic action.

1. For the application of (2), in case of the force-based approach, the base shear force should be calculated with a behaviour factor equal to *q*S.
2. For geotechnical structures as defined in prEN 1998-5, values of acceptable displacements, given in a) to e) as appropriate, should be determined at the Limit States considered for the building, and their effects taken into account in the verification of the building and of the geotechnical structures:
3. residual ground displacements of slopes;
4. settlements of soils under cyclic loading or liquefaction;
5. foundation sliding displacement;
6. residual rotation and settlement of the foundation;
7. permanent displacements of retaining structures.

NOTE prEN 1998-5 provides rules for geotechnical structures to comply with these limitations.

1. Anchorages to concrete should be verified using prEN 1998-1-1:2022, Annex G.

## Verification of Significant Damage (SD) limit state

### General

1. In application of prEN 1998-1-1:2022, 6.7.1 and 6.7.2, the SD limit state may be considered as verified if the conditions in 6.2.2 to 6.2.12 are satisfied.
2. In the primary structure of the building, for all ductility classes, global plastic mechanisms should be controlled through limitation of drift (6.2.5) and of second-order effects (6.2.4) and, in DC2 and DC3, through capacity design (6.2.6 and 6.2.7).

### Equilibrium condition

1. In application of prEN 1998-1-1:2022, 6.7.1(4), the building structure should be stable – including overturning or sliding – in the seismic design situation.

### Resistance conditions

1. prEN 1998-1-1:2022, 6.7.1 and 6.7.2, should be applied, supplemented as given in a) and b):
2. For the calculation of action effects, *E*d, redistribution of bending moments in accordance with prEN 1992-1-1, EN 1993-1-1 and prEN 1994-1-1 may be applied.
3. The design resistance*, R*d, should be calculated in accordance with the rules specific to the material used (in terms of the characteristic values of material properties *f*k and partial factor **M) and in accordance with the mechanical models which relate to the specific type of structural system, as given in Clauses 10 to 15 and in other relevant Eurocode parts. In application of prEN 1998-1-1:2022, 6.7.1(1), the resistance conditions should be expressed in terms either of generalized stresses (internal forces) or of generalized strains such as interstorey drifts or member chord rotations.
4. In case a displacement-based approach is used, generalized deformations, according to case a) or b) of prEN 1998-1-1:2022, 6.7.1(2), as specified in Clauses 10 to 15, should be used for verification of ductile mechanisms to SD. Generalized stresses should be used for the verification of brittle failure modes.
5. In case a displacement-based approach is used, resistances (calculated using mean values of material properties *f*m) should be divided by a partial factor **Rd, given in Clauses 10 to 15.

NOTE According to NOTE 2 in prEN 1998-1-1:2022, 6.7.1(1), the partial factor on resistance **Rd is function of the target reliability for the LS and CC, **t,LS,CC (suggested values in prEN 1998-1-1:2022, F.3) and the total uncertainty of the resistance model, **lnR, according to **Rd=exp(**\*R**t,LS,CC**lnR), where the resistance sensitivity factor is **\*R=0,8.

1. It should be verified that the structural members comply with the design provisions that are required by the selected ductility class.

NOTE These provisions are given in Clauses 10 to 15. They aim, in particular, at preventing brittle mechanisms and instability in critical zones. They complement provisions of the corresponding material Eurocode.

1. Fatigue resistance verification may be neglected under the seismic design situation.

### Control of second-order effects

1. Second-order effects (*P-* effects) may be neglected if the condition given by Formula (6.1) is fulfilled in all storeys.

(6.1)

where ** is the interstorey drift sensitivity coefficient, given by Formula (6.2) in the force-based approach and by Formula (6.3) in the displacement-based approach, unless different formulae are given for a specific material in the relevant clause.

(6.2)

(6.3)

where

|  |  |
| --- | --- |
| *P*tot | is the total gravity load at and above the storey, due to the masses considered in the seismic analysis of the structure, in accordance with 5.1.2(1) and prEN 1998-1-1:2022, 6.2.1(3); |
| *d*r,SD | is the design interstorey drift, defined as the difference of the average lateral displacements *d*sat the top and bottom of the storey under consideration; It is calculated in accordance with prEN 1998-1-1:2022, Formula (6.9), for the force-based approach, and calculated at the SD limit state, from the analysis, for the displacement-based approach; |
| *V*tot | is the total storey shear in the seismic design situation for the force-based approach; |
| *V*s,SD | is the total storey shear at SD limit state, calculated from the analysis, for the displacement-based approach; |
| *h*s | is the interstorey height; |
| *q*S and *q*R | are the behaviour factor components according to prEN 1998-1-1:2022, 6.4.1(1), specified in Clauses 10 to 15 for structural materials, for the force-based approach. |

1. If 0,1 < **  0,2, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to 1/ (1 – **).
2. If 0,2 < **  0,3 at any storey, the second-order effects should be taken into account directly by using an established method of second-order analysis which takes account of geometric non-linearity, i.e. consider the equilibrium conditions on the deformed structure.
3. The value of ** should not exceed 0,3.
4. If the second-order effects are taken into account directly by using an established method of second-order analysis, (2) and (3) do not apply and ** calculated with Formula (6.2) or (6.3) may be divided by (1+**) in the verification of (4).

### Limitation of interstorey drift

1. Interstorey drift should be limited at any storey of the building by complying with the condition given by Formula (6.4).

(6.4)

where

|  |  |
| --- | --- |
| *d*r,SD and *h*s | are defined in 6.2.4(1); |
| *λ*s | is a coefficient reflecting the drift limit, given in 10.4.4, 11.6.4, 12.6.4, 13.6.2 and 15.6.3 for the different structural types. |

NOTE Drift limitation at SD applies to all ductility classes and all design approaches.

### Capacity design in DC2

1. Specific material-related provisions defined in Clauses 10 to 15 should be satisfied.
2. Structures with dissipative zones should be designed such that yielding, cracking, local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.
3. Capacity design provisions defined in Clauses 10 to 15 should be satisfied.

NOTE These provisions are meant to avoid brittle failure modes and to obtain the hierarchy of resistance of the various structural components within specific zones, like the most stressed zones of the structure or the connections between structural members.

1. Dissipative zones may be in the structural members or in the connections.
2. If dissipative zones are in the structural members, the non-dissipative zones and the connections of the dissipative zones to the rest of the structure should have sufficient overstrength to allow the development of cyclic yielding in the dissipative zones.
3. If dissipative zones are in the connections, the connected members should have sufficient overstrength to allow the development of cyclic yielding in the connections.
4. In multi-storey buildings, formation of a soft-storey plastic mechanism should be prevented.

NOTE 1 Such a mechanism can entail excessive local ductility demands in the columns of the soft storey.

NOTE 2 The *q* factors given in this standard are valid if and only if (7) is fulfilled.

1. For concrete wall systems, concrete wall-equivalent dual systems with concrete or dual with steel structures or dual with composite steel-concrete structures, (7) may be considered satisfied.
2. For a seismic action index *S*δ not greater than the limits for moment-resisting frames and for moment frame-equivalent dual systems in DC2 given in Clauses 10 to 15, (7) may be considered satisfied for systems other than in (8) if they satisfy a) and b):
3. they are regular in elevation;
4. they comply with the interstorey drift limitations given in 6.2.5(1).
5. In case a force-based approach is used, (7) may be considered satisfied for moment-resisting frame systems and moment frame-equivalent dual systems regular in elevation, if the condition given by Formula (6.5) is satisfied in all storeys of the moment resisting frame.
6. In case a displacement-based approach is used, (7) may be considered satisfied for moment-resisting frame systems and moment frame-equivalent dual systems, if the plastic rotation demand in the seismic design situation nowhere exceeds the corresponding resistance .

### Capacity design in DC3

1. 6.2.6(1) to (8) should be applied.
2. Structures with dissipative zones should be designed such that yielding or cracking or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.
3. Unless otherwise specified in Clauses 10 to 15, to satisfy 6.2.6(7) in moment resisting frame buildings, including moment resisting frame equivalent ones as defined in 10.1.2(1), with two or more storeys, the condition given by Formula (6.5) should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns.

(6.5)

where

|  |  |
| --- | --- |
| *M*Rc | is the sum of the design absolute values of the moments of resistance of the columns framing the joint. The minimum absolute value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in Formula (6.5); |
| *M*Rb | is the sum of the design absolute values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are considered in the calculation of *M*Rb. |

NOTE A rigorous interpretation of Formula (6.5) requires calculation of the moments at the centre of the joint. These moments correspond to development of the design values of the moments of resistance of the columns or beams at the outside faces of the joint, plus a suitable allowance for moments due to shears at the joint faces. However, an acceptable approximation, with minor loss of accuracy, consists of neglecting the moments due to shear at the joint faces.

1. Formula (6.5) should be satisfied in two orthogonal vertical planes of bending, which, in buildings with frames arranged in two orthogonal directions, are defined by these two directions. It should be satisfied for both directions (positive and negative) of action of the beam moments around the joint, with the column moments always opposing the beam moments. If the structural system is a moment resisting frame or equivalent to a moment resisting frame in only one of its two main horizontal directions, Formula (6.5) should be satisfied only within the vertical plane through that direction.
2. (2) may be neglected at the top level of multi-storey buildings.

### Resistance of horizontal diaphragms and bracings

1. Diaphragms and bracings in horizontal planes and quasi horizontal planes (roofs) should be able to transmit, with appropriate overstrength when required, the effects of the design seismic action to the lateral load-resisting systems to which they are connected.

NOTE Design provisions for diaphragms are given in Clauses 10 to 15 according to the structural material used.

1. (1) may be considered satisfied if, for the relevant resistance verifications, the seismic action effects in the diaphragm or bracing obtained from the analysis are multiplied by an overstrength factor **d given in Table 6.1.

Table 6.1 — **d values for diaphragms

|  |  |  |  |
| --- | --- | --- | --- |
| **Diaphragm failure mode** | **DC1** | **DC2** | **DC3** |
| **Ductile** | 1,0 | 1,1 | 1,2 |
| **Brittle** | 1,0 | 1,2 | 1,5 |

### Resistance of foundations

1. The foundation system shall conform to prEN 1998-5:2022, Clause 9.

### Seismic joint condition

1. Buildings should be protected from earthquake-induced pounding with adjacent structures or between structurally independent units of the same building.
2. (1) may be considered satisfied if a) or b) applies:
3. for buildings, or structurally independent units, that do not belong to the same property, if the distance from the property line to the potential points of impact is not less than the maximum horizontal displacement of the building at the corresponding level, calculated in accordance with prEN 1998-1-1:2022, Formula (6.9), for the force-based approach or corresponding to the target displacement of the SDOF system, given by prEN 1998-1-1:2022, Formula (6.29), for the displacement-based approach;
4. for buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum of the squares (SRSS) of the maximum horizontal displacements of the two buildings or units at the corresponding level, calculated in accordance with prEN 1998-1-1:2022, Formula (6.9), for the force-based approach or corresponding to the target displacement of the SDOF system, given by prEN 1998-1-1:2022, Formula (6.29), for the displacement-based approach.
5. If the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be multiplied by a factor of 0,7.

### Verification of transfer zones in DC2 and DC3

NOTE The verification provisions for the structural members of transfer zones are given in 6.2.11. Some specific material related provisions are given in 10.5.2, 11.18, 12.16 and 13.16.

1. A transfer zone shall be designed to resist the action effects including a magnification factor.
2. The magnification factor in (1) should be applied to the seismic action effects from the analysis which enter in the calculation of the design forces in the transfer zone and should be taken equal to 1,25*ω*d, as given in Formula (6.6).
3. The effect of the vertical component of earthquake should be taken into account in the design of the structural members of transfer zones as given in 4.2(2).
4. There may be more than one transfer zone in a building.
5. The model of the structure for analysis should be in 3D with explicit modelling of the structural members of the transfer zone and below it.
6. The analysis should be a 3D response spectrum analysis or a 3D non-linear response-history analysis.
7. Buildings in which the interstorey drift at a storey of the primary structure in the transfer zone or below is more than two (2) times greater than the interstorey drift of the first storey above the transfer zone, *d*r,SD, as defined in 6.2.4(1), should be designed to DC3 with q ≤ 2.
8. The resistance to the bending moment, shear and vertical forces at the bottom of an interrupted wall or column at any level may be provided by one or more of provisions a) to c):
9. the diaphragm at the level of the interruption and a beam placed in the plane of the wall which is supported by columns placed in the same plane;
10. a couple of horizontal forces in two diaphragms, one at the bottom level of the interrupted member and the other one at an upper level;
11. columns placed in the plane of the interrupted wall.
12. In a force–based approach, the design action effects in structural members in the height of a transfer zone under the interrupted member, like beams, columns, walls and diaphragms, should be calculated using Formula (6.6).

with (6.6)

where

|  |  |
| --- | --- |
| *E*F,G | is the action effect of the non-seismic actions in the transfer zone included in the combination of actions for the seismic design situation (EN 1990:2023, 8.3.4.4); |
| *E*F,E | is the action effect of the design seismic action in the transfer zone calculated from the seismic analysis of the structure; |
| *Ω*d | is the ratio of the design resistance *R*di to the action effect in the seismic design situation *E*di at the dissipative zone of member *i* of the structure above the transfer zone which has the highest influence on the effect *E* in the members of the transfer zone under consideration -with *Ω*d = 1 in large reinforced concrete walls (see 10.9); |
| "+" | means combined with + or – sign. |

NOTE The action effect considered in the calculation of *Ω*d is the one by which the dissipative zone under consideration dissipates energy, e.g. bending moment for plastic hinge in bending, tension in the diagonal of a bracing.

1. Condition given by Formula (6.6) may be neglected in the transfer zone.

### Verification of underground basements

1. Underground basements should be verified under the action of surrounding soil in the seismic design situation, considering, for each component of the seismic action, the most unfavourable effects resulting from the application of the action as defined in (2) in one direction or the opposite.
2. The soil pressures due to the seismic action on the windward side of the basement should be as given in prEN 1998-5:2022, 10.3.2.
3. The verification of overall stability of the building and resistance of soil should be made as specified in prEN 1998-5:2022, 9.4.2.
4. The verification of the underground structures of the basement should take into account the effects of actions resulting from (1), (2) and (3).

## Verification to other limit states

### Verification to Near Collapse (NC) limit state

1. In case the NC limit state is used, it should be verified that, under the specified seismic action, the deformation limits or other relevant limits given in prEN 1998-3 are not exceeded.

### Verification of Damage Limitation (DL) limit state

#### General

1. The DL limit state may be considered as verified when the interstorey drifts due to the seismic action associated to the DL limit state are limited in accordance with 6.3.2.2.
2. Additional damage limitation verifications may be required for buildings of consequence class CC3-b, specific to the building under consideration.

NOTE The additional verifications at DL limit state applied to specified buildings can be specified by a relevant Authority or can be found in the National Annex or, where not specified, agreed for a specific project by the relevant parties.

#### Limitation of interstorey drift

1. Unless otherwise specified in Clauses 10 to 15 or if (2) applies, the condition given by Formula (6.7) should be satisfied.

(6.7)

where

|  |  |
| --- | --- |
| *d*r,DL | has a similar definition as *d*r,SD in 6.2.4(1), but calculated for the DL Limit state; |
| *h*s | is the storey height; |
| *λ*ns | coefficient accounting for sensitivity of ancillary elements to interstorey drift: |
|  | = 0,002 5 for buildings having ancillary elements of unreinforced masonry units of Group 4 attached to the structure, |
|  | = 0,004 5 for buildings having ancillary elements of brittle materials attached to the structure, in particular unreinforced masonry with clay units of Groups 1, 2 or 3 with a thickness greater than 200 mm and the normalized mean compressive strength *f*b ≥ 3 MPa, |
|  | = 0,007 5 for buildings having ductile ancillary elements attached to the structure. |

NOTE Masonry unit groups are defined in EN 1996-1-1:2022, 5.1.

1. Building structures and ancillary elements which satisfy 7.4.3 at SD are considered to satisfy Formula (6.7).

### Verification of fully Operational (OP) limit state

1. Criteria applicable to the building structure in addition to prEN 1998-1-1:2022, 6.7.3(7), should be derived from the analysis of the components the operability of which is required as well as from the analysis of their supporting systems.

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 provisions.

# Ancillary elements

## General

1. Ancillary elements of buildings that might, in case of failure, pose risks to human life or affect the main structure of the building or the services of facilities, shall, together with their supports, be verified to resist the design seismic action in two orthogonal horizontal directions.

NOTE Examples of ancillary elements are claddings, parapets, gables, antennas, mechanical appendages and equipment, curtain walls, partitions, railings, ceilings.

1. For application of (1), the seismic analysis should be based on a realistic model of the relevant structures and on an appropriate floor response spectrum derived from the response of the supporting structural members of the main seismic resisting system.
2. If the ratio of the ancillary component mass, *m*a, to floor mass, *m*i, is greater than 0,01, the action effects in the ancillary elements and in the structural members may be calculated taking into account their dynamic interaction.

## Verification at Significant Damage (SD) limit state

### Seismic action effects

1. The ancillary elements, as well as their connections to the structure, should be verified for the seismic design situation in terms of acceleration and displacement in two orthogonal horizontal directions.
2. The local transfer of action effects to the structure due to the fastening of ancillary elements and their influence on the seismic response should be accounted for.

NOTE The provisions for fastenings to concrete are given in prEN 1998-1-1:2022,Annex G**.**

1. The seismic action effects on ancillary elements may be determined by applying to them in the horizontal direction considered a horizontal force *F*ap defined by Formula (7.1).

*F*ap = *γ*ap *m*ap *S*ap/*q*ap’ (7.1)

where

|  |  |
| --- | --- |
| *F*ap | is the horizontal seismic force, acting at the centre of mass of the ancillary element in the most unfavourable direction; |
| *m*ap | is the mass of the element; |
| *S*ap | is the value in the floor acceleration spectrum, see (4); |
| **ap | is the performance factor of the element, see 7.2.2; |
| *q*ap’ | is the period dependent behaviour factor of the ancillary element, given in Annex C. |

1. The floor acceleration spectral values *S*ap should be determined for two horizontal directions.
2. In case of ancillary elements with uniformly distributed mass (e.g. partition wall), the resultant force *F*ap may be distributed proportionally to the mass or its deformed shape.
3. In case floor response spectra are available, the *S*ap value at floor *j*, *S*ap*,j*, in the direction under consideration may be taken as given by Formula (7.2).

(7.2)

where

|  |  |
| --- | --- |
| *T*ap | is the natural period of the ancillary element; |
| **ap | is the critical damping ratio of the ancillary element, which may be taken equal to 2 %, unless greater values are demonstrated; |
| *S*floor,j | is the response spectrum at the floor *j* in the direction under consideration. |

NOTE The mode shape of the ancillary elements is affected by its boundary conditions which depend on its connections to the structure.

1. In case the ancillary element meets the conditions a) or b) given in (8), the *S*ap value at floor *j*, *S*ap*,j*, in the direction under consideration may be taken as given by Formula (7.3).

(7.3)

where

|  |  |
| --- | --- |
| *H* | is the height of the building above the foundations or the top of a rigid basement; |
| *z*j | is the height of floor *j* above the foundation or the top of a rigid basement; |
| *S***, *F*A | are values associated to the response elastic spectrum associated to the limit state under consideration, defined in prEN 1998-1-1:2022, 5.2.2.2; |
| *Γ1* | is the participation factor of the fundamental mode of the building in the direction under consideration, which may be taken as given by Formula (7.4): |

(7.4)

|  |  |
| --- | --- |
| *N*s | is the number of storeys; |
| *q*D’ | is the period dependent behaviour factor, taken as given by Formula (7.5): |

(7.5)

|  |  |
| --- | --- |
| *T*P1 | is the fundamental period of the building in the considered direction; |
| *T*A , *T*C | are corner periods introduced in prEN 1998-1-1:2022, 5.2.2.2(1); |
| *q*D | is the building behaviour factor component accounting for deformation capacity and energy dissipation capacity provided in the relevant parts of EN 1998. |

1. The ancillary element may be considered rigid if a) or b) are satisfied:
2. The element is one of the following:
3. cantilevering parapets or ornamentations;
4. signs or billboards;
5. chimneys or masts not taller than 4 m.
6. If *T*ap  *T*B, where *T*B is a corner period determined in prEN 1998-1-1:2022, 5.2.2.2(1).
7. When the ancillary element does not satisfy a) or b) in (8), Sap may be calculated using Annex C.
8. If the displacement-based approach is used for the analysis of the structure, the ratio *ηS*α/*q*D’ in Formula (7.3) should be replaced by the acceleration of the inelastic primary structure determined as *S*ay = *F*\*y/*m*\*, where *F*\*y and *m*\* are given in prEN 1998-1-1:2022, 6.5.3. In case of hardening after yielding, the force at yield should be used as *F*\*y instead of the force at the target displacement.

### Performance factors

1. The performance factor **ap of ancillary elements should not be smaller than 1,0.

NOTE Except for elements participating to safety systems in (2), the value of **ap is 1,0, unless a relevant Authority or the National Annex or the relevant parties for a specific project set greater values.

1. For anchorage elements of machinery or for equipment participating to safety systems, the performance factor **ap should be greater than in (1).

NOTE The value of **ap for this type of elements is 1,5, unless a relevant Authority or the National Annex or, in the absence of such guidance, the relevant parties for a specific project set different values.

## Verification at Near Collapse (NC) limit state

1. In case the NC limit state is used, it should be verified that, under the specified seismic action, the deformation limits or other relevant limits given in prEN 1998-3 are not exceeded.

## Masonry infilled frames

### General

1. Masonry infills should be classified as interacting or non-interacting as defined in 3.1.16 and 3.1.19; interacting infills may be ductile or non-ductile.
2. Infills in contact with the frame without separation joints should be classified as interacting.
3. Ductile infills should accommodate without degradations the deformations of the frame up to the SD design stage.
4. Masonry infills may be classified as ductile if they can sustain without degradation drifts not smaller than the limits at OP, DL and SD as given in Table 7.1.
5. Infills with horizontal or vertical sliding planes along which a displacement capacity satisfying (3) exist may be considered ductile.
6. Masonry infills with only vertical or only horizontal reinforcement or with reinforcement mesh in a layer of plaster should be considered as unreinforced.
7. For concrete wall systems, the in-plane interaction with the masonry infills may be neglected.
8. The limits of seismic action and the behaviour factors for design of frames with infills should be taken as given in Clauses 10, 11, 12 and 13.
9. Braced timber frame structures with interacting masonry infill should be designed to 13.13.

### Design of frames with interacting infills

#### Basis of design and limitation of drift

1. 7.4.2 should be applied in DC1, DC2 and DC3 for the design of frames with interacting masonry infills.
2. Interacting infills may be realized with unreinforced, confined and reinforced masonry as defined in EN 1996-1-1.
3. The effects of interacting infills on the global response of a structure and the local effects due to the frame-infill interaction-like local additional shear in columns should be taken into account.

NOTE In comparison to bare frames, the presence of interacting infills increases the stiffness of structures, reduces their periods and modifies the action effects. An unsymmetrical distribution of interacting infills in plan in an otherwise symmetrical structure induces torsion and an increase in the ductility demand in members of the frame. An irregular distribution of interacting infills in elevation can increase the action effects in the non-infilled storeys. The lack of strength and/or the limited drift capacity of interacting infills can cause premature failure of some of them so that the subsequent distribution of effective infills can correspond to an irregularity in elevation or to a more unsymmetrical distribution of stiffness in plan.

1. The effects on the global response of a structure of in plan non-symmetrical layout of interacting infills should be taken into account.
2. The effects on the global response of a structure of irregularity in elevation due to interacting infills should be taken into account.
3. It should be verified that the out-of-plane seismic action effects on interacting infills are smaller than their out-of-plane resistance.
4. The calculated interstorey drift, *d*r,SD and *d*r,DL as defined in 6.2.4(1) and calculated according to 7.4.2.2 or 7.4.2.3 should not be greater than the limits at SD and, when applicable, at DL given in Table 7.1.

NOTE (7) is meant to protect infills from premature failure, to prevent the structural irregularities induced by infill failure and to protect human lives.

Table 7.1 — Limits of interstorey drift in % in buildings with masonry infills

|  |  |  |  |
| --- | --- | --- | --- |
| **Masonry Type** | **Drift limits**  **at OP [%]** | **Drift limits**  **at DL [%]** | **Drift limits**  **at SD [%]** |
| Ductile masonry infills as per 7.4.1 (4) | 0,40 | 0,75 | 2,00 |
| Unreinforced masonry with clay units in Groups 1, 2 or 3 with a thickness ≥ 200 mm and *fk* ≥ 3 MPa | 0,25 | 0,45 | 1,40 |
| Unreinforced masonry with units of Group 4 | 0,15 | 0,25 | 0,90 |

1. For units of unreinforced masonry other than those in Table 7.1, values of drift justified by tests may be used.

NOTE EN 1990:2023, Annex D, provides guidance on design assisted by testing.

1. In case of infills with openings, the limits of drift given in Table 7.1 should be reduced of at least 30 %.
2. With confined and reinforced masonry infills or in the case of insertion of vertical or horizontal reinforcement in the masonry panel or in the case of insertion of a reinforcement mesh in the plaster, the values in Table 7.1 may be increased by a factor not greater than 1,20.

#### Analysis with a model of the bare frame only

1. Regular structures with regular infill distribution may be analysed with a bare frame model and the interstorey drift demand may be calculated using a simplified procedure assuming a linear elastic behaviour and taking into account the infills only for their masses, layout and type. This simplified procedure should take into account the contribution of infills to the stiffness of the structure.

NOTE Annex K gives a simplified procedure for regular RC structures with infills regularly distributed in elevation.

1. (1) may be applied if at any storey both ratios *R*sym,x and *R*sym,y are smaller than 0,2, where *R*sym,x is given by Formula(7.6) and *R*sym,y by a similar Formula, replacing *x* by *y*.

(7.6)

where

|  |  |
| --- | --- |
| *l*infill,left,x,i | is the length of an infill parallel to axis x situated at left of axis *x*; |
| *l*infill,right,x,i | is the length of an infill parallel to axis x situated at the right of axis *x*; |
| axis *x* | is the axis in direction *x* passing through the centre of rigidity of the bare frame; |
| *di* | is the distance from infill *i* to axis *x*. |

NOTE The ratios *R*sym,x and *R*sym,y characterize the regularity in plan of the interacting infills layout.

1. Structures with infilled frames for which the ratios *R*sym,x and *R*sym,y  do not satisfy (2) but are both smaller than 0,4 may be analysed with a bare frame model; the seismic design action effects should be taken as their values from the analysis amplified by 1,3 and the design interstorey drift demand as those calculated as in (1) amplified by 1,3.
2. The analysis of structures with infilled frames for which one of the ratios *R*sym,x or *R*sym,y is greater than 0,4 should conform to 7.4.2.3.
3. If the total horizontal cross-sectional area of infills is reduced by more than 30 % in one or more consecutive storeys compared to the storey below or above, an increase of the seismic action effects in the vertical members of the storey where the reduction takes place should be taken into account.
4. (5) may be implemented by multiplying the seismic action effects calculated in the storey where infills are reduced by a factor *K*IR given by Formula (7.7).

(7.7)

where

|  |  |
| --- | --- |
| *ΔV*Rw | is the difference between the resistance of the storey above storey *i* which contains the largest quantity of infills and the resistance of the storey *i* under consideration, the resistance of a storey being the sum of the shear resistance of infills at that storey; |
| Ʃ*V*Ed | is the sum of the calculated seismic shear forces in all vertical primary seismic members of storey *i*; |
| *q* | is the behaviour factor used in the design of the structure. |

1. If Formula (7.5) leads to a magnification factor *KIR* smaller than 1,1, the amplification of the calculated action effects may be neglected.
2. In (6), the resistance of an infill panel with one or more openings may be calculated as the resistance of a panel without openings, multiplied by factor *ρ*op given by Formula (7.11).
3. In (1), (2) and (6), infill panels with one or more openings should be neglected if the ratio of the sum of the areas of openings to the total area of the infill is greater than 0,4.

NOTE Panels with openings have a very limited strength and stiffness. Overestimating the stiffness of a panel with openings induce wrong estimates of periods, torsion effects, etc.

#### Analysis with a model of the interaction between frame and infills

1. In a linear elastic analysis, each infill may be modelled as a strut having the elastic modulus of the infilled masonry, the thickness *tp* of the infill and a width *ws* as given by Formula (7.8).

(7.8)

where

*ls* is the length of the panel diagonal.

NOTE A single concentric strut framing into beam-column joints and with the same stiffness in tension and compression can be used to model the two diagonals of infilled panels and calculate their effect on global deformation and action effects in structural members, but not their local effects like local additional shear in columns where compression struts are supported. The local effects can be taken into account with 7.4.3.4 but they can also be obtained directly in a model in which the eccentricity of struts to the intersection of the beam and column axis is represented.

1. 7.4.2.2(8) and (9) should be applied.
2. In (1), the stiffness of an infill panel with one or more openings may be calculated as the stiffness of a panel without openings, multiplied by factor *ρ*op given by Formula (7.13).

#### Non-linear analysis of frames with interacting infills

1. Non-linear analysis should conform to 5.3.5.

NOTE 1 In non-linear analysis, the in-plane response of masonry infills can be modelled in various ways, from detailed micro-modelling to simple macro-modelling, like for example by diagonal single elements or multi-strut elements.

NOTE 2 With interacting infills, the additional local shear in the columns can be evaluated directly using eccentric struts or, indirectly, considering the horizontal component of the axial action in concentric struts.

#### Analysis of out-of-plane action effects on infills

1. The out-of-plane seismic action effect should be calculated using 7.2.1.
2. The design lateral load per unit area *f*a,pmay be calculated using Formula (7.9).

(7.9)

where

|  |  |
| --- | --- |
| *F*ap | is the horizontal seismic force applied at the centre of the infill wall calculated as in 7.2.1(3); |
| *l*ap | is the length of the infill wall; |
| *h*ap | is the height of the infill wall. |

#### Verification at SD limit state of concrete columns adjacent to infills

1. In DC1, DC2 and DC3, the entire length of ground floor columns surrounding infills should be considered as a critical region and 10.6.3.2(5) and (7) for critical regions of columns should be applied as a minimum.
2. In DC2 and DC3, if an infill extends over the entire length of an adjacent column and there is no infill on the other side of that column (e.g. a corner column), the entire clear length *l*cl of the column should be considered as a critical region and (1) should be applied as a minimum.

NOTE The explicit calculation of the shear effect due to the infill required by (4) can be more demanding than (1) and (2) and then it overrules (1) and (2).

1. The length *lcs* of a column over which the diagonal strut force of the infill is applied may be taken from Formula (7.10).

(7.10)

where

|  |  |
| --- | --- |
| *w*s | is the width of the strut given by Formula (7.6); |
| *Α* | is the angle between the infill panel diagonal and the horizontal (beam). |

1. In DC2 and DC3, if an infill extends on the entire clear length *l*cl of the adjacent columns, the top and bottom of these columns should be designed to resist over a length *l*cs as defined in (3) a shear force *V*Ed,infill given by Formula (7.11).

(7.11)

where

|  |  |
| --- | --- |
| *V*ap,Rd | is the horizontal component of the strut force of the infill, assumed to be equal to the horizontal resistance in shear of the infill panel as given by Formula (7.10); |
| *V*i,d | is the shear force calculated according to 10.6.2 assuming that the overstrength flexural capacity of the column, *γ*rd *M*Rc,i develops at the two ends of the contact length *lcs*. |

1. The application of (4) should not be taken to preclude the application of 10.6.2.
2. In DC2 and DC3, if the height of an infill is smaller than the clear length *l*cl of the adjacent column, a) to c) should be applied:
3. as a minimum, the clear length *l*cl of the column should be considered a critical region and 10.6.3.2 for critical regions should be applied;
4. (4) and Formula (7.9) should be applied with the column shear force *V*i,d calculated according to 10.6.2(2), assuming that the overstrength flexural capacity of the column *γ*Rd *M*Rc,i  develops over a length of the column equal to the height *h*op of the opening in the infills;
5. If *h*op is smaller than 1,5*hc*, where *h*c is the depth of the column cross-section in the direction of the infills, the shear action effect should be resisted by diagonal reinforcement over the length *h*op extended of *hc* above and below the opening.

#### Verification at SD limit state of steel columns adjacent to infills

1. In DC2 and DC3, if an infill extends on the entire clear length *l*cl of the adjacent columns, the top and bottom of these columns should be designed to resist over a length *l*cs as defined in 7.4.2.6(3) a shear force *V*Ed,infill given by Formula (7.9) where *V*i,d shouldbe taken as the shear force calculated assuming that the overstrength flexural capacity of the column, *γ*rd*M*Rd,i calculated according to 10.6.2(2) develops at the two ends of the contact length *l*cs.
2. In DC2 and DC3, if the height of an infill is smaller than the clear length *l*cl of the adjacent column, a) and b) should be applied:
3. as a minimum, the clear length *l*cl of the column should be considered a critical region and 11.8.3 for dissipative members in compression or bending should be applied;
4. (1) and Formula (7.9) should be applied with a column shear force *V*i,d calculated assuming that the overstrength flexural capacity of the column *γ*rd*M*Rd,i develops over a length of the column equal to the height *h*op of the opening in the infills.

#### Verification at SD limit state of composite columns

1. 7.4.2.7(1) should be applied.
2. 7.4.2.7(2) a) and b) should be applied, however considering in a) that 12.8.3 for dissipative members in compression or bending should be applied to the clear length *l*cl of the column considered as a critical region.

#### Verification of interacting infills at SD limit state

1. Interacting infills may be designed to resist to the maximum in-plane and to the maximum out-of-plane action effects taken separately.
2. The in-plane drift demand should be smaller than the in-plane drift capacity as given in Table 7.1.
3. The resistance of infill panels should not be smaller than the corresponding action effects calculated with 7.4.2.3 or 7.4.2.4.
4. The in-plane design resistance in shear *V*ap,Rd of an infill panel without openings may be taken from Formula (7.12).

(7.12)

where

|  |  |
| --- | --- |
| *F*ap | is the horizontal seismic force applied at the centre of the infill wall calculated as in 7.2.1(3); |
| *l*ap | is the length of the infill wall; |
| *h*ap | is the height of the infill wall. |

1. The in-plane design shear resistance of a masonry infill panel with an opening may be calculated as the product of the design shear resistance *V*ap,Rd of an infill panel without opening from Formula (7.12) and factor *ρ*op from Formula (7.13).

(7.13)

where

|  |  |
| --- | --- |
| *α*a = *l*op*h*op / *l*ap *h*ap | is the ratio of the opening's area to that of the infill panel; |
| *α*l = *l*op / *l*ap | is the ratio of the length of the opening to the infill panel length; |
| *h*ap | is the infill panel height; |
| *l*op | is the length of the opening; |
| *h*op | is the height of the opening; |
| *a, b, c* and *d* | are coefficients which should be taken from Table 7.2 for unreinforced, partially reinforced and reinforced openings (see Figure 7.1). |

Table 7.2 — Coefficients *a, b, c* and *d* for the calculation of the shear resistance of panels with openings

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Type of opening | *a* | *b* | *c* | *d* |
| Non reinforced | 0,55 | - 0,035 | 0,44 | - 0,025 |
| Partially reinforced | 0,58 | - 0,030 | 0,42 | - 0,020 |
| Reinforced | 0,63 | - 0,020 | 0,40 | -0,010 |

Figure 7.1 — Types of reinforcement of infills around openings

1. The design lateral force (out-of-plane) per unit area *f*ap calculated according to 7.4.2.5(1) and (2) should not exceed the design lateral (out-of-plane) resistance.
2. The design lateral (out-of-plane) resistance may be taken as 0,5 times the flexural or the arching effect resistance as in EN 1996-1-1:2022, 8.4.2 or 8.4.3.

NOTE The factor 0,5 takes into account the reduction in resistance due to the interaction between in-plane and out-of-plane action effects. More sophisticated methods relating the reduction to the drift demand are available.

1. Connections of infill wall to the bounding frame distributed regularly along the infill perimeter should be designed to resist the horizontal seismic force *F*ap as given in 7.2.1(3).

NOTE Some measures can reduce the risk of brittle failure of infills and of out-of-plane collapse of slender panels, in particular if their slenderness ratio greater than 15. It can be light wire meshes anchored on the faces of the wall; cement-based mortar/plasters reinforced by multi-directional textiles applied to the masonry panel; wall ties fixed to the columns and cast into the bedding planes; reinforced masonry; trimmed edges of the openings with belts and posts.

1. Values of the design lateral resistance of the infill corresponding to special reinforcement techniques and greater than those in EN 1996-1-1 may be used if they have been established by tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

### Design of frames with non-interacting infills

#### Basis of design

1. The design of non-interacting infills should be such that their contribution to the lateral stiffness and resistance of the structure against seismic actions may be neglected.
2. The analysis of the structure should be made considering a bare frame structure. Infills should be considered only for their mass.
3. The out-of-plane and in-plane seismic action effect on infills should be taken into account.

#### Detailing and in-plane verifications at SD limit state

1. The connection between the bounding frame and infill panel should be such that the effects of the interaction between infill and frame up to the interstorey drift demand at SD limit state of the bare frame may be neglected.

NOTE The connection is generally a material filling the separation joint between frame and infill.

1. The connection should be such that the contribution of the infills is in accordance with 7.4.3.1(1).
2. (2) may be considered satisfied if the closure of the gap is realized under a force which is not greater than 20 % of the shear resistance of the adjacent columns.
3. (1) may be considered satisfied if the contribution of the infill panel to the lateral resistance of the structure is not greater than 20 % of the lateral resistance of the bare frame.

NOTE Adequately soft materials are available to satisfy (2) or (3).

#### Detailing and out-of-plane verifications at SD limit state

1. The design out-of-plane force per unit area *f*ap should conform to 7.4.2.9(6).
2. 7.4.2.9(9) may be applied.
3. The connections of infill wall to the bounding frame should be distributed regularly along the infill perimeter, or on parts of it, either laterally or on top.
4. The connections should be designed to resist the resultant *F*ap of lateral action effect as given in 7.2.1(3).
5. The connections should not induce local failure of the infill walls due to stress concentrations in the bricks or blocks.
6. The connections should accommodate the design interstorey drift at SD.

## Structures with claddings

### Basis of design

1. Claddings connection systems may be either isostatic or integrated.
2. The seismic analysis of structures with cladding panels and integrated connections should consider their influence on the global behaviour of the structure.
3. Out-of-plane collapse of claddings should be prevented.
4. Hammering between adjacent panels and between panels and the structure should be prevented.

### Analysis

#### Analysis applicable to structures with any cladding systems

1. prEN 1998-1-1:2022, Clause 6, should be applied.
2. 10.14.4 should be applied to precast concrete cladding panels.
3. The out-of-plane seismic action effect should be calculated according to 7.2.1.
4. The design lateral load per unit area *f*apmay be calculated using Formula (7.9).

#### Analysis of structures with isostatic arrangements of wall panel connections, applicable to DC1, DC2 and DC3

1. The analysis may use a model of the primary structural system alone.
2. If the wall panels are introduced as structural members in the model, they may be modelled as prismatic elements with their weight distributed along their axis.
3. If the wall panels are introduced only as masses in the model, their total massshould be added to that of the members restraining them.
4. Cladding panels may be considered to rock if 7.5.3.1(3) and 7.5.3.2 are satisfied.

#### Analysis of structures with integrated panels and non-dissipative panel connections, applicable to structures designed in DC1 and DC2

1. The model for the analysis should include the cladding panels and their connections to the frame and to other panels.
2. The stiffness of the connections may be taken into account in the model.
3. Walls panels may be modelled by means of plate elements or beam-column elements.

#### Analysis of structures with integrated panels and dissipative panel connections, applicable to structures designed in DC3

1. The analysis of structures with integrated panels and dissipative panel connections should be non-linear.
2. The panel connections should be represented by their constitutive laws.

### Cladding panels

#### Verifications applicable to all types of cladding panels

1. The out-of-plane resistance of claddings should be greater than the calculated action effects in bending and shear due to the design out of plane force per unit area *f*ap calculated according to according to 7.4.2.5(2).
2. The resistance of claddings to in-plane forces should be greater than the calculated action effects.
3. The resistance and deformation capacity of the connections should not be smaller than the calculated action effects.
4. Friction due to compression forces other than by prestressing should be neglected in the evaluation of the resistance of connections to in-plane and out-of-plane forces.

NOTE Additional rules specific to steel and timber cladding panels in lightweight steel structures are given in 11.14.5 and 11.14.6. Additional rules specific to framed wall timber structures with wood-based cladding panels or with other types of sheathing material panels are given in 13.8.1.

#### Verifications of cladding panels with isostatic connection systems

NOTE In isostatic systems, action effects in the cladding and in the connections are determined by gravity and inertia forces; they do not depend on relative displacements between the supporting structure and the cladding. Thermal expansion is unrestrained. In precast concrete structures, isostatic arrangements are often used for both horizontal and vertical panels.

1. Isostatic connection systems may consist of pinned, seated and sliding connection devices or by a combination of those.

NOTE Different concepts correspond to an isostatic connection of a cladding to a structure. If the cladding is connected by a single hinge, it is a pendulum; connected by two hinges, it is a cantilever cladding; connected by two special hinges which allow, besides rotation, a free move upwards but not downwards, it is a rocking cladding; connected by one hinge and one sliding support, it is a beam cladding, either sitting or hanging.

1. Sliding devices should allow free alternate displacements in both directions with negligible reactions.
2. The design of sliding connections should be based on the displacement between the main structure and the claddings and comply with 7.5.3.1(3).
3. Installation of sliding devices should keep positioning deviations within the admissible linear and angular tolerances of the joint.
4. Compatibility of displacements at the corners of a building where orthogonal panels are jointed, with or without interposed angle elements, should be verified.

#### Verifications of interacting cladding panels

NOTE 1 Integrated arrangement are recommended mainly for vertical panels. In case of horizontal panels, integrated connections applied to the columns can induce significant shear forces in the columns, which can reduce their ductility.

NOTE 2 There are different integrated fastening schemes. Vertical panels connected to beams by two bottom and two top connections, each restraining horizontal and vertical displacement, realize a doubly fixed panel. A hinged-fixed panel can be realized by means of two bottom connections restraining horizontal and vertical displacement and two top connections restraining horizontal displacement and allowing vertical free displacements.

1. Connections of integrated panels may be dissipative. Their effectiveness should be confirmed by tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

## Partitions

### Basis of design

1. Out of plane collapse of partitions shall be prevented at SD limit state.
2. The connections of partitions to the structure should resist seismic action effects.

### Verification of partitions

1. The out-of-plane resistance of partitions should be greater than the calculated action effects in bending and shear due to the design out of plane force per unit area *f*ap calculated as given in 7.4.2.5.
2. The connections of a partition should be designed to resist the resultant *F*ap of lateral action effect as given in 7.2.1(3).
3. The connections of a partition to the bounding frame should be distributed regularly along the partition perimeter, or on parts of it, either laterally on two opposite sides or on top and bottom.
4. For masonry partitions, 14.6.3 should be applied.

# Base isolated buildings

## Field of application

1. This clause should be used for the design of seismically fully isolated buildings in which the isolation system, located below the main mass of the structure, aims at reducing the seismic response of the lateral force resisting system.

NOTE Clause 8 does not cover passive energy dissipation systems that are not arranged on a single interface but are distributed over several storeys or levels of the structure. These are covered in Clause 9.

## Basis of design

### Compliance criteria

1. The design of base isolated buildings should comply with prEN 1998-1-1:2022, 4.4, with additional rules given in 8.2.
2. The rules given in prEN 1998-1-1:2022, 6.8, should be satisfied.

### Control of undesirable movements

1. To minimize torsional effects, the effective stiffness centre and the centre of damping of the isolation system should be as close as possible to the projection of the centre of the superstructure mass on the isolation interface.
2. To minimize different behaviour of isolators, the compressive stress induced in them by the permanent actions should be as uniform as possible.
3. Devices should be fixed to the superstructure and the substructure by means of mechanical anchors. Anchorages should be in accordance with prEN 1998-1-1:2022, Annex G, and reliably ductile over any brittle failure modes.
4. The isolation system should be designed so that shocks between the substructure and the isolated structure and potential torsional movements of the isolated structure are controlled by appropriate measures.
5. (4) may be considered to be satisfied if potential shock effects are avoided through suitable devices (e.g. dampers, shock-absorbers, etc.).
6. Rotation of isolator ends around horizontal axis should not be greater than 0,5.

### Control of differential seismic ground motions

1. The structural members and the diaphragms located above and below the isolation interface should be sufficiently rigid both horizontally and vertically, so that the effects of differential seismic grounddisplacements are minimized.
2. In buildings, (1) may be considered as satisfied if conditions a) and b) are satisfied:
3. A rigid diaphragm is provided above and under the isolation system, consisting of a reinforced concrete slab or a grid of tie-beams, designed taking into account all relevant local and global modes of buckling. This rigid diaphragm is not necessary if the structure consists of rigid boxed parts.
4. The devices constituting the isolation system are fixed at both ends to the rigid diaphragms defined in a), either directly or, if not practicable, by means of vertical members, the relative horizontal displacement of which in the seismic design situation should be less than 5 % of the relative displacement of the isolation system.

### Control of displacements relative to surrounding ground and constructions

1. Sufficient space should be provided between the isolated superstructure and the surrounding ground or constructions, to allow its displacement in all directions in the seismic design situation, taking into account the amplification factor **x on displacement.

## Structural analysis

### General

1. Analysis methods in prEN 1998-1-1:2022, 6.8.5, may be applied as appropriate.
2. The vertical component of the seismic action, as defined in prEN 1998-1-1:2022, 5.2.2.3, should be taken into account for base isolated buildings.
3. Torsional effects about the vertical axis should be taken into account.
4. In buildings of consequence classes CC1 or CC2, mean values of physical and mechanical properties of isolators may be used, provided that extreme (maximum or minimum) values do not differ by more than 15 % from the mean values.

### Fundamental-mode equivalent linear response-spectrum analysis

1. If conditions a) to e) in prEN 1998-1-1:2022, 6.8.5.3(2), and a) to g) are satisfied, fundamental-mode equivalent linear response-spectrum analysis may be used, in which the horizontal displacements and the torsional movement about the vertical axis are considered, but the substructures and the superstructures are assumed to behave rigidly.
2. the conditions for vicinity to well identified seismically active faults according to prEN 1998‑11:2022, 5.1.1(5), are not met;
3. the largest dimension of the superstructure in plan is not greater than 50 m;
4. all devices are located above members of the substructure which support vertical loads;
5. the site is of category A or B;

NOTE prEN 1998-1-1:2022, 5.1.2, gives the categorization of sites. When the site is softer than A or B, there can be vertical displacement and rocking that require more refined modelling including soil-structure interaction (see prEN 1998-5:2022, 8.1).

1. the effective period *T*eff satisfies Formula (8.1):

(8.1)

where

|  |  |
| --- | --- |
| *T*eff | is the horizontal seismic force applied at the centre of the infill wall calculated as in 7.2.1(3); |
| *T*f | is the length of the infill wall; |

1. the ratio between the vertical and the horizontal stiffness of the isolation system satisfies Formula (8.2):

(8.2)

where

is defined in prEN 1998-1-1:2022, 6.8.5.2(3).

1. the fundamental period in the vertical direction, *T*V, given by Formula (8.3), is not longer than 0,1 s.

(8.3)

1. Displacements at every point of the structure should be calculated combining the translational and torsional rotation displacements. This should apply for the evaluation of the effective stiffness of each isolator.
2. The inertial forces and moments should be taken into account for the verification of the isolators and their connections, and of the substructures and the superstructures.

## Verification of Significant Damage Limit State

1. The substructure shall be verified under the combination of actions in the seismic situation directly applied to it and the forces and moments transmitted to it by the isolation system.
2. Safety verifications regarding equilibrium and resistance in the substructure and in the superstructure should be performed in accordance with Clause 6.
3. The structural members of the substructure and the superstructure should be designed as non-dissipative. Ductility class 1 (DC 1) may be adopted for them. Capacity design and global or local ductility conditions may be neglected.
4. The resistance condition of the structural members of the substructure and the superstructure may be satisfied taking into account seismic action effects calculated with *q* = 1,5.
5. Depending on the type of device, the resistance of the isolators and their connections to the substructure and superstructure should be verified at the SD Limit State in terms of either a) or b):
6. in terms of forces, taking into account the maximum and minimum vertical forces due to non-seismic actions, as well as maximum possible vertical and horizontal forces due to the seismic action, including overturning effects;
7. in terms of total relative horizontal displacement between lower and upper faces of the isolator. The total horizontal displacement should include the deformation due to the design seismic action and the effects of shrinkage, creep, temperature and post tensioning (if the superstructure is prestressed).

NOTE The reliability factor **x defined in prEN 1998-1-1:2022, 6.8.2.2, applies to forces and displacements.

1. In buildings, ancillary elements should be analysed in accordance with 7.2, with due consideration to the dynamic effects of the isolation.

# Buildings with energy dissipation systems

## General

1. This clause should be applied to the design of building structures with passive energy dissipation devices classified as displacement dependent or velocity dependent.

NOTE 1 Two types of displacement-dependent energy dissipation devices are covered: (i) devices that exhibit rigid plastic behaviour; and (ii) devices that exhibit multilinear hysteresis.

NOTE 2 Two types of velocity-dependent energy dissipation devices are covered: (i) viscoelastic (solid and fluid) devices; and (ii) viscous (fluid) devices.

1. The response of displacement-dependent energy dissipation devices should be independent of velocity and frequency of excitation.

NOTE 1 The force-displacement relationship of a displacement-dependent energy dissipation device with rigid plastic behaviour changes its shape only if loaded above its yield force, and then deforms at approximately constant force.

NOTE 2 A displacement-dependent energy dissipation device with multilinear hysteresis deforms according to a linear elastic force-displacement relation characterized by an elastic stiffness *k*ebelow the yield force, and then deforms following plastic branches with stiffness smaller than *k*e*.*

## Basis of design

### Compliance criteria

1. If not otherwise specified, the seismic performance of the structure with an energy dissipation system should be verified according to prEN 1998-1-1:2022, 4.4.
2. prEN 1998-1-1:2022, 6.8, should be satisfied.

### Main structural system

1. The primary seismic members of the main structural system should conform to one of the types covered in 10 to 15.
2. The primary seismic members of the main structural system should provide a complete lateral load path; the main structural system should maintain support of the gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition.

NOTE The main structural system includes both primary and secondary structural members.

### Energy dissipation system

NOTE This standard does not cover the systems where conditions given in 9.2.3 are not satisfied.

1. The yield displacement of displacement-dependent energy dissipation devices installed in a storey, in the direction under consideration, should not be greater than 0,4 times the yield displacement of the primary seismic members of the main structural system at the same storey.
2. The yield resistance to shear of displacement-dependent energy dissipation devices installed in a storey, in the direction under consideration, should not be greater than the maximum storey resistance to shear of the primary seismic members of the main structural system at the same storey.
3. All structural members of the energy dissipation system, exclusive of the members that are common with the main structural system and exclusive of the energy dissipation devices, should remain in the elastic domain for the action effects of the design seismic action magnified by 1,5.

### Control of torsional effects

1. The in-plan distribution of energy dissipation devices should provide torsional resistance and stiffness.

NOTE Layouts of energy dissipation devices close to the periphery of the building are more effective.

## Structural analysis

### General

1. The model should conform to 5.1 for the structural system, to prEN 1998-1-1:2022, 6.8.2, and to 9.3 of this document.
2. The method described in 9.3.2 may be applied when 9.3.2.1(1) is satisfied.
3. The method described in 9.3.3 may be applied when 9.3.3.1(1) is satisfied.
4. The method described in 9.3.4 may always be applied.

### Non-linear response spectrum analysis

#### General

1. The non-linear response spectrum analysis defined in prEN 1998-1-1:2022, 6.8.5.4, may be used for analysis and design of building structures with velocity-dependent energy dissipation devices.
2. The non-linear response spectrum analysis may be used for analysis and design provided that all the conditions in a) to c) are satisfied:
3. The floor diaphragms of the buildings are rigid in their planes in accordance with 5.1.2(2).
4. In each direction, the energy-dissipation system has at least two energy dissipation devices in each storey, arranged in accordance with 9.2.4.
5. All storeys of the building above ground level have energy dissipation devices of the same type (viscous or viscoelastic), all of them should have similar velocity exponent *a* and similar loss factor **loss and, in case of viscoelastic type energy dissipation devices, *a*= 1.
6. If one of a) to c) in (2) is not fulfilled, a response-history analysis should be used.
7. At SD limit state, the amount of energy that can be dissipated by the primary seismic members of the main structural system on each storey *k* before it reaches the SD limit state should be greater than the maximum energy dissipation demand on the main structural system at the *k*-storey under the design seismic action.
8. (4) may be considered as satisfied if conditions a) to c) are satisfied:
9. in DC3, the primary seismic members of the main structural system meet the global ductility conditions given in 6.2.7;
10. the primary seismic members of the main structural system meet the material dependent provisions to be categorized as ductility class DC1, DC2 or DC3;
11. the displacement ductility ratio **f*,*in the direction under consideration where **f is given by Formula (9.1) and calculated in accordance with Annex D, D.2, does not exceed the highest *q*D behaviour factor in the same direction that can be assigned to the primary seismic members of the main structural system to meet the provisions of material dependent Clauses.

(9.1)

where

|  |  |
| --- | --- |
| *d*roof 1 | is the maximum horizontal displacement relative to the ground at roof level for the first mode in the direction under consideration; |
|  | is a horizontal displacement at yield which takes different values (given in Annex D, D.2) according to the type of energy dissipation system. |

1. The amount of energy that can be dissipated by the energy dissipation devices installed on each storey *k* before one of them reaches the SD limit state should be greater than the maximum energy dissipation demand on the energy dissipation system at the *k*-th storey under the design seismic action.
2. (6) may be considered as satisfied if the energy dissipation capacity of the energy dissipation devices installed on each storey *k* in the direction under consideration when any energy dissipation device on the same storey attains its SD limit state, evaluated in accordance with EN 15129, is greater than the maximum energy dissipation demand given by Formula (9.2).

(9.2)

where

|  |  |
| --- | --- |
| *T*s1 | is a period that gives the maximum value of [*T*s1S(*T*s1,5%)**(*T*s1*,*I)] between *T*1,e and 1,4 *T*1,e; |
| *T*1,e | is the fundamental period of the structure with energy dissipation system in elastic conditions, in the direction under consideration; |
| *m* | is the total mass of the building; |
| *S*e(*T*s1, 5*%*) | is the spectral acceleration value of the 5 % damped elastic response spectrum given in prEN 1998-1-1:2022, 5.2.2.2(1), corresponding to a period equal to 1,4 times the fundamental period *T*1,e; |
| **(*T*s1,**I) | is the damping correction factor given by prEN 1998-1-1:2022, 5.2.2.2(12), corresponding to a period equal to 1,4 times the fundamental period *T*1,e and to the inherent damping ratio **Idefined in Annex D, D.3.2. |

1. Every velocity-dependent device with non-negligible stiffness (e.g. viscoelastic energy dissipation devices) should be modelled with an effective stiffness corresponding to the amplitude and predominant period of its response.
2. The seismic action effects in the primary seismic members of the main structural system should be calculated for both the upper-bound and lower-bound properties of the energy dissipation devices in three distinct stages: stage of maximum displacement, stage of maximum velocity, stage of maximum acceleration.

NOTE The design value of a seismic action effect *E*E in primary members of the main structural system is the maximum among *E*E,max.disp*,E*E,max.veland *E*E,max.accel obtained after combination of the corresponding effects of the seismic components.

#### Response in the first mode

NOTE First mode or fundamental mode refers to the mode of vibration that corresponds to the largest value of the effective modal mass for the direction under consideration

##### Elastic response

1. The maximum displacement relative to the ground at roof level in the direction under consideration corresponding to the elastic behaviour of the structure with energy dissipation system should be calculated using Formula (9.3).

(9.3)

where

|  |  |
| --- | --- |
| *T*1,e | as in 9.3.2.1(7); |
| *T*p1 | is the fundamental period of the primary seismic members of the main structural system in elastic conditions, exclusive of energy dissipation system; |
| **1 | is the first mode participation factor of the structure with energy dissipation system in the elastic range; |
| **I | is the component of effective damping of the structure due to inherent dissipation of energy by the primary seismic members of the main structural system, exclusive of the energy dissipation system, at or just below its yield displacement, **I should be calculated according to Annex D, D.3.2; |
| **V1 | is the component of effective damping of the first mode of vibration of the structure due to viscous dissipation of energy by the damping system, at or just below the yield displacement of the primary seismic members of the main structural system, **V1 should be calculated according to Annex D, D.3.2; |
| *S*De (*T*1,e, *5%*) | is the spectral displacement value of the 5 % damped elastic displacement response spectrum given in prEN 1998-1-1:2022, 5.2.2.2(13), corresponding to period *T*1,e; |
| **(*T*1,e,**I+**V1) | is the value of the damping correction factor given in prEN 1998-1-1:2022, 5.2.2.2(12), corresponding to period *T*1,eand damping ratio (**I+**V1). |

##### Inelastic response

1. The maximum displacement relative to the ground at roof level in the direction under consideration for inelastic behaviour of the structure with energy dissipation system, including the elastic part, should be calculated using Formula (9.4).

(9.4)

where

|  |  |
| --- | --- |
| *S*De(*T*eff 1,5*%*) | is the spectral displacement value of the 5 % damped elastic displacement response spectrum given in prEN 1998-1-1:2022, 5.2.2.2(13), at period *T*eff 1; Teff1 should be calculated according to Annex D, D.3.1; |
| *h* (*T*eff 1*,*eff 1) | is the value of the damping correction factor given in 5.2.2.2(12), corresponding to the period *T*eff 1and damping ratio **eff 1; eff1 should be calculated according to Annex D, D.3.2. |

1. The base shear of the structure with energy dissipation system at roof displacement *d*roof1 in the direction under consideration may be obtained by either a) or b):
2. By summing i) the base shear *V*py1 associated with the formation of a plastic mechanism in the primary seismic members of the main structural system, obtained with a pattern of loads proportional to first mode, and ii) the base shear exerted by the viscoelastic dampers corresponding to the roof displacement *d*roof 1 given in (1).

NOTE *V*py 1 obtained by a plastic analysis and the base shear force sustained by the elastic component of the viscoelastic dampers for the roof displacement *d*roof 1are both calculated assuming a displacement according to the first mode. The meaning of *V*py1 and other variables is shown in Figure 9.1.

Figure 9.1 — Roof displacement *d*roof vs. base shear force *V*b of the structure with energy dissipation system; (a) is the overall curve of the structure with energy dissipation system; (b) is the contribution of the main structure; (c) is the contribution of the elastic part of the viscoelastic dampers

1. Conducting a pushover analysis with a model that includes the primary seismic members of the main structural system and the storage stiffness *K*k,*j* of the viscoelastic-type energy dissipation devices evaluated at the fundamental period *T*1.

NOTE *K*k,j = *G*’ *s*f where *G*’ is the storage shear moduli of the damper and *s*f is a shape factor, both provided by the producer of the damper according to EN 15129.

1. If, for the roof displacement *d*roof 1, the base shear of the structure with energy dissipation system in the direction under consideration, is less than 0,95*V*b1max or greater than 1,05*V*b1max, where *V*b1max is given by Formula (9.5), an iterative procedure should be applied by varying *d*roof 1.

(9.5)

where

|  |  |
| --- | --- |
| *m*eff 1 | is the first mode effective mass of the structure with energy dissipation system in the elastic range, in the direction under consideration; |
| *S*e(*T*eff 1, 5*%*) | is the spectral acceleration value of the 5 % damped elastic acceleration response spectrum given in prEN 1998-1-1:2022, 5.2.2.2(1), corresponding to period *T*eff 1. |

NOTE To obtain the response in the first mode, the structure is idealized with an equivalent SDOF system of period *T*eff 1and damping ratio **eff 1. *T*eff 1is associated with a secant stiffness *k*effdetermined from the capacity curve of the structure determined as described in prEN 1998-1-1:2022, 6.5.2, with the control node located at roof level. *d*roof 1 is the (target) displacement of the capacity curve associated with *k*eff. The iteration is needed due to the fact that *T*eff 1 and **eff 1 depend on the ductility ratio **f.

##### Maximum response displacement

1. The maximum horizontal displacements at *i*-th floor relative to the ground, in the direction under consideration, should be calculated with Formula (9.6).

(9.6)

where

|  |  |
| --- | --- |
| *i*1 | is the component of the first mode vector **1 at the *i*-th floor of the structure with energy dissipation system in the elastic range, in the direction under consideration; **1 should be normalized so that at roof level the value is unity; |
| *d*roof1e | is defined in 9.3.2.2.1(1). |

1. The design lateral force at *i*-th floor in the direction under consideration should be calculated using Formula (9.7).

(9.7)

where

|  |  |
| --- | --- |
| *m*i | is the mass of level *i*; |
| *q*R and *q*S | are behaviour factor components of the main structural system. |

NOTE The forces given by Formula (9.7) are used to calculate the contribution of the first mode to the design value of the action effects in the primary seismic members of the main structural system at the stage of maximum displacement (see 9.3.2.4 (1)).

1. The axial elastic force component *N*ej1 in a viscoelastic-type energy dissipation device *j* installed in the *k*-th storey should be calculated using Formula (9.8).

(9.8)

where

|  |  |
| --- | --- |
| *d*j1 | is the relative displacement of the two ends of the energy dissipation device measured along its axis; |
| *K*k,j | is the axial elastic storage stiffness component of the viscoelastic energy dissipation device *j* installed at the *k*-th storey, corresponding to the first mode of vibration. |

NOTE The elastic forces in the viscoelastic dampers given by Formula (9.8) are used in conjunction with Formula (9.11) to determine the maximum forces in the dampers. These maximum forces are used to verify the damper.

##### Maximum response velocity

1. The maximum horizontal velocity at a given floor relative to the ground should be calculated by multiplying the maximum horizontal displacement relative to the ground at the same floor *di*1 given by Formula (9.6) by (2/*T*eff 1) and by the correction factor for velocity *CFV* given by Formula (9.9).

(9.9)

1. The axial viscous damping force component in energy dissipation device *j, N*v*j*1, should be determined using Formula (9.10).

(9.10)

where

|  |  |
| --- | --- |
| *v*j1 | is the relative velocity between each end of the energy dissipation device *j* measured along its axis; it is obtained multiplying *d*j1 given in 9.3.2.2.3(3) by (2/*T*eff 1) and by the correction factor *CFV* given by Formula (9.9); |
| **ve | is the velocity exponent of the velocity-dependent energy dissipation devices, provided by the producer of the device; |
| *C*j | is the damping coefficient of the velocity-dependent energy dissipation device *j*, provided by the producer of the device. |

1. The maximum axial force *N*j1 in energy dissipation device *j* should be calculated using Formula (9.11).

(9.11)

where

|  |  |
| --- | --- |
| **loss | is the loss factor of viscoelastic energy dissipation devices; |
| *N*ej1 and *N*vj1 | are given by Formulas (9.8) and (9.10) respectively. |

NOTE 1 In viscous-type energy dissipation devices *N*ej1 = 0 and sin(tan-1**loss) = 1.

NOTE 2 **loss is provided by the producer of the damping device.

##### Maximum response acceleration

1. The design value of a seismic action effect at stage of maximum acceleration *E*Ei,max,.acc in the direction under consideration should be obtained weighting the responses at stages of maximum displacement *E*Ei,max,disp and of maximum velocity *E*Ei,max,velaccording to Formula (9.12).

(9.12)

where

|  |  |
| --- | --- |
| *CF*1 | is given by Formulas (9.13) to (9.16): |

if **f 1 and **ve 0,25: *CF*1*=*1 (9.13)

if **f 1 and 0,25 < **ve: *CF*1=cos**lag0,5 (9.14)

if1 *< *f< 1/cos**lag: *CF*1= **fcos**lag (9.15)

if **f 1/cos**lag: *CF*1= 1 (9.16)

|  |  |
| --- | --- |
| *CF*2 | is given by Formulas (9.17) to (9.19): |

if **ve 0,25: *CF*2= 1 (9.17)

if 0,25 < **ve1: *CF*2= (sin**lag)**ve (9.18)

if **ve> 1: *CF*2= sin**lag (9.19)

|  |  |
| --- | --- |
| *m*f | is the displacement ductility factor defined in 9.3.2.1(4); |
| **ve | is defined in 9.3.2.4(2); |
| **lag | is a phase lag angle that should be calculated with Formula (9.20): |

(9.20)

where **H is the component of effective damping of the structure in the direction under consideration, due to post-yield hysteretic behaviour of the primary seismic members of the main structural system associated with the ductility factor **f, calculated in accordance with Annex D, D.3.2.

#### Response in higher modes

1. The response of the structure in higher modes of vibration (*m* > 1) in the direction under consideration should be obtained assuming that: (i) the primary seismic members of the main structural system remain elastic; and (ii) the nonlinear viscous-type energy dissipation devices behave as linear with an effective damping coefficient *C*eff*j* given by Formula (9.21).

(9.21)

where is given in 9.3.2.2.3(2).

1. The response at the stages of maximum displacement, maximum velocity and maximum acceleration in the higher mode *m* in the direction under consideration should be calculated as in the first mode using Formulae (9.3), (9.4), (9.6) to (9.20) and Formulas of Annex D, replacing the subscript 1that indicates the first mode with subscript *m* for the *m*-th mode, and modifying the Formulae as given in a) to i):
2. **f should be taken equal to 1;
3. **ve should be taken equal to 1 for all energy-dissipation devices for the response in the first mode;
4. *Cj*  should be taken equal to *C*eff*j* given by Formula (9.21);
5. *T*1,eshould be replaced by the *m*-th mode period of the structure with energy-dissipation system in elastic conditions *Tm*;
6. *T*p1should be replaced by the *m-th* mode period of the main structural system in elastic conditions *T*p*m*;
7. **i1should be replaced with the component **im of the *m*-th mode **m at the *i*-th floor of the structure with energy dissipation system in the elastic range, in the direction under consideration; **m should be normalized so that it equals 1 at the roof level;
8. **1 should be replaced by the *m*-th mode participation factor *m* of the structure with energy dissipation system in the elastic range, in the direction under consideration;
9. *q*R*q*Sshould be taken equal to 1;
10. For the determination of *CF*1, the ductility demand should be taken equal to that of the fundamental mode **f;
11. 9.3.2.2.1(4) should not be applied for higher modes.

#### Combination of modal responses

1. At the stage of maximum displacement, the design value of the action effects in the primary seismic members of the main structural system should be calculated by combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects obtained for each mode by applying to the model of the structure with energy dissipation system the design lateral forces *F*i1 given by Formula (9.7) for the first mode, and the design lateral forces calculated in accordance with 9.3.2.3 for higher modes. Torsional effects should be included.

NOTE At the stage of maximum displacement, the viscous damping force components of the energy dissipation devices are zero, but the elastic force components of the viscoelastic dampers are not and the model accounts for them.

1. At the stage of maximum velocity, the design value of the action effects in the primary seismic members of the main structural system and in the elements of the energy dissipation system exclusive of the energy dissipation devices should be calculated, combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects obtained for each mode by applying to the model the axial viscous damping force components of the energy dissipation devices *N*v*j*1 given by Formula (9.10) for the first mode and the axial viscous damping force components of the energy dissipation devices calculated in accordance with 9.3.2.3 for the higher modes. These forces should be applied at the points of attachment of the devices and in the directions consistent with the deformed shape of the building at maximum displacement. The inertia forces at each floor level should be applied concurrently with the viscous component of forces so that the displacement of each floor is zero.
2. At the stage of maximum acceleration, the design value of the action effects in the primary seismic members of the main structural system and in the elements of the energy dissipation system exclusive of the energy dissipation devices should be calculated combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects for each mode as the sum of the action effects obtained at the stage of maximum displacement times *CF*1 and of the action effects obtained at the stage of maximum velocity times *CF*2. Factors *CF*1 and *CF*2should be calculated in accordance with 9.3.2.2.4(1).
3. The design force in a velocity-dependent energy dissipation device should be based on the maximum force in the device obtained combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the maximum axial force of the energy dissipation device *N*j1 obtained with Formula (9.11) for the first mode and the maximum axial force of the energy dissipation device calculated in accordance with 9.3.2.3 for the higher modes. Torsional effects should be included.
4. At the stage of maximum displacement, the design value of the action effect in the elements of the energy dissipation system, exclusive of the elements that are common with the main structural system and exclusive of the energy dissipation devices, should be calculated combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects obtained for each mode by applying to the model of the structure with energy dissipation system the design lateral forces *F*i1 given by Formula (9.7) times *q*R *q*S for the first mode, and the design lateral forces *F*im calculated in accordance with 9.3.2.3 for the higher modes. Torsional effects should be included. prEN 1998-1-1:2022, 6.4.3.1(2), (3), and (4), should be applied.

#### Torsional effects

1. Torsional effects should be considered in accordance with 5.2(3) by setting the horizontal force acting on the considered storey in a given direction equal to the design lateral forces given by Formula (9.7) for the first mode, or to the design lateral forces calculated in accordance with 9.3.2.3 for the higher modes.

### Energy-balance based analysis

#### General

1. The energy-balance based analysis may be used for analysis and design of building structures with displacement-dependent energy-dissipation devices.

NOTE Energy-balance based analysis applies energy concepts through the basic formula *E*I = *E*k + *E*s + *E* + *E*H, where *E*I is the energy input at the foundation, *E*k the kinetic energy, *E*s the fully recoverable elastic strain energy, *E* the viscous damping energy and *E*H the energy dissipated through hysteretic plastic deformation. *E*His the sum of the energy dissipated by the primary seismic members of the main system, *E*pH, and by the energy dissipation devices, *E*dH.

1. The energy-balance based analysis may be used for analysis and design provided that the conditions in a) to c) are satisfied.
2. The floor diaphragms of the buildings are rigid in their planes in accordance with 5.1.2(2);
3. In the direction under consideration, the energy-dissipation system has at least two energy dissipation devices in each storey, arranged in accordance with 9.2.4;
4. All storeys of the building above ground level have energy dissipation devices.
5. If at least one of the conditions a) to c) in (2) is not fulfilled, a response-history analysis should be used.
6. The stiffness of displacement-dependent energy dissipation devices should be modelled with an effective stiffness that represents the force in the energy dissipating device at the response displacement of interest (e.g. design interstorey drift). Alternatively, design forces in these devices may be applied as external loads, in which case the stiffness of the energy dissipation devices may be neglected in the model.
7. The amount of energy that the structure can absorb in the direction under consideration while the primary seismic members of the main structural system remain within the elastic domain should exceed one-fourth of the amount of input energy in the building due to the reference seismic action.

NOTE This provision is intended to avoid the damage in the primary seismic members of the main system under low intensity earthquakes characterized by an amount of energy (*E*I*- E*) equal to one fourth of the corresponding value in case of the design earthquake.

1. (5) may be considered satisfied if the condition given by Formula (9.22) is satisfied.

(9.22)

where

|  |  |
| --- | --- |
| *m* | is the total mass of the building; |
| *E*e | is the amount of energythat the structure with energy dissipation system can store in form of elastic strain energy and absorb in form of plastic strain energy dissipated by the dampers while the primary seismic members of the main structural system remain in the elastic range, in the direction under consideration, calculated according to Annex D, D.4.1; |
| *S*e(*T*1,e *,* 5%) | is the spectral acceleration value of the 5 % damped elastic response spectrum given in prEN 1998-1-1:2022, 5.2.2.2(1), corresponding to period *T*1; |
| **(*T*1,e ,**I) | is the value of the damping correction factor in prEN 1998-1-1:2022, 5.2.2.2(12), corresponding to period *T*1and inherent damping ratio **I in accordance with Annex D, D.3.2. |

NOTE In the basic energy-balance Formula, *E*e represents the sum *E*k*+ E*s*+E*H specialized for a low intensity earthquake for which the primary seismic members of the main system remain elastic (i.e. *E*H = *E*dH and *E*pH = 0).

1. The amount of energy that the primary seismic members of the main structural system at the *k*-th storey can dissipate in the direction under consideration before reaching the SD limit state should be greater than the maximum energy dissipation demand on the primary seismic members of the main structural system at the *k*-th storey under the design seismic action.
2. (7) may be considered satisfied at SD limit state if the condition given by Formula (9.23) is satisfied at each storey *k*.

(9.23)

where

|  |  |
| --- | --- |
| *p* | index standing for primary seismic members; |
| *E*pH,k,SD | is the amount of energy that can be dissipated by the primary seismic members of the main structural system at the *k*-th storey in the direction under consideration before reaching the SD limit state as specified in (9); |
| *E*pH,k,max | is the maximum energy dissipation demand at the *k-*th storey in the primary seismic members of the main structural system under the design earthquake, according to Annex D, D.4.3. |

NOTE Formula (9.23) can be checked and satisfied without need to assign a ductility class to the primary seismic members of the main structural system and without need to meet the global ductility provision of 6.2.5(9).

1. *E*pH,k,SDshould be taken as a value not exceeding the amount of energy that can be dissipated by the primary seismic members of the main structural system at the *k*-th storey in the direction under consideration under cyclic deformations, until any primary seismic member of the main structural system attains the SD limit state.
2. As an alternative to (8) and (9), (7) may be considered satisfied if conditions a) to c) are satisfied:
3. In DC3, the primary seismic members of the main structural system meet the global ductility conditions prescribed in 6.2.7;
4. the primary seismic members of the main structural system meet the conditions in Clauses 10 to 15 to be categorized as ductility class DC1, DC2 or DC3;
5. the displacement ductility ratio **f,, in the direction under consideration, calculated in accordance with 9.3.2.1(5), where droof1 is the value of *d*i1 given in 9.3.3.2(2) at roof level, does not exceed the highest *q*Dbehaviour factor that can be assigned to the main structural system to meet the requirements of 10 to 15.
6. The amount of energy that the energy dissipation system at the *k*-th storey can dissipate in the direction under consideration before it reaches the SD limit state should be greater than the maximum energy dissipation demand on the energy dissipation system at the *k*-th storey under the reference seismic action.
7. (11) may be considered satisfied if the condition given by Formula (9.24) is satisfied at SD limit state at each storey *k*.

(9.24)

where

|  |  |
| --- | --- |
| *d* | index standing for device; |
| *E*dH,*k,*SD | is the amount of energy that can be dissipated by the energy dissipation devices at the *k*-th storey in the direction under consideration under cyclic deformations when any energy dissipation device on the storey attains its SD limit state evaluated in accordance with EN 15129; |
| *E*dH,*k*,max | is the maximum energy dissipation demand on the energy dissipation system at the *k*-th storey under the seismic action, calculated in accordance with Annex D, D.4.4. |

1. The design value of seismic action effects in the primary seismic members of the main structural system should be calculated for both the upper- and lower-bound properties of the energy dissipation devices at the stage of maximum displacement.

#### Response in the first mode

NOTE First mode refers to the mode of vibration that corresponds to the largest value of the effective modal mass for the direction under consideration.

1. The maximum interstorey drift at the *k-*th storey *d*r,*k*1, in the direction under consideration, for inelastic behaviour of the main structural system including the elastic part should be determined by Formula (9.25).

(9.25)

where

|  |  |
| --- | --- |
| *E*H,*k* | is the required amount of energy dissipation in form of plastic strain energy on the *k*-th storey of the building, in the direction under consideration, as a result of the design seismic action, and should be calculated according to Annex D, D.4.2; |
| *V*dy,*k* | is the *k-*th storey yield shear of the energy dissipation system in the direction under consideration; |
| *V*py,*k* | is the *k*-th storey yield shear corresponding to the formation of a plastic mechanism in the primary seismic members of the main structural system in the direction under consideration; |
| *d*py,*k* | is the *k*-th storey drift corresponding to the formation of a plastic mechanism in the primary seismic members of the main structural system in the direction under consideration. |

NOTE The meaning of *V*dy,*k , V*py,*k , d*py,*k* and other variables is shown in Figure 9.2 that represents the typical relationship between the interstorey drift and the storey shear *Vk* at a given storey *k*. *V*pyk and *d*pyk can be obtained conducting a pushover analysis or with approximate formulas, in both cases for a pattern of lateral loads proportional to the first mode

Figure 9.2 — Interstorey drift vs. storey shear *V*k at a given storey *k*. *V*py,k is the storey shear of primary seismic members of main structural system; *V*dy,k is the storey shear provided by the energy-dissipation devices at the *k*-th storey

1. The maximum horizontal displacements at *i*-th floor relative to the ground, in the direction under consideration, for inelastic behaviour of the main structural system and including the elastic part, should be calculated with Formula (9.26), where the summation extends to all the storeys below the *i*-th floor.

(9.26)

1. The design seismic force acting on the structure with energy-dissipation system at the *i*-th floor level located above storey *k* and below storey *k*+1, in the direction of interest, should be calculated using Formula (9.27).

(9.27)

where

|  |  |
| --- | --- |
| *V*pD,1 | is the maximum base shear that can be resisted by the primary seismic members of the main structural system within the elastic domain in the direction under consideration; |
| *V*dy,1 | is the base storey yield shear of the energy dissipation system in the direction under consideration; |
|  | , is the mass above the *k*-th storey normalized by the total mass of the building. |

#### Response in higher modes

1. The response in higher modes (*m* > 1) should be obtained using modal analysis, assuming elastic response for the primary seismic members and the displacement-dependent energy dissipation devices.
2. The response at the stage of maximum displacement in the higher mode *m* should be calculated with Formulae (9.3), (9.4), (9.6) and (9.7) and Formulae of Annex D, D.4, replacing the subscript 1that indicate the first mode with subscript *m* and modifying formulas as given in a) to f):
3. **v1should be taken equal to zero;
4. *T*1,eand *T*eff,1 should be replaced by the *m*-th mode period of the structure with energy-dissipation system in elastic conditions *Tm*;
5. **i1 should be replaced with the component **im of the *m*-th mode*m* at the *i*-th floor of the structure with energy dissipation system in the elastic range, in the direction under consideration. *m* should be normalized so that the value at roof level is unity;
6. **eff1should be replaced by the inherent damping ratio **I calculated according to Annex D, D3.2;
7. **1 should be replaced with the *m*-th mode participation factor *m* of the structure with energy dissipation system in the elastic range, in the direction under consideration;
8. *q*R*q*Sshould be taken equal to 1.

NOTE With the modifications a) to f), Formulae (9.2) and (9.3) give always the same value, i.e. *d*roof1e= *d*roof1.

#### Combination of modal responses

1. The design value of action effects in the primary seismic members of the main structural system should be calculated by combining, in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects obtained for each mode by applying to the structural model with the energy dissipation system the design lateral forces *F*i1 given by Formula (9.27) for the first mode, and those in accordance with 9.3.3.3 for the higher ones.
2. The design displacement in a displacement-dependent energy dissipation device should be based on the maximum displacement obtained by combining, in accordance with prEN 1998-1-1:2022, 6.4.3.2, the relative displacement between the two ends of the device along its axis calculated for the first mode on the basis of Formula (9.25) and the relative displacement due to the higher modes in accordance with 9.3.3.3.
3. Design values of action effects in the energy dissipation system, except of elements common with the main structural system and of the energy dissipation devices, should be calculated combining in accordance with prEN 1998-1-1:2022, 6.4.3.2, the action effects obtained for each mode by applying to the structural model with the energy dissipation system the design seismic forces *F*i1 given by Formula (9.27) multiplied by {*q*R*q*S[1+(*V*dy,1/*V*py,1)]/[1+*q*R*q*S(*V*dy,1/*V*py,1)]} for the first mode, and those from 9.3.3.3 for the higher ones.
4. prEN 1998-1-1:2022, 6.4.3.1(2) to (4), should be applied.

NOTE The energy dissipation system includes: (i) elements that can be common with the main structural system; their calculation is made in accordance with 9.4.3.4(1) (see also 9.5(1)); (ii) the energy dissipation devices; their calculation is made in accordance with 9.3.3.4(2); (iii) elements required to transfer the forces from the energy dissipation devices to the main structural system and to the base of the structure; their calculation is made in accordance with 9.3.3.4(3). The magnification factor {*q*R*q*S[1+(*V*dy,1/*V*py,1)]/[1+*q*R*q*S(*V*dy,1/*V*py,1)]}applied in the calculation of the latter removes reductions of the design value of the action effects due to overstrength.

#### Torsional effects

1. Torsional effects should be considered in accordance with 5.2(3) setting the horizontal force acting on the considered storey in a given direction equal to the design lateral forces given by Formula (9.27) for the first mode, or to the design lateral forces calculated in accordance with 9.3.3.3 for the higher modes.

### Non-linear response history analysis

1. prEN 1998-1-1:2022, 6.6 and 6.8.5.5, should be applied.

### Combination of the effects of the components of seismic action

1. prEN 1998-1-1:2022, 6.4.4, should be applied.

## Verification to Limit States

### General

1. Structural members that are common to the main structural system and to the energy dissipation system should be considered as part of the main structural system in the verifications.

### Verification to Significant Damage (SD) limit state

1. Verifications regarding equilibrium and resistance of the primary seismic members of the main structural system as well as seismic joints should comply with 6.2.2 to 6.2.8.
2. The primary seismic members of the main structural system should be verified according to prEN 1998-1-1:2022, 6.7.2. In case of structures with velocity-dependent energy dissipation devices, the verification should be performed at the stages of maximum displacement, maximum velocity and maximum acceleration, using the effects of the design seismic actions calculated according to 9.3.2.4. In case of structures with displacement-dependent energy dissipation devices the verification should be performed at the stage of maximum displacement using the design seismic actions calculated according to 9.3.3.4.
3. All components of the energy dissipation system should be verified according to prEN 1998‑1‑1:2022, 6.7.2(1), taking into account 6.9.1(3), (4), (5), and 9.2.3(4). Verification should be made in terms of forces in velocity-dependent energy dissipation devices, of displacements in displacement-dependent ones and of forces in the other components of the energy dissipation system.

### Verification to Near Collapse (NC) limit state

1. 6.3.1 should be applied to the primary seismic members of the main structural system.
2. It should be verified that, under the specified seismic action, the amount of energy that can be dissipated by the energy dissipation devices of each storey *k* in the direction under consideration before one of them reaches the NC limit state is greater than the maximum energy dissipation demand in the same direction on the energy dissipation system at the *k*-th storey.
3. In case of velocity-dependent energy dissipation devices, (2) may be considered satisfied if the energy dissipation capacity of the energy dissipation devices of each storey *k* at the point in the response when any energy dissipation device on that storey attains its NC limit state, evaluated in accordance with EN 15129, is greater than the maximum energy dissipation demand given by Formula (9.2) for the spectral acceleration value *S*e corresponding to the design seismic action associated to NC.
4. In case of displacement-dependent energy dissipation devices, (2) may be considered satisfied if the energy dissipation capacity of the energy dissipation devices installed on each storey *s* at the point in the response when any energy dissipation device on that storey attains its NC limit state evaluated in accordance with EN 15129, is greater than *E*dH,*k,*max calculated in accordance with Annex D, D.4.4, for the spectral acceleration value *Se* corresponding to the design seismic action associated to NC.
5. It should be verified that, under the corresponding design seismic action, all the structural members of the energy dissipation system, exclusive of the elements that are common with the main structural system and of the energy dissipation devices, remain in the elastic range.

### Verification to Damage Limitation (DL) limit state

1. At the DL limit state, 6.3.2 should be applied to the primary seismic members of the main structural system.

### Verification of fully Operational (OP) limit state

1. At OP limit state, 6.3.3 should be applied to the primary seismic members of the main structural system.

# Specific rules for concrete buildings

## General

1. This clause should be applied to the design and the verification of reinforced and prestressed concrete buildings, both monolithic cast-in-situ and precast, henceforth called concrete buildings, in seismic regions.
2. This clause should be applied as a complement to FprEN 1992-1-1.
3. This clause should be applied to the design of structural members and the detailing of their critical regions. Outside the critical regions, detailing of structural members should satisfy FprEN 1992-1-1.
4. FprEN 1992-1-1, prEN 1998-5:2022, Clause 9, and 10.15 of this document should be applied to the design of concrete foundation members, such as footings, tie-beams, foundation beams, foundation slabs, foundation walls, pile caps and piles, as well as of connections between such members, or between them and vertical concrete members, and the verification of sliding and bearing capacity.
5. Seismic design for DC1, adopted according to the seismic action indices limits in 10.4.3, should comply with 10.2.2, 10.3.2, 10.3.4, 10.4.2.1, 10.4.3, 10.4.4 and 10.7(1), (3) and (4).
6. Seismic design for DC2 and DC3, adopted according to the seismic action indices limits in 10.4.3, should comply with Clause 10.
7. prEN 1998-1-1:2022, 4.1(7), may be used if the conditions in 10.4.1(6) and 10.4.2.1(2) and (3) are satisfied.
8. For the stiffness of members considered in the analysis, 5.1.3 should be satisfied.

## Basis of design and design criteria

### General rules on design action effects

1. With the exception of ductile walls and large walls, to which 10.8.2 and 10.9.2 should be applied, the design values of bending moments and axial forces should be obtained from the analysis of the structure in the seismic design situation according to EN 1990:2023, 8.4.3.5, taking into account second order effects in accordance with 6.2.4, and the capacity design provisions of 6.2.7(2).
2. In addition to (1), the design values of axial forces in columns should comply with 10.6.2(1).
3. Redistribution of bending moments in accordance with PrEN 1992-1-1:2021, 7.3.2, may be used.
4. The design values of shear forces of primary seismic beams, columns, ductile walls and large walls, should be determined in accordance with 10.5.3, 10.6.2, 10.8.2 and 10.9.2, respectively.
5. The condition in 5.3.5.1(3) may be considered as satisfied in reinforced concrete large walls buildings if the number of storeys is no more than three and if the average aspect (height to width) ratio of structural walls is smaller than 1,0.

### Local resistance condition

1. All critical regions of the structure should satisfy prEN 1998-1-1:2022, 6.7.1(1) and (2).

### Local ductility condition

1. For the required overall ductility of the structure to be achieved, the potential regions for plastic hinge formation, to be defined for each type of building member, should satisfy the conditions a) and b), in addition to the material properties provisions in 10.3.1 and 10.3.3:
2. a sufficient ductility and plastic rotation capacity should be provided in all critical regions of primary seismic members (at column ends, depending on the potential for plastic hinge formation there) (see (2));
3. local buckling of compressed steel bars in potential plastic hinge regions of primary seismic members should be prevented by applying relevant provisions in 10.5.4, 10.6.3, 10.7, 10.8.3 and 10.9.3.
4. To satisfy (1)a), the product of the global displacement ductility factor, taken equal to the product of the components of the behaviour factor *q*R and *q*D and the chord rotation at yielding of each member end, determined according to prEN 1998-1-1:2022, 7.2.2.1.1, should not exceed the chord rotation for the SD Limit State given in prEN 1998-1-1:2022, Formula (6.32).

### Capacity design rule for moment resisting frames

1. In accordance with 6.2.6 and 6.2.7, brittle failure or other undesirable mechanisms (e.g. a soft-storey mechanism, shear failure of members or joints, plastic hinging in foundation members intended to remain elastic) shall be prevented, by deriving the design action effects of selected regions from equilibrium conditions, assuming that plastic hinges with their possible overstrength have formed in adjacent areas.
2. The primary seismic columns of moment resisting frame and moment resisting frame-equivalent concrete structures should satisfy 6.2.7(2) for DC3, and in the situations listed in 6.2.6(9) for DC2, except in cases a) to c) for DC3, and b) for DC2:
3. in one column out of every four columns in a plane moment resisting frame, if its seismic acting shear force does not exceed 25 % of the total seismic shear force in the set of columns;
4. at the bottom storey of two-storey buildings, if the normalized axial load *ν*d in the seismic design situation does not exceed 0,3 in any column;
5. at the base of discontinued columns (see 10.5.2) supported by primary seismic beams.
6. On each side of a beam, slab reinforcement parallel to the beam and within an effective width up to one-quarter of the beam’s clear span, or up to mid-distance to the nearest parallel beam, or to the edge of the slab, whichever is smallest, should be assumed to contribute to the beam flexural capacity taken into account for the calculation of Σ*M*Rb in Formula (6.5), if it is anchored beyond the beam section at the face of the joint.

## Materials requirements

### General

1. In primary beams and columns and in the critical regions of ductile walls of DC2 or DC3, reinforcing steel of ductility class B or C in PrEN 1992-1-1:2021, Table 5.5, should be used.

### Design for DC1

1. Concrete of a class lower than C16 should not be used in primary seismic members.

### Design for DC2 and DC3

1. Concrete of a class lower than C20 should not be used in primary seismic members.
2. Ribbed bars should be used as reinforcing steel in critical regions of primary seismic members.

### Safety verifications

1. For the Significant Damage (SD) limit state verifications, partial factors for concrete and steel strengths *γ*c and *γ*s not smaller than 1 should be used.

NOTE *γ*c and *γ*s are equal to 1,5 and 1,15 respectively, unless the National Annex gives different values for use in a country.

## Structural types, behaviour factors, limits of seismic action, limits of drift and partial factors for the displacement-based approach

### Structural types

1. Concrete buildings designed to be dissipative should be classified into one of the structural types given in a) to i), according to their behaviour under horizontal seismic actions:
2. Moment resisting frame structure: structure in which both the vertical and lateral loads are mainly resisted by plane concrete moment resisting frames, whose shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structure (*V*Rd,MRF/*V*Rd,total  65 %);
3. Dual structure (moment resisting frame-equivalent or wall-equivalent): structure in which the resistance to lateral loads is contributed to in part by the moment resisting frame structure and in part by ductile walls, coupled or uncoupled (*V*Rd,MRF/*V*Rd,total = 35 % to 65 %);
4. Moment resisting frame-equivalent dual structures: those in which the shear resistance of the moment resisting frames at the building base is greater than 50 % of the total (*V*Rd,MRF/*V*Rd,total = 50 % to 65 %);
5. Wall-equivalent dual structures: those in which the shear resistance of ductile walls at the building base is greater than 50 % of the total (*V*Rd,MRF/*V*Rd,total = 35 % to 50 %);
6. Wall structures (coupled or uncoupled): those in which lateral loads are mainly resisted by concrete ductile walls, coupled or uncoupled, whose shear resistance at the building base exceeds 65 % of the total shear resistance (*V*Rd,walls/*V*Rd,total  65 %);
7. Coupled walls structures: ductile wall structure in which at least 50 % of the total shear resistance is provided by walls; comprising two or more single ductile walls, connected by a regular pattern of ductile beams (“coupling beams”), where at least 25 % of the total overturning moment at the base is supported by frame action of the vertical walls (“coupled walls”) to the coupling beams;
8. Large walls structures: wall structures with at least two large walls in the horizontal direction of interest, which collectively support at least 20 % of the total gravity load and have a fundamental fixed base period *T*1 not greater than *T*c. It is sufficient to have only one wall meeting these conditions in one of the two directions, provided that: (a) the basic value of the behaviour factor, *q*, in that direction is divided by a factor of 1,5 over the value in Table 10.1 and (b) there are at least two walls meeting these conditions in the orthogonal direction;
9. Flat slab structure: those composed of flat slabs and columns considered as primary seismic members which contribute to the resistance to lateral loads by a slab-column mechanism;
10. Inverted pendulum structures: those in which 50 % or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building member.
11. Concrete buildings may be classified to one structural type in one horizontal direction and to another in the other direction.
12. If, in one direction, a structure does not qualify as a large walls structure according to (1)g), then all of its walls in that direction should be designed and detailed as ductile walls, and the foundations should be designed to fix the rotation at their base.
13. Buildings in which the stiff vertical earthquake resisting structures are located near the perimeter in plan, with a balanced stiffness distribution between parallel sides (Figure 10.1), may be considered to have the torsional rigidity which is necessary to be classified as not torsionally flexible (see 4.4.3(1)), without analytical verification.

Figure 10.1 — Concrete buildings with vertical members mainly located near the perimeter and a balanced stiffness distribution between parallel sides

1. One-storey moment resisting frame structures with column tops connected (rigidly or hinged) along both main directions and with the value of the column normalized axial load *ν*d in the seismic design situation nowhere exceeding 0,3, may be classified as moment resisting frame structures. Otherwise, they should be classified as inverted pendulum structures.
2. Structural systems which cannot be assigned to one of the types in (1) may be used; they should be designed for strength according to 10.4.2.1(3).

### Behaviour factor for horizontal components of the seismic action in force-based analysis

#### Design for DC1

1. In the design of structural systems of the types in 10.4.1(1) a) to j), a behaviour factor *q* equal to 1,5 may be used, regardless of the regularity.
2. 10.2.2, 10.3.1, 10.3.2, 10.3.4, 10.4.4 and 10.7(1), (3) and (4), should be satisfied.
3. prEN 1998-1-1:2022, 4.1(7), may be used without limitation of the seismic action index *S*δ in the design of structural systems which cannot be assigned to one of the types in 10.4.1(1) a) to i), provided that the conditions in (2) are satisfied.

#### Design for DC2 and DC3

1. Except as given in 10.14.1 for precast structures, the default values of the behaviour factor components *q*R and *q*D, and of the behaviour factor *q*, are given in Table 10.1.

Table 10.1 — Default values of the behaviour factors

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| **Structural type** | | ***q*R** | ***q*D** | | ***q = q*R*q*S*q*D** | |
|  | **DC2** | **DC3** | **DC2** | **DC3** |
| Moment resisting frame or moment resisting frame-equivalent dual structures | multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual structures | 1,3 | 1,3 | 2,0 | 2,5 | 3,9 |
| multi-storey, one-bay moment resisting frames | 1,2 | 2,3 | 3,6 |
| one-storey moment resisting frames | 1,1 | 2,1 | 3,3 |
| Moment resisting frame or  moment resisting frame-equivalent dual structures with interacting masonry infills |  | 1,1 | 1,2 | 1,4 | 2,0 | 2,3 |
| Wall or wall-equivalent dual structures | wall-equivalent dual structures | 1,2 | 1,3 |  | 2,3 | 3,6 |
| coupled walls structures | 1,2 | 1,4 | 2,0 | 2,5 | 3,6 |
| uncoupled walls structures | 1,0 | 1,3 |  | 2,0 | 3,0 |
| large walls structures | -- | -- | | 3,0 *k*w | |
| Flat slab structures |  | 1,1 | 1,2 | -- | 2,0 | -- |

1. To account for the prevailing failure mode in large walls structures, the behaviour factor *q* should be calculated by multiplying the value of *q* given in Table 10.1 by the factor *k*w given by Formula (10.1)

(10.1)

where *α*0 is the prevailing aspect ratio (height-to-length ratio) of the walls of the structural system, which may be calculated by Formula (10.2).

(10.2)

where

|  |  |
| --- | --- |
| *h*wi | is the height of wall *i*; |
| *l*wi | is the length of the section of wall *i*. |

1. The behaviour factor *q* should be reduced for structures non-regular in elevation (see 4.4.4.2(1)) according to 5.3.2(2) and for torsionally flexible structures (see 4.4.3(1)) according to 5.3.2(5), but a minimum value of 1,5 may be adopted for *q*.
2. For inverted pendulum structures, 1,5 should be used for the behaviour factor *q*.

### Limits of seismic action for design to DC1, DC2 and DC3

1. Seismic design for DC1 should not be adopted in cases a) and b):
2. for moment resisting frame structures with or without interacting infills, for dual structures (moment resisting frame-equivalent or wall-equivalent, with or without interacting infills) and for flat slab structures: if *S*δ > 2,5 m/s2;
3. for wall structures: if *S*δ > 5,0 m/s2 (see prEN 1998-1-1:2022, 4.1(4)).
4. For *S*δ > 5,0 m/s2, moment resisting frame structures with or without interacting infills should be designed for DC3.
5. Flat slab structures should not be adopted for *S*δ > 5,0 m/s2.
6. Flat slab structures should not be designed for DC3.
7. With the exceptions of (2) to (4), seismic design for DC2 and DC3 may be adopted for all levels of *S*δ and structural types.

### Limits of drift

1. For structural systems of the types in 10.4.1(1) a) to i), the interstorey drift at SD limit state according to 6.2.5(1) should be limited to: *d*r,SD ≤ 0,02 *h*s where *d*r,SD is defined in 6.2.4(1) and *h*s is the storey height.
2. For moment resisting frames with interacting infills, 7.4.2.1 should be applied.
3. For all other structures, *d*r,SD should be limited to 0,015 *h*s.

### Partial factors on resistances for the displacement-based approach

1. In case a displacement-based approach is used, local generalized deformations, according to case a) of prEN 1998-1-1:2022, 6.7.1(2), should be used for verification of ductile mechanisms to SD and NC.
2. For beams and columns in frames detailed according to this standard, deformation criteria and shear strength should be taken as given in prEN 1998-1-1:2022, 7.2.
3. For beams and columns and walls in frames detailed according to this standard, the total logarithmic standard deviation of the resistance model required to calculate the partial factors on chord rotation (see prEN 1998-1-1:2022, 6.7.2(2)) and shear strength (see prEN 1998-1-1:2022, 6.7.2(3)) at SD and NC according to 6.2.3(3) are given in Table 10.2, depending on cross-section shape. Values of the partial factors at SD and NC are given in Table 10.3 (NDP).

Table 10.2 — Total logarithmic standard deviation **lnR of the resistance model

|  |  |  |
| --- | --- | --- |
| **Section shape** | **Chord rotation **u** | **Shear** |
| Rectangular | 0,45 | 0,35 |
| Hollow | 0,45 | 0,35 |
| Circular | 0,35 | 0,30 |

Table 10.3 (NDP) — Partial factors at SD and NC

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Section shape** | **Chord rotation **u** | | **Shear** | |
| **SD** | **NC** | **SD** | **NC** |
| Rectangular, hollow | 1,8 | 2,3 | 1,6 | 1,9 |
| Circular | 1,6 | 1,9 | 1,5 | 1,75 |

NOTE The values in Table 10.3 (NDP) are computed with the values of **t,LS,2 (i.e. CC2) suggested in prEN 1998-1-1:2022, F.3. Values for CC other than CC2 can be calculated according to the note in 6.2.3(3). If the National Annex gives different values of **t,LS,CC for use in a country, the partial factors can be updated according to the note in 6.2.3(3).

## Beams

### Geometrical and other provisions

1. Prismatic concrete members should be designed as beams when subjected mainly to transverse loads and to a normalized design axial force in the seismic design situation *ν*d = *N*Ed/(*A*c *f*cd) not greater than 0,1.
2. The eccentricity of the primary seismic beam axis relative to that of the column into which it frames should not be greater than *b*max/3, where *b*max is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam.
3. The width *b* of the primary seismic beam should not be greater than the minimum of *b*max+*h* and 2*b*max, where *h* is the beam depth and *b*max is as defined in (2).
4. The effective flange width of beams cast monolithically with the slab may be taken in the model for the analysis equal to the constant value specified in PrEN 1992-1-1:2021, 7.2.3(4), for the whole span.

### Specific rules for beams supporting discontinued vertical members

1. Structural walls should not be supported on beams, unless the beams are designed as transfer zones in the seismic design situation according to 6.2.11 and 10.15.
2. For a primary seismic beam which support discontinued columns, conditions a) and b) should be fulfilled:
3. there should be no eccentricity of the column axis relative to that of the beam;
4. the beam should be supported by at least two direct supports, such as walls or columns directly founded, unless the beam is part of the transfer structure designed according to 6.2.11 and 10.15.
5. 10.5.4.2(2) should be applied.

### Design action effects

1. For DC2 and DC3, in primary seismic beams of all structural types, with clear length *l*cl, the design values of shear forces *V*i,d should be determined on the basis of the equilibrium of the beam under the effects of actions in a) and b) (Figure 10.2a):
2. the transverse load acting on it in the seismic design situation;
3. end moments *M*i,d (with *i* = 1, 2 denoting the ends of the beam), as given by Formula (10.3), corresponding to plastic hinge formation at the ends of the beam or, if they form there first (as in the exceptions in 10.2.4(2) for frame or frame-equivalent structures), in the vertical members connected to the joints into which the beam ends frame, for positive and negative directions of seismic demand (Figure 10.2b):

(10.3)

where

|  |  |
| --- | --- |
| *γ*Rd | is a factor accounting for possible overstrength due to steel strain hardening and confinement of the concrete of the compression zone, which may be taken equal to 1,1 for DC2 and 1,15 for DC3; |
| *M*Rd,b,i | is the design value of the beam resisting moment at end *i* in the sense of bending moment considered, taking into account the slab reinforcement within an effective width defined in 10.2.4(3); |
| Σ*M*Rd,c | is the sum of the design values of the resisting moments of the columns framing into the joint (see 6.2.7(2)), corresponding to the column axial force(s) in the seismic design situation for the considered sense of beam bending; |
| Σ*M*Rd,b | is the sum of the design values of resisting moments of the beams framing into the joint (see 6.2.7(2)). |

Figure 10.2 — Capacity design shear forces on beams: (a) determination of design shear forces; (b) design end moment

1. At a beam end *i* supported by another beam, instead of framing into a vertical member, the beam end moment *M*i,d there may be taken equal to the moment at the beam end section obtained from the analysis in the seismic design situation.

### SD limit state verifications and detailing

#### Resistance in bending and shear

1. The bending resistance should be calculated in accordance with prEN 1992-1-1:2022, 8.1.
2. The shear resistance should be calculated in accordance with PrEN 1992-1-1:2021, 8.2, as modified in prEN 1998-1-1:2022, 7.2.3.

#### Detailing for local ductility

1. The regions of a primary seismic beam up to a distance *l*cr = *h* (where *h* denotes the beam depth) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, should be considered as critical regions.
2. In primary seismic beams supporting discontinued (cut-off) vertical members, the regions up to a distance of 2*h* on each side of the supported vertical member should be considered as being critical regions.
3. Along the entire length of a primary seismic beam, the reinforcement ratio of the tension zone, *ρ*l, should not be smaller than the minimum value *ρ*l,min given by Formula (10.4).

(10.4)

Alternatively to the explicit calculation of *ρ*l,min given by Formula (10.4), the values in Table 10.4 may be used.

Table 10.4 — Minimum longitudinal reinforcement ratio in tension zones of beams (*ρ*l,min)

|  |  |  |
| --- | --- | --- |
|  | **Steel grade** | |
| **Concrete grade** | **B400** | **B500** |
| **C20-C25** | 0,35 % | 0,25 % |
| **C30-C45** | 0,50 % | 0,35 % |
| **C50-C90** | 0,60 % | 0,45 % |

1. At the compression zone of critical regions of primary seismic beams, reinforcement not smaller than half of the reinforcement provided at the tension zone should be placed.
2. The tension reinforcement ratio should not exceed a maximum value *ρ*l,max in Table 10.5, where *ρ*l’ is the reinforcement ratio in the compression zone. For beams with reinforcing steel grade B450, the maximum tension reinforcement ratio may be obtained by linear interpolation of the values in Table 10.4.

Table 10.5 — Maximum longitudinal reinforcement ratio in tension zones of beams (*ρ*l,max)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | **DC2** | | **DC3** | |
|  | **Steel grade** | | **Steel grade** | |
| **Concrete grade** | **B400** | **B500** | **B400** | **B500** |
| **C20-C25** | *ρ*l’ + 0,7 % | *ρ*l’ + 0,5 % | *ρ*l’ + 0,5 % | *ρ*l’ + 0,3 % |
| **C30-C45** | *ρ*l’ + 1,2 % | *ρ*l’ + 0,8 % | *ρ*l’ + 1,0 % | *ρ*l’ + 0,6 % |
| **C50-C90** | *ρ*l’ + 1,7 % | *ρ*l’ + 1,2 % | *ρ*l’ + 1,5 % | *ρ*l’ + 1,0 % |

1. Within the critical regions of primary seismic beams, hoops should satisfy the conditions in a) to d):
2. the diameter *d*bw of the hoops should not be smaller than 6 mm;
3. the spacing *s* of hoops should not exceed:
4. for DC2: *s* = min{*h*/4; 30*d*bw; 12*d*bL,min};
5. for DC3: *s* = min{*h*/4; 24*d*bw; 8*d*bL,min},

where

|  |  |
| --- | --- |
| *d*bL,min | is the minimum longitudinal bar diameter; |
| *h* | is the beam depth. |

1. the distance of the first hoop to the beam end section should not be greater than 50 mm (Figure 10.3).
2. 10.11.1(2) should be satisfied.

Figure 10.3 — Transverse reinforcement in critical regions of beams

## Columns

### Geometrical and other provisions

1. Prismatic concrete members with a depth to width ratio *h*c/*b*c not greater than 4 should be designed as columns when supporting gravity loads by axial compression or when subjected to a normalized design compression axial force in the seismic design situation *ν*d = *N*Ed/(*A*c*f*cd) greater or equal to 0,1.
2. The minimum cross-sectional dimension of primary seismic columns should not be smaller than a) or b):
3. If *θ* ≤ 0,05: 200 mm (where *θ* is defined in 6.2.4(1)).
4. If *θ* > 0,05: the maximum of:
5. one tenth of the longer distance between the point of contraflexure of the deflected shape and the ends of the column, for bending within a plane parallel to the column dimension considered;
6. 200 mm for DC2 or 250 mm for DC3.

### Design action effects

1. In primary seismic columns of DC2 and DC3 the design column axial force should be obtained from Formula (10.5).

(10.5)

where

|  |  |
| --- | --- |
| *N*Ed,G,i | is the column axial force fraction due to the gravity loads in the seismic design situation; |
| *N*Ed,E,i | is the column axial force fraction due to the seismic action in the seismic design situation; |
| *Ω* | is the magnification factor of the column axial force, which may be taken equal to 2,0; |
| “+” | means combined with + or – sign. |

1. For DC2 and DC3, in primary seismic columns, the design shear forces *V*i,d should be determined on the basis of equilibrium of the column under end moments *M*i,d (*i* = 1, 2 denoting the end sections), calculated from Formula (10.6), corresponding to plastic hinge formation for positive and negative directions of the response (Figure 10.4):

(10.6)

where

|  |  |
| --- | --- |
| *γ*Rd | accounts for overstrength due to strain hardening and confinement and may be taken equal to 1,1; |
| *M*Rd,c,i | is the design value of the column resisting moment at end *i*. |
| Σ*M*Rd,c and Σ*M*Rd,b | are defined in 10.5.3.1(1)b). |

1. The value~~s~~ of *M*Rd,c,i and Σ*M*Rd,c should correspond to the column axial force *NEd,I* from Formula (10.5) for the considered sense of seismic response.

Figure 10.4 — Determination of capacity design shear forces in columns

### SD limit state verifications and detailing

#### Resistance in bending and shear

1. In primary seismic columns, the normalized axial force *ν*d based in Formula (10.5) should not exceed:

a) for DC2: 0,65;

b) for DC3: 0,55.

1. The resistance to combined bending moment and axial force and to shear action effects in the seismic design situation should be calculated in accordance with prEN 1992-1-1:2021, 8.1 and 8.2.

#### Detailing for local ductility

1. The total longitudinal reinforcement ratio *ρ*l should not be less than 1 % and not greater than 4 %.
2. The diameter of the longitudinal bars should not be smaller than 12 mm.
3. In symmetrical cross sections, symmetrical reinforcement should be provided (*ρ*l = *ρ*l’).
4. At least one intermediate bar should be provided between corner bars along each column side.
5. The region up to a distance *l*cr given by Formula (10.7) from an end section of a primary seismic column should be considered a critical region:

(10.7)

where

|  |  |
| --- | --- |
| *b*max | is the largest cross-sectional dimension of the column; |
| *l*cl | is the clear length of the column. |

1. If *l*cl/*b*max < 3, the entire height of the primary seismic column should be taken as critical region.
2. In a critical region of a primary seismic column, hoops and cross-ties, of at least 6 mm in diameter or *d*bL,min/4, whichever is greater, should be provided with a pattern such that the cross-section benefits from confinement (examples in Figure 10.5a). 10.11.1(2) should be satisfied. The spacing, *s*, of hoops and cross-ties (Figure 10.5b) should not exceed:

a) for DC2: *s* ≤ min{*b*0C/2; 200 mm; 9*d*bL,min};

b) for DC3: *s* ≤ min{*b*0C/2; 175 mm; 8*d*bL,min};

where

|  |  |
| --- | --- |
| *b*0C | is the smallest dimension of the concrete core (to the centreline of the hoops) (Figure 10.5a); |
| *d*bL,min | is the minimum diameter of the longitudinal bars. |

1. The distance between consecutive longitudinal bars engaged by hoops or cross-ties should not exceed 250 mm for DC2 and 200 mm for DC3, taking into account PrEN 1992-1-1:2021, 12.6.

Figure 10.5 — Transverse reinforcement in columns: (a) examples of hoops and cross-ties pattern and confined concrete core; (b) critical regions

1. A minimum value of mechanical volumetric ratio of confining hoops within the critical regions, given by Formula (10.8), should be provided within all critical regions in the primary seismic columns equal to:

a) for DC2: 0,05;

b) for DC3: 0,08.

(10.8)

1. The mechanical volumetric ratio of confining hoops of columns with rectangular cross sections with dissimilar amount of reinforcement in the two directions should be taken from Formula (10.9).

(10.9)

where *ρ*w,x and *ρ*w,y are the volumetric ratios of confining hoop legs in the perpendicular directions *x* and *y*.

## Beam-column joints

1. For DC1, the horizontal confinement reinforcement in beam-column joints should not be smaller than that of the columns framing into the joint, with the exception in (3).
2. For DC2 and DC3, the horizontal confinement reinforcement in beam-column joints should not be smaller than that specified in 10.6.3.2(7) to (10) for the critical regions of columns, with the exception in (3). 10.11.1(2) should be satisfied.
3. If beams frame into all four sides of the joint and their width is at least three-quarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that specified in (1) and (2), but should not exceed 150 mm.
4. For DC1, DC2 and DC3, at least one intermediate vertical bar should be provided between column corner bars at each side of the primary beam-column joint.
5. For DC3, unless (6) is used, the verification of the joint should be done according to prEN 1998‑1‑1:2022, 7.2.4, with the modifications in a) to d):
6. The mean values of resistances are replaced by their design values: by , by , by , by , *V*Rj,c by *V*Rdj,c, *V*Rj,v by *V*Rdj,v;
7. The characteristic value of the steel yield strength *f*yk (or *f*yk,h or *f*yk,v) is replaced by the corresponding design value (or *f*yd,h or *f*yd,v);
8. The mean value of concrete strength is replaced by the design value ;
9. The mean value of concrete tensile strength is replaced by the design value .
10. Alternatively to (5), the values in Table 10.6 may be taken as minimum horizontal shear reinforcement ratio in joints (*ρ*sh  *A*sh/(*b*j,ef *h* *b*)), when a) to f) are satisfied:
11. Axial load ratio: **  ≥ 0,1 for interior joints; and 0,1 ≤ ** ≤ 0,25 for exterior joints;
12. *b*b ≤ *b*c;
13. 0,20 m ≤ *b*b ≤ 0,40 m;
14. 0,30 m ≤ *h*b ≤ 0,60 m;
15. 0,30 m ≤ *b*c ≤ 0,60 m;
16. 0,30 m ≤ *h*c ≤ 0,60 m.

Table 10.6 — Minimum horizontal reinforcement ratio in beam-column joints (*ρ*sh)

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | **Concrete grade** | Top (*ρ*s1) and bottom (*ρ*s2) beam longitudinal reinforcement ratio of beams generating shear in the joints | **Steel grade** | | | |
|  | **B400** | | **B500** | |
|  | 0,75 ≤ *h*c/*h*b < 1,25 | 1,25 ≤ *h*c/*h*b ≤ 2,0 | 0,75 ≤ *h*c/*h*b < 1,25 | 1,25 ≤ *hc*/*h*b ≤ 2,0 |
| **Interior beam-column joints** | **C20-C25** | *ρ*s1 + *ρ*s2 ≤ 0,8 % | 0,45 % | 0,40 % | 0,55 % | 0,45 % |
| 0,8 % < *ρ*s1 + *ρ*s2 ≤ 1,2 % | 0,90 % | 0,80 % | 1,00 % | 0,85 % |
| **C30-C45** | *ρ*s1 + *ρ*s2 ≤ 0,8 % | 0,40 % | 0,40 % | 0,45 % | 0,45 % |
| 0,8 % < *ρ*s1 + *ρ*s2 ≤ 1,2 % | 0,80 % | 0,65 % | 0,85 % | 0,70 % |
| **C50-C90** | *ρ*s1 + *ρ*s2 ≤ 0,8 % | 0,40 % | 0,40 % | 0,45 % | 0,45 % |
| 0,8 % < *ρ*s1 + *ρ*s2 ≤ 1,2 % | 0,55 % | 0,45 % | 0,70 % | 0,55 % |
| **Exterior beam-column joints** | **C20-C25** | max{*ρ*s1; *ρ*s2} ≤ 0,6 % | 0,45 % | 0,35 % | 0,50 % | 0,45 % |
| 0,6 % < max{*ρ*s1; *ρ*s2} ≤ 0,8 % | 0,70 % | 0,60 % | 0,70 % | 0,65 % |
| **C30-C45** | max{*ρ*s1; *ρ*s2} ≤ 0,6 % | 0,40 % | 0,40 % | 0,40 % | 0,40 % |
| 0,6 % < max{*ρ*s1; *ρ*s2} ≤ 0,8 % | 0,60 % | 0,50 % | 0,65 % | 0,55 % |
| **C50-C90** | max{*ρ*s1; *ρ*s2} ≤ 0,6 % | 0,40 % | 0,40 % | 0,40 % | 0,40 % |
| 0,6 % < max{*ρ*s1; *ρ*s2} ≤ 0,8 % | 0,45 % | 0,40 % | 0,55 % | 0,45 % |

## Ductile walls

### Geometrical and other constraints

1. Concrete walls are defined as the vertical members which have an elongated cross-section with a length to thickness ratio *l*w/*b*w greater than 4.
2. Concrete ductile walls should be fixed at the base so that the relative rotation of their base with respect to the rest of the structure is prevented and should be designed and detailed to dissipate energy in a flexural plastic hinge zone free of large openings or large perforations just above their base and at levels where the change in wall length *l*w is greater than 30 %.

NOTE Limits of the dimensions of perforations are given in 10.8.3.3(2).

1. The thickness of the web, *b*w0, should satisfy Formula (10.10).

(10.9)

where *h*s,cl is the clear storey height.

1. Openings not regularly arranged to form coupled walls should be avoided in primary seismic walls, unless their effect is either insignificant or accounted for in the analysis, dimensioning and detailing.

NOTE The conditions for insignificant effect of not regularly arranged openings are given in 10.8.3.3(6) and (7).

### Design action effects

1. Seismic action effects may be redistributed between primary seismic walls up to 30 %, provided that the total resistance is not reduced. Shear forces should be redistributed along with bending moments, so that in the individual walls the ratio of bending moment to shear force is not changed by more than 20 %. If the variation of the axial force is large, as e.g. in coupled walls, bending moments and shear forces should be redistributed from the wall(s) which are under low compression or in net tension, to those which are under greater compression.
2. In coupled walls, as defined in 10.4(1)f), seismic action effects may be redistributed between coupling beams of different storeys up to 20 %, provided that the seismic axial force at the base of each individual wall is not affected.
3. The flexural resistance of the wall at its base should not be lower than the design moment *M’*Edw,base from the analysis. Elsewhere along the height of the wall, the design moment *M*Edw should be given by a vertically displaced (tension shift) envelope of the bending moment diagram from the analysis, with the value at the base section replaced by the flexural overstrength moment, *M*0 = *γ*Rd*M*Rdw,base, with *γRd* = 1,2, where *M*Rdw,base is the flexural resistance of the wall at the base section including the vertical web reinforcement. Within the critical height of the wall, defined in 10.8.3.2(1), the design moment should be taken equal to *M*Rdw,base. The tension shift and the critical height of the wall should be consistent with the strut inclination taken in the SD limit state verification for shear (Figure 10.6).

Key

|  |  |
| --- | --- |
| *M’*Edw | moment diagram from analysis |
| *M*Rdw,base | flexural resistance at the base section |
| *M*Edw | design moment envelope |
| *h*cr | critical height |

Figure10.6 — Design envelope of bending moments in ductile walls: (a) cantilever wall structures; (b) dual structures

1. In ductile walls, when the force-based approach is used, the design shear force *V*Edw at level *z* should be derived from Formula (10.11).

(10.11)

where

|  |  |
| --- | --- |
| *V’*Edw(*z*) | is the shear force at level *z* calculated as the combination of shear force in all modes from the analysis; |
| *V’*Edw,1(*z*) | is the shear force at level *z* calculated by the analysis; if a modal response spectrum analysis is used, *V’*Edw,1(*z*) is the effect due to the mode with the largest participating mass in the direction of *V*Edw; |
| *q* | is the behaviour factor used in the design; |
| *ε*(*z*) | is the shear magnification factor from Formula (10.12), but not smaller than 1,5 nor greater than *q*; |

(10.12)

|  |  |
| --- | --- |
| *M’*Edw,base | is the design moment at the base of the wall (from analysis); |
| *M*Rdw,base | is the design moment of resistance at the base of the wall; |
| *γ*Rd | accounts for overstrength due to steel strain-hardening and may be taken equal to 1,2; |
| *S*e(*T*) | is the ordinate of the elastic response spectrum; |
| *T*C | is the upper corner period of the constant spectral acceleration range; |
| *T*1 | is the period of the mode with the largest participating mass in the direction of *V*Edw; |
| *m*s (*z*) | is a factor varying along the height of the wall, *h*w, as given by Formula (10.13). |

(10.13)

1. For DC2, alternatively to Formula (10.12), *ε*(*z*) may be assumed equal to *q*.
2. In walls in dual structures, the design shear forces *V*Edw,env should be in accordance with Figure 10.7.

Key

|  |  |
| --- | --- |
| *V’*Edw | shear diagram from analysis |
| *V*Edw | magnified shear diagram (obtained from (4)) |
| *V*Edw,env | design shear envelope |
| *h*w | wall height |
| *V*Edw,base | design shear at the base of the wall |
| *V*Edw,top | design shear at the top of the wall (*V*Edw,top *≥ V*Edw,base/2) |

Figure10. 7— Design envelope of the shear forces for ductile walls in dual structures

### SD limit state verifications and detailing

#### Resistance in bending, shear and sliding shear

1. Flexural resistance should be calculated according to PrEN 1992-1-1:2021, 8.1 and 8.2, unless specified otherwise in (2) to (6), using the value of the axial force from the analysis in the seismic design situation.
2. Vertical web reinforcement should be taken into account in the moment resistance of wall sections.
3. Composite wall sections consisting of connected or intersecting rectangular segments (T-, L-, U-, I- or similar sections) should be taken as integral units, consisting of a web or webs parallel, or approximately parallel, to the direction of the acting seismic shear force and a flange or flanges normal, or approximately normal, to it. In the calculation of the moment resistance and in the analysis (considering the properties of the cracked cross-section), the effective flange width on each side of a web should be taken to extend from the web face by the minimum of a) and b):
4. the actual flange width;
5. one-half of the distance to an adjacent web of the wall.
6. In primary seismic walls, the normalized design axial load *ν*d in the seismic design situation should not exceed:

a) for DC2: 0,40;

b) for DC3: 0,35.

1. The shear resistance should be calculated in accordance with 10.5.4.1(2).

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for experimental tests devoted to the determination of the shear resistance at the interfaces.

#### Detailing for local ductility

1. The height of the critical region *h*cr above the base of the wall (defined as the top of the foundation or of a rigid basement) may be taken from Formula (10.14).

(10.14)

with the limitation given by Formula (10.15).

(10.15)

where *h*s is the storey height and *n* is the number of storeys of the wall.

1. 10.6.3.2(1), (2), (7), (8), (9) and (10) should be applied within boundary elements of the wall extending vertically over the height *h*cr of the critical region as defined in (1), and horizontally along a length *l*c measured from the extreme compression fibre in the confined part of the wall up to a length not smaller than 0,15*l*w and 1,5*b*w (Figure 10.8).

Figure 10.8 — Confined boundary element of free-edge wall end

1. Confined boundary element may be omitted over wall flanges with thickness *b*f ≥ *h*s,cl/15 and width *l*f ≥ *h*s,cl/5 (Figure 10.9), where *h*s,cl denotes the clear storey height.

NOTE Confined boundary elements can be required at the ends of such flanges against out-of-plane bending.

Figure 10.9 — Confined boundary element not needed at wall end with large transverse flange

1. In the boundary elements of walls, every other longitudinal bar should be engaged by a hoop or cross-tie.
2. The thickness *b*w of confined parts of walls (boundary elements) should satisfy a) to c) (Figure 10.10):
3. *b*w should not be smaller than 200 mm;
4. if the length of the confined part does not exceed the maximum of 2*b*w and 0,2*l*w, *b*w should not be smaller than *h*s,cl*/*15, where *h*s,cl is the clear storey height;
5. if the length of the confined part exceeds both 2*b*w and 0,2*l*w, *b*w should not be smaller than *h*s,cl/10.

Figure 10.10 — Minimum thickness of confined boundary elements

1. The relevant rules of PrEN 1992-1-1:2021, 12.7, regarding vertical, horizontal and transverse reinforcement in the wall, should be applied, complemented with (7) to (13).
2. In the critical region of the wall, the total vertical reinforcement ratio should not be smaller than 0,25 % and the horizontal reinforcement ratio at each wall face should not be smaller than 0,125 %. 10.11.1(2) should be satisfied.
3. In the critical region of the wall, the distance between consecutive vertical and horizontal bars should not exceed 300 mm for DC2 or 250 mm for DC3.
4. In the critical region of the wall, horizontal reinforcement contributing to shear strength should be continuous, without lapping of bar outside of the boundary zones, and their distribution should be uniform.
5. Vertical reinforcement should be extended of at least 0,8*l*w beyond the wall section at which it is calculated that no reinforcement for flexure is necessary At locations where yielding of vertical reinforcement is likely to occur, anchorage lengths should be 1,25 times the values obtained from prEN 1992-1-1:2021, 11.4.2(2) and (3).
6. Above the critical region of the wall, the vertical reinforcement ratio should be at least 0,5 % in the parts of the section where under the seismic design situation the compressive strain *ε*c exceeds 0,002.
7. The confining reinforcement of boundary elements should be extended 300 mm below the bottom of the critical region if the boundary element is not near the edge of a footing and over the anchor length if the boundary element is near the edge of a footing.
8. The transverse reinforcement of the boundary elements in (2) to (5) may be determined according to prEN 1992-1-1:2022, 12.6, alone, if one of the conditions in a) or b) is fulfilled:
9. the value of the normalized design axial force *ν*d in the seismic design situation is not greater than 0,15;
10. the value of *ν*d in the seismic design situation is not greater than 0,20 and either:
11. in a force-based approach, the behaviour factor *q* used in the analysis is reduced by 15 %;
12. in a displacement-based approach, the displacement demand at SD is increased of 15 %.
13. Where confinement boundary elements are provided, the horizontal web reinforcement should be anchored by hooks or bends in the confined core of the boundary elements at not less than 150 mm from the end of the section. If the horizontal web reinforcement is not more than the transverse reinforcement of the boundary element parallel to it, it may terminate with a straight anchorage provided that the boundary element has sufficient length to accommodate it.

#### Openings and coupling beams in ductile walls

1. Perforations and openings in the critical region should satisfy a) to e), unless (9) is satisfied:
2. there should not be more than two perforations;
3. their height and width should both be smaller than 0,05 *l*w;
4. they should be in the central third of *l*w;
5. the distance between centres of perforations should be smaller than 0,2 *l*w;
6. hairpin U reinforcement or crossties should be placed at each interruption of the face layers reinforcement by the perforations and they should have the same diameter as the face layers reinforcement.
7. Around openings, ties should be provided which should not be less than given in a) and b):
8. on both sides of openings greater than 1 m2, vertical boundary elements extending to the storey above and consisting of not less than four 10 mm diameter bars in DC2 or four 12 mm diameter bars in DC3; on both sides of openings up to 1 m2, vertical ties not less than two 10 mm diameter bars in DC2 or two 12 mm diameter bars in DC3;
9. above and below openings, horizontal ties consisting of not less than two 10 mm diameter bars in DC2 or two 12 mm diameter bars in DC3.
10. Vertical bars satisfying (2) should be engaged by hoops or crossties with a diameter of not less than 6 mm or one-third of the vertical bar smallest diameter, *d*bL, whichever is greater. Hoops and crossties should be at a vertical spacing *s* of not more than 100 mm or 8*d*bL, whichever is less.
11. The effect of the openings should be considered in the analysis of the structure and in the design of walls, unless (5) is satisfied.
12. If in a wall the openings are not aligned vertically and if, at each storey, the area of openings is less than 10 % of the area of the wall at that storey, calculated as the product of the wall length *l*w by the storey height *h*s , the analysis of the structure may ignore the influence of the openings in that wall.
13. To be considered not aligned vertically, openings at each storey should be such that any vertical line intersecting them does not intersect an opening neither in the storey immediately above, nor in the one below.
14. If (6) applies, the action effect in the wall may be calculated considering a strut and tie model around openings, complying with prEN 1992-1-1:2021, 8.5.
15. The resistance and detailing of struts and ties in (7) should satisfy prEN 1992-1-1:2021, 8.5; compression struts in which the stress due to the design seismic action *E*d is greater than 0,2 *f*cd should be confined by means of closed stirrups or crossties connecting the two face layers of reinforcement.
16. If in a wall the openings are aligned vertically, the structure should be considered as being two walls coupled by beams with a clear length equal to the width of the openings and a depth equal to the distance from the top of an opening to the bottom of the opening at the storey above and 10.5.3 and 10.5.4 should be satisfied for the action effects *M*Edcband *V*Edcb which are the moment and shear in the coupling beam calculated by the analysis.
17. In (9), the coupling beams may be analysed in a strut and tie model under the shear action effects *V*Edcb acting vertically at its ends.
18. If the resistance to shear requires diagonal reinforcement of the coupling beams, these reinforcement should consist of a minimum of four bars anchored in the walls to develop *fyd*; reinforcement transverse to these diagonals should be provided in compliance with 10.5.4.2(6).
19. The reinforcement transverse to the diagonals in (11) should be as in a) or b):
20. hoops enclosing separately each group of four diagonal bars satisfying 10.6.3.2(7) to 10.6.3.2(10);
21. stirrups enclosing the entire cross section of the coupling beam completed by crossties to satisfy 10.6.3.2(7) to (10).

#### Tying systems

1. Outside of boundary elements, bars in the form of continuous steel ties, horizontal or vertical, should be provided, as given in a) to c), but not less than those specified in PrEN 1992-1-1:2021, 12.9:
2. along all intersections of walls and along the connections of walls with flanges: not less than four vertical bars with 10 mm diameter;
3. an effectively continuous peripheral tie with at least 300 mm2 cross-sectional area at each floor and roof level;
4. a horizontal tie at interior wall-floor connections with a cross sectional area of at least max {150 mm2; 28 *L*}, where *L* is the distance between the horizontal tie and the adjacent wall, in metres.

## Large walls

### Geometrical provisions

1. Concrete large walls should have a horizontal dimension *l*w at least equal to 4,0 m or to two-thirds of the height of the wall *h*w, whichever is less.

NOTE A concrete large wall, due to its dimensions, or to lack-of-fixity at the base, or to connection with large transverse walls cannot form a plastic hinge at the base and plastic hinging cannot be an effective energy dissipation mechanism. It develops limited cracking and non-linear behaviour in the seismic design situation.

1. 10.8.1(3) should be applied to large walls.

### Design action effects

1. The additional dynamic axial forces developing in large walls due to geometrical non-linearities or contact effects, should be taken into account in the SD limit state verification of the wall for bending with axial force.
2. Unless a response-history non-linear analysis takes into account the effects listed in (1), the additional dynamic axial forces may be taken as being 50 % of the axial force due to the gravity loads in the seismic design situation, with a plus or a minus sign, whichever is most unfavourable.
3. The dynamic axial force in (1) and (2) may be neglected in cases a) or b), as appropriate:
4. if the force-based approach is used and the behaviour factor *q* does not exceed 2,0;
5. if the displacement-based approach is used and the geometrical non-linearity not accounted for, if the displacement on the force-displacement curve does not exceed 150 % of the elastic limit, as defined in prEN 1998-1-1, 6.5.3.
6. To ensure that flexural yielding precedes attainment of the SD limit state in shear, the shear force from the analysis should be increased.
7. (4) may be considered satisfied if at every storey of the wall the design shear force *V*Ed is obtained from a) or b), as appropriate:
8. In the force-based approach, using Formula (10.16).

(10.16)

1. In the displacement-based approach, the amplification factor should be taken as **Rd = 1,3.
2. When using a non-linear static analysis of large walls structures, the elastic stiffness *k*\* of a bilinear idealization as in prEN 1998-1-1:2022, 6.5.3(2), should correspond to their uncracked stiffness.

### SD limit state verifications and detailing

#### Resistance in bending

1. The SD limit state in bending with axial force should be verified according to prEN 1992-1-1:2021, 8.1.
2. Normal stresses in the concrete shall be limited, to prevent out-of-plane instability of the wall.
3. (2) may be satisfied on the basis of the rules of prEN 1992-1-1:2021, 7.4, for second-order effects.
4. In the SD limit state, verification for bending with axial force taking into account the dynamic axial force defined in 10.9.2(1) and (2) may be performed in modifying the mean strain in the section, while keeping constant the curvature; the limiting strain *ε*cu may be increased to a greater value in accordance with prEN 1992-1-1:20211, 8.1.4, provided that spalling of the unconfined concrete cover is taken into account in the verification.

#### Resistance in shear and sliding shear

1. Wherever the value of *τ*Ed calculated from *V*Ed in 10.9.2(5) in accordance with prEN 1992-1-1:2021, 8.2.1(3) is less than the design value of shear resistance without shear reinforcement, *τ*Rd,c in prEN 1992‑1-1:2021, 8.2, the web minimum shear reinforcement ratio *ρ*w,min may be omitted.

NOTE This is due to the safety margin provided by the magnification of design shear forces in 10.9.2(4) and (5) and because the response (including possible inclined cracking) is deformation controlled.

1. Wherever the condition *τ*Ed ≤ *τ*Rd,c is not fulfilled, web shear reinforcement should be calculated in accordance with prEN 1992-1-1:2021, 8.2.3, on the basis of a variable inclination truss model, or a strut-and-tie model, whichever is most appropriate for the particular geometry of the wall.
2. If a strut-and-tie model is used, the strut width should take into account any openings in the wall and should not exceed 0,25*l*w or 4*b*w0, whichever is smaller, where *l*w is the wall length and *b*w0 the web thickness.
3. prEN 1998-1-1:2022, 7.2.3(8), should be applied.

#### Detailing for local ductility

1. The amount of vertical reinforcement placed in the wall should not unnecessarily exceed the amount required for the verification of the SD limit state in flexure with axial load and for the integrity of concrete.
2. The wall vertical reinforcement should be at least the minimal reinforcement of flexural members given in prEN 1992-1-1:2021, 12.2(2) and (3), or the minimum vertical wall reinforcement given in prEN 1992-1-1:2022, 12.7(2) and (3), whichever is greater.
3. Vertical bars for the verification of the SD limit state in bending with axial force should be concentrated in boundary elements at the ends of the cross-section. These boundary elements should extend in the direction of the length *l*w over a length not smaller than *b*w0 or 3*b*w0*σ*cm/*f*cd, whichever is greater, where *σ*cm is the mean value of the concrete stress in the compression zone in the SD limit state of bending with axial force. They should be anchored to develop *f*yd.
4. The vertical reinforcement in each boundary element mentioned in (3) should not be less than four bars with 12 mm diameter in the lower storey of the building, or 10mm in all other storeys.
5. When in any storey the length *l*w of the wall is reduced over that of the storey below by more than one-third of the storey height *h*s, this storey should be considered a critical region and the vertical reinforcement in each boundary element of this storey should not be less than four bars with 12 mm diameter and they should be anchored in the storeys above and below.
6. 10.8.3.3(3) and (4) and 10.8.3.4 should be applied.
7. Vertical bars for the verification of the SD limit state in bending with axial force in accordance with (3) should be engaged by hoops or crossties with a diameter of not less than 6 mm and one-third of the vertical bar with smallest diameter, *d*bL, whichever is greater. Hoops and crossties should be at a vertical spacing *s* not greater than 100 mm or 8*d*bL, whichever is less.

## Flat slabs

### Basis of design

NOTE A concrete flat slab is a two-way slab (solid or waffle), with constant thickness or with drop panels, supported directly on columns (with or without capitals) or walls.

1. 10.10 should be applied to cast-in-place flat slabs considered as primary seismic members, in flat slab structures.

NOTE Use of edge beams framing into supports or short slab overhangs improves the seismic behaviour of slab connections to edge and corner supports in terms of punching shear and reduces the flexural reinforcement.

1. Vertical supporting members should be arranged in a regular pattern in two orthogonal horizontal directions.
2. The connection of flat slabs to edge and corner columns should use the full section of the vertical supporting members and should have edge beams.
3. The thickness of each flat slab should not be smaller than 3,5 % of its largest span.
4. In waffle flat slabs, a solid area with constant thickness should be adopted up to a distance from the column or wall support not less than 3 times the effective depth *d*v of the slab.
5. Openings in flat slabs should be avoided at a distance from the column or wall support less than the shear-resisting effective depth *d*v of the slab or the drop panel. In the presence of openings at a distance from the column or wall support up to 5,5 times the shear-resisting effective depth *d*v of the slab or the drop panel, the reduction of the control perimeter according with prEN 1992-1-1:2021, 8.4.2(3) and Figure 8.19, should be adopted.
6. In a model of a flat-slab frame as a moment resisting frame in 3D for linear or nonlinear analysis, “equivalent beams” which are prismatic elements connecting adjacent columns may be used; their properties should satisfy a) to e):
7. a theoretical span equal to the axial distance between the columns’ centroids;
8. a cross-sectional depth equal to the slab thickness;
9. a concrete strength equal to that of the slab;
10. a half-width on each side of the axis joining the columns centroids equal to the smallest of 1) to 3): 1) the distance to the slab’s edge; 2) one-half the axial distance to the nearest parallel “equivalent beam”; 3) the sum of one-half the cross-sectional dimension of the column transverse to the axis of the equivalent beam, and one-fifth of the clear span between the columns;
11. top and bottom reinforcement ratios, reinforcement grade and cover to reinforcement within the width of the “equivalent beam” equal to those in the support strip between the connected columns (see Figure 10.11).
12. If the entire flat slab is simulated with plate models, the elastic flexural and shear stiffness properties of the flat slab used in the analysis should take into account cracking.
13. (8) may be satisfied by taking elastic flexural and shear stiffness of the flat slab equal to one-fourth of their uncracked stiffness.

### SD limit state verifications and detailing

1. Flexural reinforcement should be concentrated over the supporting columns and walls and in slab strips between adjacent supports called support strips.
2. Support strip width (Figure 10.11) should be taken as the maximum of a) and b):
3. the sum of 25 % of the panel width on each side of the column or wall centreline;
4. the dimension of the supporting vertical member perpendicular to the support strip.
5. The slab top and bottom flexural reinforcement to resist the full hogging and sagging moments in the slab at the ULS in the seismic design situation should be placed within the support strip.
6. A minimum flexural reinforcement should be provided in the support strips, according to a) to c):
7. over the whole span, top reinforcement of not less than one-fourth of the top reinforcement at the supports;
8. over the whole span, bottom reinforcement of not less than one-third of the top reinforcement at the supports;
9. at the face of the column or wall, bottom reinforcement of not less than the mid-span bottom reinforcement.
10. In the slab strip between two support strips (Figure 10.11), called “middle strip”, bottom flexural reinforcement over its intersection with the support strip in the orthogonal direction of not less than half of its mid-span bottom reinforcement should be provided.

Key

|  |  |
| --- | --- |
| *A* | support strip |
| *B* | middle strip |
| *C* | internal support |
| *D* | external support |

Figure 10. 11— Support-strips, middle strips and slab effective width for one direction

1. In each direction, the slab top and bottom flexural reinforcement corresponding to the moments transferred from the slab to the supports should be placed within the effective slab width *b*ef defined in (7).
2. The effective slab width *b*ef should be taken as the width of the column, column capital or wall plus, on each side of the support, the column or wall depth or the slab overhang length *a*, but not exceeding on each side 1,5 times the slab or drop panel shear-resisting effective depth *d*v (see Figure 10.11 for *b*ef).
3. For internal slab-support connections, the slab top and bottom flexural reinforcement corresponding to the fraction of the hogging and sagging design moment in the slab transferred by flexure to the support in the seismic design situation should be placed within the effective slab width *b*ef. This reinforcement should not be less than 50 % of the reinforcement in (3).
4. For external slab-support connections, the slab top and bottom flexural reinforcement required to resist the total hogging and sagging design moments in the slab in the seismic design situation should be placed within the effective slab width *b*ef and should be anchored at the support according to 10.11.2.
5. In addition to (3) to (9), the slab bottom reinforcement over the column or wall support width should satisfy a) and b) in each direction (Figure 10.12):

NOTE This reinforcement is termed “integrity reinforcement”.

1. a minimum of four continuous bars with diameter not greater than 0,12*d*v should be placed over the support width with an anchor length from the support face not smaller than *d*v+*l*b, where *d*v is the slab or drop panel shear-resisting effective depth at the support face, and *l*b the anchorage length;
2. the vertical force resistance it provides, *V*Rd,int, from Formula (10.17), should be greater than the vertical load transferred to the support.

(10.17)

where

|  |  |
| --- | --- |
| *A*s,int | is the integrity reinforcement’s cross-sectional area; |
| *f*yd | is the design yield strength of the steel; |
| *k* | is the characteristic tensile strength to yield strength ratio of the reinforcement (*f*t/*f*y)k defined in prEN 1992-1-1:2021, 5.2.2(1), Table 5.5; |
| *α*ult | is the angle between the integrity bars and the slab plane at failure (*α*ult = 20° for reinforcing steel of ductility class B; *α*ult = 25° for reinforcing steel of ductility class C); |
| *f*ck | is the characteristic compressive resistance of concrete; |
| *γ*c | is the partial factor for concrete, according to 10.3.4(1); |
| *b*int | is the control perimeter activated by the integrity reinforcement from Formula (10.18); |

(10.18)

where

|  |  |
| --- | --- |
| *s*int | is the width of the integrity bars group at each support face (Figure 10.12). |

Figure 10.12 — Integrity reinforcement

1. Punching shear resistance should be verified in all slab-column, slab-wall connections and drop panel borders, based on the control perimeter *b*0,5 defined in prEN 1992-1-1:2021, 8.4.2(2) and (3), where the slab or drop panel shear effective depth *d*v should be determined according to prEN 1992-1-1:2021, 8.4.2(1).
2. Punching shear should be verified considering the eccentricity of the shear forces, according to prEN 1992-1-1:2021, 8.4.2(6) to (8) using for all types of support the refined value of eccentricity *e*b of the shear force for internal columns as given in prEN 1992-1-1:2021, 8.4.2(6), Table 8.3. The components *e*b,x and *e*b,y of the eccentricity should be calculated from capacity design moments, determined in each one of two orthogonal directions of the supporting column as the smaller of a) and b):
3. The sum of design values of moment resistances of the supporting column at the interfaces with the slab above and below its joint with the column;
4. The slab’s flexural resistance at a support section normal to the plane of bending calculated according to 1) or 2): 1) 80 % of the sum of design values of sagging and hogging moment resistances of the full width of the slab tributary to an interior joint with the column or 75 % of the hogging moment resistance at edge joints in the direction of bending; 2) at interior columns, as 1.25-times the sum of design values of sagging and hogging moment resistances of the support sections of “equivalent beams” per 10.10.1(7) framing into opposite sides of the column in the plane of bending; at edge columns, the design value of hogging moment resistance of the support section of the equivalent beam per 10.10.1(7) connected to the column.
5. prEN 1992-1-1:2021, 8.4, as modified in (12) should be used for the verification of flat slabs in punching shear at SD. Punching shear reinforcement may be omitted when the maximum design punching shear stress *τ*Ed in the control perimeter in the seismic design situation satisfies Formula (10.19).
6. At joints with columns surrounded by others in all directions, the punching shear resistance in prEN 1992-1-1:2021, 8.4.3, Formula (8.94), may be multiplied by *η*pm as given in Formula (10.19a):

(10.19)

1. Alternatively, to (13) and prEN 1992-1-1:2021, 8.4, as modified in (12) to (14), the flat slab may be verified in punching shear at SD according to the modifications to prEN 1992-1-1:2021, 8.4 given in (16) and (17).
2. The punching shear resistance in slabs without shear reinforcement should be calculated with Formula (10.20), instead of prEN 1992-1-1:2021, Formula (8.94):

(10.20)

where the symbols are as defined in prEN 1992-1-1:2021, 8.4.3(1), with the reinforcement ratio referring to the tension (i.e., top) face of the slab.

1. For slabs with shear reinforcement, the constant values given by Formulae (10.21) and (10.22) should be used in prEN 1992-1-1:2021, 8.4.4, Formulas (8.104) and (8.112):

(10.21)

(10.22)

1. Formula (10.23) should be used instead of prEN 1992-1-1:2021, 8.4.4(5), Formula (8.102):

(10.23)

where *η*sys is given by prEN 1992-1-1:2021, 8.4.4(3), Formula (8.110) and (8.111).

NOTE Reference is made only to prEN 1992-1-1:2021, 8.4, but (14) and the alternative approach in (15), (16) are a simplifying adaptation of the provisions of prEN 1992-1-1:2021, Annex I, to punching shear under reversed moments.

1. Punching shear reinforcement detailing should follow prEN 1992-1-1:2021, 12.5.1(1) and (2), but bent-up bars or other type of inclined reinforcement should not be used.
2. The radial spacing *s* of the punching shear reinforcement link legs should not exceed 0,5*d*v, and the first line of link legs should be placed no farther than 0,25*d*v from the support face (Figure 10.13).

Figure 10.13 — Punching shear reinforcement within critical sections of flat slabs

## Provisions for anchorages and laps

### General

1. prEN 1992-1-1:2021, 11.4 and 11.5, for the detailing of reinforcement should be applied for DC2 and DC3, complemented with (2), 10.11.2 and 10.11.3.
2. For the transverse reinforcement in beams, columns, beam-column joints, walls or slabs, closed stirrups with 135° hooks and extensions of 10-diametre length should be used. Crossties should also be closed with 135° hooks and extensions of 10-diametre length.
3. The anchorage length of beam or column bars anchored within beam-column joints should be measured from a point on the bar at a distance 5*d*bL inside the face of the joint (see Figure 10.14a an example of a beam bar anchorage).

NOTE (3) takes into account the yield penetration due to cyclic post-elastic deformations.

1. When post-installed bars are used, 10.11 should be applied and they should satisfy a) and b):
2. they should not be used in critical zones;
3. the anchoring system should comply with prEN 1992-1-1:2021, Annex C, C.8.

### Anchorage of reinforcement in beams

1. The part of beam longitudinal reinforcement bent in beam-column joints for anchorage should always be placed inside the corresponding column hoops.
2. To prevent bond failure, the diameter *d*bL of beam longitudinal bars crossing beam-column joints should be limited in accordance with a) or b), as appropriate:
3. for interior beam-column joints, Formula (10.24):

(10.24)

1. for exterior beam-column joints, Formula (10.25):

(10.25)

where

|  |  |
| --- | --- |
| *h*c | is the cross-sectional dimension of the column parallel to the bars; |
| *f*ctm | is the mean value of the tensile strength of concrete; |
| *f*yd | is the design yield strength of steel reinforcement; |
| *ν*d | = *N*Ed/(*f*cd *A*c) is the normalized axial force in the column above the joint in the seismic design situation; |
| *k*D | is the factor reflecting the ductility class equal to 1/2 for DC2 and to 2/3 for DC3; |
| *ρ’*l | is the compression steel ratio of the beam bars passing through the joint; |
| *ρ*l,max | is the maximum allowed tension steel ratio (see 10.5.4.2(5)); |
| *γ*Rd | accounts for overstrength due to strain-hardening and may be taken equal to 1,1 for DC2 and 1,2 for DC3. |

1. The limitations in (2) may be disregarded for diagonal bars crossing joints.
2. Alternatively to the explicit calculation of *d*bL given by Formulas (10.20) and (10.21), the values in Table 10.7 may be used, for reinforcing steel of ductility class B or C, when a) and b) are satisfied:
3. if the normalized design axial force in the column is not less than 0,15;
4. at interior beam-column joints, if the ratio of beam compression bars crossing the joint is less than 1 %.

Table 10.7 — Maximum diameter of beam longitudinal bars passing through beam-column joints

|  |  |  |  |
| --- | --- | --- | --- |
| **Interior beam-column joints** | **Concrete grade** | **Steel grade** | |
| **B400** | **B500** |
| **C20-C25** | DC2: *d*bL ≤ 4,0 % *h*c  DC3: *d*bL ≤ 3,0 % *h*c | DC2: *d*bL ≤ 3,0 % *h*c  DC3: *d*bL ≤ 2,0 % *h*c |
| **C30-C45** | DC2: *d*bL ≤ 6,0 % *h*c  DC3: *d*bL ≤ 4,5 % *h*c | DC2: *d*bL ≤ 5,0 % *h*c  DC3: *d*bL ≤ 3,5 % *h*c |
| **C50-C90** | DC2: *d*bL ≤ 8,5 % *h*c  DC3: *d*bL ≤ 7,0 % *h*c | DC2: *d*bL ≤ 7,0 % *h*c  DC3: *d*bL ≤ 5,5 % *h*c |
| **Exterior beam-column joints** | **C20-C25** | DC2 or DC3: *d*bL ≤ 4,5 % *h*c | DC2 or DC3: *d*bL ≤ 3,5 % *h*c |
| **C30-C45** | DC2 or DC3: *d*bL ≤ 6,5 % *h*c | DC2 or DC3: *d*bL ≤ 5,5 % *h*c |
| **C50-C90** | DC2 or DC3: *d*bL ≤ 9,5 % *h*c | DC2 or DC3: *d*bL ≤ 7,5 % *h*c |

1. If (2) cannot be satisfied in exterior beam-column joints because the depth, *h*c, of the column parallel to the bars is too shallow, one of the additional measures in a) to c) may be taken, to ensure anchorage of the longitudinal reinforcement of beams:
2. the beam or slab may be extended horizontally in the form of exterior stubs (Figure 10.14a);
3. headed bars according to prEN 1992-1-1:2021, 11.4.7, may be used (Figure 10.14b);
4. bends according to prEN 1992-1-1:2021, 11.4.4, and transverse reinforcement placed tightly inside the bend of the bars may be added (Figure 10.14c).

Key

|  |  |
| --- | --- |
| A | anchor plate |
| B | hoops around column bars |

Figure 10.14 — Additional measures for anchorage in exterior beam-column joints

1. Top or bottom bars crossing interior joints should be anchored in the members framing into the joint beyond a distance not less than *l*cr (length of critical region, see 10.5.4.2(1)) from the face of the joint.

### Laps and mechanical couplers

1. There should not be laps in the critical regions of beams.
2. There should not be any lap-splicing by welding within the critical regions of structural members. For lap-splicing by welding outside of the critical regions within a distance of twice the member depth from the critical regions of primary seismic columns and beams, the design resistance of the welded splices should not be smaller than 1,25 times the design yield strength of the bars they connect.
3. There should not be splicing by mechanical couplers in the critical regions of primary seismic columns and beams and within a distance of twice the member depth from the critical regions, unless (4) is satisfied.
4. There may be splicing by mechanical couplers in the critical regions of primary seismic walls or within a distance of twice the member depth from the critical regions of primary seismic columns and beams, if the devices are qualified by testing for the seismic design situation and if the yielding and the failure of the lapped bars are obtained before those of the coupler in all tests.

NOTE 1 (4) is a capacity design rule to assure that the bar will yield first and that no failure occurs in the splicing system while the bar elongates in the plastic domain.

NOTE 2 prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. The transverse reinforcement to be provided within the lap length should be calculated in accordance with prEN 1992-1-1:2021, 11.5. In addition, a) to c) should be satisfied:
2. if the anchored and the continuing bar are arranged in a plane parallel to the transverse reinforcement, the total area of all lapped bars, Σ*a*sL, should be used in the calculation of the transverse reinforcement;
3. if the anchored and the continuing bar are arranged within a plane normal to the transverse reinforcement, the area of transverse reinforcement should be calculated on the basis of the area of the greater lapped longitudinal bar, *A*sL;
4. the spacing, *s*lap, of the transverse reinforcement in the lap zone (in mm) should not exceed the value given by Formula (10.26).

(10.26)

where *h* is the minimum cross-sectional dimension (*s* and *h* in mm).

1. The required area of transverse reinforcement *A*st within the lap zone of the longitudinal reinforcement of columns spliced at the same location (as defined in prEN 1992-1-1:2021, 11.5), or of the longitudinal reinforcement of boundary elements in walls, may be calculated with Formula (10.27).

(10.27)

where

|  |  |
| --- | --- |
| *A*st | is the area of one leg of the transverse reinforcement; |
| *d*bL | is the diameter of the lapped longitudinal bars; |
| *s* | is the spacing of the transverse reinforcement; |
| *f*yd | is the design yield strength of the longitudinal reinforcement; |
| *f*ywd | is the design yield strength of the transverse reinforcement. |

## Provisions for concrete diaphragms

### Cast in place diaphragms

1. A solid reinforced concrete slab may be considered as a rigid diaphragm, as defined in 5.1.3(6), if its thickness is not less than 70 mm and if it is reinforced in both horizontal directions with at least the minimum reinforcement specified in prEN 1992-1-1:2021, 12.4.1(1).

NOTE A solid slab is defined prEN 1992-1-1:2021, 3.1.79, as a slab without voids or ribs within its full depth.

1. A cast-in-place topping on a precast floor or roof structure may be considered as a rigid diaphragm, if a) to d) are fulfilled:
2. the thickness of the topping layer is not less than 40 mm for span between supports not longer than 8 m, or 50 mm for longer spans;
3. its mesh reinforcement is connected to the beams or walls supporting the diaphragm;
4. it is designed to provide alone the required diaphragm stiffness and resistance, according to 6.2.8;
5. it is cast over a clean, rough substrate, or connected to it through shear connectors.
6. Design action effects should take into account overstrength according to 6.2.8.
7. The design resistances should be derived in accordance with prEN 1992-1-1:2021, 12.4, 12.5, 12.9.2 and 13.6.

### Precast concrete diaphragms

1. 10.14 should be applied in addition to 10.12.1 and prEN 1992-1-1:2021, 13.6.2.

## Prestressed concrete

1. Prestressed concrete beams and columns should be designed according to 10.5 and 10.6, adding to the calculated action effects the hyperstatic axial force and bending moment generated by prestress.
2. In the design of prestressed concrete beams and columns, the vertical component of the seismic action should be taken into account according to 4.2(2).
3. Bonded prestressing tendons should not be used in critical regions of primary seismic members.
4. Anchorage of the prestressing tendons (bonded or unbonded) should not be located in the critical regions of primary seismic members.

## Precast concrete structures

### Structural types and behaviour factor *q*

1. 10.14 should be applied to all structural types defined in 10.4.1, to wall panel structures and to cell structures (precast monolithic room cell structures). Structures may be made partly or fully of precast members.
2. Beam to column joints should be considered as strong partial strength joints for moment resisting frames in Table 10.8 if they are characterized by a design moment of resistance *M*Rd greater than *M*Rd,inf/2, where *M*Rd,inf is the moment resistance at the bottom of the connected column; beam to column joints should be considered as weak partial strength joints for moment resisting frames in Table 10.8 if they are characterized by a design moment of resistance *M*Rd smaller than *M*Rd,inf/2.

NOTE Weak partial strength joints developing negligible bending moments, like for instance dowel type joints, are called nominally pinned joints.

1. The rotational capacity *θ*tot of strong and weak partial strength joints for moment resisting frames should not be smaller than 25 mrad in DC2 and 35 mrad in DC3. The rotational capacity *θ*tot should be established by tests.

NOTE The rotational capacity *θ*tot can be defined experimentally using prCEN/TS 1998-1-101 loading protocol and acceptance criteria.

1. The values of the behaviour factor components *q*R, *q*D and *q* defined in Table 10.8 should be applied.

Table 10.8 — Default values of the behaviour factors of precast concrete structures

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| **Structural type** | | ***q*R** | ***q*D** | | ***q = q*R*q*S*q*D** | |
|  | **DC2** | **DC3** | **DC2** | **DC3** |
| Moment resisting frame or moment resisting frame-equivalent dual structures | Multi-storey, multi-bay moment resisting frames or moment resisting frame-equivalent dual structures with beams with strong partial strength joints or rigid joints without cladding or with isostatic cladding | 1,3 | 1,3 | 2,0 | 2,5 | 3,9 |
| Multi-storey, one-bay moment resisting frames with strong partial strength joints or rigid joints without cladding or with isostatic cladding | 1,2 | 2,3 | 3,6 |
| One-storey moment resisting frames with strong partial strength joints or rigid joints without cladding or with isostatic cladding | 1,1 | 2,1 | 3,3 |
| One-storey one bay or multi-bay, moment resisting frames with beams with weak joints without cladding or with isostatic cladding and *ν*d ≤ 0,3 | 1,1 | 1,3 | 2,1 | 2,1 | 3,0 |
| Multi-storey, one bay or multi-bay, moment resisting frames with beams with weak joints without cladding or with isostatic cladding and *ν*d ≤ 0,3 | 1,1 | NA\* | 2,1 | NA\* | 2,0 |
| One-storey one bay or multi-bay, or multi-storey, one bay or multi-bay moment resisting frames with beams with weak joints without cladding or with isostatic cladding and *ν*d > 0,3 | NA\*\* | | | | |
| Wall- or wall-equivalent dual structures | Wall-equivalent dual structures with strong partial strength joints or rigid joints with or without cladding | 1,2 | 1,3 | 2,0 | 2,3 | 3,6 |
| Coupled wall structures with strong partial strength joints or rigid joints with or without cladding | 1,2 | 1,4 | 2,5 | 3,6 |
| Uncoupled wall structure with only two uncoupled walls per horizontal direction, with or without cladding and other uncoupled wall structure, with or without cladding | 1,0 | 1,3 | 2,0 | 3,0 |
| Multi-storey moment resisting frame with beams simply supported or with weak partial strength connections and with integrated cladding | 1,2 | 1,3 | NA\* | 2,3 | NA\* |
| Multi-storey moment resisting frame with beams with strong partial strength connections and with integrated cladding | 1,2 | 1,4 | NA\* | 2,5 | NA\* |
| Large walls structures and cell structures (factor *k*w given by Formula (10.1)) | | - | - | - | 3*kw* | 3*kw* |
| NA\*: Not Applicable NA\*\*: Inverted pendulum – DC1 design only | | | | | | |

### Rules applicable to all structural types and to DC1, DC2 and DC3

1. Unless otherwise specified, 10.1 to 10.13 and 10.15 should be applied, complemented with 10.14.
2. The models for analysis should comply with a) to i):
3. the eccentricities between members should be modelled;
4. the joints between the members should be represented with their degrees of freedom in the different planes;
5. the joints should be characterized by parameters related to the type of analysis as defined in prEN 1998-1-1:2022, 6.2;
6. the data on joints required for the analysis and for verifications should be based on experimental evidence or on models based on such experimental evidence;
7. experimental evidence may be based on existing data; if such data is not available, they should be established by experiments;

NOTE prCEN/TS 1998-1-101 gives information concerning cyclic testing of components.

1. the in-plane flexibility of the floors and roof precast members, the flexibility of the joints of floors and roof members to the beams and the flexibility of peripheral beams in the horizontal plane should be taken into account in the model;
2. the effect of rocking panels attached to the main structure by means of isostatic connection system and of the interaction between rocking panels on the response of the whole building should be taken into account;

NOTE Isostatic connection systems are defined in 3.1.16.

1. if the diaphragm’s horizontal displacements in the seismic design situation calculated considering the in-plane flexibility of the diaphragm nowhere exceed by more than 10 % those found with a rigid diaphragm assumption, then the diaphragm may be taken as rigid;
2. diaphragms realized by reinforced concrete topping cast in place over precast concrete members and complying with 10.12 a, c), d) and with topping reinforcement properly connected to the vertical resisting elements may be considered as rigid.
3. If diaphragms do not comply with the limit of deformability defined in (2)h) or with (2)i), the model of the structure should be 3D and the method of analysis should be either modal response spectrum analysis or pushover analysis or dynamic non-linear analysis.
4. The method of analysis of multi storey precast structures should be either modal response spectrum analysis or pushover analysis or response-history analysis.
5. Shear resistance at cast in place to precast interface and at precast-to-precast interfaces should be verified using prEN 1992-1-1:2021, 8.2.6.
6. In (5), the calculation of the compressive stress *σ*N used in prEN 1992-1-1:2021, Formulae (8.76) and (8.77), should take into account the reduction of *σ*N due to the vertical component of the design seismic action as given in prEN 1998-1-1:2022, 6.4.1(7).

NOTE Rules are given for the design of dowel joints, for the design of joints between precast concrete structural members in ISO 20987 and for the design of joints of concrete claddings to concrete structures in ISO 22502.

1. Data for verification of action effects calculated through non-linear pushover or response-history analysis should be as given in prEN 1998-1-1 for concrete joint zones and for dowel joints.

### Precast moment resisting frames

#### Rules applicable to DC1, DC2 and DC3

1. Precast beams and columns of precast moment resisting frames may be connected by cast in place concrete zones or by other types of joints.
2. Cast in place concrete joints should conform to the detailing for local ductility and the relevant requirements of 10.1 to 10.13. Other types of joints may be strong or weak partial strength joints as defined in 10.14.1.
3. In precast moment resisting frames with weak partial strength or nominally pinned beam-to-column joints or beam-to-floor element joints, columns should be fully fixed against translation and rotation at their base.
4. The shear resistance of structural members in the vicinity of joints and of all types of joints should be verified for 1,5*V*Ed, where *V*Ed is the shear action effect calculated at the joints in the seismic design situation.

#### Verification in DC1

1. The design shear *V*Ed in weak partial strength or nominally pinned connections at supports of beams and at supports of floor members should not be smaller than *ΔM*Ed/*h*s,cl, where *ΔM*Ed = *M*Ed,i – *M*Ed,i+1 is the variation of the moment in the column adjacent to the beam support at the storey level *i* considered, in the seismic design situation, and *h*s,cl is the clear storey height.

#### Verification in DC2 and DC3

1. 10.5 for beams and 10.6 for columns should be applied.
2. In case of cantilever columns, 10.6(5) should not be applied and the minimum cross-sectional dimension of primary seismic columns should not be smaller than either a) or b):
3. If *θ* ≤ 0,05: 200 mm, where *θ* is defined in 6.2.4(1).
4. If *θ* > 0,05: the maximum of: (i) 1/20 of the height of the column; (ii) 200 mm for DC2 or 250 mm for DC3.
5. In moment resisting frames with beams with weak joints and without cladding or with isostatic cladding, only the base of columns should be taken as critical region, with a length *l*cr as given by Formula (10.28):

*l*cr = max {2 bmax; 1,00 m} (10.28)

where

*b*max is the greatest cross-sectional dimension of the column.

1. Dissipative zones may be in the structural members or in the joints.
2. If dissipative zones are in the structural members, a), and b) should be applied:
3. The design bending moment and shear in the non-dissipative structural members, their joints and the joints of the members where dissipative zones are located should be those established by application of the capacity design rules in 10.5.3 and 10.6.2, multiplied by a factor *γ*Rd = 1,2.

NOTE Non-dissipative joints in the vicinity of dissipative zones are called overstrength joints (see Figure 10.15).

1. 10.11.3 should be satisfied.

Key

|  |  |
| --- | --- |
| A | coupler |
| B | critical section |
| C | plastic hinge zone |
| D | coupler length |

Figure10.15 — Examples of dissipative zones in the RC beam or column and adjacent joints: (a) and (b) beam to column joints; (c) column to footing joints

1. If dissipative zones are in the joints, a) to c) should be applied:
2. The design bending moment and shear outside the dissipative joints should be those established by application of the capacity design rules in 10.5.3 and 10.6.2 multiplied by a factor *γ*Rd = 1,25.
3. The yield strength of elements of the joints that are not designed to yield should be greater than 1,2 times the yield strength of the yielding portion of the joints.
4. (5)b should be applied.
5. The design shear *V*Ed in weak partial strength or nominally pinned connections at supports of beams and at supports of floor members should not be smaller than *γ*Rd *M*Rd*/h*s,cl where *M*Rd is the bending resistance of the column adjacent to the beam support at the storey level considered, *h*s,cl is the clear storey height and *γ*Rd = 1,15 in DC2 and 1,25 in DC3.

### Precast walls

#### Rules applicable to DC1, DC2 and DC3

1. Precast walls, considered as those made of precast concrete panels connected along their edges, should be designed to behave as cast in place walls.

#### Verification in DC2 and DC3

1. Precast walls should be designed as cast in place ductile walls or as large walls and comply respectively with 10.8.2 or 10.9.2 in the determination of the design action effects, and with 10.8.3.1 or 10.9.3.1 and 10.9.3.2 in the determination of the wall resistance in bending and shear and with 10.8.3.4 for tying systems.
2. Boundary elements should comply with the provisions on concrete confinement of 10.8.3.2 for DC2 ductile walls and 10.9.3.3 for large RC walls.

NOTE The boundary elements are the edge regions of the overall cross-section of precast walls.

1. Precast panels should be reinforced by two layers of mesh with pitch not greater than 200 mm and bar diameter not smaller than 8 mm. Perimeter reinforcement by two longitudinal bars of diameter at least 12 mm and edge links of diameter not smaller than 8 mm at 200 mm spacing should be placed.
2. Splices of vertical bars between adjacent panels may be lapped, welded or with mechanical splices.
3. The resistance of mechanical or welded splices should not be smaller than 1,25 times the yield strength of the bars that they connect. The length of straight lap splices should be at least 1,25 times the straight lap length of bars in cast-in-place concrete.
4. Welded splices of vertical reinforcement should be within recesses (Figure 10.16).

Key

|  |  |
| --- | --- |
| A | lap-welding of bars |

Figure 10.16 — Tensile reinforcement possibly needed at the edge of walls (Elevation view; dimensions are indicative)

1. The shear resistance of cast in place concrete joints between panels and of concrete along joints between panels should be verified for 1,5*V*Ed, where *V*Ed is the shear action effect at the joints in the seismic design situation.
2. The horizontal reinforcement geometrical ratio across vertical joints of panels should be between 0,2 % and 2 %.
3. Horizontal reinforcement may be distributed in three bands at the top, middle and bottom of the vertical joint of panels.
4. Continuity of horizontal reinforcement across vertical panel-to-panel joints should be ensured.

NOTE Horizontal continuity can be obtained by using loops of the horizontal reinforcement with a vertical bar placed in the loop (Figure 10.17a) or by welding of horizontal bars across the joint (Figure 10.17b).

1. The vertical reinforcement ratio relative to the area of grout in vertical joints should not be less than 1 %.

Key

|  |  |
| --- | --- |
| A | reinforcement protruding across joint |
| B | vertical reinforcement along joint |
| C | trace of indentations in concrete panel |
| D | grout filling between panels |

Figure 10.17 — Plan view. Cross-section of vertical joints between precast large panels: (a) joint with two accessible faces; (b) joint with one accessible face

#### Verification in DC3

1. Precast walls should be designed as cast in place ductile walls and comply with 10.8.3.2.
2. Boundary elements should comply with the requirements on confinement in 10.8.3.2.
3. A square part of the section at each edge of a precast panel, with side equal to the wall panel thickness, *b*w, should be confined according to 10.6.3.2.
4. 10.14.4.2(4) to (6) on splices should be applied.
5. The total tension force in the wall should be resisted by vertical reinforcement placed in the zone of the wall panel under tension.
6. Shear sliding should be verified according to prEN 1998-1-1:2022, 7.2.3(8).
7. The shear resistance of non-dissipative components should be verified for 1,3*V*Ed, where *V*Ed is the shear action effect at the joint in the seismic design situation as defined in 10.8.3.1(4) and (5) for cast in place similar components designed to DC3.
8. The horizontal reinforcement ratio across vertical joints of panels should not be smaller than 0,10 % in fully compressed joints and 0,25 % in partly compressed joints; it should not be more than 2 %.

### Precast floors and roof diaphragms. Rules for ductility classes DC1, DC2 and DC3

1. Precast concrete floors and roof diaphragms should not be dissipative components of the structure.

NOTE Possible ways to achieve a diaphragm effect with precast floor members are: by a concrete topping cast in place over precast members and connected to peripheral tie-beams: by connecting wide precast floor members to peripheral beams in such a way that the precast members act as fixed beams in a horizontal frame; by floor to floor joints between precast beam or slab members which are connected to peripheral beams; by precast slabs or beams simply connected to peripheral moment resisting frames.

1. Rigid diaphragms or flexible diaphragms may be used.

## Design and detailing of foundations

1. The foundation members should be designed according to prEN 1998-5:2022, Clause 9.

# Specific rules for steel buildings

## General

1. Clause 11 should be applied to the design and the verification of steel buildings, in seismic regions.
2. Clause 11 should be applied as a complement to EN 1993-1-1.

## Basis of Design

### Ductility classes

1. Earthquake resistant steel buildings should be designed in one of the Ductility Classes introduced in prEN 1998-1-1:2022, 4.5.2(3), and 4.5.2(7) (see Table 11.1), according to their dissipation capacity, i.e. DC1 for low-dissipative structural behaviour and DC2 or DC3 for dissipative structural behaviour.

Table 11.1 — Structural ductility classes and upper limit reference values   
of the behaviour factors

|  |  |
| --- | --- |
| **Structural ductility class** | **Range of the reference values of the behaviour factor *q*** |
| DC1 | *q* =1,5 |
| DC2 | 1,5  *q*  3,5 |
| DC3 | limited by the values of Table 11.2 |

1. In DC2 and DC3, the capability of parts of the structure (dissipative zones) to resist seismic action through inelastic behaviour may be taken into account with upper limit values of *qD* and *qR* depending on the Ductility Class and the structural type (see 11.4). 11.3 to 11.6 and 11.8 to 11.18 should be satisfied.
2. In DC1, 11.7 should be applied.
3. Design to ductility class DC1 or DC2 or DC3 should satisfy the limitations of the seismic action index *S*δ as given in 11.4.3 and the limitations of drift as given in 11.6.4.

### Safety verifications

1. Partial factors **M,i should comply with EN 1993-1-1:2022, 8.1(1).
2. In the capacity design verifications in 11.8 to 11.18, the resistance at yield of dissipative members should be calculated considering the randomness material factor *ω*rm as given in prEN 1998-1-1:2022, Table 7.1.

## Materials

1. The material properties, such as yield strength and toughness, in the dissipative zones shall be such that plastic deformations occur where they are intended to in the design.
2. If the design of dissipative zones is made using a single steel grade with a specified upper value *f*y,max smaller than the nominal value *f*y of the steel grades specified for non-dissipative zones, factor, *ω*rm may be taken as being 1,00 in the verifications of non-dissipative zones defined in 11.8 to 11.18.

NOTE The actual value of *ω*rm is greater than 1,00. (2) is a simplification provided that the material properties of dissipative and non-dissipative zones are consistently checked.

1. For dissipative zones, the steel grade to be used should be specified and noted on the drawings. A higher grade should not be supplied for these zones (see 11.19 (3)).
2. The steel categories for the base material and for the welding consumables should be taken as given in prEN 1993-1-10:2023, 4, for a stress level σEd = 0,75 *f*y(t) and the quasi-permanent value of service temperature.

NOTE 1 (4) defines steel with toughness and thickness adequate for yielded sections.

NOTE 2 The quasi-permanent value of the service temperature is defined in EN 1990:2023, Annex A, A.1.5.3.

1. The required toughness of steel and welds and the lowest service temperature adopted in combination with the seismic action should be defined.
2. For primary Execution Class EXC1 members, the minimum toughness requirement should be JR. For primary EXC2 members, the minimum toughness requirement should be J0. For primary EXC3 members, the minimum toughness requirement should be J2.

NOTE 1 Required Execution Class EXC are related to design conditions as given in 11.19.

NOTE 2 The requirements in (4), (5) and (6) complement the rules in prEN 1993-1-10.

1. In bolted connections of primary seismic members, high strength bolts of grade 8.8 or 10.9 should be used.

## Structural types, behaviour factors and limits of seismic action

### Structural types

1. Steel buildings designed to be dissipative structures should be assigned to one of the structural types in a) to i) according to the behaviour of their primary resisting structure under seismic actions.
2. Moment resisting frames, in which horizontal forces are mainly resisted by members acting in an essentially flexural manner (Figure 11.1; see the design rules in 11.9);
3. Frames with concentric bracings, in which horizontal forces are mainly resisted by members subjected to axial forces (Figures 11.2 and 11.3; see the design rules in 11.10);

NOTE Frames with concentric X bracings or split X bracings can be designed considering that only the tension diagonals resist the seismic action or that both tension and compression diagonals contribute to resist the seismic action – see Figure 11.2, Figure 11.3 and 11.10.2. The “tension only” design is an approximation in comparison to design considering both diagonals contributions.

1. Frames with eccentric bracings, in which horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear (Figure 11.4; see the design rules in 11.11);
2. Frames with buckling restrained bracings, in which horizontal forces are mainly resisted by members subjected to axial forces that are active both in tension and in compression and which are prevented from buckling by a restraining mechanism (Figure 11.5; see the design rules in 11.12);
3. Dual frames, in which at least 25 % of the lateral resistance is provided by moment resisting frames; the rest is provided by frames with concentric, eccentric or buckling restrained bracings (Figure 11.6; see the design rules in 11.13);
4. Lightweight steel systems using flat strap bracing or sheathed with steel sheets or wood sheeting or gypsum sheeting, in which horizontal forces are mainly resisted by lightweight members made of cold-formed members covered by prEN 1993-1-3, which are mostly subjected to either a truss force pattern or to a plastic deformation pattern in the connections between the sheeting and its supports (Figure 11.7; see the design rules in 11.14);
5. Inverted pendulum structures in which 50 % or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building member or at the bases of columns (Figure 11.8; see the design rules in 11.15);
6. Structures with concrete cores or concrete walls, in which horizontal forces are mainly resisted by these cores or walls, whose shear resistance at the building base exceeds 75 % of the total shear resistance (Figure 11.9; see the design rules in 11.16.1);
7. Moment resisting frames combined with infills (Figure 11.10; see the design rules in 11.16.2).

Figure 11.1 — Moment resisting frames (dissipative zones in beams and at bottom of columns): (a) portal frame; (b) single-storey MRF; (c) single-span multi-storey MRF; (d) multi-span multi-storey MRF

Figure 11.2 — Frames with concentric bracings where the concept of tension-only diagonals is allowed

Figure 11.3 — Frames with concentric bracings where the concept of tension-compression diagonals is mandatory

Figure 11.4 — Frames with eccentric bracings (dissipative zones in bending or shear links)

Figure 11.5 — Frames with buckling restrained bracings (dissipative zones in tension and compression diagonals)

Figure 11.6 — Dual frames with moment resisting frame combined with either concentric, eccentric or buckling restrained bracing (dissipative zones in both moment and braced frames)

Figure 11.7 — Lightweight steel systems: (a) strap braced walls; (b) shear walls with steel sheet or wood sheathing or gypsum sheathing

Figure 11.8 — Inverted pendulum: (a) dissipative zones at the column base; (b) dissipative zones in columns (*N*Ed,G/*N*pl,Rd ≥ 0,3)

Figure 11.9 — Structures with concrete cores or concrete walls

Figure 11.10 — Moment resisting frame combined with infills

Figure 11.11 — Frame with K bracings

1. Single-storey moment resisting frames in which the primary structure possesses more than one column in each resisting plane should be considered as inverted pendulum structures if *N*Ed,G ≥ 0,3 *N*pl,Rd in all primary columns.

NOTE *N*Ed,G is the axial force due to the non-seismic actions in the seismic design situation.

1. Structural systems which cannot be assigned to one of the structural types in (1) may be used; they should be designed for strength according to 11.7.3.
2. Bracings in which the intersection of the diagonals lies on a column (usually named K bracings, see Figure 11.11) and moment resisting frames made with trussed beams and/or trussed columns belong to the structural systems mentioned in (3).

### Behaviour factors

1. For regular structural systems which can be assigned to one of the structural types in 11.4.1(1), the behaviour factor components *q*D and *q*R and the behaviour factor *q* may be taken with the default values given in Table 11.2, provided that 11.8 to 11.16 are satisfied.

Table 11.2 — Default upper limit values of behaviour factors for steel systems regular in elevation

| **Structural Type** | **Ductility Class** | | | | | |
| --- | --- | --- | --- | --- | --- | --- |
| **DC2** | | | **DC3** | | |
| ***q*D** | ***q*R** | ***q*** | ***q*D** | ***q*R** | ***q*** |
| a) Moment resisting frames (MRFs) |  |  |  |  |  |  |
| Portal frames and single-storey MRFs with class 3 and 4 cross sections | 1,3 | 1 | 2 | - | - | - |
| Portal frames and single-storey MRFs with class 1 and 2 cross sections | 1,8 | 1,1 | 3 | 3,3 | 1,1 | 5,5 |
| Multi-storey MRFs | 1,8 | 1,3 | 3,5 | 3,3 | 1,3 | 6,5 |
| b) Frames with concentric bracings | 1,7 | 1 | 2,5 | 2,4 | 1,1 | 4 |
| Diagonal bracings |
| V-bracings |
| X-bracings on either single or two-storey |
| c) Frames with eccentric bracings | 1,8 | 1,1 | 3 | 3,1 | 1,3 | 6 |
| d) Frames with buckling restrained braces |  |  |  | 3,3 | 1,2 | 6 |
| e) Dual frames |  |  |  |  |  |  |
| MRFs with concentric bracing | 1,8 | 1,1 | 3 | 2,9 | 1,1 | 4,8 |
| MRFs with eccentric bracing | 2,1 | 1,1 | 3,5 | 3,3 | 1,3 | 6,5 |
| MRFs with buckling restrained braces | - | - | - | 3,3 | 1,3 | 6,5 |
| f) Structures with concrete cores or concrete walls | See Table 10.1:  walls or wall equivalent structures | | | | | |
| g) Lightweight steel frame wall systems |  |  |  |  |  |  |
| with flat strap bracing | 1,3 | 1 | 2 | 1,7 | 1 | 2,5 |
| with steel sheeting | 1,3 | 1 | 2 | 1,7 | 1 | 2,5 |
| with wood sheathing | 1,3 | 1 | 2 | 1,7 | 1 | 2,5 |
| with gypsum sheathing | 1,1 | 1 | 1,7 | 1,3 | 1 | 2 |
| h) Inverted pendulum | 1,3 | 1 | 2 | 1,5 | 1 | 2,3 |
| 1. Moment resisting frames with class 1 & 2 cross sections   with interacting unconnected concrete or masonry infills | 1,4 | 1,1 | 2,3 | 1,4 | 1,1 | 2,3 |
|  |  |  |  |  |  |  |
| with connected reinforced concrete infills | See Table 12.2:  composite structural wall systems | | | | | |
| with non-interacting infills | [see a) MRFs] | | | | | |

NOTE The values of *q*R and *q*D in Table 11.2 are default values in the sense of 5.3.2 and the values of *q* are obtained as the product of the default values of *q*D and *q*R with *q*S equal to 1,5.

1. For buildings non-regular in elevation (see 4.4.4.2(1)), the behaviour factor *q* should be reduced according to 5.3.2(2). For torsionally flexible structures (see 4.4.3(1)) and according to 5.3.2(5), *q* should not be smaller than 1,5*.*
2. prEN 1998-1-1:2022, 4.1(7), may be used in design within the limitations given in 11.6.4(1) f) and 11.7.1 and 11.7.3.

### Limits of seismic action for design to DC1, DC2 and DC3

1. For each structural type in 11.4.1(1), design to a given Ductility Class should not be made above levels of seismic action index *S*δ (see prEN 1998-1-1:2022, 4.1(4)) given in Table 11.3.

Table 11.3 — Limits of seismic action index for design to DC1, DC2 and DC3

|  |  |  |  |
| --- | --- | --- | --- |
| **Structural type** | **Limits of seismic action index**  ***S*δ(m/s2)** | | |
| **DC1** | **DC2** | **DC3** |
| Moment frames | 5,0 | 6,5 | no limit |
| Frames with concentric or eccentric bracings | 5,0 | 6,5 | no limit |
| Buckling-restrained braced frames | - | - | no limit |
| Dual frames (moment frames with bracings) | 5,0 | 7,5 | no limit |
| Steel structure with concrete cores/walls | 5,0 | 7,5 | no limit |
| Lightweight steel frame wall systems | 5,0 | 7,5 | no limit |
| Inverted pendulum | 2,5 | 5,0 | no limit |
| Moment resisting frames with unconnected interacting concrete or masonry infills | 2,5 | 5,0 | no limit |
| Moment resisting frames with non-interacting infills | 5,0 | 6,5 | no limit |
| Moment resisting frames with connected reinforced concrete infills | See 12.4.3 | | |

## Structural analysis

1. The composite action between steel members and concrete should be taken into account by complying with Clause 12.
2. For the analysis of frames in relation with the type of joints between beams and columns, Annex E should be used: Annex E, E.3.3.5, for haunched joints, Annex E, E.3.3.7, for joints with rib stiffeners, Annex E, E.3.3.9, for unstiffened joints, Annex E, E.3.3.11, for joints with reduced beam sections, and Annex E, E.3.3.13, for friction connections.
3. Second-order effects should be verified in accordance with 6.2.4.
4. In DC3 steel structures, if *q*S*<ω*rm *Ω*d where *Ω*d is given in 11.8.5(3), the interstorey drift sensitivity coefficient *θ* should be calculated at all storeys with Formula (11.1).

(11.1)

where

|  |  |
| --- | --- |
| *ω*rm | is the material randomness factor of the steel of dissipative zones (see 11.2.2); |
| *Ω*d | is the design overstrength ratio, as specified in 11.8.5(2) and (3) and in 11.9 to 11.15; |
| *qR* | is given in Table 11.2. |

## Verification to Limit States

### General

1. The verification of structural members to limit states should comply with 6.1, 6.2 and 6.3.

### Verification at Significant Damage limit state in a force-based approach

1. For the force-based approach, the resistance of structural members and connections at Significant Damage should be verified using EN 1993.

### Verification at Significant Damage limit state in a displacement-based approach

1. Resistance of ductile mechanisms should be verified to SD and NC in terms of local generalized deformations *δ* according to prEN 1998-1-1:2022, 6.7.1(3) a) and 6.7.2.
2. For members designed to this standard, deformation criteria should be taken as in prEN 1998‑1‑1:2022, 7.3.
3. For members following the design rules in this standard, total logarithmic standard deviation of the resistance model required to calculate the partial factors for the verification to SD and NC (see prEN 1998-1-1:2022, 6.7.2 and 6.7.3) according to 6.2.3(3) should be taken as given in Table 11.4 and Table 11.5.

NOTE For beams and columns, the partial factors are meant to divide the resistance in terms of chord rotation , for bracings they are applied to axial deformation and for panel zones they apply to shear distortion .

Table 11.4 — Total logarithmic standard deviation **lnR for chord rotation

|  |  |  |
| --- | --- | --- |
| **Member type** | **Section** | ****lnR** |
| Steel beam | Wide flange | 0,39 |
| RBS | 0,40 |
| Steel column | Wide flange | 0,39 |
| Hollow | 0,37 |
| Composite beam | - | 0,39 |
| Composite column | Filled rectangular (interior) | 0,31 |
| Filled rectangular (exterior) | 0,23 |
| Filled circular (interior) | 0,17 |
| Filled circular (exterior) | 0,31 |
| Encased (exterior) | 0,40 |

Table 11.5 — Total logarithmic standard deviation **lnR for axial displacement and panel zone distortion

|  |  |  |
| --- | --- | --- |
| **Member type** |  | ****lnR** |
| Bracing | Tension (Yielding) | 0,21 |
| Compression (Buckling) | 0,27 |
| Buckling-restrained braces | Tension and compression | 0,07 |
| Beam-column web joints (panel zone) | Shear distortion | 0,07 |

1. Values of the partial factors at SD and NC are given in Table 11.6 (NDP) and Table 11.7 (NDP).

Table 11.6 (NDP) — Partial factors for chord-rotation

|  |  |  |  |
| --- | --- | --- | --- |
| **Member type** | **Section** | **SD** | **NC** |
| Steel beam | Wide flange | 1,7 | 2,1 |
| RBS | 1,7 | 2,1 |
| Steel column | Wide flange | 1,7 | 2,1 |
| Hollow | 1,6 | 1,9 |
| Composite beam | - | 1,7 | 2,1 |
| Composite column | Filled rectangular (interior) | 1,5 | 1,75 |
| Filled rectangular (exterior) | 1,4 | 1,6 |
| Filled circular (interior) | 1,2 | 1,3 |
| Filled circular (exterior) | 1,5 | 1,75 |
| Encased (exterior) | 1,7 | 2,1 |

Table 11.7 (NDP) — Partial factors for axial displacement and panel zone distortion

|  |  |  |  |
| --- | --- | --- | --- |
| **Member type** |  | **SD** | **NC** |
| Bracing | Tension (Yielding) | 1,3 | 1,45 |
| Compression (Buckling) | 1,4 | 1,6 |
| Buckling-restrained braces | Tension and compression | 1,05 | 1,1 |
| Beam-column web joints (panel zone) | Shear distortion | 1,05 | 1,1 |

NOTE The values in Table 11.6 (NDP) and 11.7 (NDP) are computed with the values of **t,LS,2 (i.e. CC2) suggested in prEN 1998-1-1:2022, F.3. Values for CC other than CC2 can be calculated according to the note in 6.2.3(3). If the National Annex gives different values of **t,LS,CC for use in a country, the partial factors can be updated according to the note in 6.2.3(3).

1. For the displacement-based approach, elements not listed in Table 11.5, e.g. connections, should be verified in terms of generalized stresses according to (1).

### Limitation of interstorey drift at Significant Damage limit state

1. For all ductility classes and for design made according to 11.7.3, the interstorey drift at SD limit state should be limited to:
2. *d*r,SD ≤ 0,020 *h*s for moment frames;
3. *d*r,SD ≤ 0,015 *h*s for braced frames and inverted pendulum structures;
4. for frames with buckling restrained bracings see 11.12.4(5);
5. *d*r,SD ≤ 0,020 *h*s for dual frames;
6. *d*r,SD ≤ 0,010 *h*s for light weight structures;
7. for moment frames with interacting infills, 7.4.2.1 should be applied;
8. *d*r,SD ≤ 0,015 *h*s for all other structures;

where

|  |  |
| --- | --- |
| *d*r,SD | is as given in 6.2.4; |
| *h*s | is the interstorey height. |

## Design rules for low-dissipative (DC1) and non-dissipative structural behaviour for all structural types

### General

1. 11.7.2 should be applied to the primary members of structures designed for respectively low-dissipative behaviour and non-dissipative behaviour.
2. The resistance of the structural members and of their connections should be determined according to EN 1993.

### Design rules for low-dissipative structures

1. Low-dissipative structures should be designed to resist seismic actions almost in the elastic range. Thedesign forces in their members and connections should be calculated assuming that *q* is equal to 1,5 with both *q*D and *q*Requal to 1,0.
2. The structural members and connections should have stiffness and resistance in accordance with Eurocode 3.

### Design rules for non-dissipative structures

1. Non-dissipative structures should be designed to resist seismic actions in the elastic range and their behaviour factor *q* should be taken equal to 1,0.

NOTE Non-dissipative design is used for structures for which a ductile behaviour can hardly be conceived, e.g. steel domes, or cannot be supported by research reference. It is also used for specific structural systems for which non-dissipative design can be safe and more economical.

1. prEN 1998-1-1:2022, 4.1(7), may be used in the design of single-storey buildings; the limits of seismic action index *S*δ for their design to DC1 in 12.4.3 may be waived, provided that the condition in (4) is satisfied.
2. prEN 1998-1-1:2022, 4.1(7), may be used without limitation of the seismic action index *S*δ in the design of structural systems which cannot be assigned to one of the types in 11.4.1(1) a) to i), provided that the condition in (4) is satisfied.
3. The buckling resistance of members in compression and the connections should be designed for action effects calculated for the return period *T*LS,CC at the NC limit state given in Table 4.3 (when using prEN 1998-1-1, 4.3(3)) or the performance factor *γ*LS,CC at the NC limit state given in Table 4.4 (when using prEN 1998-1-1, 4.3(5)).

## Design rules for dissipative (DC2 and DC3) structural behaviour common to all structural types

### General

1. 11.8.2 to 11.8.5 should be applied to the members of the primary structures designed for dissipative behaviour.

### Design criteria for dissipative structures

1. Structures with dissipative zones shall be designed so that yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.
2. Dissipative zones should have adequate ductility and resistance.
3. Dissipative zones may be located in the structural members or in the connections.
4. If dissipative zones are located in the structural members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure should have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts.
5. When dissipative zones are located in the connections, the bolts and welds of the connections and the connected members should have sufficient overstrength to allow the development of cyclic yielding or friction sliding in the weakest components of the connections.

### Verification for dissipative members in compression or bending

1. Sufficient local ductility of members which dissipate energy in compression or bending should be ensured by restricting the width to thickness ratio *b*/*t* according to the cross-sectional classes specified in EN 1993-1-1:2022, 7.5.

NOTE Further rules dependent on structural types and are given in 11.9 to 11.16.

1. Depending on the ductility class and the behaviour factor *q*, cross-sectional classes of dissipative members should be chosen according to Table 11.8.

Table 11.8 — Provisions on cross-sectional class of dissipative members depending on Ductility Class and behaviour factor

|  |  |  |
| --- | --- | --- |
| **Ductility class** | **Value of *q*** | **Required cross-sectional class** |
| DC2 | 1,5 < *q*  2 | class 1, 2, 3 and 4  for portal frames, lightweight systems and single-storey moment frames |
| 1,5 < *q*  2 | class 1, 2  for inverted pendulum |
| 2 < *q*  3,5 | class 1, 2  for MRFs, CBFs, EBFs and dual frames |
| DC3 | *q >* 3,5 | class 1 |
| 2  *q*  2,5 | class 1, 2, 3 or 4  for lightweight systems |

### Verification for dissipative parts of members in tension

1. For tension members or parts of members in tension, the ductility provision of EN 1993-1-1:2022, 8.2.3(3), should be satisfied.

NOTE If the section with holes mentioned in EN 1993-1-1:2022, 8.2.3(4) is reinforced by additional plates, its net cross section includes those plates.

### Verification of members

1. In DC2, the resistance and stability of members defined by structural type in Table 11.9 and of the connections of these members should be verified under the most unfavourable combination of the axial force *N*Ed, bending moments *M*Ed and shear force *V*Ed from Formula (11.2).

(11.2)

where

|  |  |
| --- | --- |
| *N*Ed,G, *M*Ed,G and *V*Ed,G | are the effects of the non-seismic actions in the seismic design situation; |
| *N*Ed,E, *M*Ed,E and *V*Ed,E | are the effects of the design seismic action; |
| ** | is the seismic action magnification factor, see (Table 11.9); |
| "+" | means combined with + or – sign. |

NOTE The non-dissipative members are defined in Table 11.9.

1. The resistance and stability of non-dissipative members in DC3 should be verified under the most unfavourable combination of the axial force *N*Ed, bending moments *M*Ed and shear force *V*Ed from Formula (11.3).

(11.3)

where

|  |  |
| --- | --- |
| *ω*rm | is the material overstrength factor of the steel in the dissipative zone (see 11.2.2); |
| *ω*sh | is the factor accounting for hardening of the dissipative zone (see Table 11.10); |
| *Ω*d | is the minimum design overstrength, see (3); |
| "+" | means combined with + or – sign. |

NOTE 1 The members to which (2) applies are the members which in DC3 are considered non-dissipative and which are defined in Table 11.9.

NOTE 2 If friction connections are used the dissipation mechanism does not involve any plasticity, the *ω*rm can be assumed equal to 1,0.

Table 11.9 — Members to which (1) or (2) apply. Values of seismic action magnification factor *Ω* in DC2

|  |  |  |
| --- | --- | --- |
| **Structural type** | ***Ω*** | **Members to which (1) or (2) apply** |
| **Moment resisting frames (MRFs)** |  | columns |
| Portal frames with class 3 and 4 cross sections | 1,5 |
| Single-storey MRFs with class 3 and 4 cross sections | 1,5 |
| Portal frames and single-storey MRFs with class 1 and 2 cross sections | 1,7 |
| Multi-storey MRFs | 2 |
| MRFs with friction connections | 2 |
| **Frames with concentric bracings** | 1,5 | beams and columns |
| Diagonal bracings |
| V-bracings |
| X-bracings on either single or two-storey |
| **Frames with eccentric bracings** | 2 | beams outside the link, braces and columns |
| **Dual frames** |  |  |
| MRFs with concentric bracing | 1,7 | beams and columns of the concentric bracing;  columns of the MRF; |
| MRFs with eccentric bracing | 2 | beams out of the link, braces and columns of the eccentric bracing; columns of the MRF |
| **Structures with concrete cores or concrete walls** | See Clause 10 |  |
| **Lightweight steel frame wall systems** |  | connections and framing: chord studs and tracks |
| with flat strap bracing | 1,5 |
| with steel sheeting | 1,5 |
| with wood sheathing | 1,5 |
| with gypsum sheathing | 1,3 |
| **Inverted pendulum structures** | 1,5 | columns |
| **Moment resisting frames with infills** |  |  |
| with unconnected with non-interacting concrete or masonry infills | 1,5 | columns |
| with connected reinforced concrete infills | See Clause 10 | See Clause 10 |
| with non-interacting infills | (see MRFs) | columns |

1. In DC3 the design overstrength factor *Ω*d should be taken not smaller than 1 and to depend on the structural system, as specified in 11.9 to 11.15.
2. In DC2 and DC3 structures the resistance *R*d of non-dissipative members connected to dissipative connections should satisfy Formula (11.4).

### Verification of connections in dissipative zones

1. In DC2 and DC3, the design of a connection and of the member attached to it should satisfy either a) or b):
2. The member is dissipative and the connection is non-dissipative;
3. The member is non-dissipative and the connection is dissipative.
4. In DC2 and DC3, the resistance *R*d of a non-dissipative structural component attached to a dissipative structural component should satisfy Formula (11.4).

(11.4)

where

|  |  |
| --- | --- |
| *R*fy | is the resistance of the dissipative structural component in accordance the definition of resistance with subscript y in prEN 1993-1-8; |
| *ω*rm | is the factor for variability of steel yield strength in the dissipative zone (see 11.2.2 and (10)); |
| *ω*sh | is the overstrength factor for hardening of the dissipative zone (see (8) and Table 11.10). |

1. The resistance *R*d of non-dissipative connections should satisfy (1)a) and Formula (11.4) in which *Rfy*, *ω*rm and *ω*sh as defined in (2) characterize the member attached to the connection.
2. The resistance *R*d of dissipative connections should satisfy (1)b) and Formula (11.4) in which *Rfy*, *ω*rm and *ω*sh as defined in (2) characterize the connection attached to the member.
3. Non-dissipative connections should be designed for the relevant forces/moments at the location of the connection calculated in accordance with 11.8.5.
4. Fillet welded or bolted non-dissipative connections at the location of the dissipative zones should satisfy Formula (11.4), where *R*d is the resistance of the connection in accordance with prEN 1993-1-8 calculated with the plastic resistance *f*y of the connected dissipative member at the expected position of the plastic hinge and based on the nominal yield stress of the material as defined in EN 1993-1-1:2022, 5.2.1.
5. Non-dissipative connections of dissipative members made by means of full penetration butt welds may be considered to satisfy Formula (11.4).
6. In DC2, *ω*sh may be assumed equal to 1,1.
7. In DC3, unless estimated with either analytical method, experimental validation or numerical simulation, *ω*sh should be taken from Table 11.10.
8. With friction connections, the dissipation mechanism does not involve plastic deformations and *ω*rm should be taken equal to 1,0.
9. In all DC, category A bolted joints in shear as defined in prEN 1993-1-8:2021, 5.4.1a), may be used provided that a) and b) are satisfied:
10. All bolts in each joint are tightened in accordance with prEN 1993-1-8:2021, 5.1.2(1);
11. Clamped surfaces in contact are treated as class A or B in accordance with EN 1090-2.
12. In all DC, bolted joints in simple tension should be Category E of prEN 1993-1-8:2021, 5.4.1(2)b).
13. Hybrid joints with both bolts and welds either in shear or in tension should not be used in seismic zones unless action effects can be resisted by the welded connection alone.
14. For bolted shear connections, the design shear resistance of the bolts should not be less than the design bearing resistance in at least one of the connected plates. In addition, bolted joints in shear should have end distances in the line of seismic force not less than two bolt diametres when the bearing force in the seismic combination of actions exceeds 75 % of the bearing resistance.
15. The resistance and ductility of members and their connections under cyclic loading should be demonstrated by tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. Past test results from the literature and refined numerical simulations as defined in prEN 1993‑1‑14 may be used to demonstrate the effectiveness of the designed partial and full-strength connections in or adjacent to dissipative zones of DC2 and DC3 structures.
2. Experimental validation for partial and full-strength connections may be omitted if prequalified connections are used.

NOTE Annex E gives complementary rules on seismic prequalification of beam-to-column joints and design rules for gusset connections of braced structures.

Table 11.10 — Overstrength factor **sh accounting for hardening of the dissipative zones

| **Structural type** | **Dissipative zones** | **Plastic mechanism** | ***ω*sh** |
| --- | --- | --- | --- |
| Moment resisting frames | beams | bending |  |
| yielding connections |
| columns at base |
| friction connections | friction | 1,3*ω*sr *ω*μ ≤ 2,2  *ω*sr and*ω*μ as defined in Annex E |
| Frames with concentric bracings (simple and dual) | diagonal members | axial | 1,1 |
| all members | bending (see 11.10.5 and 11.10.6) | 1,1 |
| dissipative connections | axial | 1,1 |
| bending | 1,2 |
| shear | 1,5 |
| Frames with eccentric bracings (simple and dual) | short links | shear  *e* ≤ *M*p,link*/V*p,link  (very short links) | 1,8 |
| shear  *M*p,link*/V*p,link < *e* ≤ 1,6*M*p,link*/V*p,link  (short links) | 1,5 |
| intermediate links | bending and shear  *e* ≤ 2,6*M*p,link*/V*p,link | 1,5 |
| bending and shear  2,6*M*p,link*/V*p,link< *e* ≤ 3*M*p,link*/V*p,link | 1,35 |
| long links | Bending  3*M*p,link*/V*p,link< *e* ≤ 5*M*p,link*/V*p,link | 1,25 |
| Bending  *e* > 5*M*p,link*/V*p,link |  |
| beams - columns | bending (see 11.11.5) | 1,1 |
| Frames with buckling restrained braces | diagonal members | axial | see 11.12.3(4) |
| beams - columns | bending (see 11.12.6) | 1,2 |

### Verification of column-to-column splices

1. Column splices may be either bolted or welded or welded to one column and bolted to the other.
2. For all columns of the primary structure, column-to-column splices should be at least 1,20 m away from beam-to-column joints to avoid interaction; if the storey height is smaller than 2,40 m, column splices should be at half the clear height.
3. The required bending moment, shear and axial force resistance of column-to-column splices in the primary structure should be the greater of the resistances of the spliced columns, as specified in 11.9 to 11.15.
4. For all columns, including those which are not part of the primary structure (e.g. pinned columns), the required shear resistance *V*c,Rd of column splices with respect to both orthogonal axes of the column should satisfy Formula (11.5).

(11.5)

where

|  |  |
| --- | --- |
| *M*pl,Rd,c(*N*Ed) | is the design moment resistance of the column in the considered direction, taking into account the interaction with the axial load *N*Ed, in the seismic design situation; |
| *h*ss | is the distance between the upper flanges of the steel profiles or the tops of floor slabs at each of the levels above and below the splice. |

## Design rules for moment resisting frames

### Design criteria

1. In DC3, moment resisting frames should be designed so that plastic hinges form in the beams or in the connections of the beams to the columns, but not in the columns. This rule may be neglected in cases a) to c):
2. at the base of the frame in which *N*Ed,G in primary columns satisfies the inequality: *N*Ed,G / *N*pl,Rd < 0,3;
3. at the top of primary columns in the upper storey of multi-storey buildings;
4. at the top and bottom of primary columns in single storey buildings in which *N*Ed,G in columns satisfies the inequality: *N*Ed,G / *N*pl,Rd < 0,3.
5. Depending on the location of the dissipative zones, either 11.8.2(4) or 11.8.2(5) should be applied.
6. The target hinge formation pattern should d be achieved by conforming to 11.9.2, 11.9.3 and 11.9.4.

### Verification of beams

1. Beams should be verified to have sufficient resistance against lateral and lateral torsional buckling in accordance with EN 1993-1-1:2022, 8.3.5 assuming the formation of a plastic hinge at one end of the beam namely the plastic moment in one end and the design moment in the other end. The most stressed beam end in the seismic design situation should be considered for the verification. This condition may be considered satisfied if lateral-torsional restraints are detailed in accordance with (3) in DC2 and (4) in DC3.

NOTE If partial strength or friction connections are used, the lateral torsional buckling of the beam can be verified assuming the ultimate expected moment resistance of the connection at one end of the beam.

1. There should not be openings in the web of beams in dissipative zone and in the vicinity of dissipative zones up to a distance equal to 3 times the beam depth from the beam end or from the centre of the reduced beam section, if any.
2. In DC2, the spacing and the resistance of lateral-torsional restraints (e.g. bracing) should comply with EN 1993:1-1:2022, 8.3.5.3.
3. In DC3, the distance between lateral-torsional restraints should not be greater than the length *L*st given by Formula (11.6).

(11.6)

where

|  |  |
| --- | --- |
| *i*z | is the radius of gyration with respect to the weak axis of the cross section; |
| *E*s | is the elastic modulus of steel. |

1. The lateral-torsional restraints in DC3 should be designed to resist the effects of local forces *Q*m applied at each stabilized member at the plastic hinge locations, given by Formula (11.7).

(11.7)

where *N*f,Ed is the axial force in the compressed flange of the stabilized member at the plastic hinge location, which should be determined as *N*f,Ed = *b*f *t*f *f*y with *b*f and *t*f the width and the thickness of the beam flange, respectively.

1. In accordance with Table 11.8, section classes 1 and 2 should be used for beams in DC2 MRFs and section classes 3 and 4 may be used in DC2 portal frames and single-storey MRFs. In DC3, only section class 1 should be used for beams or the portions of the beams (e.g. the beam segment with reduced section) where plastic hinges are expected to form.
2. Formulae (11.8) to (11.10) should be satisfied.

(11.8)

(11.9)

(11.10)

where

|  |  |
| --- | --- |
| *M*Ed, *N*Ed and *V*Ed | are the bending moment, the axial force and the shear force, respectively, in the seismic design situation; |
| *N*Rd,b, *M*Rd,b, *V*Rd,b | are the corresponding design resistances of the cross sections of the beams to be determined in accordance with EN 1993-1-1 in function of their class. |

1. For beams in DC3, Formula (11.10) should be satisfied with the design shear *V*Ed given by Formula (11.11).

(11.11)

where

|  |  |
| --- | --- |
| *V*Ed,G | is the design value of the shear force to be determined on the basis of the equilibrium of the beam under the non-seismic actions; |
| *V*Ed,M | = (*M*pl,Rd,b,A+*M*pl,Rd,b,B)/*Lh*, is thedesign value of the shear force due to the application of the plastic moments *M*pl,Rd,b,A and *M*pl,Rd,b,B  with opposite signs at the end sections “A” and “B” of the beam; |
| *Lh* | is the distance between the dissipative zones at both ends of the span. |

NOTE The ductility of moment resisting frames can be improved by weakening of the beam flanges at desired locations of dissipative zones. Some guidance to detail the weakening of the beam is given in Annex E.

1. If Formula (11.9) is not satisfied, (6) may be considered satisfied if EN 1993-1-1:2022, 8.2.9.1 is satisfied.
2. If partial strength connections are used, Formulae (11.8) to (11.10) should also be satisfied considering for *N*Rd,b, *M*Rd,b, *V*Rd,b the design resistances of the connections.

### Verification of columns

1. The resistance and stability of columns should be verified in compression, bending and shear considering the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed from Formula (11.2). Columns in DC3 should satisfy Formula (6.5) in 6.2.7(2).
2. Columns in DC3 should be verified in compression, bending and shear under the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed from Formula (11.3), where **d is the minimum value of (*M*pl,Rd,b - *M*Ed,G)/*M*Ed of all beams if dissipatve zones are in the beams, or of all connections, if dissipative zones are in dissipative partial strength connections of the beams to the columns. *M*Ed,G is the moment due to the non-seismic actions.
3. The resistance and stability verifications of columns should satisfy EN 1993-1-1:2022, 8.2.
4. If a plastic hinge is expected in the column, its shear force *V*Ed from the analysis should satisfy the condition given by Formula (11.12).

(11.12)

where *V*Rd,c is the design resistance of the column cross sections in accordance with EN 1993-1-1:2022, 8.2.

1. Columns in DC3 should satisfy Formula (11.13).

(11.13)

where

|  |  |
| --- | --- |
| Σ*M*pl,Rd,c(*N*Ed) | is the sum of the design moment resistances of the column framing into the joint in accordance with EN 1993-1-1:2022, 8.2.9.1, taking into account the axial load *N*Ed determined according to Formula (11.3); |
| Σ*M*pl,Rd,b | is the sum of the design moments of resistance of the beams framing the joint or the sum of the design values of the moments of resistance of the beam connections if dissipative partial strength connections are used; |
| *V*Ed,M | is the shear force in the beam due to the formation of plastic hinges at both beam ends; *V*Ed,M  is equal to (*M*pl,Rd,b,A+*M*pl,Rd,b,B )/*L*h~~,~~ where *L*h is the distance between the dissipative zones at both ends of the span. |
|  | NOTE *V*Ed,M  =(*M*pl,Rd,b,A+*M*pl,Rd,b,B )/*L*h is the upper bound of the possible *V*Ed,M |
| *V*Ed,G | is the shear force due to the non-seismic actions in the seismic design situation; |
| *s*h | is the distance between the centre of the expected plastic hinge and the column axis; |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *ω*sh | is the hardening factor of the plastic hinge (see Table 11.10). |

NOTE 1 *V*Ed,M =(*M*pl,Rd,b,A+*M*pl,Rd,b,B)/*L*h is the upper bound of the possible *V*Ed,M.

NOTE 2 If friction connections are used the dissipation mechanism does not involve plastic deformations and *ω*rm is equal to 1,0.

1. The non-dimensional slenderness of columns where a plastic hinge is expected to form should not be greater than 0,85.
2. The cross-sectional class of columns should comply with Table 11.8.
3. In columns of single storey MRFs with cross sections class 3 or 4, *N*Ed should be not greater than 0,6 *N*Rd,c, where *N*Rd,c is the design resistance of the column cross section in accordance with EN 1993-1‑1.

### Verification of beam to column joints

1. The composite action between the beam and the concrete slab should be prevented using measures specified in 12.8.6.2.3. Otherwise, the frame should be designed to Clause 12 and beam-to-column joints should be designed according to 12.8.6.

NOTE Further rules and design detailing to avoid the composite action are given in Annex E.3.1 (3).

1. In structures designed to dissipate energy in the beams, the connections of the beams to the columns and the column web panels should satisfy Formula (E.1) for both external and internal joints.
2. Dissipative semi-rigid and/or partial strength connections may be used for DC2 and DC3 structures, if a) to c) are all satisfied:
3. the rotation capacity of the connections is consistent with the global deformations; this condition may be neglected if prequalified connections are used;
4. the resistance and stability of members framing into the connections are verified at the SD limit state;
5. the effects of connection deformation on global drift are taken into accountusing linear analysis if prequalified partial strength connections are used, while non-linear analyses should be performed for all other types of connections.
6. The design resistance of the connection should be obtained from prEN 1993-1-8:2021, Clause 8.
7. Column web panels framed by flanges and stiffeners (see Figure 11.12) should satisfy Formula (11.14).

(11.14)

where

|  |  |
| --- | --- |
| *V*wp,Ed | is the design shear force in the web panel due to the action effects, taking into account the plastic resistance of the adjacent dissipative zones in beams or connections, see Figure 11.12 and prEN 1993-1-8:2021, 8.2.6; |
| *V*wp,Rd | is the shear resistance of the web panel. |

Figure 11.12 — Web panel framed by column flanges and continuity plates

1. Web panels should satisfy Formula (11.15).

(11.15)

where *V*wb,Rd is the shear buckling resistance of the web panel.

1. In DC3 frames, the total thickness *t* of the web panel should satisfy Formula (11.16).

(11.16)

where

|  |  |
| --- | --- |
| *d*b | is the depth of the deeper connected beam; |
| *t*b,f | is the thickness of the flange of the deeper connected beam; |
| *d*c | is the depth of the column section; |
| *t*c,f | is the thickness of the flange of column; |
| *t*w | is thethickness of the web of a steel profile; |
| *t*wsp | is thethickness of a supplementary web plate. |

1. If Formula (11.16) is satisfied, the condition given by Formula (11.15) may be considered satisfied.
2. The joints should be designed to provide a plastic part of the rotation capacity rotation **u (see Figure 11.13) not smaller than 0,02 rad in DC2 and 0,03 rad in DC3.

NOTE 1 Detailed rules for the design of joints compliant with (9) are given in Annex E.

NOTE 2 In absence of specific tests on materials of the joints, the real plastic moment of resistance *M*pl,act can be assumed equal to the expected plastic moment *ω*rm*M*pl of the dissipative element .

NOTE 3 In case of friction connections, is the rotation due to the slippage of the damper as detailed in Annex E.

1. The part  of the rotation capacity of the joint **u should be ensured under cyclic loading without degradation of the expected post-capping strength (see Figure 11.13) independently of the intended location of the dissipative zones. It should be evaluated on the first cycle envelope curve from the tests.

NOTE In case of friction connections, the expected strength *M*p,act can be evaluated under both sagging and hogging deflection and is due to the dynamic friction coefficient of the slipping surface, as detailed in Annex E.

1. When dissipative partial strength connections are used, the column capacity design should be based on the plastic capacity of the connections.
2. When friction connections are used, the column capacity design should be based on the upper bound static friction resistance of the connections.

NOTE 1 prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for tests.

NOTE 2 New experimental tests are not required for prequalified connections because these guarantee the rotation capacity

Figure 11.13 — Definition of plastic rotations **u and

### Verification of column base joints

1. If the column base joint is fully restrained, the resistance of the base connections *M*,j,Rd,c should comply with Formula (11.17).

(11.17)

where

|  |  |
| --- | --- |
| *M*pl,Rd,c(*N*Ed) | is the design moment resistance of the column taking into account the axial loads *N*Ed in the seismic design situation; |
| *ω*rm | is the material overstrength factor given in 11.2.2; |
| *ω*sh | is the hardening factor of plastic hinge, which may be taken equal to 1,1 for DC2 and from Table 11.10 for DC3. |

1. Formula (11.17) may be satisfied by using either exposed or embedded column base joints.

NOTE Fixity at embedded column base joints can be obtained by extending the column below the assumed seismic base into a basement, if any, and by restraining the horizontal displacements of the ground floor relative to the basement. Other options to ensure the column fixity can be to encase the base joint into a grade beam or to embed it in a stocky concrete wall. Details for column base joints are given in Annex H.

1. Plastic hinges may develop at a column base joint, provided that all conditions a) to d) are satisfied:
2. the base joint has a rotation capacity consistent with the global deformations and not smaller than 0,04 rad;
3. the column exhibits adequate overstrength to allow cyclic yielding in the base joint at the SD limit state;
4. the effects of deformability of base joints on lateral drifts and global stability are taken into account in the analysis;
5. the column base joints satisfy the rules for connections in dissipative zones.
6. The resistance to shear and axial force of column bases and their attachments to the foundations should be greater than the shear force *V*Ed and the axial force *N*Ed as given by Formula (11.12) in DC2 and Formula (11.13) in DC3.
7. The resistance to shear force of a column base and its attachments to the foundations should satisfy Formula (11.5).

## Design rules for frames with concentric bracings

### Design criteria for DC2 and DC3

1. Concentrically braced frames shall be designed so that yielding of the diagonals in tension takes place before failure of the connections and before yielding or buckling of the beams or columns.
2. The diagonal members of bracings should be placed in such a way that the structure exhibits similar behaviour at each storey in opposite senses of the same braced direction under load reversals. To this end, the rule given by Formula (11.18) should be satisfied at each storey:

(11.18)

where and are the areas of the vertical projections of the cross sections of the tension diagonals, when the horizontal seismic actions have a positive or negative direction respectively (see Figure 11.14).

Figure 11.14 — Example of application of 11.10.1(2)

1. Eccentricities of diagonal members in the end connections as respect to the beam-column axes should not be greater than the beam depth and their effects on the members and connections forces should be taken into account.

### Analysis

1. Beams and columns should be considered to resist gravity loads in the persistent and transient design situation, without taking into account the resistance of the bracing members. In addition, the buckling resistance of diagonal bracings should be verified against the axial forces due to the imposed and variable loads as given in prEN 1991-1-1, prEN 1991-1-3 and prEN 1991-1-4 at ultimate limit state in non-seismic design situation.
2. The diagonals should be taken into account using an elastic analysis of the structure for the seismic action according to a) to c):
3. The “tension-only” model may be only used for DC2 frames with X diagonal bracings or split X diagonal bracings;
4. In DC2 frames with V bracings and two-storey X bracings, both the tension and compression diagonals should be taken into account;
5. in DC3 frames, both the tension and compression diagonals should be taken into account.
6. The compression diagonals in DC2 may be neglected in the analysis as defined in (2)a) provided that the lateral resistance of the building in pre-buckling range of diagonal members is smaller than the lateral resistance of the building evaluated with only the tension diagonals at yield.
7. Both tension and compression diagonals may be taken into account in the analysis of any type of concentric bracing provided that both pre-buckling and post-buckling situations of diagonals are taken into account in both design and modelling.

### Verification of diagonal members

1. The cross section of diagonal bracings should be of class 1 in DC3 and class 1 or 2 in DC2 according to EN 1993-1-1:2022, 5.5. For DC3 frames, a) and b) should be also fulfilled:
2. If circular hollow sections are used for diagonal bracings, their local slenderness *D*/*t* should not be greater than , where *D* is the external diametre and *t* the thickness of the cross section and ;
3. If either rectangular or square hollow sections are used for diagonal bracings, their maximum local slenderness *c*/*t* should not be greater than , where *c* is the side width in accordance with EN 1993-1-1 and *t* the thickness of the cross section.
4. The length of the bracing may be taken as the theoretical node-to-node length disregarding the gusset connections at both brace ends. The buckling length should also account for the restraint given by the brace end-connections and the mutual restraint at the mid-length connection between the diagonals of X bracings. The assumed degree of connection restraint between the diagonals should be verified through analytical calculations, refined numerical simulations as defined in prEN 1993-1-14 or experimental results from the literature.
5. Unless more refined data or analysis are available, the buckling length accounting for the restraint given by the brace end connections may be taken equal to 80 % of its centreline (node-to-node) length if the gusset plates are designed according to Annex E.
6. In DC2 frames with X diagonal bracings, when the tension-only model is adopted, the non-dimensional slenderness should be limited to: 1,3 ≤  2,5.
7. In frames with tension-compression diagonal bracings (see Figure 11.3), the non-dimensional slenderness should not be greater than 2,0 in DC3 and 2,5 in DC2.
8. In structures of up to two storeys with tension-compression diagonal bracings, there is no limitation of non-dimensional slenderness .
9. In frames designed with tension-only bracings, the yield resistance *N*pl,Rd  of the gross cross-section of the diagonals should not be smaller than the axial force *N*Ed in the bracing member in the seismic design situation.
10. In frames with tension-compression bracings, the buckling resistance *N*b,Rd of the bracing members should be such that *N*b,Rd  *N*Ed.
11. In DC3 frames with tension-compression bracings and with 6 storeys or more, the bracings of the top storey should not buckle.
12. To satisfy (9) in a force based approach, the buckling resistance *N*b,Rd,ts of the bracing members at the top storey should be such that *N*b,Rd,ts  *N*Ed,ts, with *N*Ed,ts from Formula (11.19).

(11.19)

where

|  |  |
| --- | --- |
| *N*Ed,G,ts | is the axial force due to non-seismic actions in the seismic design situation; |
| *N*Ed,E,ts | is the axial force due to seismic actions; |
| *q* | is the behaviour factor. |

1. The connections of the diagonals to any member should satisfy 11.8.6(1)a), (2) and (3). The required resistance of the brace connection should satisfy 11.8.6(1)a), (2) and Formula (11.4), with **sh from Table 11.10 for DC3 and **sh = 1,1 for DC2. The net area of the brace cross-section in the joint zone (i.e. the area with deductions for all holes and other openings) should be increased such that its axial resistance is not smaller than the axial resistance of its corresponding gross area.
2. In DC3 frames, at all storey with the exception of the top storey, the maximum buckling overstrength **b,i in the structure should not differ from the minimum value **b by more than 25 %, with **b,i from Formula (11.20).

(11.20)

where

|  |  |
| --- | --- |
| *N*b,Rd,i | is the buckling resistance of the diagonal bracing; |
| *N*Ed,i | is the axial force in the bracing member in the seismic design situation; |
| *Ns* | is the total number of storeys. |

1. Dissipative partial strength connections as well as reduced yielding segments in the bracings may be used, provided that conditions a) to f) are satisfied:
2. the buckling of the gusset plates of the connections is prevented;
3. the buckling resistance of the diagonal bracings *N*b,Rd is greater than , where *R*d is the resistance of the connection in accordance with prEN 1993-1-8, or the resistance of the reduced yielding segments in accordance with EN 1993-1-1;

NOTE 1 Concentrical bracings with reduced yielding segments are slender diagonal members with one cut-out portion at each end of the diagonal that do not modify the compression resistance of the member and which is detailed to have a local tension resistance similar to the global buckling resistance of the diagonal.

1. the elongation capacity of the connections and of the reduced yielding segments should be consistent with global deformations;
2. the elongation capacity used in design of partial strength connections should be demonstrated by experimental tests, either new or from the literature, or by numerical simulations.

NOTE 2 Annex E provides rules for INERD-PIN partial strength connections.

1. the length *L*r of the segment with the reduced section should not be smaller than 5,65(*A*rs)0,5, where *A*rs is the area of the reduced section; the local slenderness of the parts of the reduced area should comply with (1);
2. the effect of deformations of partial strength connections on global drifts are taken into account using linear analysis if connections covered by Annex E are used. Non-linear analysis should be carried out in all other cases.

### Verification of beams and columns

1. In DC2, the resistance and stability of beams and columns should be verified in compression, bending and shear under the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed from Formula (11.2).
2. In DC3 frames, beams and columns should be designed to resist the forces given in a) to c):
3. the total action effects *N*Ed, *M*Ed and *V*Ed determined according to 11.8.5(2), where **d is the minimum value of the ratio *N*pl,Rd,i/*N*Ed,Ei of all bracings where *N*pl,Rd,I is the tension axial resistance of brace“I” and *N*Ed,Ei is the design value of the axial force due to the seismic actions in brace “i”;
4. internal forces calculated considering a free-body distribution of axial forces in both tension and compression diagonals with values of forces equal to their expected buckling resistance *ω*rm *N*b,Rd in combination with the forces due to all non-seismic actions in the seismic situation;
5. internal forces calculated considering a free-body distribution of forces transmitted by the braces under tension forces equal to *ω*rm *ω*sh *N*Rd and those under compression forces equal to *ω*rm *ω*pb *N*b,Rd in combination with the forces due to all non-seismic actions in the seismic design situation.
6. In (2) a) and c), the overstrength factor *ω*sh should be taken from Table 11.10.
7. In (2) c), *ω*pb may be taken equal to 0,30.

NOTE *ω*pb is the multiplicative factor for the estimation of the residual post-buckling resistance of the compression brace.

1. In frames with V bracings or in two-storey frames with X bracings (see Figure 11.3), the beams should be designed to resist all non-seismic actions in the seismic situation without considering the intermediate support by the diagonals.
2. In DC2 and DC3 frames with V bracings or two-storey frames with X bracings, the flexural stiffness *k*b of the beams which intersect a bracing should not be smaller than 0,2 times the vertical stiffness *k*br of the intersecting bracings, where:

|  |  |
| --- | --- |
| *k*b |  (48*E*s*I*b/*L*b3) is the flexural stiffness of the beam; |
| *E*s | is the elastic modulus of steel; |
| *I*b | is the second moment of area of the beam section; |
| *L*b | is the beam length; |
|  | depends on the beam boundary condition ( = 4 for fixed ends and  = 1 for pinned ends); |
|  | is the vertical stiffness of the bracing; |
| *A*br | is the area of the brace section; |
| *L*br | is the brace length; |
| α | is the angle of the brace with respect to the horizontal line. |

1. If non-linear static or dynamic analyses are performed to verify the overall deformations and the distribution of damage, (4) may be waived.
2. If zipper V-bracings (where the brace-intercepted beams are tied by vertical struts, see Figure 11.3 case 7) are used, the resistance and stability of vertical members connecting the brace-to-beam intersection should be verified against the unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal, calculated according to 11.10.4.2(c).
3. In DC3, the resistance and stability of columns should be verified under axial forces calculated according to 11.10.4.2(c) and additional uniform bending moments in the direction of the braced bays equal to 0,2 *M*c,pl,Rd where *M*c,pl,Rd is the plastic moment resistance of the gross section of the column.
4. In frames with diagonal bracings in which the tension and compression diagonals are not intersecting (see Figures 11.2 and 11.3), the tensile and compression forces which develop in the columns adjacent to the diagonals in compression should be taken into account, with values of compression forces in the diagonals equal to their expected buckling resistance.

### Verification of beam to column connections

1. In braced bays, beam-to-column connections which serve also as connection of a bracing should be designed to the transfer seismic action effects from that bracing to the surrounding frame.
2. The beam-to-column connections may be either continuous or pinned and a) or b), as appropriate, should be satisfied.
3. the total rotation capacity of pinned and partial strength connections should not be smaller than 0,02 rad;
4. the resistance *M*j,Rd of continuous beam-to-column connections should be greater than the minimum expected moment resistance of the connected members, taken as given in Formula (11.21):

(11.21)

where

|  |  |
| --- | --- |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *ω*sh | is the hardening factor, which may be taken equal to 1,1 for DC2 and as in Table 11.10 for DC3. |

NOTE Annex E provides complementary rules on the beam-to-column joints of braced structures.

### Verification of brace connections

1. Brace connections should resist the axial force due to yielding and hardening of tensile diagonals and they should either restrain the rotation at the end brace (see (2)) or accommodate brace buckling under repeated cyclic loading (see (3)).
2. For brace connections restraining buckling of diagonal members, the bending of bracings should be considered as restrained in both main directions of their cross section. The resistance of the connections should be verified against design forces acting on the connections both in-plane and out-of-plane separately evaluated from Formulas (11.22) to (11.24).

(11.22)

(11.23)

(11.24)

where

|  |  |
| --- | --- |
| *N*pl,Rd | is the design plastic resistance of the brace in tension; |
| *N*b,Rd | is the design buckling resistance of the brace in compression; |
| *M*pl,Rd | is the design plastic moment of the cross section of the brace evaluated in the plane of the frame; |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *ω*sh | is the hardening factor of bracings, which may be taken equal to 1,0 for DC2 and from Table 11.10 for DC3 |

1. Brace connections accommodating buckling of diagonal members in one plane should be verified against forces separately evaluated from Formulas (11.22), (11.23) and (11.24) in the restrained plane. In the buckling plane, they should resist the expected buckling resistance of the connected diagonal bracing, using Formula (11.23). Their plastic rotation capacity should not be lower than the required rotation at the design storey drift.

NOTE A brace terminating before a linear or an elliptical clearance that corresponds to the theoretical line of restraint can provide the required plastic rotation capacity; see Annex E an elliptical clearance that corresponds to the theoretical line of restraint. For detailing see Annex E.

1. For out-of-plane brace buckling, if gusset plate connections are adopted, the connection in DC3 should resist a shear force in the direction in which the brace buckles and along the theoretical line of restraint, calculated as given by Formula (11.25).

(11.25)

where

|  |  |
| --- | --- |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *f*y | is the yield stress of the material of the gusset plate; |
| *t*gp | is the thickness of gusset plate; |
| *w*gp | is the effective width of the gusset plate, evaluated assuming spreading from the connected end of the brace to the theoretical line of restraint at an angle of 30° on both side of the diagonal. |

### Verification of column base joints

1. The resistance of column base joints should be verified under the combination of action effects due to non-seismic actions in the seismic design situation with those due to seismic action. The effects due to seismic action are the axial and shear forces and moments induced by a design axial force in the connected diagonal brace which should be taken equal to the tensile resistance *ω*rm *ω*sh *N*Rd of the diagonal. *ω*shmay be taken equal to 1,0 for DC2 and should be taken as in Table 11.10 for DC3.
2. Where diagonal braces frame to both sides of a column, the effects of the compression brace buckling force *ω*rm *N*b,Rd should be added to the effects of the adjacent tension brace.
3. Where column bases are designed as moment connections to the foundation, the required flexural resistance should be at least equal to the required flexural resistance of diagonal brace connections from Formula (11.24).
4. Moments at column to column-base connections designed as pinned connections may be ignored.

NOTE The detailing provisions for column base joints are described in Annex H.

## Design rules for frames with eccentric bracings

### Design criteria

1. Frames with eccentric bracings should be designed so that specific members or parts of members called seismic links are able to dissipate energy in plastic bending and/or plastic shear mechanisms.
2. The structural system should be designed so that uniform dissipative behaviour of the whole set of seismic links is realized.
3. Seismic links may be horizontal or vertical components (see Figure 11.4).

### Verification of seismic links

1. Links should be I- or H-shaped cross sections (hot-rolled or built-up sections), or built-up box sections. Cold-formed hollow sections should not be used as links, unless their deformation and resistance capacity are demonstrated by tests or detailed numerical models.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. Cross section of seismic links should satisfy the provisions for cross section class 1 in DC3 and class 1 or 2 in DC2 in accordance with EN 1993-1-1.
2. The web of a link should be of single thickness without doubler plate reinforcement, hole or penetration.
3. Seismic links should belong to one of the three categories a) to c), according to the type of plastic mechanism developed:
4. short links, which dissipate energy by yielding essentially in shear;
5. long links, which dissipate energy by yielding essentially in bending;
6. intermediate links, in which the plastic mechanism involves bending and shear.
7. For I and H sections, the design resistances should be taken as given by Formulae (11.26) and (11.27).

(11.26)

(11.27)

Figure 11.15 — Definition of symbols for link sections: a) I and H sections; b) box sections

1. For box sections, design resistances should be taken as given by Formulas (11.28) and (11.29).

(11.28)

(11.29)

1. If *N*Ed/*N*pl,Rd  0,15, the design resistance of the link should satisfy Formulas (11.30) and (11.31) at both ends of the link.

(11.30)

(11.31)

where

*N*Ed, *M*Ed, *V*Ed are, respectively, the design axial force, design bending moment and design shear force, at both ends of the link in the seismic design situation.

1. If *N*Ed/*N*Rd > 0,15, Formulas (11.30) and (11.31) should be satisfied with the reduced values *V*p,link,r and *M*p,link,r given in Formulas (11.32) and (11.33), instead of *V*p,link and *M*p,link.

(11.32)

(11.33)

1. If *N*Ed/*N*Rd  0,15, the link length *e* should not exceed the value given by Formula (11.34) or by Formula (11.35), as appropriate.

when (11.34)

when (11.35)

where

|  |  |
| --- | --- |
| *R* | = *t*w (*d* –2*t*f) *N*Ed / (*A V*Ed) for I and H section, in which *A* is the gross area of the link; |
| *R* | = 2*t*w (*d* –2*t*f) *N*Ed / (*A V*Ed) for box section. |

1. In DC3 frames with eccentric bracings, the individual values of the design overstrength ratios **d,i at each link, given by Formula (11.36) or Formula (11.37), as appropriate, should not exceed the relevant minimum value **d by more than 25 % of this minimum value with the exception of the links at the top storey.

= min(*V*p,link,i /*V*Ed,i) for short links and intermediate links with (11.36)

= min(*M*p,link,i/*M*Ed,i) for intermediate links with e >2,6and long links (11.37)

where

|  |  |
| --- | --- |
| *V*Ed,i, *M*Ed,i | are the design values of the shear force and of the bending moment in link *i* in the seismic design situation; |
| *V*p,link*,*i, *M*p,link,i | are the shearand bending plastic design resistances of link *i* from11.11.2(5) and 11.11.2(6). |

1. Where equal moments would form simultaneously at both ends of the link (with symmetric moment diagram (see Figure 11.16a), links may be classified according to the length *e* for all types of cross sections as given in Formulas (11.38) to (11.40).

short links (11.38)

long links (11.39)

intermediate links (11.40)

1. Where the moments at the link ends differ (see Figure 11.16b, c and d), links may be classified according to the length *e* for all types of cross sections as given in Formulas (11.41) to (11.43).

short links: (11.41)

long links: (11.42)

intermediate links: (11.43)

where ** is the ratio of the smaller moments *M*Ed,A at one end of the link in the seismic design situation, to the greater moments *M*Ed,B at the end where the plastic hinge would form, both moments taken as absolute values.

Figure 11.16 — Link rotation angle *θ*p between the link and the member outside of the link

1. If non-linear analyses are used, the link plastic rotation angle **p between the link and the member outside of the link as defined in Figure 11.16should be consistent with global deformations. It should not exceed the values given by Formulas (11.44) to (11.46), as appropriate.

short links: *θ*p ≤ *θ*pR = 0,08 rad (11.44)

long links: *θ*p ≤ *θ*pR = 0,02 rad (11.45)

intermediate links: *θ*p ≤ *θ*pR = from linear interpolation between the above values (11.46)

1. For I and H sections, full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link with a combined width of not smaller than (*b*f – 2*t*w) and a thickness not smaller than 0,8*t*w.
2. Links with I and H sections should be provided with intermediate web stiffeners as given in a) to e):
3. short links should be provided with intermediate web stiffeners at intervals not exceeding (30*t*w – *d*/5) for a link rotation angle **p of 0,08 radians or (52*t*w – *d*/5) for **p of 0,02 rad or less and linear interpolation in-between;
4. long links should be provided with one intermediate web stiffener placed at a distance of 1,5 *b* from each end of the link where a plastic hinge would form;
5. intermediate links should be provided with intermediate web stiffeners which satisfy a) and b);
6. intermediate web stiffeners are not required in links of length *e* greater than 5 *M*p,link/*V*p,link;
7. intermediate web stiffeners should be full depth. For links that are less than 600 mm in depth *d*, stiffeners may be placed on only one side of the link web. The thickness of one-sided stiffeners should not be smaller than *t*w or 10 mm, whichever is the greater, and the width should be not smaller than (*b*/2) – *t*w. For links that are 600 mm in depth or greater, similar intermediate stiffeners should be provided on both sides of the web.
8. Either full penetration or fillet welds may be used to connect a link stiffener to the relevant link. Fillet welds connecting a link stiffener to the link web of I and H sections should be designed to resist the tensile resistance of the web stiffener *ω*rm *f*y *A*st, where *A*st is the cross area of the stiffener parallel to the web. The design resistance of fillet welds fastening the stiffener to the flanges should resist a force at least equal to *ω*rm *A*st*f*y/4.
9. For box sections, full-depth web stiffeners should be provided on one side of each link web at the diagonal brace connection. These stiffeners may be welded to the outside face of the link webs or each link web segments may be welded on full plate stiffeners. They should each have a width not smaller than (*b –*2*t*w)/2 and a thickness not smaller than 0,8 *t*w.
10. Links with box sections should be provided with intermediate web stiffeners satisfying a) and b):
11. for a link rotation angle **p of at least 0,08 rad, short links should be provided with intermediate web stiffeners spaced at intervals not exceeding [20*t*w –(*d*- 2*t*f*)*/8]. Stiffeners may be omitted for link rotation angles **p not greater than 0,02 rad. Linear interpolation should be used for values of **p between 0,08 and 0,02 rad;
12. for long and intermediate links with box sections, stiffeners may be omitted.
13. Fillet welds connecting a link stiffener to the link with box sections should have a design resistance adequate to resist a force *ω*rm *f*y *A*stif, where *A*stif is the area of the stiffener.
14. In DC3 frames, lateral supports should be provided at both the top and bottom link flanges at the ends of the link. End lateral supports of links should have a design axial resistance sufficient to provide lateral support to a force equal to 6 % of the expected axial resistance of the link flange calculated as *ω*rm *b**t*f *f*y.
15. In beams where a seismic link is present, the shear buckling resistance of the web panels outside of the link should conform to prEN 1993-1-5.

### Verification of members and connections not containing seismic links

1. In DC2, the resistance and stability of members not containing seismic links should be verified against the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed from Formula (11.2).
2. In DC3, columns should be designed to resist the total effect *E*Ed determined according to 11.8.5(2), taking the overstrength factor *ω*sh for hardening of links from Table 11.10 and the design overstrength factor *Ω*d as specified in 11.11.2(10).
3. In DC3, the beam segment outside the link, the diagonal bracings and the brace-to-link connections should be verified to resist the effects transferred by the relevant connected seismic link at a rotation of 0,08 rad in combination with the effects induced by the non-seismic actions in the seismic design situation. The link induced effects on the diagonal bracings and their end-connections should be obtained according to a) or b):
4. in short and intermediate links with *e* ≤ 2,6 , from the free-body distribution of the ultimate shear *V*u,link  (*ω*rm *ω*sh *V*p,Link) and the corresponding bending moment *M*u,link  (0,5 *e* *ω*rm *ω*sh *V*p,Link),
5. in long links or intermediate links with *e* > 2,6, from the ultimate moment resistance of the link *M*u,link  *ω*rm *ω*sh *M*p,Link and the corresponding shear force *V*u,link   2*M*u,Link/*e*.
6. In DC2 and DC3, eccentricities in the brace connections away from the link should not be greater than the link depth; the resulting effects in members and connection should be taken into account in their design.

### Verification of connections of the seismic links

1. The shear and bending resistance of the connections of the links should be verified in accordance with Formula (11.4), where *R*fy should be taken equal to *V*p,link, to which corresponds the bending moment 0,5 *e* *V*p,Link  for short and intermediate links. For long links *R*fy should be taken equal to *M*p,link, to which corresponds the shear force 2 *M*p,Link/*e*.
2. In partial strength connections (i.e. those with resistance smaller than the expected ultimate resistance of the link), energy dissipation may be considered to come from the connections only, provided that all conditions in a) to c) are satisfied:
3. the connections ductility is greater than the deformation demands; this should be demonstrated through experimental results or detailed numerical model;

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. members framing into the connections are verified to be stable under the design action effects at SD;
2. the effect of the deformation of the connection on global drift is taken into account.
3. When partial strength bolted or welded connections are used for the seismic links, the capacity design of the other members in the structure should be based on the plastic resistance of the links connections, calculated in accordance with prEN 1993-1-8:2021.
4. In DC3 frames with full strength bolted connections at both ends of short links of length *e* not greater than (*M*p,link*/V*p,link), the connections resistance should not be smaller than a tensile axial force *N*u,link equal to *N*u,link  (2*ω*rm *f*y *b*f*t*f), where *b*f and *t*f are the width and thickness of the link flange, in combination with the ultimate shear resistance *V*u,link =(*ω*rm *ω*sh *V*p,Link) and the corresponding bending moment resistance *M*u,link (0,5 *e* *ω*rm *ω*sh *V*p,Link ), where *ω*sh is equal to 1,8.

### Verification of beam to column connections

1. In braced bays, beam-to-column connections which serve also as connection of a bracing should be designed to transfer the seismic action effects of the diagonal bracing in accordance with 11.11.3(1) in DC2 and with 11.11.3(3) in DC3.
2. Beam-to-column connections may be either continuous or pinned as specified in 11.10.5(2).

## Design rules for frames with buckling restrained bracings

### Design criteria

1. Buckling restrained bracings shall be designed in DC3 so that yielding of the diagonals in both tension and compression occurs before failure of connections and before yielding or buckling of beams or columns.

Key

|  |  |
| --- | --- |
| 1 | “core”, bracing member |
| 2 | “sleeve”, buckling restraining system |
| 3 | longitudinal “gap” |
| 4 | transverse “clearance” |
| A | yielding zone |
| B | transition zones |
| C | connection zones |

Figure 11.17 — Geometrical features and main components of a typical BRB

1. Lateral restraint of the core of the buckling restrained braces should be realized either by an unbonded brace or by an all-steel brace.
2. Buckling restrained bracings should be designed, tested and detailed to accommodate minimum expected deformations corresponding to the greater between an interstorey drift of 2 % of the storey height and two times the design interstorey drift as given in 11.6.4, in addition to brace deformations resulting from deformation of the frame due to non-seismic actions in the seismic design situation.
3. Eccentricities of BRB-to-beam axis and BRB-to-column axis should comply with 11.10.1(3).

### Analysis

1. Compression and tension diagonals should be considered as active in elastic or non-linear analyses.
2. The model expressing the axial stiffness should take into account the axial stiffness of the internal core and that of the terminal tapered plates.

NOTE This can be made with multiple springs or with a constitutive law taking into account several contributions to stiffness.

1. If non-linear analyses are used, the inelastic response of the buckling restrained bracings should be simulated accounting for the initial stiffness, the post-yield stiffness and the differences between tension and compression resistance of the diagonal members. In addition, the hysteretic behaviour of the bracing should be accurately simulated if non-linear time history analysis is used.

### Verification of buckling restrained bracings

1. The yield resistance *N*Rd of the cross-section of the core (segment “A” in Figure 11.17) should satisfy *N*Rd  *N*Ed, where *N*Ed is the axial force in the bracing member in the seismic design situation.
2. At all storey except the top storey, the maximum design overstrength **d,I in the structure should not be greater than the minimum value **d by more than 25 %, where **di is defined by Formula (11.47).

(11.47)

where

|  |  |
| --- | --- |
| *N*Rd,i | is the design axial resistance of the core of the buckling restrained bracing “i”; |
| *N*Ed,i | is the design axial effect in the bracing member “I” in the seismic design situation; |
|  | is the minimum design overstrength, see 11.12.5(1); |
| *N*s | is the total number of storeys. |

1. The compression strength adjustment factor, *γ*CT, should be calculated as the ratio of the maximum resistance in compression to the maximum resistance in tension measured in qualification tests performed in accordance with 11.12.4 at strains corresponding to twice the expected deformations. The greater of *γ*CT values measured in the qualification tests should be used in Formulas (11.49) and (11.51).
2. The strain hardening factor, *ω*sh, should be calculated as the ratio of the maximum tension force measured from the qualification tests at strains corresponding to twice the expected deformations to the measured yield force of the test specimen. The greater value of *ω*sh from qualification tests should be used and should not be greater than 1,50.
3. The overall stability of the sleeve or buckling restraining system may be verified using Formula (11.48).

(11.48)

where

|  |  |
| --- | --- |
|  | is the Eulerian critical load of the sleeve considering as buckling length the node-to-node length of the bracing which includes the length of the portions of beams, columns and gussets connecting the core; |
| *N*Rd | is the design axial resistance of the core of the buckling restrained bracing. |

### Conformity criteria

1. Buckling restrained bracings should be qualified according to tests given in a) and b):
2. EN 15129 tests for displacement-dependent devices;
3. Two cyclic tests satisfying the loading protocol given by prCEN/TS 1998-1-101 one test on a brace sub-assemblage that includes brace connection rotational demand; another test which should be either a uniaxial or a sub-assemblage test.

NOTE The tests defined in EN 15129 establish the tension and compression resistance, the compression strength adjustment factor *γ*CT and the hardening factor *ω*sh.

1. Interpolation or extrapolation of test results for different member sizes should be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that addresses the adverse effects of variations in material properties. Extrapolation of test results should be based on similar combinations of steel core and buckling-restraining system sizes.
2. *γ*CT should not be greater than 1,30.
3. The interstorey drift at SD limit state should be limited to 50 % the value of the maximum interstorey drift imposed by the loading protocol given by prCEN/TS 1998-1-101.

### Verification of beams and columns

1. Beams and columns should be designed to resist the most severe condition between a) and b):
2. the total action effect *E*Ed given by Formula (11.49);

(11.48)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Ed,E | is the seismic action effect in the beam or in the column due to the design seismic action; |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *ω*sh | is the hardening factor of the core based on the results of the qualification tests but not be smaller than 1,2 (see Table 11.10); |
| **CT | is the compression strength adjustment factor based on the results of the qualification tests but not smaller than 1,1 and not greater than 1,3 (see 11.12.3(3)); |
|  | is the minimum value of the design overstrength of all BRBs (see Formula 11.47). |

1. compression diagonals with values equal to their expected ultimate resistance, which may be calculated assuming that the braces develop a compression force equal to *ω*rm *ω*sh **CT *N*Rd and a tension force equal to *ω*rm **sh *N*Rd.
2. In frames with V bracings and in two-storey frames with X bracings, the flexural stiffness of the brace-intersected beams should comply with 11.10.4(6).

### Verification of beam to column connections

1. The beam-to-column connections in the bays equipped with buckling restrained bracings may be either continuous or pinned provided that adequate rotation capacity is verified; they should conform to 11.10.5(2).

NOTE Annex E gives additional rules on beam-to-column joints of frames with buckling restrained bracings.

### Verification of brace connections

1. Brace connections should be designed to resist yielding and hardening of buckling restrained diagonals without any buckling under cyclic loading.
2. In connections providing flexural restraint to the diagonal members in both main directions of their cross section, the resistance and stability of the gusset should be greater than the forces transferred by the connected brace in its non-linear range of behaviour both in-plane and out-of-plane, given by Formulas (11.50) to (11.52).

(11.50)

(11.51)

(11.52)

where

|  |  |
| --- | --- |
| *N*Rd | is the design plastic resistance in tension of the core (segment “A” in Figure 11.17); |
| *M*Rd | is the design plastic moment of the cross section of the tapered segment of the core that is not laterally restrained by the sleeve (segment “C” in Figure 11.17), and it is evaluated in the plan of the frame |
| *ω*rm | is the material overstrength factor (see 11.2.2); |
| *ω*sh | is the hardening factor of the core based on the results of the qualification tests (see 11.12.3(4)) but not smaller than 1,2; |
| **CT | is the compression strength adjustment factor based on the results of the qualification tests but not smaller than 1,1 and not greater than 1,3 (see 11.12.3(3)); |

1. Brace connections conceived as “pinned” should resist a force not smaller than the expected resistance of the connected diagonal bracing as given by Formulas (11.50) and (11.51) and should accommodate the rotation at the design storey drift.
2. To prevent out-of-plane brace buckling, if gusset plate connections are adopted, the resistance of the connection to shear in the transverse direction of the BRB and applied in the theoretical line of restraint, should not be smaller than *V*gp,Ed as given by Formula (11.53).

(11.53)

where is the expected ultimate resistance of the BRB (see Formula (11.51)).

### Verification of column base joints

1. The column base joint should be designed to resist the action effects at the base of the column due to gravity forces and the axial and shear forces transferred by the expected tensile resistance of the connected diagonal brace, which should be taken equal to *ω*rm *ω*sh *N*Rd.
2. Where column bases are designed as moment connections to the foundation, the required flexural resistance should be at least equal to the action effect at the base of the column due to gravity forces and the required flexural resistance of diagonal brace connections, which should be calculated from Formula (11.52).
3. Bending moments at column base connections designed as pinned may be ignored in the analysis.

## Design rules for dual frames - moment resisting frames combined with either concentric, eccentric or buckling restrained bracings

1. In dual structures with both moment resisting frames and braced frames acting in the same direction the horizontal forces should be distributed between the different frames according to their stiffness.
2. The moment resisting frames should contribute with at least 25 % to the total resistance.
3. The moment resisting frames should conform to 11.9 and the braced frames either to 11.10 for concentrically braced frames or 11.11 for eccentrically braced frames or 11.12 for frames with buckling restrained braces.

NOTE DC3-compliant frames can provide re-centring capacity after severe earthquake if the yield deformation at each storey of the moment resisting sub-system *Δ*y,MRF is not smaller than the ultimate storey deformation of the corresponding braced sub-system *Δ*u,Braced (i.e. either CBF, or EBF, or BRB). In these cases, the dissipative components can be designed to be replaceable.

1. In DC3, the resistance and stability of non-dissipative members should be verified in bending, axial and shear forces under the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed calculated with Formula (11.3), where should be taken as the greater of the minimum design overstrength ratios of the moment resisting frames and the braced frames.

## Design rules for lightweight steel systems

### General

1. Lightweight cold-formed steel frames shall be made of cold-formed steel frames braced by steel strap braces (i.e. strap braced walls) or by sheathing panels (i.e. shear walls) connected by means of screws to the steel frame.
2. Walls should be made of studs, i.e. vertical load bearing equally spaced members screwed at each end to tracks, i.e. horizontal steel members, which restrain the studs at their ends.
3. Each wall should be provided at each end with hold-downs to resist in-plane shear forces.

NOTE Hold-downs are connections and shear anchorages used to resist the walls overturning forces.

1. Floor and roof diaphragms should be composed of equally spaced joists, on top of which the sheathing materials should be connected by means of screws along the whole perimeter of the sheathing panel.
2. The sheating material may be a composite steel concrete slab made with profiled steel sheets.

### General verification rules for low-dissipative (DC1) and dissipative (DC2 and DC3) structural behaviour common to all lightweight steel systems

1. The individual resistance of different components of walls such as strap braces, chord studs, tracks, connections and other cold-formed steel components should be determined in accordance with prEN 1993-1-3:2022.
2. The design of composite steel concrete slab made with profiled steel sheets should be made in accordance with prEN 1994-1-1.
3. The design of the connection between wood sheathing and cold-formed steel frame in floor or roof diaphragms, should be made in accordance with prEN 1995-1-1:2023, Clause 8, and prEN 1993‑1‑3:2022, Clause 10.
4. The height-to-length ratio of strap braced walls and shear walls should not be greater than 2,0.
5. Hold-downs should be designed to resist wall overturning at each storey.
6. Shear anchorage should be designed to transmit the in-plane shear force from the horizontal diaphragms into the top of the wall and from the base of the wall into the foundation.
7. Chord studs, or other vertical boundary elements at the ends of strap braced walls and shear walls should be anchored so that the track is not required to resist uplift by bending.
8. Deflections should be determined by considering the deformation of the bracing components (i.e, strap braces, steel sheets, wood sheathings, gypsum sheathings), of the connections of the bracing component and of their chord studs.
9. The effect of eccentricity on connections, chord studs or other vertical boundary elements at the ends of wall, hold-downs and shear anchorage and all other components in the wall should be considered in the design.
10. Screws used to make connections between sheathing material and cold-formed steel frame or connections between cold-formed steel frame elements should belong to one of the types allowed in prEN 1993-1-3.

NOTE The types allowed in prEN 1993-1-3 are: the EN ISO 1478 Tapping screws thread; the EN ISO 1479 Hexagon head tapping screws; the EN ISO 2702 Heat-treated tapping screws; the EN ISO 7049 Cross recessed pan head tapping screws.

1. The pull-out resistance of screws should be not used to resist seismic forces.
2. For connections of strap braces, the design shear resistance of the screws should be greater than 1,2 times the design bearing resistance or the design net area resistance of the strap brace.
3. For connections of steel sheet sheathing, the design shear resistance of the screws should be greater than 1,2 times the design bearing resistance.
4. For connections of wood or gypsum sheathing, the design shear resistance of the screws should be greater than 1,2 times the design bearing resistance of the steel structural member or the design embedment resistance of wood or gypsum sheathing.
5. For strap braced walls, where the track is not designed to resist the horizontal shear force from the strap braces by compression or tension, the horizontal shear force should be resisted by a device connected directly to the strap braces and anchored directly to the foundation or supporting structural member.

### Additional verification rules for dissipative (DC2 and DC3) structural behaviour common to all lightweight steel systems.

1. In DC2, non-dissipative components should be designed to resist the action effect *E*Ed calculated with Formula (11.54):

(11.54)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Ed,E | is the seismic action effect due to the design seismic action; |
| *Ω* | is the seismic action magnification factor, see Table 11.9. |

1. In DC3, non-dissipative components should be designed to provide adequate overstrength with respect to the expected resistance of the dissipative members in accordance with Annex F.

### Specific verification for dissipative (DC2 and DC3) strap braced walls

1. The mechanism of dissipative behaviour in a strap braced wall should be the tension yielding of the strap braces. Other failure mechanisms should be avoided.
2. Strap braced walls should be designed to resist in-plane lateral forces mainly with tension-only steel straps applied diagonally to form a vertical truss and should be provided with chord studs or other vertical boundary elements, hold-downs and shear anchorage at each end of the wall.
3. The yield resistance *N*pl,Rd of the gross cross-section of the strap braces should be such that *N*pl,Rd ≥ *N*Ed, where *N*Ed is the design value of the axial force in the strap brace in the seismic design situation, and should not be smaller than *N*u,Rd, the design net area resistance of the strap brace.

NOTE Complementary design rules are given in Annex F, F.3.

### Specific verification for dissipative (DC2 and DC3) shear walls with steel sheet sheathing

1. In shear walls with steel sheet sheathing, the dissipative components should be the member-to-sheathing connections.
2. The design value of in-plane lateral resistance *R*c,Rd corresponding to the resistance of the member-to-sheathing connections within the effective sheathing strip should not be smaller than *F*Ed where *F*Ed is the design value of the lateral force acting on the shear wall in the seismic design situation, and should not be greater than *R*y,Rd, the design value of the in-plane lateral resistance corresponding to the yielding resistance of the effective sheathing strip.
3. The resistance of the single member-to-sheathing connection *F*c,Rd should satisfy Formula (11.55).

(11.55)

where *F*v,Rdand *F*b,Rd are the design shear resistance of the screws and the design bearing resistance of the member-to-sheathing connection, respectively, calculated as given in Annex F.4.

NOTE Complementary design rules are given in Annex F.4.

### Specific verification for dissipative (DC2 and DC3) shear walls with wood sheathing

1. In shear walls with wood sheathing, the structural member-to-wood sheathing connections should be the dissipative components.
2. The design value of in-plane lateral resistance *R*c,Rd corresponding to the resistance of the member-to-wood sheathing connection should not be smaller than *F*Ed where *F*Ed is the design value of the lateral force acting on the shear wall in the seismic design situation.

NOTE Complementary design rules are given in Annex F.5.

### Specific verification for dissipative (DC2 and DC3) shear walls with gypsum sheathing

1. In shear walls with gypsum sheathing, the structural member-to-gypsum sheathing connections should be the dissipative components.
2. The design value of in-plane lateral resistance *R*c,Rd corresponding to the resistance of the member-to-gypsum sheathing connection should not be smaller than *F*Ed where *F*Ed is the design value of the lateral force acting on the shear wall in the seismic design situation.

NOTE Complementary design rules are given in Annex F.6.

## Verification of inverted pendulum structures

1. In inverted pendulum structures (defined in 11.4.1(1)g)), the columns should be verified in compression considering the most unfavourable combination of the axial force and bending moments.
2. In the verifications, *N*Ed, *M*Ed*, V*Edshould be calculated as given in 11.8.5.
3. The geometrical slenderness of the columns should not exceed 70, as given in 11.9.3(6) The non-dimensional slenderness 𝜆̅ of columns should not be greater than 0,85.
4. The interstorey drift sensitivity coefficient defined in 11.6.2(2) should be not greater than 0,20.

## Design rules for steel structures with concrete cores or concrete walls and for moment resisting frames combined with infills

### Structures with concrete cores or concrete walls

1. Concrete wall equivalent systems which are dual with a steel structure and conform to the definition in 10.4.1(d), (e) or (f) should be considered as satisfying 6.2.6(7).
2. The steel members should be verified in accordance with the relevant parts of prEN 1993, while the concrete members should be designed in accordance with Clause 10.
3. Members in which there is an interaction between steel and concrete should be verified in accordance with Clause 12.

### Moment resisting frames combined with infills

1. Moment resisting frames in which reinforced concrete infills are connected to the steel structure in order to behave as a composite structure should be designed in accordance with 7.4 and Clause 12.
2. The moment resisting frames in which the infills are structurally disconnected from the steel frame on the lateral and top sides should be designed as steel structures.
3. The moment resisting frames in which the infills are in contact with the steel frame, but are not positively connected to that frame, should be designed to 7.4.

## Steel diaphragms

1. The design of floor diaphragms should conform to 6.2.8.
2. Chord and collector beams should be designed to resist the in-plane diaphragm forces.
3. In a floor truss used as a diaphragm, all members of the truss and their connections should be designed to resist the design action effects resulting from the load combination as given in prEN 1998-1-1:2022, 6.4.4, where the seismic action effects should be amplified by the overstrength factor *γ*d given in 6.2.8 and taken equal to the recommended value for brittle failure modes.
4. If there are shear connectors along all girders of the diaphragm, the collector beams and the corresponding connections should be designed to resist the axial forces due to distributed shear forces of the shear connectors.
5. If shear connectors are not present on the collector beams, all shear forces should be resisted by the beams of the seismic resisting frame.

## Transfer zones. Design for DC2 and DC3

1. Diaphragms in transfer zones should be designed to 6.2.11 and 11.17.
2. In DC2, **d in Formula (6.6) should be taken equal to , which is the seismic action magnification factor in the dissipative zones (see Table 11.9) to be applied to the effects (*N*Ed,E, *M*Ed,E and *V*Ed,E) of the design seismic action In DC2, the seismic action effects to be considered in the application of Formula (6.6) should be taken as given in Formula (11.2).
3. In DC3, **d in Formula (6.6) should be taken as , where *ω*rm is the material overstrength factor (see 11.2.2) In DC3, the seismic action effects to be considered in the application of Formula (6.6) should be taken as given in Formula (11.3).

## Requirements for supply of material and execution

1. The required toughness of steel and of welding consumables and the lowest service temperature adopted in combination with the seismic action should be defined in the design documents.
2. The grade of steel in the dissipative zones, the grade and the tightening force of bolts and the quality of the welds should be indicated in the drawings for fabrication.
3. The technical drawings and reports should clearly state that steel grade higher than specified should not be used for the dissipative zones.
4. For Consequence Classes 1, 2 and 3a and all ductility classes, the Execution Class should not be lower than EXC2 for all parts of the primary structure, except pre-qualified connections, if adopted, and with the exception in (6). The Execution Class for Consequence Class 3b should be EXC3.
5. For Consequence Classes 1, 2 and 3a, if a behaviour factor *q* = 1,0 is used in design, the Execution class should not be lower than EXC2 for all parts of the primary structure, with the exception in (6).
6. If *S*δ is not greater than 2,5m/s2, the execution class of structures of Ductility Class 1 and of structures designed with *q* =1 may be EXC1.

# Specific rules for composite steel–concrete buildings

## General

1. Clause 12 should be applied to the design and the verification of composite steel-concrete buildings, in seismic regions.
2. Clause 12 should be applied as a complement to prEN 1994-1-1.
3. Except where modified by the provisions of Clause 12, Clauses 10 and 11 should be applied where applicable.

## Basis of design

### Design concepts

1. Earthquake resistant composite steel-concrete buildings should be designed in accordance with one of the three Ductility Classes introduced in prEN 1998-1-1:2022, 4.4.2(3) (see Table 12.1).

Table 12.1 — Structural ductility class (DC) and upper limit reference values of the behaviour factor

|  |  |
| --- | --- |
| **Structural ductility class** | **Range of the behaviour factor *q*** |
| DC1 |  |
| DC2 |  |
| DC3 | only limited by the values of Table 12.2 |

1. In DC1, the seismic design of composite steel-concrete buildings should be verified according to 12.7.
2. Composite steel-concrete buildings in DC2 and DC3 may be designed with upper limit values of and , which depend on the Ductility Class and the structural type according to 12.4.
3. In DC2 and DC3, dissipative zones of composite steel-concrete buildings may resist earthquake actions through inelastic behaviour. Dissipative zones should be designed according to 12.8 to 12.14 and 12.16.

### Safety verifications

1. 11.2.2 should be applied.
2. Partial factors and for reinforced concrete should conform to 10.3.4.
3. The partial factor for shear connectors should conform to prEN 1994-1-1:—, 4.4.1.2(5).

## Materials

### Concrete

1. In design for DC1, 10.3.2(1) and prEN 1994-1-1 should be applied.
2. In design for DC2 and DC3, 10.3.3(1) should be applied.

### Reinforcing steel

1. 10.3.1(1) and 10.3.3(2) should be applied.

### Structural steel

1. 11.3 should be applied.
2. When the dissipative zones occur in composite cross sections, either beam or columns, the steel grade of these components should not be higher than S460. This requirement includes beams with partial concrete encasement.

## Structural types, behaviour factors, limits of seismic action and limits of drifts

### Structural types

1. Composite steel-concrete buildings designed to be dissipative should be assigned to one of the structural types in a) to g), according to the behaviour of their primary resisting structure under seismic actions:
2. Composite moment resisting frames, are those with the same definition and limitations as in 11.4.1(1)a, but in which beams and/or columns may be either structural steel or composite steel-concrete (see Figure 11.1) with full- or partial- strength beam-to-column connections. If columns are structural steel, the limitations as in 11.4.1(2) should be applied. Partial-strength beam-to-column connections are these that connect fully composite beams to structural steel columns with flexural resistance provided by a force couple achieved with continuous steel reinforcement in the slab and a steel seat angle or comparable connection at the bottom flange connection.
3. Composite frames with concentric bracings, are those with the same definition and limitations as in 11.4.1(1)b, 11.4.1(3), 11.4.1(4), 11.4.1(5), 11.4.1(6) and Figures 11.2 and 11.3. Columns and beams may be either structural steel or composite steel-concrete. Braces should either be structural steel or filled composite.
4. Composite frames with eccentric bracings, are those with the same definition and limitations as in 11.4.1(1)c and Figure 11.4. The members, which do not contain the links may either be structural steel or composite steel-concrete. Other than the slab, the links should be structural steel. Energy dissipation should occur only through yielding in bending and/or in shear of these links.
5. Composite frames with buckling restrained bracings, are those with the same definition and limitations as in 11.4.1(1)d and Figure 11.5. Columns and beams may either be structural steel or composite steel-concrete. Buckling restrained braces should be such that they are active both in tension and compression without experiencing local and/or global buckling, so that energy is dissipated in the braces by means of cyclic axial force.
6. Dual composite frames have the same definition and limitations as in 11.4.1(1) e) (see Figure 11.6).
7. Inverted pendulum structures have the same definition and limitations as in 11.4.1(1) g) (see Figure 11.8) and 11.4.1(7).
8. Composite structural wall systems are those which behave essentially as reinforced concrete walls. The composite wall systems may belong to one of the following types:
9. Type 1 corresponds to a steel or composite moment resisting frame working together with concrete infill panels connected to the steel frame (see Figure 12.1a).
10. Type 2 is a reinforced concrete wall in which encased structural steel cross sections connected to the concrete structure are used as vertical reinforcement (see Figure 12.1b an example with edge reinforcement).
11. Type 3, in which steel or composite steel beams are used to couple two or more reinforced concrete or composite walls (see Figure 12.1c).

Figure 12.1 — Composite structural wall systems: (a) Type 1 – steel or composite moment frame with connected concrete infill panels; (b) Type 2 – composite walls reinforced by connected encased vertical steel sections; (c) Type 3 – composite or concrete walls coupled by steel or composite beams

1. Composite moment resisting frames with partial-strength beam-to-column connections may be used in DC1. When designed as dissipative, composite moment resisting frames with partial-strength beam-to-column connections should only be designed in DC2 and should also be verified against global hierarchy rules in accordance with 11.9.3(2) and 6.2.7(2).
2. In composite structural wall systems of Types 1 and 2, the energy dissipation should take place in the vertical steel sections and in the vertical reinforcement of the walls. In composite structural wall systems of Type 3, energy dissipation may also take place in the coupling beams.
3. If in composite structural wall systems, the wall elements are not connected to the steel structure, Clauses 10 and 11 should be applied, respectively, for concrete and steel members.
4. prEN 1998-1-1:2022, 4.1(7), may be used in design within the limitations given in 12.6.4(1) f) and 12.7.1 and 12.7.3.
5. Structural systems which cannot be assigned to one of the structural types in (1) may be used; they should be designed for strength according to 12.7.3.

### Behaviour factors

1. The behaviour factor should account for the relevant sources of overstrength, the deformation and energy dissipation capacity, of the structural types of 12.4.1. For regular buildings, the behaviour factor components and may be taken with the upper limit values of Table 12.2 provided that 12.8 to 12.14 are applied.
2. 11.4.2(2) should be applied.
3. prEN 1998-1-1:2022, 4.1(7), may be used of in the design of structural systems which cannot be assigned to one of the structural types in 12.4.1(1) a) to g).

Table 12.2 — Default upper limit values of behaviour factors for composite steel concrete systems regular in elevation

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
|  | **Ductility Class** | | | | | | |
| **Structural type** | **DC2** | | | | **DC3** | | |
|  |  |  |  |  | |  |  |
| a) Composite moment resisting frames (CMRFs)  Full-strength connections  Portal frames and single-storey CMRFs with class 3 and 4 cross sections  Portal frames and single-storey CMRFs with class 1 and 2 cross sections  Multi-storey CMRFs  Types above without or with unconnected interacting infills | See Table 11.2 | | | | | | |
| b) CMRFs with dissipative partial-strength connections | 2,3 | 1,0 | 3,5 | - | | - | - |
| c) Composite frames with concentric bracings | See Table 11.2 | | | | | | |
| d) Composite frames with eccentric bracings | See Table 11.2 | | | | | | |
| e) Composite inverted pendulum | See Table 11.2 | | | | | | |
| f) Composite dual frames  CMRFs with concentric bracings  CMRFs with eccentric bracings  CMRFs with buckling restrained braces | See Table 11.2 | | | | | | |
| g) Composite structural wall systems  Composite walls (Type 1 and 2)  Composite or concrete walls coupled by steel or comp. beams (Type 3) | 2,2  2,2 | 1,0  1,0 | 3,5  3,5 | 3,0  3,3 | | 1,0  1,0 | 4,5  5,0 |

NOTE The values of *q*R and *q*D in Table 12.2 are default values in the sense of 5.3.2 and the values of *q* are obtained as the product of the default values of *q*D and *q*R with *q*S equal to 1,5.

### Limits of seismic action for design to DC1, DC2 and DC3

1. For each structural type, design to a given Ductility Class should not be made above the levels of the seismic action index given in Table 12.3.

Table 12.3 — Limits of seismic action index for design to DC1, DC2 and DC3

| **Structural type** | **Limits of seismic action index**  **(m/s2)** | | |
| --- | --- | --- | --- |
|  | **DC1** | **DC2** | **DC3** |
| Composite moment resisting frames (full strength connections) | 5,0 | 6,5 | No limits |
| Composite moment resisting frames (full strength connections) with unconnected interacting infills moment resisting frames | 5,0 | 6,5 | No limits |
| Composite moment resisting frames (partial-strength connections) | 5,0 | 6,5 | Not permitted |
| Composite frames with concentric or eccentric bracings | 5,0 | 6,5 | No limits |
| Composite frames with buckling-restrained braces | - | - | No limits |
| Composite dual frames | 5,0 | 7,5 | No limits |
| Composite inverted pendulum | 2,5 | 5,0 | No limits |
| Composite structural wall systems | 5,0 | 7,5 | No limits |

## Structural analysis

### General

1. 11.5(1) should be applied.
2. In composite moment resisting frames designed with joints as those specified in Annex G, the composite joint deformations should be considered in the lateral deflection calculations.
3. In composite moment resisting frames designed with dissipative partial-strength connections, the joint deformations should be considered in the lateral deflection calculations.

### Stiffness of sections

1. The stiffness of composite cross sections in which the concrete is in compression should be calculated using the modular ratio *n*0 in application for short duration loading according to prEN 1994‑1‑1:—, 5.4.2.2(2). In this case, Formula (12.1) should be applied. The stiffness of composite cross sections in which the concrete is in compression should be calculated using the modular ratio *n*0 in application for short duration loading according to prEN 1994-1-1:—, 7.4.2.2(2) or according to Formula (12.1).

(12.1)

1. For composite steel beams under positive bending, the second moment of area of the cross section, , should be calculated based on the effective width of the slab according to prEN 1994-1-1:—, 7.4.1.2.
2. The stiffness of composite beam cross sections in which the concrete is in tension should be calculated by assuming that the concrete is cracked and that only the steel portion of the cross section is active. The contribution of the longitudinal steel reinforcement may be considered.
3. The structural model should take into account that concrete is in compression in some zones and in tension in other zones.

NOTE These zones are defined in 12.9 to 12.14.

1. For stiffness calculations, the total effective width of the concrete flange associated with each steel beam web should be taken as the sum of the partial effective widths *b*e1 and *b*e2 from each side of the steel beam web (see Figure 12.2) and should be calculated according to prEN 1994-1-1:—, 7.4.1.2; *b*eff should not be greater than the actual available widths *b*1 and *b*2 defined in (6).

Figure 12.2 — Definition of effective width *b*e and *b*eff

1. The actual width, *b*1 or *b*2 from each side of the steel web should be taken as half the distance from the web to the adjacent web; at a free edge, the actual width should be the distance from the web to the free edge.

## Verification to limit states

### General

1. The verification of structural members to limit states should comply with 6.1, 6.2 and 6.3.

### Verifications at Significant Damage limit state in a force-based approach

1. The resistance of structural members and connections at Significant Damage should be verified using EN 1993-1-1, prEN 1993-1-8 and prEN 1994-1-1.

### Verifications at Significant Damage limit state in a displacement-based approach

1. Resistance of ductile mechanisms should be verified to SD and NC in terms of local generalized deformations , according to case a) of prEN 1998-1-1:2022, 6.7.1(2).
2. For members following the design rules to this standard, deformation criteria should be taken as in prEN 1998-1-1:2022, 6.7.1(3)a).
3. For members following the design rules in this standard, total logarithmic standard deviation of the resistance model required to calculate the partial factors for the verification to SD and NC (see prEN 1998-1-1:2022, 6.7.2 and 6.7.3) according to 6.2.3(3) should be taken as given in Table 11.4 and Table 11.5.

### Limitation of interstorey drift at Significant Damage limit state

1. The interstorey drift at SD limit state of composite steel-concrete buildings should be limited to:
2. for composite moment resisting frames with full - or partial-strength connections and for composite dual frames;
3. for composite moment resisting frames with interacting infills, 7.4.2.1 should be applied;
4. for composite frames with concentric or eccentric bracings and for inverted pendulum structures;
5. for composite frames with buckling restrained bracings see 11.12.4(5);

1. for composite structural wall systems of Type 1, 2 and 3.
2. for all other composite structural systems

where

|  |  |
| --- | --- |
| *d*r,SD | is defined in 6.2.4; |
| *h*s | is the interstorey height. |

## Design rules for low-dissipative (DC1) and non-dissipative structural behaviour for all structural types

### General

1. 11.7.1 should be applied.

### Design rules for low-dissipative structures

1. 11.7.2(1) should be applied.

### Design rules for non-dissipative structures

1. Non-dissipative structures should be designed to resist seismic actions in the elastic range and their behaviour factor *q* should be taken equal to 1,0.

NOTE Non-dissipative design is used for structures for which a ductile behaviour can hardly be conceived, e.g. composite domes, or cannot be supported by research reference. It is also used for specific structural systems for which non-dissipative design can be safe and more economical.

1. prEN 1998-1-1:2022, 4.1(7), may be used in the design of single-storey buildings; the limits of seismic action index *S*δ for their design to DC1 in 12.4.3 may be waived, provided that the condition in (4) and in 12.6.4 are satisfied.
2. prEN 1998-1-1:2022, 4.1(7), may be used without limitation of the seismic action index *S*δ in the design of structural systems which cannot be assigned to one of the types in 12.4.1(1) a) to g), provided that the condition in (4) is satisfied.
3. The connections should be designed for action effects calculated for the return period *T*LS,CC at the NC limit state given in Table 4.3 (when using prEN 1998-1-1, 4.3(3)) or the performance factor *γ*LS,CC at the NC limit state given in Table 4.4 (when using prEN 1998-1-1, 4.3(5)).

## Design rules for dissipative (DC2 and DC3) structural behaviour common to all structural types

### General

1. Composite members participating into energy dissipation should conform to prEN 1994-1-1.
2. The design criteria given in 12.9 to 12.14 should be applied to all the components of composite steel-concrete structural types designed in accordance with the concept of dissipative structural behaviour.
3. 12.8.2 to 12.8.9 should be applied to members of the primary structural types of 12.4 designed for dissipative behaviour.

### Design criteria for dissipative structures

1. Dissipative zones in composite steel-concrete structural types shall have adequate ductility and resistance such that their hysteretic behaviour does not affect the overall stability of the structure.
2. 11.8.2 should be applied.
3. The resistance of dissipative zones solely relying on the bare steel cross-section should be determined in accordance with Clause 11.

### Verification of dissipative members in compression or bending

1. For dissipative steel members, 11.8.3 should be applied.
2. For dissipative composite members under compression and/or bending, sufficient local ductility should be ensured by restricting the local slenderness of their cross sections.
3. Steel dissipative zones and the non-encased steel parts of composite members should conform to 11.8.3(1) and the local slenderness limits in Table 11.8 depending on their ductility class and the selected behaviour factor .
4. Dissipative zones of encased or filled composite members under compression and/or bending should conform with Table 12.4. Complementary design and detailing rules according to 12.8.7, 12.8.9 and 12.9 to 12.14 should be applied depending on the dissipative composite member.

Table 12.4 — Local slenderness limits of encased or filled composite cross sections in dissipative members depending on Ductility Class and reference behaviour factor

|  |  |  |  |
| --- | --- | --- | --- |
| **Ductility class** | **DC2** | | **DC3** |
| **Reference value of behaviour factor** | *q* = 1,5 | 1,5 *q* 3,5 | *q* 3,5 |
| Partially or fully encased H- or I-cross section: limits: |  |  |  |
| Filled rectangular cross section: limits: |  |  |  |
| Filled circular cross section: limits: |  |  |  |

where

|  |  |
| --- | --- |
|  |  |
| fy | is the nominal yield stress of the steel material; |
|  | is as defined in Figure 12.3(a); |
| and | are the ratio between the maximum external dimension-to-the wall thickness of the steel profile as defined in Figure 12.3(b) and (c). |

Key

|  |  |
| --- | --- |
| A | straight links |

Figure 12.3 — Definitions of local slenderness limits of encased or filled composite cross sections in dissipative composite members

### Verification of dissipative members in tension

1. For dissipative steel members or composite members or their parts in tension, the ductility provisions in 11.8.4 should be applied.

### Verification of members in DC2 and DC3

1. In DC2, the resistance and stability of members defined by structural type in Table 12.5 and of the connections of these members should be verified by considering the most unfavourable combination of the axial force *N*Ed, bending moments *M*Ed and shear force *V*Ed, calculated according to Formula (11.2). The seismic action magnification factor in Formula (11.2) should be taken as given in Table 12.5 and 11.8.5(1).
2. In DC3, the resistance and stability of non-dissipative members should be verified considering the most unfavourable combination of the axial force *N*Ed, bending moment *M*Ed and shear force *V*Ed, calculated according to Formula (11.3) and 11.8.5(2).
3. In DC3, the design overstrength factor, *Ω*d should not be smaller than 1,0. It should depend on the structural type as specified in 12.9 to 12.14. The non-dissipative members to which Formula (11.3) should be applied with 11.8.5(2) are those defined in Table 12.5.
4. 11.8.6 should be applied.
5. In DC3, composite structural wall systems should be designed with a seismic action magnification factor *Ω*sh *Ω*rm *Ω*dequal to 2,0.

Table 12.5 — Members to which (1) or (2) should be applied. Values of seismic action magnification factor *Ω* for structural types in DC2

|  |  |  |
| --- | --- | --- |
| **Structural types** | ***Ω*** | **Members** |
| **Composite moment resisting frames (CMRFs):**  With full-strength connections | See Table 11.9 | Columns |
| With partial-strength connections | 2,0 | Columns |
| **Composite frames with concentric bracings** | See Table 11.9 | Beams and columns |
| **Composite frames with eccentric bracings** | See Table 11.9 | Beams outside the link, braces and columns |
| **Composite inverted pendulum structures** | See Table 11.9 | Columns |
| **Composite dual frames** | See Table 11.9 |  |
| CMRFs with concentric bracings | Beams and columns of the concentric bracing; columns of the CMRF |
| CMRFs with eccentric bracings | Beams outside the link, braces and columns of the eccentric bracing; columns of the CMRF |
| **Composite structural wall systems** | 2,0 | None |
| Types 1, 2 and 3 | None |

### Verification of beams

#### General

1. 12.8.6 should be applied to dissipative composite or steel beams in composite structural types of 12.4.
2. Secondary beams in all composite structural types of 12.4 should be either designed with EN 1993‑1-1 or with prEN 1994-1-1, whichever applies.
3. In DC2 and DC3, composite steel beams with slab (see 12.8.6.2) and concrete-encased composite beams (see 12.8.6.3) should be braced laterally according to prEN 1994-1-1; alternatively, the beam cross section should be braced with point torsional bracing.
4. In DC2, the spacing and resistance of lateral-torsional restraints of composite steel beams with slab, concrete-encased composite beams or bare steel beams, should conform to EN 1993-1-1:2022, 8.3.5.3.
5. In DC3, the distance between lateral-torsional restraints in dissipative composite steel beams should not be greater than the stable length *L*st given by Formula (12.2).

(12.2)

where

|  |  |
| --- | --- |
|  | is the radius of gyration in the plane of buckling calculated based on the elastic transformed composite cross section by using the modular ratio according to Formula (12.1); |
|  | is the elastic modulus of steel; |
|  | is the steel material overstrength factor (see 11.2.2). |

1. In DC3, for dissipative composite steel beams satisfying 12.8.6.2.3, 11.9.2(3) should be applied.
2. In DC3, 11.9.2(4) should be applied for dissipative composite steel or bare steel beams of composite structural systems.
3. Additional transverse bracing should be placed adjacent to the expected dissipative zones of composite steel or steel beams of composite structural types in DC2 and DC3, where required in 12.9 to 12.14.
4. In DC3, the resistance *M*b of torsional bracing adjacent to plastic hinges in dissipative zones of composite steel beams with slab or concrete encased-composite beams should not be smaller than the value given by Formula (12.3).

(12.3)

where *M*pl,Rd is the plastic moment of resistance of the composite steel beam with slab or the concrete encased composite beam which may be calculated as given in prEN 1994-1-1:—, 8.2.1.

#### Composite beams: steel beams with slab

##### General

1. The integrity of the concrete slab shall be maintained in the seismic design situation, while yielding takes place in the bottom part of the steel cross section and/or in the reinforcing rebars of the slab.
2. If it is not intended to use a dissipative composite beam with slab, 12.8.6.2.3 should be applied. In this case, the composite steel beam should be designed in accordance with Clause 11 for the seismic design situation.
3. Composite beams in dissipative zones of the structure may be designed with full or partial shear connections in accordance with prEN 1994-1-1:—, 8.6.3. The degree of connection *η* as defined in prEN 1994-1-1:—, 8.6.3.3, should not be smaller than 0,8. The total resistance of the shear connectors within any negative (hogging) moment region should not be less than the plastic resistance of the longitudinal reinforcement. The shear connectors should be placed outside the plastic hinge region in the steel beam.
4. The design resistance of shear connectors in composite beams with slab should conform to prEN 1994-1-1:—, 8.6.8 and 8.6.9. For dissipative composite beams with slab with steel beam cross sectional depths greater than 500mm, the design resistance of the shear connectors should be multiplied by a reduction factor of 0,75.
5. When connectors of ductility category D0 or D1, according to Table 5.1 of prEN 1994-1-1:—, are used, the connection should be designed according to prEN 1994-1-1:—, 8.6.4.
6. When a profiled steel sheeting with ribs transverse to the supporting beams is used, the reduction factor of the design shear resistance of connectors given by prEN 1994-1-1:—, 8.6.9.2, should be further multiplied by the rib shape efficiency factor, ,given in Figure 12.4.

Figure 12.4 — Values of the rib shape efficiency factor

1. To achieve ductility in plastic hinges forming in dissipative zones under positive bending, the ratio zc/*d*cof the plastic neutral axis depth *z*c to the depth *d*c of the composite beam cross section, should satisfy Formula (12.4).

(12.2)

where

|  |  |
| --- | --- |
|  | is the ultimate compressive strain of concrete by considering confinement ; |
|  | is the total strain of the steel profile at SD. |

1. To comply with (7), Table 12.6 may be used for limiting zc/*d*c of composite beams with slab.
2. In dissipative zones of composite beams, seismic rebars should be present in the connection zone of the beam and the column. The seismic rebars should be transverse to the steel beam and be designed and placed according to Annex I.

NOTE “Seismic rebars” are specific ductile steel reinforcement of the slab.

Table 12.6 — Limit values of zc/dc for ductility of steel beams with slab

|  |  |  |  |
| --- | --- | --- | --- |
| **Ductility class** | ***q*** | **(N/mm2)** | **zc/*d*c**  **upper limit** |
| DC2 | 1,5 < *q*  3,5 | 355 | 0,45 |
| 275 | 0,50 |
| 235 | 0,55 |
| DC3 | 3,5 < *q*  5,0 | 355 | 0,35 |
| 275 | 0,40 |
| 235 | 0,45 |
| *q* > 5,0 | 355 | 0,30 |
| 275 | 0,35 |
| 235 | 0,40 |

##### Slab effective width for plastic bending resistance calculations

1. The partial effective width of the slab *b*eff,Rd to be used for the calculation of the plastic bending resistance of composite steel beams with slab should be taken as defined in Table 12.7.

NOTE In Table 12.7, the *b*eff,Rd values are valid for primary composite steel beams with slab positioned with their strong axis in the direction of the seismic action.

Table 12.7 — Partial effective width *b*eff,Rd of slab for evaluation of plastic bending resistance, of composite steel beams with slab designed according to Annex I

|  |  |  |  |
| --- | --- | --- | --- |
| **Stresses in slab** | **Column location** | **Transverse element** | ***b*eff,Rd** |
| Tensile | Interior | With seismic re-bars | *b*eff |
| Tensile | Exterior | With re-bars anchored to façade beam or to concrete cantilever edge strip. With seismic rebars | *b*eff |
| Tensile | Exterior | With re-bars not anchored to façade beam or to concrete cantilever edge strip. With seismic rebars | 0,0 |
| Compressive | Interior/Exterior | Transverse beam with connectors and rigidly connected to column. With seismic re-bars | *b*eff |
| Compressive | Interior/Exterior | No transverse beam with connectors. With seismic re-bars | *b*c+0,7 |
| Compressive | Exterior (perimeter frame) | No transverse beam with connectors. With seismic re-bars | *b*c |

where

|  |  |
| --- | --- |
| *b*eff | is the effective width defined in prEN 1994-1-1; |
| *b*c | is the column width intersecting the composite steel beam with slab; |
| *h*c | is the depth of the respective column. |

##### Condition for disregarding the composite action of steel beams with slab

1. If the slab is totally disconnected around a column from the steel beam in a circular zone of diametre 2 *b*eff where *b*eff is the largest of the effective widths of the steel beams intersecting to the respective column, the plastic moment resistance of a composite steel beam with slab may be calculated based on the steel profile alone.
2. The slab should be considered as totally disconnected when there is no contact between the slab and any steel element in the transverse direction (e.g. columns, shear connectors, connecting plates, corrugated flange, steel deck nailed to flange of steel section).

#### Concrete-encased composite beams

1. In partially concrete-encased composite steel beams with slab, the contribution to plastic bending resistance of concrete between the flanges of the steel profile may be neglected.

### Verification of composite columns

#### General rules

1. In the design of all composite column types in 12.8.7.1, 12.8.7.2, 12.8.7.3 and 12.8.7.4, the resistance of the steel section alone or the combined resistances of the steel section and the concrete encasement or infill may be taken into account.
2. Encased composite columns with dissipative behaviour should have minimum cross-sectional dimensions *b*, *b*c*, h*, *h*cor *d* (see Figure 12.3) of at least 250 mm.
3. The axial, shear and flexural resistance of non-dissipative encased and filled composite columns should be determined in accordance with prEN 1994-1-1:—, 8.8.
4. The design of encased composite columns in which the member resistance is only provided by the steel section may be carried out in accordance with 11.
5. The design shear resistance of composite columns calculated as given in prEN 1994-1-1:—, 8.2.2.2, should be multiplied by a reduction factor of 0,5.
6. In dissipative encased or filled composite columns, when the concrete encasement or infill are assumed to contribute to the axial and/or flexural resistance of the member, 12.8.7.2 to 12.8.7.4 should be applied. The full shear transfer between the concrete and the steel parts in a cross section should be ensured by bond, friction and/or shear connectors.
7. In dissipative encased composite columns, when the concrete encasement or infill do not contribute to the axial and/or flexural resistance of the member, the transverse shear resistance should be determined on the basis of the structural steel cross section alone.
8. Wherever a composite column is subjected to predominant axial forces, sufficient longitudinal shear transfer should be provided to ensure that the steel and concrete parts share the loads applied to the column at connections to beams and/or bracing members.
9. At both ends of a column, the clear length should be defined as being in critical regions of columns in composite moment resisting frames. In composite frames with eccentric bracings, the portion of columns adjacent to the seismic links should be defined as a critical region.

NOTE Except at their base in some composite structural types, columns are not designed to be dissipative; (6) and (9) are prescribed to mitigate uncertainties in the action effects in columns.

#### Encased composite columns

1. The steel contribution ratio, δ of encased composite columns should conform to prEN 1994-1-1:—, 8.8.1(4).
2. Concrete encasement of the steel core should be reinforced with continuous longitudinal rebars and lateral ties or stirrups.
3. In DC2 and DC3, the critical region length, *l*cr (in mm) of fully encased composite columns should be taken as given in Formula (12.5) or Formula (12.6), as appropriate.

In DC2: (12.5)

In DC3: (12.6)

where

|  |  |
| --- | --- |
| *h*c | is the depth of the composite cross section (in mm); |
| *d*bl | is the clear length of the composite column (in mm). |

1. If , the full height of the encased composite column should be considered as a critical region.
2. The spacing *s* (in mm) of confining hoops in critical regions should be taken as given in Formula (12.7) or Formula (12.8), as appropriate.

In DC2: (12.7)

In DC3: (12.8)

where

|  |  |
| --- | --- |
| *b*oc | is the minimum dimension of the concrete core to the center line of the confining hoops (in mm); |
| *d*bl | is the minimum diameter of the longitudinal rebars (in mm). |

1. In DC3, the spacing *s* (in mm) of confining hoops near the column base should satisfy Formula (12.9).

(12.9)

1. The confining hoop diameter *d*bw (in mm) should not be smaller than the value given in Formula (12.10) or Formula (12.11), as appropriate.

In DC2: (12.10)

In DC3: (12.11)

1. In dissipative fully encased composite columns of composite structural types in DC2 and DC3, the confining hoop diameter *d*bw (in mm) should satisfy Formula (12.12).

(12.12)

where

|  |  |
| --- | --- |
| *f*yd,w | is the design yield stress of the confining hoops; |
| *f*yd,f | is the design yield stress of the flange of the steel cross section; |
| *b* | is the flange width of the cross section (see Figure 12.3); |
| *t*f | is the flange thickness of the cross section (see Figure 12.3). |

1. In critical regions, the distance between consecutive longitudinal rebars restrained by hoop bends or cross-ties should not exceed 250 mm in DC2 or 200 mm in DC3.
2. In the lower two storeys of a building, hoops in accordance with (7), (8) and (9) should be provided beyond the critical regions for an additional length equal to 0,50 *l*cr.
3. Dissipative fully encased composite columns should conform to 12.8.3.
4. The longitudinal spacing *s* of confining hoops should be smaller than half the flange outstand *c* (see Figure 12.3). If *s* is such that 0,5<*s*⁄*c*<1,0, the limits in Table 12.4 should be decreased by 50 %.

NOTE The flange outstand *c* is defined in Figure 12.3.

1. The reinforcement ratio for continuous longitudinal reinforcing, *ρ*sl, should not be smaller than 0,004.

#### Partially encased composite columns

1. The spacing of transverse reinforcement s should satisfy 12.8.7.2(5) over a length greater or equal to *l*cr at the ends of a member. In DC3, 12.8.7.2(6) should be applied for confining hoops near a column base.
2. If longitudinal rebars are present, the straight links may be welded to these, as shown in Figure 12.3.
3. The yield strength of the welds of the straight links in (2) should not be less than the yield strength of the straight links.
4. The minimum diameter *d*bw of the straight links in (2) should satisfy Formula (12.12).
5. The clear concrete cover of straight links in (2) should not be smaller than 20 mm and not greater than 40 mm.
6. 12.8.7.2(12) should be applied.
7. If the longitudinal spacing *s*l of straight links welded to the inside of the flanges is smaller than one half of the flange outstand *c* (see Figure 12.3a), then the spacing *s* of the confining hoops in 12.8.7.2(12) may be increased by 50 %.
8. If the transfer of longitudinal shear between the steel and reinforced concrete parts of the composite cross section is not verified, the moment resistance should be taken equal to that of the steel cross section alone; else, the moment resistance should be taken equal to that of the composite cross section.

#### Filled composite columns

1. The condition on the reinforcing steel contribution ratio of encased composite columns *δ* as defined in prEN 1994-1-1:—, 6.7, should be satisfied.
2. Longitudinal reinforcement may be omitted. If longitudinal reinforcement is provided, internal transverse reinforcement may be omitted.
3. The shear resistance of non-dissipative filled composite columns should be calculated as given in prEN 1994-1-1:—, 8.2.2.2.
4. The shear resistance of dissipative filled composite columns should be determined on the basis of either the structural steel cross section or the reinforced concrete cross section with the steel hollow structural section taken only as shear reinforcement.

### Verification of composite joints in dissipative zones

1. The design of composite joints should limit localization of plastic strains, high residual stresses and fabrication defects. The integrity of the concrete in compression should be maintained in the seismic design situation; yielding should be limited to the steel cross sections.
2. 11.8.6(2) and 11.8.6(3) should be applied.
3. In DC3, unless otherwise specified in Annex G, 11.8.6(4) should be applied to composite connections in dissipative zones. *ω*rm and *ω*sh should be considered in the calculation of *f*y in 11.8.6(2).
4. 11.8.6(5) and 11.8.6(6) should be applied for the design of bolted joints in shear.
5. 11.8.6(6) should be applied for the design of hybrid joints with both bolts and welds either in shear or in tension.
6. In dissipative zones of composite steel beams with slab, 12.8.6.2 should be satisfied.
7. Design of the reinforcing rebars needed in the concrete of the joint region should use models that satisfy joint equilibrium (e.g. Annex I for slabs).
8. The resistance and ductility of members and their connections other than those covered by Annex G should be demonstrated by tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. Composite connections between steel and/or composite steel beams and reinforced concrete or composite columns should comply with Annex G, G.5.
2. Composite connections between steel and/or composite steel beams and steel columns should comply with Annex E for composite - steel moment resisting frames.
3. Composite connections using diaphragm plates should comply with Annex G, G.6.
4. Composite connections with double-split tees in filled composite columns should comply with Annex G, G.7.
5. In DC2 and DC3, past test results available in the literature and/or refined numerical simulations as defined in prEN 1993-1-14 may be used to verify the designed full-strength and partial-strength composite connections in or adjacent to dissipative zones.
6. Experimental validation for partial- or full-strength connections may be omitted if prequalified connections according to Annexes E and/or G are used.
7. In DC2 and DC3, gusset plate connections of composite-steel frames with concentric bracings should be designed according to Annex E.

NOTE Annex E provides complementary rules on seismic prequalification of beam-to-column joints and design rules for gusset plate connections of frames with bracings. Annex G provides complementary rules on seismic prequalification of full-strength composite beam-to-column joints.

### Verification of column-to-column splices

1. In steel columns, splices should conform to 11.8.7.
2. In reinforced concrete columns, anchorage and splices should conform to 10.6.
3. In encased composite columns, splices should conform to 12.8.7.2, 11.8.7 and 10.6.
4. In partially encased composite columns, splices should conform to 12.8.7.3, 11.8.7 and 10.6.
5. In filled composite columns, splices should be bolted and should conform to 11.8.7. Bolts should be of grade 8.8 or 10.9 and preloaded. Controlled tightening should be used in accordance with prEN 1993‑1-8:2021, 5.1.2(1) and clamped surfaces in contact should be treated as class A or class B in accordance with EN 1090-2.

## Design and detailing rules for composite moment resisting frames in DC2 and DC3

### Design criteria

1. In composite moment resisting frames with steel columns, 11.9.1 should be applied.
2. In DC3, composite moment resisting frames with composite columns should be designed so that plastic hinges form in the beams or in the connections of the beams to the columns, but not in the columns. This rule may be neglected in cases a) to c):
3. at the base of the frame in which *N*Edin primary composite columns satisfies: ;
4. at the top of primary composite columns in the upper storey of multi-storey buildings;
5. at the top and bottom of primary composite columns in single storey buildings in which, in primary composite columns satisfies , where:

|  |  |
| --- | --- |
| *N*pl,Rd | is the plastic design resistance to compression of the composite cross section according to prEN 1994-1-1:—, 8.8.3.2; |
| *N*Ed | is the total design axial compressive force, calculated in accordance to 12.8.5(2); |

1. In DC2, composite moment resisting frames with partial strength connections other than stiffened end plate connections, should conform to 11.9.3(2).
2. The target hinge formation pattern should be achieved by conforming to 12.9.3, 12.9.4 and 12.9.6.

### Analysis

1. 12.5 should be applied.
2. In composite steel beams, two different flexural stiffnesses should be taken into account: for the part of the spans subjected to positive (sagging) bending (uncracked cross section) and for the part of the span subjected to negative (hogging) bending (cracked cross section).
3. As a simplification of (2), the analysis may be performed with an equivalent moment of inertia *I*eq for the entire beam span based on Formula (12.13).

(12.13)

1. The effective flexural stiffness of composite cross sections, should be calculated according to prEN 1994-1-1 using Formula (12.14).

(12.14)

where

|  |  |
| --- | --- |
| *E*a, *E*sand *E*cm | are the modulus of elasticity of the structural steel cross section, of the steel reinforcement and of the concrete, respectively; long-term effects on the effective elastic flexural stiffness should be considered according to prEN 1994-1-1:—,7.4.2.2(2); |
| *I*a, *I*sand *I*c | are the moments of inertia of the structural steel cross section, of the steel reinforcement and of the uncracked concrete section, respectively, with respect to the axis through the centroid of the transformed section; |
|  | is a correction factor that should be taken equal to 0,5; |
|  | is a calibration factor that should be taken equal to 0,9. |

### Verification of beams

1. 12.8.6 should be applied.
2. Steel beams with slab that satisfy 12.8.6.2.3, should conform to 11.9.2.
3. Composite steel beams with slab should conform to 12.8.6.2.
4. Concrete-encased composite beams should conform to 12.8.6.3.
5. Beams should be verified against lateral buckling and lateral torsional buckling in accordance with EN 1993-1-1, assuming the development of a negative (hogging) plastic moment at one end of the beam.
6. Composite steel beams with slab or encased-composite beams should conform to 11.9.2(6) and 11.9.2(8), with the exception that in Formula (11.8) *M*b,Rd should be the design value of the bending resistance of the composite steel beam according to prEN 1994-1-1:—, 8.2.1.
7. In DC3, 11.9.2(7) should be applied with the exception that the plastic moment resistance of the composite steel beam under positive bending (sagging) should be calculated according to prEN 1994‑1‑1:—, 8.2.1.
8. If partial-strength connections are used, 11.9.2(9) should be applied.
9. Composite trusses should not be used as dissipative beams.

### Verification of columns

1. Columns should be structural steel, reinforced concrete, encased-, partially-encased or filled-composite.
2. In DC2, if composite moment resisting frames are designed with full strength connections, 11.9.3(1) should be applied; if they are designed with partial-strength connections, 11.9.3(2) and 6.2.7(2) should be applied.
3. If plastic hinges form in steel columns, 11.9.3 should be applied.
4. If plastic hinges form in reinforced concrete columns, 10.6, 11.9.3(1) and 11.9.3(2) should be applied.
5. If plastic hinges form in composite columns, 12.8.7, 11.9.3(1), 11.9.3(2), 11.9.3(3), and 11.9.3(5) should be applied.
6. The design resistance of composite columns should satisfy prEN 1994-1-1:—, 6.7.
7. In DC3, encased, partially encased or filled composite columns should satisfy Formula (12.15).

(12.15)

where

|  |  |
| --- | --- |
| *N*Ed | is the design axial compressive force in the column from Formula (11.3); |
| *N*pl,Rd | is the plastic design resistance to compression of the composite cross section according to prEN 1994-1-1:—, 8.8.3.2. |

1. 11.9.3(8) should be applied with (6) where applicable.

### Verification of column diaphragm plates

1. Diaphragm plates present in joints of filled composite columns with composite steel or steel beams may be external or internal to the column.
2. The thickness of column diaphragm plates should not be smaller than that of the respective beam flange.
3. The column diaphragm plates should be welded around the full perimeter of the column using either full penetration groove welds or two-sided fillet welds. The strength of these joints should not be smaller than the available strength of the contact area of the plate with the column sides.
4. Internal column diaphragm plates should have circular openings large enough for placing the concrete in such a way that the concrete filling occupies the complete internal volume.
5. Complementary design rules for column diaphragm plates of Annex G (see G.6) should be satisfied.

### Verification of beam to column joints

1. In composite moment resisting frames with joints between steel beams and steel, reinforced concrete or composite columns conforming to 12.8.6.2.3, 11.9.4, Annexes E and G whichever is applicable, should be applied.
2. 11.9.4(2), 11.9.4(9), 11.9.4(10), 11.9.4(11) and 11.9.4(12) should be applied.
3. In composite moment resisting frames with joints between steel and/or composite steel beams and reinforced concrete or composite columns, 12.8.8(9) should be applied.
4. In composite moment resisting frames with composite joints using diaphragm plates, 12.8.8(11) should be applied.
5. In composite moment resisting frames with composite joints with double-split tee connections in concrete filled tube columns, 12.8.8(12) should be applied.
6. In composite moment resisting frames in which the condition for disregarding the composite action of beams with slab (see 12.8.6.2.3) is not satisfied, dissipative semi-rigid and/or partial strength connections may be used only in DC2. In this case, 11.9.4(3) should be applied.
7. In composite moment resisting frames in DC3, with composite steel beams with slab, the design shear force *V*wp,Edin the column web panel should be calculated as being the sum of the plastic moment resistance of the adjacent dissipative zones in beams or connections, with a positive (sagging) moment on one side and a negative (hogging) moment on the other side, divided by the web panel depth. The shear buckling resistance of the web panels should satisfy Formula (12.16).

(12.16)

where *V*wp,Rd is the shear buckling resistance of the web panel for steel columns or the joint shear resistance according to Annex G (see G.5.6) for reinforced concrete or composite columns.

1. In (7), the positive (sagging) and negative (hogging) plastic moment resistance of composite steel beams with slab should be calculated according to 12.8.6.2.2.
2. In composite moment resisting frames, with steel columns, 11.9.4(4), 11.9.4(5), 11.9.4(6) and 11.9.4(8) should be applied.
3. In composite moment resisting frames in DC3, with steel columns, 11.9.4(7) should be applied.

### Verification of column base joints

1. 11.9.5 and Annex H should be applied.

## Design and detailing rules for composite frames with concentric bracings in DC2 and DC3

### Design criteria

1. Columns should be either structural steel, encased or partially encased composite, filled composite or reinforced concrete.
2. Beams should be either structural steel or composite steel with slab or concrete-encased composite.
3. Braces should be structural steel or filled composite.
4. 11.10.1 should be applied.

### Analysis

1. 11.10.2 should be applied.
2. 12.5 should be applied.
3. 12.9.2 should be applied.
4. In composite frames with filled composite bracings, the compression diagonals should be considered in the analysis.
5. The resistance of filled composite braces under tension should be calculated based on the structural steel cross section only.

### Verification of diagonal members

1. 11.10.3 should be applied.
2. The local slenderness of filled composite braces should satisfy the limits defined in Table 12.4 for class 1 in DC3 and class 1 or 2 in DC2.

### 12.10.4. Verification of beams and columns

1. 11.10.4 should be applied.
2. Composite steel beams with slab should conform to 12.8.6.2.
3. Concrete-encased composite beams should conform to 12.8.6.3.
4. Encased of filled composite columns should conform to 12.8.7.
5. Reinforced concrete columns should conform to 10.6.

### Verification of beam to column connections

1. 11.10.5 should be applied with Annex E, wherever applicable.
2. In composite frames with bracings and composite steel beams with slab or concrete-encased composite beams, 11.10.5(2) should be applied with *M*b,pl,Rd calculated according to 12.8.6.2 and 12.8.6.3, whichever is applicable.

### Verification of brace connections

1. 11.10.6 should be applied.
2. In composite frames with filled composite bracings, 11.10.6(2) should be applied with *N*b,Rd and *N*pl,Rd calculated based on the entire composite cross section.
3. In composite frames with bracings, brace connections accommodating buckling of diagonal members should conform to Annex E.

### Verification of column base joints

1. 11.10.7 and Annex H should be applied.
2. In composite frames with filled composite bracings, 11.10.7(1) should be applied with *N*b,Rd calculated based on the entire composite cross section.

## Design and detailing rules for composite frames with eccentric bracings in DC2 and DC3

### Design criteria

1. 11.11.1 should be applied.
2. Columns should be either structural steel, reinforced concrete, encased composite or filled composite.
3. Beams should be either structural steel or composite with a concrete slab or concrete-encased composite.
4. Braces should be structural steel or filled composite.
5. Braces, columns and beam segments outside the seismic links should be designed to remain elastic under the maximum forces corresponding to fully yielded and cyclically strain-hardened beam links.

### Analysis

1. 12.5 should be applied.
2. 12.9.2 should be applied.
3. 12.10.2(4) and 12.10.2(5) should be applied.

### Verification of seismic Links

1. 11.11.2 should be applied.
2. Seismic links should not be encased.
3. The concrete slab should be disregarded for calculating *M*p,link according to Formula (11.28). Only the steel components of the seismic link cross section should be taken into account.
4. When a seismic link frames into an encased or reinforced column, face bearing plates should be provided on both sides of the link at the column face and in the end section of the link. The design of the face bearing plates should conform to Annex G, G.5.9.
5. The design of composite connections adjacent to dissipative links should conform to 12.8.8.

### Verification of diagonal members

1. 12.10.3 should be applied.

### Verification of beams and columns

1. 12.10.4 should be applied.

### Verification of members and connections not containing seismic links

1. Members not containing seismic links should conform to 11.11.3, taking into account the combined resistance of composite members and the design rules for members in 12.8 and prEN 1994-1-1, wherever applicable.
2. Where a seismic link is adjacent to a fully encased composite column, transverse reinforcement conforming to 12.8.7.1 should be provided above and below the link connection over a length of twice the column depth.

### Verification of connections of the seismic links

1. 11.11.4 should be applied.

### Verification of beam to column connections

1. 11.11.5 should be applied.
2. 12.10.5(2) should be applied.

### Verification of column base joints

1. 12.10.7 and Annex H should be applied.

## Design and detailing rules for composite frames with buckling restrained bracings

### Design criteria

1. 11.12.1 should be applied.
2. Columns should be either structural steel, reinforced concrete, encased or filled composite.
3. Beams should be either structural steel or composite with a concrete slab or concrete-encased composite.

### Analysis

1. 11.12.2 and 12.9.2 should be applied.

### Design rules of buckling restrained bracings

1. 11.12.3 should be applied.

### Conformity criteria

1. 11.12.4 should be applied.

### Verification of beams and columns

1. 11.12.5 should be applied.
2. Composite steel beams with slab should conform to 12.8.6.2.
3. Concrete-encased composite beams should conform to 12.8.6.3.
4. Encased of filled composite columns should conform to 12.8.7.
5. Reinforced concrete columns, should conform to 10.6.

### Verification of beams to column connections

1. Beam-to-column connections should be designed according to 11.12.6 and Annex E for joints of frames equipped with buckling restrained bracings.
2. In composite frames with buckling-restrained bracings and composite steel beams with slab or concrete-encased composite beams, 12.10.5(2) should be applied.

### Verification of brace connections

1. 11.12.7 should be applied.
2. In composite frames with buckling-restrained bracings and composite steel beams with slab or concrete-encased composite beams, 11.11.7(2) should be applied with *M*Rd calculated according to 12.8.6.2 and 12.8.6.3, whichever is applicable.

### Verification of column base joints

1. 11.12.8 and Annex H should be applied.

## Design and detailing rules for composite dual frames in DC2 and DC3

### Design criteria

1. In composite dual frames with both composite moment resisting frames and/or composite frames with concentric, eccentric or buckling-restrained bracings acting in the same loading direction, the horizontal forces should be distributed between the different frames according to their lateral stiffness and comply with 12.4.1(1)e).
2. In composite dual frames, the component moment resisting frames should conform to 12.9.
3. In composite dual frames, the composite frames with concentric, eccentric or buckling-restrained bracings should conform 12.10, 12.11 and 12.12, respectively.
4. 11.13.1(4) should be applied.

## Design and detailing rules for structural wall systems made of reinforced concrete shear walls composite with structural steel elements in DC2 and DC3

### Design criteria

1. The provisions in 12.14.1 should be applied to composite structural wall systems defined in 12.4.1(1)g).
2. Type 1 and Type 2 composite structural systems should be designed to behave as shear walls and dissipate energy in the vertical steel sections and in the vertical reinforcement. The vertical fully encased or partially encased structural steel cross sections should act as boundary elements of the reinforced concrete wall or the reinforced concrete infill panels. The infills should be tied to the boundary elements to prevent separation and to contribute to the lateral resistance of the shear wall.
3. The design action effects in walls should conform to 10.8.2.
4. In Type 1 composite structural wall systems, the storey shear forces should be carried by horizontal shear in the wall and in the interface between the wall and beams.
5. Type 3 composite structural wall systems should be designed to dissipate energy in the shear walls and in the coupling beams.

### Analysis

1. Concrete wall cross section properties should conform to Clause 10 and 12.5.
2. In Type 1 and 2 composite structural systems, the analysis should be made by assuming that the seismic action effects are represented by axial forces only in the vertical boundary elements of the composite structural system. The axial forces should be calculated by assuming that the shear forces are carried by the reinforced concrete wall acting composedly with the steel profiles. The entire gravity and overturning forces should be carried by the shear wall acting composedly with the vertical boundary elements.
3. In Type 3 composite structural systems, if composite coupling beams are used, 12.9.2(2) and 12.9.2(3) should be applied.

### Verification of composite walls in DC2

1. The reinforced concrete infill panels in Type 1 and the reinforced concrete walls in Types 2 and 3 should conform to the provisions of 10.8 for ductile walls or 10.9 for large walls, with the exception of 10.8.3.1(4) for ductile walls which should be replaced by (2).
2. In primary seismic walls, the normalized design axial load *ν*d,c for ductile composite walls in the seismic design situation should satisfy:

— for DC2: *ν*d,c ≤ 0,40,

— for DC3: *ν*d,c ≤ 0,35,

where

|  |  |
| --- | --- |
| *ν*d,c | is defined by Formula (12.17) |

(12.17)

where

|  |  |
| --- | --- |
| *A*a, *A*sand *A*c | are the total area of the structural steel cross sections, of the steel reinforcement and of the concrete, respectively; |
| *n*0 | is the ratio of steel and concrete modulus by Formula (12.1). |

1. The ductility class of partially encased structural steel cross sections used as boundary elements in reinforced concrete panels should comply with Table 12.4, based on the assumed behaviour factor.
2. Fully encased structural steel cross sections used as boundary elements in reinforced concrete panels should be designed in accordance with 12.8.7.1 and 12.8.7.2.
3. Partially encased structural steel cross sections used as boundary elements of reinforced concrete panels should be designed in accordance with 12.8.7.1 and 12.8.7.3.
4. Headed shear connectors or tie reinforcement (welded to the steel members or anchored through holes in the steel members or anchored around the steel member) should be provided to transfer vertical and horizontal shear forces between the structural steel of the boundary elements and the reinforced concrete.

### Detailing and verification of coupling beams in DC2

1. Coupling beams should have an embedment length into the reinforced concrete wall sufficient to resist the most adverse combination of moment and shear generated by the bending and shear resistance of the coupling beam. The embedment length le should begin inside the first layer of the confining reinforcement in the wall boundary member (see Figure 12.5). The embedment length le should not be smaller than 1,5 times the depth of the coupling beam.
2. The design of beam to wall connections should conform to 12.8.8.
3. The vertical wall reinforcement, defined in Annex G (see G.5.8), with design axial resistance equal to the shear resistance of the coupling beam, should be placed over the embedment length of the coupling beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement should extend to a distance of at least one anchorage length above and below the flanges of the coupling beam. Vertical reinforcement placed for other purposes, such as for vertical boundary elements, may be part of the required vertical reinforcement.
4. Transverse reinforcement should conform to 12.8.6.3 and 12.8.7.2 and 12.8.7.3.

Key

|  |  |
| --- | --- |
| A | additional wall reinforcement at embedment of steel beam |
| B | steel coupling beam |
| C | face bearing plate |

Figure 12.5 — Details for coupling beam framing into a wall in DC2 or DC3

### Additional detailing rules for DC3

1. Transverse reinforcement should confine the concrete of composite boundary elements, either partially or fully encased. It should extend to a distance of 2*h* into the concrete walls where *h* is the depth of the boundary element in the plane of the wall as shown in Figure 12.6.
2. The transverse reinforcement should be placed over a height *h*cr of the wall as given in 10.8.
3. The transverse reinforcement should comply with 12.8.7.1. For fully encased steel cross sections 12.8.7.2 should be applied. For partially encased steel cross sections 12.8.7.3 should be applied.
4. The coupling beam design should comply with 12.11.3, 12.11.5 and 12.11.6, for links in composite frames with eccentric bracings.

Key

|  |  |
| --- | --- |
| A | bars welded to column |
| B | transverse reinforcement |
| C | shear connectors |
| D | cross tie |

Figure 12.6 — Transverse reinforcement details for composite boundary elements in DC3: (a) partially encased; (b)fully encased

## Composite diaphragms, chords and collectors

1. The design of floor diaphragms, chords and collectors should satisfy 6.2.8, 10.12 and 11.17, as appropriate.

## Transfer zones: Design for DC2 and DC3

1. 6.2.11 and 11.18 should be satisfied.

## Checking of design and construction

1. 11.19 should be applied.

# Specific rules for timber buildings

## General

1. Clause 13 should be applied to the design and the verification of timber buildings, in seismic regions.
2. Clause 13 should be applied as a complement to prEN 1995-1-1.

## Basis of design

### Design concepts

1. Earthquake-resistant timber buildings should be designed in accordance with prEN 1998-1-1:2022, 4.4.2(3) 4.5.2(3), aiming at either a) or b):
2. low-dissipative structural behaviour (DC1);
3. dissipative structural behaviour (DC2 or DC3).
4. In concept a) of (1), when using the reduced spectrum defined in prEN 1998-1-1:2022, 6.4.1, the behaviour factor components *q*D and *q*R should be taken equal to 1,0. The behaviour factor component *q*S should be taken equal to 1,5.
5. In concept a) of (1), (9), 13.2.2(2), 13.3.2(1), 13.3.2(2), 13.4.1, 13.4.4, 13.5, 13.6, 13.15 and 13.17 should be applied. In addition, also the General Rules and Detailing Rules for each structural type as described in 13.7 to 13.14 should be applied.
6. In concept b) of (1), when using the reduced spectrum defined in prEN 1998-1-1:2022, 6.4.1(1), for a force-based analysis, the behaviour factor components *q*D and *q*R may be taken greater than 1,0. The values of *q*D and *q*R should depend on the ductility class and the structural type (see 13.4).
7. In concept b) of (1), dissipative zones should be located in joints and connections or outside the joints and connections in purposely developed energy dissipation systems.
8. If dissipative zones are located in joints and connections, the energy dissipation should take place by flexural yielding of metal fasteners, whereas the timber members themselves should be designed to remain elastic.
9. In DC2 and DC3, a) and b) should be applied:
10. timber or wood-based members, glued joints and connections with axially loaded fasteners according to 13.4.2(12) should be designed as non-dissipative;
11. carpentry connections as defined in 3.1.5 should be designed as non-dissipative unless they comply with 13.4.3(5) and provide sufficient energy dissipation capacity, according to 13.3.1(2).
12. If dissipative zones are located in energy dissipation systems, both the timber members and the connections should be regarded as behaving elastically and should be capacity designed according to prEN 1998-1-1:2022, 4.4.2(2) 4.5.2(2), in relation to the resistance of the energy dissipation systems. The design should be carried out according to prEN 1998-1-1:2022, 6.8, and prEN 1998-1-2:2022, Clause 9 and Annex D, rather than to 13.4.
13. Horizontal diaphragms should be connected to the underlying primary and secondary (see 3.1.27) members to restrain them in- and out-of-plane and transfer the seismic action.

### Safety verifications

1. For verifications of DC2 and DC3 design at SD Limit State, the design strength of dissipative zones should be calculated by Formula (13.1).

with the limitation (13.1)

where

|  |  |
| --- | --- |
| *F*Rd,d | is the design value of the strength of the dissipative zones; |
| *k*deg | is the strength reduction factor due to degradation under cyclic loading, given in 13.3.1(1); |
| *k*mod | is the modification factor for duration of load and moisture content according to prEN 1995-1-1:2023, 5.1.3, Table 5.1; |
| *F*Rk,d | is the characteristic value of the strength of the dissipative zones, according to prEN 1995-1-1:2023, Clause 11; |
| *γ*M | is the partial factor for a material property according to prEN 1995-1-1:2023, 4.5.2.2, Table 4.6. |

1. The design strength of the non-dissipative components of DC2 and DC3 design and of all members of DC1 design should be calculated as given by Formula (13.2).

(13.2)

where

|  |  |
| --- | --- |
| *F*Rd,b | is the design value of the strength of the non-dissipative components; |
| *F*Rk,b | is the characteristic value of the strength of the non-dissipative components, according to prEN 1995-1-1:2023, Clauses 8, 11 and 12; |
| *γ*M | is as defined in (1). |

NOTE For DC1, the values of *γ*M are those given by prEN 1995-1-1:2023, 4.5.2.2, Table 4.6, for persistent and transient situations; for DC2 and DC3 the values of *γ*M are those given in prEN 1995-1-1:2023, 4.5.2.2, Table 4.6, for accidental situations, unless the National Annex gives different values for use in a country. For design according to prEN 1998-1-1:2022, 4.1(8), the values of *γ*M are those given in prEN 1995-1-1:2023, 4.5.2.2, Table 4.6, for accidental situations, unless the National Annex gives different values for use in a country.

1. The maximum allowable rope-effect contribution as given in prEN 1995-1-1:2023, 11.3.6, should be considered when assessing the strength of the dissipative connections.

NOTE The rope effect can however be neglected when assessing the strength in seismic conditions of the non-dissipative connections.

## Materials

### 13.3.1 Mechanical properties of dissipative zones

1. The mechanical properties of dissipative zones should be determined by cyclic tests in accordance with EN 12512; they should correspond to a strength impairment factor *φ*imp not greater than 0,3 and a strength reduction factor *k*deg not smaller than 0,8. Testing conditions should represent the design intent (for example type of dissipative zone, loading conditions, restraints). The loading should consist in three reversed cyclic loads at each equal target deformation, this target deformation being progressively increased.

NOTE 1 The strength impairment factor *φ*imp in a cyclic test performed in accordance with EN 12512 is the ratio of the reduction *ΔF*1-3 = *F*1 – *F*3 of resistance of the tested component from the first to the third cycle at equal target deformation *δ* and the resistance *F*1 at the first cycle. The strength reduction factor *k*deg is the ratio of the resistance at the first cycle at the ultimate deformation *δ*u determined according to EN 12512 to the maximum resistance in monotonic tests.

NOTE 2 Typical dissipative zones are single timber connections (e.g. a set of one or more fasteners connecting at least two timber and/or metal elements), 2D- or 3D-connectors (hold-down, foundation tie-down, angle bracket, shear plate, tie-down, or a 2D shear plate connecting a timber member with one or more members, etc), timber joints (a junction of two or more members involving one or more timber connections and/or 2D- or 3D-connectorsand parts of a structure (a subassembly like a shear wall or a portal frame).

NOTE 3 prCEN/TS 1998-1-101 can be used to determine the mechanical properties of a dissipative zone such as the slip modulus *K*SLS,v,mean,c (see 13.5(2)), the strength reduction factor *k*deg and the ductility *μ* in accordance with EN 12512.

1. The displacement ductility *μ* of carpentry connections in cyclic tests performed to EN 12512 withthe conditions on *φ*imp and *k*deg in (1) should not be smaller than 2.

NOTE Carpentry connections satisfying this condition have the needed energy dissipation capacity.

### 13.3.2 Material properties

1. The thickness of cross laminated timber (CLT) and glue-laminated timber (glulam or GLT) panels should be not smaller than 60 mm.
2. Glulam, Solid Wood Panels (SWP), LVL and GLVL panels may be used in shear walls, floor and roof diaphragms, providing that measures are taken in order to limit in-plane shrinkage in the direction perpendicular to face grain. In floor and roof diaphragms, the designer should consider the different stiffness of glulam, SWP~~,~~ LVL and GLVL in both in-plane directions.
3. Material properties in (4) and (5) should be satisfied by panels in dissipative zones and their connections.
4. The sheathing material should satisfy a) to h):
5. Particleboard-sheathing should comply with EN 312, be at least 12 mm thick and have a characteristic density of at least 550 kg/m3.
6. Plywood-sheathing should comply with EN 636, be at least 9 mm thick, have at least 5 layers and have a characteristic density of at least 450 kg/m3.
7. Fibreboard-sheathing should comply with EN 622 (all parts), be at least 12 mm thick and have a characteristic density of at least 550 kg/m3.
8. Oriented Strand Board (OSB) sheathing should comply with EN 300, be at least 12 mm thick and have a characteristic density of at least 550 kg/m3.
9. Gypsum Fibre board (GF) sheathing should comply with EN 15283-2, be at least 12 mm thick and have a characteristic density of at least 1000 kg/m3.
10. Densified Veneer Wood sheathing should comply with EN 61061-3-1 and have a characteristic density of at least 1200 kg/m3.
11. Solid wood panel (SWP) sheathing should comply with EN 13353, have at least 3 layers and have a characteristic density of at least 400 kg/m3.
12. For LVL sheathing panels, only LVL-C complying with EN 14279 and EN 14374 with a characteristic density of at least 450 kg/m3 should be used.
13. Steel material for connections in dissipative zones should conform to a) to d):
14. Steel plate elements should comply with EN 1993-1-1:2022, 5, and be designed to remain elastic according to EN 1993-1-1:2022, 8.
15. Fasteners should belong to low cycle ductility classes S2 and S3 according to EN 14592:2022, 5.5, for Ductility Classes DC2 and DC3 structures, respectively.
16. Screws and annular ringed shank nails used in steel-to-timber connections should be checked according to EN 12512 for the ductility ratios in Table 13.3 to ensure that the fastener head does not shear off during cyclic tests.
17. The ratio between the 95th percentile and the declared characteristic values of the mechanical properties defined according to EN 14592 should not exceed 1,4 for all metal fasteners.
18. In order to avoid human errors during construction of structures designed to DC2 and DC3, the fasteners in non-dissipative connections should either be different for diameter or shape from those used in dissipative connections, or made of steel complying with (5).

## Structural types, behaviour factors, capacity design rules and limits of seismic action

### Structural types

1. Buildings with a primary timber structure should be classified into one of the structural types defined in Table 13.1.

Table 13.1 — Timber structural types and examples of structures

|  |  |
| --- | --- |
| **Examples of structural types\*** | **Timber structural types** |
|  | 1. Cross laminated timber (CLT) structures\*\*   CLT structures are those where the primary structure (see 3.1.23) is composed of shear walls made of cross laminated timber panels according to 13.3.2(1). Glulam, LVL or GLVL may be used as an alternative to CLT only in DC1 and DC2 design and for a seismicity index *S*δ ≤ 4,0 [m/s2]. CLT structures should be designed according to 13.7. |
|  | 1. Framed wall structures   Framed wall structures are those where the primary structure is composed of framed shear walls made of timber frames to which a wood-based panel (e.g. plywood or OSB) or other type of sheathing material is connected. Framed wall structures should be designed according to 13.8.  b1) With fully anchored walls;  b2) With non-fully anchored walls. |
|  | 1. Log structures   Log structures are those where the primary structure is composed by the superposition of rectangular or round solid or glulam timber elements (‘logs’), prefabricated with carpentry connections at their ends and with upper and lower grooves.  Log structures should be designed according to 13.9. |
|  | 1. Moment-resisting frame structures   Moment-resisting frame structures are those where the primary structure is composed of frames made by timber elements with semi-rigid (as defined in 3.1.28) moment-transmitting joints between the members, achieved with mechanical fasteners. Moment-resisting frames structures should be designed according to 13.10. |
|  | 1. Braced frame structures with dowel-type connections   Braced frame structures with dowel-type connections are those consisting of timber columns and beams, where the primary structure is composed of timber diagonal bracings, with all pin-jointed dowel-type connections (as defined in 3.1.9). Braced frame structures with dowel-type connections should be designed according to 13.11. |
|  | 1. Vertical cantilever structures\*\*   Vertical cantilever structures are those where the primary structure is composed of vertically continuous cantilever glulam, LVL, GLVL or CLT walls or columns without any horizontal joints. Vertical cantilever structures should be designed according to 13.12. |
|  | 1. Braced frame structures with carpentry connections and interacting masonry infill   Braced frame structures with carpentry connections are those consisting of timber columns and beams, where the primary structure is composed of vertical timber bracing with compression-only carpentry connections and interacting masonry infill. Braced frame structures with carpentry connections and interacting masonry infill should be designed according to 13.13. |
|  | 1. Braced frame structures with carpentry connections   Braced frame structures with carpentry connections are those consisting of timber columns and beams, where the primary structure is composed of vertical timber bracing with compression-only carpentry connections. Braced frame structures with carpentry connections should be designed according to 13.14 as non-dissipative systems. |
|  | 1. Two-pin and three-pin arches, three-pin frames and dome structures   Structures composed by two-pin and three-pin timber arches, three-pin timber frames and timber dome structures should be designed as non-dissipative systems. |
|  | 1. Large span timber truss portal frame structures   Large span truss portal frame structures are those consisting of timber trusses with semi-rigid moment-transmitting joints between the chords and the columns, forming a moment frame. Large span truss portal frame structures should be designed as non-dissipative systems. |
| \* The drawings in Table 13.1 depict a part of a structure. Different number of storeys and structural layout may be used.  \*\* CLT structures can fall into either category a) or f) depending on whether the shear walls have heights equal to one interstorey height (platform frame construction – see 3.1.20) or more (balloon frame construction – see 3.1.2). | |

1. The primary structure (see 3.1.23) should be made of shear walls, cantilever walls, moment-resisting frames or braced frames, which should be structurally continuous from the base of the timber part of the building to the roof (Figure 13.1 A) or an intermediate level (Figure 13.1 B). If they are interrupted below an intermediate level (Figure 13.1 C), 6.2.11 and 13.16 should be applied for design to DC2 and DC3.
2. Partitions as defined in 3.1.18 should comply with 7.1, 7.2 and 7.3 and should be detailed not to attract lateral load according to 5.1.3(1).

Key

|  |  |
| --- | --- |
| A | primary structure with all shear walls structurally continuous from the foundation to the roof |
| B | primary structure with part of the shear walls structurally continuous from the foundation to the roof and part of the shear walls interrupted at the top storey |
| C | primary structure with part of the shear walls interrupted below the second and third storey |
| 1 | structural wall |
| 2 | lintel |
| 3 | opening |
| 4 | parapet |

Figure 13.1 — Example of primary structure with shear walls

1. Buildings with a primary timber structure other than those listed in (1) may be used, provided that either a) or b) is fulfilled:
2. they are designed in DC1;
3. for a design as DC3 or DC2 dissipative behaviour, the properties of both dissipative zones (timber connections, 2D or 3D connectors, or timber joints) and subassemblies (e.g. shear walls, portal frames, etc.) are determined by cyclic tests in accordance with EN 12512 and 13.3.1(1), and with EN 1990:2023, Annex D.

### Behaviour factors

1. For timber buildings, which are regular in elevation in accordance with 4.4.4.2 and are assigned to DC2 or DC3, the default values of the component *q*D of the behaviour factor *q* should be taken from Table 13.2 provided that either (4), (7), (10) or (11) is satisfied, and provided that the capacity design rules defined in 13.4.3 for all structural types and in 13.7 to 13.13 for each structural type are also satisfied. The values of *q* are obtained according to prEN 1998-1-1:2022, 6.4.1(1), as the product of the values of *q*D, *q*R and *q*S, with *q*S = 1,5.
2. The behaviour factor component *q*D of a hybrid building where the primary structure is made with a combination of CLT and fully anchored framed shear walls at the same level may be calculated by Formula (13.3) instead of 5.3.2(4).

(13.3)

where

|  |  |
| --- | --- |
| *q*D,Hyb | is the value of component *q*D of the behaviour factor *q* for the hybrid primary structure; |
| *q*D,FW | is the default value for the Ductility Class of component *q*D for the framed wall primary structure; |
| *q*D,CLT | is the default value for the Ductility Class of component *q*D for the CLT primary structure; |
| *k*stif min(*k*stif,i) for *i* = 1,...,*N*s | *N*s is the minimum value of the ratio between the horizontal stiffness of the CLT walls and the total horizontal stiffness of the hybrid primary structure in each main direction at the *i*th storey; |
| *N*s | is the number of storeys of the building. |

Table 13.2 — Default values of the behaviour factors *q* for buildings regular in elevation with maximum values of the seismic action index *S*δ for design in DC1

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **Structural type** | **Maximum *S*δ for design in DC1 [m/s2]** | **Ductility class** | | | | | | |
| **DC1** | **DC2** | | | **DC3** | | |
| ***q*** | ***q*D** | ***q*R** | ***q*** | ***q*D** | ***q*R** | ***q*** |
| a) Cross laminated timber (CLT) structures | 4,0 | 1,5 | 1,2 | 1,3 | 2,3 | 1,4 | 1,5 | 3,2 |
| b) Framed wall structures |  |  |  |  |  |  |  |  |
| b1) With fully anchored walls | 5,0 | 1,5 | 1,5 | 1,1 | 2,5 | 2,4 | 1,1 | 4,0 |
| b2) With non-fully anchored walls | 3,0 | 1,5 | N/A | N/A | N/A | N/A | N/A | N/A |
| 1. Log structures | 4,0 | 1,5 | 1,2 | 1,1 | 2,0 | N/A | N/A | N/A |
| 1. Moment-resisting frames structures |  |  |  |  |  |  |  |  |
| d1) Single storey | 4,0 | 1,5 | 1,3 | 1,1 | 2,1 | 2,0 | 1,1 | 3,3 |
| d2) Multi-storey, one-bay | 4,0 | 1,5 | 1,3 | 1,2 | 2,3 | 2,0 | 1,2 | 3,6 |
| d3) Multi-storey, multi-bay | 4,0 | 1,5 | 1,3 | 1,3 | 2,5 | 2,0 | 1,3 | 3,9 |
| 1. Braced frame structures with dowel-type connections | 4,0 | 1,5 | 1,3 | 1,0 | 2,0 | N/A | N/A | N/A |
| 1. Vertical cantilever structures | 4,0 | 1,5 | 1,2 | 1,3 | 2,3 | N/A | N/A | N/A |
| 1. Braced frame structures with carpentry connections and interacting masonry infills | 4,0 | 1,5 | 1,3 | 1,1 | 2,0 | N/A | N/A | N/A |
| h) Braced frame structures with carpentry connections | 3,0 | 1,5 | N/A | N/A | N/A | N/A | N/A | N/A |
| i) Two-pin and three-pin timber arches, three-pin timber frames and timber dome structures | 3,0 | 1,5 | N/A | N/A | N/A | N/A | N/A | N/A |
| j) Large span timber truss portal frame structures. | 3,0 | 1,5 | N/A | N/A | N/A | N/A | N/A | N/A |
| N/A: Not Applicable | | | | | | | | |

NOTE Conditions to design structural types h), i) and j) for *S*δ greater than 3 m/s2 are given in 13.4.4(3).

1. The behaviour factor component *q*R evaluated through an explicit calculation according to 5.3.2(6) should not exceed the default values in Table 13.2 by more than 10 %.
2. The dissipative zones, specified for each structural type designed in DC2 and DC3, should attain a ductility not smaller than the values in Table 13.3 in cyclic tests performed according to EN 12512.
3. For each structural type designed in DC2 and DC3, (4) should be fulfilled by at least one type of dissipative sub-assembly/joint/2D- or 3D-connector/connection in Table 13.3.
4. In buildings with a primary timber structure listed in 13.4.1(1) where the energy dissipation does not take place by flexural yielding of metal fasteners but occurs in alternative systems used in the same dissipative zones listed in 13.7 to 13.14, the default values of the behaviour factors in Table 13.2 for design to DC2 and DC3 may be used provided that (4) applies. In this case, (7), (10) and (11) should not be used.
5. For each structural type designed in DC2 and DC3, the default values of the behaviour factor *q* may be taken from Table 13.2 in all of the following cases a) to e), provided that (8) and (9) are also satisfied:
6. In all DC2 structural types different from framed shear walls with dissipative sheathing-to-framing stapled connections;
7. In DC2 framed shear walls according to 13.8.2 with dissipative sheathing-to-framing stapled connections, when a ductile mechanism with at least one flexural plastic hinge in the mechanical fasteners is attained in the seismic design situation;
8. In DC3 CLT systems with segmented walls according to 13.7.3;
9. In DC3 framed shear walls according to 13.8.3 with dissipative sheathing-to-framing nailed connections, when a ductile mechanism with at least one flexural plastic hinge in the nail (or screw) is attained in the seismic design situation;
10. In all other DC3 structural types, when a ductile mechanism with at least two flexural plastic hinges in the mechanical fasteners is attained in the dissipative zones in the seismic design situation.
11. Failure modes (a), (b) and (c) for fasteners in single shear as given in prEN 1995-1-1:2023, 11.3.2(1), and failure modes (a) and (b) for fasteners in double shear as given in prEN 1995-1-1:2023, 11.3.2(2), should be avoided in the dissipative zones by satisfying Formula (13.5). Failure modes (a) and (b) for fasteners in multiple shear as given in prEN 1995-1-1:2023, 11.3.5, should be avoided in the dissipative zones by satisfying Formula (13.5).
12. Brittle failure modes like splitting, row shear, block shear, plug shear, and net tensile failure of wood in the connection regions, as defined in prEN 1995-1-1:2023, 11.6, should be avoided by satisfying condition given by Formula (13.4).

NOTE Reinforcement can be used in a dissipative zone as a means to prevent brittle failure modes, see for example prEN 1995-1-1:2023, 11.8.

1. As an alternative to (7), for each structural type designed in DC3, the default values of the behaviour factor *q* may be taken from Table 13.2, if condition a) and either condition b) or c) are satisfied:
2. (9) is satisfied;
3. for timber-to-timber connections, the thicknesses of the timber members are greater than the required minimum embedment lengths which ensures failure mode (f) according to prEN 1995‑1‑1:2023, 11.3.2(6), Table 11.4;
4. for steel-to-timber connections, the thickness of the timber member is greater than the required minimum embedment lengths which ensures failure mode (f) according to prEN 1995-1-1:2023, 11.3.2(9).
5. In cases where neither b) nor c) in (10) are satisfied, the dissipative zones of all structural types may be considered as belonging to DC2 and the default values of the behaviour factor *q* may be taken from Table 13.2, provided a) in (10) is satisfied and, for timber-to-timber connections, the thicknesses of the timber members are greater than the required minimum embedment lengths which ensure the attainment of failure modes (d) or (e) according to prEN 1995-1-1:2023, 11.3.2(11), Table 11.5.
6. (11) may be applied to framed shear walls with gypsum fibre boards with stapled connections and particle boards with stapled connections.

Table 13.3 — Minimum required ductility *μ* as defined in EN 12512 of   
dissipative zones tested accordingly

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Structural type** | **Dissipative sub-assembly/joint/2D-or 3D connector/connection** | **Type of ductility** | ***μ***  **DC2** | ***μ***  **DC3** |
| a) Cross laminated timber structures | Shear wall\* | Displacement | 1,5 | 2,5 |
| Hold-downs, tie-downs, foundation tie-downs, angle brackets, shear plates | Displacement | 1,5 | 1,5 |
| Screwed wall panel-to-panel joints | Displacement | - | 5,5 |
| b) Framed wall structures | Shear wall\* | Displacement | 2,2 | 3,5 |
| Connection (nail/screw/staple) | Displacement | 3,5 | 5,5 |
| c) Log structures | Shear wall\* | Displacement | 1,4 | - |
| d) Moment-resisting frames | Portal Frame\* | Displacement | 2,0 | 3,0 |
| Beam-column joint | Rotational | 4,0 | 7,0 |
| e) Braced frame structures with dowel-type connections | Braced Frame\* | Displacement | 1,4 | - |
| f) Vertical cantilever structures | Shear wall\* | Displacement | 2,0 | - |
| g) Braced frame structures with carpentry connections and interacting masonry infill | Shear wall\* | Displacement | 1,4 | - |
| \*The values provided refer to the system ductility of the sub-assembly, taking into account the ductility of all the individual connections and components. | | | | |

1. Connections made of dowel-type fasteners transferring load mainly via axial resistance (Figure 13.2 A and B) should not be considered as dissipative.
2. Dowel-type fasteners in dissipative zones should be perpendicular to the shear force in the connection (Figure 13.2 C).

Key

|  |  |
| --- | --- |
| A and B | connection with fasteners inserted inclined with respect to the direction of the shear force, transferring most of the load via axial resistance, which should not be considered as dissipative |
| C | connection with fasteners inserted perpendicular with respect to the direction of the shear force, transferring most of the load via shear resistance, which may be considered dissipative |
| 1 | direction of the shear force being transferred |
| 2 | fasteners inclined with respect to the direction of the shear force being transferred |
| 3 | fasteners perpendicular to the direction of the shear force being transferred |

Figure 13.2 — Examples of non-dissipative and dissipative connections

### Capacity design rules common to all dissipative structural types

1. To ensure yielding of the dissipative zones, all non-dissipative members and connections in DC2 or DC3 structures shall be capacity designed according to (2) and either (4) or (5).
2. For DC2 and DC3 design of the structural types in Table 13.2, the design strength *F*Rd,b of the brittle components should satisfy Formula (13.4).

with the limitation (13.4)

where

|  |  |
| --- | --- |
| **Rd | is the design value of the strength of the dissipative zones; |
| *k*deg | is the overstrength factor, given in Table 13.4; |
| *k*deg | is the strength reduction factor defined in 13.3.1(1), for which the value given in 13.3.1(1) should be used; |
| *F*Rd,d | is the design strength of the ductile component, calculated according to 13.2.2(1); |
| *F*Rd,b | is the design strength of the brittle component, calculated according to 13.2.2(2). |

1. In DC2, the design forces in the non-dissipative connections, elements and anchorages to the foundation calculated with Formula (13.4) need not be taken as greater than the values from an analysis made considering a completely elastic behaviour (*q* = 1).
2. If less ductile failure modes may occur according to prEN 1995-1-1:2023, 11.3.2, in the dowel-type metal fasteners of the dissipative zones, the less ductile failure modes should be designed with a safety margin with respect to the selected ductile failure mode providing energy dissipation according to Formula (13.5).

(13.5)

where

|  |  |
| --- | --- |
| *F*v,Rk,d | is the characteristic strength of the selected ductile failure mode providing energy dissipation, according to prEN 1995-1-1:2023, 11.3.2; |
| *F*v,Rk,nd | is the characteristic strength of the less ductile failure mode, according to prEN 1995-1-1:2023, 11.3.2. |

NOTE An example of a dissipative zone with dowel-type metal fasteners with more possible failure modes is a timber-to-timber connection with failure mechanisms characterized by only timber embedment (less ductile and less dissipative) (modes (a), (b) and (c) in prEN 1995-1-1:2023, 11.3.2) or by one or two flexural plastic hinges in the dowel (more ductile and more dissipative) (modes (d), (e) and (f)).

1. Formula (13.5) may be applied also to dissipative carpentry connections designed to DC2 to ensure non-ductile failure modes (e.g. longitudinal shear and tension perpendicular to grain) have sufficient overstrength with respect to the selected ductile one (compression). In this case, the safety margin should be assumed equal to 1,3.
2. Additional capacity design rules are given in 13.7 to 13.14 for the different structural types.

### Limits of seismic action for design to DC1

1. For each structural type listed in 13.4.1(1), design to DC1 should not be made above levels of the seismicity index *S*δ given in Table 13.2, unless (3) is satisfied.
2. For structural types alternative to those listed in 13.4.1(1) in accordance with 13.4.1(4), design to DC1 should not be made above level of the seismicity index *S*δ of 3,0 m/s2.
3. Structural types h), i) and j) of Table 13.2 may be designed to DC1 for values of the seismic action index *S*δ greater than the limits in Table 13.2, according to prEN 1998-1-1:2022, 4.1(7), using a value of the behaviour factor *q* = 1 and the values of **M for design to DC1 according to 13.2.2.

Table 13.4 — Values of the overstrength factors **Rd to be used in capacity design

| **Capacity design at** | **Brittle/non-dissipative failure mode** | **Overstrength factor **Rd** | **Formula No.** |
| --- | --- | --- | --- |
| **Connection and 2D- or 3D-connector level** | Failure of timber by fracture in tension (net tensile failure, prEN 1995-1-1:2023, 11.6.8), failure by row shear, block shear or plug shear (prEN 1995-1-1:2023, 11.6.5, 11.6.6, 11.6.7), failure by splitting (prEN 1995-1-1:2023, 11.4.4.2) | 1,6\* | (13.4) |
| Tensile (EN 1993-1-1:2022, 8.2.3) and shear (EN 1993-1-1:2022, 8.2.6) failure of the steel plates (angle brackets, hold-downs, tie-downs, etc.), tensile (prEN 1993-1-8:2021, 5.7) and pull-through failure of anchor bolts or screws | 1,6\* | (13.4) |
| Tensile (EN 1992-4:2018, 7.2.1 and prEN 1998-1-1:2022, Annex G) and shear (EN 1992-4:2018, 7.2.2 and prEN 1998-1-1:2022, Annex G) of headed and post-installed fasteners between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc) and reinforced concrete member. | 1,6\* | (13.4) |
| Axial (prEN 1995-1-1:2023, 11.2) failures of dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members (head pull-through, withdrawal, tensile or buckling failure of the fastener, prEN 1995-1-1:2023, 11.2.2, 11.2.3, 11.2.4, 11.2.5) | 1,6\* | (13.4) |
| Failure in shear (prEN 1995-1-1:2023, 11.3) of dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members | 1,3 | (13.4) |
| **Wall and building level** | Failure of timber members (prEN 1995-1-1:2023, 8) | 1,6\* | (13.7) |
| Axial (prEN 1995-1-1:2023, 11.2) failures (head pull-through, withdrawal, tensile or compression failure of the fastener, prEN 1995-1-1:2023, 11.2.2, 11.2.3, 11.2.4, 11.2.5) of:  - dowel-type joints between adjacent floor panels; floors and supporting walls underneath; orthogonal walls, including the ones at the building corners;  - dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members | 1,6\* | (13.7) |
| Failure in shear (prEN 1995-1-1:2023, 11.3) of:  - dowel-type joints between adjacent floor panels; floors and supporting walls underneath; orthogonal walls, including the ones at the building corners;  - dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members. | 1,3 | (13.7) |
|  | Stabilizing moment due to gravity loads in log shear walls. | 1,3 | (13.19) |
| \*For high ductility moment-resisting frames with expanded tube fasteners and Densified Veneer Wood (according to 13.10.3(2)) and log structures, the value of **Rd may be reduced to 1,3. | | | |

## Structural analysis

1. The stiffness of the semi-rigid joints (as defined in 3.1.28) should be taken into account in the analysis.
2. The value of slip modulus *K*SLS,v,mean,c of the connections measured in cyclic tests according to EN 12512 should be used. If experimental values of *K*SLS,v,mean,c are not available, the slip modulus *K*SLS,v,mean,c at serviceability limit state, calculated according to prEN 1995-1-1:2023, 11.4.7, Table 11.11a, may be used.
3. Two different axial stiffnesses, in compression and tension, should be used for uplift-restraining 2D- and 3D-connectors (hold-downs, tie-downs, foundation tie-downs, shear plates). If experimental values are not available, these 2D- and 3D-connectors may be considered as rigid in compression to represent the direct bearing of the wall on the support.
4. Timber floor and roof diaphragms designed according to prEN 1995-1-1:2023, 13.4, 13.5, and to 13.6.2 and supported by a primary timber structure may be modelled as rigid in plane if conditions a) and either b), c) or d) are satisfied:
5. Their openings do not markedly affect the overall in-plane rigidity of the floor: in a floor with a compact shape, namely a convex shape where the ratio between the maximum dimensions in the two principal directions as defined in 3.1.21 does not exceed 2,0, compact openings of less than 10 % of the floor area which are not located along the perimeter may be assumed not to markedly affect the overall in-plane rigidity.
6. For all structural types in DC1, the diaphragm and its connections should be designed to transfer the in-plane seismic shear to the primary structure according to 6.2.8 using an overstrength factor **d of 1,5 instead of the values provided in Table 6.1.
7. For all structural types other than CLT and framed wall structures in DC2 and DC3, the diaphragm and its connections should be designed to transfer the in-plane seismic shear to the primary structure according to 6.2.8 using an overstrength factor **d of 2,0 instead of the values provided in Table 6.1.
8. For CLT and framed wall structures in DC2 and DC3, the diaphragm and its connections should be designed to transfer the in-plane seismic shear to the primary structure according to Formula (13.7).
9. Timber-concrete composite floor and roof diaphragms designed according to CEN/TS 19103, and 6.2.8(1) may be considered as in-plane rigid if both conditions a) to b) are satisfied:
10. Their openings do not significantly affect the overall in-plane rigidity of the floors: in a floor with a compact shape according to (4), compact openings of less than 20 % of the floor area may be assumed not to markedly affect the overall in-plane rigidity;
11. The concrete topping should be at least 50 mm thick and should be connected to all primary members as defined in 3.1.22.
12. As a simplification of 5.3.3(3), for CLT and framed wall buildings, the fundamental period of vibration of the building in a principal direction, *T*1,est, may be calculated by Formula (13.6).

(13.6)

where *H*bis the height of the building, in m, measured from the foundation or from the top of a rigid basement.

1. The fundamental period *T*1 calculated via numerical and analytical modelling in accordance with 5.3.3(3) should not be taken longer than 2*T*1,est, where *T*1,est is the estimate from Formula (13.6).
2. In linear-elastic analyses, the elastic horizontal displacement of a point in the structure *d*e defined in prEN 1998-1-1:2022, 6.4.2(2), may be calculated using prEN 1995-1-1:2023, 13.3.3, for CLT and framed shear walls.
3. In linear-elastic analyses, prEN 1995-1-1:2023, 13.3.3, may also be used to calculate the stiffness matrix of a building with primary structure made of CLT and framed walls, and to determine the distribution of the seismic forces in the different shear walls.
4. Non-linear behaviour of timber components may be considered in the analysis only if it has been demonstrated by experimental tests performed in accordance with EN 12512.
5. In non-linear static analysis performed according to prEN 1998-1-1:2022, 6.5, and Annex L, timber components and mechanical connections or devices characterized by a brittle failure should be modelled as elastic elements, and ductile components and connections as non-linear elements with mean values of mechanical properties.
6. The values of the ultimate deformation *δ*u and yield deformation *δ*y of connections, 2D- and 3D-connectors, joints and subassemblies for non-linear static analysis should be obtained from Annex L by the trilinear curve determined from tests carried out according to EN 12512. If experimental results are not available, the trilinear curve may be determined by Formulas (L.4) to (L.9).
7. In non-linear static analysis, the design deformations *δ*NC and *δ*SD for Near Collapse and Significant Damage Limit States should be obtained from Annex L, Formula (L.3) and (L.2) respectively.

## Verification of limit states

### General

1. The strength values of the timber material and of the connections should be determined taking into account the *k*mod-values for instantaneous loading in accordance with prEN 1995-1-1:2023, 5.1.3, Table 5.1.

### Limitation of interstorey drift at Significant Damage limit state

1. For all ductility classes, the interstorey drift at SD limit state should be limited to:
2. *d*r,SD ≤ 0,025 *h*s for log structures;
3. *d*r,SD ≤ 0,020 *h*s for moment-resisting frames;
4. for braced frame structures with carpentry connections and interacting masonry infills, 7.4.2.1 should be applied;
5. *d*r,SD ≤ 0,015 *h*s for all other structural types;

where

|  |  |
| --- | --- |
| *d*r,SD | is as given in 6.2.4; |
| *h*s | is the interstorey height. |

### Non-linear static analysis

1. Ductile components, modelled as non-linear, should be verified at Significant Damage Limit States by comparing the deformation (displacement or rotation) obtained by the analysis at the target displacement *d\**t of the equivalent SDOF model in prEN 1998-1-1:2022, 6.5.4, with the design deformation *d*d obtained at Significant Damage Limit States in accordance with 13.5(12) and 13.5(13).
2. Brittle components modelled as elastic should be verified at Significant Damage Limit States by comparing the generalized stresses corresponding to the target displacement *d\**t of the equivalent SDOF model in prEN 1998-1-1:2022, 6.5.4, with the design resistance defined in Annex L, L.4(1), Formula (L.10), in accordance with prEN 1998-1-1:2022, 6.7.2(3).

## Rules for cross laminated timber (CLT) structures

### General rules

1. The primary structure should be made of CLT panels according to 13.3.2(1). Glulam, LVL or GLVL as defined in 13.3.2(2) may be used in DC1 and DC2 design provided that the seismicity index *S* is not greater than 4,0 m/s2.
2. The secondary structure should be made of either CLT panels, or other types of solid wood panels as defined in 13.3.2(2). Post-and-beam members may also be used.
3. The joint of the walls to the foundation should comply with conditions a) to f):
4. It should be made by means of 2D- or 3D-connectors (e.g. hold-downs, foundation tie-downs, angle brackets, shear plates) and metal fasteners (e.g. anchoring bolts, nails and screws, etc.);
5. It should prevent uplift and sliding of the walls;
6. Anchoring connections against overturning (hold-downs or foundation tie-downs) should be placed at wall ends, adjacent to door openings in wall panels, and at opening ends either: when the wall is made by separate panel elements (i.e. wall segments connected with lintels and parapets), or when the ratio between the area of the window opening and the area of the wall panel exceeds 0,50;
7. Shear connection (shear plates, angle brackets, anchoring bolts, nails and screws, etc.) should be distributed uniformly along the wall width (Figure 13.3);
8. Shear connections and anchoring connections against overturning should be connected to the CLT panels using metal fasteners such as nails and screws, and to the foundation using anchor bolts;
9. Fastening to foundation should comply with prEN 1998-1-1:2022, Annex G.
10. The upper walls should bear on the floor panels (platform frame construction – see 3.1.20) and be connected to the lower walls with 2D- and 3D-connectors among those in (3). Tie-down connections nailed or screwed to the cross laminated timber walls may be used to restrain the external walls against uplift (Figure 13.3).
11. Wall panels should have heights at least equal to the interstorey height, *hs*. Along their width, they should be either made of a single element (‘monolithic wall’, Figure 13.3(a)) or composed of more than one panel, which should measure in width not less than 0,25*hs*. Each of these panels should be connected to the other panels by means of vertical joints made with metal fasteners such as screws or nails ('segmented wall', Figure 13.3(b)). The vertical joints between adjacent panels may be omitted ('single uncoupled walls') provided that all conditions a) to c) are satisfied:
12. the structure is not designed in DC3;
13. the width of each panel is not less than 0,40*h*s for glulam, LVL and GLVL, and 0,25*h*s for CLT;
14. anchoring connections against overturning caused by the seismic action are placed at both edges of each single panel.
15. The joints between orthogonal walls should be made with metal fasteners (usually screws).
16. Floor and roof diaphragms should be made of CLT according to 13.15.2.
17. Other types of floor and roof diaphragms than in (8) may be used, provided that their in-plane resistance and stiffness is ensured (e.g. 13.15.3 or 13.15.4).

Key

|  |  |
| --- | --- |
| A | horizontal joint between CLT floor panels |
| B | CLT floor panel |
| C | single piece CLT wall panel |
| D | tie-down |
| E | hold-down |
| F | foundation tie-down |
| G | angle bracket |
| H | vertical CLT panel joint |
| I | CLT wall panel |
| J | shear plate |
| K | concrete base beam connected to the foundation and waterproof layer |

Figure 13.3 — Walls and floors in CLT structures: (a) exterior and   
monolithic; (b) interior and segmented

### Verification in DC2

1. In CLT structures designed for DC2, each wall panel at every floor may be composed either of only one monolithic CLT panel or of multiple panels according to 13.7.1(5) when the design rules for DC3 do not apply.
2. The dissipative connections should be of type a) and b):
3. the shear connections between walls and the floor underneath, and between walls and foundation (usually the nailed or screwed connections between the angle brackets and the CLT wall panels, Figure 13.3(a) or the nailed or the screwed connections between the shear plates the CLT wall panel, Figure 13.3(b));
4. the anchoring connections against overturning placed at wall ends and at wall openings (usually the nailed or screwed connections between hold-down anchors and CLT wall panel, Figure 13.3(a) or between tie-downs and CLT wall panel, Fig. 13.3(b)).

Key

|  |  |
| --- | --- |
| A | joint between adjacent floor panels |
| B | floor panel |
| C | joint between floors and walls underneath |
| D | joint between perpendicular walls |
| E | wall panel |

Figure 13.4 — Connections and members to be designed with overstrength in order to fulfil the capacity design criteria in CLT structures in DC2

1. Vertical joints between adjacent parallel wall panels within the segmented shear walls may be designed either as dissipative or as non-dissipative.
2. 13.4.2(13) and (14) should be applied.
3. Structural members and joints listed in a) to d) (see Figure 13.4) and non-dissipative vertical joints should be capacity designed using Formula (13.7):
4. all CLT wall and floor panels;
5. joints between adjacent floor panels or joints of other types of sheathing material like in 13.7.1(9);
6. joints between floors and supporting walls underneath;
7. joints between orthogonal walls, including the ones at the building corners.

with the limitation *k*deg ≤ 1 (13.7)

where

|  |  |
| --- | --- |
| *F*Rd,b | is the design strength of the non-dissipative joint or structural member from Formula (13.2); |
| **Rd | is the overstrength factor, given in Table 13.4; |
| *k*deg | is the strength reduction factor defined in 13.3.1(1), for which the value given in 13.3.1(1) should be used; |
| *F*Ed,E | is the action effect in the non-dissipative joint or member due to the design seismic action; |
| *F*Ed,G | is the action effect in the non-dissipative joint or member due to the non-seismic actions in the seismic design situation; |
| *Ω*d | is the minimum value of all overstrength ratios *Ω*d.i, calculated by Formula (13.8); |

(13.8)

where

|  |  |
| --- | --- |
| *Ω*d.i | is the overstrength ratio at the *ith* storey calculated by Formula (13.9); |

(13.9)

where

|  |  |
| --- | --- |
| *V*Rd,LLRS,i | is the design lateral strength of the primary structure at the *ith* storey; |
| *V*Ed,E,LLRS,i | is the design global shear of the *ith* storey due to the seismic action. |

NOTE The design forces in the non-dissipative joints or structural elements can be taken not greater than the forces determined using a behaviour factor 𝑞 equal to 1.

1. When a unidirectional behaviour of the shear connections and the anchoring connections against overturning is assumed, so that shear connections and the anchoring connections against overturning are only considered effective along the shear-horizontal and tensile-vertical direction, respectively, the overstrength ratio *Ω*d,i of the primary structures at the *ith* storey should be calculated by Formula (13.10):

(13.10)

where

|  |  |
| --- | --- |
| *V*Rd,a,i,j | is the design lateral strength related to shear connections of the *jth* shear wall at the *ith* storey; |
| *M*Rd,rock,i,j | is the design rocking strength related to the anchoring connections against overturning of the *jth* shear wall at the *ith* storey including the stabilizing effect of the vertical load; |
| *V*Ed,E,i,j | is the design shear of the *jth* shear wall at the *ith* storey due to the seismic action; |
| *M*Ed,E,i,j | is the design rocking moment of the *jth* shear wall at the *ith* storey due to the seismic action; |
| *N*i | is the number of shear walls parallel to the seismic action at the *ith* storey. |

1. The design rocking strength *M*Rd,rock,i,j related to the anchoring connections against overturning of the *jth* monolithic shear wall at the *ith* storey including the stabilizing effect of the vertical load and neglecting the contribution of the shear connections should be calculated by Formula (13.11):

(13.11)

where

|  |  |
| --- | --- |
|  | is the tensile-vertical design strength of the anchoring connections against overturning of the *jth* shear wall at the *ith* storey; |
|  | is the total compressive axial load acting on the *jth* shear wall at the *ith* storey; |
| *B*CLT,i,j | is the length of the *jth* shear wall at the *ith* storey; |
|  | is the distance from the anchoring connections against overturning to the nearest vertical edge of the cross-laminated timber panel; |
|  | is the effective compressive design strength of the *jth* shear wall at the *ith* storey. It may be assumed to be equal to either (i) the compressive design strength parallel to the grain of wood laminations for shear walls at ground level when directly placed on concrete or steel members, or (ii) the compressive design strength perpendicular to the grain of wood laminations for the shear walls at the upper levels when placed on CLT floor panels; |
|  | is the effective compressive thickness of the *jth* shear wall at the *ith* storey. It may be assumed to be equal to (i) the total thickness of vertical layers for shear walls at the ground level when directly placed on either concrete or steel members, or (ii) the total thickness of the CLT panel for shear walls at the upper levels when placed on CLT floor panels. |

NOTE A rectangular stress-block model on 80 % of the length of the compression zone is adopted in the caldulation of the design rocking strength as shown in Figure 13.5.

Figure 13.5 — Calculation model for the design rocking strength of a single-panel CLT shear wall

1. The design lateral strength *V*Rd,a,i,j related to shear connections of the *jth* shear wall at the i*th* storey should be calculated as the sum of the horizontal-shear strength of all shear connections used to anchor the shear wall to the foundation or the floor underneath.
2. Alternatively to (6) to (8), the minimum value of all overstrength ratios may be assumed equal to 1,1.

NOTE (6) to (8) give a rigorous approach for the determination of .

1. The ductility of the dissipative connections defined in (2) should be achieved by satisfying either 13.4.2(7) or 13.4.2(11).
2. Brittle failures in the dissipative connections listed in (2), such as net tensile failure of timber, failure by row shear, block shear, plug shear and failure by splitting according to prEN 1995-1-1:2023, 11.6, should be avoided by satisfying Formula (13.4) with consideration of the overstrength factors given in Table 13.4 at connection level.
3. The overstrength factors in Table 13.4 at connection level should be used to satisfy Formula (13.4) such that brittle failure mechanisms of 2D- and 3D-connectors, listed in a) to f), do not occur:
4. withdrawal failure of anchor bolts or screws connecting the hold-down to the floor or the foundation;
5. shear failure of anchor bolts, screws or nails connecting the tie-down with the wall underneath or the foundation;
6. shear failure of screws or anchor bolts connecting the angle bracket or the shear plate with the floor or foundation underneath;
7. shear, tensile and pull-through failure of anchor bolts or screws;
8. steel plate tensile fracture in the weakest section of hold-down and tie-downs;
9. shear failure in the weakest section of angle brackets and shear plates.

### Verification in DC3

1. In CLT structures designed for DC3, the primary shear walls should all be segmented walls in each principal direction.
2. Segmented walls (Figure 13.3(b)) should be composed of more than one CLT panel, each with a length not smaller than 0,25*h*s and not greater than *h*s, where *h*s is the interstorey height.
3. In a segmented wall, the CLT panels should be connected with vertical joints made with metal fasteners (screws or nails) inserted perpendicular to the shear plane. Sheathing material according to 13.3.2(4) may be used in the joint.
4. In addition to 13.7.2(2), vertical screwed or nailed joints between adjacent parallel wall panels within the segmented shear walls should be designed for dissipative behaviour.
5. Structural members and joints listed in a) to d) in 13.7.2(5) should be capacity designed using Formula (13.7).
6. If the sum of the vertical stiffness of the shear resistant 2D- or 3D-connectors (angle brackets, shear plates) in a single panel of the segmented wall is smaller than one-fourth of the vertical stiffness of the anchoring 2D- or 3D-connectors against overturning (hold-downs, tie-downs, foundation tie-downs), and the latter ones are located at the corners of the wall (see Figure 13.5), 13.4.3(1) may be considered satisfied at wall and building level if (7), (8), (9) and (10) are verified.
7. For each segmented wall, conditions given by Formula (13.12) should be satisfied.

(13.12)

where

|  |  |
| --- | --- |
| *F*Rd,hd | is the design strength of the anchoring 2D- or 3D-connectors against overturning from Formula (13.1); |
| *F*Rd,c | is the design strength of the single timber-to-timber connection used in the vertical joint from Formula (13.1); |
| *K*SLS,anc | is the elastic stiffness of the anchoring 2D- or 3D-connectors against overturning; |
| *K*SLS,con | is the elastic stiffness of the single timber-to-timber connection used in the vertical joint; |
| *n*vj | is the number of fastener connections used in the vertical joint; |
| *N*Ed | is the total compressive axial load acting on the entire segmented shear wall; |
| *m*lp | is the number of CLT panels in a segmented wall (e.g. *m*lp  3 in Figure 13.6). |

1. The overstrength ratio, *Ω*d.i, of the primary structure at the *ith* storey should be calculated by Formula (13.13).

(13.13)

where

|  |  |
| --- | --- |
| *M*Rd,rock,i,j | is the design rocking strength related to the anchoring connections against overturning of the *j*th shear wall at the *i*th storey including the stabilizing effect of the vertical load; |
| *M*Ed,E,i,j | is the design rocking moment of the *j*th shear wall at the *i*th storey due to the seismic action. |

1. The maximum storey overstrength ratio max(*Ω*d,i) and the minimum storey overstrength ratio **d, with **d given by Formula (13.8), should verify Formula (13.14).

(13.14)

Key

|  |  |
| --- | --- |
| A | anchoring 2D- or 3D-connector against overturning (hold-down, tie-down, foundation tie-down) |
| B | single fastener connection in vertical joint |
| C | 2D- or 3D-connector resisting shear (angle-bracket, shear plate) |

Figure 13.6 — Segmented CLT wall

1. For each segmented wall, the shear connections should be verified using Formula (13.15).

(13.15)

where

|  |  |
| --- | --- |
|  | is the design strength of the shear connections; |
|  | is the action effect in the shear connections due to the design seismic action; |
| *M*Rd,rock | is the design rocking strength related to the anchoring connections against overturning of the shear wall including the stabilizing effect of the vertical load; |
| *M*Ed,E | is the design rocking moment of the shear wall due to the seismic action. |

1. In segmented walls with panels of the same length *b*CLT and vertical joints with the same type of fastener at the same spacing, when the wall is anchored against uplift at the corners and the shear connection do not affect rocking behaviour, the design rocking strength *M*Rd,rock should be calculated by Formula (13.16).

(13.15)

where

|  |  |
| --- | --- |
| *M*Rd,rock | is the design rocking strength; |
| *F*Rd,hd | is the design strength of the anchoring 2D- or 3D-connector against overturning calculated using Formula (13.1); |
| *F*Rd,c | is the design strength of the single timber-to-timber connection used in the vertical joint using Formula (13.1); |
| *n*vj | is the number of fasteners used in the vertical joint; |
| *b*CLT | is the length of each single panel. |

1. The ductility of the dissipative connections defined in (4) and in 13.7.2(2) should be achieved by satisfying either 13.4.2(7) or 13.4.2(10).
2. 13.7.2(10) should be applied to the dissipative connections defined in (4) and in 13.7.2(2).
3. 13.7.2(11) should be applied to 2D- and 3D-connectors defined in 13.7.2(2).

### Detailing rules

1. In the case of tension perpendicular to the grain, additional design measures should be taken to avoid splitting (e.g. by reinforcing with screws, glued-in rebars, nailed metal plates or plywood plates).
2. CLT wall-to-wall and floor-to-wall connection screws should be inserted inclined (as per Figure 13.7 C) unless a correct insertion of cross layer screws within layers with grain direction perpendicular to the screw axis can be ensured (Figure 13.7 B).

Key

|  |  |
| --- | --- |
| A | wrong – screws inserted in layers with grain direction parallel to the screw axis |
| B | correct, but difficult to achieve – screws inserted in layers with grain direction perpendicular to the screw axis |
| C | correct – screws inserted inclined |

Figure 13.7 — CLT wall-to-wall connection

## Rules for framed wall structures

### General rules

1. The primary system should be made of shear walls composed by timber frames with a sheathing material (Figure 13.8). The upper walls should bear on the floor (platform frame construction – see 3.1.20).
2. In framed shear walls, the frames (Figure 13.8 F) should be composed by equally spaced vertical studs (Figure 13.8 G), a bottom plate (Figure 13.8 H) and a top plate.
3. The sheathing material should be connected to the frame on one or both sides by means of screws, nails or staples (Figure 13.8 C).
4. Floor and roof diaphragms should be made according to 13.15.3.

+

Key

|  |  |  |  |
| --- | --- | --- | --- |
| A | nailed, screwed or stapled sheathing-to-framing floor connection | G | vertical stud |
| B | floor-to-wall connection | H | plate |
| C | nailed, screwed or stapled sheathing-to-framing wall connection | I | perpendicular wall-to-wall connection |
| D | base and interstorey anchoring connection against overturning | J | joist |
| E | base and interstorey shear~~-~~ connection against sliding | K | blocking |
| F | framing member | L | perimeter edge beam |

Figure 13.8 — Example of framed wall structure with typical connections

1. Other types of floor and roof diaphragms than in (4) may be used, provided that their in-plane resistance is ensured (e.g. 13.15.2 or 13.15.4).
2. The joint of the walls to the foundation or to the walls underneath should be made by means of anchoring 2D- or 3D-connectors (e.g. hold-downs (Figure 13.8 D), angle brackets, tie-downs) and/or shear connections (e.g. anchor bolts (Figure 13.8 E), nails and screws) and should restrain the wall against overturning and sliding.
3. In fully anchored walls, specific anchoring connections against overturning (hold-downs and tie-downs, Figure 13.8 D) should be placed at wall ends and at opening ends.

NOTE If a vertical action applied directly either on top or onto the framing member at wall ends and at opening ends in accordance with prEN 1995-1-1:2023, 13.3.2(1), is used as an alternative to specific anchoring connections against overturning, the walls cannot be regarded as fully anchored. They can be designed for earthquake actions by following the provisions for non-fully anchored walls according to (9).

1. Connections against sliding (Figure 13.8 E) should be distributed uniformly along the wall length.
2. In non-fully anchored walls, as an alternative to (7), walls may be restrained against overturning, according to prEN 1995-1-1:2023, 13.3.2(1), by vertical action applied directly onto or immediately above the framing member at wall ends and at opening ends.
3. Wall height at each level of the building should be the same as the interstorey height.
4. Perpendicular walls should be connected by joining together two vertical studs with mechanical fasteners (nails, screws or bolts, Figure 13.8 I).

### Verification in DC2

1. Shear walls may be sheathed with all the types of sheathing material defined in 13.3.2(4), connected to the wall framing by means of nails, screws or staples according to EN 14592:2022, Annex F.
2. Framed wall structures designed to DC2 should be made only of fully anchored walls.
3. Connections with dissipative behaviour should be those given in a) to c):
4. screwed, nailed or stapled connections between sheathing material and timber frame in shear walls;
5. shear connections between upper and lower walls, and between walls and foundation;
6. anchoring connections against overturning placed at wall ends and at wall openings.
7. Structural members, connections and joints listed in a) to e) (see Figure 13.8) should be capacity designed using Formula (13.7), with *Ω*d given by Formula (13.8) and *Ω*d,i given by Formula (13.17):
8. sheathing panels under in-plane shear induced by seismic actions;
9. timber framing members (studs, plates, and joists) under axial forces induced by seismic actions;
10. nailed, screwed or stapled shear connections between sheathing and timber joists/beams at each floor;
11. joints between floors and walls underneath;
12. joints between orthogonal walls, particularly at the building corners.

(13.17)

where

|  |  |
| --- | --- |
| *V*Rd,sh,i,j | is the design lateral strength related to connections between sheathing material and timber frame of the *j*th shear wall at the *i*th storey calculated according to prEN 1995-1-1:2023, 13.3.2, Formula (13.13); |
| *V*Rd,a,i,j | is the design lateral strength related to shear connections of the *j*th shear wall at the *i*th storey; |
| *M*Rd,rock,i,j | is the design rocking strength of the *j*th shear wall at the *i*th storey; |
| *V*Ed,E,i,j | is the design global shear of the *j*th shear wall at the *i*th storey due to the seismic action; |
| *M*Ed,E,i,j | is the design rocking moment of the *j*th shear wall at the *i*th storey due to the seismic action; |
| *N*i | is the number of shear walls parallel to the seismic action at the *ith* storey. |

1. As an alternative to a rigorous approach using Formula (13.8) and Formula (13.17), the minimum value of all overstrength ratios may be assumed to be equal to 1,1.
2. 13.7.2(9) should be applied for the ductility of the dissipative connections defined in (3).
3. 13.7.2(10) should be applied to the dissipative connections defined in (3).
4. 13.7.2(11) should be applied.

### Verification in DC3

1. In framed wall structures designed for DC3, only plywood or OSB panels as defined in 13.3.2(4) should be used as sheathing material. The sheathing material should be connected to the wall framing only by means of nailsaccording to EN 14592:2022, Annex F. Stapled and screw connections should not be used for the sheathing-to-frame connection. For smooth nails, the anchorage depth *t*w should be increased by 25 % compared to the minimum values according to prEN 1995-1-1:2023, 11.2.3, Table 11.2.
2. 13.8.2(2) should be applied.
3. Connections with dissipative behaviour should be only the nailed connections between sheathing material and timber frame in the shear walls.
4. Capacity designed structural members should be (Figure 13.8) all elements, 2D- and 3D-connectors, and connections except nails in dissipative zones, listed in a) to c):
5. all those given in a) to e) in 13.8.2(4);
6. shear connections between upper and lower walls, and between walls and foundation;
7. anchoring connections against overturning placed at wall ends and at wall openings.
8. The non-dissipative structural members, 2D- and 3D-connectors and connections listed from a) to c) in (4) should satisfy Formula (13.7) with *Ω*d given by Formula (13.8) and *Ω*d,i given by Formula (13.18).

(13.18)

where

|  |  |
| --- | --- |
| *V*Rd,sh,i,j | is the design lateral strength related to connections between sheathing material and timber frame of the *j*th shear wall at the *i*th storey calculated according to prEN 1995-1-1:2023, 13.3.2, Formula (13.13); |
| *V*Ed,E,i,j | is the design global shear of the *j*th shear wall at the *i*th storey due to the seismic action; |
| *N*i | is the number of shear walls parallel to the seismic action at the *ith* storey. |

1. 13.7.3(9) should be applied.
2. 13.7.3(12) should be applied for the ductility of the dissipative connections defined in (3).
3. 13.7.2(10) should be applied to the dissipative connections defined in (3).

### Detailing rules

1. 13.7.4(1) should be applied.

## Rules for log structures

### General rules

1. The primary structure should be made by the superposition of rectangular or round solid or glulam elements (‘logs’), prefabricated with upper and lower grooves.
2. The joint between orthogonal walls should be made by means of carpentry connections obtained by notching the logs of the two walls or by means of screws, dowels or bolts (Figure 13.9).
3. Floor and roof diaphragms may be detailed according to 13.15.2 to 4.
4. 13.8.1(6) to (8) and 13.8.1(10) should be applied (Figure 13.9).

Key

|  |  |
| --- | --- |
| A | possible steel rods as uplift restraint for timber logs |

Figure 13.9 — Typical corner joint and connection details in log structures

1. Uplift of the logs due to overturning moment should be prevented by either a) or b):
2. By verifying that the stabilizing moment due to the non-seismic actions in the seismic design situation is greater than or equal to the overturning moment due to seismic action multiplied by the overstrength factor **Rd according to Formula (13.19).

(13.19)

where

|  |  |
| --- | --- |
| **Rd | is the overstrength factor, given in Table 13.4; |
| *M*dst,d,E | is the overturning moment due to the design seismic action; |
| *M*stb,d,G | is the stabilizing moment due to the non-seismic actions in the seismic design situation. |

1. If Formula (13.19) is not satisfied, uplift is resisted by using steel tie-rods or screws (Figure 13.9).

Key

|  |  |
| --- | --- |
| A | perpendicular to the grain verification area |
| B | longitudinal shear verification area |

Figure 13.10 — Perpendicular to the grain and longitudinal shear verification of a typical carpentry connection between logs: (a) plan view of a log; (b) cross-section of a log; (c) side view of a log; (d) side view of a joint with dashed compression perpendicular to the grain verification area; (e) 3D view of a connection with dashed compression perpendicular to the grain verification area; (f) 3D view of a log with dashed longitudinal shear verification area

### Verification in DC2

1. Log structures should be designed to behave as box-type structures.
2. Energy dissipation should take place by friction at the interface between superimposed logs, and compression perpendicular to the grain in the carpentry connections between orthogonal walls.
3. Timber diaphragms, timber logs and their connections to the foundation or between any massive sub-element should be capacity designed by complying with Formula (13.4).
4. Carpentry connections should be capacity designed against shear failure by complying with Formula (13.5) in accordance with 13.4.3(5).

### Detailing rules

1. 13.7.4(1) should be applied.
2. Compression members and their connections (e.g. carpentry connections) should be designed to prevent separation of elements.
3. The shear resistance of shear walls should be provided by the perpendicular to the grain and longitudinal shear resistance of carpentry connections obtained by notching the logs of the two walls like in Figure 13.10.
4. Alternative solutions by means of screws, dowels or bolts like in Figure 13.11 may be used but in such case the design shear resistance should be entirely due to the mechanical connection without any possible contribution from the carpentry connections.

Key

|  |  |
| --- | --- |
| A | connection between timber logs by means of self-tapping screws |
| B | bolted connection to foundation (right: side view; left: cross-section) |

Figure 13.11 — Connection between timber logs by means of self-tapping screws

## Rules for moment-resisting frames

### General rules

1. Moment-resisting frames should be made of timber members connected with semi-rigid joints made of dowel-type metal fasteners. The column-foundation connections may be pinned or semi-rigid.
2. 13.9.1(3) should be applied.

### Verification in DC2

1. In order to ensure yielding of the fasteners in the dissipative joints, the timber members and metal plates used in joints should be capacity designed according to Formula (13.4), where the timber members and metal plates are the brittle components and the semi-rigid joints the ductile ones.
2. 13.7.2(9) should be applied for the ductility of the dissipative joints.
3. 13.7.2(10) should be applied to the dissipative connections.
4. The columns should be designed to the magnified axial force *N*Ed given by Formula (13.20):

(13.20)

where

|  |  |
| --- | --- |
| *N*Ed,G | is the axial force in the column due to the non-seismic actions in the seismic design situation; |
| *N*Ed,E | is the axial force in the column due to the design seismic action; |
| ** | is the seismic action magnification factor; |
| "+" | means combined with + or – sign. |

1. The seismic action magnification factor ** should be taken as given in a) to c), as appropriate:
2. 1,6 for single-storey moment-resisting frames;
3. 1,8 for multi-storey, one-bay moment-resisting frames;
4. 2,0 for multi-storey, multi-bay moment-resisting frames.

### Verification in DC3

1. 13.10.2(1) should be applied.
2. 6.2.7(3) should not be applied.
3. High-ductility joints shall be used, namely joints with a rotational ductility *μ* defined in accordance with EN 12512 not less than 7 determined accordingly, with a value of *k*deg defined in 13.3.1(1) not less than 0,8 and a value of *φi*mp defined in 13.3.1(1) not greater than 0,3.

NOTE Joints with expanded tube fasteners reinforced with densified veneer wood designed and constructed in accordance with prEN 1995-1-1:2023, Annex R, can be considered as high ductility joints according to (3).

1. 13.7.3(9) should be applied.
2. 13.7.3(12) should be applied for the ductility of the high ductility joints.
3. 13.7.2(10) should be applied to the dissipative connections.
4. 13.10.2(4) and (5) should be applied.

### Detailing rules

1. 13.7.4(1) should be applied.
2. Dowels, smooth nails and staples should be used with measures against withdrawal (e.g., some fasteners with withdrawal resistance, see Figure 13.12) to avoid member separation due to possible out-of-plane loads. Such measures may be omitted in secondary members.
3. Splitting of wood due to secondary tensile stresses induced by the bending moment should be avoided by reinforcing the connection region with screws or glued-in rods (Figure 13.12). Reinforcing elements should be designed according to prEN 1995-1-1:2023, 11.4.2 and 11.2, to resist a tensile force *F*t,90.Ed given by Formula (13.21).

(13.21)

where

|  |  |
| --- | --- |
| *F*t,90,Ed | is the design tensile force in the reinforcement; |
| *F*M,Ed | is the design shear force in a fastener in the outer circle due to the bending moment transmitted by the joint; |
| *n*e | is the number of fasteners in the outer circle. |

Key

|  |  |
| --- | --- |
| A | self-tapping screws or glued-in rods reinforcement |
| B | dowels |
| C | bolts |

Figure 13.12 — Possible detail to avoid splitting of wood and member separation in the joint region

1. In high ductility joints with expanded tube fasteners according to prEN 1995-1-1:2023, Annex R, the connection area should be reinforced with glued densified veneer wood in accordance with 13.3.2(4) f) or any equivalent type of reinforcing panels in order to allow yielding of the metal tube.

## Rules for braced frame structures with dowel-type connections

### General rules

1. The primary structure should be composed of timber trusses with dowel-type connections as defined in 3.1.9. The secondary structure should be made of timber columns and beams pin-jointed with dowel-type connections.
2. Primary reinforced concrete walls and cores, designed according to Clause 10, may be used as an alternative to primary timber bracings. In such a case the behaviour factors in Table 10.1 should be used.
3. Primary steel bracings with the frame and the diagonals made of steel, designed according to Clause 11, may be used as an alternative to primary timber bracings. In such a case the behaviour factors in Table 11.2 should be used.
4. Primary structure with a timber frame and steel diagonals pinned connected to the timber frame may be used as an alternative to primary timber bracings. In such a case the steel diagonals should be designed according to Clause 11, and the behaviour factors listed in Table 11.2 should be used.
5. Framed or CLT shear walls may be used as an alternative to primary timber bracings. They should be designed for DC2 according to 13.8 or to 13.7 respectively. In such a case the relevant behaviour factor for framed wall and CLT structures in Table 13.2 for DC2 should be used.
6. 13.9.1(3) should be applied.

### Verification in DC2

1. In order to ensure yielding of the fasteners, the timber members, metal plates in the joint, and floor diaphragms should be capacity designed according to Formula (13.4), where the timber members, metal plates in the joint, and floor diaphragms are the brittle components and the metal fasteners the ductile ones.
2. 13.7.2(9) should be applied for the ductility of the dissipative connections.
3. 13.7.2(10) should be applied to the dissipative connections.
4. When reinforced concrete walls, steel bracings, framed shear walls or CLT walls are used as primary structure according to 13.11.1(2), 13.11.1(3) and 13.11.1(5) as an alternative to timber trusses with dowel-type connections, the beam-to-column pinned joints, the timber members and the floor diaphragms. should be capacity designed according to Formula (13.4).
5. When a timber frame with steel diagonals is used as primary structure according to 13.11.1(4) as an alternative to primary timber trusses with dowel-type connections, the timber frame, the diagonal-to-frame pinned connections, the timber members and floor diaphragms should be capacity designed using Formula (13.4).

### Detailing rules

1. 13.7.4(1) should be applied.
2. Mechanical joints with dowel-type fasteners between beams and columns should be always regarded as pinned. Only glued joints (e.g. with glued-in rods) may be regarded as rigid as defined in 3.1.25.
3. Brittle failures of dowel-type connections due to splitting of wood caused by shrinkage perpendicular to the grain in the connection area should be prevented by using a) to c):
4. ovalized holes in the timber and metal plates, for the free movement of wood;
5. as few fasteners as possible, near to each other in a way not to restrain shrinkage of wood perpendicular to the grain;
6. reinforcement in the connection region to resist the tension perpendicular to the grain due to the restrained shrinkage of wood.
7. Reinforcement may be used in post-to-beam connections against brittle failure due to load reversal.

## Rules for vertical cantilever structures

### General rules

1. Vertical cantilever structures should be composed of either walls or columns made of CLT, glulam, LVL or GLVL according to 13.3.2(1) and (2) connected to the foundation and floors by means of mechanical devices (usually, metal plates connected with dowel-type fasteners to the wall or column, see Figure 13.13).
2. The walls or columns should be structurally continuous across the floors (balloon frame construction – see 3.1.2).
3. 13.9.1(3) should be applied.

Figure 13.13 — Vertical cantilever wall with metal plates and dowel-type fasteners

### Verification in DC2

1. In order to ensure yielding of the fasteners at the wall-foundation or column-foundation joint, the timber members, the metal plates, the floor diaphragms and the other possible wall or column spliced joints should be capacity designed according to Formula (13.4), where the ductile components are the connections with dowel-type fasteners.
2. 13.11.2(2) and 13.11.2(3) should be applied.
3. 13.7.2(11) should be applied to the metal plates linked to the dissipative connections.

### Detailing rules

1. 13.7.4 should be applied.
2. The intermediate floor-to-wall or floor-to-column joint (see Figure 13.14) should be capacity designed using Formula (13.4).

Key

|  |  |
| --- | --- |
| A | possible intermediate floor-to-wall joint |

Figure 13.14 — Joint between intermediate floor and shear walls

## Rules for braced frame structures with carpentry connections and interacting masonry infill

### General rules

1. The primary structure should be made by timber frames with carpentry connections between posts and beams, and a masonry infill inside.
2. Bracings such as diagonal timber members with carpentry connections may be added in the timber frames.
3. Finishings, such as a plaster layers, should be detailed so that the increase in lateral stiffness of the wall is negligible.
4. The joint of orthogonal walls should be made by means of carpentry connections, connections with dowel-type fasteners or steel plates to promote a box-type structural behaviour.
5. The joint between the walls and the timber diaphragm should prevent uplifting of the posts and out-of-plane overturning mechanisms of the walls.
6. When external masonry walls are used, either a timber or a steel skeleton should be embedded into these walls and connected to the inner timber frame walls with carpentry connections and masonry infills to ensure a box-type behaviour. Carpentry connections, connections with dowel-type fasteners, steel plates or injected anchors should be used to connect the skeleton of the external masonry walls to the inner timber frame walls.
7. 13.9.1(3) should be applied.
8. The joints of the walls to the foundation should be made by means of 2D- or 3D-connectors (hold-downs, angle-brackets), tie rods, anchor bolts, nails and screws, etc.
9. If timber frame walls with carpentry connections and masonry infills are used above the masonry walls of a lower floor, the timber frames should be connected (e.g. using 2D- or 3D-connectors) to a perimeter edge beam linked to the masonry walls.
10. The analysis and design of timber frames with infills should satisfy 7.4.

### Verification in DC2

1. Buildings with timber frame walls with carpentry connections and interacting masonry infill should be designed to behave as box-type structures.
2. Local failures which may compromise the box-type behaviour should be prevented.
3. Energy dissipation due to friction should take place in the walls at the interface between posts, beams and bracing members, and the infill.
4. Floor diaphragms and the joints with the walls should be capacity designed using Formula (13.4).
5. Carpentry connections should be capacity designed to prevent brittle shear failure using Formula (13.5) in accordance with 13.4.3(5).

### Detailing rules

1. 13.7.4(1) and 13.9.3(2) should be applied.
2. Joints between bracing members (e.g. diagonal timber members) and the timber frame (posts and beams) should allow movement (e.g., by using nails or screws in timber-to-timber joints).

## Rules for braced frame structures with carpentry connections

### General rules

1. Braced frame structures with carpentry connections should be composed of timber columns and beams pinned connected with mechanical joints and of eccentric timber diagonals connected to beams with carpentry connections (Figure 13.15). They should not be used in buildings of more than two-storeys, with an interstorey height not exceeding 4,5 m.
2. Since carpentry connections work in compression, a bracing wall should have two symmetrical diagonals which should not cross each other.
3. The pinned connection between columns and beams should be made either of dowel-type fasteners or 3D-connectors like hold-downs.
4. 13.9.1(3) should be applied.

Figure 13.15 — Examples of braced frame structures with carpentry connections

### Detailing rules

1. 13.7.4(1) should be applied.
2. 13.9.3(2) should be applied.

## Verification of floor and roof diaphragms

### General rules

1. Floor and roof diaphragms should be made according to 13.15.2, 13.15.3 or 13.15.4. Other types of floors such as those made of solid wood panels listed in 13.3.2(2) may be used provided that they are capable of transferring in-plane horizontal actions to the primary structure.
2. Openings, discontinuities and irregularities in horizontal diaphragms should be designed to ensure continuity of in-plane shear and flexural forces and to avoid any type of failures including those due to stress concentration. Analysis and design should be based on strut-and-tie, truss or finite-element models to determine an effective load path. Where required by the design, framing of the openings, in the form of perimetral beams, supporting walls or ties, should be provided.
3. Perimeter diaphragm chords should be assessed for the tensile and compressive forces arising from diaphragm action. Chords should always incorporate a perimeter beam to resist the full tensile and compressive forces, unless properly connected supporting walls are provided underneath the diaphragm.
4. 6.2.8(2) may be applied for the design to DC2 and DC3 of timber diaphragms for building systems other than CLT and framed shear walls with the value of *γ*d for brittle failure as given in Table 6.1 multiplied by 1,5.

### Cross laminated timber (CLT) floor and roof diaphragms

1. CLT floor and roof diaphragms should be made of CLT panels according to 13.3.2(1) connected together and to the supporting walls or beams using metal fasteners (screws and nails).

Key

|  |  |
| --- | --- |
| A | joints between horizontal CLT panels staggered with respect to joints between vertical panels |
| B | joints between horizontal CLT panels corresponding to joints between vertical CLT panels |
| C | steel strap to connect the CLT floor panels and transfer the tension forces due to the horizontal seismic actions |

Figure 13.16 — Correspondence between horizontal joints in CLT diaphragms and   
vertical joints in CLT walls

1. Holes for installations placed across two CLT panels should be such that they ensure the transmission of horizontal seismic forces in the panel-to-panel reduced length of joint.
2. Joints between horizontal panels in CLT diaphragms should be staggered with respect to joints between vertical panels in CLT walls like in Figure 13.16 A). If they are not staggered like in Figure 13.16 B, additional measures (such as continuous beams or steel straps like in Figure 13.16 C) should be taken to connect the floor panels above the wall and across the joint in order to transfer the tension forces due to the in-plane bending moment.

### Framed floor and roof diaphragms

1. Framed floor and roof diaphragms should be composed of equally spaced beams or joists (Figure 13.8 J) and timber blocking (Figure 13.8 K) in between and sheathing panels.
2. The sheathing panels should be placed on top of the beams, joists and blockings, and connected by means of screws, nails or staples along the whole perimeter of the sheathing panel (Figure 13.8 A).
3. At each floor, a perimeter edge beam (Figure 13.8 L) should be provided to resist the tensile and compressive forces arising from the diaphragm action when the floor is loaded by horizontal forces acting in its plane.
4. Sheathing edges not meeting on framing members should be supported on and connected to the transverse blocking placed between the wooden joists.
5. Blocking should be provided in the horizontal diaphragms above the primary vertical members (e.g. walls).
6. Alternative systems to (4) and (5) like steel straps, wooden boards or plywood straps running along the unsupported edge of the sheathing panel may replace the timber blocking. In that case, such a system should be designed to transfer the in-plane shear forces between adjacent sheathing panels.
7. The structural continuity of the beams and joists along the perimeter of the floor diaphragm should be verified; such beams and joists should be designed to resist the tension and compression forces due to the in-plane seismic forces and transfer them to the primary structure.
8. Structurally continuous trimmer joists should be placed around holes in the diaphragms and designed to resist the tension and compression forces due the in-plane seismic forces.
9. If intermediate transverse blocking is not present in the floor diaphragm over the full depth of the beams and joists, the depth-to-width ratio (*h/b*) of the beams and joists should not be greater than 4.
10. If *S*δ > 2 m/s2, the spacing of fasteners in areas of discontinuity calculated according to prEN 1995‑1‑1:2023, 13.4.2, 13.4.3 and 13.4.4, should be reduced by 25 %, but not to less than the minimum spacing specified in prEN 1995-1-1:2023, 13.2.2(9).
11. prEN 1995-1-1:2023, 14.1, 14.2 and 14.4, 13.1, 13.2 and 13.4, should be applied.

### Timber-concrete composite floor and roof diaphragms

1. Timber-concrete composite floor and roof diaphragms according to the CEN/TS 19103 should be jointed to the lower and upper walls by means of connections with metal fasteners.
2. The concrete topping should be connected to the vertical primary members to ensure the in-plane shear due to the diaphragm action is transferred to the walls underneath and down to the foundations.

## Transfer zones. Design for DC2 and DC3

1. Diaphragms in transfer zones as defined in 3.1.32 should be designed according to 6.2.11 and 13.15.
2. 6.2.11 should be applied only for buildings up to 4 storeys. Above four storeys, at least 80 % of the primary structure should be structurally continuous, in each main direction, from the base of the timber part of the building up to the roof (Figure 13.1 A).
3. **d in 6.2.11(9) should be taken equal to the value given in Table 13.4 for capacity design at wall and building level.
4. The flexibility of the timber floor diaphragm should be taken into account.
5. The resistance of the diaphragm to in-plane forces, calculated according to (1), should be checked.

## Checking of design and construction data

1. The structural elements in a) to h) should be identified on the design drawings:
2. details and plan of the uplift and sliding restraint connections of all the primary vertical members to foundation and interstorey floor elements;
3. details of joints between horizontal diaphragms and primary vertical members underneath;
4. details of joints between perpendicular or intersecting primary vertical members;
5. details of connections between sheathing panels and timber framing in horizontal and vertical diaphragms;
6. details of joints between CLT panels in horizontal diaphragms and vertical walls;
7. details of connections in diagonal tension and compression timber trusses used for bracing;
8. details and sizes of any carpentry connection;
9. details of any tension or perpendicular to the grain reinforcement in timber members or connection area.
10. The control during construction should be performed according to prEN 1995-3:2023, Annex G, on execution of timber structures.

# Specific rules for masonry buildings

## General

1. Clause 14 should be applied to the design and the verification of masonry buildings in seismic regions.
2. Clause 14 should be applied as a complement to EN 1996-1-1.

NOTE Masonry infilled frames are covered in 7.4.

1. Clause 14 should be applied to unreinforced masonry, confined masonry and reinforced masonry as defined in EN 1996-1-1 with the complementary rules defined herein.

## Basis of design

### Design concepts

1. Masonry buildings shall be designed to achieve a box behaviour of the building.

NOTE In a box behaviour, the horizontal components of the seismic action and the consequent inertia forces are transferred to the foundation prevailingly through in-plane forces in walls and diaphragms; the out-of-plane stability of walls is improved because the restraint reactions on their boundaries can be transferred to the foundation or equilibrated by in-plane action in other walls and/or diaphragms. See also 3.1.4.

1. Masonry buildings should be designed to either DC1 or DC2.
2. For *S*δ greater than 3 m/s2, unreinforced masonry buildings should be designed to DC2.

### Rules applicable to structures designed to DC1 or DC2

1. Masonry buildings should be composed of floors and walls.
2. There should be walls in two principal directions; they should be connected to each other at intersections and to the floors above and below.
3. Floors should be designed to act as diaphragms; to this end, a) to f) should be satisfied:
4. there should be diaphragms, ring ties or ring beams at each floor level, at a vertical spacing not exceeding 4 m;
5. diaphragms should restrain wall displacements in the direction perpendicular to wall planes;
6. diaphragms should possess in-plane shear stiffness;
7. ring ties or ring beams may be part of the diaphragms;
8. ring beams, ring ties, diaphragms and walls should be connected together;
9. ring beams, ring ties, diaphragms, joints internal and peripheral to diaphragms should be designed to resist design action effects and satisfy 6.2.8.

NOTE For ring beams and ring ties, rules are given in EN 1996-1-1:2022, 10.5.1.4, and additional rules for DC2 buildings in 14.7.2 to 14.7.4. Rules for timber framed floor and roof diaphragms are given in 13.15.3 and 13.15.4.

1. Unreinforced masonry buildings with flexible diaphragms should be designed to DC1 and satisfy 14.5.1.3.
2. Unreinforced masonry buildings with rigid diaphragms may be designed to DC1 or DC2.

NOTE A great in-plane shear stiffness of the diaphragms is necessary to activate the redundancy against horizontal actions which characterizes DC2. Definitions of rigid diaphragms are given in 14.5.1.3(3) and (4).

1. Masonry slab systems with a concrete topping which satisfy 10.12.1(2) may be considered as rigid diaphragms.

## Materials

1. The normalized compressive strength *f*b of masonry units determined in accordance with EN 772-1 should satisfy a) or b) or c):
2. In DC1 for *S*δ ≤ 3 m/s2: Normal to bed face: *f*b = *f*bv ≥ 3 MPa
3. In DC2 for *S*δ ≤ 3 m/s2: Normal to bed face: *f*b = *f*bv ≥ 3 MPa

Parallel to bed face in the plane of the wall: *f*b  = *f*bh ≥ max{0,1 *f*bv; 1,5 MPa}

1. In DC2 for *S*δ > 3 m/s2: Normal to bed face: *f*b = *f*bv ≥ 4 MPa for Group 1

*fb = f*bv ≥ 3 MPa for Group 1 with thin layer mortar

*f*b = *f*bv ≥ 5 MPa for groups 2 to 4

Parallel to bed face in the plane of the wall: *f*b = *f*bh ≥ max{0,1 *f*bv; 2,0 MPa}

1. For unreinforced masonry and confined masonry, the strength of general purpose mortar as defined in EN 1996-1-1:2022, 3.4.2, should not be smaller than given in a) or b):
2. for *S*δ ≤ 3 m/s2: *f*m,min = 2,5 MPa
3. for *S*δ > 3 m/s2: *f*m,min = 5 MPa
4. For reinforced masonry with reinforcement embedded in general purpose mortar, the strength of mortar should not be smaller than *f*m,min = 10 MPa.
5. Values of *f*b smaller than in (1) may be used if the resulting characteristic compressive strength of masonry *f*k, according to EN 1996-1-1:2022, 5.7.1, is not smaller than given in a) or b):
6. for *S*δ ≤ 3 m/s2:

Normal to the bed face: *f*k= *f*kv ≥ 1,6 MPa for unreinforced and confined masonry;

*f*k= *f*kv ≥ 2,8 MPa for reinforced masonry;

Parallel to the bed face: *f*k= *f*kh ≥ 0,15 *f*kv for unreinforced, confined and reinforced masonry;

1. for *S*δ > 3 m/s2:

Normal to the bed face: *f*k= *f*kv ≥ 2,2 MPa for unreinforced and confined masonry;

*f*k= *f*kv ≥ 3,5 MPa for reinforced masonry;

Parallel to the bed face: *f*k= *f*kh ≥ 0,15 *f*kv for unreinforced, confined and reinforced masonry.

1. For the Significant Damage (SD) limit state verifications, partial factors, partial factors for masonry and steel strengths *γ*M and *γ*s not smaller than 1,0 should be used.

NOTE *γ*M and *γ*s are equal to 1,5 and 1,15 respectively, unless the National Annex gives different values for use in a country.

## Behaviour factors

### Behaviour factors for in-plane analysis

#### Design for DC1

1. In DC1, a behaviour factor *q* up to 1,5 may be used, regardless of the structural type, the masonry type, the diaphragm stiffness and the regularity.

#### Design for DC2

1. The default values of the behaviour factor components *q*R and *q*D and of the behaviour factor *q* for the design in DC2 of buildings regular in elevation and not torsionally flexible are given in Table 14.1; the values of *qD* in Table 14.1 require that Formula (14.1) and (4) and (5) be satisfied in all primary walls:

(14.1)

where **G,k,i  is the wall axial load ratio in wall *i* due to non-seismic actions calculated as given in (4).

Table 14.1 — Structural layout, masonry types and default values of the behaviour factor *q* in DC2

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Structural layout in the earthquake direction** | ***q*R** | **Masonry type** | ***q*D** | ***q*** |
| At least 6 walls of various lengths and a significant coupling effect | 1,4 | URM (general) | 1,2 | 2,6 |
| 1,4 | Calcium silicate (hollow and solid) | 1,0 | 2,2 |
| 1,4 | AAC Gr 1 and 1s | 1,4 | 2,8 |
| 1,4 | URM Gr 1 and 1s clay | 1,6 | 2,8 |
| 1,4 | Confined masonry (general) | 1,65 | 3,4 |
| 1,4 | Reinforced masonry (general) | 1,8 | 3,8 |
| At least 6 walls of various lengths without significant coupling effect, or at least 4 walls and a significant coupling effect | 1,2 | URM (general) | 1,2 | 2,1 |
| 1,2 | Calcium silicate (hollow and solid) | 1,0 | 1,8 |
| 1,2 | AAC Gr 1 and 1s | 1,4 | 2,5 |
| 1,2 | URM Gr 1 and 1s clay | 1,6 | 2,8 |
| 1,2 | Confined masonry (general) | 1,65 | 2,9 |
| 1,2 | Reinforced masonry (general) | 1,8 | 3,2 |
| Less than 6 walls of various lengths, and no significant coupling effect; | 1,0 | URM (general) | 1,2 | 1,8 |
| 1,0 | Calcium silicate (hollow and solid) | 1,0 | 1,5 |
| 1,0 | AAC Gr 1 and 1s | 1,4 | 2,1 |
| 1,0 | URM Gr 1 and 1s clay | 1,6 | 2,4 |
| 1,0 | Confined masonry (general) | 1,65 | 2,5 |
| 1,0 | Reinforced masonry (general) | 1,8 | 2,7 |
| Buildings with flexible diaphragms | 1,0 | Confined masonry | 1,35 | 2,0 |
| 1,0 | Reinforced masonry | 1,6 | 2,4 |
| 1,0 | Unreinforced masonry | 1,0 | 1,5 |

NOTE Hybrid structures are covered in 4.4.1(4) and (5) and their behaviour factor in 5.3.2(4).

1. In Table 14.1, walls in one direction should be considered to have various length if the longest length of each of 75 % of the walls, or each of the percentage just below 75 %, is smaller than 0,8 times the length of the longest of all walls in that direction.
2. In Table 14.1, a coupling effect by floors, ring beams and/or reinforced or confined spandrels may be considered significant if a) and b) are satisfied:
3. the building is more than one storey high;
4. the stiffness of the building is at least 1,5 times greater when the coupling is considered than when it is not considered.
5. The axial load ratio **G,k,i should not be greater than 0,4 in any primary wall *i* and the average axial load ratio of primary walls in each horizontal direction of any storey, **G,k,av, should not be greater than 0,2, where:

|  |  |
| --- | --- |
| G,k,i= *N*Ed,G*,*i/*A*p,i *f*ki | is the axial load ratio in the primary wall *i* |
| G,k,av= *N*Ed,G*,*i/(*A*p*f*ki) | is the average axial load ratio; |
| *i* | refers to a primary wall *i* in the considered direction; |
| *N*Ed,G,i | is the axial load in the considered wall *i* due to non-seismic actions in the seismic design situation; |
| *A*p,i | is the area of the considered primary wall, calculated as *A*p,i = *l*mp,i x *t*ef,I; |
| *f*ki | is the characteristic compressive strength of the considered primary wall; |
| *tef,i* | is the effective thickness of the wall or pier *i* as given in EN 1996‑1‑1:2022, 7.5.1.4. Unless there are transverse stiffening piers, *t*ef,i is equal to the wall thickness *t*i; |
| *l*mp,i | is the length of the pier that the wall may contain. |

1. If, in a given direction of a storey in which inelastic deformations are expected, the wall axial load ratio *ν*G,k,i is greater than 0,2 in more than 25 % of the total area of walls, the behaviour factor component *q*D should be reduced by 20 %, but not taken smaller than 1,0. It should also be checked that the axial loads carried by the primary walls with νG,k,i >0,2 may be redistributed to other walls according to 14.5.3.
2. If the axial load at the top of a wall acts at with an eccentricity to the central plane of the wall, the deformation capacity of the wall is reduced and *q*D should be reduced.

NOTE Annex J gives information for the reduction of *q*D.

1. The values given in table 14.1. should be applied unless other values are supported by test results obtained in accordance with EN 1996-1-1.

NOTE prCEN/TS 1998-1-101 gives a loading protocol, interpretation and acceptance criteria for such tests.

### Behaviour factors for out-of-plane analysis

1. For unreinforced and confined masonry walls, the behaviour factor *q*oop for out-of-plane bending should be taken equal to 1,25.
2. For reinforced masonry walls, the behaviour factor *q*oop for out-of-plane bending should be taken equal to 1,5.

## Structural analysis

### Modelling rules for linear analyses

#### Walls

1. The stiffness of the walls should be evaluated taking into account their flexural, shear and axial deformations.
2. Piers of all types of masonry that are expected to crack under seismic loading should be assigned a reduced stiffness values which may be taken equal to the secant stiffness at 70 % of the pier’s ultimate strength.
3. For all types of masonry, the elastic flexural and shear stiffness of cracked piers may be taken equal to respectively 50 % and 35 % of the flexural and shear stiffness of the uncracked pier.

#### Spandrels and ring beams

1. Masonry spandrels may be considered as coupling beams between two walls if they are regularly bonded to the adjacent members.
2. Reinforced concrete ring beams may be considered as coupling beams between two walls.
3. Ring beams and masonry spandrels may be modelled as beams connected to the walls by means of stiff beams or rigid links over the distance from the edge of the wall to the axis of the wall modelled as a column in order to form an equivalent frame model.
4. In equivalent frame models, unless more detailed information is available, it may be assumed that the joints are rigid and have dimensions similar to those of the adjacent walls and spandrels.
5. The flexural stiffness of cracked ring beams may be taken as 50 % of that of the uncracked beams. The shear deformation of ring beams may be neglected.
6. The elastic modulus and shear modulus of a horizontal masonry member such as a spandrel may be calculated as given in EN 1996-1-1:2022, 5.8.2 and 5.8.3, taking into account *f*k of the units in the relevant horizontal direction.
7. Unless a more accurate analysis of the cracked members is performed, the elastic flexural and shear stiffness properties of the cracked spandrel may be taken as given in a) or b):
8. Unreinforced masonry spandrel: 25 % of the stiffness of uncracked member;
9. Reinforced and confined masonry spandrel: 50 % of the stiffness of uncracked member.
10. A composite spandrel consisting of a masonry spandrel and a reinforced concrete beam or slab strip may be modelled as two parallel entities, neglecting the interaction between them.
11. In the case of force-based verification, the flexural stiffness of the horizontal members may be set to zero if this does not result in a design base shear smaller than the one found considering those stiffnesses.

#### Diaphragms and slabs

1. The model should represent the actual stiffness of the diaphragm, unless the diaphragm is rigid according to (3) or (4). If the diaphragm is rigid, the stiffness may be set to a value greater than the actual stiffness.
2. In the absence of a more detailed analysis, diaphragms may be considered as uncracked.
3. A diaphragm may be considered rigid in tension, compression and shear if the period *T*i of each mode *i* that contributes significantly to the global response does not change by more than max(0,1*T*i, 0,05s) between a model of the system with the actual diaphragm stiffness and a model with rigid diaphragms.
4. A reinforced concrete slab may be considered as rigid in tension, compression and shear, if it complies with 10.12.1(1) or 10.12.1(2).
5. Flexible diaphragms may be modelled using either shell members or strut-and-tie models.
6. The global structural model may consider the coupling effect provided by the slab.

NOTE This applies if the floor slab is supported by piers or walls in such a way that horizontal loading of the structure induces bending of the slab. See also note to (7)c).

1. In buildings where several walls lie in the same vertical plane and are continuous over the height, the coupling effect provided by the slabs may be modelled by means of horizontal beams between the walls in the vertical plane of the walls. The horizontal beams may be modelled as rigid from the centreline to the end of each wall and deformable over the free span between two walls. Their effective width may be taken equal to the values in a), b) or c):
2. In buildings with flat slabs, for all types of masonry, 0,1*l*0 at the edge of the slab and 0,2*l*0 in the interior of the slab, where *l*0 is the distance between the centrelines of the two walls.
3. In unreinforced masonry buildings with flat slabs, respectively 25 % of the values in a) for the top floor and 50 % of the values in a) for the floor below the top floor, but not smaller than the wall thickness.
4. In buildings with slabs casted monolithically with beams, the effective width of beam may be determined according to prEN 1992-1-1:2021, 7.2.3,and assumed constant along the beam length.

NOTE In unreinforced masonry buildings subjected to horizontal loads, slabs can experience uplift from the masonry walls. The wall length over which the uplift takes place depends on the axial force in the walls above and below the slab.

1. In buildings where walls do not lie in few vertical planes, the out-of-plane stiffness of slabs may be modelled with shell elements.
2. The uplift of the slab from unreinforced masonry walls may be modelled by reducing the wall length over which the slab is connected to the wall or, if the walls are modelled as beams, by reducing the length of the rigid links.

#### Secondary seismic members

1. Walls should be considered as secondary if they do not satisfy the limits of *l*mp/*h*op of Table 14.2.
2. Spandrels may be classified as primary or secondary spandrels.
3. If secondary walls and spandrels are included in the model, they should be modelled such that they do not transfer shear and bending forces due to seismic action.

### Modelling rules for non-linear analyses

#### General

1. 14.5.2 should be applied to the global non-linear analysis of unreinforced, confined and reinforced masonry structures, in addition to 14.5.1.
2. For non-linear response-history analyses, appropriate hysteretic laws should be assigned to individual elements.
3. For non-linear static analyses, simplified constitutive laws may be applied.
4. The condition in 5.3.5.1(3) may be considered as satisfied if the average aspect ratio of structural walls is smaller than 1,0 in two storey masonry buildings or smaller than 0,5 in three storeys buildings.

#### Walls

1. A bilinear force-displacement relationship with zero post-yield stiffness may be used in which the elastic stiffness should correspond to cracked section properties and the plateau should extend up to **NC, where *θ*NC is the interstorey drift in the near collapse NC situation. Beyond **NC, the horizontal strength of the walls should drop to a residual strength.
2. Unless more detailed information is available, it should be assumed that the residual horizontal strength is equal to zero.
3. It may be assumed that walls maintain their axial load bearing capacity beyond **NC.

NOTE The deformation capacity of walls is given in Table 14.4.

1. The force-displacement models for walls should take into account the interaction between the shear resistance and axial forces, and between the moment resistance and axial force.

#### Spandrels and ring beams

1. The shear force–chord rotation relationship of an unreinforced masonry spandrel that is not supported by a reinforced concrete beam or a steel beam or a slab strip may be assumed elastic up to the peak shear strength *V*R,peak calculated by Formula (14.2).

(14.2)

where

|  |  |
| --- | --- |
| *V*R, lim | is given by Formula (14.3); |

(14.3)

where

|  |  |
| --- | --- |
| *f*vk0 | is the characteristic initial shear strength of masonry, under zero compressive stress; |
| *d*sp | is the spandrel depth; |
| *t*sp | is the spandrel thickness; |
| *h*sp | is the spandrel length; |
| *b*sp | is a correction coefficient related to the shear stress distribution in the middle section of the panel and to the aspect ratio *h*sp/*d*sp of the panel, however not smaller than 1,0 and not greater than 1,5; |
| *f*vlt | *f*vlt is a limit to the value of *f*vk as in EN 1996-1-1:2022, 5.7.2.1(1). |

1. More refined modelling than in (1) may be used if the axial force in the spandrel can be reliably estimated.
2. If the model used for an unreinforced masonry spandrel does not capture the elongation due to non-linear flexural deformations, its shear force-chord relationship may be assumed as elastic-perfectly plastic.
3. With a model of unreinforced masonry spandrel model able to represent the elongation due to non-linear flexural deformations, the shear force-chord rotation relationship of the spandrel may be modelled as trilinear with the three branches a), b) and c):
4. a linear elastic branch up to the chord rotation *θ*y  *V*R,peak/*K*eff where *V*R,peak is given by Formula (14.2);
5. a linear decreasing branch down to a residual strength;
6. a horizontal branch representing the residual strength; the latter may be taken as a function of the axial force in the spandrel only, but not greater than *V*R,peak.
7. When using the model in (4), the shear action effect may be calculated assuming flexural rocking in the two end sections of the spandrel for a zero tensile strength
8. The flexural and shear capacity of reinforced and confined masonry spandrels and reinforced concrete ring beams may be calculated assuming zero axial force. The shear force-chord relationship may be assumed as elastic-perfectly plastic.
9. Spandrels and ring beams may be modelled as elastic-perfectly plastic in compression with an unlimited deformation capacity. In tension, unreinforced masonry spandrels may be modelled as elastic-perfectly brittle; reinforced and confined masonry spandrels and ring beams may be modelled as elastic-perfectly plastic with an unlimited deformation capacity.

NOTE The deformation capacity of spandrels is given in 14.9(5). The deformation capacity of reinforced concrete beams is given in prEN 1998-1-1:2022, 7.2.

#### Diaphragms and slabs

1. If the diaphragm stiffness does not satisfy the condition of rigidity in 14.5.1.3(3), its actual stiffness should be reflected in the model.
2. Slabs and diaphragms may be modelled with a linear behaviour.

### Force based analysis

1. The control of second order effects according to 6.2.4 may be neglected.
2. The shear force and bending moments in walls obtained by linear analysis may be redistributed among the walls if (3) and (4), or (5) and (8), and (9) are satisfied.
3. Whenever redistribution is carried out, global equilibrium should be satisfied.

NOTE This is realized if the total storey shear and the position of the resultant force are the same.

1. If *q*R is taken greater than 1,0, the reduction or increase *V* of the absolute value of the shear force in wall *i* in any type of masonry resulting from a redistribution of forces should not be greater than *V*max,i given by Formula (14.4).

(14.4)

where

|  |  |
| --- | --- |
| *V*i | is the shear force in wall *i*; |
| *V*storey | is the storey shear force in the direction parallel to the plane of the wall; |
| *V*max, | is the maximum value of the reduction or increase *V* of the absolute value of the shear force in a wall resulting from a redistribution of forces. |

1. If the behaviour factor component *q*R is taken equal to 1,0 and redistribution effects are not taken into account, the redistribution of the shear forces in walls in any type of masonry building with rigid diaphragms may be carried out according to (6) and (7).
2. If *q*R = 1,0, the maximum reduction or increase *ΔV*max,i of the absolute value of the shear force in wall *i* is defined in (7) if the storey in the direction of the shear force that is redistributed is torsionally restrained or in (8) if it is unrestrained. The storey *j* may be considered as torsionally restrained in the horizontal direction *x* (or *y*, respectively), if Formula (14.5) is satisfied.

(14.5)

where *r*y,j, *r*x,j and *l*s,j are defined in 4.4.3.

1. If storey *j* is torsionally restrained in the direction *x* (or *y*), redistribution may be carried out so that the increase of shear in any wall along that direction is not greater than that given by Formula (14.6).

(14.6)

where

|  |  |
| --- | --- |
| *V*i | is the shear force along direction *x* (or *y*)obtained from linear analysis in the pier *i* where the shear force is increased; |
| **i | is the largest chord rotation in pier *i* calculated from the linear analysis; |
| *h*i | is the free height of the wall over which the shear force is incremented; |
|  | is the interstorey displacement along direction *x* (or *y*) at the pier where the shear force is incremented, corresponding to the attainment of the ultimate rotation in any wall of the storey along direction *x* (or *y*); *θ*ult,i may be taken as given by Formula (14.7); |

*θ*ult,i = *θ*i *h*i *θ*SD / *θ*\* ≤ *θ*SD *h*min (14.7)

where

|  |  |
| --- | --- |
| **SD | is the chord rotation capacity of a wall failing in shear given by Formula (14.18); |
| *h*min | is the free height of the shortest wall (i.e. with the lowest *h*i) in storey *j*, along the considered direction; |
| ** | is the largest chord rotation in the shortest wall (i.e. with the lowest *h*i) in the storey *j*, along the considered direction, calculated from the linear analysis. |

 condition is imposed to the reduction of shear in walls resulting from the application of (7) or (8).

1. If storey *j* is not torsionally restrained in the direction *x* (or *y*), the increase of shear due to redistribution in any wall along that direction should not be greater than Δ*V*max,i given by Formula (14.8). There is no limitation to the reduction of shear in a wall.

(14.8)

where

|  |  |
| --- | --- |
| *V*i | is the shear force obtained by the linear analysis in the wall where the shear force is increased or decreased; |
| *V*storey | is the corresponding total storey shear force in the same direction. |

1. Shear force redistribution should not be carried out when the *q*R factor has been obtained by a non-linear static analysis.
2. The effects of shear force redistribution for the diaphragms, ring beams and drag ties should be taken into account.
3. If diaphragms are flexible, shear forces may be redistributed among walls in the same vertical plane according to (6), (7) and (8) provided that walls are connected at each floor level by beams designed to resist and transmit the effects of shear force redistribution.

NOTE In such case, the criteria associated to Formula (14.4), (14.6), (14.7) and (14.8) are applicable assuming that the system of walls in the same vertical plane is a stand-alone planar structure (e.g. *V*storey is the sum of the shears in the walls at the same storey, belonging to the same vertical plane).

1. The redistribution of moments in beams of reinforced masonry buildings should satisfy EN 1996‑1‑1:2022, 7.5.3.5.

### Linear structural analysis for determining the out-of-plane bending moment demand on walls

1. If *E*Ed,E is calculated with a first order analysis, second order effects should be controlled according to 6.2.4.
2. The seismic action effects *E*Ed,E may be calculated by means of a floor acceleration response spectrum according to Annex C, considering the first vibration period.
3. The height *z*j may be assumed as the height at the centre of the wall. The damping ratio *ξ*a may be taken equal to 5 %. Factor *q*D in Formula (7.3), in Formula (C.1) and in Formula (C.3) may be taken as given in a) or b):
4. for the SD verification of systems designed for DC1: 1,0;
5. for the SD verification of systems designed for DC2: *q*oop according to 14.4.2.
6. The boundary conditions of out-of-plane loaded walls should be determined according to EN 1996‑1-1:2022, 7.5.1. Restraint at a vertical edge may be considered or, as a simplification, neglected. Walls may be modelled as 1D elements spanning between two floors or standing as a cantilever on one floor.
7. For walls modelled as 1D elements, the first period *T*0 of the out-of-plane mechanism may be calculated by Formula (14.9):

(14.9)

where

|  |  |
| --- | --- |
| ** | is a coefficient to account for the boundary conditions which may be taken equal to:  1,8 for **n= 2,0,  0,6 for **n = 1,0,  0,45 for **n = 0,75; |
| **n | is a factor related to the effective height defined in EN 1996-1-1:2022, 7.5.1.3(10); |
| *h*wall | is the clear height of the wall defined in EN 1996-1-1:2022, 7.5.1.3(9); |
| *m*wall | is the mass of the wall, mass of plaster and non-loadbearing leave included, if any; |
| *E*sec | is the short-term secant modulus of elasticity of masonry; |
| *I*oop | is the moment of inertia of the wall with regard to out-of-plane bending calculated based on the effective stiffness *t*ef. |

1. A pier may be assumed as rigid in its out-of-plane direction if *T*0/*T*1 < 0,2 where *T*1 is the period of the first mode in the considered horizontal direction.
2. The resultant horizontal force *F*oop acting on the out-of-plane loaded wall may be calculated using Formula (14.10). If the wall is rigid in its out-of-plane direction, Formula (14.11) may be used.

(14.10)

(14.11)

where

|  |  |
| --- | --- |
| *S*an,ij | is given in Annex C, Formulas (C1) and (C2), or by (7.2); |
| *PFA*ii | is given in Annex C, Formula (C3); |
| *q*oop | is the behaviour factor for out-of-plane bending defined in 14.4.2. |

1. The design bending moment *M*oop,Ed due to out-of-plane action in a wall may be calculated with the boundary conditions as in (4) and *F*oop as in (7). *F*oop may be distributed over the out-of-plane loaded wall in proportion to the distribution of mass in the wall.

## Verification of limit states

### General requirements

1. In unreinforced and confined masonry buildings, the in-plane and the out-of-plane resistances of members should satisfy 6.2.3.
2. In reinforced masonry buildings, the in-plane and the out-of-plane resistances of members should satisfy 6.2.3 for the action effects calculated as given in 14.6.2 and 14.6.3 each being considered to act alone, unless (3) is applied.
3. The verification of out-of-plane resistance of reinforced masonry walls spanning between rigid floor diaphragms or between rigid floor and roof diaphragms may be neglected if the free storey height is smaller than 4 m.

### Verification for in-plane actions

#### General

1. The SD limit state should be verified using the force-based approach in prEN 1998-1-1:2022, 6.4, or the displacement-based approach in prEN 1998-1-1:2022, 6.5.
2. The resistance in terms of generalized forces of each masonry member should be calculated in accordance with EN 1996-1-1 taking into consideration the factor *γ*M in 14.3(5).
3. Diaphragms and bracings in horizontal planes and sub-horizontal planes (roofs) should satisfy 6.2.8.
4. Force-based verifications of in-plane loaded primary members should be carried out in terms of axial force, shear force and bending moment.
5. Displacement-based verification should be carried out in terms of displacements at the control nodes.
6. The shear and flexural resistance of in-plane loaded walls should be calculated taking into account the axial force of the seismic design situation.
7. If the displacement-based approach is used, the variation of axial forces with increased displacement should be taken into account in the calculation of the shear and flexural resistances.
8. The shear and flexural resistance of reinforced and confined masonry spandrels and of reinforced concrete ring beams may be calculated assuming a zero-axial force.
9. In force-based and displacement-based approach, the verification, the verification of flexural resistance of unreinforced masonry spandrels may be neglected when ties or ring beams or stiff diaphragms are present in the proximity of the spandrel.

#### Verification of the SD limit state by the force-based approach

1. The seismic action effects Ed should be calculated according to prEN 1998-1-1:2022, 6.4.
2. The resistance of horizontal masonry members should be greater than the design action effects; if not, their design stiffness should be reduced until at least one wall is first to reach its design resistance.
3. prEN 1998-1-1:2022, Formula (6.31), should be satisfied for all primary members.

#### Verification of the SD limit state through the displacement-based approach

1. The target displacement *d*t\* may be calculated according to prEN 1998-1-1:2022, 6.5.4, without considering the upper limit of 3*d*et\* for *T*\* < *T*C.
2. The resistance *d*SD at the SD limit state in terms of displacement at the control node of the masonry building should be taken as given in Formula (14.12).

(14.12)

where

|  |  |
| --- | --- |
| **RdSD | is the partial factor on resistance in terms of generalized displacements at SD; |
| *d*SD, | is the displacement of the control node at which a primary wall first reaches its resistance in terms of generalized in-plane deformations, that is the member drift ratio. The resistance in terms of member drift ratio is given in 14.9. *d*SD,corresponds to *d*m in prEN 1998-1-1:2022, Figure 6.1. |

NOTE 1 The value of **RdSD is 1,75 unless the National Annex gives different values of **RdSD for use in a country. The National Annex can specify different values of **RdSD for unreinforced masonry, confined masonry and reinforced masonry buildings.

NOTE 2 This definition replaces the one in prEN 1998-1-1:2022, 6.7.2(2).

1. The structure should satisfy Formula (14.13).

(14.13)

where

|  |  |
| --- | --- |
|  | is the equivalent yield displacement of the equivalent single degree of freedom (SDOF) model; dy\* should be calculated from *d*m\* according to prEN 1998-1-1, Formula (6.27). |
| *d*SD\* | should be calculated from *d*SD according to prEN 1998-1-1:2022, Formula (6.25). |

#### Verification of the NC limit state

1. In case the NC limit state is used, the seismic action effects *E*d may be determined according to prEN 1998-1-1:2022, 6.5.
2. 14.6.1.3(1) should be applied.
3. The resistance *d*NC of the masonry building should be taken as given in Formula (14.14).

(14.14)

where

|  |  |
| --- | --- |
| **RdNC | is the partial factor on resistance in terms of generalized displacements at NC; |
| *d*y | is the displacement of the control node for which the first primary wall reaches its resistance in terms of generalized in-plane forces; |
| *d*NC,θ | is the displacement of the control node for which the total base shear of the system has dropped below 80 % of the peak resistance or the displacement at which the first primary member reaches an element drift ratio of 1,5 times its resistance at the NC limit state, whichever occurs first. The resistance in terms of element drift ratio is given in 14.9. |

NOTE 1 The value of *γ*Rd,NC is 1,75 unless the National Annex gives different values of *γ*Rd,NC for use in a country. The National Annex can specify different values of *γ*Rd,NC for unreinforced masonry, confined masonry and reinforced masonry buildings.

NOTE 2 Formula (14.14) replaces prEN 1998-1-1:2022, 6.7.3(2), Formula (6.32).

1. The structure should satisfy Formula (14.15).

≤ (1+3*T*C/*T*\*)*d*y\* (14.15)

where *d*NC\* should be calculated from *d*NC according to prEN 1998-1-1:2022, Formula (6.25).

### Verification for out-of-plane actions at SD limit state

1. For out-of-plane action effects, primary and secondary members such as walls and piers, including parapets and gables walls, should satisfy in all critical sections Formula (14.16), unless they comply with 14.7.1(3).

(14.16)

where

|  |  |
| --- | --- |
| *M*oop,Ed | is the design bending moment due to out of plane action; |
| *M*oop,Rd | is the design bending resistance against out of plane action effect. |

1. The design resistance, *M*oop,Rd, may be calculated according to EN 1996-1-1:2022, 8.5.4, assuming a rectangular stress distribution on the cross section of the wall and taking into consideration the factor *γ*M in 14.3(5).

NOTE Second-order effects are accounted for on the resistance side by referring to the provisions given in EN 1996-1-1:2022, Annex F, for unreinforced masonry, EN 1996-1-1:2022, 8.7.4, for reinforced masonry and EN 1996-1-1:2022, 8.10.4, for confined masonry.

1. *M*oop,Rd may be calculated assuming an axial load on the wall *N*Ed,G of the non-seismic actions in the seismic design situation.
2. The action effect *M*oop,d at the SD limit state may be calculated according to 14.5.4.

NOTE As alternative method to check the out-of-plane stability at the SD limit state is the displacement-based approach in prEN 1998-3:2023, 11.3.3.

1. Ancillary walls and partitions should satisfy 7.6.
2. The design lateral (out-of-plane) resistance of ancillary walls and partitions may be calculated as given in EN 1996-1-1:2022, 8.4.2 and 8.4.3.
3. The resistance of wall ties of non-loadbearing leaves of cavity walls and of veneer walls should be greater than the seismic action effects calculated as given in 7.2.1(3) in which the performance factor should be taken as given in 7.2.2(2) and the behaviour factor should be taken equal to *q*oop as given in 14.4.2, in the case of cavity walls, and as given in Annex C, C4, for veneers.

## Design rules for members

### Limitations of piers and walls dimensions in DC1 and DC2

1. For piers in primary walls, the ratio of their length *l*mp to the greater clear height *h*op of the adjacent openings should not be smaller than (*l*mp/*h*op)min given in Table 14.2.

NOTE The values of (*l*mp/*h*op)min in Table 14.2 are absolute limits.

1. The out-of-plane failures of primary and secondary walls should be prevented.

NOTE The conditions for a member to be secondary are given in 4.4.2 and 14.5.1.4.

1. (2) may be considered satisfied if the ratio *h*ef/*t*ef of the effective wall height to its effective thickness does not exceed (*h*ef/*t*ef)max given in Table 14.2. for the relevant values of *S*δ.

NOTE The effective height *h*ef and the effective thickness *t*ef are defined in EN 1996-1-1:2022, 7.5.1.3 and 7.5.1.4.

1. If the limitations of *S*δ and *h*ef/*t*ef in (3) are not satisfied, the out-of-plane capacity of primary and secondary walls should satisfy 14.6.3.
2. The resistance of wall ties of non-loadbearing leaves of cavity walls and of veneer walls should be greater than the seismic action effects calculated as given in 7.2.1(3) in which the performance factor should be taken as given in 7.2.2(2) and the behaviour factor should be taken equal to *q*oop as given in 14.4.2 in the case of cavity walls and as given in Annex C, C4, for veneers.

Table 14.2 — Geometric reference limit values for walls and piers\*

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Masonry type** | ***S* (m/s2)** | **Wall**  **(*h*ef/*t*ef)max**  **Config. 1** | **Wall**  **(*h*ef/*t*ef)max**  **Config. 2** | **Pier**  **(*l*mp/*h*op)min** |
| Unreinforced  Group 1 and 1S | 5,0 < *S* ≤ 6,50 | 7 | 5 | 0,50 |
| 3,25 < *S* < 5,0 | 12 | 7 | 0,45 |
| *S* < 3,25 | 20 | 10 | 0,35 |
| Unreinforced  Other Groups | 5,0 < *S* ≤ 6,50 | 10 | 5 | 0,40 |
| 3,25 < *S* < 5,0 | 15 | 7 | 0,40 |
| *S* < 3,25 | 20 | 10 | 0,35 |
| Confined masonry | *S* ≤ 6,50 | 20 | 20 | 0,30 |
| Reinforced masonry | *S* ≤ 6,50 | 20 | 20 | No limit |
| \* The table differentiates between two configurations:  a) Configuration 1 comprises walls spanning between two floors.  b) Configuration 2 comprises parapets, cantilever piers or walls and chimneys. | | | | |

### Design rules for unreinforced masonry in DC2

1. The action effects in the ring ties or ring beams should be determined as for other structural members and satisfy Clauses 10, 11, 12, 13 or 15 as appropriate for the material considered. The axial resistance of ring ties and ring beams should not be smaller than 150 kN.
2. Reinforced concrete ring beams should satisfy a) to d):
3. their depth *h*RB should not be smaller than 150 mm or the slab thickness, whichever is greater;
4. their width *t*RB should be such that the beam extends over the entire wall thickness *t*w minus a set-back; the set-back should not be greater than max(60 mm, 0,3 *t*w) if *t*w ≤ 300 mm and 0,3*t*w if *t*w > 300 mm;
5. the area of longitudinal reinforcement should not be smaller than *A*sl  450 mm2;
6. the shear reinforcement should be closed stirrups with an area not smaller than *A*sw = 225 mm2/m.

### Design rules for confined masonry in DC2

1. The horizontal and vertical confining members should be bonded together, anchored to the members of the main structural system and to the foundation. Longitudinal reinforcement from vertical confining members should be anchored at the bottom of the foundation.
2. The concrete of the confining members should be cast after the masonry is constructed. Effective bond should be ensured either by toothing at the wall-to-vertical confining member interface or by providing horizontal dowel bars at regular spacing over the wall height. Horizontal dowel reinforcement should be placed either in the form of a single reinforcing bar of minimum 6 mm diametre and 400 mm length at maximum 600 mm vertical spacing, or in the form of ladder-shaped wire reinforcement at maximum 400 mm vertical spacing.
3. The cross-sectional dimensions of both horizontal and vertical confining members should not be smaller than 150 mm.
4. In double-leaf walls in which both leaves are structural walls, each confining member should confine the two leaves simultaneously.
5. Vertical confining members should be placed as given in a) to d):
6. at the free edges of each structural wall;
7. at both sides of any wall opening of area greater than 1,5 m2;
8. with a spacing not exceeding 5 m;
9. at all structural wall intersections.
10. Vertical confining members complying with (4) but which are distant of less than 1 m from the confining member present at a wall intersection may be omitted.
11. Vertical confining members should be placed at wall intersections.
12. The horizontal confining members should be placed in the plane of the wall at every floor level with a vertical spacing not greater than 4 m.
13. The longitudinal reinforcement ratio in confining members should not be smaller than 0,8 % or four diameter 10 mm whichever the greater.
14. The diameter of stirrups should not be smaller than 6 mm and their spacing not greater than 150 mm. The spacing of stirrups in vertical confining members should not be greater than 100 mm within 0,6 m of the member ends at each storey level.
15. If *S*δ > 6,25 m/s2, longitudinal reinforcement should be Class B or C; stirrups may be of Class B. If *S*δ ≤ 6,25 m/s2, longitudinal reinforcement may be Class B and stirrups Class A.
16. Lap splices should not be shorter than 60 times the diameter of the bars being spliced.

### Design rules for reinforced masonry in DC2

1. Horizontal reinforcement should be placed either in the bed joints or in grooves in the units, with a vertical spacing not exceeding 600 mm.
2. Masonry units with recesses should accommodate the reinforcement needed in lintels and parapets. Prefabricated lintels with the needed reinforcement may also be used.
3. Parapets and lintels should be regularly bonded to the masonry of the adjoining walls and connected by means of continuous longitudinal reinforcement.
4. The diametre of horizontal reinforcing bars should not be less than 5 mm; they should be anchored by bends or hooks or loops around the vertical bars at the edges of the wall.
5. The percentage of horizontal reinforcement in the wall, normalized with respect to the gross area of the section, should not be smaller than 0,05 %.
6. The amount of horizontal reinforcement should not lead to a failure of compression struts prior to the yielding of the horizontal steel reinforcement.

NOTE Compression strut failure can be verified according to EN 1996-1-1:2022, 8.8.2(4).

1. The amount of horizontal reinforcement should not lead to a failure of compression struts prior to the yielding of the horizontal steel reinforcement.
2. The total amount of vertical reinforcement in a wall should not be smaller than 0,05 % of the gross area of its horizontal section.
3. Vertical reinforcement should be placed in pockets, cavities or holes in the units.
4. Vertical reinforcement with a cross-sectional area not smaller than 200 mm2 should be placed as given in a) to d):
5. at both free ends of a wall;
6. at each wall intersection;
7. within the wall, not exceeding a horizontal spacing of 5 m between such reinforcing bars;
8. at both sides of any wall opening of area greater than 1,5 m2.
9. The amount of horizontal reinforcement should not lead to a failure of compression struts prior to the yielding of the horizontal steel reinforcement.
10. If *S*δ > 3m/s2, longitudinal reinforcement should be Class C while shear reinforcement may be of Class B. If *S*δ ≤ 3m/s2, longitudinal reinforcement may be Class B and shear reinforcement Class A.
11. Lap splices should not be shorter than 60 times the diametre of the bars being spliced.

## Rules for simple masonry buildings

### General

1. Buildings conforming to 14.8.2 may be classified as “simple masonry buildings”.
2. For simple masonry buildings, the verifications in 14.6 may be replaced by those in 14.8.2(6)

### Design rules

1. The plan configuration of buildings should satisfy a) to c):
2. the plan should be approximately rectangular;
3. the ratio between the length of the small side and the length of the long side should not be smaller than 0,25;
4. the area of projections of recesses from the rectangular shape should be not greater than 15 % of the total floor area above the level considered.
5. Buildings should be regular in elevation and comply with 4.4.4.2(1) a) and b).
6. The roof and slab diaphragms should comply with 14.2.2.
7. The shear walls of simple masonry buildings should satisfy conditions a) to k):
8. The primary walls should be placed almost symmetrically in plan in two orthogonal directions.
9. The primary walls should be placed far from the centre of mass so that the building is not classified as torsionally flexible.
10. The primary walls should be continuous from the foundation to the top of the building.
11. The primary and secondary walls should satisfy the limitations to *h*ef/*t*ef in Table 14.2; primary walls should also satisfy the limitations to *l*mp/*h*op in Table 14.2.
12. For *S*δ > 3 m/s2, primary walls in unreinforced masonry should satisfy 14.7.2, primary walls in confined masonry should satisfy 14.7.3 and primary walls in reinforced masonry should satisfy 14.7.4.
13. At least 75 % of the vertical loads should be supported by primary walls and should be distributed almost equally to primary walls in both principal directions.
14. There should be at least two primary shear walls in each principal direction which should have a length *l*mp not less than that corresponding to twice the minimum value of *l*mp/*h*op in Table 14.2. In each horizontal direction, the spacing of these two primary walls should not be smaller than 60 % of the building dimension orthogonal to the wall plane.
15. In unreinforced masonry buildings, if *S*δ ≤ 3 m/s2, the spacing of two primary walls required in g) should not be smaller than 75 % of the building dimension orthogonal to the wall plane in one principal direction.
16. In unreinforced masonry buildings, if *S*δ > 3 m/s2, the spacing of the two primary walls required in g) should not be smaller than 60 % of the building dimension orthogonal to the wall plane in each principal direction.
17. In reinforced and confined masonry buildings, the average primary wall length should not be smaller than 1,5 m.
18. In reinforced and confined masonry buildings of 3 or 4 storeys and for *S*δ > 3 m/s2, the minimum average primary wall length should not be smaller than 2,5 m.

NOTE The average wall length is defined as the sum of the lengths of all walls divided by the number of walls.

1. In unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7 m. The connection may be achieved through interlocking of units or details that have similar effects.
2. The ratio *p*Aof the total cross-sectional area *A*mp,tot of walls in each direction to the total floor area per storey should be greater than the ratio *p*A,min satisfying (7). Only walls satisfying (4)c) should be taken into account in calculating *A*mp,tot .
3. The ratio *p*A,min should be established considering the influence of parameters a) to h):
4. number *N*s of storeys;

NOTE The number *N*s of storeys includes the ground storey and does not include the underground storeys and the roof space above full storeys.

1. seismic action *S*δ
2. type of masonry: unreinforced, reinforced or confined;
3. characteristic compressive strength *f*k of the masonry units;
4. use of masonry units of different *f*k in different walls
5. axial load ratio **Gk,i in each wall *i;*

NOTE *ν*Gk,i is defined in 14.4.1.2(4).

1. average axial load ratio *ν*Gk,i
2. stiffness of the storey beams and slab and of the foundation influencing the in-plane clamping moments.

NOTE Given the great variability of masonry types, masonry units, architectural styles and constructional practice over Europe, the ratio *p*A,min to be used in a country are given in its National Annex.

## Ultimate deformations

### General

1. The drift ratio or chord rotation **e of a masonry member should be calculated as given by Formula (14.18).

(14.18)

where**i (resp. **j) is the angle between the member axis at end section *i* (respectively *j*) and the chord line that joins the centroids of the two end sections.

### Unreinforced masonry members

1. **u2,SD of walls failing in shear should be taken as a) or b):
2. the value given in Table 14.4, for axial load ratios **Ek≤ 0,3;
3. the value given in Table 14.4 divided by 3 and multiplied by (1- vE,k )/0.7 or the deformation capacity at the elastic limit for axial load ratios **E,k > 0,3, whichever is greater.

where

|  |  |
| --- | --- |
| **E,k | is the wall axial load ratio calculated as **E,k  *N*Ed/(*A*p *f*k); |
| *N*Ed | is the axial force in the seismic design situation; |
| *A*p | is the wall section area as given in 14.4.1.2(4); |
| *f*k | is the characteristic compressive strength of the masonry wall. |

1. If *ν*G,k≤ 0,2, (1)a) may be applied.
2. The drift capacity of piers failing in flexure may be taken equal to twice the capacity of a pier failing in shear.
3. The deformation capacity in terms of member drift ratio of the pier at 20 % drop in strength may be taken as 1,5 times *θ*SD.
4. The deformation capacity of unreinforced masonry spandrels in terms of chord rotation may be taken equal to four times the drift capacity of unreinforced masonry piers.
5. The deformation capacity of a composite spandrel in terms of chord rotation may be calculated from the rotational capacity of the beam.

### Reinforced masonry members

1. The drift capacity of reinforced masonry members may be assumed as 1,5 times those of unreinforced masonry, unless other values are demonstrated by tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol, interpretation and acceptance criteria for such tests.

### Confined masonry members

1. Unless drift capacity estimates are available from tests on confined masonry members, the drift capacity of confined masonry members may be assumed as 1,25 times those of unreinforced masonry ones.

**Table 14.4**— **Unreinforced masonry: deformation capacities for piers or walls failing in shear. Applicable for axial load ratios **Ek ≤ 0,3**

|  |  |
| --- | --- |
| **Masonry type** | **Drift limits at SD [%]** |
| Clay units, Gr 2 | 0,30 |
| Clay units, Gr 1s | 0,68 |
| Calcium silicate (hollow and solid) | 0,28 |
| AAC Gr 1 and 1s | 0,38 |
| Light-weight concrete units, Gr 1 | 0,30 |

# Specific rules for aluminium buildings

## General

1. Clause 15 should be applied to the design and the verification of aluminium buildings in seismic regions.
2. Clause 15 should be applied as a complement to EN 1999-1-1.

## Basis of Design

### Design concepts

1. Earthquake resistant aluminium buildings should be designed in one of the Ductility Classes in a) or b) (see Table 15.1):
2. DC1: Low-dissipative structural behaviour;
3. DC2: Dissipative structural behaviour.

Table 15.1 — Structural ductility classes and upper limit reference values   
of the behaviour factors

|  |  |
| --- | --- |
| **Structural ductility class** | **Range of the reference values of the behaviour factor *q*** |
| DC1 | ≤1,5 |
| DC2 | only limited by the values of Table 15.3 |

1. In DC1, the action effects may be calculated on the basis of an elastic global analysis without taking into account the non-linear material behaviour.
2. In DC1, the resistance of the members and of the connections should be evaluated in accordance with EN 1999-1-1 without any additional provisions.
3. In DC2, the capability of parts of the structure (dissipative zones) to resist earthquake actions through inelastic behaviour may be taken into account with upper limit values of *q*D and *q*R depending on the Ductility Class and the structural type (see 15.4). 15.3 to 15.6 and 15.8 to 15.14 should be applied.
4. In order for DC2 class to provide an increased ability of the structure to dissipate energy in plastic mechanisms, specific provisions in one or more of the following aspects should be satisfied: plastic deformation capacity of structural members and connections, ability to form global plastic mechanisms.

### Safety verifications

1. In the capacity design verifications specified in 15.8 to 15.14, the possibility that the actual strength of aluminium is greater than the nominal one should be taken into account by a factor *ω*rm, which is the ratio of the expected (i.e. average) value of the conventional elastic limit to its design value from EN 1999-1-1:2022, 5.2. In absence of experimental characterization of the material of the dissipative zones, *ω*rm may be assumed equal to 1.5.
2. The partial factors *γ*M,i given by EN 1999-1-1:2023, 4.4.3, should be used for both non-dissipative and dissipative members.

## Materials

1. Structural aluminium should conform to standards referred to in EN 1999-1-1:2023, Bibliography. For the dissipative members of DC2, Table 15.2 should be used. The alloys to be used for DC1 structures and DC2 non-dissipative members should conform to EN 1999-1-1:2023.
2. Alloys different from those specified in Table 15.2 may be used, provided that the ratio *f*u/*f*0 is not smaller than 1,10 and the elongation at failure is not smaller than 10 %, where *f*u is the ultimate tensile strength and *f*0 is the conventional elastic strength.
3. In bolted connections high strength pre-loaded galvanized steel bolts grade 8.8 or 10.9 or stainless-steel bolts should be used.
4. Material properties should comply with ISO 6892-1.

Table 15.2 — Aluminium alloys for dissipative parts of DC2 seismic resisting frames

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| **Structural member** | **Product form** | | **alloy** | **temper** | **thickness** |
| Sheet, strip and plate | - | | 5052 | H12 | H22/H32 | ≤ 40 |
| 5049 | O / H111 | ≤ 100 |
| 5083 | O/H111 | ≤ 80 |
| 5383 | O/H111 | ≤ 120 |
| H116/H321 | ≤ 80 |
| 5454 | O/H111 | ≤ 80 |
| 5754 | O/H111 | ≤ 100 |
| 6061 | T4 / T451 | ≤ 12,5 |
| 6082 | T4 / T451 | ≤ 12,5 |
| Extruded profiles, extruded tube, extruded rod/bar and drawn tube | ET,EP,ER/B | | 5083 | O/H111  F/H112 | ≤ 200 |
| ET,EP,ER/B | | 5454 | O/H111  F/H112 | ≤ 25 |
| ET,EP,ER/B | | 5754 | O/H111  F/H112 | ≤ 25 |
| DT | | 6060 | T6 | ≤ 20 |
| EP,ET,ER/B | | T64 | ≤ 15 |
| EP,ET,ER/B | | 6061 | T4 | ≤ 25 |
| DT | | T4 | ≤ 20 |
| EP,ET,ER/B | | 6082 | T4 | ≤ 25 |
| Legend:  EP- Extruded profiles  ER/B- Extruded rod and bar | | ET- Extruded tube  DT- Drawn tube | | | |

## Structural types, behaviour factors and limits of seismic action

### Structural types

1. Aluminium buildings may be assigned to one of the structural types in a) to d) according to the behaviour of their primary resisting structure under seismic actions:
2. Moment resisting frames, in which horizontal forces are mainly resisted by members acting in an essentially flexural manner (see Figure 11.1);
3. Frames with concentric bracings, in which horizontal forces are mainly resisted by members subjected to axial forces (see Figures 11.2 and 11.3);
4. Dual frames, in which at least 25 % of the lateral force resistance is provided by moment resisting frames that are combined with frames with concentric bracings, which provide the rest of the lateral force resistance (see Figure 11.6):
5. Inverted pendulum structures, in which 50 % or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building member or at the bases of columns (see Figure 11.8).
6. 11.4.1(2), 11.4.1(3), 11.10(1) and 11.10.2(2) a) and b) should be applied to aluminium buildings.
7. Structural systems which cannot be assigned to one of the structural types in (1) may be used; they should be designed for strength according to 15.7.3.

### Behaviour factors

1. The behaviour factor *q*, complying with prEN 1998-1-1:2022, 6.4.1, should account for the deformation capacity, the energy dissipation capacity of the structure and the relevant sources of overstrength. For regular structural systems, where softening effects of Heat Affected Zones (HAZ) are not detrimental for their performance, the behaviour factor components *q*D and *q*R may be taken with default values given in Table 15.3, provided that 15.8 to 15.14 are satisfied.

Table 15.3 — Default values of behaviour factors for aluminium building regular in elevation

|  |  |  |  |
| --- | --- | --- | --- |
|  | **Ductility Class** | | |
| **Structural Type** | **DC2** | | |
|  | ***q*D** | ***q*R** | ***q*** |
| **Moment resisting frames (MRFs)** |  |  |  |
| Single-storey MRFs | 1,5 | 1,1 | 2,5 |
| Multi-storey MRFs | 1,5 | 1,3 | 3,0 |
| **Frames with concentric bracings** | 1,5 | 1,0 | 2,3 |
| Diagonal bracings |
| V-bracings |
| X-bracings on either single or two-storey |
| **Dual frames (MRFs with concentric bracing)** | 1,7 | 1,2 | 3,0 |
| **Inverted pendulum** | 1,3 | 1,0 | 2,0 |

1. 11.4.2(2) should be applied.
2. prEN 1998-1-1:2022, 4.1(7), may be used within the limitations given in 15.6.3(1)b) and 15.7.3.

### Limits of seismic action for design to DC1 and DC2

1. For each structural type, design to a given Ductility Class should not be made above levels of seismic action index *S*δ (see prEN 1998-1-1:2022, 4.1(4)) given in Table 15.4.

Table 15.4 — Limits of seismic action index *S*δ for design of aluminium buildings to DC1 and DC2

|  |  |  |  |
| --- | --- | --- | --- |
| **Structural type** | **Limits of seismicity index *S*δ (m/s2)** | | |
|  | **DC1** | **DC2** |
| Moment resisting frames (MRFs) | 5,0 | no limits |
| Frames with concentric bracings | 5,0 | no limits |
| Dual frames (MRFs with concentric bracing) | 5,0 | no limits |
| Inverted pendulum | 2,5 | 5,0 |

## Structural analysis

1. Except where otherwise specified in Clause 15 (e.g. frames with tension-only concentric bracings, see 15.10.2(2)), the analysis of the structure may be made assuming that all primary members are active.

## Verification to Limit States

### General

1. The verification of structural members to limit states should comply with 6.1, 6.2 and 6.3.

### Resistance conditions at Significant Damage limit state

1. The resistance of structural members and connections at SD should be verified using EN 1999-1-1.
2. The second-order effects should be verified in accordance with 6.2.4.

### Limitation of interstorey drift at Significant Damage limit state

1. The interstorey drift at SD limit state should be limited as given in a) or b):
2. *d*r,SD ≤ 0,020 *h*s for moment frames;
3. for moment resisting frames with interacting infills, 7.4.2.1 should be applied;
4. *d*r,SD ≤ 0,015 *h*s for frames with concentric bracings, for dual frames, for inverted pendulum structures and for all other structural types;

where

|  |  |
| --- | --- |
| *d*r,SD | is as given in 6.2.4; |
| *h*s | is the interstorey height. |

## Design rules for low-dissipative (DC1) and non-dissipative structural behaviour common to all structural types

### General

1. 15.7.2 should be applied to the primary members of structures designed for low-dissipative behaviour.

### Design rules for low-dissipative structures

1. Structural members and connections should have adequate stiffness, resistance and stability, which should be verified in accordance with EN 1999-1-1:2023, Clauses 8 and 10.

### Design rules for non-dissipative structures

1. Non-dissipative structures should be designed to resist seismic actions in the elastic range and their behaviour factor *q* should be taken equal to 1,0.

NOTE Non-dissipative design is used for structures for which a ductile behaviour can hardly be conceived, e.g. aluminium domes, or cannot be supported by research reference. It is also used for specific structural systems for which non-dissipative design can be safe and more economical.

1. prEN 1998-1-1:2022, 4.1(7), may be used in the design of single-storey buildings; the limits of seismic action index *S*δ for their design to DC1 in 15.4.3 may be waived, provided that the condition in (4) and in 15.6(3)b) are satisfied.
2. prEN 1998-1-1:2022, 4.1(7), may be used without limitation of the seismic action index *S*δ in the design of structural systems which cannot be assigned to one of the types in 15.4.1(1) a) to d), provided that the conditions in (4) and in 15.6(3)b) are satisfied.
3. The connections should be designed for action effects calculated for the return period *T*LS,CC at the NC limit state given in Table 4.3 (when using prEN 1998-1-1, 4.3(3)) or the performance factor *γ*LS,CC at the NC limit state given in Table 4.4 (when using prEN 1998-1-1, 4.3(5)).

## Design rules for dissipative (DC2) structural behaviour common to all structural types

### General

1. 15.8.2 to 15.8.5 should be applied to the members of the primary structures designed for dissipative behaviour.

### Design criteria for dissipative structures

1. 11.8.2(1) and (2) should be applied in aluminium buildings.
2. Dissipative zones may be located in the structural members.
3. The non-dissipative parts and the connections of the dissipative parts to the rest of the structure should have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts.
4. The use of transversal welds in dissipative zones should be avoided.

### Design rules for dissipative members in compression or bending

1. Members which dissipate energy in compression or bending should be limited to cross-sectional class 1 in EN 1999-1-1.

### Design rules for dissipative parts of members in tension

1. For tension members or parts of members in tension, the ductility requirement of EN 1999-1-1 should be satisfied.

### Design rules for non-dissipative members

1. The strength and stability of non-dissipative members in DC2 should be verified considering the most unfavourable combination of the axial force *N*Ed, bending moments *M*Ed and shear force *V*Ed, calculated according to Formula (15.1).

(15.1)

where

|  |  |
| --- | --- |
| *N*Ed,G , *M*Ed,G and *V*Ed,G | are the axial force, the bending moment and shear force in the non-dissipative member due to the non-seismic actions in the seismic design situation; |
| *N*Ed,E , *M*Ed,E and *V*Ed,E | are the axial force, the bending moment and shear force in the non-dissipative member due to the design seismic action; |
| ** | is the action magnification factor given in (3). |

1. The action magnification factor *Ω* should be taken to depend on the type of plastic behaviour of the dissipative zone and on the structural system according to Table 15.5.

Table 15.5 — Members to which (1) apply. Values of seismic action magnification factor Ω in DC2

|  |  |  |
| --- | --- | --- |
| **Structural type** | ***Ω*** | **Members to which (1) applies** |
| **Moment resisting frames (MRFs)** |  |  |
| Single-storey MRFs | 1,8 | columns |
| Multi-storey MRFs | 2,0 |
| **Frames with concentric bracings** | 1,5 |  |
| Diagonal bracings | beams and columns |
| V-bracings |
| X-bracings on either single or two-storey |
| **Dual frames (MRFs with concentric bracing)** | 2,0 | beams and columns of the concentric bracing; columns of the MRF |
| **Inverted pendulum** | 1,5 | columns |

### Design rules for connections in dissipative zones

1. The resistance of non-dissipative connections of dissipative members made by means of welds should be evaluated by accounting for the deteriorated contribution of the heat affected zone in order to satisfy Formula (15.2).
2. Non-dissipative connections should be designed for the relevant forces/moments at the location of the connection in accordance with 15.8.5. For fillet welded or bolted non-dissipative connections at the location the dissipative zones, the condition given by Formula (15.2) should be satisfied.

(15.2)

where

|  |  |
| --- | --- |
| *R*d | is the design resistance of the connection in accordance with EN 1999-1-1; |
| *R*fo | is the plastic resistance of the connected dissipative member evaluated in the expected position of the plastic hinge and based on the nominal conventional elastic strength of the material as defined in EN 1999-1-1; |
| *ω*rm | is the material overstrength factor in the dissipative zones (see 15.2.2); in absence of experimental characterization, *ω*rm may be taken equal to 1,5; |
| *ω*sh | is the hardening factor in the dissipative zones (see 15.8.6(2)). |

1. Factor *ω*sh should be taken to depend on the type of plastic behaviour of the dissipative zone and on the structural system. Unless estimated with a refined analytical method, experimental validation or numerical simulation, *γ*sh should be taken as given in a) or b):
2. for members in plastic bending: as 1,3 or the value calculated in accordance with EN 1999-1-1:2023, Annex L, whichever is greater;
3. for members in plastic tension: as 1,5 or the ratio , whichever is greater.
4. Category C bolted joints in shear and category E bolted joints in tension in accordance with EN 1999-1-1:2023, 10.5.3.1, should be used for the connections of dissipative members in DC2.

### Design rules for column-to-column splices

1. 11.8.7(1), (2) and (4) should be applied.
2. The resistance of the welded splice should be evaluated accounting for the influence of the heat affected zone.

## Design rules for moment resisting frames

### Design criteria

1. 11.9.1(1) should be applied.
2. Depending on the location of the dissipative zones, either 15.8.2(4) or 15.8.2(5) should be applied.
3. The target hinge formation pattern should be achieved by conforming to 15.9.2, 15.9.3 and 15.9.4.

### Beams

1. Beams should be verified to have sufficient resistance against lateral and lateral torsional buckling in accordance with EN 1999-1-1:2023, Annex I, assuming the formation of a plastic hinge at one end of the beam. The most stressed beam end in the seismic design situation should be considered for the verification.
2. 15.8.3(1) should be applied.

### Columns

1. The strength and stability of columns should be verified in compression, bending and shear considering the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed from 15.8.5(1).
2. The resistance verifications of columns should be made in accordance with EN 1999-1-1:2023, 8.2. and 8.3.
3. If a plastic hinge is expected in the column, the shear force *V*Ed from the analysis should satisfy Formula (15.3).

(15.3)

1. 11.9.3(6) and (7) should be applied.

### Beam to column joints

1. The connections of the beams to the columns and the column web panels should be designed for overstrength as given by Formula (15.2) for both single- and double-sided joints.
2. 11.9.4(5) and (6) should be applied.

### Column base joints

1. The resistance of rigid full strength non-dissipative base connections *M*c,j,Rd should comply with Formula (15.4).

(15.4)

where

|  |  |
| --- | --- |
| *M*Rd,c(*N*Ed) | is the design moment resistance of the column taking into account the axial load *N*Ed, in the seismic design situation; |
| *ω*rm | is the material overstrength factor given in 15.2.2; |
| *ω*sh | is the hardening factor of the plastic hinge (see 15.8.6(3)). |

## Design rules for frames with concentric bracings

### Design criteria for DC2

1. 11.10.1(1) shall be applied.
2. 11.10.1(2) and (3) should be applied.

### Analysis for DC2

1. 11.10.2(1) should be applied.
2. The diagonals should be taken into account using an elastic analysis of the structure for the seismic action, according to a) and b):
3. The “tension-only” model may be only used for frames with X diagonal bracings or split X diagonal bracings;
4. In frames with V bracings, both the tension and compression diagonals should be taken into account.
5. 11.10.2(3) and (4) should be applied.

### Diagonal members

1. The cross section of diagonal bracings should be of class 1 according to EN 1999-1-1.
2. 11.10.3(2) to (7) should be applied.
3. The connections of the diagonals to any member should satisfy the rules of 15.8.6 with *γ*sh given in 15.8.6(4).

### Beams and columns

1. The strength and stability of both beams and columns should be verified in compression, bending and shear considering the most unfavourable combination of the axial force *N*Ed, bending moments *M*Ed and shear force *V*Ed calculated in accordance with Formula (15.1).
2. 11.10.4(4) and (8) should be applied.

### Beam to column connections

1. 11.10.5(1) should be applied.
2. 11.10.5(2) should be applied with the material variability factor *ω*rm as in 15.2.2 and *ω*sh, the hardening factor, as in 15.8.6(3).

### Brace connections

1. 11.10.6(1) to (4) should be applied.

### Column base joints

1. 11.10.7(1) should be applied, with *ω*rm and *ω*sh as in 15.10.5(2).
2. 11.10.7(2) and (3) should be applied.

## Design rules for dual frames - moment resisting frames combined with concentric bracings

### Design criteria

1. 11.13.1(1) and (2) should be applied.
2. The moment resisting frames and the braced frames should conform to 15.9. and 15.10, respectively.

## Design rules for inverted pendulum structures

1. 11.15(1) to (4) should be applied to inverted pendulum aluminium structures (defined in 15.4.1(d)).

## Aluminium diaphragms

1. The design of floor diaphragms, chord and collector beams should conform to 6.2.8 and 11.17.

## Transfer zones. Design for DC2

1. Diaphragms in transfer zones should be designed to 6.2.9(7) and 11.17.

## Checking of design, supply of material and execution

1. The type of aluminium alloy and the relevant maximum yield stress *f*0 in the dissipative zones, the grade and the tightening force of bolts and the quality of the welds should be indicated in the drawings for fabrication.
2. The technical drawings and reports should clearly specify that aluminium alloys with resistance greater than specified should not be supplied for the dissipative zones.
3. Execution Class 3 should be considered for all consequence classes and ductility classes, with the exception of Execution class 2 that may be adopted for Consequence Class 1.
4. (informative)  
     
   Characteristics of earthquake resistant buildings and in plan regularity
   1. Use of this annex
5. This Informative Annex provides complementary/supplementary guidance to 4.4.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

* 1. Scope and field of application

1. This annex gives good practice rules governing earthquake resistant design relative to:
2. structural simplicity;
3. uniformity, symmetry and redundancy;
4. bi-directional resistance and stiffness;
5. torsional resistance and stiffness;
6. diaphragmatic behaviour at storey level;
7. adequate foundation;
8. structural regularity in plan.

These good practice rules are further elaborated hereunder.

1. Good practice rules should not be considered as mandatory, as they cannot in practice be all satisfied.

NOTE Satisfying good practice rules allows for more economical design.

* 1. Structural simplicity

1. Structural simplicity, characterized by the existence of clear and direct paths for the transmission of the seismic actions, should be pursued, since the modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behaviour is much more reliable.
   1. Uniformity, symmetry and redundancy
2. Uniformity, symmetry and redundancy should be pursued by means appropriate to the considered structure.

NOTE 1 Uniformity in planis characterized by an even distribution of the structural members which allows short and direct transmission of the inertia forces created in the distributed masses of the building.

NOTE 2 Uniformity can be realized by subdividing the entire building by seismic joints into dynamically independent units, if these joints are designed against pounding of the individual units in accordance with 6.2.8.

NOTE 3 Uniformity in the development of the structure along the height of the building tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.

NOTE 4 A close relationship between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness.

NOTE 5 If the building configuration is symmetric or quasi-symmetric, a symmetric layout of well-distributed in-plan structural members is appropriate for the achievement of uniformity.

NOTE 6 The use of evenly distributed structural members increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

* 1. Bi-directional resistance and stiffness

1. To satisfy 4.4.1(1), the structural members should be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.
2. The choice of the stiffness characteristics of the structure, while attempting to minimize the effects of the seismic action (considering its specific features at the site) should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.
   1. Torsional resistance and stiffness
3. Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness to limit the development of torsional motions which tend to stress the different structural members in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building should be preferred.
   1. Diaphragmatic behaviour at storey level
4. Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are in the vicinity of the main vertical structural members, thus hindering such effective connection between the vertical and horizontal structure.

NOTE In buildings, floors (including the roof) play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the seismic actions to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

1. Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis (e.g. rigidity of the diaphragm), particularly when there are significant changes in stiffness or offsets of vertical members above and below the diaphragm.
   1. Adequate foundation
2. The design and construction of the foundations and of the connection to the superstructure should ensure that the whole building is subjected to a uniform seismic excitation.
   1. Regularity in plan
3. Implementing good practices given in a) to e) may contribute to obtain favourable in-plan regularity and to minimize undesirable torsional effects about the vertical axis:
4. With respect to the lateral stiffness and mass distribution, the building structure is approximately symmetric in plan with respect to two orthogonal axes.
5. The plan configuration is compact, i.e., each floor is delimited by a polygonal convex line. If in plan set-backs (re-entrant corners or edge recesses) exist, they do not affect the floor in-plan stiffness. For each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 15 % of the floor area.
6. The in‑plan stiffness of the floors is sufficiently large in comparison with the lateral stiffness of the vertical structural members, so that the deformation of the floor has a small effect on the distribution of the forces among the vertical structural members. The effect may be considered small if the forces among the vertical structural members does not differ of more than 20 % of the values found with an infinitely stiff diaphragm. In this respect, L, C, H, I, and X plan shapes should be carefully examined concerning the stiffness of the lateral branches, which should be comparable to that of the central part, to satisfy the rigid diaphragm condition. The application of this paragraph should be considered for the global behaviour of the building.
7. For buildings in which the primary structure is in the facades, if the slenderness λ= *L*max/*L*min of the building in plan is not greater than 4, where *L*max and *L*min are respectively the greater and smaller in plan dimension of the building, measured in orthogonal directions.

NOTE If the number of lines of primary seismic structures perpendicular to the long side of a building is greater than 0,5 *λ*, like for instance in long industrial halls, with many parallel frames perpendicular to the long dimension, there is no need to satisfy that condition.

1. At each level *i* and for each direction of analysis *x* or *y*, the structural eccentricity *e*o and the torsional radius *r* satisfy the two conditions given by Formulae (A.1) and (A.2), expressed for the direction of analysis *y*.

(A.1)

(A.2)

where

|  |  |
| --- | --- |
| *e*ox,i | is the *x* component of the natural eccentricity, normal to the direction of analysis *y* considered; |
| *r*x,i | is the *x* component of the torsional radius; |
| *l*s,i | is the radius of gyration of floor *i* defined in 4.4.3(2). |

NOTE 1 Annex B gives procedures for calculating the natural eccentricity and the torsional radius.

NOTE 2 Formulae similar to (A.1) and (A.2) apply for the direction of analysis *x*.

1. (informative)  
     
   Natural eccentricity and torsional radius
   1. Use of this annex
2. This Informative Annex provides complementary / supplementary guidance to 4.4.3.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

* 1. Scope and field of application

1. This annex gives possible methods to calculate the natural eccentricity and the torsional radius.
   1. General
2. The natural eccentricity *e*0,i of floor *i* of a building should be defined as the horizontal vector *C*i*G*i joining the two points defined in a) and b):
3. the centre of masses *G*i of floor *i,* including the mass of the vertical members in the mid height of the storeys below and above the floor, the mass being calculated according to 5.1.2(1);
4. the projection *C*i on floor *i* of the stiffness centre of the storey below the floor, including all vertical structures taken into account in the model for seismic analysis.
5. The two components *e*0x,i and *e*0y,i of *e*0,i in orthogonal horizontal axes may be obtained as the difference of coordinates of *G*i and *C*i in the same axes.

NOTE The two components *e*0x,i and *e*0y,i of ***e***0,i in orthogonal horizontal axes are used in 5.2(2).

1. The *x* component of the torsional radius, *r*x,i, of the *i*-th floor should be defined as the square root of the ratio of the torsional stiffness with respect to the stiffness centre *C*i to the floor lateral stiffness in the *y* direction.

NOTE Similar definition applies to the *y* component *r*y,i,

* 1. Uniform type of lateral load resisting system

1. When the primary structure is composed of the same type of primary bracings continuous from the basement to the level under consideration, the coordinates of the natural eccentricity may be calculated as given by Formula (B.1), see Figure B.1.

; (B.1)

where

|  |  |
| --- | --- |
| *x*Ci and *y*Ci | are the coordinates of the stiffness centre in orthogonal horizontal axis *x* and *y*; |
| *x*j and *y*j | are the coordinates of bracing element *j* of the floor under consideration; |
| *K*x,ij | is the lateral stiffness in the *x* direction of primary bracing *j* in the storey below floor *i*, as given in (2); |
| *K*y,ij | is defined similarly to *K*x,ij, but in the *y* direction. |

Figure B.1 — Buildings with a uniform type of lateral load resisting system: (a) with columns; (b) with walls and box walls

1. For a primary structure composed of columns (Figure B.1(a)), the lateral stiffness *K*x,ij in the x direction may be taken as given by Formula (B.2).

(B.2)

where

|  |  |
| --- | --- |
|  | is the flexural stiffness of the column *j* in the storey below floor *i* in the direction x; |
|  | is the free height of the column *j* in the storey below floor *i*. |

1. For a primary structure composed of walls and box walls (Figure B.1(b)), the lateral stiffness *K*x,ij in the *x* direction may be taken as given by Formula (B.3). For box walls, stiffnesses may be calculated considering the tubular inertia and area.

(B.3)

where

|  |  |
| --- | --- |
|  | is the flexural stiffness of the wall j in the storey below floor i in the direction *x*; |
|  | is the gross area of the section of the wall. |

1. For a primary structure composed of frames with bracings, the lateral stiffness *K*x,ij in the *x* direction may be taken as given by Formula (B.4).

(B.4)

where

|  |  |
| --- | --- |
| *d*i | is the length of the bracing element; |
| ** | is the angle of the bracing element to the horizontal; |
|  | is the axial stiffnesses of the bracing element considered. |

NOTE (2) to (4) apply similarly to *K*y ,ij.

1. The *x* component of the torsional radius, *r*x,i, of the *i*-th floor may be taken as given by Formula (B.5).

(B.5)

whereand may be taken as in (2) to (4), as appropriate.

NOTE A Formula similar to (B.5) applies to .

* 1. Calculation by a 3D model

1. The components of the natural eccentricity *e*0,i in two orthogonal horizontal axes *x* and *y* may be calculated at all floors of a building using a 3D model and the procedure in (2) to (4), with the condition that the floors may be considered as rigid in their planes.
2. A profile of forces should be defined in applying at each floor *i* a unit acceleration to the mass which, as defined in B3(1)a), is calculated according to 5.1.2(1).

NOTE This simple profile allows for an approximate value of the natural eccentricity at each floor considered sufficient for its use in this Standard.

1. Three load cases should be applied separately to the structure, as given in a) to c):
2. Case 1: the profile of forces defined in (2) applied in the *x* direction, which gives
3. Case 2: the profile of forces defined in (2) applied in the *y* direction, which gives .
4. Case 3: a profile of moments about the vertical axis applied at the centres of masses of the floors, which gives . The values of the moments should be equal to the values of the forces in the profile in (2).
5. The components *e*0x,i and *e*0y,i of the natural eccentricity should be taken as given by Formula (B.6).

(B.6)

where

|  |  |
| --- | --- |
|  | is the difference of rotations about the vertical axis between floor *i* - 1 and floor *i* in Case 1; |
|  | is the difference of rotations about the vertical axis between floor *i* - 1 and floor *i* in Case 2; |
|  | is the difference of rotations about the vertical axis between floor *i* - 1 and floor *i* in Case 3. These rotations may be calculated from the displacements at opposite sides of the floors. |

1. The torsional radius *r*x,i in the *x* direction should be taken as given in Formula (B.7).

(B.7)

where *∆d*y,C,i = *∆d*y,G,i – *∆*y,i *e*0x,i is the difference of displacements of the stiffness centre *C* in the *y* direction between floor *i* - 1 and floor *i* in Case 2 and *∆d*y,G,i is the corresponding difference of displacements of the centre of mass *G*.

NOTE A Formula similar to (B.7) applies to .

1. (normative)  
     
   Floor accelerations for ancillary elements
   1. Use of this normative annex
2. This Normative Annex contains additional provisions to 7.2.1 for calculating pseudo-absolute accelerations applicable to ancillary elements.
   1. Scope and field of application
3. This Normative Annex should be applied to calculate the floor acceleration spectrum applicable to an ancillary element in the case where 7.2.1(6) or (7) are not applied.
   1. Floor spectra
4. The floor acceleration spectrum value *S*ap introduced in 7.2.1 may be calculated by the procedure presented in this annex.
5. For mode *i* and floor *j*, the value of the *S*ap component in the direction under consideration may be determined by Formulas (C.1) to (C.3).

(C.1)

(C.2)

with

(C.3)

where

|  |  |
| --- | --- |
| indices "p" and "ap" | correspond to the primary structure and the ancillary element, respectively;  and, in addition to symbols introduced in 7.2.1 |
| *S*ep,i = *S*e(*T*p,i, *ξ*p,i) | is the value in the elastic response spectrum which applies to the *i*th mode of the primary structure; |
| *S*eap = *S*e(*T*ap, *ξ*ap) | is the value in the elastic response spectrum which applies to the ancillary element; |
| *T*p,i | is the natural period of the *i*th mode of the structure; |
| *ϕ*ij | is the *i*th mode shape value at the *j*th floor, considering also (3) and (4); |
| *Γ*i | is the modal participation factor for the *i*th mode. In a planar model with concentrated masses, *Γ*i is defined by Formula (C.4), where *m*j is the mass at the *j*th floor |

(C.4)

where

|  |  |
| --- | --- |
| *ξ*p,i | is the critical damping ratio (in %) of the primary structure for buildings, equal to 5 %. |
| *qD*’ | is the behaviour factor defined in 7.2.1(7), Formula (7.5). |

1. In the case of lateral force analysis, the displacement shape used for the determination of lateral forces given in 5.3.3 may be used.
2. In the case of pushover analysis, the inelastic deformation shape (corresponding to the design seismic action at the limit state under consideration) should be used instead of the fundamental mode shape {**1}.
3. At every floor, *j*, the resulting floor acceleration *S*ap,j should be calculated as a combination of values for all modes of vibration contributing significantly to the global response as specified in prEN 1998-1-1:2022, 6.4.3.1(3). *S*ap,j should not be smaller than *S*eap.
   1. Modelling
4. The value of the behaviour factor *q*ap’ defined in 7.2.1 should be calculated using Formula (C5).

(C.5)

where

|  |  |
| --- | --- |
|  | is the behaviour factor component accounting for all sources of overstrength; it can be taken as equal to 1.3 unless another value is specified or documented for the ancillary element under consideration; |
|  | is the frequency dependent behaviour factor component accounting for the deformation capacity and energy dissipation capacity of the ancillary element, given by Formula (C.6). |

(C.6)

where *q*ap,D is given in (2).

1. Except if properly documented or provided in the relevant parts of EN 1998, *q*ap,D values should not exceed those given in Table C1.

NOTE Values of *q*ap,D for out-of-plane assessment of ancillary masonry walls are given as *q*oop in 14.4.2

Table C.1 — Maximum values of *q*ap,D for ancillary elements

|  |  |
| --- | --- |
| **Type of ancillary element** | ***q*ap,D** |
| **Elements not able or not allowed to dissipate energy by inelastic deformation:** | 1 |
| Cantilevering parapets or ornamentations  Signs and billboards  Chimneys, masts and tanks on legs acting as unbraced cantilevers along more than one half of their total height |
| **Elements dissipating energy by inelastic deformation:** | 2 |
| Exterior and interior walls  Partitions and façades, claddings, veneers  Chimneys, masts and tanks on legs acting as unbraced cantilevers along less than one half of their total height, or braced or guyed to the structure at or above their centre of mass  Anchorage elements for permanent cabinets and book stacks supported by the floor  Anchorage elements for false (suspended) ceilings and light fixtures |

1. Alternatively to (1), in case the behaviour of a rigid ancillary element is documented as rigid-plastic, with an acceptable displacement, , intended for verification to SD, may be calculated as given by Formula (C.7).

(C.7)

where

|  |  |
| --- | --- |
|  | is the acceptable displacement of the ancillary element, |
| *T*P1 | is the fundamental period of the building, |
|  | is the acceleration capacity of the ancillary element, given by its force capacity, *F*ap,R, divided by its total mass *m*ap*.* |

1. (normative)  
     
   Buildings with energy dissipation systems
   1. Use of this normative annex
2. This Normative Annex contains additional provisions to Clause 9 for buildings with energy dissipation system.
   1. Displacement ductility ratio
3. The definition of the displacement ductility ratio should be as given in 9.3.2.1(5).
4. The displacement ductility ratio **f for structures with displacement-dependent energy dissipation devices should be defined as the maximum displacement relative to the ground in the direction of the considered seismic action at roof level divided by *d*roof y calculated by Formula (D.1).

(D.1)

where

|  |  |
| --- | --- |
| *V*py,1 and *V*dy,1 | are defined in 9.3.3.2(1); |
| *m*eff 1 | is the first mode effective mass of the structure with energy dissipation system in the elastic range, in the direction under consideration. |

1. The displacement ductility ratio **f for structures with velocity-dependent energy dissipation devices should be defined as the maximum displacement relative to the ground at roof level divided by *d*roof y calculated by Formula (D.2).

(D.2)

where

|  |  |
| --- | --- |
| *V*py,1 | is as in (1); |
| *T*p1 | is the first mode vibration period of the primary seismic members in elastic conditions, exclusive of energy dissipation system; |
| **p1 | is the first mode participation factor of the primary seismic members in elastic conditions, exclusive of energy dissipation system in the direction under consideration; |
| *m*p,eff 1 | is the first mode effective mass of the primary seismic members in elastic conditions, exclusive of energy dissipation system in the direction under consideration. |

* 1. Complementary rules for structures with velocity-dependent energy dissipation devices
     1. Effective period

1. The first mode effective period *T*eff 1 in the direction under consideration for structures with velocity-dependent energy dissipation devices should be calculated with Formula (D.3).

(D.3)

where **V1 is the component of the effective damping of the structure in the direction of interest due to viscous dissipation of energy by velocity-dependent energy dissipation devices, at or just below the yield displacementof the main structural system, calculated with Formula (D.5).

* + 1. Effective damping

1. The first mode effective damping **eff 1 in the direction under consideration for structures with velocity-dependent energy dissipation devices should be calculated with Formula (D.4).

(D.4)

where

|  |  |
| --- | --- |
| **V1 | as in D3.1(1); |
| **I | is the component of effective damping of the structure due to inherent dissipation of energy by the main structural system and ancillary components, exclusive of energy dissipation system, at or just below its yield displacement. **I is given in (3); |
| **H | is the component of effective damping of the structure in the direction of interest due to post-yield hysteretic behaviour of the primary seismic members of the main structural system at the ductility ratio **f; **H is calculated with Formula (D.6). |

1. For structures with velocity-dependent energy dissipation devices, **V1 in the direction under consideration should be calculated with Formula (D.5).

(D.5)

where the summation over *i* extends over all masses, summation over *j* extends over all velocity-dependent energy dissipating devices and

|  |  |
| --- | --- |
| **r*j*1 | is the difference between the first modal ordinates (horizontal displacements in the direction under consideration) associated with degrees of freedom corresponding to the ends of the energy dissipating device *j*; |

NOTE If the ends of the energy dissipating device *j* are connected to degrees of freedom *i* and (*i-1*) then **r*j*1 = *i*1 - **(*i*-1)1 where *i*1 and **(*i*-1)1 are the componentsof the first modal vector ****1 associated with the degrees of freedom *i* and (*i*-1).

|  |  |
| --- | --- |
| *j* | is the angle of inclination of energy dissipation device *j* to the horizontal; |
| *Cj* | is the damping coefficient of the velocity-dependent energy dissipation device *j*, provided by the producer of the device; |
| *a* | is the velocity exponent of the velocity-dependent energy dissipation devices, provided by the producer of the device. |

1. **I should be based on the material type, configuration, and behaviour of the main structural system and ancillary components responding dynamically at or just below yielding of the primary seismic members of the main structural system. Unless analysis or test data supports other values, inherent damping should not be taken greater than 5 % of critical damping for all modes of vibration.
2. For structures with velocity dependent energy dissipation devices **H should be calculated with Formula (D.6).

(D.6)

where *F*H is the hysteresis loop adjustment factor; it may be taken as 0,67*T*C/*T*p1; it should not be taken greater than 1 and may be taken greater than 0,5.

* 1. Complementary rules for structures with displacement-dependent energy-dissipation devices
     1. Calculation of *E*e

1. The amount of energy *E*ethat can be absorbed by a structure with displacement-dependent energy dissipation devices in the direction under consideration while the primary seismic members of the main structural system remain in the elastic domain should be calculated by Formula (D.7).

(D.7)

where the summation over *k* extends to all stories and

|  |  |
| --- | --- |
| *E*pes,k | is the elastic strain energy stored by the primary seismic members of the main structural system at the *k-*th storey in the direction under consideration while these members remain in the elastic domain; *E*pes,k is given in (2); |
| *E*des,k | is the elastic strain energy stored by the energy dissipation devices at *k-*th storey in the direction under consideration while the primary seismic members of the main structural system at the *k*-th story remain in the elastic domain; *E*des,k is given in (3); |
| *E*dH,k | is the energy dissipated in form of plastic strain by the energy dissipation devices at the *k-*th storey in the direction under consideration while the primary seismic members of the main structural system at the *k*-th story remain in the elastic domain; *E*dH,k is given in (4). |

1. *E*pes,k should be calculated using Formula (D.8).

(D.8)

where

|  |  |
| --- | --- |
|  | is the maximum shear for which the primary seismic members of the main structural system at the *k*-th storey in the direction under consideration remains in the elastic domain; |
| *d*D,k | is the maximum interstorey drift for which the primary seismic members of the main structural system at the *k*-th storey in the direction under consideration remains within the elastic range. |

NOTE *E*pes,k is represented by the area of triangle o-c-d in Figure 9.2.

1. *E*des,kshould be calculated using Formula (D.9).

(D.9)

where

|  |  |
| --- | --- |
| *V*dy,k | is defined in 9.3.3.2(1); |
| *d*dy,k | is the *k-*th storey drift at yielding of the energy dissipation devices of this storey in the direction under consideration. |

NOTE *E*pes,k is represented by the area of triangle o-c-d in Figure 9.2.

1. *E*dH,k should be calculated using Formula (D.10).

(D.10)

NOTE *E*dH,k is taken as equivalent to the energy dissipated by the dampers in one cycle of displacement at amplitude *d*D,k, that is, four times the area of rectangle a-b-d-e in Figure 9.1.

* + 1. Calculation of *E*H,*k*

1. The required amount of energy dissipation *E*H,*k* in form of plastic strain at the *k*-th storey of the building in the direction under considerationshould be calculated with Formula (D.11).

(D.11)

where

|  |  |
| --- | --- |
|  |  |
| *T*s1 | is a period that gives the maximum value of [*T*s1*S*e(*T*s1*,*5%)**(*T*s1*,*I)] between *T*1and 1,4*T*1; |
| *s*k | is a standard value representing the ratio of the required amount of dissipation at storey *k* to the required amount of dissipation at the first storey; it should be calculated using Formula (D.12); |

(D.12)

where

|  |  |
| --- | --- |
| *p*k | represents the deviation of the k-th storey seismic shear from an optimum value that would provide an approximately even distribution of damage among storeys, calculated using Formula (D.13). |

(D.13)

where

|  |  |
| --- | --- |
| *p*t,k | is a coefficient that accounts for the increase in the required dissipation capacity in a storey k resulting from torsional effects, calculated using Formula (D.14). |

(D.14)

where

|  |  |
| --- | --- |
| *e*ox | is the distance between the centre of stiffness and the centre of mass, measured along the *x* direction, which is normal to the direction of analysis *y* considered; |
| rx | is the square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction (“torsional radius”), which may be calculated by Formula (A3) in Annex A; |
| *n* | represents the extent to which the required amount of energy dissipation of the building is distributed to each storey according to the stiffness and strength of each storey. It should be taken as *n*= 4 if the main structural system satisfies the global and local ductility conditions of 6.2.4 and 6.2.5, as appropriate, and as *n*= 8 otherwise. |

* + 1. Calculation of *E*pH,*k*,max

1. The maximum energy dissipation demand *E*pH,*k*,max at the *k-*th storey on the primary seismic members of the main structural system in the direction under consideration should be calculated with Formula (D.15).

(D.15)

where

|  |  |
| --- | --- |
| *E*H,k | is as calculated in D4.2; |
| *d*py,k, *V*py,kand *V*dy*,k* | are defined in 9.3.3.2(1); |
| *d*r,k,1 | is defined in 9.3.3.2(1); |
| *d*dy,k | as in D4.1(3). |

NOTE In one cycle of displacement at maximum interstorey drift , the primary seismic members of the main system dissipate an amount of energy equal to , and the energy dissipation system dissipate an amount of energy equal to . The second member of Formula (D.15) distributes the energy to be dissipated by the storey *k, E*H,k, between the primary seismic members of the main system and the energy dissipation system in the same proportion that is distributed in a single cycle of displacement.

* + 1. Calculation of *E*dH,*k*,max

1. The maximum energy dissipation demand *E*dH,*k*,max at the *k*-th storey on the energy dissipation devices in the direction under considerationshould be calculated with Formula (D.16).

(D.16)

where all notations are defined in D.4.3.

NOTE 1 The first term of the second member distributes the *E*H,*k* between the primary seismic members of the main system and the energy dissipation system as explained in the NOTE to D.4.3.

NOTE 2 Under the design earthquake, the dampers dissipate energy through plastic deformations before the primary seismic members of the main structural system reach the yield displacement *d*py,*k*. The second term of the second member of Formula (D.16) represents this energy that is taken as equivalent to ten cycles of amplitude *d*py,*k*.

NOTE 3 The energy dissipation devices dissipate energy through plastic deformations under low intensity earthquakes, for which the main system remains in the elastic domain. The third term of the second member represents this energy. It is assumed that the structure can be subjected to five low intensity earthquakes, and in each one the energy to be dissipated by the energy dissipation devices is equivalent to five cycles of amplitude *d*D,*k.*

1. (normative)  
     
   Seismic design of connections for steel buildings
   1. Use of this normative annex
2. This Normative Annex contains additional provisions to Clause 11 for the seismic design of connections.
   1. Scope and field of application
3. This annex should be used for the design of beam-to-column joints of moment resisting and dual steel frames and for the design of gusset connections in concentrically, eccentrically and buckling restrained bracings. Annex E should be applied for joints of primary DC3 structures in addition to the provisions in Clause 11 and in prEN 1993-1-8.
4. Annex E may be used for joints of primary DC2 and DC1 structures.
5. Annex E may be applied to connections different from those specified in Annex E, however, in those cases the validity and effectiveness of their performance should be demonstrated by means of either experimental evidence, past experimental results available in the literature or refined numerical simulations.
6. The moment resisting beam-to-column joints covered by Annex E may be used for beam-to-column joints in the braced bay of bracing systems (i.e. concentrically, eccentrically and buckling restrained bracings).
   1. Pre-qualified moment resisting beam-to-column joints
      1. General

NOTE 1 A beam-to-column joint is defined as the zone where at least one beam and one column are interconnected. A planar major-axis beam-to-column joint consists of a web panel and either one connection for external joints (or “single-sided” joint) or two connections for internal joints (or “double-sided” joint).

NOTE 2 Where present, the requirement to full strength connections and strong web panels lead to nominally rigid connections and web panels.

1. The joints specified in E.3 may be considered as conforming to 11.9.4; they may be considered as prequalified and may be used in DC3 and DC2 moment resisting frames and moment resisting sub-system of dual frames without any further qualifying cyclic tests.
2. The beam-to-column connections specified in E.3 may be also used in the braced bay of concentrically, eccentrically and buckling restrained bracings.

NOTE Pre-qualified joints are such that their rotation capacity is not smaller than the limit specified in 11.9.4(9) or 11.10.5(2)a, as appropriate.

1. The beams should be torsionally restrained with bracings spaced by the length *L*st calculated as given by Formula (11.6).
2. If a concrete or composite slab is used and if the structure is calculated as a steel structure, the composite action between the beam and the slab should be prevented by detailing the slab-to-beam zone as specified in 11.9.4(1) and 12.8.6.2.3 and with the detailing given in a) to c):
3. no shear connectors should be used on the beams belonging to the column (i.e. both the primary beams and the transverse beams) within a length equal to 1,5 times the depth of the primary beam from any protruding parts of the connection;
4. a gap not less than 25 mm should be kept between the concrete slab and any protruding parts of the connection and all around the column; it should be either void or filled with compressible material.
5. steel rebars and steel mesh in the concrete slab should not cross the joint in the gap defined in (c).
   * 1. Classification of pre-qualified moment resisting beam-to-column joints
        1. Classification by rotational stiffness
6. Stiffness classification of the connection should be as defined in prEN 1993-1-8.
   * + 1. Classification by moment resistance
7. Unlike in the resistance classification of connections in prEN 1993-1-8, the type and the localization of plastic mechanism shall be explicitly considered.
8. To satisfy (1), the random material variability (through *ω*rm, see 11.2.2) and strain hardening (through *ω*sh, see Table 11.10) of the dissipative zone (i.e. either beam or connection) should be accounted for.
9. In the use of Annex E for DC2 connections, *ω*sh may be considered equal to 1,1.

Key

|  |  |
| --- | --- |
| *A* | idealized position of the plastic hinge |
| *B* | beam |
| *C* | column |
| *D* | rib or haunch stiffener (if adopted) |

Figure E.1 — Definition of design moment: position of plastic hinges in stiffened joints

1. The categories of the connections should be classified on the basis of the localization of the dissipative mechanism in the joint in accordance with Formula (E.1) as in a) to d):
2. Full strength or “non-yielding” connections, where the plastic deformations are localized in the beam, should be designed for the expected strength of the plastic hinge in its location by accounting for the effects of material over-strength and strain hardening as given in Formula(E.1).

(E.1)

where other variables are as defined in 11.9.3(2) and

|  |  |
| --- | --- |
|  | is the design resistance of the connection; |
| *M*b,pl,Rk | is the characteristic value of the resistance to bending moment of the beam framing the joint; |

NOTE The partial factor *γ*M0 for the nominal resistance of cross-sections is defined in EN 1993-1-1:2022, 8.1.

|  |  |
| --- | --- |
| *s*h,c | is the distance between the centre of the expected plastic hinge and the connection (i.e. the column face), see Figure E.1; |

1. Equal strength or “balance yielding” connections, where the plastic deformations occur in both the beam and the connection, should be designed for the nominal strength of the plastic hinge in its location as given in Formula (E.2). Both the members and the connections should be ductile.

(E.2)

NOTE Due to variability of material characteristics and strain hardening in the connection and in other parts of the structure, plastic deformations can occur in the connection and/or the column web panel.

1. Partial strength “yielding” connections, where the plastic deformations are localized in the connection, should be designed for the action effects in the seismic design situation as given in Formula (E.3).

(E.3)

where *ω*rm is the material overstrength factor of the beam (see 11.2.2), but 1/*ω*rm should not be taken smaller than 0,75.

1. Partial strength “friction” connections, where the dissipation mechanism is due to the slippage of the clamped friction surfaces between the lower part of the beam and its connection, should be designed for the action effects in the seismic design situation as given in Formula (E.4).

(E.4)

where *ω*rm is the material overstrength factor of the beam (see 11.2.2).

1. Pinned connections should comply with prEN 1993-1-8:2021, 5.5 and 5.7.3.
2. Members with partial-strength connections or friction connections should be designed for the ultimate expected resistance developed in the connection, including the effects of material overstrength and strain hardening and friction overstrength in accordance with Formula (E.5).

(E.5)

where

|  |  |
| --- | --- |
| *M*pl,Rd | is the design moment resistance of the member framing the connection; |
| *ω*rm | is the material overstrength factor of the yielding components of the connection (see 11.2.2); |
| *ω*sh | is the hardening factor of the yielding components of the connection (see Table 11.10). |
| *ω*sr | is the strain rate overstrength factor of the friction components of the connection. |
| *ω*μ | is the friction overstrength factor of the friction components of the connection. |

NOTE 1 The strain rate overstrength factor *ω*sr of the friction components of the connection can be assumed equal to 1,1 unless a different value of *ω*sr result from specific experimental tests realized and processed as described in prCEN/TS 1998-1-101.

NOTE 2 The friction overstrength factor *ω*μ of the friction components of the connection is the ratio between the 95 % fractile value of the static friction coefficient *μ*s.upper and the design value of the 5 % fractile value of the dynamic friction coefficient *μ*d. In the absence of experimental characterization, *ω*μ can be assumed equal to 1,6.

1. The column web panel should be designed according to categories a) to c) in terms of their contribution to the plastic mechanism:
2. strong web panel, where most of the plastic demand is not in the column web panel (e.g. concentrated in the connection and/or in the beam): it should be designed using Formula (11.14) in which the shear resistance *V*wp,Rd of the web panel is taken equal to the web panel's “elastic” resistance without the additional resistance of continuity plates (if present);
3. balanced web panel, where the plastic demand is balanced between the column web panel and the connection and/or the beam; it should be designed using Formula (11.14) in which *V*wp,Rd is taken equal to the web panel's plastic resistance, namely accounting for the additional resistance of continuity plates (if present) and the shear force *V*wp,Ed in the web panel is taken equal to the minimum moment resistance between the beam and connection;
4. weak web panel, where all the plastic demand is concentrated in the column web panel: it should be designed using Formula (11.14) in which the shear force *V*wp,Ed is taken equal to the design moments in the seismic situation divided by the web panel depth.
5. For full strength connections in the primary structure of moment resisting and dual frames, strong column web panel should be used.
6. For equal strength connections in the primary structure of moment resisting and dual frames, any types of column web panel may be used.
7. For partial strength connections in the primary structure of moment resisting and dual frames, balanced or weak column web panel should be used.
8. For friction connections in the primary structure of moment resisting and dual frames, strong column web panel should be used.
   * 1. Types of pre-qualified joints and technological requirements
        1. General
9. Moment-resisting beam-to-column joints should have either welded or bolted configurations (see Figure E.2) and be designed in order to ensure the required resistance and ductility in the seismic design situation.
10. Unstiffened connections in FigureE.2 a) and e) may be designed with reduced beam section in the zone where plastic hinge is expected to form.
11. To fulfil the resistance and stiffness requirements, full and equal strength connections with welded or bolted configuration may be strengthened through stiffeners (e.g. haunches, rib stiffeners, cover plates, etc., see Figures E.2 (b) to (d) and (f) to (h) and the column web panel may be reinforced through supplementary welded web plates. The beams of joints with friction connections should be strengthened through rib stiffeners, see Figures E.2 (i) and (j).
12. The beams should have I cross section (e.g. IPE profiles). The weak beam-strong column hierarchy (see 11.9.3(2)) should be satisfied. The beam may be either hot-rolled or welded built-up sections. Hot-rolled profiles should comply with EN 10034. The web and flanges of built-up beams should be connected using full penetration groove welds with a pair of reinforcing fillet welds within a zone extending from the beam end to a distance not less than one beam depth beyond the plastic hinge location *S*h, (see Figure E.1). The throat thickness of each reinforcing fillet weld should be the smaller than of 8 mm and of the thickness of the beam web.
13. For both full strength and equal strength connections, the beam web and flange in compression or in tension components should be ignored in the calculation of the resistance.
14. The column may be either hot-rolled, built-up, cruciform or cold-formed hollow section.
15. Depending on the type of cross section, the columns should be manufactured in accordance with a) to d):
16. hot-rolled profiles with H shape (e.g. HE sections) should comply with EN 10034;
17. either H or box built-up columns may be used. The column web should be connected to the flanges using full penetration groove welds with a pair of reinforcing fillet welds, within a zone extending from 300 mm above the upper beam flange to 300 mm below the lower beam flange and the minimum throat thickness of the reinforcing fillet welds should be the smaller of 8 mm and of the thickness of the column web;
18. the elements of flanged cruciform columns should be fabricated from rolled shapes or built up from plates and should comply with EN 1993-1-1; the web-to-web welds if rolled shapes are used or all welds constituting the cross section if built-up solution is adopted should comply with (b);
19. cold-formed hollow profiles should be class 1 and comply with EN 1993-1-1.
20. For full strength, equal strength and friction connections, if supplementary plates are added onto the column web and detailed as specified in E.3.3.2 and Figures E.5 a, b, the column web in transverse tension and compression components may be ignored in the calculation of the resistance.
21. In welded connections and in bolted connections conforming with (5) and (7) and having a single bolt row between the beam tension flange and the axis of the beam, the resistances of the connection and the column web panel should be calculated independently as two separate macro-components.
22. If (9) is applied, the shear resistance of the column web panel should be separately calculated in accordance with prEN 1993-1-8 and compared to the minimum resistance of either the connection or the beam for panel zone classification; for the application of the local hierarchy of resistances, see E.3.2.2(4).
23. In bolted connections, if more than one bolt row is between the beam tension flange and the beam axis, the connection and column web panel should not be considered as separate macro-components and their interaction should be considered in the calculation of the joint in accordance with prEN 1993-1-8.

Figure E.2 — The types of welded and bolted joints covered by Annex E: (a, e) unstiffened; (b, d, f, h) stiffened with ribs; (c, g) stiffened with haunches; (i) friction joint parallel to the beam flange; (j) friction joint parallel to the beam web

* + - 1. Detailing of column web panel

1. Supplementary web plates may be used to satisfy Formula (11.14), considering a) to c):
2. If these plates are placed either onto the column web panel with longitudinal groove welds, the gap *g* between the column web and the plate should be not greater than 1 mm (see Figure E.5a).
3. If fillet longitudinal welds are used, there should be no gap *g* between the column web and the plate (see Figure E.5b) and plug welds should be used as detailed in Figure E.5c for every thickness of the plate.
4. If supplementary web plates are spaced away from the column web, the distance *e*f of the spaced plates from the edge of the column flange should be not smaller than 25 mm (see Figure E.5) and each spaced plate should satisfy Formula (11.16) in order to be considered as active strengthening plate.
5. The resisting shear area *A*v should be calculated as the sum of the column shear area Av,c and the gross area of the additional web plates *A*v,p.
6. The width-to-thickness ratio *b*s/*t*swp of a supplementary web plate should be not smaller than 40ε, where . Plates with *b*s/*t*swp greater than 40ε may be used only if placed onto the column web, provided that plug welds are used to prevent plate buckling (see Figure E.5 c).
7. The depth of the supplementary web plates in welded or bolted connections should be such that they extend throughout the effective width of the column web in tension and compression. In bolted extended end-plate joints (either stiffened by rib plates or haunch) the depth of the supplementary web plates should not be less than that of the end plate.
8. Continuity plates are transverse stiffeners (see Figure E.7) and should be used to strengthen the column flange and the column web panel. Their thickness should not be smaller than the thickness of the beam flange or of the haunch flange. Continuity plates should not extend beyond the edge of the column flange (see Figures E.7, b and c).
9. long the web of an H shape column, the corner clip *c*c of the continuity plate (see Figure E.7 a) should be detailed to avoid interference with the fillet radius of hot-rolled columns (see Figure E.7 b) or the longitudinal welds of built-up columns (see Figure E.7 c). To this end, the corner clip should be not smaller than the fillet radius + 1 mm in hot rolled column or 15mm + the side of the weld in built-up column.
   * + 1. Detailing of welds
10. The joint details adopted shall make use of an appropriate type of weld for the component to be connected and for the plastic engagement of the joint.
11. Three types of weld details may be adopted: full penetration groove welds (FPGW), fillet welds (FW) and plug welds (PW), as appropriate.
12. FPGW should be detailed with single or double bevel using V and K shape for plates and I or H members and using J or U shapes for cold-formed hollow profiles. The slope of the bevels should not be smaller than 45° and not greater than 60°. FPGW should be adopted in the details a) to e):
13. to connect the beam flange to the endplate or to the column flange (see Figure E.3);
14. to connect the rib stiffener (see Figure E.4 and Figure E.9 b) and the flange of a haunch (see Figure E.9 a) to the beam flange and to either the end plate or column flange;
15. to connect supplementary plates onto the column web panel in the flange corner fillet radius (see Figure E.5 a) and for supplementary plates spaced from the column web (see Figure E.5 d);
16. to connect continuity plates to the column flanges if the shear resistance *V*wp,Rd of the column web panel is calculated accounting for their additional resistance (see Figure E.6 a);
17. to connect diaphragms to columns made of hollow profiles.
18. FW width should not be smaller than 50 mm and their throat not smaller than 0,8 times the thickness of the connected element; they should be adopted in a) to d):
19. to connect the beam web to either the end plate or the column flange;
20. to strengthen with internal fillet welds the beam flange-to-end-plate groove welds (see Figure E.3);
21. to connect the continuity plates to the column flanges (if the shear resistance *V*wp,Rd of the column web panel is calculated without their contribution) and web (see Figure E.6 b);
22. to connect the outermost edges of supplementary plates to the column web (see Figure E.5 c).
23. PW should be not less than four (see Figure E.5 c) and should be adopted if *b*s/*t*swp ≥ 40ε (see E.3.3.2(3)).
24. For full welded unstiffened beam-to-column joints, weld access holes should be made in the web of the beam close to the beam flange-to-column flange connection. The geometrical features and details of weld access holes should comply with Figure E.8. The access holes should be made by means of initial drilled holes, see detail (C) in Figure E.8, with diametre between 18 mm and 20 mm, and finished with either oxy cutting, plasma cutting or laser cutting. Flame cutting should be avoided. Backing plates (see detail (E) in Figure E.8) should be removed after the welding.
25. For bolted end plate connections with or without stiffeners (i.e. rib – see Figure E2 (b) or (d)- or haunch- see Figure E2 (c)) and welded stiffened joints, weld access holes should not be made. The geometrical features and details of the weld of beam flange-to-column flange connection should comply with Figure E.3.
26. In both welded or bolted connections with either rib or haunch stiffeners, the stiffener plates should be terminated at the beam flange and at the end of the end plate or column flange with landings equal to the maximum between the thickness of the beam flange *t*fb and 25 mm (see Figure E.9(5)).
27. In both welded and bolted connections with rib stiffeners, the groove full penetration welds should be parallel to the rib depth *h*r and length *l*r and should not wrap around the tip of the stiffener, see Figure E.9b.

Key

|  |  |
| --- | --- |
| *1* | endplate |
| *2a* | upper beam flange |
| *2b* | lower beam flange |
| *3* | full penetration groove weld |
| *4* | reinforcing fillet weld |

Figure E.3 — DC3 details of beam flange-to-end-plate connection:   
(a) upper beam flange; (b) lower beam flange

Key

|  |  |
| --- | --- |
| *1* | web of the rib stiffener |
| *2* | beam flange, column flange or end-plate |
| *3* | full penetration groove weld with V bevel and reinforcing fillet weld |

Figure E.4 — DC3 details of the rib stiffeners welds to either beam flange, column flange or endplate

Key

|  |  |
| --- | --- |
| *1* | column web |
| *2* | column flange |
| *3* | supplementary plate |
| *4* | full penetration groove weld |
| *5* | reinforcing fillet weld |
| *6* | plug welds connecting slender supplementary plates to column web (see G.3.3.3(5)) |

Figure E.5 — DC3 details of welds of supplementary plates to column web: (a) tolerances of supplementary plate onto column web; (b) details of welds of supplementary plate onto column web; (c) supplementary plate spaced from column web; (d) top view

Key

|  |  |
| --- | --- |
| 1 | continuity plate |
| 2 | column flange |
| 3 | full penetration groove weld with V bevel |
| 4 | fillet weld |

Figure E.6 — DC3 details of welds between continuity plates and column flange: (a) detail for the case compliant with G.3.3.3(2)d); (b) detail for the case compliant with G.3.3.3(3)c)

Key

|  |  |
| --- | --- |
| 1 | continuity plate |
| 2 | column |
| 3 | corner clip |

Figure E.7 — DC3 details of corner clip for continuity plates: (a) corner clip *c*c; (b) details of corner clip of continuity plates onto hot-rolled H column; (c) details of corner clip of continuity plates onto built-up H column

Key

|  |  |
| --- | --- |
| A | beam flange |
| B | column flange |
| C | diametre of drilled hole in the range [18mm-20mm] |
| D | beam web |
| E | backing plate (to be removed after welding) |

Figure E.8 — DC3 details of weld access hole for welded beam-to-column joints

Key

|  |  |
| --- | --- |
| 1 | end-plate or column flange |
| 2 | beam flange |
| 3 | full penetration groove weld |
| 4 | reinforcing fillet weld |
| 5 | access landings |
| 6 | rib stiffener |
| 6a | web of the haunch |
| 6b | flange of the haunch |
| 7 | full penetration groove weld with reinforcing fillet |
| 8 | fillet weld |

Figure E.9 — DC3 details of the stiffener landings at the beam flange and at the end of plate/column flange: (a) detail for haunch stiffener; (b) detail for rib stiffener

* + - 1. Joints with haunched stiffeners

1. Beam-to-column joints may be reinforced using a haunch below the bottom flange of the beam to provide a full-strength and rigid connection, with strong or balanced column web panel. Either fully welded or bolted connection may be used, see Figure E.10.
2. A bolted connection should include an extended end plate with not less than six rows of high-strength pre-loadable bolts with two bolts per row (see Figures E.10 c, d). Three bolt rows should be used to resist the tensile force of sagging moment (see Figures E.10 c) and the other three rows the tensile force of the hogging moment (see Figures E.10 d).
3. Both welded and bolted haunched joints may be used.
4. The action effect at the connection should be calculated with Formula (E.1). The design resistance of the connection should be calculated with prEN 1993-1-8 taking into account the additional assumptions in (5).
5. In the case of bolted connection, the resistance of the connection should be calculated considering a) to c):
6. the active bolt rows in tension are those above the axis of the beam under hogging moment and those below the axis under sagging moment;
7. the centre of compression is located at the centroid of the upper beam flange under sagging moment (see Figure E.10 c) and at 0,5 times the depth of the stiffener *h*h under hogging moment (see Figure E.10 d);
8. conditions given in E.3.3.1(5) and E.3.3.1(8) should be satisfied.
9. The clear span-to-depth ratio (i.e. the distance between the assumed location of plastic hinges divided by the beam depth) should be not less than 7 for MRFs and 5 for dual frames.
10. E.3.3.1(4) should be satisfied for beams. Steel grade from S235 to S355 should be used for the beams and the beam cross sections should be class 1 according to EN 1993-1-1:2022, 7.5.2, in DC3. In DC2 the beam cross sections may be class 1 or 2 according to EN 1993-1-1:2022, 7.5.2. For all DC, the beam depth should be in the range of 240 mm to 770 mm.
11. E.3.3.1(6) should be satisfied for columns. Steel grade from S235 to S690 should be used for the columns and their cross sections should be class 1 according to EN 1993-1-1:2022, 7.5.2. In DC2 the column cross sections may be class 1 or 2 according to EN 1993-1-1:2022, 7.5.2. For all DC, the column depth should be in the range of 200 mm to 716 mm.
12. The thickness of stiffeners transverse to columns and of beam stiffeners should not be smaller than the thickness of the connected beam flange, as shown in Figure E.10. These stiffeners may be considered as secondary, and they may be connected with fillet welds along their perimeter. The thickness of these welds should comply with E.3.3.3(4).
13. If supplementary web plates are necessary, they should satisfy E.3.3.2.
14. Continuity plates should comply with E.3.3.2(5) and E.3.3.2(6). Inner continuity plates, like (F) in Figure E.10 b, may be used in bolted joints to increase the bending resistance of the column flange at the inner bolt-rows.
15. The haunch angle measured between the bottom flange of the beam and the flange of the haunch should be not smaller than 30° and not greater than 40°.
16. The thickness of the web of the haunch should satisfy Formula (E.6).

(E.6)

where

|  |  |
| --- | --- |
| *t*w,h | is the thickness of the haunch web; |
| *t*w,b | is the thickness of the beam web; |
| *f*y,h | is the yield strength of the haunch web; |
| *f*y,b | is the yield strength of the beam. |

1. The types of welds to be used for haunched beam-to-column joints should satisfy E.3.3.3, Figure E.9 a and a) to e):
2. The resistance of all welds should not be smaller than the resistance of the welded parts.
3. The web of the haunch should be welded with two fillet welds (one on each side of the web) with a throat depth not smaller than 0,8 times the plate thickness.
4. Critical welds, i.e. the welds of the top beam flange, of the haunch flange, of the supplementary web plate to column flange should be full-penetration groove welds.
5. In DC3 the top beam flange and haunch flange groove welds should be reinforced with additional fillet welds as specified in Figure E.3.
6. The reinforcing fillet welds at the top beam flange and haunch flange may be omitted for joints in DC2.

Key

|  |  |
| --- | --- |
| A | haunch |
| B | beam |
| C | column |
| D | vertical beam stiffener at the section of the haunch tip |
| E | end-plate |
| F | transverse column stiffeners, also named continuity plates |
| 1 | centre of compression under sagging moment |
| 2 | centre of compression under hogging moment |

Figure E.10 — Configurations of joints with haunch stiffener: (a) welded joint; (b) bolted end-plate joint; (c) bolted joint under sagging moment; (d) bolted joint under hogging moment

1. Weld access holes may not be made at the beam flange to end-plate or column flange connections – see E.3.3.3(6).
2. The end plate thickness should be in the range of 16 mm to 40 mm. The steel grade of the end plates should be S235 to S355. The width of the end plate should not be smaller than the width of the beam flange increased by 30 mm and not greater than the width of column flange.
3. The Z grade of the steel in the end plate in bolted connections and the column flange in welded connections should comply with prEN 1993-1-10, but it should not be smaller than Z15.
4. High strength preloaded bolts according to EN 14399-3 (system HR) and EN 14399-4 (system HV) should be used. Bolts should be fully preloaded according to EN 1090-2. The nominal bolt diametre should be in the range of 14 mm to 40 mm. The width of the beam flanges should be smaller than the width of the column flange minus 30 mm.
5. The bolt holes should comply with prEN 1993-1-8. They should be made by drilling or sub-punching and reaming. Bolt holes should not be oversized.
6. Finger shims may be interposed between the end plate and the column flange at both ends of the beam to adjust the constructional tolerances. Each shim thickness should not be greater than 2 mm and the total thickness of the stacked shims should not be greater than 8 mm at each end of the beam.
   * + 1. Specific requirements for the analysis of frames with haunched joints
7. In linear elastic analysis, frames equipped with haunched joints may be modelled by means of a centre-line structural model. The deformability of the column web panel and of the connection may be ignored.
8. In non-linear analyses the column web panel and the connection should be distinctly modelled as given in a) to d):
9. the column web panel should be simulated with alternative models accounting for its stiffness and resistance if designed as specified in E.3.2.2(4)b) and c);
10. the column web panel should be simulated using rigid segments per its axis with length equal to half dimension of the column web panel if designed as specified in E.3.2.2(4)a);
11. the connection should be simulated as rigid;
12. the monotonic and cyclic behaviour of the beam should be modelled.
    * + 1. Joints with rib stiffeners
13. Beam-to-column joints reinforced using rib stiffener above the upper flange of the beam and below its bottom flange may be either welded (see Figure E.11) or bolted (see Figure E.12). Bolted stiffened end-plate joints may be either full strength or equal strength, with strong or balanced column web panel.
14. The bolted connection should use an extended end plate with different configurations of bolt rows (see Figure E.12) depending on the target design performance of the connection and the beam depth, as specified in a) to e):
15. four bolt row configurations (see Figure E.12 b) for equal strength connection with depth of I beams from 230 mm to 500 mm;
16. four bolt row configurations (see Figure E.12 b) for full strength connection with depth of I beams from 230 mm to 360 mm;
17. six bolt row configurations (see Figure E.12 c) for full strength connection with depth of I beams from 360 mm to 600 mm;
18. six bolt row configurations (see Figure E.12 c) for equal strength connection with depth of I beams from 500 mm to 910 mm;
19. eight bolt row configurations (see Figure E.12 d) for full strength connection with depth of I beams from 600 mm to 910 mm.
20. Full (both welded and bolted) and equal (bolted) strength connections should be used for DC3 structures.
21. The design action effects in full-strength connections (both welded and bolted) should be calculated with Formula (E.1). Their design resistance should be calculated with prEN 1993-1-8 and should comply with the additional assumptions for bolted connections in (6).
22. The design action effects in equal strength bolted connection should be calculated with Formula (E.2). Their design resistance should be calculated with prEN 1993-1-8 and should comply with the additional assumptions in (6).
23. The resistance of bolted connections should be calculated considering the additional assumptions in a) to d):
24. The active bolt rows in tension are those above the axis of the beam under hogging moment and below under sagging moment;
25. The centre of compression is located at the centroid of the beam flange and rib web (see Figure E.13);
26. the conditions given in E.3.3.1(5) and (8) apply;
27. the conditions given in E.3.3.1(8) and (9) apply but only in the case of four or six bolt rows configurations.
28. E.3.3.4(6) should be applied. The clear span-to-depth ratio (i.e. the distance between the assumed location of plastic hinges divided by the beam depth) should not be less than 7 for MRFs and 5 for dual frames.
29. E.3.3.4(7) should be applied, but with a beam depth range from 230 mm to 910 mm.
30. E.3.3.4(8) should be applied, but with a column depth range from 200 mm to 1 000 mm.
31. E.3.3.4(10) should be applied.
32. Continuity plates should comply with E.3.3.2(5) and E.3.3.2(6).
33. The rib angle measured between the flange of the beam and the line intersecting the edge of tip of the rib and the corresponding tip on the end plate or the column flange (see Figure E.14) should be between 30° and 40°. The stiffener should be clipped where it meets the beam flange and the end plate or the column flange to provide clearance between the stiffener and the weld (see Figure E.14).
34. The thickness of the rib should comply with Formula (E.7).

(E.7)

where

|  |  |
| --- | --- |
| *t*w,r | is the thickness of the rib web; |
| *t*w,b | is the thickness of the beam web; |
| *f*y,r | is the yield strength of the rib web; |
| *f*y,b | is the yield strength of the beam. |

1. In full strength connection, the depth-to-thickness ratio of ribs should comply with Formula (E.8).

(E.8)

where *h*r is the depth of the rib web and *E* is the elastic modulus of the steel.

1. The depth of the rib web *h*r should be between 100 mm and 250 mm.
2. The types of welds to be used for rib stiffened beam-to-column joints should comply with those in E.3.3.3, and in Figure E.9 b, considering a) to g):
3. The resistance of all welds should not be smaller than the resistance of the welded parts.
4. The web of the rib should be welded with grove full penetration welds and reinforcing fillet welds.
5. The rib-to-beam and the rib-to-end-plate welds should comply with E.3.3.3(9).
6. The beam web should be connected to either the column flange or the end-plate with fillet welds with a minimum side of 0,8 times the beam web thickness.
7. Critical welds (beam flange and supplementary web plate to column flange and beam flange to end-plate) should be full-penetration groove welds.
8. In DC3, beam flange groove welds should be reinforced with additional fillet welds as in Figure E.3.
9. In DC2, the reinforcing fillet welds at beam flanges may be omitted for full strength connections but should be added for equal strength connections.

Key

|  |  |
| --- | --- |
| A | rib |
| B | beam |
| C | column |
| 1 | single rib |
| 2 | double rib |

Figure E.11 — Configurations of welded joints with rib stiffeners: (a) the assembly; (b) connection with single rib per beam flange; (c) connection with double rib per beam flange

Key

|  |  |
| --- | --- |
| A | rib |
| B | beam |
| C | column |
| D | end-plate |

Figure E.12 — Configurations of bolted end-plate joints with rib stiffener: (a) the assembly; (b) connection with four bolt rows; (c) connection with six bolt rows; (d) connection with eight bolt rows

Key

|  |  |
| --- | --- |
| 1 | rib web |
| 2 | beam flange |
| 3 | centre of compression in the centroid of T-area |

Figure E.13 — Location of centre of compression in connections with rib stiffener: a) welded connection; (b) bolted connection; (c) T-area made of beam flange and rib web

Figure E.14 — Slope and corner clips of rib stiffener

1. End-plate thickness should be in the range of 12 mm to 60 mm. Steel grade S235 to S355 should be used for the end plate. The width of the end plate should be between the width of the beam flange increased by 30 mm and the width of the column flange.
2. For equal strength bolted connections, the end-plate thickness *t*p should comply with Formula E.9.

(E.9)

where

|  |  |
| --- | --- |
| *d*bolt | is the nominal diametre of the bolt; |
| *f*u,b | is the ultimate tensile strength of the bolt; |
| *f*y,p | is the tensile yield strength of the material of the end plate. |

1. E.3.3.4(17) should be applied.
2. E.3.3.4(18) should be applied.
3. E.3.3.4(19) should be applied.
4. The horizontal distance between bolts per row should be between 80 mm and 160 mm.
5. E.3.3.4(20) should be applied.
   * + 1. Specific requirements for the analysis of frames with full strength and equal strength rib stiffened joints
6. In linear elastic analysis, frames equipped with full strength rib stiffened joints may be modelled with centre-line models where the deformability of both column web panel and connection is neglected. The deformability of equal strength bolted connection should be modelled.
7. In non-linear analyses the column web panel and the connection should be distinctly modelled as given in a) to e):
8. the column web panel should be modelled with alternative models accounting for its stiffness and resistance if designed as specified in E.3.2.2(4)b);
9. the column web panel should be modelled using rigid segments per its axis with length equal to half dimension of the column web panel if designed as specified in E.3.2.2(4)a);
10. the connection should be modelled as rigid for full strength joints;
11. the stiffness and resistance of the connection should be modelled for equal strength joints;
12. the monotonic and cyclic behaviour of the beam should be modelled.
    * + 1. Unstiffened Joints
13. Unstiffened beam-to-column joints may be either welded (see Figure E.15a) or bolted (see Figure E.15b). Bolted unstiffened end-plate joints may be either full, equal or partial strength, with strong or balanced or weak column web panel.
14. Bolted connections should make use of extended end plates.
15. The configurations of bolt rows (see Figure E.15) should depend on the target design performance of the connection and on the beam depth, as specified in a) to f):
16. four bolt row configurations (see Figure E.15 c) for full strength connection with depth of I beams between 230 mm to 300 mm;
17. four bolt row configurations (see Figure E.15 c) for equal strength connection with depth of I beams between 230 mm to 500 mm;
18. four bolt row configurations (see Figure E.15 c) for partial strength connection with depth of I beams between 230 mm to 910 mm;
19. six bolt row configurations (see Figure E.15 d) for full strength connection with depth of I beams between 300 mm to 450 mm;
20. six bolt row configurations (see Figure E.15 d) for equal strength connection with depth of I beams between 500 mm to 910 mm;
21. six bolt row configurations (see Figure E.15 d) for partial strength connection with depth of I beams between 230 mm to 910 mm.

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | beam flange-to-column flange weld access hole |
| D | end-plate |

Figure E.15 — Configurations of unstiffened joints: (a) welded connection; (b) bolted connection; (c) bolted connection with four bolt rows; (d) bolted connection with six bolt rows

1. Full (both welded and bolted) or equal (bolted) strength connections should be used for DC3 and DC2 moment and dual frames. For DC3 and DC2 dual frames, partial strength bolted connections with either balanced or weak column web panel may be used.
2. E..3.5(4) should be applied for full strength connections.
3. E.3.3.5(5) should be applied for equal strength connections.
4. The design action effect in a partial strength bolted connection should be evaluated in accordance with Formula E.3 and the design resistance should be calculated in accordance with prEN 1993-1-8 and assumptions specified in (8).
5. The resistance of bolted connection should be calculated considering the additional assumptions a) to c):
6. The active bolt rows in tension are those above the central axis of the beam under hogging moment and below the axis of the beam under sagging moment;
7. the conditions given in E.3.3.1(5) and E.3.3.1(7);
8. for four bolt row configurations, the conditions given in E.3.3.1(8) and E.3.3.1(9).
9. E.3.3.5(7) should be applied.
10. E.3.3.5(8) should be applied.
11. E.3.3.5(9) should be applied.
12. E.3.3.4(10) should be applied.
13. Continuity plates should comply with E.3.3.2(5) and E.3.3.2(6).
14. The types of welds to be used for unstiffened beam-to-column joints should be as in E.3.3.3, considering a) to d):
15. The beam web should be connected with fillet welds with a minimum side of 0,8 times the beam web thickness.
16. Critical welds (beam flange and supplementary web plate to column flange and beam flange to end-plate) should be full-penetration groove welds.
17. In DC3, beam flange groove welds should be reinforced with additional fillet welds as in Figure E.3.
18. In DC2, the reinforcing fillet welds at beam flanges may be omitted for full strength connections but should be added for equal and partial strength connections.
19. For welded connections, the access holes at the beam flange to column flange weld should be as detailed in E.3.3.3(6) and Figure E.8.
20. For bolted connections, there should not be access holes in the web of the beams in accordance with E.3.3.3(7).
21. End-plate thickness should be in the range of 10 mm to 60 mm. The steel grade of end plate should be S235 to S355. The width of the end plate should not be smaller than the width of the beam flange increased by 30 mm and not greater than the width of column flange.
22. For equal and partial strength bolted connections, the end-plate thickness *t*p should comply with Formula E.8.
23. E.3.3.4(17) should be applied.
24. E.3.3.4(18) should be applied.
25. E.3.3.4(19) should be applied.
26. The horizontal distance between bolts per row should not be smaller than 80 mm and not greater than 170 mm.
27. E.3.3.4(20) should be applied.
    * + 1. Specific requirements for the analysis of frames with welded unstiffened joints
28. In linear elastic analysis, frames with welded unstiffened joints may be modelled with centre-line models in which the deformability of the column web panel is modelled. The deformability of both equal and partial strength bolted connection should be modelled.
29. In non-linear analyses the column web panel and the connection should be distinctly modelled as given in a) to e):
30. the column web panel should be modelled with alternative models accounting for its stiffness and resistance if designed as specified in E.3.2.2(b) or in E.3.2.2(c);
31. the column web panel should be modelled using rigid segments per its axis with length equal to half dimension of the column web panel if designed as specified in E.3.2.2(a);
32. the stiffness of the connection should be modelled for full strength bolted joints;
33. the stiffness and resistance of the connection should be modelled for equal and partial strength bolted joints;
34. the monotonic and cyclic behaviour of beams should be modelled.
    * + 1. Joints with reduced beam section
35. Joints with reduced beam section (RBS or “dog-bone joints”) should be designed to avoid plastic deformations in non-yielding connections and to form a plastic hinge in the portion of the beam with reduced section. Both welded and bolted end plate connections may be used (see Figure E.16).
36. The flanges of the beam should be cut as given in a) to d):
37. The distance “*a*” between the column face and the beginning of the RBS should satisfy Formula E.10.

(E.10)

where *b*f is the width of the beam flange.

1. The length “*b*” over which the flange should be reduced should satisfy Formula E.11.

(E.11)

where *d*b is the beam depth.

1. The depth of the flange cut “*c*” on each side should be not greater than 0,25 *b*f. The value given by Formula E.12 may be used.

(E.12)

1. The radius of the flange cut “*r*” should be taken as given by Formula E.13.

(E.13)

1. A grinding of the cut surface of the beam reduced section such that the indentations due to flame cutting are removed should be specified.

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | reduced beam flange |
| D | protected zone |

Figure E.16 — Configurations of joints with reduced beam section: a) welded connection; (b) bolted connection

1. The plastic modulus (*W*pl,RBS) and the plastic moment (*M*pl,Rd,RBS) of the plastic hinge section at the centre of the RBS should be calculated from Formulas (E.14) and (E.15).

(E.14)

(E.15)

where

|  |  |
| --- | --- |
| *W*pl,b | is the plastic modulus of the beam; |
| *f*y,b | is the yield strength of the steel in the beam; |
| *t*f | is the thickness of the beam flange. |

1. The distance *s*h,c between the column face and the intended plastic hinge section at the centre of the RBS should be taken as given by Formulas (E.16) and (E.17).

(E.16)

(E.17)

where *t*p is the thickness of the end plate in bolted connections.

1. Welded and bolted connections with RBS should be designed as full strength in accordance with Formula E.1; their resistance should be calculated in accordance with prEN 1993-1-8 and the assumptions in E.3.3.6(7).
2. Both welded and bolted connections with RBS may be used.
3. E.3.3.4(6) should be applied.
4. Beams should comply with E.3.3.6(10).
5. Lateral bracings should be used to restrain the beam section close to the dog-bone zone against lateral torsional buckling. The stable length *L*st should be taken equal to the length of the protected zone (see Figure E.16), as given by Formula (E.18).

(E.18)

NOTE For joints with RBS, Formula (11.6) is not valid.

1. Columns should comply with E.3.3.5(9).
2. If supplementary web plates are necessary, they should comply with E.3.3.2.
3. Continuity plates should comply with E.3.3.2(5) and E.3.3.2(6).
4. The types of welds to be used should comply with E.3.3.6(14).
5. E.3.3.4(15) should be applied.
6. The end-plate thickness should comply with E.3.3.6(17).
7. E.3.3.4(17) should be applied.
8. E.3.3.4(18) should be applied.
9. E.3.3.4(19) should be applied.
10. E.3.3.5(22) should be applied for bolted connections.
11. E.3.3.4(20) should be applied.
    * + 1. Specific requirements for the analysis of frames with reduced beam sections
12. Centre-line structural model of frames equipped with reduced beam sections may be used in linear elastic analysis. The deformability of both column web panels and connections may be ignored.
13. In non-linear analyses the column web panel and the connection should be distinctly modelled as given in a) to d):
14. the column web panel should be modelled with alternative models accounting for its stiffness and resistance if designed as specified in E.3.2.2(4)b or in E.3.2.2(4)c;
15. the column web panel should be modelled using rigid segments per its axis with length equal to half dimension of the column web panel if designed as specified in E.3.2.2(4)a;
16. the stiffness of the connection should be modelled for full strength bolted joints;
17. the monotonic and cyclic behaviour of RBS should be modelled.
    * + 1. Joints with friction connections
18. Friction connections may be designed either with the slippage surface parallel to the beam flange (horizontal configuration, Figure E.17) or with the slippage surface parallel to the beam web (vertical configuration, Figure E.18).
19. The design resistance of friction connection should be calculated as given in Formula (E.19).

(E.19)

where

|  |  |
| --- | --- |
| *n*b | is the number of bolts; |
| *n*s | is the number of friction surfaces, |
| *F*p,lt | is the long-term value of the bolt preload due to creep phenomena, |
| z | is the lever arm of the connection, see Figures E.17 and E.18, |
| *μ*d | is the 5 % fractile value of the dynamic friction coefficient of the contact surfaces; |
| γMcf | is the partial safety factor. |

NOTE The partial safety factor γMcf can be assumed equal to 1,10 unless differently verified by specific experimental tests.

1. The long-term value of the bolt preload due to creep phenomena should be calculated as given in Formula (E.20).

(E.20)

where *F*p,st is the short term preload of bolts that should be assumed in the range 0,4 to 0,6 times the preload of bolts *F*p,C recommended by prEN 1993-1-8.

1. The design action effects in the elements and the components of joints with friction connections should be calculated with Formula (E.5). Their design resistance should be calculated with prEN 1993‑1‑8 and should comply with the additional assumptions in (5), (6) and (7).
2. The design shear force acting in the web and bolts of upper T-stub connection should be considered equal to the shear force in the beam projected at the column face. The design shear force in the lower L-Stub connections should be considered equal to twice the shear force in the beam projected at the column face.
3. The centre of rotation is located at the centroid of the web of upper T-Stub connection (see Figure E.19).
4. The beam-to-column gap Sg (see Figure E.19) should be greater than 2*t*w,T , being *t*w,T the thickness of the web of the upper T-Stub, and 0,04z in DC3 and 0,03z in DC2.
5. Friction connections should be designed to avoid any slippage at ultimate limit state in non-seismic design situation, where the resistance of friction connection should be calculated as given in Formula (E.19).

(E.21)

where *μ*s,lower is the 5 % fractile value of the static friction coefficient.

1. The 5 % and 95 % fractile values of static and dynamic friction coefficients, strain rate overstrength factor *ω*sr and the stability of the hysteretic behaviour of the connections should be evaluated from cyclic tests on lap-shear sliding connections with clamped interfaces having the same tribological properties of those adopted for the friction connections. The test procedure and the acceptance criteria are detailed in prCEN/TS 1998-1-101.
2. The experimentally measured total overstrength of the friction connection should not be greater than 2,20.
3. The tightening procedure of the bolts of the friction connections should be controlled in order to guarantee the application of the design value of *F*p,s as specified in (3).
4. E.3.3.4(6) should be applied. The clear span-to-depth ratio (i.e. the distance between the assumed location of plastic hinges divided by the beam depth) should not be less than 7 for MRFs and 5 for dual frames.
5. E.3.3.4(7) should be applied, but with a beam depth range from 230 mm to 910 mm.
6. E.3.3.4(8) should be applied, but with a column depth range from 200 mm to 1000 mm.
7. E.3.3.4(10) should be applied.
8. Continuity plates should satisfy E.3.3.2(5) and E.3.3.2(6).
9. The thickness of the web of haunch and the rib should satisfy Formula (E.6).
10. The types of welds to be used for haunch and rib should comply with those in E.3.3.3 and E.3.3.4(14). The types of welds to be used for the upper T-Stub should comply with those in E.3.3.3(3) and Figure E.4.

Key

|  |  |  |  |
| --- | --- | --- | --- |
| A | slotted haunch | F | triangular web stiffeners |
| B | beam | G | continuity plates |
| C | column | H | friction shims |
| D | bolted t-stub | I | pre-loadable bolts |
| E | l-stubs |  |  |

Figure E.17 — Friction connection with horizontal slip surface

Key

|  |  |  |  |
| --- | --- | --- | --- |
| A | slotted haunch | F | triangular web stiffeners |
| B | beam | G | continuity plates |
| C | column | H | friction shims |
| D | bolted t-stub | I | pre-loadable bolts |
| E | slotted l-stubs |  |  |

Figure E.18 — Friction connection with vertical slip surface

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | location of centre of rotation in the web of upper T-Stub connection (in this drawing, the T-stub web is parallel to the beam flange) |

Figure E.19 — Location of centre of rotation and required beam-to-column gap to accommodate the rotation due to the slippage

1. Flange thickness of the upper T-Stub should be in the range of 25 mm to 60 mm. Steel grade not lower than S355 should be used for the flange.
2. E.3.3.4(17) should be applied.
3. High strength preloaded bolts according to EN 14399-3 (system HR) and EN 14399-4 (system HV) should be used to clamp the friction surfaces. Bolts should be tightened according to EN 1090-2, but the applied preload should comply with Formula (E.20). The nominal bolt diameter should be in the range of 16 mm to 24 mm.
4. The bolt holes should comply with prEN 1993-1-8. They should be made by drilling or sub-punching and reaming. The slotted holes should be designed to accommodate slippage consistent with the required rotation.
5. E.3.3.4(20) may be applied.
   * + 1. Specific requirements for the analysis of frames with friction connections
6. In linear elastic analysis, frames equipped with partial strength friction connections may be modelled with centre-line models where the deformability of both column web panel and connection is neglected.
7. In non-linear analyses the column web panel and the connection should be distinctly modelled as given in a) to c):
8. the column web panel should be modelled using rigid segments per its axis with length equal to half dimension of the column web panel;
9. the connection should be modelled as rigid;
10. the monotonic and cyclic behaviour of the friction connection should be modelled.
    * + 1. Other types of joints
11. Beam-to-column joints other than those specified from E.3.3.4 to E.3.3.7 may be used provided that the conditions in a) and b) are satisfied:
12. prequalification tests are performed in accordance with 11.9.4(10);
13. local details comply with E.3.3.1, E.3.3.2 and E.3.3.3.
14. The condition b) in (1) may be waived if experimental evidence proves the ductility and capacity of the joints for the full range of designed beam-column profiles.
15. Prequalified connections may be used in DC2 and DC3 design, within the applicability limits of prequalification and without any request for further qualifying cyclic tests.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for tests for prequalification.

* 1. E.4 Beam-to-column connections allowing rotations in braced frames

1. Beam-to-column connections allowing rotation in accordance with 11.10.5(2)a should have a splice in the beam outside the gusset plate, see Figure E.20.

NOTE The beam-to-beam splice relieves the gusset plate of in-plane moments and guarantees the required rotation capacity.

1. The beam-to-beam splice should be detailed with either bolted double plates (see Figure E.20 a) or bolted flush end plates (see Figure E.20 b) and spaced from the edge of the gusset not less than the width *b*f of the beam.
2. The beam-to-column connections should be detailed as specified in E.3.
3. The gusset plate connections should be detailed as specified in E.5, E.6 and E.7 for concentrically, eccentrically and buckling restrained bracings, respectively.
4. The shear resistance of bolts of flush end-plate beam-to-beam splice should not be smaller than 1,2 times the design bearing resistance of the weaker connected plates.
5. The thickness of flush end-plate beam-to-beam splice should comply with Formula E.9.
6. The welds of flush end-plate splice should comply with E.3.3.6(16).
7. E.3.3.4(18) should be applied.
8. E.3.3.4(19) should be applied.

Figure E.20 — Beam-to-column connections allowing rotations in braced frames: (a) beam-to-beam splice with bolted double plates; (b) beam-to-beam splice with bolted flush end-plates

* 1. Gusset plate connections in concentric bracings
     1. General

1. Gusset plate connections should be designed to accommodate brace buckling under repeated cyclic loading in accordance with 11.10.6(3) and 11.10.6(4).
2. Buckling of the gusset plates should be prevented and restraint-free plastic out-of-plane rotations allowed about either a yield line (either linear or elliptical) in the gusset plate or in a short yielding segment in the axis of the brace.
3. The net areas of gusset plates and bracings should be strengthened by supplementary welded plates to restore the resistance of the original gross areas.
4. The supplementary strengthening plates should be centered as respect to the net areas and their lap length should be at least 25 mm longer than the covered net zones.

NOTE (3) and (4) avoid brittle necking and are necessary because E.5.2 and E.5.3 are based on the assumption that necking does not occur.

* + 1. Gussets with linear clearance

1. Yield lines at each end of the brace should be perpendicular to the brace axis. A 2*t*gp to 4*t*gp clearance (where *t*gp is the gusset plate’s thickness) should be detailed between the end of the brace and the assumed geometric line of the gusset restraints; this line is drawn from the gusset plate point constrained from out-of-plane rotation that is the nearest to the brace end (in Figure E.21 the gusset-plate yield line is denoted E).
2. If the offset of the yield line is within the thickness of a concrete slab, the gusset plate should be isolated from the slab by means of compressible material (e.g. polystyrene, fire caulking, etc.), see Figure E.22 a. Alternatively, the gusset may be detailed to shift the offset of the yield line out of the slab thickness, see Figure E.22 b.

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | diagonal brace |
| D | gusset plate |
| E | linear clearance |

Figure E.21 — Configurations of gusset plate connections for out-of-plane buckling: (a) welded connection; (b) bolted connection

Key

|  |  |
| --- | --- |
| 1 | linear clearance |
| 2 | compressible material |
| 3 | edge stiffener |

Figure E.22 — Slab-to-gusset details: (a) isolated from the slab; (b) restrained by the slab

1. If the free edge of the gusset plate is greater than 4*t*gp beyond the theoretical yield line, an edge stiffener should stabilize the gusset plate and restrain the yield line, see Figure E.22 b. The edge stiffener should terminate outside the gusset-plate hinge.
2. There should be an offset not smaller than 25 mm from each side of the brace to the free edge of the gusset plate, see Figure E.21.
3. The angle between the edge of the gusset plate and the brace axis should be 30 degrees, see Figure E.21.
4. The brace lap length of a welded gusset plate should be 25 mm longer than the specified weld length *Lw*, see Figure E.23 a.
5. To calculate the tension and buckling resistance of the gusset plate against forces transferred by the connected brace, the resisting area of the gusset plate should be calculated with Formula (E.22).

(E.22)

where

|  |  |
| --- | --- |
| *t*gp | is the thickness of the gusset plate; |
| *W*d | is the effective width of the gusset plate, see (8). |

1. The effective width *W*d of the gusset plate at the hinge zone, defined between two 30° lines drawn from either the tips of the welds in welded connection or the first bolts on the gusset that intersect the centreline axis of the last bolt (see Figure E.23) should be calculated using Formula (E.23).

(E.23)

where

|  |  |
| --- | --- |
| *L*w | is the length of the weld connecting the bracing member to the gusset plate; |
| *L*b | is the length of the bolted connection of the bracing member to the gusset plate; |
| *b* | is the distance between the weld lines; |
| *g* | is the distance between the bolt lines. |

Figure E.23 — Effective width of gusset plate: (a) welded brace; (b) bolted brace

1. The buckling resistance of the gusset plate should be calculated from the buckling length using Formula (E.24).

(E.24)

where *L*1, *L*2 and *L*3 are the projections of the effective widths to the edges of the gusset plate, see Figure E.23.

1. The welds of welded gusset plate should be either full penetration groove welds or fillet welds with throat depth not smaller than the thickness of the gusset plate.
   * 1. Gussets with elliptical clearance

NOTE An elliptical clearance is the plastic hinge zone of the gusset around which the brace rotates when out-of-plane buckling occurs. An elliptical clearance is shaped as an elliptical band with a clear 8*t*gp width, see Figure E.24 where the gusset-plate hinge-zone is indicated as (D).

1. The dimensions “*a”* and “*b”* of a gusset with elliptical clearance should be selected so that the imaginary corner of the gusset intersects the centroidal axis of the brace (see Figure E.24) for welded or bolted gussets. The radii *a*’ and *b*’ of the ellipse should be calculated using Formula (E.25).

and (E.25)

1. The welds of welded gusset plates should comply with E.5.2(10).
2. The resistance of the gusset plate should be calculated as in E.5.2(7) and E.5.2(8).
3. The buckling length of the gusset plate should be calculated as in E.5.2(9).

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | diagonal brace |
| D | elliptical clearance |

Figure E.24 — Elliptical clearance of gusset plate: (a) welded brace; (b) bolted brace

* + 1. Gussets detailed for in-plane rotations

1. In a gusset plate detailed with knife plate, see (E) in Figure E.25, the length of the yielding clearance should be equal to 3*t*gp where *t*gp is the thickness of the knife plate, see Figure E.25.

NOTE This arrangement is made to allow in-plane inelastic rotations in the yielding segment between the gusset plate and the diagonal brace. around which the brace rotates when in-plane buckling occurs.

Key

|  |  |
| --- | --- |
| A | beam |
| B | column |
| C | diagonal brace |
| D | gusset plate |
| E | knife plate |

Figure E.25 — Elliptical clearance of gusset plate: (a) welded brace; (b) bolted brace

1. The knife plate may be welded or bolted to the gusset plate.
2. The resistance of the knife plate and of the connection between the knife plate and the gusset plates should be greater than the ultimate axial resistance of the brace calculated using Formula 11.23.
3. The resisting area of the gusset plate should be calculated as in E.5.2(7).
4. The buckling length of the gusset plate should be calculated as in E.5.2(9).
5. The resistance and stability of the gusset should be verified against the expected forces transmitted by the brace in accordance with 11.10.6(2).
   1. Partial strength connections in concentric bracings
      1. General
6. Partial strength connections may be used in concentric bracings only if energy dissipation takes place entirely in these connections while there is no buckling and yielding of braces and other components.
7. Dissipative partial strength connections in bracings should comply with 11.10.3(11).
   * 1. Dissipative pin connection
8. Dissipative pin connections may be placed at one or both ends of the diagonals and may be connected to both strong and weak axis of the column.

NOTE In dissipative pin connection, a pin crosses two external plates connected to the frame columns/beams, and one or two internal plates connected to the brace (see Figure E.26).

1. Dissipative pin connections should be designed in such a way that yielding of the pins in bending will take place before buckling or yielding of the braces and of the adjacent members and parts.
2. The length *a*pl of the pin should comply with Formula (E.26).

(E.26)

where (see Figure E.27):

|  |  |
| --- | --- |
| *h*pin | is the height of the pin; |
| *a*pj | is the clear distance between the inner and outer gusset plates. |
| *b*pin | is the width of the pin; |
| cpj | is the distance between the outer gusset plates |

Figure E.26 — Dissipative pin connections: (a) single plate; (b) double plate

Figure E.27 — Geometrical features of dissipative pin connections: (a) single plate; (b) double plate

1. The thickness of the external plates *t*ext should not be smaller than 0,5*t*int in single plate configuration and equal or greater than *t*int in double plate configurations, where *t*int is the thickness of the internal plate, see Figure E.27.
2. The resistance *R*d of welds or bolts of the dissipative pin connection should satisfy Formula (11.4) where *R*fy in Formula (11.4) should be taken as *R*j,M,Rd or *R*j,M,Rd given in (8), *ω*rm as given in (15) and *ω*sh as given in Table 11.10.
3. In the analysis of frames with dissipative pin connections both the tension and compression diagonals should be taken into account. The pin connection should be modelled as an axial spring, whose elastic stiffness should be calculated from Formula (E.27).

(E.27)

where

|  |  |
| --- | --- |
| *ΕΙ* | is the bending stiffness of the pin; |
| *ρ*pj | is the ratio *a*pj/*c*pj. |

1. The design resistance of the pin connection should be greater than *N*Ed = *N*Ed,G “+” *N*Ed,E.
2. The resistance of the pin connection *R*j,Rd should be the minimum of those due to bending and shear of the pin calculated from Formulae (E.28) and (E.29)

(E.28)

with *V*pl,Rd = *b*pin *h*pin *f*yd /√3 (E.29)

where

|  |  |
| --- | --- |
|  | is the plastic moment of the section of the pin; |
|  | is the plastic shear force of the section of the pin. |

1. The design overstrength of the pin connection **d,M yielding in bending is the minimum value of the ratio *R*j,M,Rd / *N*Ed,j where *N*Ed,j is the axial force in the connection.
2. The design overstrength of the pin connection **d,V yielding in shear is the minimum value of the ratio *R*j,V,Rd / *N*Ed,j where *N*Ed,j is the axial force in the connection.
3. In order to achieve a global behaviour of the structure, it should be checked that the maximum design overstrength ratio over the entire structure does not differ from the minimum value of **d,V and **d,M by more than 25 %.
4. The tensile and buckling resistance of diagonal members should be greater than the ultimate resistance of the pins, which should be taken equal to , where is the material overstrength factor and is given in Table 11.10.
5. Beams and columns connected to braces with INERD-PIN connections should be verified considering the most unfavourable combination of *N*Ed, *M*Ed and *V*Ed, calculated according to Formula (11.3) for DC3 connections and according to Formula (11.2) for DC2 connections.
6. For DC3 connections, when applying Formula (11.3), **d is obtained as the minimal value of **d,M and **d,V and is obtained as given in a) or b):
7. in given in Table 11.10 for dissipative connections in bending when **d = **d,M;
8. in given in Table 11.10 for dissipative connections in shear when **d = **d,V.
9. For the application of (5), (12) and (13), *ω*rm should be taken as given in prEN 1998-1-1:2022, Table 7. When the actual yield strength *f*y,ac,pin of the pin is known, *ω*rm may be assumed equal to 1,0, provided that *f*y,ac,pin is used for the application of Formulae (E.28) and (E.29).
   1. Brace connections in eccentric bracings
      1. General
10. Brace connections should be designed to resist the design action effects in the braces in accordance with 11.11.3(3).
11. The brace connections may be designed as fully rigid or pinned. In fully rigid connections, the braces should be welded with full penetration groove welds to the beam containing the link (see Figure E.28) in accordance with E.3.3.3(6) and Figure E.8. Pinned connections should comply with E.7.2.
    * 1. Gusset plate connections
12. Gusset plate connections may be either welded (see Figure E.29 a) or bolted (see Figure E.29 b). Both types of connections should be designed to restrain buckling of the braces. To this end, the clearance between the beam and the brace should be the minimum compatible with the geometry of the assembly.
13. The joint should be compact, with small free portions of the gussets.
14. The free edges of the gusset plates should be stiffened as shown in Figure E.29. The thickness and the width of the stiffeners of the gusset plates should not be smaller than those of the connected beam flange.
15. The stiffener-to-gusset welds may be either full penetration groove welds or fillet welds with throat depth not smaller than 0,8 times the thickness of the gusset plate.

Key

|  |  |
| --- | --- |
| A | full penetration groove welds in accordance with E.3.3.3(6) |

Figure E.28 — Welded brace connections of EBF

Key

|  |  |
| --- | --- |
| B | stiffeners of the free edge of the gusset |
| C | end-plate connection in bolted gusset plates |

Figure E.29 — Gusset plate connections of diagonal braces of EBF

# Gusset plate connections in buckling restrained bracings

1. Gusset plate connections should be designed to resist the ultimate resistance of the buckling restrained braces in accordance with 11.12.7.
2. Gusset plate connections may be either welded or bolted (see E.7.2). The free clearance between the gusset and the brace should be the minimum compatible with the geometry of the assembly.
3. The brace end connections may be designed as fixed or pinned. In both, the effectiveness of the details should be assessed by experimental tests on the assembly.

NOTE prCEN/TS 1998-1-101 gives a loading protocol and acceptance criteria for such tests.

1. (normative)  
     
   Steel light weight structures
   1. Use of this normative annex
2. This Normative Annex contains provisions additional to 11.14.3 to 11.14.7 for light weight steel buildings.
   1. General
      1. Scope and field of application
3. This annex should be used for DC2 and DC3 design of steel light weight structures made of cold-formed steel frames braced by steel strap braces (i.e. strap braced walls) or by sheathing panels (i.e. shear walls) connected by means of screws to the steel frame. Annex F should be applied in addition to 11.14 and prEN 1993-1-3.
4. Annex F may be also applied to teel light weight structures made of cold-formed steel frames different from those specified in Annex F, however, in those cases the validity and effectiveness of their performance should be demonstrated by means of either experimental evidence, past experimental results available in the literature or refined numerical simulations.
   * 1. Basis of design
5. The primary earthquake resistant structure of lightweight cold-formed steel systems should be made of lightweight cold-formed steel walls made of equally spaced vertical load bearing members screwed at each end to horizontal members and of a bracing system.

NOTE The vertical members are named studs; the horizontal members are named tracks.

1. Tracks may consist of only a web and two flanges.
2. All members may be class 4.
3. Tracks should restrain the studs at their ends.
4. The primary structures of lightweight cold-formed steel systems should resist the in-plane lateral forces of the seismic design situation.

NOTE There are four typical bracing systems: strap braced walls – see Figure F.1; shear walls with steel sheet sheathing – see Figure F.2; shear walls with wood or gypsum sheathing – see Figure F.4. Sheating is connected by means of screws.

1. Strap braced walls should be designed to resist in-plane lateral forces mainly with tension-only steel straps applied diagonally to form a vertical truss (see Figure F.1).

NOTE Straps are flat or coiled sheet steel material used to transfer tension.

1. Shear walls with steel sheet sheathing and shear walls with wood sheathing or gypsum sheathing should be full-height sheathed, the studs should have a lipped channel section having at least a flange width of 40 mm, a web depth of 90 mm and an edge stiffener of 10 mm, whereas tracks should have an unlipped channel section having at least a flange width of 30 mm and a web depth of 90 mm.
   1. Strap braced walls
2. Strap braced walls should be designed to resist in-plane lateral forces mainly with tension-only steel straps applied diagonally, i.e. flat or coiled sheet steel material typically used for transferring loads by tension, to form a vertical truss which forms part of the lateral force-resisting system (see Figure F.1).

Key

|  |  |
| --- | --- |
| 1 | stud |
| 2 | chord stud |
| 3 | track |
| 4 | hold-down |
| 5 | shear anchorage |
| 6 | steel strap brace |
| 7 | connection of strap brace |
| 8 | tension anchorage (with *s*s as the stud spacing) |

Figure F.1 — Strap braced walls: (a) general view; (b) detailed view

1. The non-dissipative components in a) to f) of the strap braced wall should be designed with the overstrength as given in (2) and (3):
2. connections of strap brace;
3. chord studs or other compressed vertical boundary elements at the ends of the wall;
4. tracks;
5. hold-downs and tension anchorages and their connections or other tensioned vertical boundary elements at the ends of wall;
6. shear anchorages;
7. all other components and connections in the wall.
8. In DC2, non-dissipative components in a) to f) of the strap-braced wall should be designed in accordance with 11.14.3(1).
9. In DC3, non-dissipative components in a) to f) of the strap-braced wall should be designed to resist the total action effect *E*Ed from Formula (F.1).

(F.1)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Nfy | is the action effect due to the yielding resistance *N*fy of the gross cross-section of the strap braces based on the nominal yield stress of the material as defined in EN 1993-1-1; |
| rm | is the material overstrength factor (see 11.2.2). For steel grade not included in Table 11.2 rm=1,45; |
| "+" | means combined with + or – sign. |

* 1. Shear walls with steel sheet sheathing

Key

|  |  |
| --- | --- |
| 1 | stud |
| 2 | chord stud |
| 3 | track |
| 4 | hold-down |
| 5 | shear anchorage |
| 6 | steel sheet sheathing |
| 7 | screw at panel edge |
| 8 | screw in the panel field |
| 9 | sheathing joint |
| 10 | tension anchorage (with *s*s stud spacing, *s*f screw spacing in the panel field and *s* screw spacing at panel edge) |

Figure F.2 — Shear walls with steel sheet sheathing

* + 1. Geometrical and mechanical provisions for the components and parts for dissipative DC2 and DC3 structural behaviour

1. a) to k) should be satisfied:
2. thickness of steel sheet sheathing should be in the range from 0,4 mm to 0,9 mm;
3. yield stress of steel sheet sheathing should not be greater than 350 MPa;
4. thickness of steel frame elements should be greater than 0,9 mm;
5. studs and tracks should have nominal yield strength *f*y of 220 MPa for members with thickness smaller than 1,4 mm, and of 350 MPa for members with thickness at least 1,4 mm;
6. screw spacing *s* at panel edges should be in the range from 50 mm to 150 mm and between the edges at most 300 mm;
7. screw spacing *s* in the panel field should be not more than 300 mm;
8. sheathing screw diametre should be in the range from 4,2 mm to 4,8 mm;
9. the end distance from the centre of the screw to the steel sheet sheathing edges should be at least 13 mm;
10. steel sheet panels less than 300 mm wide should not be used;
11. maximum stud spacing *s*s should be 600 mm on centres;
12. all sheathing panel edges should be attached to joists or tracks, sheathing panels should be connected without horizontal joints, and where shear walls need multiple vertical sheathing panels, a single stud should be used at the sheathing joint, unless the connection between the coupled studs is designed for the shear transfer between sheathing panels.
    * 1. Overstrength provisions
13. The non-dissipative components in a) to d) of the shear wall with steel sheathing should be designed with overstrength:
14. chord studs or other compressed vertical boundary elements at the ends of the wall;
15. tracks;
16. hold-downs and tension anchorage and their connections or other tensioned vertical boundary elements at the ends of wall;
17. all other components and connections in the wall.
18. In DC2, non-dissipative components in a) to d) of the shear wall with steel sheathing should be designed in accordance with 11.14.3(1).
19. In DC3, non-dissipative components in a) to d) of the shear wall with steel sheathing should be designed to resist the total action effect *E*Ed from Formula (F.2):

(F.2)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Rc,Rd | is the action effect due to the design resistance *R*c,Rd of the member-to-steel sheathing connections within the effective sheathing strip; |
| "+" | means combined with + or – sign. |

* + 1. Effective strip method

1. The effective strip method may be used in the range of geometrical and mechanical parameters in a) to d):
2. walls should comply with F.4.2;
3. thickness of steel frame elements should be not greater than 1,4 mm;
4. the height-to-length ratio (aspect ratio) should be not smaller than 1,0 and not greater than 2,0.
5. sheathing panels should be without horizontal stiffeners or at most a mid-height stiffener should be used.
6. The sheathing should have sufficient overstrength to allow the development of yielding in the weakest components of the member-to-sheathing connections (see Figure F.3).

Figure F.3 — Effective strip method

1. The effective strip width *w*eff may be calculated using Formulas (F.3) to (F.11).

or (F.3)

(F.4)

(F.5)

(F.6)

(F.7)

(F.8)

(F.9)

(F.10)

(F.11)

where

|  |  |
| --- | --- |
| *h*w | is the wall height; |
| *w*w | is the wall length; |
| *w*w,eff | is the effective wall length; |
| *f*u | is the ultimate tensile strength of steel sheathing (expressed in MPa); |
|  | is the minimum of the ultimate tensile strength of stud or track (expressed in MPa); |
| *t* | is the thickness of steel sheathing (expressed in mm); |
|  | is the minimum of the thickness of stud or track (expressed in mm); |
| s | is the screw spacing at panel edges (expressed in mm). |

1. The design value of the in-plane lateral resistance of wall *R*c,Rd corresponding to the strength of the member-to-sheathing connection may be obtained using Formula (F.12), but should not be greater than *R*y,Rd, the design yielding resistance of the effective sheathing given by Formula (F.16).

(F.12)

where

|  |  |
| --- | --- |
| *F*b,Rd,st, *F*b,Rd,ss, *F*b,Rd,sts | are respectively the design bearing resistance of the sheating to track, sheating to stud and sheating to track and stud connections calculated from Formula (F.13) and (F.14) and should be smaller than *F*v,Rd/1,2, with *F*v,Rd the design shear resistance of screws from Formula (F.15); |

If (F.13)

If (F.14)

where

|  |  |
| --- | --- |
| *f*u,f | is the ultimate tensile strength of stud or track; |
| *t*f | is the thickness of stud or track; |
| *d* | is the nominal diametre of screws; |
| *γ*M2 | is equal to 1,25. |

1. The shear resistance of the screws should be as given by Formula (F.15).

(F.15)

where

|  |  |
| --- | --- |
| *F*v,k | is the characteristic strength of the fastener according to prEN 1993-1-3:2022, 5.3.1.2(2); |
| *γ*M2 | is the partial strength factor given in EN 1993-1-1:2022, 8.1 |

1. The design yielding resistance of the effective sheathing strip should be calculated from Formula (F.16).

(F.16)

where

|  |  |
| --- | --- |
| *f*y,s | is the yield strength of the steel sheathing; |
| *γ*M0 | is in accordance with EN 1993-1-1:2022, 8.1. |

* 1. Shear walls with wood sheathing
     1. Geometrical and mechanical provisions for the components and parts for dissipative DC2 and DC3 structural behaviour

Key

|  |  |
| --- | --- |
| 1 | stud |
| 2 | chord stud |
| 3 | track |
| 4 | hold-down |
| 5 | shear anchorage |
| 6 | wood or gypsum sheathing panel |
| 7 | screw at panel edge |
| 8 | screw in the panel field |
| 9 | sheathing joint |
| 10 | tension anchorage (with *s*s stud spacing, *s*f screw spacing in the panel field and *s* screw spacing at panel edge) |

Figure F.4 — Shear walls with wood or gypsum sheathing

1. a) to j) should be satisfied (see Figure F.4):
2. wood structural panel sheathing should be either Oriented Strand Board (OSB (see EN 300)) with a thickness from 9 mm to 11 mm or Plywood (EN 636) with a thickness from 9,5 mm to 12,5 mm;
3. the thickness of steel frame elements should be greater than 1,1 mm;
4. studs and tracks should have nominal yield strength *f*y of 220 MPa for members with thickness smaller than 1,4 mm and of 350 MPa for those with thickness at least 1,4 mm;
5. screw spacing *s* at panel edges should be in the range from 50 mm to 150 mm and between the edges at most 300 mm;
6. screw spacing *s*f in the panel field should be not more than 300 mm;
7. for members with thickness smaller than 1,4 mm, sheathing screws should have nominal diametre of 4,2 mm and minimum head diametre of 7,2 mm; for those with thickness not smaller than 1,4 mm sheathing screws should have nominal diametre of 4,8 mm and minimum head diametre of 8,5 mm;
8. the distance from the centre of the screw to the wood sheet sheathing edges should be at least 13 mm;
9. wood panels smaller than 300 mm wide should not be used;
10. maximum stud spacing *s*s should be 600 mm on centres;
11. all sheathing panel edges should be attached to joists or tracks, sheathing panels should be connected without horizontal joints, and where shear walls require multiple vertical sheathing panels, a single stud should be used at the sheathing joint, unless the connection between the coupled studs is designed for the shear transfer between sheathing panels.
    * 1. Overstrength
12. The non-dissipative components in a) to e) of the shear wall should be designed with overstrength:
13. the wood sheathing;
14. chord studs or other compressed vertical boundary elements at the ends of the wall;
15. tracks;
16. hold-downs and tension anchorages and their connections or other tensioned vertical boundary elements at the ends of wall;
17. all other components and connections in the wall.
18. In DC2, non-dissipative components in a) to e) of the shear wall with wood sheathing should be designed in accordance with 11.14.3(1).
19. In DC3, non-dissipative components in a) to e) of the shear wall with wood sheathing should be designed to resist the total action effect *E*Ed from Formula (F.17).

(F.17)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Rc,Rd | is the action effect due to the design resistance *R*c,*R*d of the member-to-wood sheathing connections; |
| "+" | means combined with + or – sign. |

NOTE The design resistance *R*c,Rd of the member-to-wood sheathing connections can be calculated with Formula (11.13) in prEN 1995-1-1:2023, 11.2.3.

* 1. Shear walls with gypsum sheathing
     1. Geometrical and mechanical provisions for the components and parts for dissipative DC2 and DC3 structural behaviour

1. a) to j) should be satisfied (see Figure F.4):
2. gypsum panel sheathing (EN 520) should have a thickness of 12,5 mm;
3. the thickness of steel frame elements should be greater than 0,9 mm;
4. studs and tracks should have nominal yield strength *f*y of 220 MPa for members with thickness less than 1,4 mm, and of 350 MPa for those with thickness equal or greater than 1,4 mm;
5. screw spacing *s* at panel edges should be in the range from 50 mm to 150 mm;
6. screw spacing *s* in the panel field should be not more than 300 mm;
7. sheathing screws should have nominal diametre of 4,2 mm and minimum head diametre of 7,2 mm;
8. the end distance from the centre of the screw to the gypsum sheet sheathing edges should be not less than 12,5 mm;
9. Gypsum panels less than 300 mm wide may not be used;
10. maximum stud spacing *s*s should be 600 mm on centre;
11. all sheathing panel edges should be attached to joists or tracks, sheathing panels should be connected without horizontal joints, and where shear walls require multiple vertical sheathing panels, a single stud should be used at the sheathing joint, unless the connection between the coupled studs is designed for the shear transfer between sheathing panels.
    * 1. Overstrength for DC3
12. The components in a) to e) of the shear wall should be designed with overstrength:
13. the gypsum sheathing;
14. chord studs or other compressed vertical boundary elements at the ends of the wall;
15. tracks;
16. hold-downs and tension anchorages and their connections or other tensioned vertical boundary elements at the ends of wall;
17. all other components and connections in the wall in DC2.
18. Non-dissipative components in a) to e) of the shear wall with gypsum sheathing should be designed in accordance with 11.14.3(1).
19. In DC3, non-dissipative components in a) to e) of the shear wall with gypsum sheathing should be designed to resist the total action effect *E*Ed from Formula (F.18).

(F.18)

where

|  |  |
| --- | --- |
| *E*Ed,G | is the action effect due to the non-seismic actions in the seismic design situation; |
| *E*Rc,Rd | is the action effect due to the design resistance *R*c,*R*d of the member-to-gypsum sheathing connections; |
| "+" | means combined with + or – sign. |

NOTE The design resistance *R*c,Rd of the member-to-gypsum sheathing connections should be calculated based on either experimental evidence, or past experimental results available in the literature, or producer certificates, or refined numerical simulations.

1. (normative)  
     
   Design of connections of concrete or composite columns for dissipative composite steel-concrete moment resisting frames
   1. Use of this normative annex
2. This Normative Annex contains additional provisions to Clause 12 for composite buildings.
   1. Scope and field of application
3. This annex should be used for the design, fabrication and quality criteria of full-strength composite connections of dissipative composite steel or steel beams to composite or concrete columns in composite steel-concrete moment resisting frames designed to DC2 and DC3 in Clauses 11 and 12.
   1. Materials
4. All bolts should be of 10.9 class according to prEN 1993-1-8:2021, 3.1.1.
   1. Design provisions
5. The distance of the plastic hinge location from the face of the column, , should be taken in accordance with the provisions for the individual connection according to Annex E, E.3.2.2.
6. For the moment resistance of full-strength composite connections Annex E, E.3.2.2 should be applied.
7. Full-strength composite connections should be designed for the design bending resistance as defined in Annex E, E.3.2.2. In composite steel beams the nominal bending resistance under positive moment should be calculated according to 12.8.6.2 and prEN 1994-1-1:—, 8.3.2.
   1. Joints between steel beams and reinforced concrete or composite columns
      1. General
8. The joint aspect ratio should satisfy: , where:

|  |  |
| --- | --- |
|  | is the depth of the concrete or composite column in the plane of the beam (see Figure G.1); |
|  | is the depth of the steel beam (see Figure G.1). |

1. Width-to-thickness ratios of the flanges and web of the beam should satisfy the requirements of section classification and structure ductility class in Table 11.8.
2. Lateral bracing of beams should be provided according to 12.8.6 and 12.9.
3. The protected zone should consist of the portion of the composite steel or steel beam between the face of the column and 1,5 from it.
4. Beam-to-column panel zone joints should conform to 12.9.6.
5. Face bearing plates as shown in Figure G.1, should be used on all beams that frame into columns and transfer moment demands through the joint.
6. Face bearing plates should be within the beam depth, with a minimum depth, (see Figure G.1); they should be designed according to G.5.9.

NOTE 12.5 indicates in which case consideration of joint deformation in the analysis is required.

Key

|  |  |
| --- | --- |
| A | face bearing plates |
| B | column |
| C | steel or composite beam |
| D | wide face bearing plate |
| E | vertical reinforcement bars |
| F | extended face bearing plates |
| G | structural steel profile |

Figure G.1 — Joint configurations between steel beams and reinforced concrete or composite columns

* + 1. Joint forces

1. A composite joint should be designed for the interaction of bending, shear and axial forces transferred to it (Figure G.2a) by adjacent members in the seismic design situation. The forces shown in Figure G.2b and Figure G.2c may be used for interior and exterior joints, respectively.
2. In bi-directional composite moment resisting frames, the joint resistance may be checked by independently applying the provisions of G.5 for the maximum joint force demands in each of the two orthogonal directions.
3. In joints with reinforced concrete columns in bi-directional composite steel-concrete moment resisting frames, the confinement by steel beams framing into four sides of the column should be neglected.

Figure G.2 — Member forces acting on the joint and design forces for interior and exterior joint configurations

* + 1. Joint failure modes

1. The joint resistance should be verified for shear failure of the web panel of the steel profile column (see Figure G.3a) according to G.5.6 and vertical bearing failure (see Figure G.3b) according to G.5.5.
2. Vertical reinforcement, as shown in Figure G.3b, should be provided to prevent the bearing failure at locations of high compressive stresses; it should be calculated according to G.5.8.

Key

|  |  |
| --- | --- |
| A | panel shear yielding |
| B | concrete crushing |
| C | gap |
| D | vertical reinforcement |

Figure G.3 — Joint failure modes: (a) web panel shear and (b) vertical bearing

* + 1. Effective joint width

1. The effective width, of the joint within the column should be calculated using Formula (G.1).

(G.1)

where

|  |  |
| --- | --- |
|  | is the width of the inner panel as shown in Figure G.4b and Figure G.4c, calculated using Formula (G.2); |

(G.2)

where

|  |  |
| --- | --- |
|  | is the width of the face bearing plate (see Figure G.1); |
|  | is the width of the steel flange of the beam; |
|  | is the width of the outer panel (Figures G.4b and G.4c), calculated from Formula (G.3); |

(G.3)

where

|  |  |
| --- | --- |
|  | is the greater of the steel column or extended face bearing plate width |
|  | is equal to the column depth, where extended face bearing plates are present or *x* = (*h*c+ *h*sc)/2 ( when only the steel column is present; |
|  | is the width of the concrete column (see Figure G.2). |
| *h*c | is the depth of the concrete column |

1. When a steel column is present, should be taken as the minimum of *h*p and 0,25 *h*b. Otherwise, should be taken as 0,25*h*b (see Figure G.1 for the definitions of *h*0, *h*b and *h*p).

Figure G.4 — Joint design forces and internal force resultants

* + 1. Vertical bearing

1. The joint should be verified for vertical bearing under the moment and shear forces, and applied on it (see Figure G.3b).
2. Vertical joint reinforcement (reinforcing bars, rods, steel angles or other elements attached to the steel beam to transfer forces into the concrete column) should be used to prevent vertical bearing failure of composite joints.
3. The bending resistance against vertical bearing should satisfy Formula (G.4).

(G.4)

where

|  |  |
| --- | --- |
|  | = 0,6*f*cd *b*j *h*c is the design concrete bearing resistance (see Figure G.4); |
|  | is the design tensile resistance of the tension vertical joint reinforcement attached directly to the steel beam (see Figure G.4); |
|  | is the design compressive resistance of the compression vertical joint reinforcement attached directly to the steel beam (see Figure G.4); |
|  | is the distance between the tension and compression vertical joint reinforcement (see Figure G.4); |
|  | is the difference in the shear demand between the two beams intersecting the column joint (see Figure G.4). |

1. The vertical reinforcement should be fully anchored to concrete or connected to the steel beam.
2. The contribution of the vertical reinforcement should satisfy Formula (G.5).

(G.5)

* + 1. Joint shear resistance

1. The design shear resistance of a composite joint should be calculated as the sum of the design shear resistance of the steel panel, , defined in (3), the inner concrete compression strut, , defined in (4), and bearing against face bearing plates within the beam depth, and the outer concrete compressive field,  defined in (5) and forming in the outer panel width (see Figure G.4b).

NOTE The three shear resisting mechanisms are shown in Figure G.5.

1. The design shear resistance of a composite joint should satisfy the condition given by Formula (G.6).

(G.6)

where

|  |  |
| --- | --- |
|  | is the centre-to-centre distance between the beam flanges (see Figure G.5a); |
|  | is the depth of the steel web of the beam (see Figure G.5b); |
|  | is defined in G.5.4; |
|  | is defined in Figure G.5; it should be calculated using Formula (G.7); |

(G.7)

NOTE As the joint shear resistance increases while increases, assuming that *j*h = 0,7 *h*c for simplicity is safe sided.

|  |  |
| --- | --- |
|  | is the vertical bearing force acting on a bearing zone (see Figure G.4a); it should be calculated using Formula (G.8); |

(G.8)

where

|  |  |
| --- | --- |
|  | is the design compressive strength of concrete according to prEN 1992-1-1:2021, Table 5.1; |
| ac | should be calculated using Formula (G.9); |

(G.9)

where

|  |  |
| --- | --- |
|  | should be as given by Formula (G.10). |

*K* (G.10)

Figure G.5 — Joint shear mechanisms: (a) steel web panel; (b) concrete compression strut; and (c) concrete compression field

1. The shear resistance, , of the steel panel shown in Figure G.5a should be calculated using Formula (G.11).

(G.11)

where

|  |  |
| --- | --- |
|  | is the nominal yield strength of the steel material of the steel panel; |
|  | is the thickness of the steel panel; |
|  | is defined according to Formula G.7; |
|  | is the material partial factor according to EN 1993-1-1:2022, 8.1. |

1. The design shear resistance, , of the concrete compressive strut (see Figure G.5b) should be calculated using Formula (G.12).

(G.12)

where *b*p is the effective face bearing plate width (see Figure G.1); it should satisfy Formula (G.13).

(G.13)

1. The design shear resistance, , of the concrete compressive field (see Figure G.5c), should be calculated from Formula (G.14).

(G.14)

where

|  |  |
| --- | --- |
|  | is the design shear resistance due to horizontal column ties within the beam depth calculated with Formula (G.7); |
| *b*0 | effective width of outer concrete panel (see Figure G.4). |

(G.15)

where

|  |  |
| --- | --- |
|  | is the design yield strength of the reinforcement bars, according to prEN 1992‑1‑1:2021, 4.3.3, 5.2.4 and Table 5.4; |
|  | is the cross-section area of reinforcing bars in each layer of ties spaced at (see Figure G.6) through the web depth, ; it should not be smaller than 0,004; |
|  | is the effective face bearing plate width (see Figure G.1); it should satisfy Formula (G.13). |

Key

|  |  |
| --- | --- |
| A | holes in the web |
| B | 3 layers minimum (4 leg closed hoops) |

Figure G.6 — Joint shear reinforcement

* + 1. Column horizontal reinforcing bar tie requirement

1. Horizontal ties should be provided in the column within the beam depth and above and below the beam to carry tensile forces developed in the joint. They should be calculated according to G.5.6(5).
2. Perimeter and cross ties should have 135o hooks and may be lap spliced according to 10.11.3.
3. At least three layers of closed rectangular ties (see Figure G.6) should be provided in all joints above and below the beam, as given in a) to c):
4. For a column width, , four 10 mm rebars.
5. For a column width, , four 12 mm rebars.
6. For a column width, , four 16 mm rebars.
7. The layers in (3) should be placed within a distance of above and below the beam.
8. Where the outer compressive field is mobilized to resist the joint shear, the minimum total cross-sectional area, , of the tie reinforcement located within the vertical distance of above and below the beam should be at least equal to , where may be calculated by Formula (G.14).
   * 1. Column vertical reinforcing bars
9. The diametre of vertical column bars passing through the joint should satisfy Formula (G.16).

(G.16)

NOTE For bundled bars, *d*b corresponds to the diametre of a bar of equivalent area to the bundle.

* + 1. Face bearing plate provisions

1. The face bearing plate within the beam depth should resist the horizontal shear force in the concrete strut .
2. The required thickness, , of the face bearing plate should satisfy Formula (G.17).

(G.17)

where

|  |  |
| --- | --- |
|  | is the design yield strength of the bearing plate; |
|  | is the design ultimate strength of the bearing plate; |
|  | is the design yield strength of the web panel; |
|  | may be calculated from Formula (G.6) by assuming |

* + 1. Steel beam flanges

1. The beam flange thickness, should be such that the steel beam resists the vertical bearing force due to joint shear in the steel panel and should satisfy Formula (G.18).

(G.18)

where

|  |  |
| --- | --- |
|  | is the design yield strength of the steel web panel; |
|  | is the design yield strength of the beam flanges; |
|  | is the thickness of the steel web panel. |

1. Additional vertical stiffeners or horizontal bearing plates welded to the flanges may be used to reinforce the beam flanges to increase its transverse bending resistance if the condition in Formula (G.18) is not satisfied.
   * 1. Extended face bearing plates and steel columns
2. Extended face bearing plates and/or steel columns may be used and should be designed to resist a force equal to the joint shear carried by the outer compressive field.
3. The average concrete bearing strength against extended face bearing plates and/or columns should not be greater than and should act over a height above the beam flange not greater than .
4. The extended face bearing plates or the steel column flanges may be assumed capable of resisting transverse bending if their thicknesses, *t*f, satisfies Formula (G.19):

(G.19)

where

|  |  |
| --- | --- |
|  | is the design yield strength of the face bearing plate; |
|  | is the flange width of the extended face bearing plate or the steel column; |
|  | is the flange thickness of the steel beam. |

* 1. Composite joints using diaphragm plates
     1. General

1. Composite joints using diaphragm plates in composite moment resisting frames designed to DC2 and DC3 should be designed according to G.6.

NOTE Figure G.7 shows the main components of a composite joint with a diaphragm plate.

1. Composite joints using diaphragm plates should be designed as full strength, with all plastic deformations localized in the composite or steel beam.
2. One of three types of diaphragm plates should be used: internal (see Figure G.7a), external (see Figure G.7b) or through diaphragm (see Figure G.7c).
3. The diaphragm plates should be welded around the full perimeter of the column cross-section using either fillet welds or full penetration groove welds. In DC3, full penetration groove welds should be used.
4. The thickness of the diaphragm steel plates should be equal to the beam flange thickness and should be made of the same steel grade as that of the steel beam.
5. For connections with internal diaphragm plates, the beam flanges should be directly welded to the steel tube walls. A shear tab steel plate should be used to attach the steel beam web to the steel tub through structural bolts.
6. For connections with external (see Figure G.7b) or through diaphragm plates (see Figure G.7c), the beam flanges should be directly welded to the diaphragm plates with full penetration groove welds. A shear tab plate should be used to attach the steel beam web to the steel tab through structural bolts.
7. Internal or through diaphragm plates should have 25 mm holes for concrete pouring.
   * 1. Joint forces
8. A full-strength composite joint with diaphragm plates should be designed for the plastic flexural resistance of the composite or steel beam at the face of the column.
   * 1. Joint horizontal shear resistance
9. The design shear resistance of a joint should be calculated as the sum of the design shear resistance of the steel tube walls , and the design shear resistance of the inner concrete compression strut, , according to Formula (G.20).

(G.20)

where

|  |  |
| --- | --- |
|  | is the design yield strength of the steel tube walls; |
|  | is the cross-section area portion of the tube walls; |
|  | is the material partial factor according to EN 1993-1-1:2022, 8.1; |
|  | is the cross section area of the concrete infill. |

1. The shear resistance of the joint may be calculated by ignoring the axial load effects.
   * 1. Detailing of welds
2. Full penetration groove welds between I- or H-shape steel beams and diaphragm plates should be detailed according to Annex E, E.3.3.3(2).
   * 1. Shear tab steel plates
3. Shear tab steel plates should be designed for the design shear force effect in the composite steel or steel beam in the seismic design situation.

Key

|  |  |
| --- | --- |
| A | internal diaphragm plate |
| B | concrete-filled tube |
| C | I-shape beam |
| D | external diaphragm plate |
| E | through diaphragm plate |

**Figure G.7** — **Composite joints with: (a) internal; (b) external; and (c) through-diaphragm plates**

* 1. Full-strength composite joints with double-split tee connections in concrete filled tube columns
     1. General

1. Full-strength composite joints with double-split tee connections (see Figure G.8) should consist of concrete-filled tube columns, I- or H-shape steel beams, tapered tee stubs and structural bolts of 10.9 class.

Key

|  |  |
| --- | --- |
| A | tee-stub |
| B | concrete-filled tube |
| C | steel beam |
| D | web panel joint |
| E | structural bolt |
| F | compressive length of tee-stub |
| G | fillet weld of tee-stub to the steel beam flange |

Figure G.8 — Composite joint with double-split tee connections in filled composite columns

1. The tee stub flange should be attached to the concrete filled tube column using slip-critical through structural bolts.
2. The tee stub stem should be either bolted or fillet welded to the beam flanges.
   * 1. Joint shear resistance
3. G.5.3 should be applied.
   * 1. Length and size of welds required to resist the beam flange forces in the joint
4. Welds in a tee-stem welded to the beam flanges should be calculated for the flange forces due to the plastic bending resistance of the connected composite or steel beam.
5. The weld length, , for the tee-stem should be calculated for a given weld thickness, *t*weld, according to Formula (G.21).

(G.21)

where

|  |  |
| --- | --- |
|  | is the design plastic bending resistance of the joint calculated according to G.7.2(1); |
|  | is the depth of the steel beam (see Figure G.8); |
|  | is the nominal yield strength of the fillet weld. |

* + 1. Design of T-stem

1. The tee-stem of tee-stubs should be designed to resist the flange force, , in the tee-stub due to the moment at the face of the column and should be verified against gross section yielding, net section fracture and compression due to flexural yielding.
2. The flange force, , should be calculated according to Formula (G.22).

(G.22)

1. The minimum stem thickness to resist gross section yielding should be calculated according to Formula (G.23).

(G.23)

1. The minimum stem thickness to resist net section fracture of the tee-stem should be calculated according to Formula (G.24).

(G.24)

1. The minimum stem thickness to resist buckling of the tee-stem should be calculated according to Formula (G.25).

(G.25)

where

|  |  |
| --- | --- |
|  | is the width of the tee-stem (see Figure G.8); |
|  | is the design yield strength of the steel material of the tee-stub; |
|  | is the design ultimate tensile strength of the steel material of the tee-stub; |
|  | is the distance from the face of the column to the first row of shear bolts (see Figure G.8); |
|  | is the thickness of the flange of the tee-stub (see Figure G.8). |

* + 1. Required number and size of structural bolts connecting the Tee-stub to the column

1. The minimum diametre of bolts in the tee-stub should be calculated according to Formula (G.26).

(G.26)

where

|  |  |
| --- | --- |
|  | is the number of structural bolts under tensile loading; |
|  | is the design ultimate tensile strength of the structural bolt. |

* + 1. Tee-flange thickness to resist prying forces

1. The distance,, between the tension bolts on either side of the tee-stub stem should be chosen (see Figure G.8b) in order to calculate the geometric parameters of the tee-stub flange to resist prying forces.
2. The minimum thickness, , of the tee-stub flange to resist prying forces should be calculated according to Formula (G.27).

(G.27)

where

|  |  |
| --- | --- |
|  | is the force demand of a single tension bolt and should be calculated as ; |
|  | is the effective width of each tension bolt and should be calculated as |
|  | is the distance from the edge of the tee-stub flange to the edge of the bolt; |
|  | is the distance from the face of the tee-stub stem to the edge of the bolt; |
|  | is the design tensile resistance of a tension bolt (N/bolt). |

1. The critical thickness of the tee-stub flange to eliminate prying action should be calculated according to Formula (G.28).

(G.28)

* + 1. Connection classification

1. The designed connection should be verified if it is within the parameters of a fully-strength or partial strength connection according to prEN 1993-1-8 to be used in composite steel moment resisting frames in DC2 or DC3. The connection classification should be done according to the geometric properties of the tee-stub and of the structural bolts (symbol E in Figure G.8).
   * 1. Additional verifications
2. The maximum force, *N*f, in the tee-stub should be re-calculated based on the actual tee-stub dimensions according to Formula (G.29).

(G.29)

1. In tee-stubs welded or bolted to the steel beam flanges, the resistance of welds or bolts should not be smaller than the maximum force, *N*f, calculated according to (1).
2. The tee-stem resistance according to G.6.5 should be verified against the maximum force, *N*f, calculated according to (1).
3. The flange resistance of the tee-stub should not be smaller than the maximum beam flange force. It should resist the plastic flange mechanism, mixed-mode failure, and tension bolt fracture with no prying action according to prEN 1993-1-8.
4. The shear resistance, , of the panel zone joint calculated according to G.6.3 should not be smaller than the panel zone shear demand on the column from the bending demand at the face of the column according to Formula (G.30).

(G.30)

where

|  |  |
| --- | --- |
|  | is the shear force in the concrete filled column; |
|  | is the flange thickness of the I- or H-shape steel beam. |

1. (informative)  
     
   Seismic design of exposed and embedded steel and composite column base connections
   1. Use of this annex
2. This Informative Annex provides complementary / supplementary guidance to Clauses 11 and 12.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

* 1. Scope and field of application

1. This annex should be used for the design of column base connections retaining moment in steel and/or composite steel-concrete buildings.

NOTE Free to rotate column bases are not covered by this Annex.

1. Column bases retaining moments during a seismic action may either be exposed or embedded type connections to a concrete foundation.
2. Column base connections should be designed as non-dissipative elements for *N*Ed, *V*Ed and *M*Ed as such in DC1 and including the effects of the action magnification factor, , in 11.8.5(1) or 12.8.5(1) for DC2 and the effects of the action magnification factor *ω*rm *ω*sh *Ω*d in 11.8.5(2), or 12.8.5(2) and 12.8.5(3) for DC3, depending on the structural type.
3. Dissipative column base connections may be used in steel and composite steel – concrete moment resisting frames, if their hysteretic behaviour is validated by cyclic testing according to prCEN/TS 1998-1-101.
   1. Materials
4. Steel base plates should be made of structural steels according to 11.3.
5. Anchors should be made according to EN ISO 898-1, EN 20898-2, EN ISO 3506-1 and EN ISO 3506‑2.
6. Grout layers should have a design compressive strength of at least twice the design compressive strength of the foundation concrete. If columns are set on the top of a footing, a 40 mm to 50 mm thick grout layer may be used.
   1. Exposed column base connections
      1. General
7. Exposed column bases should consist of a column, a steel base plate and an anchoring assembly (Figure H.1). Levelling nuts (Figure H.1c) may be used for positioning the column base connection.
8. Exposed column bases may be designed with unstiffened base plates or stiffened base plates.
9. The base plate dimensions should be large enough for the installation of at least four anchors as shown in Figure H.1a and Figure H.1b.
10. A uniform distribution of the resultant compressive bearing resistance per unit length , may be assumed for column base design calculations (see Figure H.1c).
    * 1. Base plate eccentricity and critical eccentricity
11. The base plate eccentricity *e*bpshould be calculated from Formula (H.1).

(H.1)

1. The critical eccentricity *e*crit of an exposed column base should be calculated from Formula (H.2).

(H.2)

where

|  |  |
| --- | --- |
| *L*BP | is the base plate length in the plane of the seismic action (see Figure H.1); |
| *N*Rd,b,max | is the maximum bearing resistance per unit length, which should be calculated as given in Formula (H.3); |

(H.3)

where

|  |  |
| --- | --- |
| *B*BP | is the base plate dimension perpendicular to the plane of the seismic action (Figures H.1a, b); |
|  | is the bearing strength of the concrete footing calculated using Formula (H.4); |

(H.4)

where

|  |  |
| --- | --- |
|  | is the bearing area of the baseplate; |
|  | is the effective concrete area, which is defined as the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area. |

NOTE For a column base plate bearing on a concrete footing far from edges or openings, = 2,0.

1. If *e*bp is not greater than *e*crit, the force combination should be considered with a small moment and the bearing stress between the concrete and the base plate should be calculated using Formula (H.5).

(H.5)

1. If *e*bp is greater than *e*crit, anchors should be used for the moment to be transferred.
   * 1. Base plate with a small moment
2. When *e*bp is not greater than *e*crit, the length, *Y,* of the bearing area should be calculated as given by Formula (H.6).

(H.6)

1. The design bearing resistance per unit length, should satisfy Formula (H.7).

(H.7)

1. The design moment resistance per unit width of the base plate at the bearing interface should be calculated from Formula (H.8).

(H.8)

1. The required base plate thickness, *,* should be calculated as given by Formula (H.9).

(H.9)

where

|  |  |
| --- | --- |
|  | is the nominal value of the yield strength of structural steel of the base plate; |
|  | is the distance shown in Figure H.1a; it should be calculated using Formula (H.10). |

(H.10)

* + 1. Base plate with a large moment

1. When *e*bp is greater than *e*crit, the condition in Formula (H.11) should be satisfied.

(H.11)

where *l* is the distance of the anchor centreline to the base plate centreline (Figure H.1c).

* + 1. Base

1. H.4.4(1) should be satisfied.
2. The base plate bearing length *Y* should be calculated as given by Formula (H.12).

(H.12)

* + 1. Anchor tensile resistance

1. To account for the ratio of the effective tension area of the threaded portion of the anchor to the area of the shank of the anchor, the required anchor yield resistance, , to be considered for the design of the anchor group on one side of the base plate should be calculated as given by Formula (H.13).

(H.13)

1. The anchor tensile strength, should be calculated using Formula (H.14).

(H.14)

where is the specified ultimate tensile strength of the anchor.

1. The design yield resistance of a single anchor should be calculated as given by Formula (H.15).

(H.15)

where is the unthreaded body area of the anchor.

1. The design resistance of anchored headed bars should satisfy prEN 1992-1-1:2021, 11.4.7.

Key

|  |  |  |  |
| --- | --- | --- | --- |
| A | anchor rods | E | shear lug |
| B | column | F | levelling nuts |
| C | base plate | G | concrete footing |
| D | grout layer | H | anchoring point |

Figure H.1 — Schematic representation of exposed column base connection

* + 1. Required base plate thickness

1. The base plate thickness of exposed column base connections should be calculated to limit potential flexural yielding at the bearing and tensile interface of the base plate to the concrete footing in the seismic design situation.
2. H.4.3(3) and (4) should be applied, with replaced by in Formulae (H.8) and (H.9).
3. To prevent flexural yielding at the bearing interface with the concrete footing, the required base plate thickness, , should be calculated from Formula (H.16).

(H.16)

1. To prevent yielding at the tension interface, the required base plate thickness, , should be calculated from Formula (H.17).

(H.17)

where

|  |  |
| --- | --- |
|  | should be calculated using Formula (H.13); |
| *x* | is the distance shown in Figure H.1c; it should be calculated using Formula (H.18). |

(H.18)

* + 1. Design of anchorage for tension

1. The column base concrete elements should be verified against concrete pull out due to the tensile axial load per anchor and concrete crushing due to bearing of the anchor head to the concrete footing as shown in Figure H.2.
2. 10.8.3.1(6), 10.11 and prEN 1992-1-5:2021, 9.4.3 should be satisfied.
   * 1. Design of anchorage for shear
3. Exposed column base connections should transfer shear from column bases into concrete.
4. The shear transfer in anchors may be achieved through friction, bearing and shear or their combination.
5. The most unfavourable design compressive loads, *N*Ed, consistent with the lateral force *V*Ed, should be considered.
6. The shear resistance due to friction should be calculated according to prEN 1992-1-1:2021, 8.2.6.
7. Shear forces may be transferred through bearing by single or multiple shear lugs as shown in Figures H.1c and H.1d. Partial embedment of the column in the concrete foundation may also be used.
8. Shear lugs may be fillet welded to the base plate and may be designed as a cantilever fixed to the steel base plate.
9. When shear lugs are used, grout pockets should be of sufficient size for ease of grout placement. Non shrink grout of flowable consistency should be used.

Key

|  |  |
| --- | --- |
| A | tensile strength in concrete along the surface of stress cone |

Figure H.2 — Concrete breakout cone: (a) single anchor; (b) group of anchors

* 1. Embedded column base connections
     1. General

1. Embedded column base connections (Figure H.3) should provide bending resistance by bearing between the column flanges and the surrounding concrete.
2. Embedded column base connections may be treated as a structural steel coupling beam embedded in a composite reinforced concrete shear wall.
3. For I- or H-shape steel columns, embedded column base connections may develop their full bending resistance with respect to the weak axis provided that the concrete bearing resistance is adequate.
4. The column axial force may be transferred through both the face bearing plate and the base plate.

Key

|  |  |
| --- | --- |
| A | column |
| B | horizontal reinforcement |
| C | line of horizontal reinforcement |
| D | face bearing plates |
| E | concrete footing |
| F | steel base plate |

Figure H.3 — Typical embedded column base connection detail

* + 1. Required shear resistance at the embedded column base

1. The design shear resistance of the column base should satisfy Formula (H.19).

(H.19)

where

|  |  |
| --- | --- |
|  | is the shear force for the design of foundation elements according to prEN 1998‑5:2022, Clause 9; |
|  | is the bending resistance of the column unreduced by the effects of the axial load, at both ends; |
|  | is the column clear storey height above the concrete foundation. |

* + 1. Required column embedment depth

1. The embedment length, , (Figure H.3) should satisfy be calculated based on the required shear resistance of the embedded column base connection as given by Formula (H.20) (units are N, mm).

(H.20)

where

|  |  |
| --- | --- |
|  | is the column flange width under bearing; |
|  | is the width of the concrete foundation. |

1. To guarantee a fixed boundary condition, the embedment length, *L*e, should not be smaller than 2,0 where *d*c is defined at Figure H.1.

NOTE The term reflects the effect of confinement, such that a high ratio between the widths of the concrete foundation to the width of the bearing flange results in a large bearing strength.

* + 1. Required horizontal foundation reinforcement

1. Horizontal reinforcement should be placed in the concrete foundation and over the embedment length, *L*e, with cross-sectional area sufficient to resist in tension the horizontal force(s) given from Formula (H.20).
2. Two-thirds of the horizontal reinforcement should be placed in the top concrete layer.
   * 1. Minimum face bearing plate thickness
3. Face bearing plates (Figure H.3) should be placed on both sides of the column at the face of the foundation and near the end of the embedded region. At a minimum, their thickness should comply with 11.11.2(14).
   * 1. Yielding in the face bearing plates
4. The column axial load should be distributed from the column to the face bearing plates and then to the foundation in direct bearing. The dimension *l* of the face plate cantilever, which is the distance from the anchor centreline to the base plate centreline, should satisfy Formula (H.21).

(H.21)

NOTE The face bearing plate refers to the embedded column bases, not the exposed ones, and *l* refers to Figure H.3, key D.

* + 1. Required transfer reinforcement

1. Transfer reinforcement should be provided at right angles to each embedded flange. The area of transfer reinforcement should be calculated from Formula (H.22).

(H.22)

where is the design value of the yield strength of the transfer steel reinforcement.

1. The sum of the areas of the transfer reinforcement and of the horizontal one according to H.5.4 should not exceed 0,08.
   * 1. Steel base plate
2. In embedded column base connections, column axial load should be transferred to the foundation through a steel base plate.
3. The steel base plate of the embedded column base connection should be designed to remain elastic under bending effects.
4. The steel base plate axial resistance at yielding, should be calculated from Formula (H.23).

(H.23)

where

|  |  |
| --- | --- |
|  | is the design value of the yield strength of the anchors; |
|  | is the total number of the anchors; |
|  | is the area of the anchor; |
|  | is the bearing area of the base plate to the concrete footing. |

1. The steel base plate width should sustain the maximum bearing resistance of the concrete by assuming a uniform stress distribution from the embedded column flange to the steel base plate edge.
2. (normative)  
     
   Design of the slab of steel-concrete composite beams at beam-column joints in moment resisting frames
   1. Use of this normative annex
3. This Normative Annex contains additional provisions to Clause 12 for composite buildings.
   1. Scope and field of application
4. This annex should be used for the design of the slab and of its connection to composite steel-concrete moment resisting frames with rigid full-strength beam-to-column composite connections in DC2 and DC3.
   1. Design of joints at exterior columns under negative moment
5. When there is no façade steel beam and no concrete cantilever edge strip, the moment resistance of exterior joints (Figure I.1a) should be taken as the plastic moment resistance of the steel beam alone (Figure I.1b).

NOTE The composite effect between slab and steel beam cannot be mobilized with the detailing in (1).

1. When there is a concrete cantilever edge strip but no façade steel beam, prEN 1994-1-1 should be applied for the calculation of the moment resistance of the joint (Figure I.1c). The reinforcing bars should be adequately anchored in the cantilever edge strip.
2. When there is a façade steel beam rigidly connected to the column, but no concrete cantilever edge strip, the moment resistance of the joint may include the contribution of the slab reinforcements, provided that (4) to (7) are satisfied (Figure I.1d).
3. Reinforcing bars of the slab should be anchored to the shear connectors of the façade steel beam.
4. The cross-sectional area of reinforcing steel should be such that yielding of the reinforcing steel takes place before failure of the connectors and of the façade beams.
5. The cross-sectional area of reinforcing steel and the connectors should be placed over a width equal to the effective width defined in 12.8.6.2.2 and Table 12.7.
6. The connectors should satisfy the condition in Formula (I.1).

(I.1)

where

|  |  |
| --- | --- |
| *n* | is the number of connectors in the effective width in negative bending; |
|  | is the design resistance of a single connector; |
|  | is the design resistance of the longitudinal bars in the effective width: ; |
|  | is thedesign yield stress of the slab reinforcement. |

1. The façade steel beams and their joints to the column should be verified in bending, shear and torsion under the horizontal force applied at the connectors.

Key

|  |  |
| --- | --- |
| A | main beam |
| B | slab |
| C | exterior column |
| D | façade steel beam |
| E | concrete cantilever edge strip |

Figure I.1 — Exterior beam-to-column joints under negative moment: (a) composite joint elevation; (b) no concrete cantilever edge strip – no façade steel beam; (c) concrete cantilever edge strip – no façade steel beam; (d) no concrete cantilever edge strip – façade steel beam; (e) concrete cantilever edge strip – façade steel beam

1. When there is both a façade steel beam and a concrete cantilever edge strip, the flexural resistance of the joint should include the contributions defined in (2) and (3) (Figure I.1e).
2. The cantilever edge strip should be designed as a reinforced concrete beam supporting horizontal action effects of tension in the slab reinforcement under positive moment and horizontal action effects of compression in the slab under negative moment.

NOTE The reinforcement defined for the cantilever edge strip is specific to the edge strip to column joint zone; it is additional to longitudinal and transverse reinforcement designed for the resistance of the cantilever edge strip in shear and bending under horizontal forces defined in (10).

* 1. Design of joints at exterior columns under positive moment

1. The flexural resistance of exterior joints (Figure I.2a) should not be taken greater than the moment for which reaction forces to the compression in the slab are provided.
2. Three reaction forces may be considered to resist the slab compression forces at beam end, as given in a) to c):
3. on the interior face of the column (Figure I.2b) if (3) and (4) are satisfied; should be calculated using Formula (I.2).

(I.2)

1. by compression struts on the column sides (Figure I.2c) if (5) and (6) are satisfied; *F*Rd2 should be calculated using Formula (I.3).

(I.3)

1. by shear connectors of a façade steel beams rigidly connected to the column (Figure I.2d); it should be calculated using Formula (I.4).

(I.4)

where

|  |  |
| --- | --- |
|  | is the overall depth of the slab, in case of solid slabs, or the thickness of the slab above the ribs of the profiled sheeting for composite slabs; for partially composite beams, should be multiplied by the degree of composite action  |
|  | is the bearing width of the concrete of the slab on the column (see Figure I.2c); |
|  | is the depth of the column steel cross section; |
| *f*cd | is the design value of the concrete compressive strength; |
|  | angle of inclination of the compressive struts (Figure I.2c); |
|  | strut resistance factor according to prEN 1992-1-1:2021, 8.5.2; |
| *n* | number of shear connectors within the effective width in positive bending. |

1. For the concrete close to the column flange should be confined and the area of confining reinforcement should satisfy Formula (I.5).

(I.5)

where

|  |  |
| --- | --- |
| *l* | is the beam span, as defined in prEN 1992-1-1:2021, 7.2.3(5); |
|  | is thedesign yield stress of the transverse reinforcement in the slab. |

1. The reinforcement in (3) should be uniformly distributed over a length of the beam equal to and the distance of the first bar to the column flange should not exceed 30 mm. may be partly or totally provided by reinforcing bars placed for other purposes.
2. For *F*Rd2, a reaction to the concrete compression strut of width *A*T should be provided on the sides of the column facing the struts.

NOTE Flanges of the column section as shown at Fig. I.2 or shear connectors on the sides of the column can provide the necessary reactions to the compression struts forces.

1. For calculating *,* the area of tension-ties (see Figure I.2c) should satisfy Formula (I.6).

(I.6)

Key

|  |  |
| --- | --- |
| A | main beam |
| B | slab |
| C | exterior column |
| D | façade steel beam |
| E | concrete cantilever edge strip |
| F | additional device fixed to the column for bearing |

Figure I.2 — Exterior beam-to-column joints under positive moment and possible transfer of slab forces: (a) composite joint elevation; (b) mechanism 1 – slab bearing on column flange (G) or additional device (H); (c) mechanism 2 – slab extending up to the column outside face or beyond as a concrete cantilever edge is required; (d) mechanism 3 – façade steel beam rigidly connected to the column is required

1. The area in (5) should be distributed over a length of beam equal to and the bars should be fully anchored. The length *l*t of the ties should satisfy Formula (I.7).

(I.7)

where is the anchorage length of these bars in accordance with prEN 1992-1-1:2021, 11.4.

1. The flexural resistance of the joint should be derived from the sum of the reaction forces effectively available at the node.
2. The façade steel beams and their joints to the column should be verified in bending, shear and torsion under the horizontal force *F*Rd3 applied at the connectors.
3. To develop the full plastic moment of resistance of a composite beam, Formula (I.8) should be satisfied.

(I.8)

where is the effective width in positive bending as defined in 12.8.6.2.2 and Table 12.7.

* 1. Interior columns

1. The action effect in the slab due to the bending on opposite sides of interior columns (Figure I.3a) should be calculated as the sum of the tension force in the reinforcing rebars on the negative moment side and of the compression force in the concrete on the positive moment side using Formula (I.9).

(I.9)

where

|  |  |
| --- | --- |
|  | is the area of bars in the effective width; |
|  | is the effective width in positive bending given in 12.8.6.2.2 and Table 12.7. |

1. The reaction forces *F*Rd1, *F*Rd2 and FRd3 defined in I.4(2) may be used to provide resistance to the slab compression forces on the positive moment side and to the tension force of the slab reinforcement on the negative moment side.
2. The number of connectors, *n*, for calculating *F*Rd3 should be taken as the number within the effective width for negative or positive bending, whichever is greater.
3. The transverse steel beams and their joints to the column should be verified in bending, shear and torsion under the horizontal force *F*Rd3 applied at the connectors.
4. The area *A*T of tension-ties (see Figure I.3c) should comply with I.4(5) and (6).
5. To develop the plastic moment resistance of a composite beam, Formula (I.10) should be verified.

(I.10)

where is the effective width in positive bending specified in 12.8.6.2.2 and Table 12.7.

Key

|  |  |  |  |
| --- | --- | --- | --- |
| A | main beam | E | additional device fixed to the column for bearing |
| B | slab | F | reaction forces |
| C | interior column | G | reaction forces |
| D | transverse beam |  |  |

Figure I.3 — Possible transfer of slab forces at an interior beam-to-column joint under negative moment on one side and positive moment on the other side: (a) interior joint elevation; (b) mechanism 1 – slab bearing on column flange (F) or additional device (G); (c) mechanism 2; (d) mechanism 3

NOTE The reaction forces *F*Rd1, *F*Rd2 or *F*Rd3 resist in part the action effects coming from the slab and its rebars on the *M*+ side and in another part to the tension forces in the rebars of the slab on the *M*- side. These parts are named respectively α*F*Rd1, β *F*Rd2 and γ *F*Rd3 on the *M*+ side and respectively (1-α) *F*Rd1, (1-β) *F*Rd2 and (1-γ) *F*Rd3) on the *M*- side. α, β and γ being qualitative coefficients.

1. (informative)  
     
   Drift limits for eccentrically loaded unreinforced masonry piers
   1. Use of this annex
2. This Informative Annex provides complementary / supplementary guidance to 14.9.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

1. Annex J may be applied only to unreinforced masonry piers and should not be used for reinforced or confined masonry piers.

NOTE This informative annex contains a preliminary approach, to be used until more detailed information is available on how axial loads that act with an eccentricity to the central plane of the pier affect the in-plane deformation capacity of the pier.

1. The bearing length *t*B should not be smaller than half the thickness of the pier or 100 mm, whichever is smaller.

NOTE The bearing length of slabs or roofs on masonry piers is sometimes less than the pier thickness *t*p. EN 1996-1-1 gives information on the effect of such reduced bearing lengths on the strength of masonry elements. Experimental results indicate that not only the strength but also the deformation capacity of unreinforced masonry piers depends on the bearing length of floors or roofs resting on the pier. This Informative Annex gives guidelines on how this effect can be considered in design.

* 1. Verification for in-plane actions

1. If a displacement-based verification procedure is applied and if *t*B is smaller than 0,75 times the thickness *t*p of the pier, the effect of the bearing length on the deformation capacity may be considered. The reduced deformation capacity **u2,red may be calculated as given by Formula (J.1).

(J.1)

where **y is the deformation capacity of the pier at the elastic limit.

1. If a force-based verification is applied and if all piers with *t*B < 0,75 *t* contribute together 50 % or more to the storey shear resistance, the *q*D factor may be reduced by 25 %.
2. (informative)  
     
   Simplified evaluation of drift demands on infilled frames
   1. Use of this annex
3. This Informative Annex provides complementary / supplementary guidance to 7.4.2 (Design of frames with interacting infills) and 7.4.2.2 (Analysis with a model of the bare frame only).

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

* 1. Scope and field of application

1. Annex K is applicable to regular RC structures with interacting unreinforced masonry infills regularly distributed in elevation.

NOTE The displacements of an infilled frame are smaller than those of a bare frame; the difference between these two displacements increases with the ratio of infill stiffness to bare frame stiffness.

1. Annex K may be used for the calculation of drift demand in an infilled frame based on the drift calculated on the bare frame.
2. Annex K should be applied for the two principal seismic resistant directions.
   1. Analysis
3. The stiffness *K*Inf,i,j of a masonry infill in bay *i* of storey *j* may be taken as given by Formula (K.1):

(K.1)

where

|  |  |
| --- | --- |
| *V*p,Rd,i,j | is the shear strength of the masonry infill from Formula (7.10) in 7.4.3.7; |
| *θ*p,i,j | is the drift limit at DL of the masonry infill taken from Table 7.1; |
| *h*j | is the height of storey *j*. |

NOTE The drift limit at DL corresponds to the peak strength of the masonry infill.

1. In (1), for masonry infill panels made of other materials than those of Table 7.1 and for masonry panels with openings, respectively 7.4.3.1(6) and 7.4.3.1(7) should be applied.
2. If different types of infills and if infills with openings are present in storey *j*, an equivalent drift capacity *θ*m,j for the whole set of bays may be calculated using Formula (K.2) and the stiffness *K*Inf,i,j of a masonry infill in bay *i* of storey *j* may be taken as given by Formula (K.3).

(K.2)

where *n*b,jis the number of bays in storey *j*.

(K.3)

1. The stiffness *K*Inf,j due to all infills in storey *j* may be calculated using Formula (K.4).

(K.4)

1. The stiffness *K*Fr,j of storey *j* of the bare frame may be calculated using Formula (K.5).

(K.5)

where

|  |  |
| --- | --- |
| *d*e,r,j | is the interstorey average elastic drift, calculated as the difference of the average lateral elastic displacements *d*e,j+1 and *d*e,j at the top and bottom of storey *j*; |
| *V*tot,j | is the total storey shear at storey *j* in the seismic design situation. |

1. The stiffness parameter *C*jwhich characterizes the relative stiffness due to the infills and due to the bare frame at the *j*-th storey may calculated using Formula (K.6).

(K.6)

1. The drift at the ground storey of the infilled structure should be taken as equal to the one calculated for the bare structure and *C*1taken equal to zero.
2. The interstorey drift demand *θ*Ed,jat storey *j* of the infilled frame may be calculated with Formula (K.7) or with Figure K.1 for the case of *θ*mj = 0,003.

(K.7)

where

|  |  |
| --- | --- |
| *θ*e, j | is the elastic drift of the *j-th* storey of the bare frame calculated with Formula (K.8); |

(K.8)

|  |  |
| --- | --- |
| *θ*m, j | is the equivalent drift capacity calculated with Formula (K.2); |
| *θ*C, j | is equal to 0,4 *θ*m, j. |

Figure K.1 — Definition of the bi-linear curve for different values of *C*j

1. (normative)  
     
   Load-deformation relationships of dissipative timber components and resistances of non-dissipative timber components for non-linear analyses
   1. Use of this normative annex
2. This Normative Annex contains additional provisions to 13.5 and 13.6 for timber buildings.
   1. Scope and field of application
3. This annex provides rules for the determination of the force-deformation relationships of dissipative timber components and the resistances of non-dissipative timber components for use in non-linear analysis in accordance with prEN 1998-1-1:2022, 6.7.2 and 6.7.3.
   1. Force-deformation relationships of dissipative timber components for non-linear analysis
4. Reference to the experimental data provided by the producers should be made for the dissipative zones (connections, 2D- or 3D-connectors, joints or subassemblies) of timber structures.

Key

|  |  |
| --- | --- |
| *Y* | yielding point |
| *SD* | Significant Damage point |
| *M* | maximum load point |
| *NC* | Near Collapse point |
| *U* | ultimate load point |

Figure L.1 — Derivation of the trilinear (a) and bilinear (b) load-deformation mean curve of dissipative zones in timber structures from the experimental curves according to EN 12512

1. A trilinear load-deformation mean curve may be used as an approximation to model the behaviour of dissipative zones in timber structures. It may be obtained from the 1st cycle envelope load-deformation curve derived from the cyclic test carried out according to EN 12512 (Figure L.1(a)) as described in a) to f):
2. The first branch should be obtained by connecting the origin with the yield point Y (; *δ*y). The yield deformation *δ*y and the yield load *F*y should be determined according to EN 12512. The slip modulus *K*SLS,v,mean,c should be calculated as given by Formula (L.1).

(L.1)

where *k*mod is the modification factor for duration of load and moisture content according to prEN 1995‑1‑1:2023, 5.1.3.

1. The second branch should be obtained by connecting the yield load point Y with the maximum load point M (; *δ*Fmax), where is the maximum load attained on the 1st cycle envelope load-deformation curve during the test, and *δ*Fmax is the deformation attained at the maximum load .
2. The third branch should be obtained by connecting the maximum load point M with the ultimate load point U (; *δ*u). The ultimate load *F*u is the load attained on the 1st cycle envelope load-deformation curve corresponding to the ultimate deformation *δ*u, which is determined according to EN 12512 assuming a limit value of the strength impairment factor *φ*imp not greater than 0,30 and a limit value of the strength reduction factor due to degradation under cyclic loading *k*deg not smaller than 0,80.
3. When the ultimate deformation *δ*u is smaller than the deformation related to the maximum load *δ*Fmax, a bilinear curve should be used (Figure L.1(b)). The first branch should be defined according to a). The second branch should be obtained by connecting the yield load point Y with the ultimate load point U, which is defined according to c).
4. The SD point should be obtained on the trilinear (or bilinear) curve at the deformation value **SDgiven by Formula (L.2) according to prEN 1998-1-1:2022, 6.7.2(1).

(L.2)

where

|  |  |
| --- | --- |
| **SD | is the deformation resistance at SD limit state; |
| *γ*Rd | is the partial factor on resistance at SD limit state, calculated according to 6.2.3(3), with the total logarithmic standard deviation of the resistance model given in Table L.1. |

1. The NC point should be obtained on the trilinear (or bilinear) curve at the deformation *δ*NCgiven by Formula (L.3) according to prEN 1998-1-1:2022, 6.7.3(2).

(L.3)

where

|  |  |
| --- | --- |
| **NC | is the deformation resistance at NC limit state; |
| *γ*Rd | is the partial factor on resistance at NC limit state. |

NOTE prCEN/TS 1998-1-101 can be used to determine all the aforementioned mechanical properties according to EN 12512.

Table L.1 — Total logarithmic standard deviation **lnR of the   
resistance model for dissipative timber members

|  |  |
| --- | --- |
| **Member type** | ****lnR** |
| Dissipative timber connections which are experimentally characterized according to (1) and (2) | TBD |
| Dissipative timber connections which are analytically characterized according to (3) | TBD |

1. If experimental results are not available for the dissipative zones, the trilinear load-deformation mean curve may be determined following a) to h) (Figure L.2):
2. The mean yield strength (Point Y in Figure L.2) of the dissipative connection under seismic loading may be calculated as given by Formula (L.4).

(L.4)

where

|  |  |
| --- | --- |
| *k*mean | is the ratio between the mean and the characteristic strength in static conditions, from Table L.2; |
| *k*y | is the ratio between the yield strength and the maximum strength, from Table L.2; |
| *F*Rk,d | is the characteristic value of the strength of the dissipative connection in static conditions, according to prEN 1995-1-1:2023, 11.3. |

1. The yield deformation (Point Y in Figure L.2) of the dissipative connection under seismic loading **y may be calculated as given by Formula (L.5).

(L.5)

where *K*SLS,v,mean is the slip modulus of the dissipative connection under service load, according to prEN 1995-1-1:2023, 11.4.7, Table 11.11a.

1. The ultimate strength at the end of the softening branch (Point U in Figure L.2) of the dissipative connection under seismic loading *F*Rm,d,U may be calculated using Formula (L.6).

(L.6)

where *k*deg is the strength reduction factor due to degradation under cyclic loading defined in 13.3.1(4), for which the value in 13.3.1(2) should be used.

1. The deformation at the end of the softening branch (Point U in Figure L.2) of the dissipative connection under seismic loading *δ*umay be calculated as given by Formula (L.7).

(L.7)

where *µ* is the ductility of the dissipative connection under seismic action, which may be taken from Table 13.3 when capacity designed in DC2 or DC3.

1. The maximum strength (Point M in Figure L.2) of the dissipative connection under seismic loading may be calculated from Formula (L.8).

(L.8)

1. The deformation at the maximum load (Point M in Figure L.2) of the dissipative connection under seismic loading *δ*Fmaxmay be calculated using Formula (L.9).

(L.9)

1. The NC point may be obtained on the trilinear curve at the deformation *δ*NCgiven by Formula (L.3).
2. The SD point may be obtained on the trilinear curve at the deformation value **SDgiven by Formula (L.2).

Key

|  |  |
| --- | --- |
| *Y* | yielding point |
| *SD* | Significant Damage point |
| *M* | maximum load point |
| *NC* | Near Collapse point |
| *U* | ultimate load point |

Figure L.2 — Trilinear load-deformation mean curve of dissipative zones in timber structures using analytical Formulas

Table L.2 — Values of the ratio *k*mean between the mean and the characteristic strength of dissipative zones in static conditions, and of the ratio *k*y between the yield and the maximum strength of dissipative zones

|  |  |  |
| --- | --- | --- |
| **Type of dissipative connection/joint/subassembly** | ***k*mean** | ***k*y** |
| High ductility semi-rigid beam-column joints with expanded tube fasteners and Densified Veneer Wood designed in DC3, Carpentry connections in log structures | 1,20 | 0,90 |
| Dissipative connections in CLT structures, Dissipative connections in framed wall structures, Shear walls in braced frame structures with carpentry connections and interacting masonry infills, Semi-rigid beam-column joints designed in DC2, Dowel-type dissipative connections in braced frame structures, Base connections of vertical cantilever structures | 1,35 | 0,90 |

* 1. Resistances of non-dissipative timber components for non-linear analysis

1. The design resistance of non-dissipative timber components for the verification to SD according to prEN 1998-1-1:2022, 6.7.2(2), should be calculated using Formula L.10.

(L.10)

where

|  |  |
| --- | --- |
| *V*Rd,b | is the design value of strength of the non-dissipative component; |
| *V*Rk,b | is the characteristic value of strength of the non-dissipative component, according to prEN 1995-1-1:2023, Clauses 8, 11 and 12; |
| *k*mod | is the modification factor for duration of load and moisture content according to prEN 1995-1-1:2023, 5.1.3, Table 5.1; |
| *k*16 | is the ratio between the 16th and the 5th percentiles of the resistance distribution, given in Table L.3. |

NOTE Formula (L.10) corresponds to 6.2.3(3).

1. The design resistance of non-dissipative timber components for the verification to NC according to prEN 1998-1-1:2022, 6.7.3(3), may be calculated using Formula L.10.

Table L.3 — Values of the ratio between the 16th and the 5th percentile of the strength distribution, *k*16

|  |  |
| --- | --- |
| **Type of product** | ***k*16** |
| Solid timber | 1,25 |
| Glulam, CLT | 1,15 |
| Wood-based panels | 1,15 |
| LVL, GLVL | 1,10 |
| Connections with laterally loaded fasteners with side members of wood and wood-based panels | 1,15 |
| Connections with laterally loaded fasteners with side members of steel | 1,05 |
| Connections with axially loaded fasteners | 1,05 |

1. (informative)  
     
   Material or product properties in EN 1998-1-2
   1. Use of this annex
2. This Informative Annex contains additional provisions to EN 1998-1-2.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this Informative Annex, it can be used.

* 1. Scope and field of application

1. Annex M covers guidance for material and product specifications used in EN 1998-1-2.
2. Table M.1 gives the list of properties of structural elements and antiseismic devices used in EN 1998‑1-2 and the related normative references.

Table M1  List of properties used in EN 1998-1-2 and related normative references

| **Sections in this document** | **Property** | **Referred standard** | **Symbol in the referred standard** | **Symbol in EN 1998-1-2** |
| --- | --- | --- | --- | --- |
| 13.3.1 (2) | The strength reduction factor due to degradation under cyclic loading *k*deg | EN 12512  (revision is needed)      EN 26891  (revision is needed) | - | *k*deg |
| 13.3.1 (3) | The strength impairment factor *φ*imp in a cyclic test | EN 12512  (revision is needed) | - | *φ*imp |
| Table 13.3 | Minimum required ductility of dissipative zones, *μ.* | EN 12512  (revision is needed) | *D* | *μ* |
| 13.3.2 (5) b | Low cycle ductility classes of fasteners S2 and S3 | EN 14592 | Classes S2 and S3 | Classes S2 and S3 |
| 13.3.2 (5) d | The ratio between the 95th percentile of the mechanical properties of fasteners and the characteristic values declared by the manufacturer | EN 14592 | - | *-* |
| 13.5 (2) | The value of slip modulus of the connections measured in cyclic tests, *K*SLS,v,mean,c | EN 12512  (revision is needed) | - | *K*SLS,v,mean,c |
| Annex L (3) a | The yield deformation *δ*y obtained from the 1st cycle envelope load-deformation curve derived from the cyclic test. | EN 12512  (revision is needed) | *V*y | *δ*y |
| Annex L (3) a | The yield load *F*y obtained from the 1st cycle envelope load-deformation curve derived from the cyclic test. | EN 12512  (revision is needed) | *F*y | *F*y |
| Annex L (3) a | The maximum load attained on the 1st cycle envelope load-deformation curve during a cyclic test . | EN 12512  (revision is needed) |  |  |
| Annex L (3) a | The deformation attained at the maximum load. | EN 12512  (revision is needed) | - | *δ*Fmax |
| Annex L (3) c | The ultimate deformation *δ*u derived from the cyclic test. | EN 12512  (revision is needed | *V*u | *δ*u |
| Annex L (3) c | The ultimate load *F*u attained on the 1st cycle envelope load-deformation curve which is determined according to EN 12512 assuming a limit value of the strength impairment factor *φ*imp not greater than 0,30 and a limit value of the strength reduction factor due to degradation under cyclic loading *k*deg not lower than 0,80. | EN 12512  (revision is needed | *F*u | *F*u |

Bibliography

**References contained in recommendations (i.e. “should” clauses)**

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative standards could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

prCEN/TS 1998-1-101, Eurocode 8 — Part 1-101: Characterisation and qualification of structural components for seismic applications by means of cyclic tests

CEN/TS 19103, Eurocode 5 — Design of Timber Structures — Structural design of timber-concrete composite structures — Common rules and rules for buildings

EN 300, Oriented Strand Boards (OSB) — Definitions, classification and specifications

EN 312 (all parts), Particleboards — Specifications

EN 622 (all parts), Fibreboards — Specifications

EN 636, Plywood — Specifications

EN 1090‑2, Execution of steel structures and aluminium structures — Part 2: Technical requirements for steel structures

prEN 1991-1-1:2023, Eurocode 1 — Actions on structures — Part 1-1: Specific weight of materials, self-weight of construction works and imposed loads for buildings

prEN 1991-1-3, Eurocode 1 — Actions on structures — Part 1-3: Snow loads

prEN 1991-1-4, Eurocode 1 — Actions on structures — Part 1-4: Wind actions

prEN 1992-1-1:2021, Eurocode 2 — Design of concrete structures — Part 1-1: General rules and rules for buildings, bridges and civil engineering structures

EN 1992‑4:2018, Eurocode 2 — Design of concrete structures — Part 4: Design of fastenings for use in concrete

EN 1993‑1-1:2022, Eurocode 3 — Design of steel structures — Part 1-1: General rules and rules for buildings

prEN 1993-1-3:2022, Eurocode 3 — Design of steel structures — Part 1-3: Cold-formed members and sheeting

prEN 1993-1-5:2022, Eurocode 3 — Design of steel structures — Part 1-5: Plated structural elements

prEN 1993-1-8:2021, Eurocode 3 — Design of steel structures — Part 1-8: Joints

prEN 1993-1-10:2023, Eurocode 3 — Design of steel structures — Part 1-10: Material toughness and through-thickness properties

prEN 1993-1-14, Eurocode 3 — Design of steel structures — Part 1-14: Design assisted by finite element analysis

prEN 1994-1-1:—,[[1]](#footnote-1) Eurocode 4 — Design of composite steel and concrete structures — Part 1-1: General rules and rules for buildings (under development)

prEN 1995-3, Eurocode 5 — Design of timber structures — Part 3: Execution

EN 1996‑1-1:2022, Eurocode 6 — Design of masonry structures — Part 1-1: General rules for reinforced and unreinforced masonry

prEN 1998-3:2023, Eurocode 8 — Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings and bridges

EN 1999‑1-1:2023, Eurocode 9 — Design of aluminium structures — Part 1-1: General rules

EN 10034, Structural steel I and H sections — Tolerances on shape and dimensions

EN 13353, Solid wood panels (SWP) — Requirements

EN 14279, Laminated Veneer Lumber (LVL) — Definitions, classification and specifications

EN 14374, Timber structures — Structural laminated veneer lumber — Requirements

EN 14592:2022, Timber structures — Dowel type fasteners — Requirements

EN 15129, Anti-seismic devices

EN 15283‑2, Gypsum boards with fibrous reinforcement — Definitions, requirements and test methods — Part 2: Gypsum fibre boards

EN 20898‑2, Mechanical properties of fasteners — Part 2: Nuts with specified proof load values — Coarse thread (ISO 898-2)

EN 61061‑3-1, Non-impregnated densified laminated wood for electrical purposes — Part 3: Specifications for individual materials — Sheet 1: Sheets produced from beech veneer (IEC 61061-3-1)

EN ISO 898‑1, Mechanical properties of fasteners made of carbon steel and alloy steel — Part 1: Bolts, screws and studs with specified property classes — Coarse thread and fine pitch thread (ISO 989-1)

EN ISO 3506‑1, Fasteners — Mechanical properties of corrosion-resistant stainless steel fasteners — Part 1: Bolts, screws and studs with specified grades and property classes (ISO 3506-1)

EN ISO 3506‑2, Fasteners — Mechanical properties of corrosion-resistant stainless steel fasteners — Part 2: Nuts with specified grades and property classes

ISO 6892‑1, Metallic materials — Tensile testing — Part 1: Method of test at room temperature

**References contained in permissions (i.e. “may” clauses)**

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

None

**References contained in possibilities (i.e. “can” clauses) and notes**

The following documents are cited informatively in the document, for example in “can” clauses and in notes.

EN ISO 1478, Tapping screws thread

EN ISO 1479, Hexagon head tapping screws

EN ISO 2702, Fasteners — Heat-treated tapping screws — Mechanical and physical properties

EN ISO 7049, Cross recessed pan head tapping screws

ISO 20987, Simplified design for mechanical connections between precast concrete structural elements in buildings

ISO 22502, Simplified design of connections of concrete claddings to concrete structures

1. Under preparation. [↑](#footnote-ref-1)