Design of composite steel and concrete structures - Part 1.1: General rules and rules for buildings

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

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

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Foreword

This European Standard EN 1994-1-1, Eurocode 4: Design of composite steel and concrete structures: General rules and rules for buildings, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1994-1-1 on YYYY-MM-DD.

No existing European Standard is superseded.

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 agreement 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:

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<th>Standard</th>
<th>Code</th>
<th>Description</th>
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<td>EN 1990</td>
<td>Eurocode :</td>
<td>Basis of Structural Design</td>
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<td>EN 1991</td>
<td>Eurocode 1:</td>
<td>Actions on structures</td>
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<td>EN 1995</td>
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<td>Design of timber structures</td>
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<td>EN 1996</td>
<td>Eurocode 6:</td>
<td>Design of masonry structures</td>
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<tr>
<td>EN 1997</td>
<td>Eurocode 7:</td>
<td>Geotechnical design</td>
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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).
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 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 Documents referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards. 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.

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.

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

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

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.

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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.
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)
- 2.4.1.2(6)
- 2.4.1.2(7)
- 2.4.2
- 3.1
- 3.5(2)
- 6.4.3(1)
- 6.6.1.1(5)
- 6.6.3.1(1)
- 6.6.3.1(3)
- 6.6.4.1(3)
- 6.8.2
- 9.1.1(2)
- 9.6(2)
- 9.7.3(4)
- 9.7.3(8)
- B.2.5(1)
- B.3.6(6)
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 Basis of structural design
EN 1991 Actions on structures
ENs, hENs, ETAGs and ETAs for construction products relevant for composite structures
EN 1090 Execution of steel structures – Technical requirements
EN 13670 Execution of concrete structures
EN 1992 Design of concrete structures
EN 1993 Design of steel structures
EN 1997 Geotechnical design
EN 1998 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) Technical rules for the execution of steel structures

1.2.2 Other reference standards

EN 1992-1-1 1 Design of concrete structures: General rules and rules for buildings
EN 1993-1-1 1 Design of steel structures: General rules and rules for buildings
EN 1993-1-3 1 Design of steel structures: Cold-formed thin gauge members and sheeting
EN 1993-1-5 1 Design of steel structures: Plated structural elements
EN 1993-1-8 1 Design of steel structures: Design of joints
EN 1993-1-9 1 Design of steel structures: Fatigue strength of steel structures
EN 10025-1: 2002 Hot-rolled products of structural steels: General delivery conditions
EN 10025-2: 2002 Hot-rolled products of structural steels: Technical delivery conditions for non-alloy structural steels
EN 10025-3: 2002 Hot-rolled products of structural steels: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
EN 10025-4: 2002 Hot-rolled products of structural steels: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels

1 To be published
EN 10025-5: 2002  Hot-rolled products of structural steels: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance

EN 10025-6: 2002  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 10147: 2000  Continuously hot-dip zinc coated structural steels strip and sheet: 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:

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 $E_d I_1$ 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
\textbf{cracked flexural stiffness}
the stiffness $E_d I_2$ 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
\textbf{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.

\textit{Latin upper case letters}

\begin{itemize}
  \item $A$ Cross-sectional area of the effective composite section neglecting concrete in tension
  \item $A_a$ Cross-sectional area of the structural steel section
  \item $A_b$ Cross-sectional area of bottom transverse reinforcement
  \item $A_{bh}$ Cross-sectional area of bottom transverse reinforcement in a haunch
  \item $A_c$ Cross-sectional area of concrete
  \item $A_{ct}$ Cross-sectional area of the tensile zone of the concrete
  \item $A_{fc}$ Cross-sectional area of the compression flange
  \item $A_p$ Cross-sectional area of profiled steel sheeting
  \item $A_{pe}$ Effective cross-sectional area of profiled steel sheeting
  \item $A_s$ Cross-sectional area of reinforcement
  \item $A_{sf}$ Cross-sectional area of transverse reinforcement
  \item $A_{sr}$ Cross-sectional area of reinforcement in row $r$
  \item $A_t$ Cross-sectional area of top transverse reinforcement
  \item $A_v$ Shear area of a structural steel section
  \item $A_1$ Loaded area under the gusset plate
  \item $E_a$ Modulus of elasticity of structural steel
  \item $E_{c,\text{eff}}$ Effective modulus of elasticity for concrete
  \item $E_{cm}$ Secant modulus of elasticity of concrete
  \item $E_s$ Design value of modulus of elasticity of reinforcing steel
  \item $(EI)_{\text{eff}}$ Effective flexural stiffness for calculation of relative slenderness
  \item $(EI)_{\text{eff,II}}$ Effective flexural stiffness for use in second-order analysis
  \item $(EI)_2$ Cracked flexural stiffness per unit width of the concrete or composite slab
  \item $F_{c,w,c,Rd}$ Design value of the resistance to transverse compression of the concrete encasement to a column web
  \item $F_l$ Design longitudinal force per stud
  \item $F_t$ Design transverse force per stud
  \item $F_{\text{ten}}$ Design tensile force per stud
  \item $G_a$ Shear modulus of structural steel
  \item $G_c$ Shear modulus of concrete
  \item $I$ Second moment of area of the effective composite section neglecting concrete in tension
  \item $I_a$ Second moment of area of the structural steel section
  \item $I_{at}$ St. Venant torsion constant of the structural steel section
\end{itemize}
# Structural Engineering Terms

- $I_c$: 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
- $K_{c.e}$, $K_{c.e,II}$: 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
- $M_{cr}$: Elastic critical moment for lateral-torsional buckling of a composite beam
- $M_{Ed}$: Design bending moment applied to the composite section
- $M_{Ed,i}$: 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
\( N_c \) 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
\( N_{c,i} \) 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
\( P_{l,Rd} \) Design value of the shear resistance of a single stud connector corresponding to \( F_i \)
\( P_{pb,Rd} \) 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 \( F_i \)
\( R_{Ed} \) Design value of a support reaction
\( S_j \) Rotational stiffness of a joint
\( S_{ji,ini} \) 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
- \( b_{\text{eff}} \) Total effective width
- \( b_{\text{eff},1} \) Effective width at mid-span for a span supported at both ends
- \( b_{\text{eff},2} \) Effective width at an internal support
- \( b_{\text{eff},c,wc} \) Effective width of the column web in compression
- \( b_{\text{ei}} \) Effective width of the concrete flange on each side of the web
- \( b_{\text{em}} \) 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
- \( d_{\text{do}} \) 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
- \( e_0 \) 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
- \( f_{\text{cd}} \) Design value of the cylinder compressive strength of concrete
- \( f_{\text{ck}} \) Characteristic value of the cylinder compressive strength of concrete at 28 days
- \( f_{\text{cm}} \) Mean value of the measured cylinder compressive strength of concrete
- \( f_{\text{ct,eff}} \) Mean value of the effective tensile strength of the concrete
- \( f_{\text{ctm}} \) Mean value of the axial tensile strength of concrete
- \( f_{\text{ct,0}} \) Reference strength for concrete in tension
- \( f_{\text{ctm}} \) Mean value of the axial tensile strength of lightweight concrete
Design value of the yield strength of reinforcing steel
Characteristic value of the yield strength of reinforcing steel
Specified ultimate tensile strength
Actual ultimate tensile strength in a test specimen
Nominal value of the yield strength of structural steel
Design value of the yield strength of structural steel
Design value of the yield strength of profiled steel sheeting
Mean value of the measured yield strength of profiled steel sheeting
Reduction factors for bending moments at supports
Overall depth; thickness
Depth of the structural steel section
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
Thickness of concrete flange; thickness of finishes
Position of neutral axis
Overall depth of the profiled steel sheeting excluding embossments
Depth between the centroids of the flanges of the structural steel section; distance between the longitudinal reinforcement in tension and the centre of compression
Overall nominal height of a stud connector
Overall thickness of test specimen
Amplification factor for second-order effects; coefficient; empirical factor for design shear resistance
Coefficient
Stiffness coefficient
Addition to the stiffness coefficient due to concrete encasement
Reduction factor for resistance of a headed stud used with profiled steel sheeting parallel to the beam
Rotational stiffness; coefficient
Stiffness of a shear connector
Stiffness reduction factor due to deformation of the shear connection
Stiffness coefficient for a row of longitudinal reinforcement in tension
Reduction factor for resistance of a headed stud used with profiled steel sheeting transverse to the beam
Factor for the effect of longitudinal compressive stress on transverse resistance of a column web
Parameter
Flexural stiffness of the cracked concrete or composite slab
Flexural stiffness of the web
Length of the beam in hogging bending adjacent to the joint
Length of slab in standard push test
Bearing lengths
Load introduction length
Slope of fatigue strength curve; empirical factor for design shear resistance
Modular ratio; number of shear connectors
Number of connectors for full shear connection
Modular ratio depending on the type of loading
Number of stud connectors in one rib
Modular ratio for short-term loading
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

\( \Delta \sigma \)  Reference value of the fatigue strength at 2 million cycles
\( \Delta \sigma_{E} \)  Equivalent constant amplitude stress range
\( \Delta \sigma_{E,\text{glob}} \)  Equivalent constant amplitude stress range due to global effects
\( \Delta \sigma_{E,\text{loc}} \)  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
\( \Delta \sigma_{s,\text{equ}} \)  Damage equivalent stress range
\( \Delta \tau \)  Stress range 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 stress range related to 2 million cycles
\( \Delta \tau_{R} \)  Fatigue strength
\( \Psi \)  Coefficient

Greek lower case letters

\( \alpha \)  Factor; parameter
\( \alpha_{cr} \)  Factor by which the design loads would have to be increased to cause elastic instability
\( \alpha_{M} \)  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
\( \alpha_{st} \)  Ratio
\( \beta \)  Factor; transformation parameter
\( \beta_{k} , \beta_{l} \)  Parameters
\( \gamma_{c} \)  Partial factor for concrete
\( \gamma_{f} \)  Partial factor for actions, also accounting for model uncertainties and dimensional variations
\( \gamma_{fE} \)  Partial factor for equivalent constant amplitude stress range
\(\gamma_M\) Partial factor for a material property, also accounting for model uncertainties and dimensional variations

\(\gamma_{M0}\) Partial factor for structural steel applied to resistance of cross-sections, see EN 1993-1-1, 6.1(1)

\(\gamma_{M1}\) Partial factor for structural steel applied to resistance of members to instability assessed by member checks, see EN 1993-1-1, 6.1(1)

\(\gamma_{Mf}\) Partial factor for fatigue strength

\(\gamma_{Mf,s}\) Partial factor for fatigue strength of studs in shear

\(\gamma_p\) Partial factor for pre-stressing action

\(\gamma_s\) Partial factor for reinforcing steel

\(\gamma_N\) Partial factor for design shear resistance of a headed stud

\(\gamma_{NS}\) Partial factor for design shear resistance of a composite slab

\(\delta\) Factor; steel contribution ratio; central deflection

\(\delta_{\text{max}}\) Sagging vertical deflection

\(\delta_s\) Deflection of steel sheeting under its own weight plus the weight of wet concrete

\(\delta_{\text{max}}\) Limiting value of \(\delta_s\)

\(\delta_k\) Characteristic value of slip capacity

\(\bar{\epsilon}\) \(\sqrt{235/f_y}\), where \(f_y\) is in N/mm\(^2\)

\(\eta\) Degree of shear connection; coefficient

\(\eta_a, \eta_{ao}\) Factors related to the confinement of concrete

\(\eta_c, \eta_{co}, \eta_{cL}\) Factors related to the confinement of concrete

\(\theta\) Angle

\(\lambda, \lambda_v\) Damage equivalent factors

\(\lambda_{\text{glob}}, \lambda_{\text{loc}}\) Damage equivalent factors for global effects and local effects, respectively

\(\lambda\) Relative slenderness

\(\lambda_{LT}\) Relative slenderness for lateral-torsional buckling

\(\mu\) Coefficient of friction; nominal factor

\(\mu_d\) Factor related to design for compression and uniaxial bending

\(\mu_{dy}, \mu_{dz}\) Factor \(\mu_d\) related to plane of bending

\(\nu\) Reduction factor to allow for the effect of longitudinal compression on resistance in shear; parameter related to deformation of the shear connection

\(\nu_a\) Poisson’s ratio for structural steel

\(\zeta\) Parameter related to deformation of the shear connection

\(\rho\) Parameter related to reduced design bending resistance accounting for vertical shear

\(\rho_s\) Parameter; reinforcement ratio

\(\sigma_{\text{com,c,Ed}}\) Longitudinal compressive stress in the encasement due to the design normal force

\(\sigma_{\text{Rd}}\) Local design strength of concrete

\(\sigma_{\text{ct}}\) 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

\(\sigma_{s_{\text{max,f}}}, \sigma_{s_{\text{min,f}}}\) Stress in the reinforcement due to the bending moment \(M_{\text{Ed,max,f}}\)

\(\sigma_s\) Stress in the tension reinforcement

\(\sigma_{s_{\text{max}}}\) Stress in the reinforcement due to the bending moment \(M_{\text{max}}\)
σ_{s,max,0}  Stress in the reinforcement due to the bending moment \( M_{\text{max}} \), neglecting concrete in tension

σ_{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

τ_{u,Rd}  Design value of longitudinal shear strength of a composite slab

τ_{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(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  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 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 $\gamma_P$ should be specified for ultimate limit states, taking into account favourable and unfavourable effects.

Note: Values for $\gamma_P$ 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) Unless an upper estimate of strength is required, partial factors shall be applied to lower characteristic or nominal strengths.

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

$$f_{cd} = f_{ck} / \gamma_C$$  (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 $\gamma_C$ is that used in EN 1992-1-1.

(3) For steel reinforcement, a partial factor $\gamma_S$ shall be applied.

Note: The value for $\gamma_S$ is that used in EN 1992-1-1.

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

Note: Values for $\gamma_M$ are those given in EN 1993.

(5) For shear connection, a partial factor $\gamma_Y$ shall be applied.

Note: The value for $\gamma_Y$ may be given in the National Annex. The recommended value for $\gamma_Y$ is 1.25.

(6) For longitudinal shear in composite slabs, a partial factor $\gamma_{VS}$ shall be applied.

Note: The value for $\gamma_{VS}$ may be given in the National Annex. The recommended value for $\gamma_{VS}$ is 1.25.

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

Note: The value for $\gamma_{Mf}$ 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 EN 10147.

Note: The value for the minimum nominal thickness 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) 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 10147 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) 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) 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) 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:

\[ \alpha_{cr} \geq 10 \]  

(5.1)

where:

\[ \alpha_{cr} \] is the factor by which the design loading would have to be increased to cause elastic instability.

(4) 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 $\alpha_{cr}$ 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) 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) 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 5.2.1(2), first-order analysis may be used. Where
second-order analysis should be used, member imperfections may be neglected within
the global analysis if:

$$\overline{\lambda} \leq 0.5 \sqrt{N_{\text{pl,Rk}} / N_{\text{Ed}}}$$  \hspace{1cm} (5.2)

where:

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

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

$N_{\text{Ed}}$ 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) 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 boltholes 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) 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 \( b_{\text{eff}} \), see Figure 5.1, may be determined as:

\[
b_{\text{eff}} = b_0 + \sum b_{ei}
\]

(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 midway 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:

$$b_{eff} = b_0 + \sum \beta_i b_{ei}$$  \hspace{1cm} (5.4)

with:

$$\beta_i = (0.55 + 0.025 L_e / b_{ei}) \leq 1.0$$  \hspace{1cm} (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.

![Figure 5.1: Equivalent spans, for effective width of concrete flange](image-url)
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.

5.4.2.2 Creep and shrinkage

(1) 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:

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

(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;
- $\varphi_t$ 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,
- $\psi_L$ is the creep multiplier depending on the type of loading, which 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 of 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 $\psi_L$ 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 $nA_c$ 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) 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 $f_{c,tm}$ or $f_{l,tm}$, 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) 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) 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) The behaviour of the shear connection shall be taken into account.

(3) Effects of the deformed geometry of the structure should 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

<table>
<thead>
<tr>
<th>Class of cross-section in hogging moment region</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>For un-cracked analysis</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>For cracked analysis</td>
<td>25</td>
<td>15</td>
<td>10</td>
<td>0</td>
</tr>
</tbody>
</table>
(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. The redistribution 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) 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:
   a) the grade of structural steel does not exceed S355,
   b) the contribution of any reinforced concrete encasement in compression is neglected when calculating the design resistance moment,
   c) all effective cross-sections at plastic hinge locations are in Class 1; and all other effective cross-sections are in Class 1 or Class 2,
   d) each beam-to-column joint has been shown to have sufficient design rotation capacity, or to have a design resistance moment at least 1.2 times the design plastic resistance moment of the connected beam,
e) adjacent spans do not differ in length by more than 50% of the shorter span,

f) end spans do not exceed 115% of the length of the adjacent span,

g) in any span in which more than half of the total design load for that span is concentrated within a length of one-fifth of the span, then at any hinge location where the concrete slab is in compression, not more than 15% of the overall depth of the member should be in compression; this does not apply where it can be shown that the hinge will be the last to form in that span and

h) the steel compression flange at a plastic hinge location is laterally restrained.

(5) Unless verified otherwise, it should be assumed that composite columns do not have rotation capacity.

(6) Where the cross-section of a steel member varies along its length, EN 1993-1-1, 5.6(3) is applicable.

(7) Where restraint is required by (3)(c) or 4(h), it should be located within a distance along the member from the calculated hinge location that does not exceed half the depth of the steel section.

5.5 Classification of cross-sections

5.5.1 General

(1) The classification system defined in EN 1993-1-1, 5.5.2 applies to cross-sections of composite beams.

(2) A composite section should be classified according to the least favourable class of its steel elements in compression. The class of a composite section normally depends on the direction of the bending moment at that section.

(3) A steel compression element restrained by attaching it to a reinforced concrete element may be placed in a more favourable class, provided that the resulting improvement in performance has been established.

(4) For classification, the plastic stress distribution should be used except at the boundary between Classes 3 and 4, where the elastic stress distribution should be used taking into account sequence of construction and the effects of creep and shrinkage. For classification, design values of strengths of materials should be used. Concrete in tension should be neglected. The distribution of the stresses should be determined for the gross cross-section of the steel web and the effective flanges.

(5) For cross-sections in Class 1 and 2 with bars in tension, reinforcement used within the effective width should have a ductility Class B or C, see EN 1992-1-1, Table C.1. Additionally for a section whose resistance moment is determined by 6.2.1.2, 6.2.1.3 or 6.2.1.4, a minimum area of reinforcement $A_s$ within the effective width of the concrete flange should be provided to satisfy the following condition:
\[ A_s \geq \rho_s A_c \]  
with  
\[ \rho_s = \delta \frac{f_y}{235} \frac{f_{ctm}}{f_{sk}} \sqrt{k_c} \]  
where:  
- \( A_c \) is the effective area of the concrete flange;  
- \( f_y \) is the nominal value of the yield strength of the structural steel in N/mm\(^2\);  
- \( f_{sk} \) is the characteristic yield strength of the reinforcement;  
- \( f_{ctm} \) is the mean tensile strength of the concrete, see EN1992-1-1, Table 3.1 or Table 11.3.1;  
- \( k_c \) is a coefficient given in 7.4.2;  
- \( \delta \) is equal to 1.0 and 1.1 for Class 2 and Class 1 cross-sections, respectively.

(6) Welded mesh should not be included in the effective section unless it has been shown to have sufficient ductility, when built into a concrete slab, to ensure that it will not fracture.

(7) In global analysis for stages in construction, account should be taken of the class of the steel section at the stage considered.

### 5.5.2 Classification of composite sections without concrete encasement

(1) A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class 1 if the spacing of connectors is in accordance with 6.6.5.5.

(2) The classification of other steel flanges and webs in compression in composite beams without concrete encasement should be in accordance with EN 1993-1-1, Table 5.2. An element that fails to satisfy the limits for Class 3 should be taken as Class 4.

(3) Cross-sections with webs in Class 3 and flanges in Classes 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance with EN1993-1-1, 6.2.2.4.

### 5.5.3 Classification of composite sections for buildings with concrete encasement

(1) A steel outstand flange of a composite section with concrete encasement in accordance with (2) below may be classified in accordance with Table 5.2.

**Table 5.2 : Classification of steel flanges in compression for partially-encased sections**
(2) For a web of a concrete encased section, the concrete that encases it should be reinforced, mechanically connected to the steel section, and capable of preventing buckling of the web and of any part of the compression flange towards the web. It may be assumed that the above requirements are satisfied if:

a) the concrete that encases a web is reinforced by longitudinal bars and stirrups, and/or welded mesh,

b) the requirements for the ratio \( b_c / b \) given in Table 5.2 are fulfilled,

c) the concrete between the flanges is fixed to the web in accordance with Figure 6.10 by welding the stirrups to the web or by means of bars of at least 6 mm diameter through holes and/or studs with a diameter greater than 10 mm welded to the web and

d) the longitudinal spacing of the studs on each side of the web or of the bars through holes is not greater than 400 mm. The distance between the inner face of each flange and the nearest row of fixings to the web is not greater than 200 mm. For steel sections with a maximum depth of not less than 400 mm and two or more rows of fixings, a staggered arrangement of the studs and/or bars through holes may be used.

(3) A steel web in Class 3 encased in concrete in accordance with (2) above may be represented by an effective web of the same cross-section in Class 2.

Section 6  Ultimate limit states

6.1 Beams

6.1.1 Beams for buildings

(1) Composite beams are defined in 1.5.2. Typical types of cross-section are shown in Figure 6.1 with either a solid slab or a composite slab. Partially–encased beams are those in which the
web of the steel section is encased by reinforced concrete and shear connection is provided between the concrete and the steel components.

![Figure 6.1: Typical cross-sections of composite beams](image)

(2) Design resistances of composite cross-sections in bending or/and vertical shear should be determined in accordance with 6.2 for composite beams with steel sections and 6.3 for partially-encased composite beams.

(3) Composite beams shall be checked for:
- resistance of critical cross-sections (6.2 and 6.3);
- resistance to lateral-torsional buckling (6.4);
- resistance to shear buckling (6.2.2.3) and transverse forces on webs (6.5);
- resistance to longitudinal shear (6.6).

(4) Critical cross-sections include:
- sections of maximum bending moment;
- supports;
- sections subjected to concentrated loads or reactions;
- places where a sudden change of cross-section occurs, other than a change due to cracking of concrete.

(5) A cross-section with a sudden change should be considered as a critical cross-section when the ratio of the greater to the lesser resistance moment is greater than 1.2.

(6) For checking resistance to longitudinal shear, a critical length consists of a length of the interface between two critical cross-sections. For this purpose critical cross-sections also include:
- free ends of cantilevers;
– in tapering members, sections so chosen that the ratio of the greater to the lesser plastic resistance moments (under flexural bending of the same direction) for any pair of adjacent cross-sections does not exceed 1.5.

(7)P The concepts "full shear connection" and "partial shear connection" are applicable only to beams in which plastic theory is used for calculating bending resistances of critical cross-sections. A span of a beam, or a cantilever, has full shear connection when increase in the number of shear connectors would not increase the design bending resistance of the member. Otherwise, the shear connection is partial.

Note: Limits to the use of partial shear connection are given in 6.6.1.2.

6.1.2 Effective width for verification of cross-sections

(1) The effective width of the concrete flange for verification of cross-sections should be determined in accordance with 5.4.1.2 taking into account the distribution of effective width between supports and mid-span regions.

(2) As a simplification for buildings, a constant effective width may be assumed over the whole region in sagging bending of each span. This value may be taken as the value \( b_{\text{eff},1} \) at mid-span. The same assumption applies over the whole region in hogging bending on both sides of an intermediate support. This value may be taken as the value \( b_{\text{eff},2} \) at the relevant support.

6.2 Resistances of cross-sections of beams

6.2.1 Bending resistance

6.2.1.1 General

(1)P The design bending resistance shall be determined by rigid-plastic theory only where the effective composite cross-section is in Class 1 or Class 2 and where pre-stressing by tendons is not used.

(2) Elastic analysis and non-linear theory for bending resistance may be applied to cross-sections of any class.

(3) For elastic analysis and non-linear theory it may be assumed that the composite cross-section remains plane if the shear connection and the transverse reinforcement are designed in accordance with 6.6, considering appropriate distributions of design longitudinal shear force.

(4)P The tensile strength of concrete shall be neglected.

(5) Where the steel section of a composite member is curved in plan, the effects of curvature should be taken into account.

6.2.1.2 Plastic resistance moment \( M_{\text{pl,Rd}} \) of a composite cross-section

(1) The following assumptions should be made in the calculation of \( M_{\text{pl,Rd}} \):

a) there is full interaction between structural steel, reinforcement, and concrete;
b) the effective area of the structural steel member is stressed to its design yield strength $f_{yd}$ in tension or compression;

c) the effective areas of longitudinal reinforcement in tension and in compression are stressed to their design yield strength $f_{sd}$ in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected;

d) the effective area of concrete in compression resists a stress of $0.85 f_{cd}$, constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete, where $f_{cd}$ is the design cylinder compressive strength of concrete.

Typical plastic stress distributions are shown in Figure 6.2.

![Diagram of plastic stress distributions](image)

**Figure 6.2**: Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending

(2) For composite cross-sections with structural steel grade S420 or S460, where the distance $x_{pl}$ between the plastic neutral axis and the extreme fibre of the concrete slab in compression exceeds 15% of the overall depth $h$ of the member, the design resistance moment $M_{Rd}$ should be taken as $\beta M_{pl,Rd}$ where $\beta$ is the reduction factor given in Figure 6.3. For values of $x_{pl}/h$ greater than 0.4 the resistance to bending should be determined from 6.2.1.4 or 6.2.1.5.

(3) Where plastic theory is used and reinforcement is in tension, that reinforcement should be in accordance with 5.5.1(5).

(4) For buildings, profiled steel sheeting in compression shall be neglected.

(5) For buildings, any profiled steel sheeting in tension included within the effective section should be assumed to be stressed to its design yield strength $f_{yp,d}$. 
6.2.1.3 Plastic resistance moment of sections with partial shear connection in buildings

(1) In regions of sagging bending, partial shear connection in accordance with 6.6.1 and 6.6.2.2 may be used in composite beams for buildings.

(2) Unless otherwise verified, the plastic resistance moment in hogging bending should be determined in accordance with 6.2.1.2 and appropriate shear connection should be provided to ensure yielding of reinforcement in tension.

(3) Where ductile shear connectors are used, the resistance moment of the critical cross-section of the beam $M_{Rd}$ may be calculated by means of rigid plastic theory in accordance with 6.2.1.2, except that a reduced value of the compressive force in the concrete flange $N_c$ should be used in place of the force $N_{cf}$ given by 6.2.1.2(1)(d). The ratio $\eta = N_c / N_{cf}$ is the degree of shear connection. The location of the plastic neutral axis in the slab should be determined by the new force $N_c$, see Figure 6.4. There is a second plastic neutral axis within the steel section, which should be used for the classification of the web.

(4) The relation between $M_{Rd}$ and $N_c$ in (3) is qualitatively given by the convex curve ABC in Figure 6.5 where $M_{pl,a,Rd}$ and $M_{pl,Rd}$ are the design plastic resistances to sagging bending of the structural steel section alone, and of the composite section with full shear connection, respectively.
(5) For the method given in (3), a conservative value of $M_{Rd}$ may be determined by the straight line AC in Figure 6.5:

$$M_{Rd} = M_{pl,a,Rd} + \left( M_{pl,Rd} - M_{pl,a,Rd} \right) \frac{N_c}{N_{cf}}$$  \hspace{1cm} (6.1)

### 6.2.1.4 Non-linear resistance to bending

(1) Where the bending resistance of a composite cross-section is determined by non-linear theory, the stress-strain relationships of the materials shall be taken into account.

(2) It should be assumed that the composite cross-section remains plane and that the strain in bonded reinforcement, whether in tension or compression, is the same as the mean strain in the surrounding concrete.

(3) The stresses in the concrete in compression should be derived from the stress-strain curves given in EN 1992-1-1, 3.1.7.

(4) The stresses in the reinforcement should be derived from the bi-linear diagrams given in EN 1992-1-1, 3.2.7.

(5) The stresses in structural steel in compression or tension should be derived from the bi-linear diagram given in EN 1993-1-1, 5.4.3(4) and should take account of the effects of the method of construction (e.g. propped or un-propped).

(6) For Class 1 and Class 2 composite cross-sections with the concrete flange in compression, the non-linear resistance to bending $M_{Rd}$ may be determined as a function of the compressive force in the concrete $N_c$ using the simplified expressions (6.2) and (6.3), as shown in Figure 6.6:

$$M_{Rd} = M_{a,Ed} + (M_{el,Rd} - M_{a,Ed}) \frac{N_c}{N_{c,el}} \hspace{1cm} \text{for} \hspace{1cm} N_c \leq N_{c,el}$$  \hspace{1cm} (6.2)
\[
M_{\text{Rd}} = M_{\text{el,Rd}} + (M_{\text{pl,Rd}} - M_{\text{el,Rd}}) \frac{N_c - N_{c,\text{el}}}{N_{c,f} - N_{c,\text{el}}} \quad \text{for } N_{c,\text{el}} \leq N_c \leq N_{c,f} \quad (6.3)
\]

with:

\[
M_{\text{el,Rd}} = M_{\text{a,Ed}} + k M_{\text{Ed}} \quad (6.4)
\]

where:

- \(M_{\text{a,Ed}}\) is the design bending moment applied to the structural steel section before composite behaviour;
- \(M_{\text{Ed}}\) is the design bending moment applied to the composite section;
- \(k\) is the lowest factor such that a stress limit in 6.2.1.5(2) is reached; where un-propped construction is used, the sequence of construction should be taken into account;
- \(N_{c,\text{el}}\) is the compressive force in the concrete flange corresponding to moment \(M_{\text{el,Rd}}\).

For cross sections where 6.2.1.2 (2) applies, in expression (6.3) and in Figure 6.6 instead of \(M_{\text{pl,Rd}}\) the reduced value \(\beta M_{\text{pl,Rd}}\) should be used.

(7) For buildings, the determination of \(M_{\text{el,Rd}}\) may be simplified using 5.4.2.2(11).

---

**Figure 6.6**: Simplified relationship between \(M_{\text{Rd}}\) and \(N_c\) for sections with the concrete slab in compression

### 6.2.1.5 Elastic resistance to bending

(1) Stresses should be calculated by elastic theory, using an effective width of the concrete flange in accordance with 6.1.2. For cross-sections in Class 4, the effective structural steel section should be determined in accordance with EN 1993-1-5, 4.3.

(2) In the calculation of the elastic resistance to bending based on the effective cross-section, the limiting stresses should be taken as:
– $f_{cd}$ in concrete in compression;
– $f_{yd}$ in structural steel in tension or compression;
– $f_{sd}$ in reinforcement in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected.

(3) Stresses due to actions on the structural steelwork alone shall be added to stresses due to actions on the composite member.

(4) Unless a more precise method is used, the effect of creep should be taken into account by use of a modular ratio according to 5.4.2.2.

(5) In cross-sections with concrete in tension and assumed to be cracked, the stresses due to primary (isostatic) effects of shrinkage may be neglected.

6.2.2 Resistance to vertical shear

6.2.2.1 Scope

(1) Clause 6.2.2 applies to composite beams with a rolled or welded structural steel section with a solid web, which may be stiffened.

6.2.2.2 Plastic resistance to vertical shear

(1) The resistance to vertical shear $V_{pl,Rd}$ should be taken as the resistance of the structural steel section $V_{pl,a,Rd}$ unless the value for a contribution from the reinforced concrete part of the beam has been established.

(2) The design plastic shear resistance $V_{pl,a,Rd}$ of the structural steel section should be determined in accordance with EN 1993-1-1, 6.2.6.

6.2.2.3 Shear buckling resistance

(1) The shear buckling resistance $V_{b,Rd}$ of an uncased steel web should be determined in accordance with EN 1993-1-5, 5.

(2) No account should be taken of a contribution from the concrete slab, unless a more precise method than the one of EN 1993-1-5, 5 is used and unless the shear connection is designed for the relevant vertical force.

6.2.2.4 Bending and vertical shear

(1) Where the vertical shear force $V_{Ed}$ exceeds half the shear resistance $V_{Rd}$ given by $V_{pl,Rd}$ in 6.2.2.2 or $V_{b,Rd}$ in 6.2.2.3, whichever is the smaller, allowance should be made for its effect on the resistance moment.

(2) For cross-sections in Class 1 or 2, the influence of the vertical shear on the resistance to bending may be taken into account by a reduced design steel strength $(1 - \rho) f_{yd}$ in the shear area as shown in Figure 6.7 where:

$$\rho = (2V_{Ed} / \sqrt{V_{Rd}} - 1)^2$$  \hspace{1cm} (6.5)
and $V_{Rd}$ is the appropriate resistance to vertical shear, determined in accordance with 6.2.2.2 or 6.2.2.3.

(3) For cross-sections in Class 3 and 4, EN 1993-1-5, 7.1 is applicable using the calculated stresses of the composite section.

![Figure 6.7: Plastic stress distribution modified by the effect of vertical shear](image1)

**6.3 Resistance of cross-sections of beams for buildings with partial encasement**

**6.3.1 Scope**

(1) Partially–encased beams are defined in 6.1.1(1). A concrete or composite slab can also form part of the effective section of the composite beam, provided that it is attached to the steel section by a shear connection in accordance with 6.6. Typical cross-sections are shown in Figure 6.8.

(2) Clause 6.3 is applicable to partially encased sections in Class 1 or Class 2, provided that $d/t_w$ is not greater than 124ε.

![Figure 6.8: Typical cross-sections of partially-encased beams](image2)

(3) The provisions elsewhere in EN 1994-1-1 are applicable, unless different rules are given in 6.3.

**6.3.2 Bending resistance**
(1) Full shear connection should be provided between the structural steel section and the web encasement in accordance with 6.6.

(2) The design resistance moment may be determined by plastic theory. Reinforcement in compression in the concrete encasement may be neglected. Some examples of typical plastic stress distributions are shown in Figure 6.9.

(3) Partial shear connection may be used for the compressive force in any concrete or composite slab forming part of the effective section.

(4) Where partial shear connection is used with ductile connectors, the plastic resistance moment of the beam should be calculated in accordance with 6.3.2(2) and 6.2.1.2(1), except that a reduced value of the compressive force in the concrete or composite slab $N_c$ should be used as in 6.2.1.3(3), (4) and (5).

Figure 6.9 : Examples of plastic stress distributions for effective sections

6.3.3 Resistance to vertical shear

(1) The design shear resistance of the structural steel section $V_{pl,a,Rd}$ should be determined by plastic theory in accordance with 6.2.2.2(2).

(2) The contribution of the web encasement to shear may be taken into account for the determination of the design shear resistance of the cross-section if stirrups are used in accordance with Figure 6.10. Appropriate shear connection should be provided between the encasement and the structural steel section. If the stirrups of the encasement are open, they should be attached to the web by full strength welds. Otherwise the contribution of the shear reinforcement should be neglected.

(3) Unless a more accurate analysis is used, the distribution of the total vertical shear $V_{Ed}$ into the parts $V_{a,Ed}$ and $V_{c,Ed}$, acting on the steel section and the reinforced concrete
web encasement respectively, may be assumed to be in the same ratio as the contributions of the steel section and the reinforced web encasement to the bending resistance $M_{pl,Rd}$.

(4) The resistance to vertical shear for the web encasement should take account of cracking of concrete and should be verified in accordance with EN 1992-1-1, 6.2 and the other relevant design requirements of that Standard.

![Figure 6.10: Arrangement of stirrups](image)

**6.3.4 Bending and vertical shear**

(1) Where the design vertical shear force $V_{a,Ed}$ exceeds half the design plastic resistance $V_{pl,a,Rd}$ of the structural steel section to vertical shear, allowance should be made for its effect on the resistance moment.

(2) The influence of the vertical shear on the resistance to bending may be expressed as in 6.2.2.4(2) with the following modification. In expression (6.5), the ratio $V_{Ed}/V_{pl,Rd}$ is replaced by $V_{a,Ed}/V_{pl,a,Rd}$ to calculate the reduced design steel strength in the shear area of the structural steel section. Then, the design reduced plastic resistance moment $M_{Rd}$ should be calculated in accordance with 6.3.2.

**6.4 Lateral-torsional buckling of composite beams**

**6.4.1 General**

(1) A steel flange that is attached to a concrete or composite slab by shear connection in accordance with 6.6 may be assumed to be laterally stable, provided that lateral instability of the concrete slab is prevented.

(2) All other steel flanges in compression should be checked for lateral stability.

(3) The methods in EN 1993-1-1, 6.3.2.1-6.3.2.3 and, more generally, 6.3.4 are applicable to the steel section on the basis of the cross-sectional forces on the composite section, taking into account effects of sequence of construction in accordance with 5.4.2.4. The lateral and elastic torsional restraint at the level of the shear connection to the concrete slab may be taken into account.

(4) For composite beams in buildings with cross-sections in Class 1, 2 or 3 and of uniform structural steel section, the method given in 6.4.2 may be used.

**6.4.2 Verification of lateral-torsional buckling of continuous composite beams with cross-sections in Class 1, 2 and 3 for buildings**
(1) The design buckling resistance moment of a laterally unrestrained continuous composite beam (or a beam within a frame that is composite throughout its length) with Class 1, 2 or 3 cross-sections and with a uniform structural steel section should be taken as:

\[ M_{b,Rd} = \chi_{LT} M_{Rd} \]

where:

\[ \chi_{LT} \] is the reduction factor for lateral-torsional buckling depending on the relative slenderness \( \lambda_{LT} \);

\( M_{Rd} \) is the design resistance moment under hogging bending at the relevant internal support (or beam-to-column joint).

Values of the reduction factor \( \chi_{LT} \) may be obtained from EN 1993-1-1, 6.3.2.2 or 6.3.2.3.

(2) For cross-sections in Class 1 or 2, \( M_{Rd} \) should be determined according to 6.2.1.2 for a beam whose bending resistance is based on plastic theory, or 6.2.1.4 for a beam whose bending resistance is based on non-linear theory, or 6.3.2 for a partially-encased beam, with \( f_{yd} \) determined using the partial factor \( \gamma_{M1} \) given by EN 1993-1-1, 6.1(1).

(3) For cross-sections in Class 3, \( M_{Rd} \) should be determined using expression (6.4), but as the design hogging bending moment that causes either a tensile stress \( f_{sd} \) in the reinforcement or a compression stress \( f_{yd} \) in the extreme bottom fibre of the steel section, whichever is the smaller; \( f_{yd} \) should be determined using the partial factor \( \gamma_{M1} \) given by EN 1993-1-1, 6.1(1).

(4) The relative slenderness \( \lambda_{LT} \) may be calculated by:

\[ \lambda_{LT} = \sqrt{\frac{M_{Rk}}{M_{cr}}} \]

where:

\( M_{Rk} \) is the resistance moment of the composite section using the characteristic material properties;

\( M_{cr} \) is the elastic critical moment for lateral-torsional buckling determined at the internal support of the relevant span where the hogging bending moment is greatest.

(5) Where the same slab is also attached to one or more supporting steel members approximately parallel to the composite beam considered and the conditions 6.4.3(c), (e) and (f) are satisfied, the calculation of the elastic critical moment \( M_{cr} \) may be based on the "continuous inverted U-frame" model. As shown in Figure 6.11, this model takes into account the lateral displacement of the bottom flange causing bending of the steel web, and the rotation of the top flange that is resisted by bending of the slab.
Figure 6.11 : Inverted-U frame ABCD resisting lateral-torsional buckling

(6) At the level of the top steel flange, a rotational stiffness $k_s$ per unit length of steel beam may be adopted to represent the U-frame model by a beam alone:

$$k_s = \frac{k_1 k_2}{k_1 + k_2}$$

(6.8)

where:

$k_1$ is the flexural stiffness of the cracked concrete or composite slab in the direction transverse to the steel beam, which may be taken as:

$$k_1 = \alpha \frac{(EI)_2}{a}$$

(6.9)

where $\alpha = 2$ for $k_1$ for an edge beam, with or without a cantilever, and $\alpha = 3$ for an inner beam. For inner beams in a floor with four or more similar beams, $\alpha = 4$ may be used;

$a$ is the spacing between the parallel beams;

$(EI)_2$ is the "cracked" flexural stiffness per unit width of the concrete or composite slab, taken as the lower of the value at mid-span, for sagging bending, and the value at the supporting steel section, for hogging bending;

$k_2$ is the flexural stiffness of the steel web, to be taken as:

$$k_2 = \frac{E_u t_w^3}{4(1-\nu_a^2) h_s}$$

(6.10)

for an uncased steel beam,

where:

$\nu_a$ is Poisson’s ratio for structural steel and $h_s$ and $t_w$ are defined in Figure 6.11.

(7) For a steel beam with partial encasement in accordance with 5.5.3(2), the flexural stiffness $k_2$ may take account of the encasement and be calculated by:

$$k_2 = \frac{E_u t_w \frac{b_c^2}{16h_s(1+4nt_w/b_c)}}$$

(6.11)

where:

$n$ is the modular ratio for long-term effects according to 5.4.2.2, and
\( b_c \) is the width of the concrete encasement, see Figure 6.8.

(8) In the U-frame model, the favourable effect of the St. Venant torsional stiffness \( G_a I_{at} \) of the steel section may be taken into account for the calculation of \( M_{cr} \).

(9) For a partially-encased steel beam with encasement reinforced either with open stirrups attached to the web or with closed stirrups, the torsional stiffness of the encasement may be added to the value \( G_a I_{at} \) for the steel section. This additional torsional stiffness should be taken as \( G_c I_{ct} / 10 \), where \( G_c \) is the shear modulus for concrete, which may be taken as \( 0.3 E_a / n \) (where \( n \) is the modular ratio for long-term effects), and \( I_{ct} \) is the St. Venant torsion constant of the encasement, assuming it to be un-cracked and of breadth equal to the overall width of the encasement.

6.4.3 Simplified verification for buildings without direct calculation

(1) A continuous beam (or a beam within a frame that is composite throughout its length) with Class 1, 2 or 3 cross-sections may be designed without additional lateral bracing when the following conditions are satisfied:

a) Adjacent spans do not differ in length by more than 20% of the shorter span. Where there is a cantilever, its length does not exceed 15% of that of the adjacent span.

b) The loading on each span is uniformly distributed, and the design permanent load exceeds 40% of the total design load.

c) The top flange of the steel member is attached to a reinforced concrete or composite slab by shear connectors in accordance with 6.6.

d) The same slab is also attached to another supporting member approximately parallel to the composite beam considered, to form an inverted-U frame as illustrated in Figure 6.11.

e) If the slab is composite, it spans between the two supporting members of the inverted-U frame considered.

f) At each support of the steel member, its bottom flange is laterally restrained and its web is stiffened. Elsewhere, the web may be un-stiffened.

g) If the steel member is an IPE section or an HE section that is not partially encased, its depth \( h \) does not exceed the limit given in Table 6.1.

h) If the steel member is partially encased in concrete according to 5.5.3(2), its depth \( h \) does not exceed the limit given in Table 6.1 by more than 200 mm for steel grades up to S355 and by 150 mm for grades S420 and S460.

Note: Provisions for other types of steel section may be given in the National Annex.

Table 6.1 : Maximum depth \( h \) (mm) of uncased steel member for which clause 6.4.3 is applicable

<table>
<thead>
<tr>
<th>Steel member</th>
<th>Nominal steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S 235</td>
</tr>
<tr>
<td>IPE</td>
<td>600</td>
</tr>
</tbody>
</table>
6.5 Transverse forces on webs

6.5.1 General

(1) The rules given in EN 1993-1-5, 6 to determine the design resistance of an unstiffened or stiffened web to transverse forces applied through a flange are applicable to the non-composite steel flange of a composite beam, and to the adjacent part of the web.

(2) If the transverse force acts in combination with bending and axial force, the resistance should be verified according to EN 1993-1-5, 7.2.

(3) For buildings, at an internal support of a beam designed using an effective web in Class 2 in accordance with 5.5.2(3), transverse stiffening should be provided unless it has been verified that the un-stiffened web has sufficient resistance to crippling and buckling.

6.5.2 Flange-induced buckling of webs

(1) EN 1993-1-5, 8 is applicable provided that area $A_{fc}$ is taken equal to the area of the non-composite steel flange or the transformed area of the composite steel flange taking into account the modular ratio for short-term loading, whichever is the smaller.

6.6 Shear connection

6.6.1 General

6.6.1.1 Basis of design

(1) Clause 6.6 is applicable to composite beams and, as appropriate, to other types of composite member.

(2) Shear connection and transverse reinforcement shall be provided to transmit the longitudinal shear force between the concrete and the structural steel element, ignoring the effect of natural bond between the two.

(3) Shear connectors shall have sufficient deformation capacity to justify any inelastic redistribution of shear assumed in design.

(4) Ductile connectors are those with sufficient deformation capacity to justify the assumption of ideal plastic behaviour of the shear connection in the structure considered.

(5) A connector may be taken as ductile if the characteristic slip capacity $\delta_{uk}$ is at least 6mm.

Note: An evaluation of $\delta_{uk}$ is given in Annex B.
(6)P Where two or more different types of shear connection are used within the same span of a beam, account shall be taken of any significant difference in their load-slip properties.

(7)P Shear connectors shall be capable of preventing separation of the concrete element from the steel element, except where separation is prevented by other means.

(8) To prevent separation of the slab, shear connectors should be designed to resist a nominal ultimate tensile force, perpendicular to the plane of the steel flange, of at least 0,1 times the design ultimate shear resistance of the connectors. If necessary they should be supplemented by anchoring devices.

(9) Headed stud shear connectors in accordance with 6.6.5.7 may be assumed to provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension.

(10)P Longitudinal shear failure and splitting of the concrete slab due to concentrated forces applied by the connectors shall be prevented.

(11) If the detailing of the shear connection is in accordance with the appropriate provisions of 6.6.5 and the transverse reinforcement is in accordance with 6.6.6, compliance with 6.6.1.1(10) may be assumed.

(12) Where a method of interconnection, other than the shear connectors included in 6.6, is used to transfer shear between a steel element and a concrete element, the behaviour assumed in design should be based on tests and supported by a conceptual model. The design of the composite member should conform to the design of a similar member employing shear connectors included in 6.6, in so far as practicable.

(13) For buildings, the number of connectors should be at least equal to the total design shear force for the ultimate limit state, determined according to 6.6.2, divided by the design resistance of a single connector \( P_{Rd} \). For stud connectors the design resistance should be determined according to 6.6.3 or 6.6.4, as appropriate.

(14)P If all cross-sections are in Class 1 or Class 2, in buildings partial shear connection may be used for beams. The number of connectors shall then be determined by a partial connection theory taking into account the deformation capacity of the shear connectors.

### 6.6.1.2 Limitation on the use of partial shear connection in beams for buildings

(1) Headed studs with an overall length after welding not less than 4 times the diameter, and with a shank of nominal diameter not less than 16 mm and not greater than 25 mm, may be considered as ductile within the following limits for the degree of shear connection, which is defined by the ratio \( \eta = n / n_t \):

For steel sections with equal flanges:

\[
L_e \leq 25: \quad \eta \geq 1 - \left( \frac{355}{f_y} \right) (0,75 - 0,03 L_e), \quad \eta \geq 0,4 \tag{6.12}
\]

\[
L_e > 25: \quad \eta \geq 1 \tag{6.13}
\]
For steel sections having a bottom flange with an area equal to three times the area of the top flange:

\[ L_{e} \leq 20: \eta \geq 1 - \left( \frac{355}{f_{y}} \right) (0.30 - 0.015 L_{e}), \quad \eta \geq 0.4 \quad (6.14) \]

\[ L_{e} > 20: \eta \geq 1 \quad (6.15) \]

where:

- \( L_{e} \) is the distance in sagging bending between points of zero bending moment in metres; for typical continuous beams, \( L_{e} \) may be assumed to be as shown in Figure 5.1;
- \( n_{f} \) is the number of connectors for full shear connection determined for that length of beam in accordance with 6.6.1.1(13) and 6.6.2.2(2);
- \( n \) is the number of shear connectors provided within that same length.

(2) For steel sections having a bottom flange with an area exceeding the area of the top flange but less than three times that area, the limit for \( \eta \) may be determined from expressions (6.12) – (6.15) by linear interpolation.

(3) Headed stud connectors may be considered as ductile over a wider range of spans than given in (1) above where:

- (a) the studs have an overall length after welding not less than 76 mm, and a shank of nominal diameter of 19 mm,
- (b) the steel section is a rolled or welded I or H with equal flanges,
- (c) the concrete slab is composite with profiled steel sheeting that spans perpendicular to the beam and the concrete ribs are continuous across it,
- (d) there is one stud per rib of sheeting, placed either centrally within the rib or alternately on the left side and on the right side of the trough throughout the length of the span,
- (e) for the sheeting \( b_{0} / h_{p} \geq 2 \) and \( h_{p} \leq 60 \) mm, where the notation is as in Figure 6.13 and
- (f) the force \( N_{c} \) is calculated in accordance with the simplified method given in Figure 6.5.

Where these conditions are satisfied, the ratio \( \eta \) should satisfy:

\[ L_{e} \leq 25: \eta \geq 1 - \left( \frac{355}{f_{y}} \right) (1.0 - 0.04 L_{e}), \quad \eta \geq 0.4 \quad (6.16) \]

\[ L_{e} > 25: \eta \geq 1 \quad (6.17) \]

Note: The requirements in 6.6.1.2 are derived for uniform spacing of shear connectors.

### 6.6.1.3 Spacing of shear connectors in beams for buildings
The shear connectors shall be spaced along the beam so as to transmit longitudinal shear and to prevent separation between the concrete and the steel beam, considering an appropriate distribution of design longitudinal shear force.

In cantilevers and hogging moment regions of continuous beams, tension reinforcement should be curtailed to suit the spacing of the shear connectors and should be adequately anchored.

Ductile connectors may be spaced uniformly over a length between adjacent critical cross-sections as defined in 6.1.1 provided that:

- all critical sections in the span considered are in Class 1 or Class 2,
- \( \eta \) satisfies the limit given by 6.6.1.2 and
- the plastic resistance moment of the composite section does not exceed 2.5 times the plastic resistance moment of the steel member alone.

If the plastic resistance moment exceeds 2.5 times the plastic resistance moment of the steel member alone, additional checks on the adequacy of the shear connection should be made at intermediate points approximately mid-way between adjacent critical cross-sections.

The required number of shear connectors may be distributed between a point of maximum sagging bending moment and an adjacent support or point of maximum hogging moment, in accordance with the longitudinal shear calculated by elastic theory for the loading considered. Where this is done, no additional checks on the adequacy of the shear connection are required.

### 6.6.2 Longitudinal shear force in beams for buildings

#### 6.6.2.1 Beams in which non-linear or elastic theory is used for resistances of one or more cross-sections

(1) If non-linear or elastic theory is applied to cross-sections, the longitudinal shear force should be determined in a manner consistent with 6.2.1.4 or 6.2.1.5 respectively.

#### 6.6.2.2 Beams in which plastic theory is used for resistance of cross sections

(1) The total design longitudinal shear shall be determined in a manner consistent with the design bending resistance, taking account of the difference in the normal force in concrete or structural steel over a critical length.

(2) For full shear connection, reference should be made to 6.2.1.2, or 6.3.2, as appropriate.

(3) For partial shear connection, reference should be made to 6.2.1.3 or 6.3.2, as appropriate.

### 6.6.3 Headed stud connectors in solid slabs and concrete encasement

#### 6.6.3.1 Design resistance
(1) The design shear resistance of a headed stud automatically welded in accordance with EN 14555 should be determined from:

\[ P_{Rd} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} \]  
(6.18)

or:

\[ P_{Rd} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \]  
(6.19)

whichever is smaller, with:

\[ \alpha = 0.2 \left( \frac{h_{sc}}{d} + 1 \right) \text{ for } 3 \leq h_{sc} / d \leq 4 \]  
(6.20)

\[ \alpha = 1 \text{ for } h_{sc} / d > 4 \]  
(6.21)

where:

\( \gamma_V \) is the partial factor;
\( d \) is the diameter of the shank of the stud, 16 mm \( \leq d \leq 25 \) mm;
\( f_u \) is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm\(^2\);
\( f_{ck} \) is the characteristic cylinder compressive strength of the concrete at the age considered, of density not less than 1750 kg/m\(^3\);
\( h_{sc} \) is the overall nominal height of the stud.

Note: The value for \( \gamma_V \) may be given in the National Annex. The recommended value for \( \gamma_V \) is 1.25.

(2) The weld collars should comply with the requirements of EN 13918.

(3) Where studs are arranged in a way such that splitting forces occur in the direction of the slab thickness, (1) is not applicable.

Note: For buildings, further information may be given in the National Annex.

**6.6.3.2 Influence of tension on shear resistance**

(1) Where headed stud connectors are subjected to direct tensile force in addition to shear, the design tensile force per stud \( F_{ten} \) should be calculated.

(2) If \( F_{ten} \leq 0.1 P_{Rd} \), where \( P_{Rd} \) is the design shear resistance defined in 6.6.3.1, the tensile force may be neglected.

(3) If \( F_{ten} > 0.1 P_{Rd} \), the connection is not within the scope of EN 1994.
6.6.4 Design resistance of headed studs used with profiled steel sheeting in buildings

6.6.4.1 Sheeting with ribs parallel to the supporting beams

(1) The studs are located within a region of concrete that has the shape of a haunch, see Figure 6.12. Where the sheeting is continuous across the beam, the width of the haunch \( b_0 \) is equal to the width of the trough as given in Figure 9.2. Where the sheeting is not continuous, \( b_0 \) is defined in a similar way as given in Figure 6.12. The depth of the haunch should be taken as \( h_p \), the overall depth of the sheeting excluding embossments.

![Figure 6.12: Beam with profiled steel sheeting parallel to the beam](image)

(2) The design shear resistance should be taken as the resistance in a solid slab, see 6.6.3.1, multiplied by the reduction factor \( k_l \) given by the following expression:

\[
k_l = 0,6 \frac{b_0}{h_p} \left( \frac{h_{sc}}{h_p} - 1 \right) \leq 1,0
\]

(6.22)

where:

\( h_{sc} \) is the overall height of the stud, but not greater than \( h_p + 75 \) mm.

(3) Where the sheeting is not continuous across the beam, and is not appropriately anchored to the beam, that side of the haunch and its reinforcement should satisfy 6.6.5.4.

Note: Means to achieve appropriate anchorage may be given in the National Annex.

6.6.4.2 Sheeting with ribs transverse to the supporting beams

(1) Provided that the conditions given in (2) and (3) are satisfied, the design shear resistance should be taken as the resistance in a solid slab, calculated as given by 6.6.3.1 (except that \( f_u \) should not be taken as greater than 450 N/mm²) multiplied by the reduction factor \( k_t \) given by:

\[
k_t = \frac{0,7}{\sqrt{n_r}} \frac{b_0}{h_p} \left( \frac{h_{sc}}{h_p} - 1 \right)
\]

(6.23)

where:

\( n_r \) is the number of stud connectors in one rib at a beam intersection, not to exceed 2 in computations, and other symbols are as defined in Figure 6.13.
(2) The factor $k_t$ should not be taken greater than the appropriate value $k_{t,\text{max}}$ given in Table 6.2.

Table 6.2: Upper limits $k_{t,\text{max}}$ for the reduction factor $k_t$

<table>
<thead>
<tr>
<th>Number of stud connectors per rib</th>
<th>Thickness $t$ of sheet (mm)</th>
<th>Studs not exceeding 20 mm in diameter and welded through profiled steel sheeting</th>
<th>Profiled sheeting with holes and studs 19 mm or 22 mm in diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_r = 1$</td>
<td>$\leq 1,0$</td>
<td>0,85</td>
<td>0,75</td>
</tr>
<tr>
<td></td>
<td>$&gt; 1,0$</td>
<td>1,0</td>
<td>0,75</td>
</tr>
<tr>
<td>$n_r = 2$</td>
<td>$\leq 1,0$</td>
<td>0,70</td>
<td>0,60</td>
</tr>
<tr>
<td></td>
<td>$&gt; 1,0$</td>
<td>0,8</td>
<td>0,60</td>
</tr>
</tbody>
</table>

(3) The values for $k_t$ given by (1) and (2) are applicable provided that:

– the studs are placed in ribs with a height $h_p$ not greater than 85 mm and a width $b_0$ not less than $h_p$ and
– for through deck welding, the diameter of the studs is not greater than 20 mm, or
– for holes provided in the sheeting, the diameter of the studs is not greater than 22 mm.

6.6.4.3 Biaxial loading of shear connectors

(1) Where the shear connectors are provided to produce composite action both for the beam and for the composite slab, the combination of forces acting on the stud should satisfy the following:

$$\frac{F_i^2}{P_{i,Rd}^2} + \frac{F_i^2}{P_{i,Rd}^2} \leq 1$$

(6.24)
where:

- $F_l$ is the design longitudinal force caused by composite action in the beam;
- $F_t$ is the design transverse force caused by composite action in the slab, see Section 9;
- $P_{l,Rd}$ and $P_{t,Rd}$ are the corresponding design shear resistances of the stud.

### 6.6.5 Detailing of the shear connection and influence of execution

#### 6.6.5.1 Resistance to separation

(1) The surface of a connector that resists separation forces (for example, the underside of the head of a stud) should extend not less than 30 mm clear above the bottom reinforcement, see Figure 6.14.

#### 6.6.5.2 Cover and concreting

(1) The detailing of shear connectors shall be such that concrete can be adequately compacted around the base of the connector.

(2) If cover over the connector is required, the minimum cover should be:
   a) not less than 20 mm, or
   b) as recommended by EN 1992-1-1, Table 4.4 for reinforcing steel, less 5 mm, whichever is the greater.

(3) If cover is not required the top of the connector may be flush with the upper surface of the concrete slab.

(4) In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations. Wherever possible, deformation should not be imposed on a shear connection until the concrete has reached a cylinder strength of at least 20 N/mm².

#### 6.6.5.3 Local reinforcement in the slab

(1) Where the shear connection is adjacent to a longitudinal edge of a concrete slab, transverse reinforcement provided in accordance with 6.6.6 should be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

(2) To prevent longitudinal splitting of the concrete flange caused by the shear connectors, the following additional recommendations should be applied where the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm:
   a) transverse reinforcement should be supplied by U-bars passing around the shear connectors,
b) where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than 6\(d\), where \(d\) is the nominal diameter of the stud, and the U-bars should be not less than 0.5\(d\) in diameter and

c) the U-bars should be placed as low as possible while still providing sufficient bottom cover.

(3)P At the end of a composite cantilever, sufficient local reinforcement shall be provided to transfer forces from the shear connectors to the longitudinal reinforcement.

### 6.6.5.4 Haunches other than formed by profiled steel sheeting

(1) Where a concrete haunch is used between the steel section and the soffit of the concrete slab, the sides of the haunch should lie outside a line drawn at 45° from the outside edge of the connector, see Figure 6.14.

![Figure 6.14 : Detailing](image)

(2) The concrete cover from the side of the haunch to the connector should be not less than 50 mm.

(3) Transverse reinforcing bars sufficient to satisfy the requirements of 6.6.6 should be provided in the haunch at not less than 40 mm clear below the surface of the connector that resists uplift.

### 6.6.5.5 Spacing of connectors

(1)P Where it is assumed in design that the stability of either the steel or the concrete member is ensured by the connection between the two, the spacing of the shear connectors shall be sufficiently close for this assumption to be valid.

(2) Where a steel compression flange that would otherwise be in a lower class is assumed to be in Class 1 or Class 2 because of restraint from shear connectors, the centre-to-centre spacing of the shear connectors in the direction of compression should be not greater than the following limits:

- where the slab is in contact over the full length (e.g. solid slab): \(22 \frac{t_f}{\sqrt{235/f_y}}\)
– where the slab is not in contact over the full length (e.g. slab with ribs transverse to the beam): $15 t_f \sqrt{235/f_y}$

where:

$t_f$ is the thickness of the flange;

$f_y$ is the nominal yield strength of the flange in N/mm$^2$.

In addition, the clear distance from the edge of a compression flange to the nearest line of shear connectors should be not greater than $9 t_f \sqrt{235/f_y}$.

(3) In buildings, the maximum longitudinal centre-to-centre spacing of shear connectors should be not greater than 6 times the total slab thickness nor 800 mm.

6.6.5.6 Dimensions of the steel flange

(1) The thickness of the steel plate or flange to which a connector is welded shall be sufficient to allow proper welding and proper transfer of load from the connector to the plate without local failure or excessive deformation.

(2) In buildings, the distance $e_D$ between the edge of a connector and the edge of the flange of the beam to which it is welded, see Figure 6.14, should be not less than 20 mm.

6.6.5.7 Headed stud connectors

(1) The overall height of a stud should be not less than $3d$, where $d$ is the diameter of the shank.

(2) The head should have a diameter of not less than $1.5d$ and a depth of not less than $0.4d$.

(3) For elements in tension and subjected to fatigue loading, the diameter of a welded stud should not exceed 1.5 times the thickness of the flange to which it is welded, unless test information is provided to establish the fatigue resistance of the stud as a shear connector. This applies also to studs directly over a web.

(4) The spacing of studs in the direction of the shear force should be not less than $5d$; the spacing in the direction transverse to the shear force should be not less than $2.5d$ in solid slabs and $4d$ in other cases.

(5) Except when the studs are located directly over the web, the diameter of a welded stud should be not greater than 2.5 times the thickness of that part to which it is welded, unless test information is provided to establish the resistance of the stud as a shear connector.

6.6.5.8 Headed studs used with profiled steel sheeting in buildings

(1) The nominal height of a connector should extend not less than $2d$ above the top of the steel deck, where $d$ is the diameter of the shank.
(2) The minimum width of the troughs that are to be filled with concrete should be not less than 50 mm.

(3) Where the sheeting is such that studs cannot be placed centrally within a trough, they should be placed alternately on the two sides of the trough, throughout the length of the span.

6.6.6 Longitudinal shear in concrete slabs

6.6.6.1 General

(1) Transverse reinforcement in the slab shall be designed for the ultimate limit state so that premature longitudinal shear failure or longitudinal splitting shall be prevented.

(2) The design longitudinal shear stress for any potential surface of longitudinal shear failure within the slab $\gamma_{Ed}$ shall not exceed the design longitudinal shear strength of the shear surface considered.

![Figure 6.15: Typical potential surfaces of shear failure](image)

<table>
<thead>
<tr>
<th>Type</th>
<th>$A_{sf}/s_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a-a</td>
<td>$A_b + A_t$</td>
</tr>
<tr>
<td>b-b</td>
<td>$2A_b$</td>
</tr>
<tr>
<td>c-c</td>
<td>$2A_b$</td>
</tr>
<tr>
<td>d-d</td>
<td>$2A_{bh}$</td>
</tr>
</tbody>
</table>

(3) The length of the shear surface b-b shown in Figure 6.15 should be taken as equal to $2h_{sc}$ plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to $(2h_{sc} + s_t)$ plus the head diameter for stud shear connectors arranged in pairs, where $h_{sc}$ is the height of the studs and $s_t$ is the transverse spacing centre-to-centre of the studs.

(4) The design longitudinal shear per unit length of beam on a shear surface should be determined in accordance with 6.6.2 and be consistent with the design and spacing of the shear connectors. Account may be taken of the variation of longitudinal shear across the width of the concrete flange.
(5) The design longitudinal shear stress $v_{Ed}$ should be taken as the design longitudinal shear per unit length of beam divided by the length $h_f$ of the shear surface considered.

**6.6.6.2 Design resistance to longitudinal shear**

(1) The design shear strength of the concrete flange (shear planes a-a illustrated in Figure 6.15) should be determined in accordance with EN 1992-1-1, 6.2.4.

(2) In the absence of a more accurate calculation the design shear strength of any surface of potential shear failure in the flange or a haunch may be determined from EN 1992-1-1, 6.2.4(4). For a shear surface passing around the shear connectors (e.g. shear surface b-b in Figure 6.15), the dimension $h_f$ should be taken as the length of the shear surface.

(3) The effective transverse reinforcement per unit length, $A_{sf} / s_f$ in EN 1992-1-1, should be as shown in Figure 6.15, in which $A_b$, $A_t$ and $A_{bh}$ are areas of reinforcement per unit length of beam anchored in accordance with EN 1992-1-1, 8.4 for longitudinal reinforcement.

(4) Where a combination of pre-cast elements and in-situ concrete is used, the resistance to longitudinal shear should be determined in accordance with EN 1992-1-1, 6.2.5.

**6.6.6.3 Minimum transverse reinforcement**

(1) The minimum area of reinforcement should be determined in accordance with EN 1992-1-1, 9.2.2(5) using definitions appropriate to transverse reinforcement.

**6.6.6.4 Longitudinal shear and transverse reinforcement in beams for buildings**

(1) Where profiled steel sheeting is used and the shear surface passes through the depth of the slab (e.g. shear surface a-a in Figure 6.16), the dimension $h_f$ should be taken as the thickness of the concrete above the sheeting.

(2) Where profiled steel sheeting is used transverse to the beam and the design resistances of the studs are determined using the appropriate reduction factor $k_t$ as given in 6.6.4.2, it is not necessary to consider shear surfaces of type b-b in Figure 6.16.

(3) Unless verified by tests, for surfaces of type c-c in Figure 6.16 the depth of the sheeting should not be included in $h_f$.

(4) Where profiled steel sheeting with mechanical or frictional interlock and with ribs transverse to the beam is continuous across the top flange of the steel beam, its contribution to the transverse reinforcement for a shear surface of type a-a may be allowed for by replacing expression (6.21) in EN 1992-1-1, 6.2.4(4) by:

$$A_{sf}f_y / s_f + A_p f_{y_p,d} > v_{Ed} h_f / \cot \theta$$

where:

- $A_p$ is the cross-sectional area of the profiled steel sheeting per unit length of the beam; for sheeting with holes, the net area should be used;
(5) Where the profiled steel sheeting with ribs transverse to the beam is discontinuous across the top flange of the steel beam, and stud shear connectors are welded to the steel beam directly through the profiled steel sheets, the term $A_p f_{yp,d}$ in expression (6.25) should be replaced by:

$$P_{pb,Rd} \times s \leq A_p f_{yp,d}$$  \hspace{1cm} (6.26)

where:

- $P_{pb,Rd}$ is the design bearing resistance of a headed stud welded through the sheet according to 9.7.4;
- $s$ is the longitudinal spacing centre-to-centre of the studs effective in anchoring the sheeting.

(6) With profiled steel sheeting, the requirement for minimum reinforcement relates to the area of concrete above the sheeting.

### 6.7 Composite columns and composite compression members

#### 6.7.1 General

(1) This clause applies for the design of composite columns and composite compression members with concrete encased sections, partially encased sections and concrete filled rectangular and circular tubes, see Figure 6.17.

(2) This clause applies to columns and compression members with steel grades S235 to S460 and normal weight concrete of strength classes C20/25 to C50/60.
(3) This clause applies to isolated columns and columns and composite compression members in framed structures where the other structural members are either composite or steel members.

(4) The steel contribution ratio $\delta$ should fulfil the following condition:

$$0.2 \leq \delta \leq 0.9$$

(6.27)

where:

$\delta$ is defined in 6.7.3.3(1).

(5) Composite columns or compression members of any cross-section should be checked for:

- resistance of the member in accordance with 6.7.2 or 6.7.3,
- resistance to local buckling in accordance with (8) and (9) below,
- introduction of loads in accordance with 6.7.4.2 and
- resistance to shear between steel and concrete elements in accordance with 6.7.4.3.

(6) Two methods of design are given:

- a general method in 6.7.2 whose scope includes members with non-symmetrical or non-uniform cross-sections over the column length and
- a simplified method in 6.7.3 for members of doubly symmetrical and uniform cross section over the member length.
(7) For composite compression members subjected to bending moments and normal forces resulting from independent actions, the partial factor $\gamma_F$ for those internal forces that lead to an increase of resistance should be reduced by 20%.

(8) The influence of local buckling of the steel section on the resistance shall be considered in design.

(9) The effects of local buckling may be neglected for a steel section fully encased in accordance with 6.7.5.1(2), and for other types of cross-section provided the maximum values of Table 6.3 are not exceeded.

**Table 6.3 : Maximum values (d/t), (h/t) and (b/t) with $f_y$ in N/mm²**

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Max (d/t), max (h/t) and max (b/t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular hollow steel sections</td>
<td>max (d/t) = 90 $\frac{235}{f_y}$</td>
</tr>
<tr>
<td>Rectangular hollow steel sections</td>
<td>max (h/t) = 52 $\frac{235}{\sqrt{f_y}}$</td>
</tr>
<tr>
<td>Partially encased I-sections</td>
<td>max (b/t) = 44 $\frac{235}{\sqrt{f_y}}$</td>
</tr>
</tbody>
</table>

**6.7.2 General method of design**

(1) Design for structural stability shall take account of second-order effects including residual stresses, geometrical imperfections, local instability, cracking of concrete, creep and shrinkage of concrete and yielding of structural steel and of reinforcement. The design shall ensure that instability does not occur for the most unfavourable combination of actions at the ultimate limit state and that the resistance of individual cross-sections subjected to bending, longitudinal force and shear is not exceeded.

(2) Second-order effects shall be considered in any direction in which failure might occur, if they affect the structural stability significantly.

(3) Internal forces shall be determined by elasto-plastic analysis.