

Eurocode 4: Design of composite steel and concrete structures —

Part 1-1: General rules and rules for buildings

ICS 91.010.30; 91.080.10; 91.080.40

National foreword

This British Standard is the UK implementation of EN 1994-1-1:2004, incorporating corrigendum April 2009. It supersedes DD ENV 1994-1-1:1994 which is withdrawn.

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

The structural Eurocodes are divided into packages by grouping Eurocodes for each of the main materials: concrete, steel, composite concrete and steel, timber, masonry and aluminium; this is to enable a common date of withdrawal (DOW) for all the relevant parts that are needed for a particular design. The conflicting national standards will be withdrawn at the end of the co-existence period, after all the EN Eurocodes of a package are available.

Following publication of the EN, there is a period allowed for the national calibration during which the National Annex is issued, followed by a co-existence period of a maximum three years. During the co-existence period Member States are encouraged to adapt their national provisions. At the end of this co-existence period, the conflicting parts of national standard(s) will be withdrawn.

In the UK, the primary corresponding national standards are:

- BS 5950-3.1:1990 — *Structural use of steelwork in building. Design in composite construction. Code of practice for design of simple and continuous composite beams.*
- BS 5950-4:1994 — *Structural use of steelwork in building. Code of practice for design of composite slabs with profiled steel sheeting.*

and based on this transition period, these standards will be withdrawn on a date to be announced, but at the latest by March 2010.

The UK participation in its preparation was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/4, Composite structures.

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

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN.

To enable EN 1994-1-1 to be used in the UK, the NDPs will be published in a National Annex, which will be made available by BSI in due course, after public consultation has taken place.

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

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

Amendments/corrigenda issued since publication

Date	Comments
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English version

Eurocode 4: Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings

Eurocode 4: Calcul des structures mixtes acier-béton -
Partie 1-1: Règles générales et règles pour les bâtiments

Eurocode 4: Bemessung und Konstruktion von
Verbundtragwerken aus Stahl und Beton - Teil 1-1:
Allgemeine Bemessungsregeln und Anwendungsregeln für
den Hochbau

This European Standard was approved by CEN on 27 May 2004.

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

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

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



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

Management Centre: rue de Stassart, 36 B-1050 Brussels

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Foreword

This document (EN 1994-1-1:2004), Eurocode 4: Design of composite steel and concrete structures: Part 1-1 General rules and rules for buildings, has been prepared on behalf of Technical Committee CEN/TC 250 "Structural Eurocodes", the Secretariat of which is held by BSI.

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

This document supersedes ENV 1994-1-1:1992.

CEN/TC 250 is responsible for all Structural Eurocodes.

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

Background of the Eurocode programme

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

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

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

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

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

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

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

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

Status and field of application of Eurocodes

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

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

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

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

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

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

- give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;
- serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

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

National Standards implementing Eurocodes

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

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

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

It may also contain

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

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

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

Additional information specific to EN 1994-1-1

EN 1994-1-1 describes the Principles and requirements for safety, serviceability and durability of composite steel and concrete structures, together with specific provisions for buildings. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1994-1-1 is intended to be used, for direct application, together with other Parts of EN 1994, Eurocodes EN 1990 to 1993 and Eurocodes EN 1997 and 1998.

EN 1994-1-1 also serves as a reference document for other CEN TCs concerning structural matters.

EN 1994-1-1 is intended for use by:

- committees drafting other standards for structural design and related product, testing and execution standards;
- clients (e.g. for the formulation of their specific requirements on reliability levels and durability);
- designers and constructors;
- relevant authorities.

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

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1994-1-1 is used as a base document by other CEN/TCs the same values need to be taken.

National annex for EN 1994-1-1

This standard gives values with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1994-1-1 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1994-1-1 through the following clauses:

- 2.4.1.1(1)
- 2.4.1.2(5)P
- 2.4.1.2(6)P
- 2.4.1.2(7)P
- 3.1(4)
- 3.5(2)
- 6.4.3(1)(h)
- 6.6.3.1(1)
- 6.6.3.1(3)
- 6.6.4.1(3)
- 6.8.2(1)
- 6.8.2(2)
- 9.1.1(2)P
- 9.6(2)
- 9.7.3(4), Note 1
- 9.7.3(8), Note 1
- 9.7.3(9)
- B.2.5(1)
- B.3.6(5)

Section 1 General

1.1 Scope

1.1.1 Scope of Eurocode 4

(1) Eurocode 4 applies to the design of composite structures and members for buildings and civil engineering works. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design.

(2) Eurocode 4 is concerned only with requirements for resistance, serviceability, durability and fire resistance of composite structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) Eurocode 4 is intended to be used in conjunction with:

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

ENs, hENs, ETAGs and ETAs for construction products relevant for composite structures

EN 1090 Execution of steel structures and aluminium structures

EN 13670 Execution of concrete structures

EN 1992 Eurocode 2: Design of concrete structures

EN 1993 Eurocode 3: Design of steel structures

EN 1997 Eurocode 7: Geotechnical design

EN 1998 Eurocode 8: Design of structures for earthquake resistance, when composite structures are built in seismic regions.

(4) Eurocode 4 is subdivided in various parts:

Part 1-1: General rules and rules for buildings

Part 1-2: Structural fire design

Part 2: Bridges.

1.1.2 Scope of Part 1-1 of Eurocode 4

(1) Part 1-1 of Eurocode 4 gives a general basis for the design of composite structures together with specific rules for buildings.

(2) The following subjects are dealt with in Part 1-1:

Section 1: General

Section 2: Basis of design

Section 3: Materials

Section 4: Durability

Section 5: Structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Composite joints in frames for buildings

Section 9: Composite slabs with profiled steel sheeting for buildings

1.2 Normative references

The following normative documents contain provisions which, through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

1.2.1 General reference standards

EN 1090-2 ¹	Execution of steel structures and aluminium structures - Technical rules for the execution of steel structures
EN 1990: 2002	Basis of structural design.

1.2.2 Other reference standards

AC1 EN 1992-1-1:2004	Eurocode 2: Design of concrete structures: General rules and rules for buildings
EN 1993-1-1:2005	Eurocode 3: Design of steel structures: General rules and rules for buildings
EN 1993-1-3:2006	Eurocode 3: Design of steel structures: Cold-formed thin gauge members and sheeting
EN 1993-1-5:2006	Eurocode 3: Design of steel structures: Plated structural elements
EN 1993-1-8:2005	Eurocode 3: Design of steel structures: Design of joints
EN 1993-1-9:2005	Eurocode 3: Design of steel structures: Fatigue strength of steel structures
EN 10025-1:2004	Hot-rolled products of structural steels: General delivery conditions
EN 10025-2:2004	Hot-rolled products of structural steels: Technical delivery conditions for non-alloy structural steels
EN 10025-3:2004	Hot-rolled products of structural steels: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
EN 10025-4:2004	Hot-rolled products of structural steels: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels
EN 10025-5:2004 AC1	Hot-rolled products of structural steels: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance

AC1 Footnote deleted **AC1**

EN 10025-6:2004	Hot-rolled products of structural steels: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition
EN 10326:2004	Continuously hot dip coated strip and sheet of structural steels – Technical delivery conditions
EN 10149-2: 1995	Hot-rolled flat products made of high yield strength steels for cold-forming: Delivery conditions for thermomechanically rolled steels
EN 10149-3: 1995	Hot-rolled flat products made of high yield strength steels for cold-forming: Delivery conditions for normalised or normalised rolled steels

1.3 Assumptions

- (1) In addition to the general assumptions of EN 1990 the following assumptions apply:
- those given in clauses 1.3 of EN1992-1-1 and EN1993-1-1.

1.4 Distinction between principles and application rules

- (1) The rules in EN 1990, 1.4 apply.

1.5 Definitions

1.5.1 General

- (1) The terms and definitions given in EN 1990, 1.5, EN 1992-1-1, 1.5 and EN 1993-1-1, 1.5 apply.

1.5.2 Additional terms and definitions used in this Standard

1.5.2.1 Composite member

a structural member with components of concrete and of structural or cold-formed steel, interconnected by shear connection so as to limit the longitudinal slip between concrete and steel and the separation of one component from the other

1.5.2.2 Shear connection

an interconnection between the concrete and steel components of a composite member that has sufficient strength and stiffness to enable the two components to be designed as parts of a single structural member

1.5.2.3 Composite behaviour

behaviour which occurs after the shear connection has become effective due to hardening of concrete

1.5.2.4 Composite beam

a composite member subjected mainly to bending

1.5.2.5 Composite column

a composite member subjected mainly to compression or to compression and bending

1.5.2.6 Composite slab

a slab in which profiled steel sheets are used initially as permanent shuttering and subsequently combine structurally with the hardened concrete and act as tensile reinforcement in the finished floor

1.5.2.7 Composite frame

a framed structure in which some or all of the elements are composite members and most of the remainder are structural steel members

1.5.2.8 Composite joint

a joint between a composite member and another composite, steel or reinforced concrete member, in which reinforcement is taken into account in design for the resistance and the stiffness of the joint

1.5.2.9 Propped structure or member

a structure or member where the weight of concrete elements is applied to the steel elements which are supported in the span, or is carried independently until the concrete elements are able to resist stresses

1.5.2.10 Un-propped structure or member

a structure or member in which the weight of concrete elements is applied to steel elements which are unsupported in the span

1.5.2.11 Un-cracked flexural stiffness

the stiffness $E_a I_1$ of a cross-section of a composite member where I_1 is the second moment of area of the effective equivalent steel section calculated assuming that concrete in tension is un-cracked

1.5.2.12 Cracked flexural stiffness

the stiffness $E_a I_2$ of a cross-section of a composite member where I_2 is the second moment of area of the effective equivalent steel section calculated neglecting concrete in tension but including reinforcement

1.5.2.13 Prestress

the process of applying compressive stresses to the concrete part of a composite member, achieved by tendons or by controlled imposed deformations

1.6 Symbols

For the purpose of this Standard the following symbols apply.

Latin upper case letters

A	Cross-sectional area of the effective composite section neglecting concrete in tension
A_a	Cross-sectional area of the structural steel section
A_b	Cross-sectional area of bottom transverse reinforcement
A_{bh}	Cross-sectional area of bottom transverse reinforcement in a haunch
A_c	Cross-sectional area of concrete
A_{ct}	Cross-sectional area of the tensile zone of the concrete
A_{fc}	Cross-sectional area of the compression flange
A_p	Cross-sectional area of profiled steel sheeting

A_{pe}	Effective cross-sectional area of profiled steel sheeting
A_s	Cross-sectional area of reinforcement
A_{sf}	Cross-sectional area of transverse reinforcement
$A_{s,r}$	Cross-sectional area of reinforcement in row r
A_t	Cross-sectional area of top transverse reinforcement
A_v	Shear area of a structural steel section
A_l	Loaded area under the gusset plate
E_a	Modulus of elasticity of structural steel
$E_{c,eff}$	Effective modulus of elasticity for concrete
E_{cm}	Secant modulus of elasticity of concrete
E_s	Design value of modulus of elasticity of reinforcing steel
$(EI)_{eff}$	Effective flexural stiffness for calculation of relative slenderness
$(EI)_{eff,II}$	Effective flexural stiffness for use in second-order analysis
$(EI)_2$	Cracked flexural stiffness per unit width of the concrete or composite slab
$F_{c,wc,c,Rd}$	Design value of the resistance to transverse compression of the concrete encasement to a column web
F_ℓ	Design longitudinal force per stud
F_t	Design transverse force per stud
F_{ten}	Design tensile force per stud
G_a	Shear modulus of structural steel
G_c	Shear modulus of concrete
I	Second moment of area of the effective composite section neglecting concrete in tension
I_a	Second moment of area of the structural steel section
I_{at}	St. Venant torsion constant of the structural steel section
I_c	Second moment of area of the un-cracked concrete section
I_{ct}	St. Venant torsion constant of the un-cracked concrete encasement
I_s	Second moment of area of the steel reinforcement
I_1	Second moment of area of the effective equivalent steel section assuming that the concrete in tension is un-cracked
I_2	Second moment of area of the effective equivalent steel section neglecting concrete in tension but including reinforcement
$K_e, K_{e,II}$	Correction factors to be used in the design of composite columns
K_{sc}	Stiffness related to the shear connection
K_β	Parameter
K_0	Calibration factor to be used in the design of composite columns
L	Length; span; effective span
L_e	Equivalent span
L_i	Span
L_o	Length of overhang
L_p	Distance from centre of a concentrated load to the nearest support
L_s	Shear span
L_x	Distance from a cross-section to the nearest support
M	Bending moment
M_a	Contribution of the structural steel section to the design plastic resistance moment of the composite section
$M_{a,Ed}$	Design bending moment applied to the structural steel section
$M_{b,Rd}$	Design value of the buckling resistance moment of a composite beam
$M_{c,Ed}$	The part of the design bending moment applied to the composite section
M_{cr}	Elastic critical moment for lateral-torsional buckling of a composite beam

M_{Ed}	Design bending moment
$M_{Ed,i}$	Design bending moment applied to a composite joint i
$M_{Ed,max,f}$	Maximum bending moment or internal force due to fatigue loading
$M_{Ed,min,f}$	Minimum bending moment due to fatigue loading
$M_{el,Rd}$	Design value of the elastic resistance moment of the composite section
$M_{max,Rd}$	Maximum design value of the resistance moment in the presence of a compressive normal force
M_{pa}	Design value of the plastic resistance moment of the effective cross-section of the profiled steel sheeting
M_{perm}	Most adverse bending moment for the characteristic combination
$M_{pl,a,Rd}$	Design value of the plastic resistance moment of the structural steel section
$M_{pl,N,Rd}$	Design value of the plastic resistance moment of the composite section taking into account the compressive normal force
$M_{pl,Rd}$	Design value of the plastic resistance moment of the composite section with full shear connection
$M_{pl,y,Rd}$	Design value of the plastic resistance moment about the y - y axis of the composite section with full shear connection
$M_{pl,z,Rd}$	Design value of the plastic resistance moment about the z - z axis of the composite section with full shear connection
M_{pr}	Reduced plastic resistance moment of the profiled steel sheeting
M_{Rd}	Design value of the resistance moment of a composite section or joint
M_{Rk}	Characteristic value of the resistance moment of a composite section or joint
$M_{y,Ed}$	Design bending moment applied to the composite section about the y - y axis
$M_{z,Ed}$	Design bending moment applied to the composite section about the z - z axis
N	Compressive normal force; number of stress range cycles; number of shear connectors
N_a	Design value of the normal force in the structural steel section of a composite beam
N_c	Design value of the compressive normal force in the concrete flange
$N_{c,f}$	Design value of the compressive normal force in the concrete flange with full shear connection
$N_{c,el}$	Compressive normal force in the concrete flange corresponding to $M_{el,Rd}$
$N_{cr,eff}$	Elastic critical load of a composite column corresponding to an effective flexural stiffness
N_{cr}	Elastic critical normal force
N_{c1}	Design value of normal force calculated for load introduction
N_{Ed}	Design value of the compressive normal force
$N_{G,Ed}$	Design value of the part of the compressive normal force that is permanent
N_p	Design value of the plastic resistance of the profiled steel sheeting to normal force
$N_{pl,a}$	Design value of the plastic resistance of the structural steel section to normal force
$N_{pl,Rd}$	Design value of the plastic resistance of the composite section to compressive normal force
$N_{pl,Rk}$	Characteristic value of the plastic resistance of the composite section to compressive normal force
$N_{pm,Rd}$	Design value of the resistance of the concrete to compressive normal force
N_R	Number of stress-range cycles
N_s	Design value of the plastic resistance of the steel reinforcement to normal force
N_{sd}	Design value of the plastic resistance of the reinforcing steel to tensile normal force
$P_{\ell,Rd}$	Design value of the shear resistance of a single stud connector corresponding to F_{ℓ}
$P_{pb,Rd}$	Design value of the bearing resistance of a stud
P_{Rd}	Design value of the shear resistance of a single connector

P_{Rk}	Characteristic value of the shear resistance of a single connector
$P_{t,Rd}$	Design value of the shear resistance of a single stud connector corresponding to F_t
R_{Ed}	Design value of a support reaction
S_j	Rotational stiffness of a joint
$S_{j,ini}$	Initial rotational stiffness of a joint
$V_{a,Ed}$	Design value of the shear force acting on the structural steel section
$V_{b,Rd}$	Design value of the shear buckling resistance of a steel web
$V_{c,Ed}$	Design value of the shear force acting on the reinforced concrete web encasement
V_{Ed}	Design value of the shear force acting on the composite section
V_{ld}	Design value of the resistance of the end anchorage
$V_{l,Rd}$	Design value of the resistance to shear
$V_{pl,Rd}$	Design value of the plastic resistance of the composite section to vertical shear
$V_{pl,a,Rd}$	Design value of the plastic resistance of the structural steel section to vertical shear
$V_{p,Rd}$	Design value of the resistance of a composite slab to punching shear
V_{Rd}	Design value of the resistance of the composite section to vertical shear
V_t	Support reaction
$V_{v,Rd}$	Design value of the resistance of a composite slab to vertical shear
$V_{wp,c,Rd}$	Design value of the shear resistance of the concrete encasement to a column web panel
W_t	Measured failure load

Latin lower case letters

a	Spacing between parallel beams; diameter or width; distance
b	Width of the flange of a steel section; width of slab
b_b	Width of the bottom of the concrete rib
b_c	Width of the concrete encasement to a steel section
b_{eff}	Total effective width
$b_{eff,1}$	Effective width at mid-span for a span supported at both ends
$b_{eff,2}$	Effective width at an internal support
$b_{eff,c,wc}$	Effective width of the column web in compression
b_{ei}	Effective width of the concrete flange on each side of the web
b_{em}	Effective width of a composite slab
b_f	Width of the flange of a steel section
b_i	Geometric width of the concrete flange on each side of the web
b_m	Width of a composite slab over which a load is distributed
b_p	Length of concentrated line load
b_r	Width of rib of profiled steel sheeting
b_s	Distance between centres of adjacent ribs of profiled steel sheeting
b_0	Distance between the centres of the outstand shear connectors; mean width of a concrete rib (minimum width for re-entrant sheeting profiles); width of haunch
c	Width of the outstand of a steel flange; effective perimeter of reinforcing bar
c_y, c_z	Thickness of concrete cover
d	Clear depth of the web of the structural steel section; diameter of the shank of a stud connector; overall diameter of circular hollow steel section; minimum transverse dimension of a column
d_{do}	Diameter of the weld collar to a stud connector
d_p	Distance between the centroidal axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression
d_s	Distance between the steel reinforcement in tension to the extreme fibre of the composite slab in compression; distance between the longitudinal reinforcement in tension and the centroid of the beam's steel section

e	Eccentricity of loading; distance from the centroidal axis of profiled steel sheeting to the extreme fibre of the composite slab in tension
e_D	Edge distance
e_g	Gap between the reinforcement and the end plate in a composite column
e_p	Distance from the plastic neutral axis of profiled steel sheeting to the extreme fibre of the composite slab in tension
e_s	Distance from the steel reinforcement in tension to the extreme fibre of the composite slab in tension
f	Natural frequency
f_{cd}	Design value of the cylinder compressive strength of concrete
f_{ck}	Characteristic value of the cylinder compressive strength of concrete at 28 days
f_{cm}	Mean value of the measured cylinder compressive strength of concrete
$f_{ct,eff}$	Mean value of the effective tensile strength of the concrete
f_{ctm}	Mean value of the axial tensile strength of concrete
$f_{ct,0}$	Reference strength for concrete in tension
f_{lctm}	Mean value of the axial tensile strength of lightweight concrete
f_{sd}	Design value of the yield strength of reinforcing steel
f_{sk}	Characteristic value of the yield strength of reinforcing steel
f_u	Specified ultimate tensile strength
f_{ut}	Actual ultimate tensile strength in a test specimen
f_y	Nominal value of the yield strength of structural steel
f_{yd}	Design value of the yield strength of structural steel
$f_{yp,d}$	Design value of the yield strength of profiled steel sheeting
f_{ypm}	Mean value of the measured yield strength of profiled steel sheeting
f_1, f_2	Reduction factors for bending moments at supports
h	Overall depth; thickness
h_a	Depth of the structural steel section
h_c	Depth of the concrete encasement to a steel section; thickness of the concrete flange; thickness of concrete above the main flat surface of the top of the ribs of the sheeting
h_f	Thickness of concrete flange; thickness of finishes
h_n	Position of neutral axis
h_p	Overall depth of the profiled steel sheeting excluding embossments
h_s	Depth between the centroids of the flanges of the structural steel section; distance between the longitudinal reinforcement in tension and the centre of compression
h_{sc}	Overall nominal height of a stud connector
h_t	Overall thickness of test specimen
k	Amplification factor for second-order effects; coefficient; empirical factor for design shear resistance
k_c	Coefficient
k_i	Stiffness coefficient
$k_{i,c}$	Addition to the stiffness coefficient k_i due to concrete encasement
k_ℓ	Reduction factor for resistance of a headed stud used with profiled steel sheeting parallel to the beam
k_s	Rotational stiffness; coefficient
k_{sc}	Stiffness of a shear connector
k_{slip}	Stiffness reduction factor due to deformation of the shear connection
$k_{s,r}$	Stiffness coefficient for a row r of longitudinal reinforcement in tension
k_t	Reduction factor for resistance of a headed stud used with profiled steel sheeting transverse to the beam

$k_{wc,c}$	Factor for the effect of longitudinal compressive stress on transverse resistance of a column web
k_ϕ	Parameter
k_1	Flexural stiffness of the cracked concrete or composite slab
k_2	Flexural stiffness of the web
ℓ	Length of the beam in hogging bending adjacent to the joint
l	Length of slab in standard push test
l_{bc}, l_{bs}	Bearing lengths
ℓ_0	Load introduction length
m	Slope of fatigue strength curve; empirical factor for design shear resistance
n	Modular ratio; number of shear connectors
n_f	Number of connectors for full shear connection
n_L	Modular ratio depending on the type of loading
n_r	Number of stud connectors in one rib
n_0	Modular ratio for short-term loading
r	Ratio of end moments
s	Longitudinal spacing centre-to-centre of the stud shear connectors; slip
s_t	Transverse spacing centre-to-centre of the stud shear connectors
t	Age; thickness
t_e	Thickness of end plate
$t_{eff,c}$	Effective length of concrete
t_f	Thickness of a flange of the structural steel section
t_s	Thickness of a stiffener
t_w	Thickness of the web of the structural steel section
t_{wc}	Thickness of the web of the structural steel column section
t_0	Age at loading
v_{Ed}	Design longitudinal shear stress
w_k	Design value of crack width
x_{pl}	Distance between the plastic neutral axis and the extreme fibre of the concrete slab in compression
y	Cross-section axis parallel to the flanges
z	Cross-section axis perpendicular to the flanges; lever arm
z_0	Vertical distance

Greek upper case letters

$\Delta\sigma$	Stress range
$\Delta\sigma_c$	Reference value of the fatigue strength at 2 million cycles
$\Delta\sigma_E$	Equivalent constant amplitude stress range
$\Delta\sigma_{E, glob}$	Equivalent constant amplitude stress range due to global effects
$\Delta\sigma_{E, loc}$	Equivalent constant amplitude stress range due to local effects
$\Delta\sigma_{E, 2}$	Equivalent constant amplitude stress range related to 2 million cycles
$\Delta\sigma_s$	Increase of stress in steel reinforcement due to tension stiffening of concrete
$\Delta\sigma_{s, equ}$	Damage equivalent stress range
$\Delta\tau$	Range of shear stress for fatigue loading
$\Delta\tau_c$	Reference value of the fatigue strength at 2 million cycles
$\Delta\tau_E$	Equivalent constant amplitude stress range
$\Delta\tau_{E, 2}$	Equivalent constant amplitude range of shear stress related to 2 million cycles
$\Delta\tau_R$	Fatigue shear strength

Ψ Coefficient

Greek lower case letters

α	Factor; parameter
α_{cr}	Factor by which the design loads would have to be increased to cause elastic instability
α_M	Coefficient related to bending of a composite column
$\alpha_{M,y}$, $\alpha_{M,z}$	Coefficient related to bending of a composite column about the y-y axis and the z-z axis respectively
α_{st}	Ratio
β	Factor; transformation parameter
β_c , β_i	Parameters
γ_c	Partial factor for concrete
γ_F	Partial factor for actions, also accounting for model uncertainties and dimensional variations
γ_{Ff}	Partial factor for equivalent constant amplitude stress range
γ_M	Partial factor for a material property, also accounting for model uncertainties and dimensional variations
γ_{M0}	Partial factor for structural steel applied to resistance of cross-sections, see EN 1993-1-1, 6.1(1)
γ_{M1}	Partial factor for structural steel applied to resistance of members to instability assessed by member checks, see EN 1993-1-1, 6.1(1)
γ_{Mf}	Partial factor for fatigue strength
$\gamma_{Mf,s}$	Partial factor for fatigue strength of studs in shear
γ_p	Partial factor for pre-stressing action
γ_s	Partial factor for reinforcing steel
γ_v	Partial factor for design shear resistance of a headed stud
γ_{vs}	Partial factor for design shear resistance of a composite slab
δ	Factor; steel contribution ratio; central deflection
δ_{max}	Sagging vertical deflection
δ_s	Deflection of steel sheeting under its own weight plus the weight of wet concrete
$\delta_{s,max}$	Limiting value of δ_s
δ_u	Maximum slip measured in a test at the characteristic load level
δ_{uk}	Characteristic value of slip capacity
ε	$\sqrt{235 / f_y}$, where f_y is in N/mm ²
η	Degree of shear connection; coefficient
η_a , η_{ao}	Factors related to the confinement of concrete
η_c , η_{co} , η_{cl}	Factors related to the confinement of concrete
θ	Angle
λ , λ_v	Damage equivalent factors
λ_{glob} , λ_{loc}	Damage equivalent factors for global effects and local effects, respectively
$\bar{\lambda}$	Relative slenderness
$\bar{\lambda}_{LT}$	Relative slenderness for lateral-torsional buckling
μ	Coefficient of friction; nominal factor
μ_d	Factor related to design for compression and uniaxial bending
μ_{dy} , μ_{dz}	Factor μ_d related to plane of bending

ν	Reduction factor to allow for the effect of longitudinal compression on resistance in shear; parameter related to deformation of the shear connection
ν_a	Poisson's ratio for structural steel
ξ	Parameter related to deformation of the shear connection
ρ	Parameter related to reduced design bending resistance accounting for vertical shear
ρ_s	Parameter; reinforcement ratio
$\sigma_{\text{com,c,Ed}}$	Longitudinal compressive stress in the encasement due to the design normal force
$\sigma_{\text{c,Rd}}$	Local design strength of concrete
σ_{ct}	Extreme fibre tensile stress in the concrete
$\sigma_{\text{max,f}}$	Maximum stress due to fatigue loading
$\sigma_{\text{min,f}}$	Minimum stress due to fatigue loading
$\sigma_{\text{s,max,f}}$	Stress in the reinforcement due to the bending moment $M_{\text{Ed,max,f}}$
$\sigma_{\text{s,min,f}}$	Stress in the reinforcement due to the bending moment $M_{\text{Ed,min,f}}$
σ_s	Stress in the tension reinforcement
$\sigma_{\text{s,max}}$	Stress in the reinforcement due to the bending moment M_{max}
$\sigma_{\text{s,max,0}}$	Stress in the reinforcement due to the bending moment M_{max} , neglecting concrete in tension
$\sigma_{\text{s,0}}$	Stress in the tension reinforcement neglecting tension stiffening of concrete
τ_{Rd}	Design shear strength
τ_u	Value of longitudinal shear strength of a composite slab determined from testing
$\tau_{\text{u,Rd}}$	Design value of longitudinal shear strength of a composite slab
$\tau_{\text{u,Rk}}$	Characteristic value of longitudinal shear strength of a composite slab
ϕ	Diameter (size) of a steel reinforcing bar; damage equivalent impact factor
ϕ^*	Diameter (size) of a steel reinforcing bar
φ	Creep coefficient
$\varphi(t, t_0)$	Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days
χ	Reduction factor for flexural buckling
χ_{LT}	Reduction factor for lateral-torsional buckling
ψ_L	Creep multiplier

Section 2 Basis of design

2.1 Requirements

(1)P The design of composite structures shall be in accordance with the general rules given in EN 1990.

(2)P The supplementary provisions for composite structures given in this Section shall also be applied.

(3) The basic requirements of EN 1990, Section 2 are deemed to be satisfied for composite structures when the following are applied together:

- limit state design in conjunction with the partial factor method in accordance with EN 1990,
- actions in accordance with EN 1991,
- combination of actions in accordance with EN 1990 and
- resistances, durability and serviceability in accordance with this Standard.

2.2 Principles of limit states design

(1)P For composite structures, relevant stages in the sequence of construction shall be considered.

2.3 Basic variables

2.3.1 Actions and environmental influences

(1) Actions to be used in design may be obtained from the relevant parts of EN 1991.

(2)P In verification for steel sheeting as shuttering, account shall be taken of the ponding effect (increased depth of concrete due to the deflection of the sheeting).

2.3.2 Material and product properties

(1) Unless otherwise given by Eurocode 4, actions caused by time-dependent behaviour of concrete should be obtained from EN 1992-1-1.

2.3.3 Classification of actions

(1)P The effects of shrinkage and creep of concrete and non-uniform changes of temperature result in internal forces in cross sections, and curvatures and longitudinal strains in members; the effects that occur in statically determinate structures, and in statically indeterminate structures when compatibility of the deformations is not considered, shall be classified as primary effects.

(2)P In statically indeterminate structures the primary effects of shrinkage, creep and temperature are associated with additional action effects, such that the total effects are compatible; these shall be classified as secondary effects and shall be considered as indirect actions.

2.4 Verification by the partial factor method

2.4.1 Design values

2.4.1.1 Design values of actions

(1) For pre-stress by controlled imposed deformations, e.g. by jacking at supports, the partial safety factor γ_p should be specified for ultimate limit states, taking into account favourable and unfavourable effects.

Note: Values for γ_p may be given in the National Annex. The recommended value for both favourable and unfavourable effects is 1,0.

2.4.1.2 Design values of material or product properties

(1)P Unless an upper estimate of strength is required, partial factors shall be applied to lower characteristic or nominal strengths.

(2)P For concrete, a partial factor γ_c shall be applied. The design compressive strength shall be given by:

$$f_{cd} = f_{ck} / \gamma_c \quad (2.1)$$

where the characteristic value f_{ck} shall be obtained by reference to EN 1992-1-1, 3.1 for normal concrete and to EN 1992-1-1, 11.3 for lightweight concrete.

Note: The value for γ_c is that used in EN 1992-1-1.

(3)P For steel reinforcement, a partial factor γ_s shall be applied.

Note: The value for γ_s is that used in EN 1992-1-1.

(4)P For structural steel, steel sheeting and steel connecting devices, partial factors γ_M shall be applied. Unless otherwise stated, the partial factor for structural steel shall be taken as γ_{M0} .

Note: Values for γ_M are those given in EN 1993.

(5)P For shear connection, a partial factor γ_V shall be applied.

Note: The value for γ_V may be given in the National Annex. The recommended value for γ_V is 1,25.

(6)P For longitudinal shear in composite slabs for buildings, a partial factor γ_{VS} shall be applied.

Note: The value for γ_{VS} may be given in the National Annex. The recommended value for γ_{VS} is 1,25.

(7)P For fatigue verification of headed studs in buildings, partial factors γ_{Mf} and $\gamma_{Mf,s}$ shall be applied.

Note: The value for γ_{Mf} is that used the relevant Parts of EN 1993. The value for $\gamma_{Mf,s}$ may be given in the National Annex. The recommended value for $\gamma_{Mf,s}$ is 1,0.

2.4.1.3 Design values of geometrical data

(1) Geometrical data for cross-sections and systems may be taken from product standards hEN or drawings for the execution and treated as nominal values.

2.4.1.4 Design resistances

(1)P For composite structures, design resistances shall be determined in accordance with EN 1990, expression (6.6a) or expression (6.6c).

2.4.2 Combination of actions

(1) The general formats for combinations of actions are given in EN 1990, Section 6.

Note: For buildings, the combination rules may be given in the National Annex to Annex A of EN 1990.

2.4.3 Verification of static equilibrium (EQU)

(1) The reliability format for the verification of static equilibrium for buildings, as described in EN 1990, Table A1.2(A), also applies to design situations equivalent to (EQU), e.g. for the design of hold down anchors or the verification of uplift of bearings of continuous beams.

Section 3 Materials

3.1 Concrete

(1) Unless otherwise given by Eurocode 4, properties should be obtained by reference to EN 1992-1-1, 3.1 for normal concrete and to EN 1992-1-1, 11.3 for lightweight concrete.

(2) This Part of EN 1994 does not cover the design of composite structures with concrete strength classes lower than C20/25 and LC20/22 and higher than C60/75 and LC60/66.

(3) Shrinkage of concrete should be determined taking account of the ambient humidity, the dimensions of the element and the composition of the concrete.

(4) Where composite action is taken into account in buildings, the effects of autogenous shrinkage may be neglected in the determination of stresses and deflections.

Note: Experience shows that the values of shrinkage strain given in EN 1992-1-1 can give overestimates of the effects of shrinkage in composite structures. Values for shrinkage of concrete may be given in the National Annex. Recommended values for composite structures for buildings are given in Annex C.

3.2 Reinforcing steel

(1) Properties should be obtained by reference to EN 1992-1-1, 3.2.

(2) For composite structures, the design value of the modulus of elasticity E_s may be taken as equal to the value for structural steel given in EN 1993-1-1, 3.2.6.

3.3 Structural steel

(1) Properties should be obtained by reference to EN 1993-1-1, 3.1 and 3.2.

(2) The rules in this Part of EN 1994 apply to structural steel of nominal yield strength not more than 460 N/mm².

3.4 Connecting devices

3.4.1 General

(1) Reference should be made to EN 1993-1-8 for requirements for fasteners and welding consumables.

3.4.2 Headed stud shear connectors

(1) Reference should be made to EN 13918.

3.5 Profiled steel sheeting for composite slabs in buildings

(1) Properties should be obtained by reference to EN 1993-1-3, 3.1 and 3.2.

(2) The rules in this Part of EN 1994 apply to the design of composite slabs with profiled steel sheets manufactured from steel in accordance with EN 10025, cold formed steel sheet in accordance with EN 10149-2 or EN 10149-3 or galvanised steel sheet in accordance with **AC1** EN 10326. **AC1**

Note: The minimum value for the nominal thickness t of steel sheets may be given in the National Annex. The recommended value is 0,70 mm.

Section 4 Durability

4.1 General

(1) The relevant provisions given in EN 1990, EN 1992 and EN 1993 should be followed.

(2) Detailing of the shear connection should be in accordance with 6.6.5.

4.2 Profiled steel sheeting for composite slabs in buildings

- (1)P The exposed surfaces of the steel sheeting shall be adequately protected to resist the particular atmospheric conditions.
- (2) A zinc coating, if specified, should conform to the requirements of **EN 10326** or with relevant standards in force.
- (3) A zinc coating of total mass 275 g/m² (including both sides) is sufficient for internal floors in a non-aggressive environment, but the specification may be varied depending on service conditions.

Section 5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

- (1)P The structural model and basic assumptions shall be chosen in accordance with EN 1990, 5.1.1 and shall reflect the anticipated behaviour of the cross-sections, members, joints and bearings.
- (2) Section 5 is applicable to composite structures in which most of the structural members and joints are either composite or of structural steel. Where the structural behaviour is essentially that of a reinforced or pre-stressed concrete structure, with only a few composite members, global analysis should be generally in accordance with EN 1992-1-1.
- (3) Analysis of composite slabs with profiled steel sheeting in buildings should be in accordance with Section 9.

5.1.2 Joint modelling

- (1) The effects of the behaviour of the joints on the distribution of internal forces and moments within a structure, and on the overall deformations of the structure, may generally be neglected, but where such effects are significant (such as in the case of semi-continuous joints) they should be taken into account, see Section 8 and EN 1993-1-8.
- (2) To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction may be made between three joint models as follows, see 8.2 and EN 1993-1-8, 5.1.1:
- simple, in which the joint may be assumed not to transmit bending moments;
 - continuous, in which the stiffness and/or resistance of the joint allow full continuity of the members to be assumed in the analysis;
 - semi-continuous, in which the behaviour of the joint needs to be taken into account in the analysis.
- (3) For buildings, the requirements of the various types of joint are given in Section 8 and in EN 1993-1-8.

5.1.3 Ground-structure interaction

- (1)P Account shall be taken of the deformation characteristics of the supports where significant.

Note: EN 1997 gives guidance for calculation of soil-structure interaction.

5.2 Structural stability

5.2.1 Effects of deformed geometry of the structure

(1) The action effects may generally be determined using either:

- first-order analysis, using the initial geometry of the structure
- second-order analysis, taking into account the influence of the deformation of the structure.

(2)P The effects of the deformed geometry (second-order effects) shall be considered if they increase the action effects significantly or modify significantly the structural behaviour.

(3) First-order analysis may be used if the increase of the relevant internal forces or moments caused by the deformations given by first-order analysis is less than 10%. This condition may be assumed to be fulfilled if the following criterion is satisfied:

$$\alpha_{cr} \geq 10 \quad (5.1)$$

where:

α_{cr} is the factor by which the design loading would have to be increased to cause elastic instability.

(4)P In determining the stiffness of the structure, appropriate allowances shall be made for cracking and creep of concrete and for the behaviour of the joints.

5.2.2 Methods of analysis for buildings

(1) Beam-and-column type plane frames may be checked for sway mode failure with first-order analysis if the criterion (5.1) is satisfied for each storey. In these structures α_{cr} may be calculated using the expression given in EN 1993-1-1, 5.2.1(4), provided that the axial compression in the beams is not significant and appropriate allowances are made for cracking of concrete, see 5.4.2.3, creep of concrete, see 5.4.2.2 and for the behaviour of the joints, see 8.2 and EN 1993-1-8, 5.1.

(2) Second-order effects may be included indirectly by using a first-order analysis with appropriate amplification.

(3) If second-order effects in individual members and relevant member imperfections are fully accounted for in the global analysis of the structure, individual stability checks for the members are un-necessary.

(4) If second-order effects in individual members or certain member imperfections (e.g. for flexural and/or lateral-torsional buckling) are not fully accounted for in the global analysis, the stability of individual members should be checked for the effects not included in the global analysis.

(5) If the global analysis neglects lateral-torsional effects, the resistance of a composite beam to lateral-torsional buckling may be checked using 6.4.

(6) For composite columns and composite compression members, flexural stability may be checked using one of the following methods:

- (a) by global analysis in accordance with 5.2.2(3), with the resistance of cross-sections being verified in accordance with 6.7.3.6 or 6.7.3.7, or

- (b) by analysis of the individual member in accordance with 6.7.3.4, taking account of end moments and forces from global analysis of the structure including global second-order effects and global imperfections when relevant. The analysis of the member should account for second-order effects in the member and relevant member imperfections, see 5.3.2.3, with the resistance of cross-sections being verified in accordance with 6.7.3.6 or 6.7.3.7, or
- (c) for members in axial compression, by the use of buckling curves to account for second-order effects in the member and member imperfections, see 6.7.3.5. This verification should take account of end forces from global analysis of the structure including global second-order effects and global imperfections when relevant, and should be based on a buckling length equal to the system length.

(7) For structures in which the columns are structural steel, stability may also be verified by member checks based on buckling lengths, in accordance with EN 1993-1-1, 5.2.2(8) and 6.3.

5.3 Imperfections

5.3.1 Basis

(1)P Appropriate allowances shall be incorporated in the structural analysis to cover the effects of imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of flatness, lack of fit and the unavoidable minor eccentricities present in joints of the unloaded structure.

(2)P The assumed shape of imperfections shall take account of the elastic buckling mode of the structure or member in the plane of buckling considered, in the most unfavourable direction and form.

5.3.2 Imperfections in buildings

5.3.2.1 General

(1) Equivalent geometric imperfections, see 5.3.2.2 and 5.3.2.3, should be used, with values that reflect the possible effects of global imperfections and of local imperfections, unless the effects of local imperfections are included in the resistance formulae for member design, see 5.3.2.3.

(2) Within a global analysis, member imperfections in composite compression members may be neglected where, according to $\overline{\lambda}_{AC1}$ 5.2.1(3), $\overline{\lambda}_{AC1}$ first-order analysis may be used. Where second-order analysis should be used, member imperfections may be neglected within the global analysis if:

$$\overline{\lambda} \leq 0,5 \sqrt{N_{pl,Rk} / N_{Ed}} \quad (5.2)$$

where:

$\overline{\lambda}$ is defined in 6.7.3.3 and calculated for the member considered as hinged at its ends;

$N_{pl,Rk}$ is defined in 6.7.3.3;

N_{Ed} is the design value of the normal force.

(3) Member imperfections should always be considered when verifying stability within a member's length in accordance with 6.7.3.6 or 6.7.3.7.

(4) Imperfections within steel compression members should be considered in accordance with EN 1993-1-1, 5.3.2 and 5.3.4.

5.3.2.2 Global imperfections

(1) The effects of imperfections should be allowed for in accordance with EN 1993-1-1, 5.3.2.

5.3.2.3 Member imperfections

(1) Design values of equivalent initial bow imperfection for composite columns and composite compression members should be taken from Table 6.5.

(2) For laterally unrestrained composite beams the effects of imperfections are incorporated within the formulae given for buckling resistance moment, see 6.4.

(3) For steel members the effects of imperfections are incorporated within the formulae given for buckling resistance, see EN 1993-1-1, 6.3.

5.4 Calculation of action effects

5.4.1 Methods of global analysis

5.4.1.1 General

(1) Action effects may be calculated by elastic global analysis, even where the resistance of a cross-section is based on its plastic or non-linear resistance.

(2) Elastic global analysis should be used for serviceability limit states, with appropriate corrections for non-linear effects such as cracking of concrete.

(3) Elastic global analysis should be used for verifications of the limit state of fatigue.

(4)P The effects of shear lag and of local buckling shall be taken into account if these significantly influence the global analysis.

(5) The effects of local buckling of steel elements on the choice of method of analysis may be taken into account by classifying cross-sections, see 5.5.

(6) The effects of local buckling of steel elements on stiffness may be ignored in normal composite sections. For cross-sections of Class 4, see EN 1993-1-5, 2.2.

(7) The effects on the global analysis of slip in bolt holes and similar deformations of connecting devices should be considered.

(8) Unless non-linear analysis is used, the effects of slip and separation on calculation of internal forces and moments may be neglected at interfaces between steel and concrete where shear connection is provided in accordance with 6.6.

5.4.1.2 Effective width of flanges for shear lag

(1)P Allowance shall be made for the flexibility of steel or concrete flanges affected by shear in their plane (shear lag) either by means of rigorous analysis, or by using an effective width of flange.

(2) The effects of shear lag in steel plate elements should be considered in accordance with EN 1993-1-1, 5.2.1(5).

(3) The effective width of concrete flanges should be determined in accordance with the following provisions.

(4) When elastic global analysis is used, a constant effective width may be assumed over the whole of each span. This value may be taken as the value $b_{\text{eff},1}$ at mid-span for a span supported at both ends, or the value $b_{\text{eff},2}$ at the support for a cantilever.

(5) At mid-span or an internal support, the total effective width b_{eff} , see Figure 5.1, may be determined as:

$$b_{\text{eff}} = b_0 + \sum b_{\text{ei}} \quad (5.3)$$

where:

b_0 is the distance between the centres of the outstand shear connectors;

b_{ei} is the value of the effective width of the concrete flange on each side of the web and taken as $L_e / 8$ but not greater than the geometric width b_i . The value b_i should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge b_i is the distance to the free edge. The length L_e should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, L_e may be assumed to be as shown in Figure 5.1.

(6) The effective width at an end support may be determined as:

$$b_{\text{eff}} = b_0 + \sum \beta_i b_{\text{ei}} \quad (5.4)$$

with:

$$\beta_i = (0,55 + 0,025 L_e / b_{\text{ei}}) \leq 1,0 \quad (5.5)$$

where:

b_{ei} is the effective width, see (5), of the end span at mid-span and L_e is the equivalent span of the end span according to Figure 5.1.

(7) The distribution of the effective width between supports and midspan regions may be assumed to be as shown in Figure 5.1.

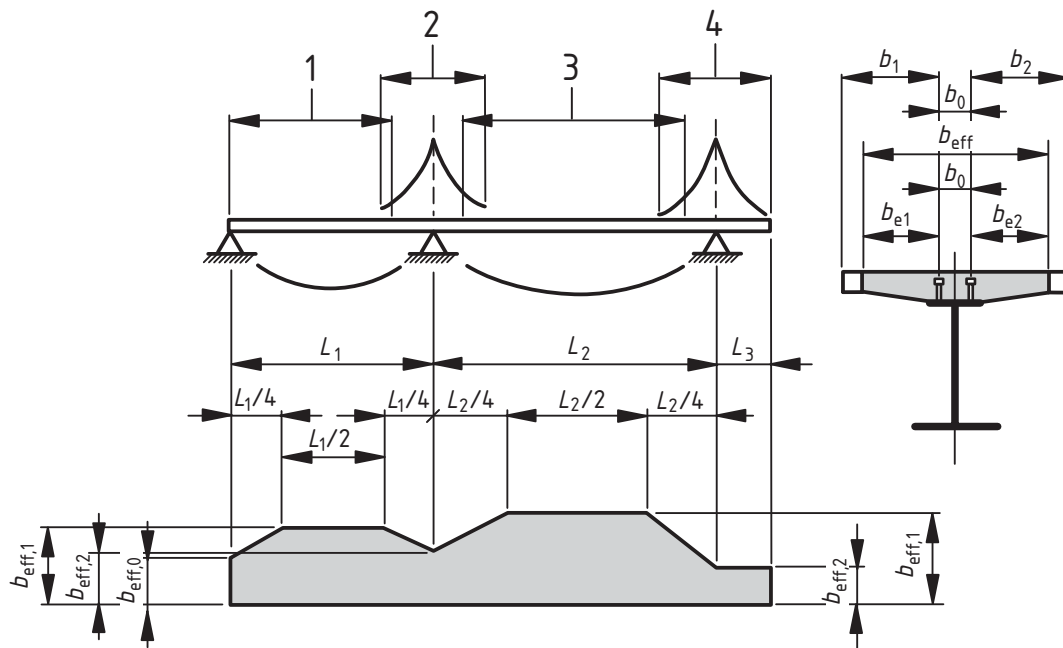
(8) Where in buildings the bending moment distribution is influenced by the resistance or the rotational stiffness of a joint, this should be considered in the determination of the length L_e .

(9) For analysis of building structures, b_0 may be taken as zero and b_i measured from the centre of the web.

5.4.2 Linear elastic analysis

5.4.2.1 General

(1) Allowance should be made for the effects of cracking of concrete, creep and shrinkage of concrete, sequence of construction and pre-stressing.



Key

- 1 $L_e = 0,85L_1$ for $b_{\text{eff},1}$
- 2 $L_e = 0,25(L_1 + L_2)$ for $b_{\text{eff},2}$
- 3 $L_e = 0,70L_2$ for $b_{\text{eff},1}$
- 4 $L_e = 2L_3$ for $b_{\text{eff},2}$

Figure 5.1 : Equivalent spans, for effective width of concrete flange

5.4.2.2 Creep and shrinkage

(1)P Appropriate allowance shall be made for the effects of creep and shrinkage of concrete.

(2) Except for members with both flanges composite, the effects of creep may be taken into account by using modular ratios n_L for the concrete. The modular ratios depending on the type of loading (subscript L) are given by:

$$n_L = n_0 (1 + \psi_L \varphi_t) \quad (5.6)$$

where:

- n_0 is the modular ratio E_a / E_{cm} for short-term loading;
- E_{cm} is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1, Table 3.1 or Table 11.3.1;
- φ_t is the creep coefficient $\varphi(t, t_0)$ according to EN 1992-1-1, 3.1.4 or 11.3.3, depending on the age (t) of concrete at the moment considered and the age (t_0) at loading,
- ψ_L is the creep multiplier depending on the type of loading, which $\overline{\text{AC1}}$ should $\overline{\text{AC1}}$ be taken as 1,1 for permanent loads, 0,55 for primary and secondary effects of shrinkage and 1,5 for pre-stressing by imposed deformations.

- (3) For permanent loads on composite structures cast in several stages one mean value t_0 may be used for the determination of the creep coefficient. This assumption may also be used for pre-stressing by imposed deformations, if the age of all of the concrete in the relevant spans at the time of pre-stressing is more than 14 days.
- (4) For shrinkage, the age at loading should generally be assumed to be one day.
- (5) Where prefabricated slabs are used or when pre-stressing of the concrete slab is carried out before the shear connection has become effective, the creep coefficient and the shrinkage values from the time when the composite action becomes effective should be used.
- (6) Where the bending moment distribution at t_0 is significantly changed by creep, for example in continuous beams of mixed structures with both composite and non-composite spans, the time-dependent secondary effects due to creep should be considered, except in global analysis for the ultimate limit state for members where all cross-sections are in Class 1 or 2. For the time-dependent secondary effects the modular ratio may be determined with a creep multiplier ψ_L of 0,55.
- (7) Appropriate account should be taken of the primary and secondary effects caused by shrinkage and creep of the concrete flange. The effects of creep and shrinkage of concrete may be neglected in analysis for verifications of ultimate limit states other than fatigue, for composite members with all cross-sections in Class 1 or 2 and in which no allowance for lateral-torsional buckling is necessary; for serviceability limit states, see Section 7.
- (8) In regions where the concrete slab is assumed to be cracked, the primary effects due to shrinkage may be neglected in the calculation of secondary effects.
- (9) In composite columns and compression members, account should be taken of the effects of creep in accordance with 6.7.3.4(2).
- (10) For double composite action with both flanges un-cracked (e.g. in case of pre-stressing) the effects of creep and shrinkage should be determined by more accurate methods.
- (11) For simplification in structures for buildings that satisfy expression (5.1) or 5.2.2(1), are not mainly intended for storage and are not pre-stressed by controlled imposed deformations, the effects of creep in composite beams may be taken into account by replacing concrete areas A_c by effective equivalent steel areas A_c/n for both short-term and long-term loading, where n is the nominal modular ratio corresponding to an effective modulus of elasticity for concrete $E_{c,eff}$ taken as $E_{cm}/2$.

5.4.2.3 Effects of cracking of concrete

- (1)P Appropriate allowance shall be made for the effects of cracking of concrete.
- (2) The following method may be used for the determination of the effects of cracking in composite beams with concrete flanges. First the envelope of the internal forces and moments for the characteristic combinations, see EN 1990, 6.5.3, including long-term effects should be calculated using the flexural stiffness $E_a I_1$ of the un-cracked sections. This is defined as “un-cracked analysis”. In regions where the extreme fibre tensile stress in the concrete due to the envelope of global effects exceeds twice the strength f_{ctm} or f_{ctm} , see EN1992-1-1, Table 3.1 or Table 11.3.1, the stiffness should be reduced to $E_a I_2$, see 1.5.2.12. This distribution of stiffness may be used for ultimate

limit states and for serviceability limit states. A new distribution of internal forces and moments, and deformation if appropriate, is then determined by re-analysis. This is defined as “cracked analysis”.

(3) For continuous composite beams with the concrete flanges above the steel section and not pre-stressed, including beams in frames that resist horizontal forces by bracing, the following simplified method may be used. Where all the ratios of the length of adjacent continuous spans (shorter/longer) between supports are at least 0,6, the effect of cracking may be taken into account by using the flexural stiffness $E_a I_2$ over 15% of the span on each side of each internal support, and as the un-cracked values $E_a I_1$ elsewhere.

(4) The effect of cracking of concrete on the flexural stiffness of composite columns and compression members should be determined in accordance with 6.7.3.4.

(5) In buildings, the contribution of any encasement to a beam may be determined by using the average of the cracked and un-cracked stiffness of the encasement. The area of concrete in compression may be determined from the plastic stress distribution.

5.4.2.4 Stages and sequence of construction

(1)P Appropriate analysis shall be made to cover the effects of staged construction including where necessary separate effects of actions applied to structural steel and to wholly or partially composite members.

(2) The effects of sequence of construction may be neglected in analysis for ultimate limit states other than fatigue, for composite members where all cross-sections are in Class 1 or 2 and in which no allowance for lateral-torsional buckling is necessary.

5.4.2.5 Temperature effects

(1) Account should be taken of effects due to temperature in accordance with EN 1991-1-5.

(2) Temperature effects may normally be neglected in analysis for the ultimate limit states other than fatigue, for composite members where all cross-sections are in Class 1 or Class 2 and in which no allowance for lateral-torsional buckling is necessary.

5.4.2.6 Pre-stressing by controlled imposed deformations

(1)P Where pre-stressing by controlled imposed deformations (e.g. jacking of supports) is provided, the effects of possible deviations from the assumed values of imposed deformations and stiffness on the internal moments and forces shall be considered for analysis of ultimate and serviceability limit states.

(2) Unless a more accurate method is used to determine internal moments and forces, the characteristic values of indirect actions due to imposed deformations may be calculated with the characteristic or nominal values of properties of materials and of imposed deformation, if the imposed deformations are controlled.

5.4.3 Non-linear global analysis

(1) Non-linear analysis may be used in accordance with EN 1992-1-1, 5.7 and EN 1993-1-1, 5.4.3.

(2)P The behaviour of the shear connection shall be taken into account.

(3)P Effects of the deformed geometry of the structure $\boxed{AC_1}$ shall $\boxed{AC_1}$ be taken into account in accordance with 5.2.

5.4.4 Linear elastic analysis with limited redistribution for buildings

(1) Provided that second-order effects need not be considered, linear elastic analysis with limited redistribution may be applied to continuous beams and frames for verification of limit states other than fatigue.

(2) The bending moment distribution given by a linear elastic global analysis according to 5.4.2 may be redistributed in a way that satisfies equilibrium and takes account of the effects of inelastic behaviour of materials, and all types of buckling.

(3) Bending moments from a linear elastic analysis may be redistributed:

- a) in composite beams with full or partial shear connection as given in (4) - (7);
- b) in steel members in accordance with EN 1993-1-1, 5.4.1(4);
- c) in concrete members subject mainly to flexure in accordance with EN 1992-1-1, 5.5;
- d) in partially-encased beams without a concrete or composite slab, in accordance with (b) or (c), whichever is the more restrictive.

(4) For ultimate limit state verifications other than for fatigue, the elastic bending moments in composite beams may be modified according to (5) – (7) where:

- the beam is a continuous composite member, or part of a frame that resists horizontal forces by bracing,
- the beam is connected by rigid and full-strength joints, or by one such joint and one nominally-pinned joint,
- for a partially-encased composite beam, either it is established that rotation capacity is sufficient for the degree of redistribution adopted, or the contribution of the reinforced concrete encasement in compression is neglected when calculating the resistance moment at sections where the bending moment is reduced,
- each span is of uniform depth and
- no allowance for lateral-torsional buckling is necessary.

(5) Where (4) applies, the bending moments in composite beams determined by linear elastic global analysis may be modified:

- by reducing maximum hogging moments by amounts not exceeding the percentages given in Table 5.1, or
- in beams with all cross-sections in Classes 1 or 2 only, by increasing maximum hogging moments by amounts not exceeding 10%, for un-cracked elastic analysis or 20% for cracked elastic analysis, see 5.4.2.3,

unless it is verified that the rotation capacity permits a higher value.

Table 5.1 : Limits to redistribution of hogging moments, per cent of the initial value of the bending moment to be reduced

Class of cross-section in hogging moment region	1	2	3	4
For un-cracked analysis	40	30	20	10
For cracked analysis	25	15	10	0

(6) For grades of structural steel higher than S355, redistribution should only be applied to beams with all cross-sections in Class 1 and Class 2. Redistribution by reduction of maximum hogging moments should not exceed 30% for an un-cracked analysis and 15% for a cracked analysis, unless it is demonstrated that the rotation capacity permits a higher value.

(7) For composite cross-sections in Class 3 or 4, the limits in Table 5.1 relate to bending moments assumed in design to be applied to the composite member. Moments applied to the steel member should not be redistributed.

5.4.5 Rigid plastic global analysis for buildings

(1) Rigid plastic global analysis may be used for ultimate limit state verifications other than fatigue, where second-order effects do not have to be considered and provided that:

- all the members and joints of the frame are steel or composite,
- the steel material satisfies EN 1993-1-1, 3.2.2,
- the cross-sections of steel members satisfy EN 1993-1-1, 5.6 and
- the joints are able to sustain their plastic resistance moments for a sufficient rotation capacity.

(2) In beams and frames for buildings, it is not normally necessary to consider the effects of alternating plasticity.

(3)P Where rigid-plastic global analysis is used, at each plastic hinge location:

- a) the cross-section of the structural steel section shall be symmetrical about a plane parallel to the plane of the web or webs,
- b) the proportions and restraints of steel components shall be such that lateral-torsional buckling does not occur,
- c) lateral restraint to the compression flange shall be provided at all hinge locations at which plastic rotation may occur under any load case,
- d) the rotation capacity shall be sufficient, when account is taken of any axial compression in the member or joint, to enable the required hinge rotation to develop and
- e) where rotation requirements are not calculated, all members containing plastic hinges shall have effective cross-sections of Class 1 at plastic hinge locations.

(4) For composite beams in buildings, the rotation capacity may be assumed to be sufficient where: