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

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

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

ICS:

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| CCMC will prepare and attach the official title page. |

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European foreword

This document (prEN 1994-1-1:2024) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

This document is currently submitted to CEN Enquiry.

This document will supersede EN 1994-1-1:2004.

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

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

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

0 Introduction

**0.1 Introduction to the Eurocodes**

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

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

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

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

**0.2 Introduction to EN 1994 (all parts)**

EN 1994 applies to the design of steel and concrete composite structures and those who undertake building 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 given in EN 1990, *Eurocode* — *Basis of structural and geotechnical design*.

EN 1994 is concerned only with requirements for resistance, serviceability, durability and fire resistance of steel and concrete composite structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

EN 1994 is subdivided in various parts:

EN 1994-1-1, *Eurocode 4 — Design of composite steel and concrete structures — Part 1 1: General rules and rules for buildings*;

EN 1994-1-2*, Eurocode 4 —Design of composite steel and concrete structures — Part 1 2: Structural fire design*;

EN 1994-2, *Eurocode 4 — Design of composite steel and concrete structures — Part 2: Bridges.*

**0.3 Introduction to EN 1994–1–1**

EN 1994–1–1 gives basic rules for the design of steel and concrete composite structures and supplementary provisions specific for buildings.

**0.4 Verbal forms used in the Eurocodes**

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

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

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

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

**0.5 National Annex for EN 1994–1–1**

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

The national standard implementing EN 1994–1–1 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works relevant to each country.

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

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

National choice is allowed in EN 1994–1–1 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.4.1.2(2) | 4.4.1.2(4) | 4.4.1.2(5) | 4.4.1.2(6) |
| 5.1(3) | 5.1(7) | 5.4.2.1(5) | 8.2.2.5(1) |
| 8.6.8.1(1) | 8.6.9.1(3) | 8.8.2(9) | 10.6(2) |
| 10.7.5(7) | B.2.2.3(3) | D.4.1.3(5) | H.2(2) |
| H.2(3) | H.2(4) |  |  |

National choice is allowed in EN 1994-1-1 on the application of the following informative annexes:

|  |  |  |  |
| --- | --- | --- | --- |
| Annex A | Annex C | Annex E | Annex G |
| Annex J |  |  |  |

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

# Scope

## Scope of EN 1994-1-1

(1) EN 1994–1–1 gives basic rules for the design of steel and concrete composite structures and supplementary provisions specific for buildings.

NOTE Specific rules for bridges are given in EN 1994–2.

## Assumptions

(1) The assumptions of EN 1990 apply to EN 1994-1-1.

(2) In addition to the general assumptions of EN 1990, the assumptions given in EN 1992–1–1, EN 1992—1-2, and EN 1993–1–1 apply to this document.

(3) EN 1994–1–1 is intended to be used in conjunction with EN 1990, EN 1991 (all parts), EN 1992-1-1, EN 1993 (all parts), EN 1997 (all parts), EN 1998 (all parts when steel and concrete composite structures are built in seismic regions), EN 1090-1, EN 1090-2, EN 1090-4, EN 13670 and ENs for construction products relevant to steel and concrete composite structures.

# Normative references

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

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

EN 1990:2023[[1]](#footnote-2), Eurocode — *Basis of structural and geotechnical design*

EN 1991 (all parts), *Eurocode 1 — Actions on structures*

EN 1991-1-5, *Eurocode 1 — Actions on structures – Part 1-5: Thermal Actions*

EN 1992-1-1:2023, *Eurocode 2* *— Design of concrete structures* *– Part 1-1: General rules and rules for buildings*

EN 1993-1-1:2022, *Eurocode 3 — Design of steel structures – Part 1-1: General rules and rules for buildings*

FprEN 1993‑1‑8:2023, *Eurocode 3 — Design of steel structures — Part 1-8: Joints*

prEN 1993-1-14:2023, *Eurocode 3 — Design of steel structures — Part 1-14: Design assisted by finite element analysis*

# Terms, definitions and symbols

## Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, EN 1992-1-1 and EN 1993-1-1 and the following apply.

3.1.1

composite member

structural member with components of concrete and of structural 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

3.1.2

shear connection

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

3.1.3

shear connector

connector used in composite members to enable the combined action of the concrete and steel components

3.1.4

composite behaviour

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

3.1.5

composite beam

composite member subjected mainly to bending, composed of a structural steel section with one or two concrete flanges or a partially-encased structural steel section with or without concrete flanges

3.1.6

composite column

composite member subjected mainly to compression or to compression and bending

3.1.7

composite slab

slab in which a profiled steel sheet is used initially as permanent formwork and subsequently combines structurally with the hardened concrete to act as tensile reinforcement in the finished floor

3.1.8

colid slab

slab with a constant depth of concrete

3.1.9

concrete slab

either a composite slab or a solid slab

3.1.10

concrete flange

concrete slab in combined action with the steel component, acting as compression or tension flange

3.1.11

locally stiff concrete slab

concrete flange of a composite beam at a web opening where the relative flexural stiffness of the concrete flange relative to the steel element significantly modifies the distribution of shear forces between the steel and concrete

3.1.12

composite frame

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

3.1.13

composite joint

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

3.1.14

propped structure or member

structure or member where the weight of concrete elements is applied to steel elements which are subject to temporary support in the span, or is carried independently through temporary supports until the concrete elements are able to resist stresses

3.1.15

un‑propped structure or member

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

3.1.16

un‑cracked flexural stiffness

the stiffness *E*a *I*1 of a cross‑section of a composite member where *I*1 is the second moment of area of the transformed composite cross-section calculated assuming that any concrete in tension is un‑cracked and that the total cross-section remains plane

3.1.17

cracked flexural stiffness

stiffness *E*a *I*2 of a cross‑section of a composite member where *I*2 is the second moment of area of the transformed composite cross section-section calculated neglecting any concrete in tension but including reinforcement and assuming that the total cross-section remains plane

3.1.18

prestress

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

3.1.19

braced members or systems

structural members or subsystems, which in analysis and design are assumed not to contribute to the overall horizontal stability of a structure

3.1.20

elastic resistance

resistance of a cross-section based on linear elastic theory

3.1.21

plastic resistance

resistance of a cross-section, based on rectangular stress blocks

3.1.22

non-linear resistance

resistance at a cross-section level, which includes several types of non-linear cross-section analyses such as the determination of the moment resistance considering plastic resistance or based on strain limitation and the materials stress-strain curve

3.1.23

global analysis

describes the analysis of the structure or system as a whole,

* elastic global analysis is based on the assumption that the stress-strain behaviour of the materials is linear, whatever the stress level, even if the resistance of a cross-section is based on its plastic or non-linear resistance.
* rigid plastic analysis neglects the effects of elastic deflections and assumes that all structural deformation takes place in discrete plastic hinge regions within the members or joints.
* non-linear plastic global analysis considers the spread of plasticity both through the cross-sections and along the members in plastic zones. This term also applies to a plastic analysis considering the impact of joints

Note 1 to entry: The purpose of global analysis is to determine deformations, internal forces and moments in beams, columns and framed structures.

3.1.24

member imperfection

member imperfections include geometrical and structural imperfections

3.1.25

equivalent bow imperfection

equivalent bow imperfections specified in this standard take into account the effects of:

* geometrical imperfections as limited by geometrical tolerances in product standards or the execution standard;
* structural imperfections due to fabrication and erection, e.g. residual stresses and variation of the yield strength

3.1.26

full shear connection

span of a beam has full shear connection when an increase in the number of shear connectors would not increase the design bending resistance of the member. Otherwise, the shear connection is partial

3.1.27

partial shear connection

when the condition for full shear connection in accordance with 3.1.26 is not fulfilled

3.1.28

full interaction

when a composite element has full interaction, the slip in the composite connection between steel and concrete may be neglected. Thus only one neutral axis exists for the composite cross-section and the cross-section remains plane

3.1.29

double composite action

section where the steel member acts compositely with concrete flanges at both the top and bottom

3.1.30

transformed composite cross-section

composite cross-section with an effective (transformed) concrete area modified by the modular ratio *n*L in accordance with 7.4.2.2(2), Formula (7.4) taking into account the effects from the type of loading and the time effects of creep and shrinkage for linear elastic analysis

3.1.31

composite hollow core slab

precast hollow core slab floor complemented by a cast in-situ topping

3.1.32

core

longitudinal void in precast hollow core slabs produced by specific industrial manufacturing techniques, located with a regular pattern and the shape of which is such that the vertical loading applied on the slab is transmitted to the webs

3.1.33

hollow core slab

prestressed or reinforced concrete element with a constant depth divided into an upper and a lower flange, linked by vertical webs, so constituting cores as longitudinal voids, the cross-section of which is constant and presents one vertical symmetric axis

3.1.34

hollow core slab floor

floor made of precast hollow core slabs after the grouting of the joints

3.1.35

open core

core with upper flange removed to receive transverse reinforcement bar and cast in-situ infill

3.1.36

precast floor plate

reinforced or prestressed concrete floor plate used as permanent formwork for cast-in-situ concrete, flat or with ribs, with or without lattice girders, but without void formers

3.1.37

precast product

precast concrete element manufactured in compliance with a specific EN standard

3.1.38

solid composite slab

slab comprising a precast concrete floor plate and bonded topping, which behave as a monolithic slab after hardening of the topping through the bond between the precast element and topping with or without connecting reinforcement

3.1.39

topping

in-situ concrete layer cast over the entire precast floor surface, so that it acts monolithically by bond intended to increase its bearing capacity and so constituting a composite slab floor

3.1.40

**European Technical Product Specification**

— a European Product Standard (EN),

— or a European Technical Assessment (ETA) based on a European Assessment Document (EAD),

— or a product documentation based on a transparent and reproducible assessment that complies with all requirements of the relevant EAD

## Symbols

For the purposes of this document, the following symbols apply.

### Latin upper case letters

|  |  |
| --- | --- |
| *A* | Cross sectional area of the effective composite section neglecting concrete in tension |
| *A*a | Cross sectional area of the structural steel section |
| *A*b | Cross sectional area of the bottom transverse reinforcement |
| *A*bh | Cross sectional area of the bottom transverse reinforcement in a haunch |
| *A*c | Cross sectional area of concrete |
| *A*c,eff | Effective cross sectional area of concrete considering the modular ratio |
| *A*c,L | Area of concrete considering time dependent effects |
| *A*ct | Cross sectional area of the tensile zone of the concrete |
| *A*fb | Cross sectional area of the bottom flange of the steel section |
| *A*fc | Effective cross-section area of the compression flange |
| *A*ft | Cross sectional area of the top flange of the steel section |
| *A*i,s | Area of the effective equivalent steel section for shrinkage actions |
| *A*i,0 | Area of the effective equivalent steel section for non-permanent actions |
| *A*i,P | Area of the effective equivalent steel section for permanent actions |
| *A*pe | Effective cross sectional area of profiled steel sheeting |
| *A*s | Cross sectional area of reinforcement |
| *A*sf | Cross sectional area of transverse reinforcement |
| *A*t | Cross sectional area of the top transverse reinforcement |
| *A*v | Shear area of a structural steel section |
| *A*vc | Shear area of a column web panel |
| *A*1 | Loaded area under the gusset plate |
| *E*a | Modulus of elasticity of structural steel |
| *E*cd | Design value of modulus of elasticity of concrete |
| *E*cm | Secant modulus of elasticity of concrete |
| *E*c,eff | Effective modulus of elasticity of concrete accounting for creep deformations |
| *E*d | Load vector |
| *E*s | Design value of modulus of elasticity of ordinary reinforcing steel |
| *(EI)*eff | Effective flexural stiffness for calculation of relative slenderness |
| *(EI)*eff,ll | Effective flexural stiffness for use in second-order analysis |
| *(EI)*2 | Cracked flexural stiffness per unit width of the concrete or composite slab |
| *F*c,wc,c,Rd | Design value of the resistance to transverse compression of the concrete encasement to a column web |
| *F*Ed | Design force, or Design concentrated load |
| *F*ℓ,Ed | Design longitudinal force per stud caused by composite action in the beam |
| *F*ten,Ed | Design tensile force per stud |
| *F*t,Ed | Design transverse force per stud caused by composite action in the slab |
| *F*s,Ed | Design longitudinal shear force per stud |
| *G*a | Shear modulus of structural steel |
| *G*c | Shear modulus of concrete |
| *G*f | Fracture energy of concrete |
| *G*k | Characteristic value of the sum of all permanent actions |
| *I* | Second moment of area of the effective composite section neglecting concrete in tension |
| *I*a | Second moment of area of the structural steel section |
| *I*at | St. Venant torsion constant of the structural steel section |
| *I*c | Second moment of area of the uncracked concrete section |
| *I*c,L | Second moment of area of the concrete slab considering the modular ratio |
| *I*cross | Second moment of area in cross-direction assuming an uncracked concrete |
| *I*ct | St. Venant torsion constant of the uncracked concrete encasement |
| *I*i,s | Second moment of area of the effective equivalent steel section for shrinkage actions |
| *I*i,0 | Second moment of area of the effective equivalent steel section for non-permanent actions |
| *I*i,P | Second moment of area of the effective equivalent steel section for permanent actions |
| *I*L,eff | Effective second moment of area of the composite cross-section |
| *I*long | Second moment of area of the slab in the longitudinal direction assuming an uncracked concrete |
| *I*s | Second moment of area of the reinforcement |
| *I*1 | Second moment of area of the effective equivalent steel section assuming that the concrete in tension is uncracked |
| *I*2 | Second moment of area of the effective equivalent steel section neglecting concrete in tension but including reinforcement |
| *K*e, *K*e,II | Correction factors to be used in the design of composite columns |
| *K*0 | Calibration factor to be used in the design of composite columns |
| *L* | Length |
| *L*A−B | Distance between two cross-sections A and B |
| *L*bc, *L*bs | Bearing lengths |
| *L*e | Equivalent span length |
| *L*1; *L*2; *L*3 | Span lengths |
| *L*p | Distance from centre of a concentrated load to the nearest support |
| *L*V | Length of shear connection |
| *L*x | Distance from a cross section to the nearest support |
| *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 before composite action is achieved |
| *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 |
| *M*Ed,Im | Maximum first-order moment within the length of the column, including the effect of imperfections and any lateral loading not included in the global analysls |
| *M*Ed,Em | Maximum moment at the ends of the column from the global analysis including second-order effects where required by 7.2 |
| *M*Ed,max | Maximum design bending moment along the beam |
| *M*el,Rd | Design value of the elastic resistance moment of the composite section |
| *M*f,Rk | Characteristic plastic moment of resistance of the cross-section consisting of the effective area of the flanges only |
| *M*i,Ed | Part of the design bending moment on the composite section |
| *M*i,D,Ed | Design value of bending moment from prestressing by imposed deformation on the transformed composite cross-section |
| *M*i,L,Ed | Design value of bending moment from actions on the time dependant transformed composite cross-section |
| *M*i,P,Ed | Design value of bending moment from permanent actions on the transformed composite cross-section |
| *M*i,PT,Ed | Design value of bending moment developing due to creep |
| *M*i,sh,Ed | Design value of bending moment from shrinkage on the transformed composite cross-section |
| *M*i,0,Ed | Design value of bending moment from non-permanent actions on the transformed composite cross-section |
| *M*pa | Design value of the plastic resistance moment of the effective cross section of the profiled steel sheeting |
| *M*pl,a,Rd | Design value of the plastic resistance moment of the structural steel section |
| *M*pl,Rd | Design value of the plastic resistance moment of the composite section with full shear connection |
| *M*pl,N,Rd | Design value of the plastic resistance moment of the composite section taking into account the compressive normal force |
| *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*Rd(*η*) | Design value of the plastic resistance moment of a composite section considering the degree of shear connection |
| *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*a | Design value of the normal force in the structural steel section of a composite beam |
| *N*a,Ed | Design axial force, respectively applied to the structural steel section before composite action is achieved |
| *N*c | Design value of the compressive axial force in the concrete flange, taking account of the degree of shear connection |
| *N*cd | Axial force in concrete in the cross-section where the design bending moment isthe maximum along the beam |
| *N*c,el | Compressive axial force in the concrete flange corresponding to *M*el,Rd |
| *N*c,f | Design value of the compressive axial force in the concrete flange with full shear connection |
| *N*cr | Elastic critical axial force |
| *N*cr,eff | Elastic critical load of a composite column corresponding to an effective flexural stiffness |
| *N*c1 | Design value of the compressive axial force in the concrete outside the steel flanges of concrete encased composite column |
| *N*Ed | Design value of the compressive axial force |
| *N*G,Ed | Design value of the part of the compressive axial force that is permanent |
| *N*i,D,Ed | Design value of axial force from prestressing by imposed deformation on the transformed composite cross-section |
| *N*i,Led | Design value of axial force from actions on the time dependant transformed composite cross section |
| *N*i,P,Ed | Design value of axial force from permanent actions on the transformed composite cross-section |
| *N*i,PT,Ed | Design value of axial force developing in time due to creep |
| *N*i,sh,Ed | Design value of axial force from shrinkage on the transformed composite cross-section |
| *N*i,0,Ed | Design value of axial force from non-permanent actions on the transformed composite cross-section |
| *N*p | Design value of the plastic resistance of the profiled steel sheeting to axial force |
| *N*p,l | Axial force in profiled steel sheeting anchored at the slab support |
| *N*pl,a | Design value of the plastic resistance of the steel section |
| *N*pl,Rd | Design value of the plastic resistance of the composite section to compressive axial force |
| *N*pl,Rk | Characteristic value of the plastic resistance of the composite section to compressive axial force |
| *N*s | Design value of the plastic resistance of the steel reinforcement to axial force |
| *N*s,l | Axial force in reinforcement anchored at the slab support |
| *P*e | Maximum value of the resistance per shear connector resulting from a standard push test |
| *P*Ed | Design value of the shear force in a shear connector |
| *P*em | Mean failure load per shear connector from a standard push test |
| *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*ℓ,Rd | Design shear resistance of a single headed stud in the longitudinal direction of the beam |
| *P*t,Rd | Design shear resistance of a single headed stud in the transverse direction of the beam |
| *P*ten,Rd | Design tension resistance of a headed stud |
| *Q*k | Characteristic value of the sum of all variable actions |
| *R*Ed | Design value of a support reaction |
| *R*m | Mean value of resistance |
| *R*pl,m | Resistance in combined bending and compression based on the full plastic interaction |
| *R*pl,d  *R*w,Rd | Resistance in combined bending and compression  Shear resistance of steel sheeting, in accordance with FprEN 1993-1-3:2023, 8.1.6 |
| *S*j | Rotational stiffness of a joint |
| *S*j,ini | Initial rotational stiffness of a joint |
| *V*a,Ed | Design value of the shear force acting on the structural steel section |
| *V*a,Rd | Design value of the shear resistance of the structural steel section |
| *V*b,Rd | Design value of the shear buckling resistance of a steel web, shear resistance of profiled steel sheeting, in accordance with FprEN 1993-1-3:2023, 8.1.5 |
| *V*b,e,Rd | Design value for vertical shear of the profiled steel sheeting, with a width |
| *V*c,cs | Shear capacity of the compression strut at the support of a composite slab |
| *V*c,ct | Shear capacity of concrete in a composite slab at crack propagation zone close to the crack tip |
| *V*c,cz | Shear capacity of the uncracked compression zone in a composite slab |
| *V*c,Ed | Design value of the shear force acting on the reinforced concrete web encasement |
| *V*c,Rd | Shear resistance of the concrete ribs in accordance with EN 1992-1-1:2023, 8.2.2 for a slab width of 1,0 meter. |
| *V*c,ks | Shear capacity of a composite slab inducing kinking of reinforcement and spalling of concrete |
| *V*c,ks,1 | Shear capacity of a composite slab inducing spalling of concrete |
| *V*c,ks,2 | Shear capacity of a composite slab inducing kinking of reinforcement |
| *V*Ed | Design value of the shear force acting on the composite section |
|  | Design value of permanent vertical shear forces during construction stage |
| *V*L,Ed | Longitudinal shear force |
| *V*p,Rd | Design value of the resistance of a composite slab to punching shear |
| *V*pl,a,Rd | Design value of the plastic resistance of the structural steel section to vertical shear |
| *V*pl,a,T,Rd | Design value of the reduced plastic shear resistance of the structural steel section making allowance for the presence of a torsional moment |
| *V*pp,Rd | Shear resistance of a composite slab for a surface parallel to the direction of the profiled steel sheeting |
| *V*pt,Rd | Shear resistance of a composite slab for a surface transverse to the direction of the profiled steel sheeting |
| *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 |

### Latin lower case letters

|  |  |
| --- | --- |
| *a* | Spacing between parallel beams |
| *a*hs | Diameter of a circular hollow section or width of a rectangular hollow section |
| *a*rp | Distance of the stud axis to the reinforcement which avoids the splitting of concrete (effective edge distance) |
| *a*sc | Distance from the centre of the stud to the end of the sheeting |
| *a*q | Distance between concentrated forces pushing against each other |
| *b* | Width of the flange of a steel section |
| *b*c | Width of the concrete encasement to a steel section;  Width of a concrete flange |
| *b*eff,c,wc | Effective width of the column web in compression |
| *b*eff | Total effective width |
| *b*eff,0 | Effective width at an end support |
| *b*eff,1 | Effective width at mid-span for a span supported at both ends |
| *b*eff,2 | Effective width at an internal support |
| *b*ei | Effective width of the concrete flange on each side of the web |
| *b*em | Effective width of a composite slab |
| *b*ev | Effective width of a composite slab for shear forces |
| *b*fp | Horizontal distance between the web-to-flange junction and the corner of the re-entrant stiffener |
| *b*i | Geometric width of the concrete flange on each side of the web |
| *b*bot | Width of the bottom flange to the profiled steel sheeting |
| *b*m | Width of a composite slab over which a load is distributed |
| *b*min | Minimum width of concrete within the through of the steel sheeting |
| *b*p | Dimension of concentrated point or line load |
| *b*r | Width of the top flange to the profiled steel sheeting |
| *b*s | Distance between centres of adjacent ribs of profiled steel sheeting |
| *b*sl | Slab width |
| *b*top | Width of the top of the concrete rib |
| *b*0 | Mean width of a concrete rib for open-through profiles or minimum width for re-entrant sheeting profiles;  Width of a concrete haunch |
| *b*0i | Distance between the centres of the outstand shear connectors; |
| *c* | Width of the outstand of a steel flange |
| *c*eff | Effective perimeter of reinforcing bar |
| Δ*c*dev | Allowance in design for deviation of the concrete cover |
| *c*min,b | Minimum concrete cover due to bond requirement |
| *c*s | Stiffness of the shear connection |
| *c*y, *c*z | Thickness of concrete cover |
| *c*z,min | Minimum concrete cover |
| *d* | Diameter of the shank of a stud connector  Overall diameter of circular hollow steel section |
| *d*do | Diameter of the weld collar to a stud connector |
| *d*ef | Height of a top reintrant stiffener on the upper side of an open through profiled sheeting |
| *d*p | Distance between the centroidal axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression |
| *d*q | Distance between centroidal axis of the sum of anchored tensile forces in profiled steel sheeting and reinforcement to the extreme fibre of composite slabs in compression |
| *d*s | Distance between the steel reinforcement in tension to the extreme fibre of the composite slab in compression |
| *d*s,c | Depth below the top of the slab to the reinforcement |
| *d*s,0 | Reference distance from the centroid of the tension forces to the extreme fibre of the concrete in compression of a composite slab |
| *d*w | Clear depth of the web of the structural steel section |
| *e* | Distance from the bottom fibre of the profile sheeting to its center of gravity |
| *e*D | Distance between the edge of a connector and the edge of the flange of the beam |
| *e*d,F | Either 2*e*h,F or 2*e*v,F |
| *e*d,min | Additional eccentricity |
| *e*g | Gap between the reinforcement and the end plate in a composite column |
| *e*h,F | Lateral distance from the point of application of force *F*Ed to the relevant steel web, if *F*Ed is applied to the concrete slab |
| *e*k | Distance from the edge of the concrete rib on the higher moment side to the centre-line of the nearest stud connector |
| *e*p | Distance from the plastic neutral axis of profiled steel sheeting to the extreme fibre of the composite slab in tension |
| *e*N | Eccentricity of loading |
| *e*v | Nominal concrete cover from the side of the haunch to the connector |
| *e*v,F | Vertical distance from the point of application of force *F*Ed to the relevant steel web, if it is applied to the slab |
| *f*cd | Design value of concrete compressive strength |
| *f*ck | Characteristic compressive cylinder strength of concrete at age *t*ref |
| *f*cm | Mean concrete cylinder compressive strength at age *t*ref |
| *f*ctm | Mean value of the axial tensile strength of concrete |
| *f*ct,eff | Mean value of the effective tensile strength of the concrete |
| *f*ct,0 | Reference strength for concrete in tension |
| *f*sd | Design value of the yield strength of reinforcing steel |
| *f*sk | Characteristic value of the yield strength of reinforcing steel |
| *f*sm | Mean value of the yield strength of reinforcing steel |
| *f*y | Nominal value of the yield strength of structural steel |
| *f*yd | Design value of the yield strength of structural steel |
| *f*yk | Characteristic value of the yield strength of steel |
| *f*ym | Mean value of the yield strength of structural steel |
| *f*yp,d | Design value of the yield strength of profiled steel sheeting |
| *f*u | Ultimate tensile strength of the material of the stud |
| *f*1, *f*2 | Reduction factors for bending moments at supports |
| *h* | Overall depth; thickness |
| *h*a | Depth of the structural steel section |
| *h*A | Embedment depth of a stud connector above the profiled steel sheeting |
| *h*c | Thickness of the concrete flange; Depth of the concrete encasement to a steel section; Thickness of concrete above the top of the profiled sheeting: *h*c=*h*cs−*h*pg or the thickness of the in-situ concrete above a semi-prefabricated concrete element |
| *h*cs | Total height of concrete slab including profiled sheeting |
| *h*e | Thickness of the finishes |
| *h*f | Length of a shear surface |
| *h*p | Overall depth of the profiled steel sheeting excluding longitudinal stiffener |
| *h*pc | Height of centroid of the lowest shear connection device above the lower chord, for re-entrant profiles midway between upper and lower chord |
| *h*pg | Overall depth of the profiled steel sheeting including longitudinal stiffener if available, otherwise *h*pg=*h*p |
| *h*s | Depth between the centroids of the flanges of the structural steel section, for symmetrical sections: *h*s=*h*a−*t*f |
| *h*sc | Overall nominal height of a stud connector in accordance with EN ISO 13918 |
| *k* | Coefficient which allows for the effect of non-uniform self-equilibrating stresses |
| *k*c | Coefficient which takes account of the stress distribution within the section immediately prior to cracking |
| *k*cc | Reduction factor considering the effect of concrete relaxation and sustained loading |
| *k*E | Factor in accordance with EN 1992-1-1:2023, 5.1.6(1) |
| *k*el | Lowest factor applied to the part of design moment on composite section to reach an elastic stress limit |
| *k*is | Coefficient which allows for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection |
| *k*ℓ | Reduction factor for resistance of a headed stud used with profiled steel sheeting parallel to the beam |
| *k*s | Rotational stiffness |
| *k*sc | Stiffness of a shear connector |
| *k*t | Reduction factor for resistance of a headed stud used with profiled steel sheeting transverse to the beam |
| *k*tc | Coefficient considering the effect of high sustained loads on concrete compressive strength, see EN 1992-1-1:2023, 5.1.6(1) |
| *k*t,max | Maximum value of *k*t |
| *k*up | Factor taking account if the beam is propped during construction or not |
| *k*wc,c | Factor for the effect of longitudinal compressive stress on transverse resistance of a column web |
| *k*1 | Flexural stiffness of the cracked concrete or composite slab |
| *k*2 | Flexural stiffness of the web |
| *k*Φ | Coefficient considering the distance between a stud and the end of the profiled sheeting for the determination of design resistance for end anchoring |
| *m*Ed | Design transverse bending moment per unit of length |
| *n* | Modular ratio |
| *n*f | Number of connectors for full shear connection |
| *n*L | Modular ratio depending on the type of loading |
| *n*r | Number of shear stud connectors in one rib |
| *n*sc | Number of shear connectors |
| *n*0 | Modular ratio for short-term loading |
| *p*Ed | Design line load |
| *q*Ed | Design distributed load |
| *r* | Ratio of end moments |
| *s*f | Longitudinal spacing of transverse reinforcement |
| *s*x | Longitudinal spacing center-to-center of the stud shear connectors |
| *s*y | Transverse spacing center-to-center of the stud shear connectors |
| *t* | Age, wall thickness of a hollow section |
| *t*e | Thickness of an end plate |
| *t*eff,c | Effective length of concrete |
| *t*f | Thickness of a flange of the structural steel section |
| *t*p | Thickness of the steel profiled sheeting |
| *t*ref | Reference time, see EN 1992-1-1:2023, 5.1.6(1) |
| *t*w | Thickness of the web of the structural steel section |
| *t*0 | Age at loading |
| *v* | Anchoring length |
| *v*L,Ed | Design longitudinal shear force per unit of length |
| *v*L,Ed,max | Maximum design longitudinal shear force per unit of length |
| *v*L,Rd | Longitudinal shear resistance per unit length at the interface between steel and concrete in a composite member |
| *w*lim,cal | Limit for the calculated crack width |
| *w*s | Width of a top reintrant stiffener on the upper side of an open through profiled sheeting |
| *w*o | Geometrical imperfection |
| y-y | Cross section axis parallel to the flanges |
| *z* | Lever arm |
| z-z | Cross section axis perpendicular to the flanges |
| *z*c | Depth of concrete in compression |
| *z*cl | Distance between center lines of concrete slab and steel section |
| *Z*p | Distance between the compression force in the concrete and the tension force in the steel sheeting |
| *z*pl | Distance between the plastic neutral axis and the extreme fibre of the concrete slab in compression |
| *z*s | Distance between the compression force in the slab and the tension force in the rib reinforcement |
| *z*0 | Vertical distance |

### Greek upper case letters

|  |  |
| --- | --- |
| Δ*σ*s | Increase of stress in steel reinforcement due to tension stiffening of concrete |

### Greek lower case letters

|  |  |
| --- | --- |
| *α* | Ratio of characteristic resistance of a shear connector from push test to the characteristic resistance used for design |
| *α*cr | Minimum force amplifier to reach the elastic critical buckling load |
| *α*M | Coefficient related to bending of a composite column |
| *α*My , *α*Mz | Coefficients related to bending of a composite column about the y-y axis and the z-z axis respectively |
| αs | Coefficient considering the slab system for determination of the flexural stiffness of the slab for verification of lateral torsional buckling |
| *α*st | Ratio of the product of the area and second moment of area of the composite cross-section to the product of area and second moment of area of the steel section |
| *β* | Reduction factor for plastic moment resistance |
| *β*c | Coefficient increasing the design shear strength for composite columns |
| *β*d | Reduction factor |
| *β*i | Parameter for the effective width of concrete flange |
| *β*L | Angle of spread for concentrated longitudinal force |
| *β*M | Equivalent moment factor |
| *β*S | Section parameter |
| *γ*C | Partial factor for concrete |
| *γ*G | Partial factor for permanent actions |
| *γ*M | Partial factor for a material property, also accounting for model uncertainties and dimensional variations |
| *γ*M0 | Partial factor for structural steel applied to resistance of cross sections, see EN 1993-1-1:2022, 8.1(1) |
| *γ*M1 | Partial factor for structural steel applied to resistance of members to instability assessed by member checks, see EN 1993-1-1:2022, 8.1(1) |
| *γ*P | Partial factor for prestressing action |
| *γ*Q | Partial factor for variable actions |
| *γ*S | Partial factor for reinforcing steel |
| *γ*Rd | Partial factor for taking into account model uncertainties in the resistance model |
| *γ*V | Partial factor for design shear resistance of a shear connector |
| *γ*VS | Partial factor for longitudinal shear in composite slabs for buildings |
| *γ*0 | Overall partial factor |
| *δ* | Slip between the concrete and steel due to the deformation of the shear connection |
| *δ*c | Steel contribution ratio |
| *δ*α | Slip value of 2,0 mm for slip between the concrete for classification of shear connector resistance |
| *δ*ek | Characteristic value of the elastic slip of connectors when reaching the characteristic resistance |
| *δ*max | Sagging vertical deflection |
| *δ*p | Deflection of steel sheeting under its own weight plus the weight of wet concrete |
| *δ*p,max | Limiting value for *δ*p |
| *δ*sh | Deflection of composite slab due to shrinkage |
| *δ*uk | Characteristic value of the slip capacity of connectors at Ultimate Limit State |
| *ε* | Material parameter given by  where is *f*y in N/mm2 |
| *ε*sh | Shrinkage strain of the concrete |
| *η* | Degree of shear connection |
| *η*a , *η*a0 | Factors related to the confinement of concrete |
| *η*c , *η*c0 | Factors related to the confinement of concrete |
| *η*cc | Factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength that can be developed in the structural component |
| *η*cc,up | Upper value of *η*cc factor |
| *η*cL | Factor related to the confinement of concrete |
| *η*j | Stiffness modification factor |
| *η*lw,fc | Coefficient related to *f*ck in lightweight aggregate concrete |
| *η*lw,fct | Influence factor of the increased brittleness of lightweight concrete axial tensile strength of concrete *f*ctm |
| *η*min | Minimum degree of shear connection in accordance with the Ductility Category of shear connector |
| *η*s | Utilisation factor of reinforcement |
| *η*V | Coefficient for interaction of shear and bending |
| *η*0 | Minimum degree of shear connection in accordance with composite beam geometry |
| *Θ*f | Angle |
|  | Relative slenderness |
|  | Relative slenderness for lateral torsional buckling |
| *μ*d | Factor related to design for compression and uniaxial bending |
| *μ*f,d | Design friction value |
| *μ*f,k | Characteristic friction value |
| *μ*dy , *μ*dz | Factor *μ*d related to plane of bending |
| *v* | Reduction factor to allow for the effect of longitudinal compression on resistance in shear |
| *v*a | Poisson’s ratio for structural steel |
| *ρ* | Parameter related to reduced design bending resistance accounting for vertical shear |
| *ρ*m | Factor |
| *ρ*s | Reinforcement ratio |
| *ρ*up | Ratio between moments |
| *σ*com,c,Ed | Longitudinal compressive stress in the encasement due to the design normal force |
| *σ*ct | Tensile stress in the extreme fibre of concrete |
| *σ*c,Rd | Local design strength of concrete |
| *σ*s | Stress in the tension reinforcement |
| *σ*s,0 | Stress in the tension reinforcement neglecting tension stiffening of concrete |
| *σ*x | Stress in the tension reinforcement in the x direction |
| *σ*y | Stress in the tension reinforcement in the y direction |
| *τ*Ed | Longitudinal shear stress, at the junction between one side of a flange in accordance with EN 1992-1-1:2023, 8.2.5 |
| *τ*Rd | Design shear strength |
| *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 |
| *φ*\* | Maximum diameter of a steel reinforcing bar |
| *ϕ*t | Creep coefficient |
| *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* | Coefficient |
| *ψ*L | Creep multiplier |

## Additional symbols used in Annex A

### Latin upper case letters

|  |  |
| --- | --- |
| *A*s,r | Cross-sectional area of the longitudinal reinforcement in row r |
| *K*sc | Stiffness of a group of connectors |
| *K*β | Coefficient |
| *M*Ed,j | Design bending moment applied to a connection j |

### Latin lower case letters

|  |  |
| --- | --- |
| *d*sb | Distance between the longitudinal reinforcing bars in tension and the centroid of the beam’s steel section |
| *d*wc | Straight height of the column web |
| *h*st | Distance between the longitudinal reinforcing bars in tension and the centre of compression |
| *k*c,wc | Stiffness coefficient for an unstiffened web and a contact plate |
| *k*c,wc,c | Additional stiffness coefficient for acolumn web encased in concrete |
| *k*i | Stiffness coefficient |
| *k*slip | Reduction factor taking account of the deformation of the shear connection |
| *k*s,r | Stiffness coefficient for a reinforcement row in tension |
| *k*wp | Stiffness coefficient for a steel column web given by FprEN 1993-1-8:2023, A.2.2(1) |
| *k*wp,c | Additional stiffness coefficient for a steel column web encased by concrete |
| *n*Ls | Number of shear connectors distributed over the length |
| *t*wc | Thickness of the column web |
| *v* | Factor |

### Greek lower case letters

|  |  |
| --- | --- |
| *β* | Transformation parameter |
| *ξ* | Parameter related to deformation of the shear connection |

## Additional symbols used in Annex B

### Latin upper case letters

|  |  |
| --- | --- |
| *L*o | Length of overhang |
| *L*s | Shear span length |
| *M*pa,m | Plastic resistance moment of profiled steel sheeting based on measure values |
| *M*pr,m | Reduced plastic moment |
| *N*c,fm | Compressive axial force in the concrete flange at moment |
| *N*c,m | Compressive axial force in the concrete flange with partial shear connection |
| *P* | Load |
| *P*e | Expected failure load |
| *V*t | Support reaction under the ultimate test load |
| *W*t | Measured failure load |

### Latin lower case letters

|  |  |
| --- | --- |
| *f*cm,t | Actual mean value of the concrete cylinder compressive strength in the test specimen |
| *f*ut | Actual ultimate tensile strength of the material of the stud in the test specimen |
| *f*yp | Nominal yield strength of profiled steel sheeting |
| *f*ypm,t | Mean value of the measured yield strength of profiled steel sheeting |
| *s*e | Experimental slip at a load of 0,7 *P*Rk |
| *z*pl,m | Location of the neutral plastic axis |

### Greek lower case letters

|  |  |
| --- | --- |
| *δ*u | Maximum slip measured in a test at the characteristic load level |
| *η*test | Degree of shear connection in the test |
| *μ*m | Mean value of the friction coefficient |
| *τ*u | Value of longitudinal shear strength of a composite slab determined from testing |

## Additional symbols used in Annexes D and E

### Latin upper case letters

|  |  |
| --- | --- |
| *A*a,bT | Cross-sectional area of bottom Tee |
| *A*a,tT | Cross-sectional area of top Tee |
| *A*v,bT | Shear area of bottom Tee |
| *A*v,tT | Shear area of top Tee |
| *F*tr.Rd | Resistance of transverse reinforcement to local loads |
| *I*bT | Second moment of area of bottom Tee |
| *I*1,gross | Second moment of area of the composite section with solid web |
| *I*1,net | Second moment of area of the composite section at the centre of the opening |
| *I*tT1 | Second moment of area of the transformed composite top Tee |
| *M*A,Ed | Design bending moment in locally stiff slab at the low moment end of the opening |
| *M*A,Rd | Bending resistance moment in locally stiff slab at the low moment end of the opening |
| *M*B,Ed | Design bending moment in locally stiff slab at the high moment end of the opening |
| *M*B,Rd | Bending resistance moment in locally stiff slab at the high moment end of the opening |
| *M*c,Rd | Bending resistance of the slab for the Vierendeel effect |
| *M*elN,bT | Moment required to cause yield in the bottom steel Tee at the low moment end of the opening taking into account the axial force |
| *M*elN,tT | Moment required to cause yield in the top steel Tee at the high moment end of the opening taking into account any axial force |
| *M*NV,bT,Rd | Reduced bending resistance of bottom Tee due to axial force and using the effective web yield strength or thickness for the effects of shear |
| *M*NV,tT,Rd | Reduced bending resistance of top Tee due to axial force and using the effective web yield strength or thickness for the effects of shear |
| *M*o,Ed | Design value of bending moment at the centre-line of an opening |
| *M*vc,Rd | Local Vierendeel bending resistance due to composite action |
| *M*wp,Ed | Design value of in plane moment at mid-height of the web post |
| *N*bT,Ed | Design value of tensile force in bottom Tee |
| *N*bT,Rd | Design value of tensile resistance of bottom Tee |
|  | Increase in design value of compression force developed by the shear connectors placed between the centre-lines of adjacent openings |
| *N*oc,Ed | Design value of compression force in slab developed from the nearer support to the centre of the opening |
| *N*oc,Rd | Design value of compression resistance of concrete slab at the centre of an opening |
| *N*sl,Rd | Cumulated shear resistance of the connectors placed between the nearer support and the center-line of the opening |
| *N*tT,Ed | Design value of compression force in top Tee |
| *N*tT,Rd | Design axial resistance of the top Tee |
| *N*w,Ed | Compressive force in the web |
| *P*comp,Ed | Vertical compression force at the lower moment side of an opening with locally stiff slab |
| *P*s,Ed | Longitudinal shear force acting on a shear connector over the opening that is used to develop the Vierendeel bending resistance due to composite action |
| *P*ten,Ed | Vertical tension force acting on the group of shear connectors at the higher moment side of the opening |
| *V*av,Ed | Average value of the shear forces at the centre of adjacent openings |
| *V*bT,Ed | Design value of the shear force acting on the bottom Tee |
| *V*bT,Rd | Plastic shear resistance of the bottom Tee |
| *V*c,Rd | Shear resistance of the concrete slab |
| *V*iT,Ed | Design value of the shear force acting on a Tee, equal to either *V*bT,Ed or *V*tT,Ed |
| *V*iT,Rd | Design value of the plastic shear resistance of a Tee, equal to either *V*bT,Ed or *V*tT,Ed |
| *V*oa,Ed | Design value of the shear force acting on the steel profile at an opening |
| *V*oc,Ed | Design value of the shear force acting on the concrete slab at an opening |
| *V*oc,Rd | Design shear resistance of the concrete slab at the opening position |
| *V*o,Ed | Design value of the shear force acting on the beam at the centre of the opening |
| *V*o,ser,Ed | Design value of shear force at the opening at the serviceability limit state |
| *V*slab | Maximum possible shear force in a stiff slab at the center of the opening |
| *V*tT,Ed | Design value of the shear force acting on the top Tee |
| *V*tT,Rd | Plastic shear resistance of the top Tee |
| *V*Vier,Rd | Shear resistance to Vierendeel bending |
| *V*wp,Ed | Design value of horizontal shear force in web post |

### Latin lower case letters

|  |  |
| --- | --- |
| *a*eff | Effective length of opening for buckling and deflection calculations |
| *a*eq | Equivalent length of opening for Vierendeel bending |
| *a*eqA | Distance from the lower moment end of the opening to the point where the local moment in the slab is equalto 0 |
| *a*eqB | Distance from the higher moment end of the opening to the point where the local moment in the slab is equal to 0 |
| *a*o | Length of opening |
| *b*eff,b | Effective width of slab due to local bending |
| *b*eff,V | Effective width of slab due to local shear |
| *b*hm,eff | Effective width of the slab determined at the high-moment end of the opening |
| *d*c | Distance of the centroid of the compressed part of the slab from the upper face of the steel profile |
| *d*s,c,b | Distance of the bottom reinforcement to the top of the slab |
| *d*s,c,ten | Distance of the transverse reinforcement in the tension region to the upper edge of the slab |
| *d*y | Width of load introduction perpendicular to the beam axis |
| *e*o | Eccentricity of centre line of opening above centre line of beam |
| *f*y,red | Reduced yield strength of the structural steel which takes account of MV interaction |
| *n*bo | Number of openings along the beam |
| *h*bT | Depth of bottom Tee |
| *h*ts,lim | Limit of slab depth for stiff slab |
| *h*tT | Depth of top Tee |
| *h*o | Depth of opening |
| *k*a | Modification factor on composite resistance to Vierendeel bending for long openings |
| *n*t | Number of shear connectors in the group acting in tension |
| *r*w1, *r*w2 | Factors |
| *s*o | Centre to centre spacing of adjacent openings |
| *w*add | Additional mid-span deflection due to the openings |
| *w*b | Mid-span deflection of the equivalent solid web composite beam |
| *w*p | Edge-to-edge spacing of openings |
| *w*v,add | Relative deflection due to Vierendeel bending |
| *x*0 | Position of centre of opening from nearer support |
| *z*bT | Distance of centroid of bottom Tee from the bottom of the steel section |
| *z*tT | Distance of centroid of top Tee from the top of the steel section |

### Greek lower case letters

|  |  |
| --- | --- |
| *ρ*min | Minimum reinforcement ratio |
| *ρ*st,lim | Limit of reinforcement ratio for stiff slabs |
| *ρ*t | Transverse reinforcement ratio |
| *φ*t | Diameter of the transverse reinforcement in the tension region of the slab |

## Additional symbols used in Annex F

### Latin upper case letters

|  |  |
| --- | --- |
| *F*L,Ed | Design longitudinal shear force |
| *F*V,Ed | Design vertical shear force |
| *N*cyc | Reference number of cycles |
| *N*cyc,f | Number of force range cycles |
| *P*L,Rd | Design longitudinal shear resistance |
| *P*V,Rd | Design vertical shear resistance |
| *T*Ed | Design splitting force |

### Latin lower case letters

|  |  |
| --- | --- |
| *a*r | Distance between the axis of the stud and the closest concrete surface |
| *a*r,o | Distance between the axis of the stud and the slab surface where the concrete cone failure would appear |
| *a*rp | Effective edge distance |
| *a*rp,o | Relevant effective edge distance for the concrete cone failure |
| *c*v | Nominal concrete cover |
| *k*V | Coefficient |
| *m* | Slope of the fatigue strength |
| *s* | Spacing of the stirrup |
| *s*v | Transverse spacing between studs |
| *v* | Distance of the stirrup to the lower face of the connector’s head |

### Greek upper case letters

|  |  |
| --- | --- |
| Δ*P*L,c | Reference value of fatigue strength for the range of longitudinal shear forces per stud at *N*cyc = 2 × 106 |
| Δ*P*L,R | Fatigue strength based on the range of longitudinal shear forces per stud |
| Δ*P*V,c | Reference value of fatigue strength for the range of vertical shear forces per stud at *N*cyc = 2 × 106 |
| Δ*P*V,R | Fatigue strength based on the range of vertical shear forces per stud |
| Δ*P*R | Fatigue strength based on the difference of shear force per stud |
| Δ*P*c | Reference value of fatigue strength at 2 million cycles |

### Greek lower case letters

|  |  |
| --- | --- |
| *β*r | Angle |
| *γ*Mf,s | Partial factor for fatigue strength of shear connectors |
| *γ*Ff | partial factor for fatigue actions |
| *η*r | Correction factor |
| *φ*s | Diameter of the stirrups |
| *φ*l | Diameter of the longitudinal reinforcement |

## Additional symbols used in Annex G

### Latin upper case letters

|  |  |
| --- | --- |
| *C*2 | Coefficient |
| *M*pl,sc | Plastic moment of the connector |
| *W*sc | Modulus of flexion |

### Latin lower case letters

|  |  |
| --- | --- |
| *f*ctk,0,05 | Characteristic value of tensile strength of concrete in accordance with EN 1992-1-1:2023, Table 5.1 |
| *k*u | Correction factor |
| *n*y | Number of plastic yielding in the stud |

## Additional symbols used in Annex H

### Latin upper case letters

|  |  |
| --- | --- |
| *A*c,N/ | Ratio taking into account the geometric effect of axial spacing and edge distance on the characteristic resistance *N*c,Rk |
| *N*c,Rd | Design resistance of the headed stud to concrete cone failure |
| *N*c,Rk | Characteristic resistance of the headed stud to concrete cone failure |
|  | Characteristic resistance of a single headed stud placed in the slab and not influenced by adjacent studs or edges |
| *N*p,Rd | Design resistance of the headed stud to concrete pull-out failure mode |
| *N*p,Rk | Characteristic resistance of the headed stud to concrete pull-out failure mode |
| *N*s,Rd | Design resistance of the steel stud to tension |
| *N*s,Rk | Characteristic resistance of the steel stud to tension |

### Latin lower case letters

|  |  |
| --- | --- |
| *b*x | Distance |
| *c* | Distance of the headed stud to the closest edge |
| *c*cr,N | Characteristic edge distance |
| *h*ef | Effective embedment depth of the stud |
| *k*1 | Factor that takes into account the load transfer mechanism |
| *n*s | Number of regularly spaced headed studs |
| *s*rc,N | Characteristic spacing of studs to ensure the characteristic resistance of the stud in case of concrete cone failure under tension load |
| *t*n | Thickness of the head |

### Greek lower case letters

|  |  |
| --- | --- |
| *γ*Mc | Partial factor for the concrete cone failure mode |
| *γ*Mp | Partial factor for the pull-out failure mode |
| *γ*Ms | Partial factor for the tension resistance of steel of the stud |
| *ψ*ec,N | Factor taking into account the group effect when different tension loads are acting on the individual fasteners of a group in case of concrete cone failure |
| *ψ*M,N | Factor |
| *ψ*P | Increasing factor for the characteristic resistance of the headed stud to concrete pull-out failure mode |
| *ψ*re,N | Shell spalling factor |
| *ψ*s,N | Factor taking into account the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the slab |

## Additional symbols used in Annex I

|  |  |
| --- | --- |
| *η*m | Moment ratio |
| *m*ybt,Ed | Design transverse bending moment per unit of length of the plate supporting the slab |
| *m*ybt,Rd | Design transverse resistant bending moment per unit of length of the plate supporting the slab |

## Additional symbols used in Annex J

### Latin upper case letters

|  |  |
| --- | --- |
| *A*s | Tensile stress area of the bolt |
| *D*upper | Largest value of the upper sieve size *D* in an aggregate for the coarsest fraction of aggregates in the concrete permitted by EN 206. |
| *F*prC | Preloading force in the bolt |
| *L*bc | Nominal bearing length |
| *L*f | Infill lengths |
| *P*b,Rd | Design value of shear resistance based on bolt failure |
| *P*c,Rd | Design value of shear resistance based on concrete failure |

### Latin lower case letters

|  |  |
| --- | --- |
| *a*b | Breadth of the chamfer |
| *a*h | Height of the chamfer |
| *b*e | Width of the hollow core slab |
| *b*g | Gap between the slabs |
| *d* | Diameter of the bolt used as connector |
| *f*ub | Ultimate tensile strength of the bolt |
| *h*j | Depth of the slab where the transfer of compression is not possible |
| *h*pc | Thickness of the precast floor plate |
| *h*sc | Overall nominal height of the non-preloaded bolted shear connector above the flange |
| *h*core | Diameter of openings in hollow core slabs |
| *k* | Factor |

### Greek lower case letters

|  |  |
| --- | --- |
| *α*b, *α*c | Factors |
| *β*h | Coefficient |
| *ε*h | Coefficient |
|  | Diameter of the transverse rebars |

# Basis of design

## General Rules

### Requirements

(1) The design of composite structures shall be in accordance with the general rules given in EN 1990 and the specific provisions for steel-concrete composite structures given in this document.

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

* limit state design in conjunction with the partial factor method in accordance with EN 1990;
* actions in accordance with EN 1991 (all parts);
* combination of actions in accordance with EN 1990; and
* resistances, durability and serviceability in accordance with this document.

### Robustness

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

(2) The general arrangement of the structure and the interaction and connection of its various parts should be such as to give appropriate robustness during construction and use.

### Reliability

(1) The provisions on reliability given in EN 1990, EN 1993-1-1 and EN 1992-1-1 shall be followed.

## Principles of limit states design

(1) The ultimate limit state and serviceability limit state shall be considered for all aspects of the composite structure.

(2) All relevant design situations shall be considered, including relevant phases in the erection stage.

NOTE For the selection of design situations, see EN 1990.

## Basic variables

### Actions and environmental influences

(1) The characteristic values of actions for the design of composite structures, including any regional, climatic and accidental situations, shall be obtained from the relevant parts of EN 1991.

(2) The actions to be considered during the erection stage may be obtained from EN 1991-1-6.

(3) In verification for steel sheeting as formwork, the effects of ponding shall be considered.

### Material and product properties

(1) Unless otherwise stated in EN 1994-1-1, actions caused by time dependent behaviour of concrete should be obtained from EN 1992-1-1.

### Classification of actions

(1) 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.

NOTE The effects of shrinkage and creep of concrete and non‑uniform changes of temperature result in internal forces in cross-sections, curvatures and longitudinal strains in members.

(2) 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 additional effects shall be classified as secondary effects and shall be considered as indirect actions.

## Verification by the partial factor method

### Design values

#### Design values of actions

(1) For the design of composite structures, combination of actions and partial factors of actions shall be derived from EN 1990:2023, Annex A.

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

NOTE The value for *γ*P for favourable effects is 1,0 and for unfavourable effects is 1,1 unless a different value is given in the National Annex.

#### 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, the design compressive strength *f*cd shall be obtained by reference to EN 1992-1-1:2023, 5.1.3 and 5.1.6(1) for normal weight concrete and to EN 1992-1-1:2023, Annex M, Tables M.1 and M.2 for lightweight concrete.

NOTE 1 For confined concrete, see EN 1992-1-1:2023, 8.1.4.

NOTE 2 More details for compressive strength *f*cd are given in 5.1.

NOTE 3 The value for γC is that used in EN 1992-1-1.

(3) For steel reinforcement, the design yield strength *f*sd corresponds to the value *f*yd as given in EN 1992-1-1:2023, 5.2.4(1). More details for steel reinforcement are given in 5.2.

NOTE The value for γS is that used in EN 1992-1-1.

(4) For structural steel, connecting devices in accordance with EN 1993-1-8 and profiled steel sheeting, partial factors γM shall be applied. Unless otherwise stated, the partial factor for structural steel shall be taken as γM0.

NOTE 1 Values for γM are those given in EN 1993-1-1 for structural steel

NOTE 2 Values for γM are those given in EN 1993-1-8 for connecting devices

NOTE 3 Values for γM are those given in EN 1993-1-3 for profiled steel sheeting. The recommended value for profiled steel sheeting in tension is 1,0 unless the National Annex specifies different values.

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

NOTE The value for γV is 1,25 unless the National Annex gives a different value.

(6) For longitudinal shear in composite slabs for buildings, a partial factor γVS shall be applied.

NOTE The value for γVS is 1,25 unless the National Annex gives a different value.

#### Design values of geometrical data

(1) Geometrical data for cross‑sections and systems may be taken from the relevant EN and EN-ISO product standards, EADs or ETAs or drawings for the execution and treated as nominal values. Reference is given to EN 1993-1-1:2022, 4.2.2 and EN 1992-1-1:2023, 4.2.2.

### Design resistances

(1) For composite structures, design resistances shall be determined in accordance with EN 1990:2023, Formula (8.20), Formula (8.21) or Formula (8.23).

### Combination of actions

(1) The combination of actions given in EN 1990:2023, Clause 8 and Annex A shall be used.

### Verification of static equilibrium

(1) The reliability format for the verification of static equilibrium for buildings, as described in EN 1990:2023, Table A1.8, shall be applied to design situations equivalent to the static equilibrium limit state, e.g. for the design of holding down anchors or the verification of uplift of bearings of continuous beams.

# Materials

## Concrete

(1) Unless otherwise given in EN 1994-1-1, properties should be obtained by reference to EN 1992-1-1:2023, 5.1.3 and 5.1.6 for normal weight concrete and to EN 1992-1-1:2023, Annex M for lightweight concrete.

(2) Except where (3) applies, the design compressive strength of the concrete *f*cd should be taken from EN 1992-1-1:2023, 5.1.6 or Annex M.

(3) The design compressive strength of the concrete *f*cd should be taken from EN 1992-1-1:2023, 5.1.6 formula (5.3), where the value of ηcc in accordance with EN 1992-1-1:2023, formula (5.4) or formula (M.4) should not exceed the value of 0,85 for

* the bending resistance determined using plastic methods in accordance with 8.2.1.2, 8.2.1.3 or 8.3.2 (2); or
* the bending resistance determined using non-linear methods in accordance with 8.2.1.5 where the complete thickness of the concrete flange is subject to compression; or
* the resistance of the cross section of a column, determined using plastic methods in accordance with 8.8.3.2.

Notwithstanding the above, where the resistance of the cross section of a concrete filled section is determined using plastic methods in accordance with 8.8.3.2, the limit of 0,85 on the value of cc may be increased to 1,0.

NOTE The proposed value for *k*tc in accordance with EN 1992-1-1 is 1,0 for the concrete compression strength at 28-days, unless other values are given by the National Annex to this standard. For concrete compression strength based on *t*ref > 28 days reference is given to EN 1992-1-1:2023, 5.1.6(1).

(4) 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 C70/85 and LC60/66.

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

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

(7) For buildings, the values of shrinkage strain should be as given in EN 1992-1-1 or in Annex C.

NOTE The choice of values can be given in the National Annex.

(8) The secant modulus of elasticity of the concrete *E*cm for short-term loading should be obtained by reference to EN 1992-1-1:2023, 5.1.4 for normal weight concrete and EN 1992-1-1:2023, Annex M for lightweight aggregate concrete.

(9) The application of fibre reinforced concrete in accordance with EN 1992-1-1:2023, Annex L requires advanced design methods based on the stress-strain curve of the fibre reinforced concrete and justification of the shear connector resistance by experimental tests in accordance with Annex B.

## Reinforcing steel

(1) Properties should be obtained by reference to EN 1992-1-1:2023, 5.2, see also 4.4.1.2(3). All reinforcing steel used in composite structures where plastic resistance is taken into account, should meet the requirements of ductility Class B or Class C in accordance with EN 1992-1-1:2023, Table 5.5.

(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 1993-1-1:2022, 5.2.5.

(3) The rules for in this part of EN 1994 does not apply for reinforcement grades higher than B500 for the following cases:

* plastic global analysis of continuous composite beams;
* plastic resistance of partially-encased composite beams sections;
* plastic resistance of composite slabs; and
* plastic resistance of composite columns in accordance with 8.8.3.

## Structural steel

(1) Properties should be obtained by reference to EN 1993-1-1:2022, 5.1 and 5.2, see also 4.4.1.2(4).

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

## Connecting devices

### Fasteners

(1) Reference should be made to EN 1993‑1‑8 for requirements for fasteners (bolts, rivets or pins) and welding consumables.

### Shear connectors

#### Ductility

(1) Shear connectors that are used in design according to this Eurocode should be classified according to the requirements given in Table 5.1. The associated load-slip relation *P*-*δ* for the Ductility Categories, together with the definition of *δ*ek and *δ*uk are given in Figure 5.1. The properties *δ*uk, *δ*ek, *P*Rk and *P*em are to be evaluated in accordance with Annex B.

**Table 5.1 — Ductility Categories for shear connectors**

|  |  |  |  |
| --- | --- | --- | --- |
| Ductility Category | Characteristic elastic slip when characteristic resistance is reached  *δ*ek | Characteristic slip capacity in ULS  *δ*uk | Definition |
| D1 | - | - | Brittle or flexible shear connector with linear or multilinear load slip curve, not fulfilling the requirements of the Ductility Categories D2 or D3 [see Figure 5.1(a) and(b)] |
| D2 | mm | 6,0 mm ≤ *δ*uk < 10 mm | Ductile shear connector in accordance with (2), (3) or (5) with sufficient deformation capacity in accordance with (4) to justify the assumption of ideal-plastic behaviour of the shear connection in the structure, where the characteristic resistance *P*Rk intersects *δ*ek and *δ*uk in the load-slip curve [see Figure 5.1(c), (d)] |
| D3 | *δ*uk ≥ 10 mm |

|  |  |
| --- | --- |
| A diagram of a graph  Description automatically generated | A diagram of a curve  Description automatically generated |
| a) | b) |
| A diagram of a curve  Description automatically generated | A diagram of a function  Description automatically generated |
| c) | d) |

**Key**

|  |  |
| --- | --- |
| a) | D1 Ductility Category D1 (brittle behaviour) |
| b) | D1 Ductility Category D1 (flexible behaviour with multilinear curve) |
| c) | D2 or D3 Ductility Category D2 and D3 (ductile behavior) |
| d) | D2 or D3 Ductility Category D2 and D3 where the design longitudinal shear stress is to be modified by the ratio α according to Clause (5) |

Figure 5.1 — P-δ relation for Ductility Categories

(2) Headed studs in solid slabs may be deemed to be Ductility Category D2 when the rules for headed studs and shear connection in 8.6.8.1 apply and the concrete strength Class does not exceed C60/75.

NOTE  For other types of shear connectors, the classification into the Ductility Category in accordance with Table 5.1 can be verified by push tests in accordance with Annex B or taken from an European Technical Specification (TS) or a transparent and reproducible assessment that complies with all the requirements of an European Assessment Document (EAD).

(3) Headed studs in profiled steel sheeting may be deemed to be at least Ductility Category D2 when the design resistance for shear is calculated in accordance with 8.6.9.1(1) or 8.6.9.2(1).

NOTE  For Ductility Category D3 shear connectors, the classification can be provided by Push-Tests in accordance with Annex B or taken from an European Technical Specification (TS) or a transparent and reproducible assessment that complies with all the requirements of an European Assessment Document (EAD).

(4) The assumption of ideal plastic behavior is to be verified to classify shear connectors in Category D2 or D3. This may be deemed to be fulfilled when the assumption of a horizontal plateau of the load-slip curve between *δ*ek and *δ*uk can be proved.

NOTE A horizontal plateau of the load-slip curve between *δ*ek and *δ*uk can be assumed if the ratio of *P*em / *P*Rk does not exceed a value of 1,3.

(5) Shear connectors fulfilling the requirements of *δ*ek for Category D2 or D3, exceeding the minimum ultimate slip capacity *δ*uk given in Table 5.1 but not fulfilling the assumption of a horizontal plateau of the load-slip curve between *δ*ek and *δuk* in accordance with (4) may be considered as Category D2 and D3 respectively, provided that design longitudinal shear stress on a shear failure for any potential surface of longitudinal shear failure in accordance with 8.6.11.2 is increased by the ratio of *P*Rk/ (*α*⋅*P*Rk), [see Figure 5.1(d)]. Otherwise those connectors are to be classified in Category D1, Table 5.1.

NOTE 1 The values α may be taken from an European Technical Specification (TS) or a transparent and reproducible assessment that complies with all the requirements of an European Assessment Document (EAD) or the value for *P*em and *P*Rk can be determined by push tests in accordance with Annex B.1.

NOTE 2 For the classification, the shear connector resistance α *P*Rk may be taken at a slip of *δ*α. The design shear resistance is then taken as *P*Rd = α *P*Rk/γv where 0 < α ≤ 1,0, [see Figure 5.1(d)]. The value *δ*α is smaller than 2,0 mm unless the National Annex gives a different value for use within specific countries.

(6) The flexibility of the shear connector is to be considered:

* in ultimate limit states where significant impact on the rotation capacity of a member is expected;
* in serviceability limit states where significant impact on the deflection of a member is expected.

#### Headed stud shear connectors

(1) Reference should be made to EN ISO 13918. The headed stud shear connectors should be type SD1 or SD3.

(2) The rules for headed studs and shear connection in accordance with 8.6 apply.

#### Other types of shear connectors

(1) Other types of shear connectors may be used if their performance (in particular load-slip behaviour) has been adequately verified.

NOTE European Standard (EN), European Technical Specification (TS) or a transparent and reproducible assessment that complies with all the requirements of an European Assessment Document (EAD) or verification in accordance with Annex B.2 provide adequate verification.

## Profiled steel sheeting for composite slabs in buildings

(1) Properties should be obtained by reference to FprEN 1993‑1‑3:2023, 5.1 and 5.2.

(2) For the design of composite slabs according to this standard, profiled steel sheets should be:

* profiled steel sheets manufactured from steel in accordance with EN 10025 (all parts) or;
* cold-formed steel sheets in accordance with EN 10149-2 or EN 10149-3 or;
* galvanized steel sheet in accordance with EN 10346 or;
* galvanized steel sheet with organic coatings in accordance with EN 10169.

# Durability

## General

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

(2) Detailing of the shear connection shall be in accordance with 8.6.10 for headed studs.

NOTE For other types of shear connector detailing is to be applied in accordance to CEN/TS 1994-1-102—1, or an European Technical Product Specification.

## 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. The relevant provisions given in EN 1993-1-3 should be followed.

(2) Zinc and other metallic coatings, if specified, shall be in accordance to EN 10346 or with relevant standards.

NOTE Hot-dipped zinc coated (Z275) profiled steel sheeting is sufficient for internal elements in a non‑aggressive environment but the specification can be varied depending on service conditions.

(3) Organic coatings, if specified, should be in accordance with EN 10169 or with relevant standards in force.

# Structural analysis

## Structural modelling for analysis

### Structural modelling and basic assumptions

(1) Clause 7 is applicable to composite structures in which most of the structural members and joints are either composite or of structural steel.

(2) The structural model and basic assumptions shall be chosen in accordance with EN 1990:2023, 7.1.1 and shall reflect the anticipated behaviour of the cross‑sections, members, joints and bearings.

(3) 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 Clause 10.

(4) For the calculation of bending effects in composite beams resulting from shrinkage and non-uniform changes of temperature, the effective width of the concrete slab in accordance with 7.4.1.2 may be used.

### 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 be neglected, but where such effects are significant (such as in the case of semi-continuous joints) they should be taken into account, see Clause 11 and EN 1993-1-8.

(2) To identify whether the effects of joint moment-rotation behaviour on the analysis need to be taken into account, a distinction should be made between three joint models as follows, see FprEN 1993-1-8:2023, 11.2 and 7.1.1.

* For a simple joint model, the joint may be assumed not to transmit bending moments.
* For a semi-continuous joint model, the behaviour of the joint needs to be taken into account in the global analysis.
* For a continuous joint model, the behaviour of the joint may be assumed to have no effect on the global analysis.

(3) For buildings, the requirements of the various types of joint given in Clause 11 and in EN 1993‑1‑8 shall apply.

### 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.

## Structural stability

### Effects of deformed geometry of the structure

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

* first‑order analysis, using the undeformed 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 not significant. It may be assumed that second-order effects are insignificant if the conditions EN 1993-1-1:2022, 7.1 and 7.2 are satisfied. For composite columns the effective stiffness should be determined in accordance with 8.8.3.4(2).

(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. For the stiffness of the joints, see Clause 11.

### 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 conditions in EN 1993-1-1:2022, 7.2.1(5) are satisfied for each storey. In these structures αcr may be calculated using the expression given in EN 1993-1-1:2022, 7.2.1(10), provided that the axial compression in the beams is not significant and appropriate allowances are made for cracking of concrete, see 7.4.2.3, creep of concrete, see 7.4.2.2 and for the behaviour of the joints, see 11.2 and FprEN 1993‑1‑8:2023, 7.1.

(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 not required.

(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 should be checked using 8.4.

(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 7.2.2(3), with the resistance of cross‑sections being verified in accordance with 8.8.3.6 or 8.8.3.7, or

b) by analysis of the individual member in accordance with 8.8.3.4, 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 7.3.2.3 with the resistance of cross‑sections being verified in accordance with 8.8.3.6 or 8.8.3.7, 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 8.8.3.5. 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 shall be verified. This requirement may be archieved by member checks based on buckling lengths, in accordance with EN 1993-1-1:2022, 7.2.2(9) and 8.3.

## Imperfections

### 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.

### Imperfections in buildings

#### General

(1) Equivalent geometric imperfections (see 7.3.2.2 and 7.3.2.3) 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 7.3.2.3.

(2) When performing the global analysis for determining forces and moments at member ends to be used in member checks in accordance with 8.4 and/or 8.8, equivalent bow imperfections may be neglected. However, for frames sensitive to second-order effects, equivalent bow imperfections of members additionally to global sway imperfections should be introduced in the structural analysis of the frame for each compressed member if the following conditions are met:

* at least one moment resistant joint at one member end; and
* 

where

|  |  |  |
| --- | --- | --- |
|  | *N*Ed | is the design value of the compression force; and |
|  | *N*cr,eff | is the critical axial force determined for in-plane flexural buckling of the member considered as hinged at its ends using the flexural stiffness given in 8.8.3.4. |

(3) Member imperfections shall always be considered when verifying stability within a member’s length in accordance with 8.8.3.6 or 8.8.3.7.

(4) Imperfections within steel compression members should be considered in accordance with EN 1993-1-1:2022, 7.3.3.

#### Global imperfections

(1) The effects of imperfections should be allowed for in accordance with 7.3.2.1(2) and EN 1993-1-1:2022, 7.3.2.

#### Member imperfections

(1) Design values of equivalent bow imperfection for composite columns and composite compression members should be taken from Table 8.7.

(2) For laterally unrestrained composite beams, the effects of imperfections are incorporated within the formula given for buckling resistance moment (see 8.4).

(3) For steel members, the effects of imperfections are incorporated within the formulae given for buckling resistance (see EN 1993-1-1:2022, 8.3).

## Calculation of action effects

### Methods of global analysis

#### General

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

(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 ultimate limit state caused by fatigue.

(4) The effects of shear lag and of local buckling shall be taken into account if they 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 7.5.

(6) The effects of local buckling of steel elements on stiffness may be ignored in typical composite sections (Figure 8.1). For cross‑sections of Class 4, see FprEN 1993‑1‑5:2023, 4.3.

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

(8) Unless non‑linear analysis is used, the effects of slip and separation on the calculation of internal forces and moments may be neglected at interfaces between steel and concrete where shear connection is provided in accordance with 8.6.

#### 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 1993-1-1:2022, 7.2.1(7) and EN 1993-1-1:2022, Clause 8.

(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*eff,1 at mid‑span for a span supported at both ends, or the value *b*eff,2 at the support for a cantilever.

(5) At mid‑span or at internal support, the total effective width *b*eff, see Figure 7.1, may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (7.1) |

where

|  |  |  |
| --- | --- | --- |
|  | *b*0i | 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 and |
|  | *b*i | is the distance from the outstand shear connector to a point mid‑way between adjacent webs, measured at mid‑depth of the concrete flange, except that at a free edge *b*i is the distance to the free edge. The length *L*e should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, *L*e may be assumed to be as shown in Figure 7.1. |

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

|  |  |  |
| --- | --- | --- |
|  |  | (7.2) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (7.3) |

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 in accordance with Figure 7.1. |

(7) The distribution of the effective width between supports and mid-span regions may be assumed to be as shown in Figure 7.1.

A picture containing diagram, technical drawing, sketch, plan

Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | for *b*eff,1 |
| 2 | for *b*eff,2 |
| 3 | for *b*eff,1 |
| 4 | for *b*eff,2 |

Figure 7.1 — Equivalent spans, for effective width of concrete flange

(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*0i may be taken as zero and *b*i measured from the centre of the web.

### Linear elastic analysis

#### General

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

#### Creep and shrinkage

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

(2) Except as given in (10), the effects of creep may be taken into account using a modular ratio *n*L. The modular ratio depends on the type of loading (subscript L) and is given by:

|  |  |  |
| --- | --- | --- |
|  |  | (7.4) |

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 in accordance with 5.1 (8); |
|  |  | is the creep coefficient *ϕ*(*t*,*t*0) in accordance with EN 1992-1-1:2023, 5.1.5 or Annex M, depending on the age (*t*) of concrete at the moment considered and the age (*t*0) at loading; |
|  |  | is the creep multiplier depending on the type of loading, which should 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. |

NOTE Symbol “*L*”:

* *L*=*0* ;  for non-permanent loads,
* *L = P*:  for permanent loads,
* *L = sh*:  for effects of shrinkage,
* *L = D*:  for pre‑stressing by imposed deformations,
* *L = PT*:  for stresses developing due to creep.

Reference is given in Figure 8.7.

(3) For permanent loads on composite structures cast in several stages, one mean value of *t*0 may be used for the determination of the creep coefficient. This value may also be used for pre-stressing by imposed deformations, if the age of all of the concrete in the relevant spans at the time of pre‑stressing is more than 14 days.

(4) For shrinkage, the age at loading should generally be assumed to be one day.

(5) Where prefabricated slabs are used or when pre‑stressing of the concrete slab is carried out before the shear connection has become effective, the creep coefficient and the shrinkage values from the time when the composite action becomes effective should be used.

(6) Where the bending moment distribution at *t*0 is significantly changed by creep, for example in continuous beams of mixed structures with both composite and non-composite spans, the time-dependent secondary effects due to creep should be considered, except in global analysis for the ultimate limit state for members where all cross‑sections are in Class 1 or 2. For the time dependent secondary effects, the modular ratio may be determined with a creep multiplier of 0,55.

(7) The primary and secondary effects caused by shrinkage and creep of the concrete flange should be taken into account. The effects of creep and shrinkage of concrete may be neglected for analysis of ultimate limit states other than fatigue for composite members with all cross-sections in Class 1 or 2 where there is sufficient plastic redistribution and in which no allowance for lateral-torsional buckling is necessary.

NOTE 1 For serviceability limit states, see Clause 9.

NOTE 2 Sufficient plastic redistribution is assumed when the moment resistance is calculated in accordance with 8.2.1.2, 8.2.1.3 or 8.3.2.

(8) In regions where the concrete slab is assumed to be cracked in line with 7.4.2.3, the primary effects due to shrinkage may be neglected in the calculation of secondary effects.

(9) In composite columns and compression members, the effects of creep in accordance with 8.8.3.4(2) should be taken into account.

(10) For double composite sections with both flanges un-cracked (as outlined in 7.4.2.3; e.g. in case of pre-stressing) the effects of creep and shrinkage shall be determined by more accurate methods than the use of modular ratios in accordance with 7.4.2.2(2).

(11) For simplification in structures for buildings where sway mode failure may be checked by first-order analysis (7.2.2(1)), that are not mainly intended for storage and are not pre‑stressed by controlled imposed deformations, the effects of creep in composite beams may be taken into account by replacing concrete areas *A*c by effective equivalent steel areas *A*c/*n* for both short‑term and long‑term loading, where *n* is the nominal modular ratio corresponding to an effective modulus of elasticity for concrete *E*c,eff taken as *E*cm / 2.

#### Effects of cracking of concrete

(1) Appropriate allowance shall be made for the effects of cracking of concrete.

NOTE 1 This requirement gives rules for the identification of regions that can be considered as cracked in the global analysis and where the concrete is therefore to be neglected. The rules are considered appropriate for the calculation of action effects i.e. moments and shear.

NOTE 2 Rules for the calculation of longitudinal shear are given in 8.6.

(2) The following method may be used for the determination of the effects of cracking in composite beams with concrete flanges:

a) First the envelope of the internal forces and moments for the characteristic combinations (see EN 1990:2023, 8.4.3.2), including long-term effects is calculated using the flexural stiffness *E*a *I*1 (see 3.1.16) of the un-cracked sections. This is defined as “un-cracked analysis”.

b) In regions where the extreme fibre tensile stress in the concrete due to the envelope of global effects, exceeding twice the strength *f*ctm or *η*lw,fct *f*ctm (see EN 1992-1-1:2023, Table 5.1 or Tables M.1 and M.2), the stiffness should be reduced to *E*a *I*2 ( see 3.1.17). The resulting distribution of stiffnesses may be used to determine the distribution of the action effects for all limit states. A new distribution of internal forces and moments, and deformations if appropriate, is then determined by re-analysis. This is defined as “cracked analysis”.

(3) For continuous composite beams with a concrete flange above the steel section and not prestressed, 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. This clause does not apply to members that are part of a moment frame that contributes to the stability of the structure.

(4) The effect of cracking of concrete on the flexural stiffness of composite columns and compression members should be determined in accordance with 8.8.3.4.

(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 should be determined from the plastic stress distribution.

#### 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 fully 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 with all cross-sections in Class 1 or 2 where there is sufficient plastic redistribution and in which no allowance for lateral-torsional buckling is necessary.

NOTE Sufficient plastic redistribution is assumed when the moment resistance is calculated in accordance with 8.2.1.2, 8.2.1.3 or 8.3.2.

#### Temperature effects

(1) Account shall 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 with all cross-sections in Class 1 or 2 where there is sufficient plastic redistribution and in which no allowance for lateral-torsional buckling is necessary.

NOTE Sufficient plastic redistribution is assumed when the moment resistance is calculated in accordance with 8.2.1.2, 8.2.1.3 or 8.3.2.

#### 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 should be calculated with the characteristic or nominal values of properties of materials and of imposed deformation, if the imposed deformations are controlled.

### Non‑linear global analysis

(1) Non‑linear analysis shall be used in accordance with EN 1992-1-1:2023, 7.3.4 and EN 1993-1-1:2022, 7.4.3 or EN 1993-1-14. The non-linear behaviour of material shall be considered.

NOTE Information about material models is given in 8.2.1.5.

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

(3) Effects of the deformed geometry of the structure shall be taken into account in accordance with 7.2.

### Linear elastic analysis with limited redistribution for buildings

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

(2) The bending moment distribution given by a linear elastic global analysis in accordance with 7.4.2 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:

1. in composite beams with full or partial shear connection as given in (4) to (8);
2. in steel members in accordance with EN 1993-1-1:2022, 7.4.1(4);
3. in concrete members subject mainly to flexure in accordance with EN 1992-1-1:2023, 7.3.2; and
4. in partially‑encased beams without a concrete flange, in accordance with (b) or (c), whichever is the more restrictive.

(4) For the verification of ultimate limit states other than fatigue, the elastic bending moments in composite beams may be modified in accordance with (5) to (8) where all the following conditions are fulfilled

* the beam is a continuous composite beam;
* 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;
* no allowance for lateral-torsional buckling is necessary;
* the loading is mainly uniformly distributed. This should not be assumed if, in any span, more than half the design load for the span is concentrated over a length less than one-fifth of the span length; and
* the beam is only subjected to vertical loading.

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

a) by reducing maximum hogging moments by amounts not exceeding the percentages given in Table 7.1, or

b) 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 7.4.2.3, unless it is verified that the rotation capacity permits a higher value.

**Table 7.1 — Limits to redistribution of hogging moments, per cent of the   
initial value of the bending moment to be reduced**

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

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

(7) For composite cross-sections in Class 3 or 4, the limits in Table 7.1 relate to bending moments assumed in design to be applied to the composite member. Moments applied to the steel member should not be redistributed.

(8) At the serviceability limit state for continuous beams with an upper concrete flange, a moment redistribution of 10% of the hogging support moment may be used based on an un-cracked analysis for action effects. Alternatively the moment distribution at SLS may be determined in accordance with 7.4.2.3.

(9) For partially-encased sections [Figure 8.1b) and 8.1e)], moment redistribution in accordance with Table 7.1 for Class 1 should not be used when the concrete encasement and reinforcement is taken into account for the bending resistance in 8.3.2, unless the rotation capacity of the section has been proven to be sufficient.

(10) For cross-sections in accordance with 8.2.1.2(2) where the plastic moment resistance cannot be reached, the limits to redistribution of hogging moment should not exceed those limits given in Table 7.1 for Class 3.

### 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 is in accordance with EN 1993-1-1:2022, 5.2.2;
* the cross‑sections of steel members are in accordance with EN 1993-1-1:2022, 7.6; 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 normally not necessary to consider the effects of alternating plasticity (shakedown).

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

1. the cross‑section of the structural steel section shall be symmetrical about a plane parallel to the plane of the web or webs;
2. the proportions and restraints of steel components shall be such that lateral‑torsional buckling does not occur;
3. lateral restraint to the compression flange shall be provided at all hinge locations at which plastic rotation may occur under any load case;
4. 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
5. 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:

1. the grade of structural steel does not exceed S355;
2. the contribution of any reinforced concrete encasement in compression is neglected when calculating the design resistance moment;
3. 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;
4. each beam-to-column joint has been shown to have sufficient design rotation capacity, or to have a design resistance moment at least that of the connected beam where the resistance of the connected beam is calculated using the mean values of concrete strength (EN 1992-1-1:2023, Table 5.1), 1,08 *f*sk for the reinforcement and a design strength of structural steel of *f*yk times the overstrength factor in FprEN 1993-1-8:2023, Table B.4.
5. adjacent spans do not differ in length by more than 50% of the shorter span;
6. end spans do not exceed 115% of the length of the adjacent span;
7. 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
8. 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 1993-1-1:2022, 7.6(3) is applicable.

(7) Where restraint is required as defined 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.

## Classification of cross‑sections

### General

(1) The Classification system defined in EN 1993-1-1:2022, 7.5.2 applies to cross‑sections of composite beams.

(2) A composite section should be classified in accordance with the least favourable Class of its steel elements in compression.

NOTE The Class of a composite section normally depends on the sign 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 1992-1-1:2023, Table 5.5. Additionally for a section whose resistance moment is determined by 8.2.1.2, 8.2.1.3 or 8.2.1.5, a minimum area of reinforcement *A*s within the effective width of the concrete flange should be provided to satisfy the condition given in:

|  |  |  |
| --- | --- | --- |
|  |  | (7.5) |

with

|  |  |  |
| --- | --- | --- |
|  |  | (7.6) |

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/mm2; |
|  | *f*sk | is the characteristic yield strength of the reinforcement; |
|  | *f*ctm | is the mean tensile strength of the concrete, see EN 1992-1-1:2023, Table 5.1 or Tables M.1 and M.2; |
|  | *k*c | is a coefficient given in 9.4.2(1); |
|  |  | is equal to 1,0 for Class 2 cross‑sections, and equal to 1,1 for Class 1 cross‑sections at which plastic hinge rotation is required. |

(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.

### 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 8.6.10.5.

(2) The Classification of other steel flanges and webs in compression in composite beams without concrete encasement should be in accordance with EN 1993-1-1:2022, Table 7.2. 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 EN 1993-1-1:2022, 8.2.2.4.

### 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 7.2.

(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:

1. the concrete that encases a web is reinforced by longitudinal bars and stirrups, and/or welded mesh;
2. the requirements for the ratio *b*c / *b* given in Table 7.2 are fulfilled;
3. the concrete between the flanges is fixed to the web in accordance with Figure 8.12 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
4. 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.

**Table 7.2 — Classification of steel flanges in compression for partially‑encased sections**

|  |  |  |
| --- | --- | --- |
| A picture containing diagram, technical drawing, sketch, plan  Description automatically generated | | A picture containing line, diagram, screenshot, sketch  Description automatically generated  Stress distribution  (compression positive) |
| Class | Type | Limit |
| 1 | (1) rolled or (2) welded |  |
| 2 |  |
| 3 |  |

# Ultimate limit states

## Beams

### General

(1) Design resistances of composite cross‑sections in bending and vertical shear should be determined in accordance with 8.2 for composite beams with uncased steel sections, and 8.3 for partially encased steel sections. For shallow floor beams in buildings, use the additional rules in Annex I (normative).

NOTE 1 In general, composite beams consist of a steel cross-section with a concrete flange, as given in Figure 8.1.

NOTE 2 For composite beams used in bridges, further guidance is given in EN 1994-2, for composite beams used in buildings refer to (2).

(2) For composite beams used in buildings, typical types of cross‑section are shown in Figure 8.1 with either a solid slab [Figure 8.1a), b), c)] or a composite slab [Figure 8.1d), e) and f)]. Partially–encased beams [Figure 8.1b) and e)], 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.

|  |  |  |
| --- | --- | --- |
|  |  |  |
| a) | b) | c) |
|  |  | A picture containing screenshot, rectangle, line, font  Description automatically generated |
| d) | e) | f) |

Figure 8.1 — Typical cross‑sections of composite beams

(3) Composite beams shall be checked for:

* resistance of critical cross‑sections (8.2 and 8.3);
* resistance to lateral‑torsional buckling (8.4);
* resistance to shear buckling (8.2.2.4) and transverse forces on webs (8.5);
* resistance to longitudinal shear (8.6);
* resistance to fatigue (8.7); and
* resistance to plate buckling for Class 4 cross-sections (8.2.1.4).

(4) Critical cross‑sections include sections:

* at positions of maximum bending moment;
* at supports;
* subjected to concentrated loads or reactions; and
* at 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; and
* 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) The concepts "full shear connection" and "partial shear connection" are applicable only to beams in which plastic bending resistance is used in accordance with 8.2.1.2, 8.2.1.3 and 8.3.2. Limits to the use of partial shear connection are given in 8.6.3.3.

### 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 7.4.1.2 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*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*eff,2 at the relevant support.

## Resistances of cross‑sections of beams

### Bending resistance

#### General

(1) The plastic design bending resistance should be applied 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 resistance for bending can be applied to cross-sections of any Class. Non–linear resistance may be applied to Class 1 or Class 2 cross-sections. For Class 3 cross-section, non-linear resistance may only be applied for structural steel in tension.

(3) When determining elastic or non‑linear resistance, it may be assumed that the composite cross‑section remains plane if the shear connection and the transverse reinforcement are designed in accordance with 8.6, considering appropriate distributions of design longitudinal shear force.

(4) The tensile strength of concrete shall be ignored.

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

(6) When any zones of the lower flange are in compression, the beam should be checked for lateral torsional buckling in accordance with 8.4.

(7) Additional rules for beams in bridges and beams subjected to biaxial bending, combined bending and torsion are given in prEN 1994-2:2024, 8.2.1.3(1).

(8) For widely spaced web openings, the impact of an opening on the moment resistance may be neglected when:

- the maximum dimension of the web opening is less than 10% of the steel section depth; and

- the eccentricity of the centre line of the opening relative to the centre line of the web does not exceed 10% of the steel section depth;

otherwise the influence on internal forces and secondary bending should be considered.

(9) For narrowly spaced openings or where (8) does not allow the impact of widely spaced openings to be neglected, use Annex D.

NOTE Annex E extends Annex D to cover web openings in beams where the flexural stiffness of the slab is significant.

#### Plastic moment resistance *M*pl,Rd of a composite cross‑section

(1) The following assumptions should be made in the calculation of *M*pl,Rd:

1. there shall be full interaction between structural steel, reinforcement, and concrete;
2. the effective area of the structural steel member shall be stressed to its design yield strength *f*yd in tension or compression;
3. the effective areas of longitudinal reinforcement in tension and in compression shall be stressed to their design yield strength *f*sd in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected; and
4. the effective area of concrete in compression shall resist a stress of *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 in accordance with 5.1(3) .

Typical plastic stress distributions are shown in Figure 8.2.

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Figure 8.2 — Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending

(2) To avoid concrete failure before the plastic moment resistance is reached, where the ratio *z*pl/*h* exceeds the value of 0,2 for a steel grade not greater than S355, or 0,15 for steel grades S420/S460, the resistance to bending *M*pl,Rd should be:

1. determined in accordance with 8.2.1.5.
2. alternatively, as a simplification, the design resistance moment *M*Rd of a composite beam fulfilling the conditions:

* Concrete Class C20/25 – C50/60;
* (*h*c + *h*pg)/*h*a is between 0,15 – 0,7;
* the overall height of any profiled sheeting, *h*pg, is smaller than 135 mm;
* the ratio of the areas of the top and bottom flange, *A*ft/*A*fb, is smaller than 1,0;
* the ratio of the areas of the bottom and top flange, *A*fb/*A*ft, is smaller than 3,0;

may be taken as *β* *M*pl,Rd, where *β* is the reduction factor given in Figure 8.3. The dimension *z*pl is the distance between the plastic neutral axis, determined assuming full interaction between steel and concrete, and the extreme fibre of the concrete flange in compression, *h* is the overall depth of the member. If there is more than one steel grade in the cross-section, the highest grade should be used.

|  |
| --- |
| A diagram of a rectangular object  Description automatically generated |
| A graph with numbers and lines  Description automatically generated |

**Key**

|  |  |
| --- | --- |
| 1 | Composite beam, concrete flange by solid slab |
| 2 | Composite beam, concrete flange with non-continuous profiled sheeting or semi-prefabricated concrete element |
| 3 | Composite beam, concrete flange with continuous profiled steel sheeting |
| 4 | Reduction factor *β* |
| 5 | Application range of simplified moment reduction by factor *β* |

**Figure 8.3 — Reduction factor *β* for *Mpl,Rd***

(3) When the conditions in (2)b are satisfied, the coefficient β may be taken as 1,0 if:

1. the compression resistance of the full concrete flange is at least 2 times the full plastic resistance *N*pl,a of the steel section; or
2. all the following conditions are satisfied:

* the section is an IPE, HEAA, HEA, HEB, UB or a welded section with equivalent geometry;
* the steel grade is not greater than S355;
* the depth of the steel section *h*a is not less than 2,5 times the height of the concrete flange (*h*c + *h*pg);
* the effective width of the concrete flanges is not less than 1,5 m;
* the plastic neutral axis assuming full interaction is not below the top flange of the steel profile; and
* the concrete flange is considered to be either solid or with profiled sheeting and a concrete depth *hc* of at least 70 mm.

(4) Where all the conditions given in (3) b are satisfied, except that the effective width of the concrete flange is less than 1,0 m (but not less than 0,5 m), the moment resistance *M*Rd given in (3)b should be used with an additional reduction factor of 0,90.

(5) Where plastic resistance is used and reinforcement is in tension, that reinforcement should be in accordance with 7.5.1(5).

(6) For buildings, profiled steel sheeting should be neglected.

#### Plastic resistance moment of sections with partial shear connection in buildings

(1) In regions of sagging bending, partial shear connection in accordance with 8.6.1 and 8.6.3 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 8.2.1.2 and appropriate shear connection should be provided to ensure yielding of reinforcement in tension.

A diagram of a machine

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**Figure 8.4 — Plastic stress distribution under sagging bending for partial shear connection**

(3) Where shear connectors of Ductility Category D2 and D3 in accordance with Table 5.1 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 8.2.1.2(1), except that a reduced value of the compressive force in the concrete flange *N*c should be used in place of the force *N*c,f given by 8.2.1.2(1)d. The ratio *η* = *N*c /*N*c,f is the degree of shear connection.

The location of the plastic neutral axis in the concrete flange should be determined by the force *N*c (see Figure 8.4). There is a second plastic neutral axis within the steel section, which should be used for the classification of the web.

A diagram of a diagram of a beam

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**Key**

|  |  |
| --- | --- |
| 1 | plastic theory |
| 2 | simplified method |

Figure 8.5 — Relation between *M*Rd(η) and *N*c (for ductile shear connectors)

(4) The relation between *M*Rd and *N*c in (3) is qualitatively given by the convex curve ABC in Figure 8.5 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 from Formula (8.1) which corresponds to the straight line AC (line 2) in Figure 8.5:

|  |  |  |
| --- | --- | --- |
|  |  | (8.1) |

(6) When a reduction of the plastic moment resistance *M*pl,Rd in accordance with 8.2.1.2(2) is required, in the absence of more advanced methods, e.g. in accordance with 8.2.1.5, 8.2.1.6 and 8.6.2, the moment resistance *M*Rd with partial shear connection may be determined in accordance with (7). The requirements defined in this sub-clause are illustrated by lines 4 and 6 respectively in Figure 8.6. The requirements in (7) apply only to beams where the plastic neutral axis for full shear connection is not below the top steel flange

(7) The reduction in the moment resistance may be taken as proportional to the degree of shear connection. The design moment resistance for partial shear connection where *β* in accordance with 8.2.1.2 (2) applies may therefore be calculated for cross-sections by reducing the moment resistance calculated in accordance with (3) and (4) (Curve 1) in Figure 8.6), using a factor (1-(1-*β*)*η*) to give the design moment resistance by rotation of Curve 1 to Curve 4. Alternatively Curve 6 can be used.

Where the curve reaches the moment resistance *β* *M*pl,Rd, the degree of shear connection should be *η* = 1,0.

where:

|  |  |  |
| --- | --- | --- |
|  | *β* | is the reduction factor from 8.2.1.2(2) based on the depth of plastic neutral axis for the section with full shear connection; |
|  | *η* | is the ratio *N*c/*N*c,f from (3). |

A diagram of a curve

Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | partial connection theory based on plastic theory (*β* = 1), in accordance with 8.2.1.3(4) |
| 2 | partial connection theory based on simplified method (*β* = 1), in accordance with 8.2.1.3(5) |
| 3 | degree of shear connection applying Curve 1 in accordance with 8.2.1.3(4) for |
| 4 | partial shear connection for based on approximat strain limited theory in accordance with 8.2.1.3(7) |
| 5 | degree of shear connection in accordance with 8.2.1.3 (7) for applying Curve 4 |
| 6 | partial connection theory for based on alternative approximate strain limited theory in accordance with 8.2.1.3(7) |
| 7 | degree of shear connection in accordance with 8.2.1.3(7) applying Curve 6 |
| 8 | when applying Curve 6 in accordance with 8.2.1.3(7), for this zone the degree of shear connection η = 1,0 |

Figure 8.6 — Relation between *M*Rd and *η* when factor *β* applies (for ductile shear connectors)

#### Elastic resistance to bending

(1) Stresses should be calculated by elastic theory, using an effective width of the concrete flange in accordance with 8.1.2. For cross‑sections in Class 4, the effective area of the structural steel section should be determined in accordance with FprEN 1993-1-5:2023, 6.3.

(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 (see Clause 5.1(2));
* *f*yd in structural steel in tension or compression (see Clause 5.3);
* *f*sd in reinforcement in tension or compression (see Clause 5.2). 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 in accordance with 7.4.2.2.

(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) The elastic resistance to bending is given by Formula (8.2). Effects due to loading sequence, temperature and pre-stressing should be taken into account.

|  |  |  |
| --- | --- | --- |
|  |  | (8.2) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*a,Ed, *N*a,Ed | are the design bending moment and axial force, respectively, applied to the structural steel section before composite action is achieved; |
|  | *M*i,Ed | is the sum of all time dependant moments Σ*M*i,L,Ed and effects of axial forces Σ*N*i,L,Ed *z*i applied to the transformed composite cross-sections, determined in accordance with 7.4.2, with: |
|  | *M*i,0,Ed, *N*i,0,Ed | are the bending moment and axial force respectively, from non-permanent actions on the transformed composite section; |
|  | *M*i,P,Ed, *N*i,P,Ed | are the bending moment and axial force respectively, from permanent actions on the transformed composite section; |
|  | *M*i,D,Ed, *N*i,D,Ed | are the bending moment and axial force respectively, due to pre-stressing by imposed deformation on the transformed composite section; |
|  | *M*i,sh,Ed, *N*i,sh,Ed | are the bending moment and axial force respectively, due to shrinkage on the transformed composite cross-section; |
|  | *M*i,PT,Ed, *N*i,PT,Ed | are the bending moment and axial force, developing in time due to creep (indirect action, see 7.4.2.2(6) |
|  | *k*el | is the lowest factor such that a stress limit in (2) is reached.  A diagram of a graph  Description automatically generated |

**Key**

|  |  |
| --- | --- |
| 1 | on steel section |
| 2 | composite section |
| 3 | resulting stresses on composite section |

**Figure 8.7 — Elastic stress distribution under sagging bending for composite section without pre-stressing**

(7) For buildings, the determination of Σ*M*i,L,Ed may be simplified by using 7.4.2.2(11).

(8) When any zones of the lower flange are in compression, the beam should be checked for lateral torsional buckling in accordance with 8.4.

(9) For composite beams with cross-sections in Class 4 designed in cordance with FprEN 1993-1-5:2023, Clause 6, the sum of stresses in a cross-section from different stages of construction and use should be used for calculating the effective steel cross-section at the time considered. These effective cross-sections should be used for checking stresses in the composite section at the different stages of construction and use.

(10) Additional rules for consideration of prestressing steel are given in EN 1994-2.

#### Non-linear resistance based on stress-strain relationships

(1) Where the bending resistance of a composite cross‑section of Class 1 to 3 is determined by a non‑linear method, the stress‑strain relationships of the materials and the effects of the method of construction (e.g. propped or un‑propped) shall be taken into account.

NOTE 1 Where the limiting strain in the concrete is reached before complete yield of the steel bottom flange, effects due to loading sequence, temperature and pre-stressing are to be considered.

NOTE 2 Creep and shrinkage are to be considered when leading to unfavourable effects.

(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.

NOTE In more advanced methods, based on stress-strain curves, the moment resistance and associated concrete compression force *N*cf for full shear interaction assume the composite cross-section remains plane. Partial shear interaction takes into account the discontinuity in strain between concrete flange and steel section for the determination of the moment resistance and associated concrete compression force *N*c.

(3) The stresses in the concrete in compression may be derived from the stress‑strain curves given in EN 1992-1-1:2023, 8.1.2 and the requirement of EN 1992-1-1:2023, 8.1.1. Alternatively, the stress-strain curve for concrete may be derived from EN 1992-1-1:2023, 5.1.6 and *f*cm shall be substituted by the design compressive strength *f*cdand *E*cm by *E*cdin accordance with EN 1992-1-1:2023, 7.4.3.3(3).

(4) The stresses in the reinforcement should be derived from the bi‑linear diagrams given in EN 1992-1-1:2023, 5.2.4.

(5) The stresses in the structural steel should be derived from the bi‑linear diagram given in EN 1993-1-1:2022, 7.4.3(3) or stress-strain relationship given in prEN 1993-1-14:2023, 5.3.2.

#### Non-linear resistance to bending

(1) 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 Formula (8.3) and Formula (8.4), as shown in Figure 8.8:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  | for | (8.3) |
|  |  | for | (8.4) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*el,Rd | is as defined in 8.2.1.4; |
|  | *M*a,Ed  *M*pl,Rd | is the design bending moment applied on the structural steel section before composite behaviour;  is the plastic moment resistance of a composite cross-section defined in 8.2.1.2; |
|  | *N*c,el | is the compressive force in the concrete flange corresponding to moment *M*el,Rd; |
|  | *N*c | is the compressive force in the concrete flange; |
|  | *N*c,f | is the design value of the compressive axial force in the concrete flange with full shear connection. |

(2) For cross-sections where 8.2.1.2(2) applies, in Formula (8.3) and in Figure 8.8 instead of *M*pl,Rd the reduced value *β* *M*pl,Rd or the nonlinear moment resistance assuming full interaction should be used.

(3) Additional rules for consideration of prestressing are given in EN 1994-2.

A graph of a function

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | propped construction |
| 2 | unpropped construction |

Figure 8.8 — Simplified relationship between *M*Rd and *N*c for sections   
with the concrete slab in compression

### Resistance to vertical shear

#### Scope

(1) Clause 8.2.2 applies to composite beams with a rolled or welded structural steel section with a solid web, which may be stiffened.

NOTE 1 Additional rules for bridges are given in EN 1994-2.

NOTE 2 Additonal rules for beams with web-opening are given in Annexes D and E.

#### Plastic resistance to vertical shear

(1) The shear resistance of a composite beam without concrete encasement should be determined as the resistance *V*pl,a,Rd of the structural steel section in accordance with EN 1993-1-1:2022, 8.2.6, unless a value for a contribution from the reinforced concrete part of the beam has been established. Where a torsional moment is acting with a shear force, the plastic shear resistance should be reduced from *V*pl,a,Rd to *V*pl,a,T,Rd as given in EN 1993-1-1:2022, 8.2.7(9).

(2) The contribution of the concrete flange to shear may be taken into account for the determination of the design shear resistance for cross-section of Class 1 or 2 only if an adequate shear connection in accordance with 8.6 is provided. The shear connectors should be designed for additional loading due to transfer of vertical shear forces. Where only the steel part is connected to the support, the contribution of the concrete should be neglected.

NOTE 1 For the consideration of a contribution from the concrete flange to vertical shear resistance, more complex design methods are required, considering non-linear behaviour of the materials and the shear connector, and verifying the shear resistance of the effective concrete section.

NOTE 2 Additional rules for shallow floor beams are given in Annex I.

#### Elastic resistance to shear

(1) The elastic resistance may be assumed for all cross-section Classes. For Class 4 cross-sections, the effective structural steel section should be used as determined in EN 1993-1-5.

(2) For elastic design, the shear resistance should be verified in accordance with EN 1993-1-1:2022, 8.2.6(4) taking account of the sequence of construction, creep, shrinkage, temperature and pre-stressing effects. The first moment of area and the second moment area to be used in the verification should be time dependent properties based on the modular ratio in 7.4.2.2.

#### Shear buckling resistance

(1) The shear buckling resistance *V*b,Rd of an uncased steel web should be determined in accordance with FprEN 1993-1-5:2023, Clause 7, if the web slenderness is such that the condition in accordance with Formula (8.27) in EN 1993-1-1:2022, 8.2.6(6) applies.

(2) No account should be taken of a shear contribution from the concrete slab, unless a more precise method than the one of FprEN 1993-1-5:2023, Clause 7 is used and unless the shear connection is designed for the relevant vertical force.

(3) When applying FprEN 1993-1-5:2023, 7.4(1) for a beam with a single composite flange, the dimensions of the non-composite flange may be used even if that is the larger steel flange. The axial force *N*Ed in FprEN 1993-1-5:2023, 7.4(2) should be taken as the axial force acting on the composite section. For the composite flange the effective area should be used.

#### Bending and vertical shear

(1) The interaction of the shear force acting on the structural steel section with bending effects should be considered in accordance with EN 1993-1-1:2022, 8.2.8 or FprEN 1993-1-5:2023, Clause 9. Where no contribution from the reinforced concrete part of the beam has been established *Va,Ed* is equal to the shear force *VEd* acting on the composite section.

Where the vertical shear force *V*Ed exceeds *η*V⋅*V*a,Rd, where *V*a,Rdis given by *V*pl,a,Rd in 8.2.2.2 or *V*b,Rd in 8.2.2.4, whichever is the smaller, allowance should be made for its effect on the resistance moment.

NOTE The value of *η*V is 0,5 unless the National Annexes provide an other value.

(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 use of a reduced design steel strength (1 - ρ) *f*yd in the shear area, as shown in Figure 8.9, where the value of ρ is in accordance with EN 1993-1-1:2022, 8.2.8 or 8.2.10(3). For elastic design the sequence of loading, effects of creep and shrinkage, of pre-stressing and temperature should be taken into account. For cross-sections that are not subject to shear buckling, the stresses from bending and shear can be considered as given by EN 1993-1-1:2022, 8.2.1(4) and 8.2.1(5). For Class 4 cross-sections, the effective steel section should be determined in accordance with EN 1993-1-5.

(4) For cross-sections in Classes 3 and 4 where shear buckling is to be taken into account in accordance with FprEN 1993-1-5:2023, 9.1, *V*Ed should be taken as equal to *V*a,Ed if no account is taken of a the contribution from the concrete slab in accordance with 8.2.2.4(2).

(5) For the calculation of *M*f,Rd in FprEN 1993-1-5:2023, 9.1(1), the design plastic resistance to bending of the effective composite section, excluding the steel web, should be used.

A diagram of a rectangular object

Description automatically generated

**Figure 8.9 — Plastic stress distribution modified by the effect of vertical shear**

## Resistance of cross‑sections of beams with partial encasement for buildings

### Scope

(1) Partially–encased beams are defined in 8.1.1(2). A concrete flange can also form part of the effective section of the composite beam, provided that it is attached to the steel section by shear connection in accordance with 8.6. Typical cross‑sections are shown in Figure 8.10.

NOTE  Additional rules for shallow floor beams are given by Annex I.

(2) Clause 8.3 is applicable to partially-encased sections in Class 1 or Class 2, provided that *d*w / *t*w is not greater than 124 *ε*.

A picture containing sketch, rectangle, diagram

Description automatically generated

Figure 8.10 — Typical cross‑sections of partially‑encased beams

### Bending resistance

(1) Full shear connection should be provided between the structural steel section and the web encasement in accordance with 8.6.

(2) The design resistance moment may be determined by plastic theory for cross-sections of Class 1 and 2. Reinforcement in compression in the concrete encasement may be neglected.

NOTE Some examples of typical plastic stress distributions are shown in Figure 8.11.

(3) Partial shear connection may be used for the compressive force in any concrete slab forming part of the effective section. For the reinforcement in concrete in tension, full-shear connection should be provided.

(4) Where partial shear connection is used with connectors of Ductility Category D2 or D3 in accordance with 5.4.2.1, the plastic resistance moment of the beam should be calculated in accordance with 8.3.2(2) and 8.2.1.2(1), except that a reduced value of the compressive force in the concrete or composite slab *N*c should be used as in 8.2.1.3(3), (4) and (5).

(5) Where the ratio *z*pl/*h* exceeds the value given by 8.2.1.2(2), the resistance to bending *M*pl,Rd should be determined in accordance with 8.2.1.4 or 8.2.1.5, ignoring the concrete in tension. Alternatively, the design resistance moment *M*Rd of composite beams fulfilling the conditions defined in Figure 8.3 may be taken as *β M*pl,Rd, where *β* is the reduction factor given in Figure 8.3.

(6) When a reduction of the moment resistance *M*pl,Rd in accordance with 8.3.2(5) and 8.2.1.2(2) is required, the concrete compression force *N*c for partial shear connection of the concrete flange may be determined by the approximation given in 8.2.1.3(7), when the position of the plastic neutral axis is in the concrete flange above the steel section.

(7) Where the bending resistance of a composite cross‑section is determined by non‑linear resistance, see 8.2.1.5 or 8.2.1.6.

A picture containing diagram, technical drawing, rectangle, sketch

Description automatically generated

**Figure 8.11 — Examples of plastic stress distributions for effective sections**

### 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 8.2.2.2.

(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 8.12. 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 1992-1-1:2023, 8.2.3 and the other relevant design requirements of that standard, where the plastic neutral axis is in the concrete encasement.

(5) Where the plastic neutral axis is in the concrete flange outside the steel section, the transfer of the strut and tie forces due to shear from the concrete flange to the encasement via the steel section and appropriate shear connection should be verified. Otherwise the contribution of the concrete encasement to the vertical shear resistance should be neglected.

A black and white picture of a rectangular object

Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | closed stirrups |
| 2 | open stirrups welded to the web |
| 3 | stirrups through the web |

Figure 8.12 — Arrangement of stirrups

### Bending and vertical shear

(1) The interaction of bending and shear should be considered in accordance with 8.2.2.5.

(2) Where the influence of shear requires the use of a reduced yield strength for the shear area of a section, the design reduced plastic resistance moment *M*Rd should be calculated in accordance with 8.3.2.

## Lateral‑torsional buckling of composite beams

### General

(1) A steel flange that is attached to a concrete flange by shear connection in accordance with 8.6 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 1993-1-1:2022, 8.3.2.1 to 8.3.2.3 and, more generally, 8.3.4 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 7.4.2.4. 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 8.4.2 may be used.

### Verification of lateral‑torsional buckling of continuous composite beams with cross- sections in Class 1, 2 and 3

(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 determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.5) |

where:

|  |  |  |
| --- | --- | --- |
|  | *χ*LT | is the reduction factor for lateral‑torsional buckling depending on the relative slenderness ; |
|  | *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 *χ*LT should be obtained from EN 1993-1-1:2022, 8.3.2.3(2).

(2) For cross‑sections in Class 1 or Class 2, *M*Rd should be determined in accordance with 8.2.1.2 for a beam whose bending resistance is based on plastic theory, or 8.2.1.5 for a beam whose bending resistance is based on non‑linear analysis, or 8.3.2 for a partially‑encased beam, with *f*yd determined using the partial factor γM1 given in EN 1993-1-1:2022, 8.1(1).

(3) For cross‑sections in Class 3, *M*Rd should be determined using Formula (8.2), 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 γM1 given by EN 1993-1-1:2022, 8.1(1).

(4) The relative slenderness should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.6) |

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 8.4.3(1)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 8.13, 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.

A picture containing sketch, drawing, diagram

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | cracks |

Figure 8.13 — 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 given by Formula (8.7) may be adopted to represent the U‑frame model by a beam alone.

|  |  |  |
| --- | --- | --- |
|  |  | (8.7) |

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 determined from: |

|  |  |  |
| --- | --- | --- |
|  |  | (8.8) |

|  |  |  |
| --- | --- | --- |
|  | αs | where αs is a parameter taken as αs = 2 for an edge beam, with or without a cantilever, αs = 3 for an inner beam. For inner beams in a floor with four or more similar beams, αs = 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 midspan, for sagging bending, and the value at the supporting member, for hogging bending; |
|  | *k*2 | is the flexural stiffness of the steel web, to be taken as that given in Formula (8.9) for an uncased steel beam;. |

|  |  |  |
| --- | --- | --- |
|  |  | (8.9) |

|  |  |  |
| --- | --- | --- |
|  | *ν*a | Poisson’s ratio for structural steel; and |
|  | *h*s and *t*w | are defined in Figure 8.13. |

(7) For a steel beam with partial encasement in accordance with 7.5.3(2), the flexural stiffness *k*2 may take account of the encasement and be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.10) |

where:

|  |  |  |
| --- | --- | --- |
|  | *n* | is the modular ratio for long‑term effects in accordance with 7.4.2.2; and |
|  | *b*c | is the width of the concrete encasement, see Figure 8.10. |

(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.

### Simplified verification of lateral‑torsional buckling of continuous composite beams with cross- sections in Class 1, 2 and 3, without direct calculation for beams in buildings

(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:

1. 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.
2. The loading on each span is uniformly distributed, and the design permanent load exceeds 40% of the total design load.
3. The top flange of the steel member is attached to a reinforced concrete or composite slab by shear connectors in accordance with 8.6.
4. 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 8.13.
5. If the slab is composite, it spans between the two supporting members of the inverted-U frame considered.
6. At each support of the steel member, its bottom flange is laterally restrained and its web is stiffened. Elsewhere, the web is un-stiffened.
7. 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 8.1.
8. If the steel member is partially-encased in concrete in accordance with 7.5.3 (2), its depth *h* does not exceed the limit given in Table 8.1 by more than 200 mm for steel grades up to S355 and by 150 mm for grades S420 and S460.

**Table 8.1 — Maximum depth *h* (mm) of uncased steel members for which 8.4.3 is applicable**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Steel member | Maximum depth *h*max, in mm, for nominal steel grade | | | |
| S235 | S275 | S355 | S420 and S460 |
| IPE | 600 | 550 | 400 | 270 |
| HEA | 800 | 700 | 650 | 500 |
| HEB | 900 | 800 | 700 | 600 |

i) As an alternative to (g) and (h), if the steel member is a doubly symmetric hot-rolled I or H section, where the section parameter calculated in accordance with Formula (8.11) is less than the limit given in Table 8.2.

|  |  |  |  |
| --- | --- | --- | --- |
|  | Section parameter |  | (8.11) |

**Table 8.2 — Section parameter limits**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | S235 | S275 | S355 | S420 and S460 |
| Uncased web | 15,1 | 13,9 | 12,3 | 10,8 |
| Encased web | 15,1 | 18,0 | 15,8 | 13,9 |

## Transverse forces on webs

### General

(1) The requirements given in FprEN 1993-1-5:2023, Clause 8 to determine the design resistance of an unstiffened or stiffened web to transverse forces applied through a flange should be used for the non‑composite steel flange of a composite beam, and to the adjacent part of the web. The simplified requirements in EN 1993-1-1:2022, 8.2.11 may also be used.

(2) If the transverse force acts in combination with bending and axial force, the resistance should be verified in accordance with FprEN 1993‑1‑5:2023, 9.2.

(3) For buildings, at an internal support of a beam designed using an effective web in Class 2 in accordance with 7.5.2(3), transverse stiffening should be provided unless it has been verified that the un‑stiffened web has sufficient resistance to crippling and buckling.

### Flange‑induced buckling of webs

(1) FprEN 1993-1-5:2023, Clause 10 is applicable provided that the effective cross-sectional area of the compression flange, *A*fc, is 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.

## Shear connection

### Basis of design

(1) Subclause 8.6 is applicable to composite beams and, as appropriate, to other types of composite members.

(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) The requirement in (4) may be assumed to be satisfied if a shear connector is in Ductility Category D2 or D3 as defined in Table 5.1.

(6) 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) 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 8.6.10.7 may be assumed to provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension.

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

(11) The requirement in (10) may be assumed to be satisfied if the detailing of the shear connection is in accordance with the appropriate provisions of 8.6.10, and the transverse reinforcement is in accordance with 8.6.11.

(12) Where concentrated longitudinal shear forces occur, local effects of longitudinal slip should be taken into account; for example, as provided in 8.6.7. Otherwise, the effects of longitudinal slip may be neglected.

(13) The effects from the loading sequence, creep, shrinkage, temperature and pre-stressing should be taken into account when they significantly affect the distribution of the longitudinal shear forces.

NOTE These effects can be neglected when the moment resistance is calculated in accordance with 8.2.1.2, 8.2.1.3 or 8.3.2.

(14) Where a method of interconnection, other than the shear connectors included in this standard 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 design model. The design of the composite member should conform to the design of a similar member employing shear connectors included in this standard, in so far as is practicable.

### General method using non-linear analysis

(1) This subclause is applicable to all Class 1 and 2 sections. It is also applicable to Class 3 sections where the steel only yields in tension.

(2) The distribution of longitudinal shear per unit length over the entire length of the composite beam should be determined by non-linear analysis.

(3) For such calculation the non-linear material behaviour of steel, reinforcement and concrete should be taken into account with reference to 8.2.1.5.

(4) The stiffness and ductility of a shear connector, defined by its load-slip relationship *P*-*δ* (with reference to Table 5.1), should be taken into account. The maximum slip *δ* should not exceed the slip capacity *δ*uk of the shear connector. Therefore, shear connectors of any Ductility Category may be used.

(5) Any tests to determine the load-slip relationship *P*-*δ* are to be carried out in accordance with Annex B.

### Beams in buildings where plastic theory is used for the resistance of the cross-section

#### General

(1) These requirements apply to beams where the bending resistance of the cross-section is determined by plastic theory using full or partial shear connection and with shear connectors of Ductility Category D2 or Category D3.

(2) The requirements are applicable to buildings and may also be relevant to other situations where fatigue is not a concern.

#### Longitudinal shear force and spacing of shear connectors in beams for buildings

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

(2) 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.

(3) In buildings, if all cross-sections are in Class 1 or Class 2, 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. For partial shear connection, reference should be made to 8.2.1.3 or 8.3.2 and for full shear connection reference should be made to 8.2.1.2, or 8.3.2, as appropriate.

(4) 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.

(5) Ductile connectors may be spaced uniformly over a length between adjacent critical cross-sections as defined in 8.1.1 provided that:

* all critical sections in the span considered are in Class 1 or Class 2;
* *η* satisfies the limit given by 8.6.3.3 or a more advanced proof in accordance with 8.6.2 is provided; and
* the plastic resistance moment of the composite section does not exceed 2,5 times the plastic resistance moment of the steel member alone.

(6) The number of connectors should be at least equal to the total design shear force for the ultimate limit state, divided by the design resistance of a single connector *P*Rd. For stud connectors, the design resistance should be determined in accordance with 8.6.8 or 8.6.9, as appropriate

(7) 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.

(8) 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.

(9) For composite cross-section where the plastic neutral axis for full shear connection is in the steel section, additional checks on the adequacy of the shear connection should be made at intermediate points approximately mid‑way between adjacent critical cross‑sections described in (5) for the arrangement of the shear connectors over the beam length.

(10) For cross-sections classified in cross-section Class 1 or Class 2 where the full-plastic moment resistance cannot be reached (see 8.2.1.2,(2)), the distribution of the shear connectors should be determined in accordance with 8.6.2, 8.6.4 or 8.6.5.

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

(1) For partial shear connection, either the calculated maximum slip should not exceed the capacity given in 5.4.2.1 or the degree of shear connection should comply with the limit in (2).

(2) Where all the following conditions apply:

* the beam is predominantly symmetrically loaded;
* ductile shear connectors of Ductility Category D2 or Category D3 are used;
* the concrete grade does not exceed concrete resistance Class C60/75; and
* the steel grade does not exceed steel grade S460,

the degree of shear connection *η* which is equal to the ratio *η* = *n*sc/*n*f , should fullfil the condition given in:

|  |  |  |
| --- | --- | --- |
|  |  | (8.12) |

where:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  | provided degree of shear connection; | |
|  | *n*f | number of connectors for full shear connection () for the length *L*e in accordance with (3); | |
|  | *P*Rd | design resistance of a shear connector | |
|  | *n*sc | number of shear connectors provided within that same length; | |
|  |  | in accordance with Formula (8.13) to Formula (8.16) | |
|  |  | ; but ; | |
|  | *M*Ed | design bending moment; | |
|  |  | design moment resistance considering applied degree of shear connection; | |
|  | *k*up | factor considering the method of construction, where: | |
|  |  |  | when the steel section is propped during construction; |
|  |  |  | when the steel section is unpropped during construction; |
|  |  | when | |
|  |  | | |
|  | *M*a,Ed | design bending moment, applied to the structural steel section before composite action is achieved, due to self-weight; | |
|  | *M*pl,Rd | bending moment resistance for full shear connection; | |
|  |  | for shear connectors of Ductility Category D2   for shear connectors of Ductility Category D3 | |

NOTE Ductility Categories are defined in Table 5.1.

(3) The value of *η*0 to be used in Formula (8.12) should be calculated in accordance with the following:

For steel sections with equal flanges:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  |  | (8.13) |
|  |  |  | (8.14) |

For steel sections having a bottom flange with an area equal to three times the area of the top flange:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  |  | (8.15) |
|  |  |  | (8.16) |

where:

|  |  |  |
| --- | --- | --- |
|  | *L*e | is the distance in sagging bending between points of zero bending moment in metres; for continuous beams, *L*e may be assumed to be as shown in Figure 7.1; |

(4) 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 ratio for use in Formula (8.12) should be determined using Formula (8.13) to Formula (8.16) by linear interpolation.

(5) Where the shear connectors are Ductility Category D3,the steel section has equal flanges and the beam length *L*e is less than 20 m, Formula (8.17) may be used as an alternative to Formula (8.12) to give the required degree of shear connection.

|  |  |  |
| --- | --- | --- |
|  |  | (8.17) |

(6) When the distance between the centroid of the steel beam and the centroid of the effective concrete slab is greater than 600 mm, the minimum degree of shear connection given by Formula (8.13) or (8.17) should be increased by 0,2, but not exceed 1,0, unless the plastic resistance moment is more than 1,6 times the plastic resistance moment of the steel member alone.

### Other beams where plastic theory is used for the resistance of the cross-sections

(1) These requirements apply to beams of cross-section Class 1 and Class 2, where one of the following criteria is fulfilled:

1. shear connectors of Ductility Category D1 fulfilling the requirements of 5.4.2.1(5) are applied; or
2. where connectors of Ductility Category D2 or Category D3 are used and 8.6.3 is not used.

(2) For parts of the beams where the applied moment does not exceed the elastic design resistance, 8.6.5 should be used.

(3) Where the concrete slab is in compression (see Figure 8.14) and where the total design bending moment *M*Ed,max= *M*a,Ed + *M*i,Ed exceeds the elastic bending resistance *M*el,Rd, account should be taken of the non-linear relationship between transverse shear and longitudinal shear within the inelastic lengths of the member.

NOTE *M*a,Ed and *M*i,Ed are defined in 8.2.1.4(6).

Diagram

Description automatically generated

Figure 8.14 — Determination of longitudinal shear in beams with   
inelastic behaviour of cross-sections

(4) Shear connectors should be provided within the inelastic length *L*A-B to resist the longitudinal shear force *v*L,Ed, resulting from the difference between the axial forces *N*cd and *N*c,el in the concrete slab at the cross-sections B and A, respectively.

If the maximum bending moment *M*Ed,max at cross-section B is smaller than the plastic bending resistance *M*pl,Rd, the axial force *N*cd at cross-section B should be determined in accordance with 8.2.1.6(1) and Figure 8.8, or alternatively using the simplified linear relationship in accordance with Figure 8.14.

NOTE The bending resistance *M*el,Rd is defined in 8.2.1.4.

(5) For regions with the slab in tension, where the effects of inelastic behaviour of a cross-section are taken into account, the longitudinal shear forces and their distribution should be determined from the differences of forces in the concrete slab within the inelastic length of the beam. The effects from tension stiffening of concrete between cracks and possible overstrength of concrete in tension should also be taken into account.

(6) As an alternative to (5), the longitudinal shear forces may be determined by elastic analysis in accordance with 8.6.5.

(7) For the stresses in pre-stressing steel, additional guidance should be sought from EN 1994-2.

### Beams in which elastic theory is used for resistances of cross-sections

(1) The number and spacing of shear connectors should provide the required longitudinal shear resistance per unit length at the interface between steel and concrete in a composite member.

(2) For any load combination and arrangement of design actions, *v*L,Ed should be determined from the rate of change of the longitudinal force in either the steel or the concrete element of the composite section.

NOTE When elastic theory is used for beams of uniform cross-section, the value of *v*L,Ed at a cross-section is directly proportional to the value of vertical shear.

(3) For composite beams with the concrete flange above the steel section, the properties of the uncracked section should be used for the determination of the longitudinal shear force, even where cracking of concrete is assumed in the global analysis.

(4) For connectors of Ductility Category D1, the longitudinal shear resistance per unit length provided should be not less than *v*L,Ed at all cross-sections.

(5) For ultimate limit states other than fatigue, the size and spacing of shear connectors of Ductility Category D2 and Category D3 may be kept constant over any length where *v*L,Ed does not exceed the longitudinal shear resistance by more than 15%. Over such a length the total design longitudinal shear resistance should exceed the average value of *v*L,Ed times the length.

(6) Notwithstanding (3), the effects of cracking of concrete on the longitudinal shear force may be taken into account, if in the global analysis and for the determination of the longitudinal shear force account is taken of the effects of tension stiffening and possible overstrength of concrete.

(7) For partially-encased and shallow floor beams, more advanced calculations are required, taking into consideration concrete cracking, concrete overstrength and tension stiffening.

### Beams in which non-linear theory is used for resistances of cross-sections

(1) Where non‑linear theory is used to determine resistance, the longitudinal shear force should be determined in accordance with 8.2.1.5.

### Local effects of concentrated longitudinal shear force

#### Local effects due to introduction of longitudinal forces

(1) Where a force *F*Ed parallel to the longitudinal axis of the composite beam is applied to the concrete or steel element by a bonded or unbonded tendon, the distribution of the concentrated longitudinal shear force *V*L,Ed along the interface between steel and concrete, should be determined in accordance with (2) or (3). The distribution of *V*L,Ed caused by several forces *F*Ed should be obtained by summation.

(2) The force *V*L,Ed may be assumed to be distributed along a length *L*v of shear connection with a maximum shear force per unit length as given in Formula (8.18) and Figure 8.15a) for load introduction within a length of a concrete flange and by Formula (8.19) and Figure 8.15b) at an end of a concrete flange.

|  |  |
| --- | --- |
|  | (8.18) |
|  | (8.19) |

where

|  |  |  |
| --- | --- | --- |
|  | *L*v | when shear connectors of Ductility Category D2 and Category D3 are applied, *L*v is the effective width *b*eff for global analysis, given by 7.4.1.2. When shear connectors of Ductility Category D1 are applied, *L*v is the larger of the values of the effective width of the concrete flange on each side of the web *b*ei given by 7.4.1.2, |
|  | *e*d,F | is either 2*e*h,F or 2*e*v,F (the length over which the force *F*Ed is applied may be added to *e*d,F); |
|  | *e*h,F | is the lateral distance from the point of application of force *F*Ed to the relevant steel web, if the force is applied to the slab; |
|  | *e*v,F | is the vertical distance from the point of application of force *F*Ed to the plane of the shear connection concerned, if the force is applied to the steel element. |

(3) Where stud shear connectors are used, at ultimate limit states, a rectangular distribution of shear force per unit length may be assumed within the length *L*v, so that within a length of concrete flange,

|  |  |
| --- | --- |
|  | (8.20) |

and at an end of a flange,

|  |  |
| --- | --- |
|  | (8.21) |

(4) In the absence of a more precise determination, the forces *F*Ed - *V*L,Ed may be assumed to disperse into the concrete or steel element at an angle of , where = arctan 2/3.

|  |  |
| --- | --- |
| A diagram of a cone  Description automatically generated | A diagram of a beam  Description automatically generated |
| a) | b) |
| A diagram of a triangle with arrows and letters  Description automatically generated with medium confidence | A diagram of a triangle  Description automatically generated |
| c) | d) |

Figure 8.15 — Distribution of longitudinal shear force along the interface

#### Local effects at sudden change of cross-sections

(1) Concentrated longitudinal shear at the end of the concrete slab, e.g. due to the primary effects of shrinkage and thermal actions in accordance with EN 1991-1-5, should be taken into account [see Figure 8.15c)]. This also applies for intermediate stages of construction of a concrete slab [see Figure 8.15d)].

(2) Concentrated longitudinal shear at a sudden change of cross-sections, e.g. change from steel to composite section in accordance with Figure 8.15d), should be taken into account.

(3) Where the primary effects of shrinkage and thermal actions cause a design longitudinal shear force *V*L,Ed, to be transferred across the interface between steel and concrete at each free end of the member, its distribution may be assumed to be triangular, with a maximum shear force per unit length [see Figure 8.15c) and d)].

|  |  |
| --- | --- |
|  | (8.22) |

Where

|  |  |  |
| --- | --- | --- |
|  | *L*v | is taken as the effective width *b*eff for global analysis, given in 7.4.1.2, when shear connectors of Ductility Category D2 and Category D3 are applied. For connectors of Ductility Category D1, it is the larger of the values of the effective width of the concrete flange on each side of the web, *b*ei, given in 7.4.1.2. |

NOTE Where stud shear connectors are used, for the ultimate limit state the distribution can alternatively be assumed to be rectangular along a length *L*v adjacent to the free end of the slab.

(4) For calculating the primary effects of shrinkage at intermediate stages of the construction of a concrete slab, the equivalent span for the determination of the width *b*eff in 8.6.7.2 should be taken as the continuous length of concrete slab where the shear connection is effective, within the span considered.

(5) Where at a sudden change of cross-section in accordance with Figure 8.15d) the concentrated longitudinal shear force results from the force *N*c due to bending, the distribution given in (3) may be used.

(6) The forces transferred by shear connectors should be assumed to disperse into the concrete slab at an angle of , where =arctan 2/3.

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

#### Design resistance for shear

(1) The design shear resistance of a headed stud with an overall height of a stud not less than 3,9 d should be determined from the smallest of the values obtained from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.23) |

and:

|  |  |  |
| --- | --- | --- |
|  |  | (8.24) |

where

|  |  |  |
| --- | --- | --- |
|  | *γ*V | is the partial factor in accordance with 4.4.1.2(5); |
|  | *d* | is the diameter of the shank of the stud, 16 mm ≤ d ≤ 25 mm; |
|  | *f*u | is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm2; |
|  | *f*ck | is the characteristic cylinder compressive strength of the concrete at the age considered, of density not less than 1750 kg/m3; |
|  | *E*cm | is the secant modulus of elasticity of the concrete for short-term loading in accordance with 5.1(8). The coefficient *k*E in accordance with EN 1992-1-1 should not be taken as more than 9500; |
|  | *k*cc | is the reduction factor considering the effect of concrete relaxation under sustained loading. |

NOTE The factor *k*cc is 1,0 unless the National Annex gives a different value for use within specific countries.

(2) The studs should be automatically welded in accordance with EN ISO 14555 and the weld collars should comply with the recommendations of EN ISO 13918.

(3) In buildings, where headed stud connectors are arranged in such a way that splitting forces can occur in the direction of the slab thickness (see Figure 8.16), and where there is no transverse shear, the design resistance may be determined in accordance with (1) when the following conditions are satisfied:

a) fatigue is not relevant;

b) transverse reinforcement is provided, as shown in Figure 8.16, such that *a*rp ≥ 6 *d*, and the anchorage length *v* is greater than or equal to 14 *d*; and

c) the splitting force is resisted by stirrups which should be designed for a tensile force of 0,3 *P*Rd per stud connector. The spacing of these stirrups should not exceed the smaller of 18 *d* and the longitudinal spacing of the connectors.

(4) Where the conditions in (3) are not satisfied, or for application in bridges, use Annex F.

A diagram of a rectangular object with a hole in the middle

Description automatically generated

Figure 8.16 — Local reinforcement for splitting forces

#### Effect 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,Ed should be calculated.

(2) If *F*ten,Ed ≤ 0,1*P*Rd, where *P*Rd is the design shear resistance defined in 8.6.8.1(1), the tensile force may be neglected.

(3) If *F*ten,Ed > 0,1*P*Rd, the tensile resistance and the interaction between shear and tension should be checked as follows:

|  |  |
| --- | --- |
|  | (8.25) |
|  | (8.26) |

where

|  |  |  |
| --- | --- | --- |
|  | *P*Rd | is the design shear resistance defined in 8.6.8.1 for solid slabs and in 8.6.9 for headed studs used with profiled steel sheeting in buildings; |
|  | *F*s,Ed  *F*ten,,Ed | is the design longitudinal shear force per stud;  is the design tensile force per stud; |
|  | *P*ten,Rd | is the design tension resistance of a headed stud, in accordance with (4) or (5). |

NOTE Studs covered by Annex G are outside the scope of this subclause.

(4) The tensile resistance of headed studs, *P*ten,Rd, should be calculated in accordance with EN 1992-4:2018, 7.2. Only checks for steel failure, pull-out failure and concrete cone failure apply.

(5) For the most commonly used headed studs, use Annex H to obtain *P*ten,Rd.

#### Biaxial loading

(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 be in accordance with:

|  |  |  |
| --- | --- | --- |
|  |  | (8.27) |

where

|  |  |  |
| --- | --- | --- |
|  | *F*,Ed | is the design longitudinal force caused by composite action in the beam; |
|  | *F*t,Ed | is the design transverse force caused by composite action in the slab (see Clause 10); and |
|  | *P*,Rd and *P*t,Rd | are the corresponding design shear resistances of the stud. |

### Design resistance of headed studs used with profiled steel sheeting in buildings

#### Sheeting with ribs parallel to the supporting beams

(1) Where studs are located within a region of concrete that has the shape of a haunch, see Figure 8.17, and 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 10.2. Where the sheeting is not continuous, *b*0 is defined in a similar way as given in Figure 8.17. The depth of the haunch should be taken as *h*p.

A picture containing diagram, line, technical drawing

Description automatically generated

Figure 8.17 — Beam with profiled steel sheeting parallel to the beam

(2) The design shear resistance should be taken as the resistance in a solid slab, multiplied by the reduction factor *k* given in Formula (8.28). For concrete grades higher than C60/70, the resistance in a solid slab in accordance with 8.6.8.1 should be calculated with *f*ck = 60 N/mm2.

|  |  |  |
| --- | --- | --- |
|  |  | (8.28) |

where

|  |  |  |
| --- | --- | --- |
|  | *h*sc | is the length after welding of the stud in accordance with EN ISO 13918, but not greater than *h*p + 75 mm. For calculations, the length after welding in accordance with Table 10 of EN ISO 13918:2018 may be assumed to apply for through-deck welded studs; |
|  | *h*p | is the overall height of the profiled steel sheeting, excluding the height of the top re-entrant stiffener in open trough sheeting (see Figure 8.21), provided that the geometry is in accordance with 8.6.10.8(3). |

NOTE For through-deck welded studs, the shorter length after welding is accounted for in the design resistance given by Formulae (8.28) and (8.29).

(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 8.6.10.4.

NOTE Means to achieve appropriate anchorage can be given in the National Annex.

(4) The effect of tension on the resistance of headed studs should be taken into account in accordance with 8.6.8.2 with the stud resistance *P*Rd in 8.6.8.2 taken as  *P*Rd with as given by (2) and *P*Rd as given by 8.6.8.1.

(5) Biaxial loading of shear connectors should be taken into account in accordance with 8.6.8.3 with the stud resistance in 8.6.8.2 taken as  *P*Rd with as given by (2) and *P*Rd as given in 8.6.8.1.

#### Sheeting with ribs transverse to the supporting beams

(1) Provided the conditions (2) and (3) are satisfied, the design shear resistance should be taken as the resistance in a solid slab, calculated as given in 8.6.8.1, (except that *f*u should not be taken as greater than 450 N/mm2, and *f*ck should not be taken as greater than 60 N/mm2), multiplied by the reduction factor *k*t determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.29) |

where

|  |  |  |
| --- | --- | --- |
|  | *n*r | is the number of stud connectors in one rib at the beam intersection, not to exceed 2. Other symbols are as defined in Figure 8.18. |

A picture containing diagram, technical drawing, sketch, plan

Description automatically generated

Figure 8.18 — Beam with profiled steel sheeting transverse to the beam

(2) The factor *k*t should not be taken as greater than the appropriate value *k*t,max given in Table 8.3.

(3) The values for *k*t given in (1) and (2) are applicable provided that all the following conditions are satisfied:

* for open trough profiles, the studs are placed within the rib with an embedment depth *h*A not less than 2,7 d, and the distance from the edge of the concrete rib on the higher moment side to the centre-line of the nearest stud connector *e*k is greater than 60 mm (Figure 8.19);
* for re-entrant trough profiles, the studs are placed within the rib with an embedment depth *h*A that is in accordance with 8.6.10.8(1);
* the nominal thickness *t*p of the steel sheet is not less than 0,70 mm;
* the nominal concrete cover over the stud is in accordance with 8.6.10.2;
* a reinforcement layer is placed underneath the head of the stud;
* the height of the profiled sheeting *h*p does not exceed a value of 105 mm;
* the number of studs in one rib *n*r does not exceed a value of 2;
* for through-deck welding, the diameter of the studs is not greater than 20 mm;
* for holes provided in the sheeting, the diameter of the studs is not greater than 22 mm.

A diagram of a beam

Description automatically generated

Figure 8.19 — Beam with profiled steel sheeting transverse to the beam

**Table 8.3 — Upper limits *k*t,max for the reduction factor *k*t**

|  |  |  |  |
| --- | --- | --- | --- |
| Number of stud connectors per rib | Thickness *t*p of sheet (mm) | Studs not exceeding 20 mm in diameter and welded through profiled steel sheeting | Profiled sheeting with holes and studs 19 mm or 22 mm in diameter |
| *n*r = 1 | ≤ 1,0 | 0,85 | 0,75 |
| > 1,0 | 1,0 | 0,75 |
| *n*r = 2 | ≤ 1,0 | 0,70 | 0,60 |
| > 1,0 | 0,8 | 0,60 |

(4) The effect of tension on the resistance of headed studs should be taken into account in accordance with 8.6.8.2 with the stud resistance *P*Rd in 8.6.8.2 taken as *k*t *P*Rd with *k*t as given in (1) and *P*Rd as given in 8.6.8.1.

(5) Biaxial loading of shear connectors should be taken into account in accordance with 8.6.8.3 with the stud resistance *P*Rd in 8.6.8.2 taken as *k*t *P*Rd with *k*t as given in (1) and *P*Rd as given in 8.6.8.1.

(6) For open trough profiles, if the requirements of (3) are not fulfilled, the design resistance may be determined in accordance with Annex G. Alternatively, the values may be determined by tests in accordance with Annex B.

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

#### Resistance to separation

(1) The surface of a connector that resists separation forces (e.g. the underside of the head of a stud) should extend not less than 30 mm clear above the bottom reinforcement in solid slabs (see Figure 8.20).

#### Cover and concreting for buildings

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

(2) The nominal concrete cover should be:

1. not less than 15 mm; or
2. as recommended in EN 1992-1-1:2023, 6.4.2 and 6.4.3 with *c*min,b = 0 and *Δc*dev = 5 mm, whichever is greater.

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

#### 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 8.6.11 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:

1. transverse reinforcement should be supplied by U‑bars passing around the shear connectors;
2. 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, with the U‑bars not less than 0,5 *d* in diameter; and
3. the U‑bars should be placed as low as possible while still providing sufficient bottom cover.

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

#### 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 8.20).

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Figure 8.20 — Detailing

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

(3) Transverse reinforcing bars sufficient to satisfy the requirements of 8.6.11 should be provided in the haunch at not less than 40 mm clear below the surface of the connector that resists uplift.

#### Spacing of connectors

(1) 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 Class 3 or Class 4 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 not be greater than the following limits:

* where the slab is in contact over the full length (e.g. solid slab): ; and
* where the slab is not in contact over the full length (e.g. slab with ribs transverse to the beam):

where:

|  |  |  |
| --- | --- | --- |
|  | *t*f | is the thickness of the flange. |

In addition, the clear distance from the edge of a compression flange to the nearest line of shear connectors should be not greater than .

(3) In buildings, the maximum longitudinal centre‑to‑centre spacing of shear connectors should be not greater than six times the total slab thickness or 800 mm.

#### 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 8.20), should not be less than 20 mm.

#### Headed stud connectors

(1) The length after welding of the stud should be in accordance with EN ISO 13918.

(2) The head should have a diameter of not less than 1,5 *d* and a depth of not less than 0,4 *d*.

(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 5 *d*; the spacing in the direction transverse to the shear force should be not less than 2,5 *d* in solid slabs and 4 *d* 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.

#### Headed studs used with profiled steel sheeting in buildings

(1) The minimum embedment depth given by *h*A = (*h*sc – *h*p) should not be less than 2 *d*, where *h*sc is as defined in 8.6.9.1(2), *h*p is the overall height of the profiled steel sheeting [excluding the height of the top re-entrant stiffener in open trough sheeting when the conditions in (3) are satisfied] and *d* is the diameter of the shank.

(2) The minimum width of the troughs that are to be filled with concrete should not be less than 50 mm.

(3) For open trough sheeting with a top re-entrant stiffener, the height of the stiffener *d*ef should not be greater than 15 mm, and the horizontal distance between the web-to-flange junction and the corner of the re-entrant stiffener *b*fp should not be less than 25 mm (see Figure 8.21).

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Figure 8.21 — Open trough sheeting with a top re-entrant stiffener

### Longitudinal shear in concrete slabs

#### General

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

(2) The design longitudinal shear stress for any potential surface of longitudinal shear failure within the slab *τ*Ed shall not exceed the design longitudinal shear strength of the shear surface under consideration.

(3) The length of the shear surface b‑b shown in Figure 8.22 should be equal to 2 *h*sc plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to (2*h*sc + *s*y) plus the head diameter for stud shear connectors arranged in pairs, where *h*sc is the length after welding of the studs and *s*y 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 8.6.2 to 8.6.7 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) For each type of shear surface considered, the design longitudinal shear stress *τ*Ed should be determined from the design longitudinal shear per unit length of beam, taking account of the number of shear planes and the length of the shear surface.

#### Design resistance to longitudinal shear

(1) The design shear strength of the concrete flange (shear planes a‑a illustrated in Figure 8.22) should be determined in accordance with EN 1992-1-1:2023, 8.2.5(6).

(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 using EN 1992-1-1:2023, 8.2.5(4). For a shear surface passing around the shear connectors (e.g. shear surface b‑b in Figure 8.22), 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 1992-1-1:2023, 8.2.5 should be as shown in Figure 8.22, in which *A*b, *A*t and *A*bh are areas of reinforcement per unit length of beam anchored in accordance with EN 1992-1-1:2023, 11.4 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 1992-1-1:2023, 8.2.6.

|  |  |
| --- | --- |
| A diagram of a beam  Description automatically generated | A diagram of a beam  Description automatically generated |
| a) | b) |
| A diagram of a structure  Description automatically generated | |  |  | | --- | --- | | type | *A*sf/sf | | a-a | *A*b + *A*t | | b-b | *2 A*b | | c-c | *2 A*b | | d-d | *2 A*bh | |
| c) |  |

Figure 8.22 — Typical potential surfaces of shear failure

#### Minimum transverse reinforcement

(1) The minimum area of reinforcement should be determined in accordance with EN 1992-1-1:2023, 12.2(4), using the definitions appropriate to transverse reinforcement.

#### 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 8.23), 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 8.6.9.2, it is not necessary to consider shear surfaces of type b‑b in Figure 8.23.

(3) For surfaces of type c‑c in Figure 8.23, the depth of the sheeting should not be included in *h*f unless verified by tests.

(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, see Formula (8.30).

|  |  |  |
| --- | --- | --- |
|  |  | (8.30) |

where:

|  |  |  |
| --- | --- | --- |
|  | *A*pe | is the effective cross‑sectional area of the profiled steel sheeting per unit length of the beam [see 10.7.2(5)]; for sheeting with holes, the net area should be used; |
|  | *f*yp,d | is its design yield strength; |
|  | τEd | is the longitudinal shear stress, at the junction between one side of a flange in accordance with EN 1992-1-1:2023, 8.2.5, with *h*fl replaced by *h*c; |
|  | cot*θ*f | 1 ≤ cot *θ*f ≤ 3,0 in compression flanges; 1 ≤ cot *θ*f ≤ 1,25 in tension flanges. |

NOTE As a simplification cot *θ*f =1,2 can be used.

|  |  |
| --- | --- |
| A diagram of a beam  Description automatically generated | A diagram of a beam  Description automatically generated |
| a) | b) |
| A diagram of a metal structure  Description automatically generated | |  |  | | --- | --- | | type | *A*sf/sf | | a-a | *A*t | | b-b | *2 A*b | | c-c | *2 A*b | | d-d | *2 A*b *+ A*t | |
| c) |  |

Figure 8.23 — Typical potential surfaces of shear failure where profiled steel sheeting is 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*pe *f*yp,d in Formula (8.30) should be replaced by that given in Formula (8.31).

|  |  |  |
| --- | --- | --- |
|  |  | (8.31) |

where:

|  |  |  |
| --- | --- | --- |
|  | *P*pb,Rd | is the design bearing resistance of a headed stud welded through the sheet in accordance with 10.7.4; |
|  | *s*x | is the longitudinal spacing centre‑to‑centre of the studs effective in anchoring the sheeting. |

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

## Fatigue

### Fatigue for buildings

(1) No fatigue assessment for structural steel, reinforcement, concrete and shear connection is required where, for structural steel, EN 1993-1-1:2022, 10(2) or 10(3) applies and, for concrete, where conditions in EN 1992-1-1:2023, 10.1(2) are fulfilled.

(2) Where (1) is not fulfilled, the requirements in prEN 1994-2:2024, 8.7 should be used.

## Composite columns and composite compression members

### General

(1) Clause 8.8 applies for the design of composite columns and composite compression members with concrete encased sections, partially-encased sections and concrete filled rectangular and circular hollow section (see Figure 8.24).

(2) Composite compression members may be verified by the following methods:

* a general method given in 8.8.2 whose scope includes members with non-symmetrical or non-uniform cross-sections over the column length; and
* a simplified method given in 8.8.3 for members of doubly symmetrical and uniform cross-section over the member length.

1. The simplified method in accordance with 8.8.3 applies to columns and compression members with steel grades S235 to S460, reinforcement grade not above B500, normal weight concrete of strength Classes C20/25 to C50/60 and those cross-sections shown in Figure 8.24.

NOTE The simplified method applies to cross-sections given in Figure 8.24. Other sections such as a concrete encased steel section with a concrete cross-section that is not rectangular or square, composite columns with elliptical, triangular or other shapes different to the sections presented in Figure 8.24 and all composite columns with single or multiple massive steel cores or a steel core formed from plates are not covered by the simplified method. Those columns have to be verified using the general method, see 8.8.2 or a specific CEN/TS.

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| A diagram of a square with a letter in it  Description automatically generated | | A diagram of a square with a square and a square with a square and a square with a square with a square and a square with a square with a square and a square with a square with  Description automatically generated | | A hexagon with a number of lines and symbols  Description automatically generated |
| a) | | b) | | c) |
| A rectangular object with black lines and black dots  Description automatically generated | A diagram of a clock  Description automatically generated | |
| d) | e) | |
|  | A black and white circle with a letter in the middle  Description automatically generated | |
| f) | g) | |

Figure 8.24 — Typical cross‑sections of composite columns and notation

(4) This clause applies to isolated columns as well as columns and composite compression members in framed structures where the other structural members are either composite or steel members.

(5) The steel contribution ratio *δ*c should fulfil the following condition given in Formula (8.32).

|  |  |  |
| --- | --- | --- |
|  |  | (8.32) |

where:

|  |  |  |
| --- | --- | --- |
|  | *δ*c | is defined in 8.8.3.3(1). |

1. Composite columns or compression members of any cross‑section should be checked for:

* resistance of the member in accordance with 8.8.2 or 8.8.3;
* resistance to local buckling in accordance with (8) and (9);
* introduction of loads in accordance with 8.8.4.2; and
* resistance to shear between steel and concrete elements in accordance with 8.8.4.3.

(7) For composite compression members subjected to bending moments and axial forces resulting from combinations of independent actions, those internal forces that, when increased, 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 8.8.5.1(2), and for other types of cross‑section provided the maximum values of Table 8.4 are not exceeded.

(10) Compression members with an axial force *N*Ed should be designed for a minimum moment of *M*Ed,min= ± *N*Ed⋅*e*d,min. unless one of the following conditions applies:

• equivalent imperfections have been used for the global analysis; or

• the resistance of the member is calculated in accordance with 8.8.3.5; or

• specific structural measures (e.g. yield plate or centre plate) are used to guarantee concentric loading.

The eccentricity *e*d,min is an additional eccentricity which covers the effects of geometric imperfections (e.g. due to an offset of superposed columns or bending of the supported slab). For composite columns in buildings with braced frames, where the restraining effects from slabs or beams on the buckling length are not taken into account and where the buckling length is assumed to be the geometric length between the centres of the upper and lower restraints, the additional minimum eccentricity may be neglected. Otherwise for columns supporting a concrete structure the eccentricity can be taken by reference to EN 1992-1-1:2023, 8.1.1(5). Rules for determination of buckling length then are given in EN 1992-1-1:2023, Annex O.

**Table 8.4 — Maximum values (*d*/*t*), (*h*/*t*) and (*b*/*t*f)**

|  |  |  |
| --- | --- | --- |
| Cross-section |  | Max (*d*/*t*), max (*h*/*t*) and max (*b*/*t*f) |
| Circular hollow steel sections | A diagram of a circular object with numbers and symbols  Description automatically generated |  |
| Rectangular hollow steel sections | A diagram of a rectangular object with black dots and lines  Description automatically generated |  |
| Partially-encased I‑sections | A black and white drawing of a square with a square in the middle  Description automatically generated |  |
| ; where *f*y is in N/mm2 | | |

### General method of design

(1) The general method is based on a non-linear analysis. Design for structural stability shall take account of non-linear 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 non-linear model shall cover all relevant failure modes. The verifications 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) Non-linear 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 geometrical and material non-linear analysis with imperfections (GMNIA).

(4) Plane sections may be assumed to remain plane. Full composite action up to failure may be assumed between the steel and concrete components of the member, provided the longitudinal shear resistance in the length of load introduction and also over the remaining member length is not exceeded.

(5) Concrete in tension shall be neglected. The influence of tension stiffening of concrete between cracks on the flexural stiffness may be taken into account.

(6) Shrinkage and creep effects shall be taken into account if they are likely to reduce the structural stability significantly.

(7) For simplification, creep and shrinkage effects may be ignored if the increase in the bending moments due to creep deformations and normal force resulting from permanent and quasi permanent loads is not greater than 10%.

(8) The general method is based on non-linear analysis and design by applying an overall safety factor for resistance and for action effects. For composite columns the inequality given in Formula (8.33) should be satisfied:

|  |  |  |
| --- | --- | --- |
|  |  | (8.33) |

where

|  |  |  |
| --- | --- | --- |
|  | *R*m | is the mean value of resistance, obtained from a non-linear analysis, as defined in (9); |
|  |  | is the vector defined by the internal force *N*Ed and the associated moment (Figure 8.25). |
|  |  | is the internal design moment determined by the geometrical and material non-linear analysis taking into account geometrical and structural imperfections based on the combination of action effects in accordance with EN 1990 and EN 1991 (all parts) taking into account the partial safety factors γG and γQ and considering 8.8.1(7). |
|  | *N*Ed | Is the Internal force determined by the geometrical and material non-linear analysis taking into account geometrical and structural imperfections based on the combination of action effects in accordance with EN 1990 and EN 1991 (all parts) taking into account the partial safety factors γG and γQ and considering 8.8.1(7) if required; for the verification of the compression member as isolated member, the design value *N*Ed is that from the global analysis. |
|  | *γ*0 | is the overall partial factor, in accordance with (10); |
|  | *γ*G | is the partial factor for permanent actions; |
|  | *γ*Q | is the partial factor for non-permanent actions. |

NOTE  Where the variation of the elasticity modulus can lead to a significant impact on the second-order analysis sensitivity study is to be taken into account.

(9) The nominal mean values of material strength for concrete, reinforcement and structural steel in accordance with 8.8.2(11) should be used in the analysis. Additional effects due to geometrical imperfection and residual stresses should be included in the analysis.

NOTE 1 The mean value *R*m of the resistance results from the geometrical and material non-linear analysis taking into account the geometrical and structural imperfections considering all effects in accordance with 8.8.2(1).

NOTE 2 Values for residual stress can be taken in accordance with EN 1993-1-14.

NOTE 3 The geometrical imperfection depends on the column type and execution process. It is *L*/400 unless the National Annex gives a different value, where L is the length of the compression member.

NOTE 4 For simplification, instead of the effects of residual stresses (structural imperfection) and geometrical imperfections, equivalent initial bow imperfections (member imperfections) can be used in accordance with Table 8.7.

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Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | Graph of applied load against displacement |
| 2 | End eccentricity and imperfection |
| 3 | Moments due to applied load, eccentricity and imperfection |

Figure 8.25 — Non-linear analysis for determination of *R*m and

(10) The overall partial factor γ0 is given in Formula (8.34) and Figure 8.26. The values of *R*pl,m and *R*pl,d, shown in this figure should be determined along the direction of the load vector, defined by *N*Ed and which should be associated to the investigated load combination.

|  |  |  |
| --- | --- | --- |
|  |  | (8.34) |

where:

|  |  |  |
| --- | --- | --- |
|  | *R*pl,m | is the resistance in combined bending and compression which may be based on the full plastic interaction Curve 1 or interaction Curve 1 determined based on nonlinear resistance, in Figure 8.26, using the material strengths *f*cm, *f*ym and *f*sm in accordance with (11), |
|  | *R*pl,d | is the resistance in combined bending and compression which may be obtained from 8.8.3.2(2) (Curve 2 in Figure 8.26). |
|  |  | is the design value of the maximum internal moment resulting from (1) to (9); |
|  | *N*pl,m | is the resistance of the cross-section to compression using the material strengths *f*cm, *f*ym and *f*sm which may be based on the full plastic or nonlinear resistance. |
|  | *N*pl,d | is the design resistance of the cross-section to compression which may be based on the full plastic or nonlinear resistance. |
|  | *M*pl,m | is the resistance of the cross-section to bending using the material strengths *f*cm, *f*ym and *f*sm which may be based on the full plastic or nonlinear resistance. |
|  | *M*pl,rd | is the design resistance of the cross-section to bending which may be based on the full plastic or nonlinear resistance. |

NOTE All values for *R*pl,m, *R*pl,d, *N*pl,m, *N*pl,d, *M*pl,m and *M*pl,Rd are to be determined either on plastic resistance or alternative all are to be determined based on nonlinear resistance.

**A diagram of a function

Description automatically generated**

Key

|  |  |
| --- | --- |
| 1 | N-M curve based on mean material properties |
| 2 | N-M curve based on design material properties |

Figure 8.26 — Interaction curve for determination of overall safety factor γ0

(11) The following stress-strain relationships should be used for the non-linear analysis and the determination of *R*m, in accordance with Figure 8.25. The mean values should also be used for the determination of *R*pl,m, in accordance with Figure 8.26:

* for concrete in compression as given in EN 1992-1-1:2023, 5.1.6(3), where *f*cm is given in EN 1992-1-1:2023, Table 5.1 and Table A.1;
* for reinforcing steel as given in EN 1992-1-1:2023, Figure 5.2, Curve 2. In this diagram, *f*yk should be replaced by *f*sm and the value *k* *f*yk should be replaced by *k* *f*ym, where *f*sm is given in EN 1992-1-1:2023, Table A.1;
* for structural steel as given in EN 1993-1-1:2022, 7.4.3(3), where *f*y should be replaced by *f*ym. Alternatively the stress-strain relation in accordance with prEN 1993-1-14:2023, 5.3.2 may be used.

NOTE The values of *f*ym can be determined from EN 1993-1-1:2022, Table E.1.

(12) For concrete filled tubes with circular cross-sections, confinement effects may be taken into account in accordance with 8.8.3.2(6).

(13) For composite columns under biaxial bending, the non-linear analysis should be provided by taking into account the load eccentricity, moments and imperfections in all direction. If no more detailed analysis based on biaxial N-My-Mz interaction curve is provided, the overall partial factor *γ*0 may be developed for each bending axis separately using the resulting moment vector (My2 +Mz2)0,5. The more conservative overall partial factor should be applied to determine the design resistance in accordance with (10).

### Simplified method of design

#### General and scope

(a) The scope of this simplified method is limited to members of doubly symmetrical and uniform cross‑section over the member length with hot-rolled, cold‑formed or welded steel sections. The simplified method is not applicable if the structural steel component consists of two or more unconnected sections. The relative slenderness defined in 8.8.3.3 should fulfil the condition in Formula (8.35).

|  |  |  |
| --- | --- | --- |
|  |  | (8.35) |

(b) For a fully-encased steel section, see Figure 8.24 a), limits to the maximum thickness of concrete cover that may be used in calculation are given in Formula (8.36):

|  |  |  |
| --- | --- | --- |
|  | max *c*z = 0,3*h* max *c*y = 0,4*b* | (8.36) |

(3) The longitudinal reinforcement that may be used in calculation should not exceed 6% of the concrete area.

(4) The ratio of the depth to the width of the composite cross‑section should be within the limits 0,2 and 5,0.

#### Resistance of cross-sections

(1) The plastic resistance to compression *N*pl,Rd of a composite cross-section should be calculated by adding the plastic resistances of its components as given in Formula (8.37).

|  |  |  |
| --- | --- | --- |
|  |  | (8.37) |

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Description automatically generated

Figure 8.27 — Interaction curve for combined compression and uniaxial bending

(2) The resistance of a cross‑section to combined compression and bending and the corresponding interaction curve may be calculated assuming rectangular stress blocks as shown in Figure 8.27, taking account of the design shear force *V*Ed in accordance with (3). The tensile strength of the concrete should be neglected.

(3) The influence of transverse shear forces on the resistance to bending and normal force should be considered when determining the interaction curve, if the shear force *V*a,Ed on the steel section exceeds *η*V *V*pl,a,Rd, see 8.2.2.5. The resistance to shear *V*c,Ed of the reinforced concrete part should be verified in accordance with EN 1992-1-1:2023, 8.2.

(4) Unless a more accurate analysis is used, *V*Ed may be distributed into *V*a,Ed acting on the structural steel and *V*c,Ed acting on the reinforced concrete section as given by Formulae (8.38) and (8.39).

|  |  |  |
| --- | --- | --- |
|  |  | (8.38) |
|  |  | (8.39) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*pl,a,Rd | is the plastic resistance moment of the steel section; and |
|  | *M*pl,Rd | is the plastic resistance moment of the composite section. |

For simplification *V*Ed may be assumed to act on the structural steel section alone.

(5) As a simplification, the interaction curve may be replaced by a polygonal diagram (the dashed line in Figure 8.28). Figure 8.28 shows as an example the plastic stress distribution of a fully-encased cross-section for the points A to D.

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Figure 8.28 — Simplified interaction curve and corresponding stress distributions

1. For concrete filled tubes of circular cross‑section, account may be taken of increase in strength of concrete caused by confinement provided that the relative slenderness  defined in 8.8.3.3(2) does not exceed 0,5 and *e*N/*d* < 0,1, where *e*N is the eccentricity of loading given by *M*Ed / *N*Ed and *d* is the external diameter of the column. The plastic resistance to compression may then be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.40) |

where:

|  |  |  |
| --- | --- | --- |
|  | *t* | is the wall thickness of the steel tube; |
|  |  | coefficients taking into accounts confinement effects according to Table 8.5. |

**Table 8.5 — Coefficients taking into accounts confinement effects**

|  |  |  |  |
| --- | --- | --- | --- |
| For members with centric loading | | | |
| *e*N/*d* = 0 |  |  | (8.41) |
|  |  | (8.42) |
| For members in combined compression and bending | | | |
| 0 < *e*N/*d* ≤ 0,1 |  |  | (8.43) |
|  |  | (8.44) |
| *e*N/*d* > 0,1 |  | *η*a = 1,0 | (8.45) |
|  | *η*c = 0 | (8.46) |

#### Effective flexural stiffness, steel contribution ratio and relative slenderness

1. The steel contribution ratio is defined by Formula (8.47).

|  |  |  |
| --- | --- | --- |
|  |  | (8.47) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*pl,Rd | is the plastic resistance to compression defined in 8.8.3.2(1). |

1. The relative slenderness for the plane of bending being considered is given in Formula (8.48).

|  |  |  |
| --- | --- | --- |
|  |  | (8.48) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*pl,Rk | is the characteristic value of the plastic resistance to compression given by Formula (8.37) if, instead of the design strengths, the characteristic values are used; |
|  | *N*cr | is the elastic critical normal force for the relevant buckling mode, calculated with the effective flexural stiffness (*EI*)eff determined in accordance with (3) and (4). |

1. For the determination of the relative slenderness  and the elastic critical force *N*cr, the characteristic value of the effective flexural stiffness (*EI*)eff of a cross-section of a composite column should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.49) |

where:

|  |  |  |
| --- | --- | --- |
|  | *K*e | is a correction factor that should be taken as 0,6. |
|  | *I*a, *I*c , *I*s | are the second moments of area of the structural steel section, the un‑cracked concrete section and the reinforcement for the bending plane being considered. |

(4) Influence of long‑term effects on the effective elastic flexural stiffness should be taken into account. The modulus of elasticity of concrete *E*cm should be reduced to the value *E*c,eff determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.50) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*Ed | is the total design normal force; |
|  | *N*G,Ed | is the part of this normal force that is permanent. |
|  |  | is the creep coefficient in accordance with 7.4.2.2(2). For concrete filled hollow sections, this coefficient may be considered as 25% of the value that would be obtained from EN 1992-1-1:2023, 5.1.5 and Annex B, for a reinforced concrete section. |

#### Methods of analysis and member imperfections

(1) For member verification, analysis should be based on second‑order linear elastic analysis.

(2) For the determination of the internal forces the design value of effective flexural stiffness  should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.51) |

where:

|  |  |  |
| --- | --- | --- |
|  | *K*e,II | is a correction factor which should be taken as 0,5; |
|  | *K*0 | is a calibration factor which should be taken as 0,9; |
|  | *E*c,eff | is the reduced modulus of elasticity of concrete to allow for long-term effects in accordance with 8.8.3.3(4). |

(3) Second‑order effects need not to be considered where 7.2.1(3) applies and the elastic critical load is determined with the flexural stiffness (*EI*)eff,II in accordance with (2).

(4) The influence of geometrical and structural imperfections may be taken into account by equivalent geometrical imperfections. Equivalent member imperfections for composite columns are given in Table 8.7, where *L* is the column length.

(5) Where in the global analysis only global second-order effects are accounted for, the design second-order moment within a column length may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.52) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*Ed,Em | is the maximum moment at the ends of the column from the global analysis including second-order effects where required by 7.2; |
|  | *M*Ed,Im | is the maximum first-order moment within the length of the column including the effect of imperfections and of any lateral loading within the column length not included in the global analysis; |
|  | *N*cr,eff | is the critical axial force for the relevant axis and corresponding to the effective flexural stiffness given in (2). The effective length may conservatively be assumed to be equal to the system length; |
|  |  | is the equivalent moment factor. Where there is no lateral load within the column length, its value is given in Table 8.6 where *r* is the ratio of the moment at the other end of the column to *M*Ed,Em and -1 ≤ *r* ≤1. Where there is lateral load within the column length a value of 1,0 should be used. |

**Table 8.6 — Factors *β*M for the determination of moments to second-order theory**

|  |  |  |  |
| --- | --- | --- | --- |
|  | Moment distribution | Moment factors *β*M | Comment |
| (a) | **A diagram of a triangle  Description automatically generated with medium confidence** | First‑order bending moments from member imperfection or lateral load: | *M*Ed,Im is the maximum bending moment within the column length ignoring second‑order effects |
| (b) | **A grey rectangular object with black text  Description automatically generated** | End moments:    And | *M*Ed,Em and *r* *M*Ed,Em are the end moments from first‑order or second‑order global analysis |

#### Resistance of members in axial compression

(1) Members may be verified using second-order analysis in accordance with 8.8.3.6 taking into account member imperfections.

(2) For simplification for members in axial compression, the design value of the axial force *N*Ed should satisfy Formula (8.53).

|  |  |  |
| --- | --- | --- |
|  |  | (8.53) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*pl,Rd | is the plastic resistance of the composite section in accordance with 8.8.3.2(1), but with *f*yd determined using the partial factor *γ*M1 given in EN 1993-1-1:2022, 8.1(1); |
|  | *χ* | is the reduction factor for the relevant buckling mode given in EN 1993-1-1:2022, 8.3.1.3 in terms of the relevant relative slenderness. |

The relevant buckling curves for cross‑sections of composite columns are given in Table 8.7, where *ρ*s is the reinforcement ratio *A*s / *A*c.

#### Resistance of members in combined compression and uniaxial bending

(1) Formula (8.54) based on the interaction curve determined in accordance with 8.8.3.2(2) to (5) should be satisfied:

|  |  |  |
| --- | --- | --- |
|  |  | (8.54) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*Ed | is the greatest of the end moments and the maximum bending moment within the column length, calculated in accordance with 8.8.3.4, including imperfections and second-order effects if necessary; |
|  | *M*pl,N,Rd | is the plastic bending resistance taking into account the axial force *N*Ed, given by *µ*d *M*pl,Rd (see Figure 8.27); |
|  | *M*pl,Rd | is the plastic bending resistance, given by point B in Figure 8.28. |

For steel grades between S235 and S355 inclusive, the coefficient *α*M should be taken as 0,9 and for steel grades S420 and S460 as 0,8.

(2) Where an increase in axial compression increases the bending resistance, values *µ*d greater than 1,0 should only be used where the bending moment *M*Ed depends directly on the action of the axial force *N*Ed, for example where the moment *M*Ed results from an eccentricity of the normal force *N*Ed. Otherwise an additional verification is necessary in accordance with Clause 8.8.1(7).

NOTE The value *µ*d = *µ*dy or *µ*dz, see Figure 8.29, refers to the design plastic resistance moment *M*pl,Rd for the plane of bending being considered.

A picture containing diagram, line, plot

Description automatically generated

**Figure 8.29 — Design for compression and biaxial bending**

**Table 8.7 — Buckling Curves and member imperfections for composite columns**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Cross‑section | Limits | Axis of buckling | Buckling Curve | Member imperfection |
| square and rectangular concrete encased section  A picture containing diagram, line, sketch, rectangle  Description automatically generated |  | *y‑y* | *b* | *L /200* |
| *z‑z* | *c* | *L /150* |
| partially concrete encased section  A picture containing sketch, line, diagram  Description automatically generated |  | *y‑y* | *b* | *L /200* |
|  | *z‑z* | *c* | *L /150* |
| circular and rectangular hollow steel section | *ρs ≤ 3%* | *any* | *a* | *L /300* |
| A diagram of a clock  Description automatically generated with low confidence |
| *3%< ρs ≤ 6%* | *any* | *b* | *L /200* |
| circular and rectangular hollow steel sections with additional I‑section |  | *y‑y* | *b* | *L /200* |
| A picture containing symbol, sketch, circle, diagram  Description automatically generated |  |  |  |
|  |  |  |
| *z‑z* | *b* | *L /200* |
|  |  |  |
| *partially concrete encased section with crossed I-sections* |  | *any* | *b* | *L /200* |
| A black and white diagram of a hexagon with arrows  Description automatically generated with low confidence |
| NOTE If the thickness of the open steel-section exceed a values of 40 mm, the buckling Curve is to be reduced by one class. | | | | |

#### Combined compression and biaxial bending

(1) For composite columns and compression members with biaxial bending, the values *µ*dy and *µ*dz in Figure 8.29 may be calculated in accordance with 8.8.3.6 separately for each axis. Imperfections should be considered only in the plane in which failure is expected to occur. If it is not evident which plane is the more critical, checks should be made for both planes.

(2) For combined compression and biaxial bending, the conditions given in Formulae (8.55) and (8.56) should be satisfied for the stability check within the column length and for the check at the end:

|  |  |  |
| --- | --- | --- |
|  |  | (8.55) |
|  |  | (8.56) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*pl,y,Rd and *M*pl,z,Rd | are the plastic bending resistances of the relevant plane of bending; |
|  | *M*y,Ed and *M*z,Ed | are the design bending moments including second‑order effects and imperfections in accordance with 8.8.3.4; |
|  | and | are defined in 8.8.3.6; |
|  |  | is given in 8.8.3.6(1);  is given in 8.8.3.6(1). |

### Shear connection and load introduction

#### General

(1) Provision shall be made in regions of load introduction for internal forces and moments applied from members connected to the ends and for loads applied within the length to be distributed between the steel and concrete components, considering the shear resistance at the interface between steel and concrete. A clearly defined load path shall be provided that does not involve an amount of slip at this interface that would invalidate the assumptions made in design.

(2) Where composite columns and compression members are subjected to significant transverse shear, as for example by local transverse loads and by end moments, provision shall be made for the transfer of the corresponding longitudinal shear stress at the interface between steel and concrete.

(3) For axially loaded columns and compression members, longitudinal shear outside the areas of load introduction need not be considered.

#### Load introduction

(1) Shear connectors should be provided in the load introduction area and in areas with change of cross-sections, if the design shear strength *τ*Rd (see 8.8.4.3), is exceeded at the interface between steel and concrete. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length. If the loads are introduced into the concrete cross-section only, the values resulting from an elastic analysis considering creep and shrinkage should be taken into account. Otherwise, the forces at the interface should be determined by elastic theory or plastic theory, to determine the more severe case.

(2) In the absence of a more accurate method, the introduction length should not exceed the minimum of 2 d or *L*/ 3, where *d* is the minimum transverse dimension of the column and *L* is the column length.

(3) For composite columns and compression members, no shear connection needs to be provided for load introduction by end plates if the full interface between the concrete section and end plate is permanently in compression, taking account of creep and shrinkage. Otherwise the load introduction should be verified in accordance with (5). For concrete filled tubes of circular cross‑section, the effect caused by the confinement may be taken into account if the conditions given in 8.8.3.2(6) are fulfilled using the values *η*a and *η*c for equal to zero.

(4) Where stud connectors are attached to the web of a fully or partially concrete encased steel I-section or a similar section, account may be taken of the frictional forces that develop from the prevention of lateral expansion of the concrete by the adjacent steel flanges. This resistance may be added to the calculated resistance of the shear connectors. The additional resistance may be assumed to be *µ* *P*Rd/2 on each flange and each horizontal row of studs, as shown in Figure 8.30, where *µ* is the relevant coefficient of friction that may be assumed. For steel sections without painting and free from oil, grease or loose scale or rust, *µ* may be taken as 0,5. *P*Rd is the resistance of a single stud in accordance with 8.6.8.1. In absence of better information from tests, the clear distance between the flanges should not exceed the values given in Figure 8.30.

(5) If the cross‑section is partially loaded (as, for example, Figure 8.31a), the loads may be distributed with a ratio of 1:2,5 over the thickness *t*e of the end plate. The concrete stresses should then be limited in the area of the effective load introduction, for concrete filled hollow sections in accordance with (6) and for all other types of cross‑sections in accordance with EN 1992-1-1:2023, 8.6.

A picture containing sketch, rectangle

Description automatically generated

**Figure 8.30 — Additional frictional forces in composite columns by use of headed studs**

(6) If the concrete in a filled circular hollow section or a square hollow section is only partially loaded, for example by gusset plates through the profile or by stiffeners as shown in Figure 8.31, the local design strength of concrete, *σ*c,Rd under the gusset plate or stiffener resulting from the sectional forces of the concrete section should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.57) |

where:

|  |  |  |
| --- | --- | --- |
|  | *t* | is the wall thickness of the steel tube; |
|  | *a*hs | is the diameter of the tube or the width of the square section; |
|  | *A*c | is the cross-sectional area of the concrete section of the column; |
|  | *A*1 | is the loaded area under the gusset plate, see Figure 8.31; |
|  | *η*cL | = 4,9 for circular steel tubes and 3,5 for square sections. |

The ratio *A*c/*A*1 should not exceed the value 20. Welds between the gusset plate and the steel hollow sections should be designed in accordance with FprEN 1993‑1‑8:2023, Clause 6.

(7) For concrete filled circular hollow sections, longitudinal reinforcement may be taken into account for the resistance of the column, even where the reinforcement is not welded to the end plates or in direct contact with the end plates, provided that:

* verification for fatigue is not required; and
* the gap *e*g between the reinforcement and the end plate does not exceed 30 mm, see Figure 8.31a).

(8) Transverse reinforcement should be in accordance with EN 1992-1-1:2023, 12.6. In cases of partially-encased steel sections, concrete should be held in place by transverse reinforcement arranged in accordance with Figure 8.12.

|  |  |
| --- | --- |
| A diagram of a machine  Description automatically generated | A diagram of a cylinder  Description automatically generated |
| a) | b) |

Figure 8.31 — Partially loaded circular concrete filled hollow section

(9) In cases of load introduction through only the steel section or the concrete section, for fully-encased steel sections, the transverse reinforcement should be designed for the longitudinal shear that results from the transmission of axial force (*N*c1 in Figure 8.32) from the parts of concrete directly connected by shear connectors into the parts of the concrete without direct shear connection (see Figure 8.32, section A‑A; the hatched area outside the flanges of Figure 8.32 should be considered as not directly connected).

The design and arrangement of transverse reinforcement should be based on a truss model assuming an angle of 45° between concrete compression struts and the member axis.

A picture containing sketch, diagram, technical drawing, plan

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | not directly connected |
| 2 | directly connected |

Figure 8.32 — Directly and not directly connected concrete areas for the design of transverse reinforcement

#### Longitudinal shear outside the areas of load introduction

(1) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and /or end moments. Shear connectors should be provided, based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength *τ*Rd.

(2) In the absence of a more accurate method, elastic analysis, considering long-term effects and cracking of concrete, may be used to determine the longitudinal shear at the interface.

(3) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in Table 8.8 may be assumed for *τ*Rd.

**Table 8.8 — Design shear strength *τ*Rd**

|  |  |
| --- | --- |
| Type of cross-section | *τ*Rd (N/mm2) |
| Completely concrete encased steel sections | 0,30 |
| Concrete filled circular hollow sections | 0,55 |
| Concrete filled rectangular hollow sections | 0,40 |
| Flanges of partially-encased sections | 0,20 |
| Webs of partially-encased sections | 0,00 |
| Completely or partially concrete encased steel sections with paint, oil, grease, loose scale or rust | 0,00 |

(4) The value of *τ*Rd given in Table 8.8 for completely concrete encased steel sections applies to sections with a minimum concrete cover of 40 mm and transverse and longitudinal reinforcement in accordance with 8.8.5.2. For greater concrete cover and adequate reinforcement, higher values of *τ*Rd may be used. Unless verified by tests, for completely encased sections the increased value *β*c *τ*Rd may be used, with *β*c determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (8.58) |

where:

|  |  |  |
| --- | --- | --- |
|  | *c*z | is the nominal value of concrete cover in mm, see Figure 8.24a; |
|  | *c*z,min | = 40 mm is the minimum concrete cover. |

(5) Unless otherwise verified, for partially-encased I‑sections with transverse shear due to bending about the weak axis due to lateral loading or end moments, shear connectors should always be provided. If the resistance to transverse shear is not be taken as only the resistance of the structural steel, then the required transverse reinforcement for the shear force *V*c,Ed in accordance with 8.8.3.2(4) should be welded to the web of the steel section or should pass through the web of the steel section.

### Detailing provisions

#### Concrete cover of steel profiles and reinforcement

(1) For fully-encased steel sections, at least a minimum cover of reinforced concrete shall be provided to ensure the safe transmission of bond forces, the protection of the steel against corrosion and spalling of concrete.

(2) The concrete cover for a flange of a fully-encased steel section should be not less than 40 mm, nor less than one‑sixth of the breadth *b* of the flange.

(3) The cover for reinforcement should be in accordance with EN 1992-1-1:2023, 6.4.

#### Longitudinal and transverse reinforcement

(1) The longitudinal reinforcement in concrete‑encased columns which is allowed for in the resistance of the cross‑section should be not less than 0,3% of the cross‑section of the concrete. Normally no longitudinal reinforcement is necessary in concrete filled hollow sections, if design for fire resistance is not required.

(2) The transverse and longitudinal reinforcement in fully or partially concrete encased columns should be designed and detailed in accordance with EN 1992-1-1:2023, 11.2 and 12.5.

(3) The clear distance between longitudinal reinforcing bars and the structural steel section may be smaller than required in (2), even zero. In this case, for bond the effective perimeter *c*eff of the reinforcing bar should be taken as half or one quarter of its perimeter, as shown in Figure 8.33 at (a) and (b) respectively.

A diagram of a beam

Description automatically generated

Figure 8.33 — Effective perimeter *c*eff of a reinforcing bar

(4) For fully or partially-encased members, where environmental conditions are Class *X*0 in accordance with EN 1992-1-1:2023, Table 6.1, and longitudinal reinforcement is neglected in design, a minimum longitudinal reinforcement of diameter 8 mm and 250 mm spacing, with a transverse reinforcement of diameter 6 mm and 200 mm spacing should be provided. Alternatively welded mesh reinforcement of diameter 4 mm may be used.

# Serviceability limit states

## General

(1) A structure with composite members shall be designed and constructed such that all relevant serviceability limit states are fulfilled in accordance with the principles of EN 1990:2023, 5.4.

(2) The verification of serviceability limit states should be based on the criteria given in EN 1990:2023, 5.4(3).

(3) Serviceability limit states for composite slabs with profiled steel sheeting should be verified in accordance with Clause 10.

## Stresses

### General

(1) Calculation of stresses for beams at the serviceability limit state shall take into account the following effects, where relevant:

* shear lag;
* creep and shrinkage of concrete;
* cracking of concrete and tension stiffening of concrete;
* sequence of construction;
* increased flexibility resulting from significant incomplete interaction due to slip of shear connection;
* inelastic behaviour of steel and reinforcement, if any; and
* torsional and distortional warping, if any.

(2) Shear lag may be taken into account in accordance with 7.4.1.2.

(3) Unless a more accurate method is used, effects of creep and shrinkage may be taken into account by use of modular ratios in accordance with 7.4.2.2.

(4) In cracked sections the primary effects of shrinkage may be neglected when verifying stresses.

(5) In section analysis, the cracking of concrete shall be taken into account.

(6) The influence of tension stiffening of concrete between cracks on stresses in reinforcement and pre‑stressing steel should be taken into account. Unless more accurate methods are used, the stresses in reinforcement should be determined in accordance with 9.4.3.

(7) The influences of tension stiffening on stresses in structural steel may be neglected.

(8) The effects of incomplete interaction may be ignored, where full shear connection is provided and where, in case of partial shear connection in buildings, 9.3.1(4) applies.

### Stress limitation for buildings

(1) Stress limitation is not required for beams if, in the ultimate limit state, no verification of fatigue is required and no pre‑stressing by tendons and/or by controlled imposed deformations (e.g. jacking of supports) is provided.

(2) For composite columns in buildings, normally no stress limitation is required.

(3) If required, the stress limitations for concrete and reinforcement given in EN 1992-1-1:2023, 9.1(3) should apply.

## Deformations in buildings

### Deflections

(1) Deflections due to loading applied to the steel member alone should be calculated in accordance with EN 1993‑1‑1.

(2) Deflections due to loading applied to the composite member should be calculated using appropriate methods. Elastic analysis in accordance with Clause 7 may be used. Otherwise the effect of possible yielding in the steel at midspan and reinforcement at the support should be taken into account. The effect of yielding of steel at the support may be considered in accordance with (8).

(3) The reference level for the vertical deflection *δ*max should be consistent with the requirements.

(4) The effects of slip [incomplete interaction, see 9.2.1(8)] may be ignored provided that:

1. the design of the shear connection is in accordance with 8.6;
2. either the number of shear connectors used is not less than the half of the number needed for full shear connection, or the forces resulting from an elastic behaviour and acting on the shear connectors in the serviceability limit state do not exceed PRd; and
3. when the beam comprises a steel section and a composite slab with ribs transverse to the beam, the height of the ribs *h*p does not exceed 80 mm.

(5) When the effects of slip have to be taken into account, the elastic deflection of a composite single span beam, where the loading is mainly uniformly distributed, and the cross-sections and the shear connectors remain elastic under the load combination for the serviceability limit state, may be estimated using an effective stiffness in accordance with Formula (9.1), unless a more accurate method is used.

|  |  |
| --- | --- |
|  | (9.1) |
|  | (9.2) |
|  | (9.3) |

Where:

|  |  |  |
| --- | --- | --- |
|  | *I*c,L | second moment of area of the concrete slab considering the modular ratio in accordance with 7.4.2.2; |
|  | *I*a | second moment of area of the structural steel section; |
|  | *n*L | modular ratio in accordance with 7.4.2.2, where the subscript index “L” represents the type of loading as defined in 7.4.2.2(2); |
|  | *A*a | area of the structural steel section; |
|  | *A*c | area of the concrete flange; |
|  | *A*c,L | area of the concrete flange considering time dependent effects; *A*c,L = *A*c/*n*L; |
|  | *L* | span length of the composite beam; |
|  | *c*s | stiffness of the shear connection cs = *k*sc*n*r/sx; |
|  | *k*sc | stiffness of a shear connector in accordance with Annexe B, Figure B.3; |
|  | *n*r | number of transversely spaced shear connectors; |
|  | *s*x | distance between uniformly spaced shear connectors; and |
|  | *z*cl | distance between centroids of the concrete slab and the steel section. |

NOTE For a concrete slab in which the reduction factor *k*t is unity, see 8.6.9.2, the approximate values for the stiffness *k*sc are given in A.3 (4).

(6) The effect of cracking of concrete in hogging moment regions on the deflection should be taken into account by adopting the methods of analysis given in 7.4.2.3.

(7) For beams with critical sections in Class 1, Class 2 or Class 3, the following simplified method may be used. At every internal support where *σ*ct exceeds 1,5 *f*ctm or 1,5 *η*lw,fct*f*ctm as appropriate, the bending moment determined by un‑cracked analysis defined in 7.4.2.3(2) is multiplied by the reduction factor *f*1 given in Figure 9.1, and corresponding increases are made to the bending moments in adjacent spans. The redistribution allowed by 7.4.4(8) should not be used. Curve *A* may be used for internal spans only, when the loadings per unit length on all spans are equal and the lengths of all spans do not differ by more than 25%. Otherwise the approximate lower bound value *f*1 = 0,6 (line B) should be used.

(8) For the calculation of deflection of un‑propped beams, account may be taken of the influence of local yielding of structural steel over a support by multiplying the bending moment at the support, determined in accordance with the methods given in (6), with an additional reduction factor as follows:

* *f*2 = 0,5 if *f*y is reached before the concrete slab has hardened; and
* *f*2 = 0,7 if *f*y is reached after concrete has hardened.

This applies to the determination of the maximum deflection but not to pre‑camber.

A diagram of a function

Description automatically generated

Figure 9.1 — Reduction factor for the bending moment at supports

(9) Unless specifically required by the client, the effect of curvature due to shrinkage of normal weight concrete need not be included when the ratio of span to overall depth of the beam is not greater than 20.

### Vibration

(1) The dynamic properties of floor beams should satisfy the criteria of EN 1990:2023, A.1.7.3.

## Cracking of concrete

### General

(1) For the limitation of crack width, the general considerations of EN 1992-1-1:2023, 9.2.1 may apply to composite structures. The limitation of crack width depends on the exposure Class in accordance with EN 1992-1-1:2023, 6.3.

(2) An estimation of crack width can be obtained from EN 1992-1-1:2023, 9.2.4, where the stress *σ*s should be calculated by taking into account the effects of tension stiffening. Unless a more precise method is used, *σ*s may be determined in accordance with 9.4.3(3).

(3) As a simplified and conservative alternative, crack width limitation to acceptable width can be achieved by ensuring a minimum reinforcement, defined in 9.4.2, and bar spacing or diameters not exceeding the limits defined in 9.4.3.

(4) In cases where beams in buildings are designed as simply supported although the slab is continuous and the control of crack width is of no interest, the longitudinal reinforcement provided within the effective width of the concrete slab, in accordance with 8.1.2, should be not less than:

* 0,4% of the area of the concrete, for propped construction;
* 0,2% of the area of concrete, for un‑propped construction.

For a beam designed as simply‑supported, the reinforcement should extend over a length of 0,25 *L* each side of an internal support, or of 0,5 *L* adjacent to a cantilever, where *L* is the length of the relevant span or the length of the cantilever respectively. No account should be taken of any profiled steel sheeting. The maximum spacing of the bars should be in accordance with 10.2.1(5) for a composite slab, or with EN 1992-1-1:2023, 12.4.1(1) for a solid concrete flange.

### Minimum reinforcement

(1) Unless a more accurate method is used in accordance with EN 1992-1-1:2023, 9.2.2, in all cross-sections without pre‑stressing by tendons and without significant tension due to restraint of imposed deformations, in combination or not with effects of direct loading, the required minimum reinforcement area *A*s for the slabs of composite beams is determined from:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  |  | (9.4) |

where:

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | *f*ct,eff | | is the mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur. Values of *f*ct,eff may be taken as those for *f*ctm, see EN 1992-1-1:2023, Table 5.1, or as  *η*lw,fct*f*ctm, see EN 1992-1-1:2023, M.1, as appropriate, taking as the Class the strength at the time cracking is expected to occur. When the age of the concrete at cracking cannot be established with confidence as being less than 28 days, a minimum tensile strength of 3 N/mm2 may be adopted; | |
|  | *k* | | is a coefficient which allows for the effect of non‑uniform self‑equilibrating stresses which may be taken as 0,8; | |
|  | *k*is | | is a coefficient which allows for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection, which may be taken as 0,9; | |
|  | *k*c | | is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and determined from: | |
|  | | |  | | (9.5) | |
|  | *h*c | | is the thickness of the concrete flange, excluding any haunch or ribs; | |
|  | *z*o | | is the vertical distance between the centroids of the un‑cracked concrete flange and the un‑cracked composite section, calculated using the modular ratio *n*0 for short‑term loading; | |
|  | *σ*s | | is the maximum stress permitted in the reinforcement immediately after cracking. This may be taken as its characteristic yield strength *f*sk. A lower value, depending on the bar size, may however be needed to satisfy the required crack width limits. This value is given in Table 9.1; | |
|  | *A*ct | | is the area of the tensile zone (caused by direct loading and primary effects of shrinkage) immediately prior to cracking of the cross-section. For simplicity the area of the concrete section within the effective width may be used. | |

(2) The maximum bar diameter for the minimum reinforcement may be modified to a value  determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (9.6) |

where:

|  |  |  |
| --- | --- | --- |
|  |  | is the maximum bar size given in Table 9.1; |
|  | *f*ct,0 | is a reference strength of 2,9 N/mm2. |

**Table 9.1 — Maximum bar diameters for high bond bars**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Steel stress | Maximum bar diameter (mm) | | | |
| σs | for design crack width *w*lim,cal | | | |
| (N/mm2) | *w*lim,cal = 0,4 mm | *w*lim,cal = 0,3 mm | *w*lim,cal = 0,2 mm | *w*lim,cal = 0,15 mm |
| 160 | 40 | 32 | 25 | 20 |
| 200 | 32 | 25 | 16 | 13 |
| 240 | 20 | 16 | 12 | 9 |
| 280 | 16 | 12 | 8 | 7 |
| 320 | 12 | 10 | 6 | 5 |
| 360 | 10 | 8 | 5 | 4 |
| 400 | 8 | 6 | 4 | 3 |
| 450 | 6 | 5 | ‑ | 3 |

(3) At least half of the required minimum reinforcement should be placed between mid‑depth of the slab and the face subjected to the greater tensile strain.

(4) For the determination of the minimum reinforcement in concrete flanges with variable depth transverse to the direction of the beam, the local depth should be used.

(5) For buildings, the minimum reinforcement in accordance with 9.4.3(1) and (2) should be placed where the concrete is in tension under the characteristic combination of actions.

(6) In buildings, minimum lower longitudinal reinforcement for the concrete encasement of the web of a steel I‑section should be determined from Formula (9.4) with *k*c taken as 0,6 and *k* taken as 0,8.

### Control of cracking due to direct loading

(1) Where at least the minimum reinforcement given by 9.4.2 is provided, the limitation of crack widths to acceptable values may generally be achieved by limiting bar spacing or bar diameters. Maximum bar diameters are given in Table 9.1 and maximum bar spacing in Table 9.2.

NOTE Maximum bar diameter and maximum bar spacing depend on the stress σs in the reinforcement and the design crack width.

**Table 9.2 — Maximum bar spacing for high bond bars**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Steel stress | Maximum bar spacing (mm) | | | |
| *σ*s | for design crack width wlim,cal | | | |
| (N/mm2) | wlim,cal = 0,4 mm | wlim,cal = 0,3 mm | wlim,cal = 0,2 mm | wlim,cal = 0,15 mm |
| 160 | 300 | 300 | 200 | 150 |
| 200 | 300 | 250 | 150 | 100 |
| 240 | 250 | 200 | 100 | 50 |
| 280 | 200 | 150 | 50 |  |
| 320 | 150 | 100 | ‑ |  |
| 360 | 100 | 50 | ‑ |  |

(2) The internal forces should be determined by elastic analysis in accordance with Clause 7 taking into account the effects of cracking of concrete. The stresses in the reinforcement should be determined taking into account effects of tension stiffening of concrete between cracks. Unless a more precise method is used, the stresses may be calculated in accordance with (3).

(3) The tensile stress in reinforcement σs due to direct loading may be calculated using Formula (9.7).

NOTE In composite beams where the concrete slab is assumed to be cracked and not pre‑stressed by tendons, stresses in reinforcement increase due to the effects of tension stiffening of concrete between cracks, compared with the stresses based on a composite section neglecting concrete.

|  |  |  |
| --- | --- | --- |
|  |  | (9.7) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (9.8) |
|  |  | (9.9) |

where:

|  |  |  |
| --- | --- | --- |
|  | *σ*s,0 | is the stress in the reinforcement caused by the internal forces acting on the composite section, calculated neglecting concrete in tension; |
|  | *f*ctm | is the mean tensile strength of the concrete, for normal concrete taken as *f*ctm from EN 1992-1-1:2023, Table 5.1 or for lightweight concrete as, *η*lw,fct*f*ctm where *η*lw,fct is from EN 1992-1-1:2023, Table M.1; |
|  | *ρ*s | is the reinforcement ratio, given by *ρ*s = (*A*s /*A*ct) ; |
|  | *A*ct | is the effective area of the concrete flange within the tensile zone; for simplicity the area of the concrete section within the effective width should be used; |
|  | *A*s | is the total area of all layers of longitudinal reinforcement within the effective area *A*ct; |
|  | *A*, *I* | are area and second moment of area, respectively, of the effective composite section neglecting concrete in tension and profiled sheeting, if any; |
|  | *A*a, *I*a | are the corresponding properties of the structural steel section. |

(4) For buildings without pre‑stressing by tendons, the quasi‑permanent combination of actions should typically be used for the determination of *σ*s.

(5) For cases where biaxial tension occurs and there are orthogonal reinforcing bars with stresses *σ*x and σy where *σ*x > *σ*y, the following should be assumed:

* The steel stress σs should be taken as;

 if ; (9.10)

 if ; (9.11)

* The bar spacing should be taken as the greater of the two centre-to-centre spacings of the orthogonal sets of bars.
* The stresses σx and σy should be determined in accordance with (3).

# Composite slabs with profiled steel sheeting for buildings

## General

### Scope

(1) Clause 10 deals with composite floor slabs spanning primarily in the direction of the ribs. Cantilever slabs are included. It applies to designs for building structures where the imposed loads are predominantly static, including industrial buildings where floors may be subject to moving loads.

(2) The scope is limited to sheets with narrowly spaced webs. Narrowly spaced webs are defined by an upper limit on the ratio *b*r / *b*s (see Figure 10.2). The value of the upper limit is to be taken as 0,65 unless it can be demonstrated that, taking account of all relevant effects including local buckling, shear lag and local loads, a higher ratio is valid.

NOTE A European Technical Product Specification can be used to demonstrate that a higher ratio is valid for a given profile.

(3) For structures where the imposed load is largely repetitive or applied abruptly in such a manner as to produce dynamic effects, composite slabs are permitted, but special care shall be taken over the detailed design to ensure that the composite action does not deteriorate in time.

(4) Slabs subject to seismic loading are not excluded, provided an appropriate design method for the seismic conditions is defined for the particular project or is given in another Eurocode.

(5) Composite slabs may be used to provide lateral restraint to the steel beams and to act as a diaphragm to resist horizontal actions, but no specific rules are given in this Standard. For diaphragm action of the profiled steel sheeting while it is acting as formwork, the rules given in FprEN 1993‑1‑3:2023, 11.4 and 11.5.4 apply.

### Definitions

#### Types of shear connection

(1) The profiled steel sheet shall be capable of transmitting horizontal shear at the interface between the sheet and the concrete; pure bond between steel sheeting and concrete is not considered effective for composite action. Composite behaviour between profiled sheeting and concrete shall be ensured by one or more of the following means, see Figure 10.1:

1. mechanical interlock provided by deformations in the profile (indentations or embossments);
2. frictional interlock for profiles shaped in a re‑entrant form;
3. end anchorage provided by welded studs or another type of local connection between the concrete and the steel sheet, only in combination with (a) or (b);
4. end anchorage by deformation of the ribs at the end of the sheeting, only in combination with (b).

Other means are not excluded but are not within the scope of this Standard.

A picture containing sketch, diagram, drawing, technical drawing

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**Key**

|  |  |
| --- | --- |
| 1 | mechanical interlock |
| 2 | frictional interlock |
| 3 | end anchorage by through‑deck welded studs |
| 4 | end anchorage by deformation of the ribs |

Figure 10.1 — Typical forms of interlock in composite slabs

#### Full shear connection and partial shear connection

(1) A span of a slab has full shear connection when an increase in the resistance of the longitudinal shear connection would not increase the design bending resistance of the member. Otherwise, the shear connection is partial.

## Detailing provisions

### Slab thickness and reinforcement

(1) The overall depth of the composite slab *h* should be not less than 80 mm. The thickness of concrete *h*c above the top of the profiled sheeting should be not less than 40 mm.

(2) If the slab is acting compositely with the beam or is used as a diaphragm, the total depth should be not less than 90 mm and *h*c shall be not less than 50 mm.

(3) Transverse and longitudinal reinforcement should be provided within the depth *h*c of the concrete.

(4) The amount of reinforcement in both directions should be not less than 80 mm2/m.

|  |
| --- |
| A diagram of a beam  Description automatically generated |
| a) |
| A diagram of a beam  Description automatically generated |
| b) |
| A diagram of a beam  Description automatically generated |
| c) |

**Key**

|  |  |
| --- | --- |
| a | Re-entrant profile |
| b | Open trough profile |
| c | Open trough profile with upper stiffener |

Figure 10.2 — Sheet and slab dimensions

(5) The spacing of the reinforcement bars should not exceed 2*h*cs and 350 mm, whichever is the lesser.

### Aggregate

(1) The nominal size of the aggregate depends on the smallest dimension in the structural element within which concrete is poured, and shall not exceed the least of:

* 0,40 *h*c (see Figure 10.2);
* *b*0 /3, where *b*0 is the mean width of the ribs (minimum width for re‑entrant profiles), see Figure 10.2; or
* 31,5 mm (sieve C 31,5).

### Bearing requirements

(1) The bearing length shall be such that damage to the slab and the bearing is avoided; that fastening of the sheet to the bearing can be achieved without damage to the bearing and that collapse cannot occur as a result of accidental displacement during erection.

(2) The bearing lengths *L*bc and *L*bs as indicated in Figure 10.3 should not be less than the following limiting values:

* for composite slabs bearing on steel or concrete: *L*bc = 75 mm and *L*bs = 50 mm;
* for composite slabs bearing on other materials: *L*bc = 100 mm and *L*bs = 70 mm.

|  |  |  |
| --- | --- | --- |
| A picture containing diagram, line, technical drawing, rectangle  Description automatically generated | A diagram of a rectangular object with a straight line  Description automatically generated | A drawing of a rectangular object  Description automatically generated |
| a) | b) | c) |

NOTE Overlapping of some sheeting profiles is impractical.

Figure 10.3 — Minimum bearing lengths

## Actions and action effects

### Design situations

(1) All relevant design situations and limit states shall be considered in design so as to ensure an adequate degree of safety and serviceability.

(2) The following situations shall be considered:

1. Profiled steel sheeting as shuttering: Verification is required for the behaviour of the profiled steel sheeting while it is acting as formwork for the wet concrete. Account shall be taken of the effect of props, if any.
2. Composite slab: Verification is required for the floor slab after composite behaviour has commenced and any props have been removed.

### Actions for profiled steel sheeting as shuttering

(1) The following loads should be taken into account in calculations for the steel deck as shuttering:

* weight of concrete and steel deck;
* construction loads including local heaping of concrete during construction, in accordance with prEN 1991‑1‑6:2024, 6.3.2;
* storage load, if any; and
* “ponding” effect (increased depth of concrete due to deflection of the sheeting).

(2) If the central deflection *δ*p of the sheeting under its own weight plus that of the wet concrete, calculated for serviceability, is less than 1/10 of the slab depth, the ponding effect may be ignored in the design of the steel sheeting. If this limit is exceeded, this effect should be allowed for. It may be assumed in design that the nominal thickness of the concrete is increased over the whole span by 0,7 *δ*p.

### Actions for composite slab

(1) Loads and load arrangements should be in accordance with EN 1991‑1‑1.

(2) In design checks for the ultimate limit state, it may be assumed that the whole of the loading acts on the composite slab, provided this assumption is also made in design for longitudinal shear.

## Analysis for internal forces and moments

### Profiled steel sheeting as shuttering

(1) The design of the profiled steel sheeting as shuttering should be in accordance with EN 1993‑1‑3.

(2) Plastic redistribution of moments should not be allowed when temporary supports are used.

### Analysis of composite slab

(1) The following methods of analysis may be used for ultimate limit states:

1. linear elastic global analysis with or without redistribution;
2. rigid-plastic global analysis provided that it is shown that sections where plastic rotations are required have sufficient rotation capacity; and
3. elastic global analysis, taking into account the non‑linear material properties.

(2) Linear methods of analysis should be used for serviceability limit states.

(3) If the effects of cracking of concrete are neglected in the analysis for ultimate limit states, the bending moments at internal supports may optionally be reduced by up to 30%, and corresponding increases made to the sagging bending moments in the adjacent spans.

(4) Plastic analysis without any direct check on rotation capacity may be used for the ultimate limit state if reinforcing steel of Class C in accordance with EN 1992-1-1:2023, Table 5.5 is used and the span is not greater than 3,0 m.

(5) A continuous slab may be designed as a series of simply supported spans. Nominal reinforcement in accordance with 10.8.1 should be provided over intermediate supports.

### Effective width of composite slab for concentrated point and line loads

(1) Where concentrated point or line loads are to be supported by the slab, they may be considered to be distributed over an effective width, unless a more exact analysis is carried out.

(2) Concentrated point or line loads parallel to the span of the slab should be considered to be distributed over a width *b*m, measured immediately above the ribs of the sheeting (see Figure 10.4), and determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.1) |

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Key

|  |  |
| --- | --- |
| 1 | Finishes |
| 2 | Reinforcement |

Figure 10.4 — Distribution of concentrated load

(3) For concentrated line loads perpendicular to the span of the slab, Formula (10.1) should be used for *b*m, with *b*p taken as the length of the concentrated line load.

(4) If *h*p/*h* does not exceed 0,6, the width of the slab considered to be effective for global analysis and for resistance may for simplification be determined with Formula (10.3) to Formula (10.5).

1. For bending and longitudinal shear:

* for simple spans and exterior spans of continuous slabs

|  |  |  |
| --- | --- | --- |
|  |  | (10.2) |

* for interior spans of continuous slabs

|  |  |  |
| --- | --- | --- |
|  |  | (10.3) |

(b) For vertical shear:

|  |  |  |
| --- | --- | --- |
|  |  | (10.4) |

where:

|  |  |  |
| --- | --- | --- |
|  | *b*sl | is the slab width; |
|  | *L*p | is the distance from the centre of the load to the nearest support; |
|  | *L* | is the span length. |

(5) If the imposed design loads do not exceed the following values, a nominal transverse reinforcement, as specified in (6) may be used without calculation:

* concentrated loads: 
* line load: 
* distributed load: 

where:

|  |  |  |
| --- | --- | --- |
|  | *F*Ed | is the design value of a single load; |
|  | *q*Ed | is the design value an uniformly distributed load; |
|  | *p*Ed | is the design value of a line load; |
|  | *d*s,c | is the depth below the top of the slab to the reinforcement in the cross direction in mm; |
|  | *h*c | is the thickness of the concrete slab above the profiled sheeting in mm; |
|  | *L* | is the span of composite slab in longitudinal direction in m. |

(6) This nominal transverse reinforcement should have a cross‑sectional area of not less than 0,2% of the area of structural concrete above the ribs, and should extend over a width of not less than the effective width *b*em. Minimum anchorage lengths should be provided beyond this width in accordance with EN 1992-1-1. Reinforcement provided for other purposes may fulfil all or part of this rule.

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Figure 10.5 — Definition of *d*s,c and *h*c

(7) Where the conditions in (5) are not satisfied, the transverse bending moments *m*Ed caused by line or point loads should be determined by more detailed calculation and adequate transverse reinforcement determined using EN 1992-1-1. The values of moment per unit length given in Formula (10.5) and Formula (10.6) may be used:

For concentrated loads *F*Ed:

|  |  |  |
| --- | --- | --- |
|  |  | (10.5) |

For line loads *p*Ed:

|  |  |  |
| --- | --- | --- |
|  |  | (10.6) |

where:

|  |  |  |
| --- | --- | --- |
|  | *I*cross | is the second moment of area of the cross-direction assuming an uncracked cross-section; |
|  | *I*long | is the second moment of area in the longitudinal direction of the slab assuming an uncracked cross-section, and |
|  | *L* | is the span of composite slab in longitudinal direction. |

For distributed loads *q*Ed:

the moment may be calculated using Formula (10.5) with a concentrated load *F*Ed equal to the maximum concentrated load that coexists with the distributed load. This may be taken as a value in kN of 1,5 times the value of distributed load in kN/m2.

The reinforcement provided to resist this moment should extend over a width equal to the span of the slab unless calculations show that a shorter length is adequate.

NOTE 1 This sub-clause does not cover shear failure. Where concentrated loads are present, punching is to be checked in accordance to 10.7.6. Where high line loads are present, shear failure in the transverse direction to the span of the slab is to be be checked in accordance with EN 1992-1-1.

NOTE 2 Where concentrated loads or line loads are present at the unsupported edge of a composite slab, hogging bending can occur. In such cases, a verification may be performed in accordance with EN 1992-1-1.

NOTE 3 The formulae for line loads were developed for a line load in the direction of the span, which is the most unfavourable case. In other cases, more complex models can lead to more economic results.

## Verification of profiled steel sheeting as shuttering for ultimate limit states

(1) Verification of the profiled steel sheeting for ultimate limit states should be in accordance with EN 1993‑1‑3. Due consideration should be given to the effect of embossments or indentations on the design resistances.

## Verification of profiled steel sheeting as shuttering for serviceability limit states

(1) Section properties should be determined in accordance with EN 1993‑1‑3.

(2) The deflection *δ*p of the sheeting under its own weight plus the weight of wet concrete, excluding the construction load, should not exceed *δ*p,max.

NOTE The value for *δ*p,max is *L* /180, where *L* is the effective span between supports (props being supports in this context), unless a different value is given in the National Annex.

## Verification of composite slabs for the ultimate limit states

### Design criterion

(1) The design values of internal forces shall not exceed the design values of resistance for the relevant ultimate limit states.

### Flexure

(1) In case of full shear connection, the bending resistance *M*Rd of any cross-section should be determined by plastic theory in accordance with 8.2.1.2(1) but with the design yield strength of the steel member (sheeting) taken as that for the sheeting, *f*yp,d. In the case of partial shear connection a reduced value of the compressive force in the concrete should be used in accordance with (8).

(2) The partial connection method should be used only for composite slabs with a ductile longitudinal shear behaviour.

(3) The longitudinal shear behaviour may be considered as ductile if the failure load exceeds the load causing a recorded end slip of 0,1 mm by more than 10%. If the maximum load is reached at a midspan deflection exceeding *L*/50, the failure load should be taken as the load at a midspan deflection of *L*/50.

(4) In hogging bending, the contribution of the steel sheeting shall only be taken into account where the sheet is continuous and when, for the construction phase, redistribution of moments due to plastification of cross-sections over supports has not been used.

(5) Where the effective area *A*pe of the steel sheeting is used in calculations, the width of embossments and indentations in the sheet should be taken into account in accordance with EN 1993-1-3 unless it has been demonstrated by testing that a larger area is effective.

(6) The effect of local buckling of compressed parts of the sheeting should be taken into account by using effective widths not exceeding twice the limiting values given in EN 1993-1-1:2022, Table 7.3 for Class 1 steel webs. Alternatively the effective section may be calculated in accordance with EN 1993-1-3.

(7) The sagging bending resistance of a cross-section with its plastic neutral axis above the main top flange of the sheeting, as shown in Figure 10.6, or in the sheeting, as shown in Figure 10.7, should be determined as follows in (8) to (12).

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Key

|  |  |
| --- | --- |
| 1 | centroidal axis of the profiled steel sheeting |
| 2 | axis of lower reinforcement |
| 3 | main top flange of sheeting |

Figure 10.6 — Stress distribution for sagging bending if the neutral axis is above the steel sheeting for full shear connection

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**Key**

|  |  |
| --- | --- |
| 1 | Centroidal axis of the profiled steel sheeting |
| 2 | Plastic neutral axis of the profiled sheeting |
| 3 | Axis of the lower reinforcement |
| 4 | Main top flange of sheeting |

Figure 10.7 — Stress distribution for sagging bending if neutral axis is in the steel sheeting for full shear connection

(8) The axial force in the concrete part of width *b*c should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.7) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (10.8) |
|  | when lower reinforcement is present | (10.9) |

where:

|  |  |  |
| --- | --- | --- |
|  |  | is the design shear strength  obtained from slab tests in accordance with Annex B. |
|  |  | is the partial factor for longitudinal shear in composite slabs for buildings, as given in 4.4.1.2(6). |
|  | *L*x | is the distance of the cross-section being considered from the nearest support reduced by 1/10 of the span for continuous slabs, where the shear span includes a region subject to hogging moment. |

The height *z*c of the compression zone of the concrete is limited to *h*c, therefore *N*p is limited to the value in Formula (10.10).

|  |  |  |
| --- | --- | --- |
|  |  | (10.10) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*c,f | is the axial force in the concrete for full shear connection (*f*cd *b*c *h*c) |

NOTE  In many cases, a lower additional reinforcement in the troughs is not necessary.

(9) For simplification *z*p, *z*s and *M*pr may be determined using Formula (10.11), (10.12) and (10.13) respectively:

|  |  |  |
| --- | --- | --- |
|  |  | (10.11) |

where:

|  |  |  |
| --- | --- | --- |
|  | *e*p | is the distance from the plastic neutral axis of the steel sheeting to the extreme fibre of the composite slab in tension; |
|  | *e* | is the distance from the bottom fibre of the sheeting to its centre of gravity; |
|  | *z*p | is the distance between the compression force in the concrete and the tension force in the steel sheeting. |

|  |  |  |
| --- | --- | --- |
|  |  | (10.12) |
|  |  | (10.13) |

where:

|  |  |  |
| --- | --- | --- |
|  | *M*pa | is the design value of the plastic resistance moment of the effective cross-section of the steel sheeting; |
|  | *M*pr | is the reduced plastic resistance moment of the steel sheeting; |
|  | *d*s | is the distance between the steel reinforcement in tension and the extreme fibre of the composite slab in compression; |
|  | *z*s | is the distance between the compression force in the slab and the tension force in the additional reinforcement. |

(10) The design moment resistance of the composite slab is given in Formula (10.14).

|  |  |  |
| --- | --- | --- |
|  |  | (10.14) |

(11) Where the reinforcement in the ribs or above the ribs contributes to the sagging moment resistance and *z*pl/*h* exceeds the value of 0,2, the resistance to bending in case of full-connection should be determined taking into account strain limits in accordance with EN 1992-1-1:2023, 8.1. The dimension *z*pl is the distance between the plastic neutral axis and the extreme top fibre of the concrete slab in compression, and h is the overall depth of the slab.

Alternatively, the design resistance moment of the composite slab may be taken as *β*d *M*pl,Rd, where *β*d is the reduction factor given in Formula (10.15) and the ratio *z*pl/*h* does not exceed a value of 0,45.

|  |  |  |  |
| --- | --- | --- | --- |
|  | when |  | (10.15) |
|  | when |  |

where:

|  |  |  |
| --- | --- | --- |
|  | *h* | total height of the composite slab, *h* = *h*c + *h*p |
|  | *z*pl | position of plastic neutral axis |

(12) Reinforcement in the troughs should only be taken into account, if the following conditions are fulfilled:

* the reinforcement is anchored in accordance with the rules given in EN 1992-1-1; and
* the longitudinal shear behaviour of the slab is ductile.

(13) If the contribution of the steel sheeting is neglected, the hogging bending resistance of a cross-section should be calculated using the stress distribution in Figure 10.8.

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Figure 10.8 — Stress distribution for hogging bending

### Longitudinal shear for slabs without end anchorage

(1) The provisions in 10.7.3 apply to composite slabs with mechanical or frictional interlock (types (a) and (b) as defined in 10.1.2.1).

(2) In Formula (10.7), *N*c may be increased by *μ*f,d *R*Ed provided that *τ*u,Rd is determined taking into account the additional longitudinal shear resistance caused by the support reaction,

where:

|  |  |  |
| --- | --- | --- |
|  | *R*Ed | is the support reaction; |
|  | *μ*f,d | is the design friction value (*μ*f,k /); |
|  | *μ*f,k | is the characteristic value of the friction coefficient; |
|  |  | is the partial factor for longitudinal shear in composite slabs for buildings, as given in 4.4.1.2(6). |

NOTE The characteristic value of the friction coefficient *µ*f,k for the deck to the concrete can be taken as 0,6 in accordance with EN 1337-1, unless the National Annex gives a different value. In other cases, *µ*f,k can be established from the results of tests.

### Longitudinal shear for slabs with end anchorage

(1) Unless a contribution to longitudinal shear resistance by other shear devices is shown by testing, the end anchorage of type I, as defined in 10.1.2.1, should be designed for the tensile force in the steel sheet at the ultimate limit state.

(2) The design resistance against longitudinal shear of slabs with end anchorage of type (c) and (d), as defined in 10.1.2.1, may be determined by the partial connection method as given in 10.7.2 with *N*c in Formula (10.7) increased by the design resistance of the end anchorage.

(3) The design resistance *P*pb.Rd of a headed stud welded through the steel sheet used for end anchorage should be taken as the smaller of the design shear resistance of the stud in accordance with 8.6.9.1 or the bearing resistance of the sheet determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.16) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (10.17) |

where:

|  |  |  |
| --- | --- | --- |
|  | *d*do | is the diameter of the weld collar which may be taken as 1,1 times the diameter of the shank of the stud; |
|  | *a*sc | is the distance from the centre of the stud to the end of the sheeting, to be not less than 1,5 *d*do; |
|  | *t*p | is the thickness of the sheeting, a nominal thickness of *t*p ≥ 0,7 mm is required |

(4) For end anchorage achieved through deformation of the ribs as shown in Figure 10.1, type 4, the design resistance *P*pb.Rd, is given by an ETA.

### Vertical shear

(1) Shear resistance should be verified at a distance *d*q from the supports or from a concentrated load, see EN 1992-1-1:2023, 8.2.2(1). Where a concentrated load is applied at a distance *a*q from the support, where *d*q ≤ *a*q ≤ 2*d*q, the contribution of this load to the design shear force *V*Ed may be multiplied by 0,5 *a*q/*d*q. Where *d*q is the effective depth of the composite slab.

(2) The vertical shear resistance *V*v,Rd of a composite slab over an effective width *b*s (see Figure 10.10), should be determined in the region of sagging bending in accordance with the following expression:

|  |  |  |
| --- | --- | --- |
|  |  | (10.18) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | partial factor for concrete, in accordance with 4.4.1.2(2); |
|  | *V*c,cz | shear resistance of the uncracked concrete compression zone; |
|  | *V*c,ks | shear resistance inducing spalling of concrete; |
|  | *V*c,cs | shear resistance due to the compression strut at the support; |
|  | *V*c,ct | shear resistance of concrete in crack propagation zone close to crack tip; |
|  | *R*w,Rd | resistance to transverse force of the web of steel sheeting, in accordance with FprEN 1993-1-3:2023, 8.1.6 or given by European Assessment Document (EAD).  A diagram of a curved object  Description automatically generated |

Figure 10.9 — Shear model for design of vertical shear resistance of composite slabs

(3) The shear resistance of the compression zone *V*c,cz should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.19) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*s,I | is the axial force in the reinforcement anchored at the front end of the slab support; |
|  | *N*p,I | is the axial force in steel sheeting anchored at the front end of the slab support; in accordance with 10.7.3 and 10.7.4, see Figure 10.9; |
|  | *f*ctm | is obtained in accordance with EN 1992-1-1:2023, Table 5.1 for normal concrete, to be replaced by in accordance with Table M.1 for lightweight concrete; |

(4) The shear resistance of vertical tension stresses in concrete inducing kinking and spalling *V*c,ks should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.20) |

where:

|  |  |  |
| --- | --- | --- |
|  | *b*min | width of concrete between the shear connection of the profiled steel sheeting, see Figure 10.11; |

|  |  |  |
| --- | --- | --- |
|  | *h*pc | height of centroid of the lowest shear connection device above the lower chord, for re-entrant profiles taken as 0,5 *h*p, see Figure 10.11.  A picture containing diagram, line, technical drawing, plan  Description automatically generated |

Figure 10.10 — Effective shear width of comb-shaped section

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**Figure 10.11 — Geometry of sheet with *b*min and *h*pc**

(5) The shear resistance due to the compression strut *V*c,cs should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.21) |

(6) The shear resistance of the concrete in the crack propagation zone close to the crack tip *V*c,ct should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.22) |

where:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  | 0,12 for normal weight concrete (NWAC), 0,09 for lightweight concrete (LWAC) | |
|  | *G*f | fracture energy of concrete: | |
|  |  |  | for NWAC, with *G*f [N/m] and *f*cm [MPa], |
|  |  |  | for LWAC with normal weight sand, |
|  |  |  | for LWAC with lightweight sand, also (ALWAC) |
|  |  | *f*ctm | in accordance with EN 1992-1-1:2023, 5.1.3 |
|  |  |  | according EN 1992-1-1:2023, Table M.1 |
|  | *b*0 | given in Figure 10.10 | |
|  | *d*s | distance from the centroid of the tension forces to the extreme fibre of the concrete in compression. *d*s,0 = 270 mm, but | |

(7) Alternatively to the method provided in (2) to (6) the vertical shear resistance for region of sagging bending may be simplified determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.23) |

where:

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | *V*c,Rd | is the governing shear resistance of the concrete ribs in [kN/m] with *τ*Rd,c in accordance with EN 1992-1-1:2023, 8.2.2 for a slab width of 1,0 meter EN 1992-1-1 in [kN/cm²]. Where *b*min and *b*s are given in Figure 10.10:   |  |  | | --- | --- | |  | (10.24) | |
|  | *z*cs | Inner lever arm to be taken as the distance between the centre of the compression zone and the centroid of the resulting tension force in [cm], |
|  | *V*b,e,Rd | is the design value for vertical shear of the profiled steel sheeting in [kN/m], with a width of 1,0 m, limited to the design value of permanent vertical shear forces Vg,Ed applied during construction, transferred to the non-propped profiled steel sheet, smaller than the shear resistance of the profiled steel sheeting Vb,e,Rd ≤ Vb,Rd, |
|  | *V*b,Rd | is the shear resistance of the profiled steel sheeting with a width of 1,0 m in [kN/m], in accordance with FprEN 1993-1-3:2023, 8.1.6 or given by an European Assessment Document (EAD). |

NOTE 1 The value *k*v can be taken as 1,0 when *V*b,e,Rd is considered as the design value of permanent vertical shear forces *V*g,Ed applied during construction, transferred to the profiled steel sheet, with *V*b,e,Rd ≤ *V*b,Rd, unless the National Annex gives a different value.

NOTE 2 The value *k*v is to be taken as 0,5 when *V*b,e,Rd is considered as the design value of vertical shear resistance of the profiled steel sheeting *V*b,Rd , unless the National Annex gives a different value.

NOTE 3 For the determination of the effective depth *d* and the reinforcement ratio ρl in accordance with EN 1992-1-1:2023, 8.2.1 and 8.2.2 as well as for the inner lever arm *z*cs, the National Annex can allow the use of the profiled steel sheeting as reinforcement for sagging moment. Sufficient anchorage of the reinforcement and the profiled sheeting is to be realised.

NOTE 4  EN 1992-1-1 is providing the shear resistance *τ*Rd,c in [MPa] while in formula (10.24) it is to be used in [kN/cm²].

(8) The vertical shear resistance *V*v,Rd of a composite slab in a region of hogging bending should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (10.25) |

where:

|  |  |  |
| --- | --- | --- |
|  | *V*c,Rd | is the governing shear resistance of the concrete ribs in [kN/m] as given by formula (10.24) whereby zcs is to be replaced by the lever arm z in accordance to EN 1992-1-1:2023, 8.2.1 and 3. |
|  | *V*g,Ed | is the design value of permanent vertical shear forces applied during construction in [kN/m], transferred to the non-propped profiled steel sheeting, with *V*g,Ed ≤ *V*b,Rd, |
|  | *V*b,Rd | is the shear resistance of the profiled steel sheeting with a width of 1,0 m in [kN/m], in accordance with FprEN 1993-1-3:2023, 8.1.6 or given by an European Assessment Document (EAD). |

NOTE The effective depth *d*s for for the determination of *V*c,Rd in accordance with EN 1992-1-1:2023, Formula 8.27 is to be taken as the distance between the centre of the compression zone and the centroid of the tension force resulting from the reinforcement.

(9) For the determination of the shear resistance *V*b,Rd of the profiled steel sheeting in (7) and (8) unfavorable effects resulting from normal stresses due to bending of the composite slab and, where parts of the profiled steel sheeting are in compression, the impact of buckling on *V*b,Rd should be considered. The interaction between normal stresses and shear stresses and impact of buckling may be taken into account in accordance with EN 1993-1-3.

### Punching shear

(1) The punching shear resistance of a composite slab at a concentrated load should be taken as the sum of the resistances of the two surfaces parallel to the direction of the profiled sheeting plus the resistances of the two surfaces transverse to the direction of the profiled sheeting.

(2) The resistance *V*pp,Rd of a surface parallel to the direction of the profiled steel sheeting should be determined in accordance with EN 1992-1-1:2023, 8.4.3 for slabs without shear reinforcement and EN 1992-1-1:2023, 8.4.4 for slabs with shear reinforcement. The width of the surface should extend *d*q either side of the concentrated load as shown in Figure 10.12.

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Key

|  |  |
| --- | --- |
| 1 | loaded area |
| 2 | surfaces parallel to the direction of sheeting |
| 3 | surfaces transverse to the direction of sheeting |
| 4 | direction of sheeting |

Figure 10.12 — Critical perimeter for punching shear

(2) The resistance *V*pt,Rd of a surface transverse to the direction of the profiled steel sheeting should be determined in accordance with Formula (10.26). The width of the surface should extend *d*cs either side of the concentrated load as shown in Figure 10.12. The determination of all components and coefficients are in accordance with section 10.7.5.

|  |  |
| --- | --- |
|  | (10.26) |

NOTE The component *V*c,cs in Formula (10.26) represents the direct compression strud to the support and is therefore neglected for the determination of the punching shear resistance in accordance with Formula (10.19).

## Verification of composite slabs for serviceability limit states

### Control of cracking of concrete

(1) The crack width in hogging moment regions of continuous slabs should be checked in accordance with EN 1992-1-1:2023, 9.2.

(2) Where continuous slabs are designed as simply‑supported in accordance with 10.4.2(5) and the control of cracking is of no interest, the cross‑sectional area of the reinforcement above the ribs should be not less than 0,2% of the cross‑sectional area of the concrete above the ribs for un‑propped construction and 0,4% of this cross‑sectional area for propped construction. The reinforcement should be placed in the zone where tensile strains are developed.

### Deflection

(1) The rules in EN 1990:2023, 5.4(3) should apply.

(2) Deflections due to loading applied to the steel sheeting alone should be calculated in accordance with FprEN 1993‑1‑3:2023, Clause 9.

(3) Deflections due to loading applied to the composite slab should be calculated using elastic analysis in accordance with Clause 7.

(4) The deflection of slabs may be considered to be acceptable without calculation where both the following conditions are satisfied:

* 

where:

|  |  |  |
| --- | --- | --- |
|  | *L* | is the span of the slab; |
|  | *d*q | is the –effective depth of the composite slab. |

* the conditions defined in (7) for neglect of the effects of end slip, are satisfied.

(5) For composite slabs used in buildings, unless a more detailed analysis is performed, the additional deflections caused by shrinkage should be determined from Formula (10.27) or (10.28).

|  |  |  |  |
| --- | --- | --- | --- |
|  |  | for single span slabs | (10.27) |
|  |  | for continuous slabs | (10.28) |

where

|  |  |  |
| --- | --- | --- |
|  | *L* | is the span; |
|  | *h* | is the overall depth of the composite slab; |
|  |  | is the deflection due to shrinkage; |
|  |  | is the shrinkage strain of the concrete. |

(6) For an internal span of a continuous slab where the shear connection is as defined in 10.1.2.1(a), (b) or (c), the deflection may be determined using the following approximations:

* the second moment of area may be taken as the average of the values for the cracked and un‑cracked section; and
* for concrete, an average value of the modular ratio for both long- and short‑term effects may be used.

(7) For external spans, no account need be taken of end slip if the initial slip load in tests according to B.2 (defined as the load causing an end slip of 0,5 mm) exceeds 1,2 times the design service load.

(8) Where end slip exceeding 0,5 mm occurs at a load below 1,2 times the design service load, then end anchors should be provided. Alternatively deflections should be calculated including the effect of end slip.

(9) If the influence of the shear connection between the sheeting and the concrete is not known from experimental verification for a composite floor with end anchorage, the composite slab should be modelled as tied arch. From that model, the lengthening and shortening gives the deflection that should be taken into account.

# Composite joints in frames for buildings

## Scope

(1) A composite joint is defined in 3.1.13. Some examples are shown in Figure 11.1. Concrete or steel joints in composite frames should be designed in accordance with EN 1992-1-1 or EN 1993‑1‑8, as appropriate.

NOTE Clause 11 concerns joints subject to predominantly static loading. It supplements or modifies EN 1993‑1‑8.

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Key

|  |  |
| --- | --- |
| 1 | Single‑sided configuration |
| 2 | Double‑sided configuration |
| 3 | Contact plate |

Figure 11.1 — Examples of composite joints

## Analysis, modelling and Classification

### General

(1) The provisions in FprEN 1993‑1‑8:2023, Clause 7 for joints connecting H- or I-sections are applicable with the modifications given in 11.2.2 and 11.2.3.

### Elastic global analysis

(1) Where the rotational stiffness *S*j is taken as *S*j,ini/in accordance with FprEN 1993-1-8:2023, 7.1.2, the value of the stiffness modification coefficient *η*j for a contact-plate connection should be taken as 1,5.

### Classification of joints

(1) Joints should be classified in accordance with FprEN 1993‑1‑8:2023, 7.3, taking account of effect of composite action on the resistance and rotational stiffness as given in 11.3.2 and 11.3.3, respectively.

(2) For the classification, the directions of the internal forces and moment should be considered.

(3) Cracking and creep in connected members may be neglected.

## Design methods

### Basis and scope

(1) FprEN 1993‑1‑8:2023, Clause 8 may be used as a basis for the design of composite beam‑to‑column joints and splices provided that the steelwork part of the joint is within the scope of that section.

(2) The structural properties of components assumed in design should be based on tests or on analytical or numerical methods supported by tests.

NOTE Properties of components are given in 11.4 and Annex A herein and in FprEN 1993‑1‑8:2023, Clause 8 and Annex A.

(3) In determining the structural properties of a composite joint, a row of reinforcing bars in tension may be treated in a manner similar to a bolt‑row in tension in a steel joint, provided that the structural properties are those of the reinforcement.

### Resistance

(1) Composite joints should be designed to resist vertical shear in accordance with relevant provisions of EN 1993‑1‑8.

(2) The design resistance moment of a composite joint with full shear connection should be determined by analogy to provisions for steel joints given in FprEN 1993‑1‑8:2023, Annex B.3, taking account of the contribution of reinforcement.

(3) The resistance of components should be determined from 11.4 below and FprEN 1993‑1‑8:2023, Annex A, where relevant.

### Rotational stiffness

(1) The rotational stiffness of a joint should be determined by analogy to provisions for steel joints given in FprEN 1993‑1‑8:2023, 7.2.6 and Annex B.2, taking account of the contribution of reinforcement.

(2) The value of the coefficient ψ, see FprEN 1993‑1‑8:2023, 7.2.6(4), should be taken as 1,7 for a contact plate joint.

### Rotation capacity

(1) The influence of cracking of concrete, tension stiffening and deformation of the shear connection should be considered in determining the rotation capacity.

(2) The rotation capacity of a composite joint may be demonstrated by experimental evidence. Account should be taken of possible variations of the properties of materials from specified characteristic values. Experimental demonstration is not required when using details which experience has proved have adequate properties.

(3) Alternatively, calculation methods may be used, provided that they are supported by tests.

## Resistance of components

### Scope

(1) The resistance of the following basic joint components should be determined in accordance with 11.4.2:

* longitudinal steel reinforcement in tension; and
* steel contact plate in compression.

(2) The resistance of components identified in EN 1993‑1‑8 should be taken as given therein, except as given in 11.4.3.

(3) The resistance of concrete encased webs in steel column sections should be determined in accordance with 11.4.4.

### Basic joint components

#### Longitudinal steel reinforcement in tension

(1) The effective width of the concrete flange should be determined for the cross‑section at the joint in accordance with 7.4.1.2.

(2) It should be assumed that the effective area of longitudinal reinforcement in tension is stressed to its design yield strength *f*sd.

(3) Where unbalanced loading occurs, a strut‑tie model may be used to verify the introduction of the forces in the concrete slab into the column (see Figure 11.2).

A picture containing diagram, line, sketch, technical drawing

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Figure 11.2 — Strut‑tie model

(4) For a single‑sided configuration designed as a composite joint, the effective longitudinal slab reinforcement in tension should be anchored sufficiently well beyond the span of the beam to enable the design tension resistance to be developed.

#### Steel contact plate in compression

(1) Where a height or breadth of the contact plate exceeds the corresponding dimension of the compression flange of the steel section, the effective dimension should be determined assuming dispersion at 45° through the contact plate.

(2) It should be assumed that the effective area of the contact plate in compression may be stressed to its design yield strength *f*yd.

### Column web in transverse compression

(1) For a contact plate connection, the effective width of the column web in compression *b*eff,c,wc should be determined assuming dispersion at 45° through the contact plate.

### Reinforced components

#### Column web panel in shear

(1) Where the steel column web is encased in concrete, see Figure 8.24b, the design shear resistance of the panel, determined in accordance with FprEN 1993‑1‑8:2023, Annex A.2.1, may be increased to allow for the encasement.

(2) For a single‑sided joint, or a double‑sided joint in which the beam depths are similar, the design shear resistance of concrete encasement to the column web panel *V*wp,c,Rd should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (11.1) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (11.2) |
|  |  | (11.3) |

where:

|  |  |  |
| --- | --- | --- |
|  | *b*c | is the breadth of the concrete encasement; |
|  | *h* | is the depth of the column section; |
|  | *t*f | is the column flange thickness; |
|  | *t*w | is the column web thickness; |
|  | *z* | is the lever arm, see FprEN 1993‑1‑8:2023, Annex B.1.2.1 and Table B.1. |

(3) The reduction factor to allow for the effect of axial compression in the column on the design resistance of the column web panel in shear should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (11.4) |

where:

|  |  |  |
| --- | --- | --- |
|  | *N*Ed | is the design compressive normal force in the column; |
|  | *N*pl,Rd | is the design plastic resistance of the column’s cross‑section including the encasement (see 8.8.3.2). |

#### Column web in transverse compression

(1) Where the steel column web is encased in concrete the design resistance of the column web in compression, determined in accordance with FprEN 1993‑1‑8:2023, Annex A.3, may be increased to allow for the encasement.

(2) The design resistance of the concrete encasement to the column web in transverse compression *F*c,wc,c,Rd should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (11.5) |

where:

|  |  |  |
| --- | --- | --- |
|  | *t*eff,c | is the effective length of concrete, determined in a similar manner to the effective width *b*eff,c,wc defined in FprEN 1993‑1‑8:2023, Annex A.5 |

(3) Where the concrete encasement is subject to a longitudinal compressive stress, its effect on the resistance of the concrete encasement in transverse compression may be considered by multiplying the value of *F*c,wc,c,Rd by a factor *k*wc,c determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (11.6) |

where:

|  |  |  |  |
| --- | --- | --- | --- |
|  |  | is the longitudinal compressive stress in the encasement due to the design normal force *N*Ed. |  |

In the absence of a more accurate method, , may be determined from the relative contribution of the concrete encasement to the plastic resistance of the column section in compression *N*pl,Rd, see 8.8.3.2.

1. (informative)  
     
   Stiffness of joint components in buildings
   1. Scope

(1) The stiffness of the following basic joint components may be determined in accordance with A.2.1 below:

* longitudinal steel reinforcement in tension;
* steel contact plate in compression.

(2) Stiffness coefficients *k*i are defined by FprEN 1993‑1‑8:2023, Formula B.6. The stiffness of components identified in that Standard may be taken as given therein, except as given in A.2.2 below.

(3) The stiffness of concrete encased webs in steel column sections may be determined in accordance with A.2.3 below.

(4) The influence of slip of the shear connection on joint stiffness may be determined in accordance with A.3.

* 1. Stiffness coefficients
     1. Basic joint components
        1. Longitudinal steel reinforcement in tension

(1) The stiffness coefficient *k*s,r for a row *r* may be obtained from Table A.1.

* + - 1. Steel contact plate in compression

(1) The stiffness coefficient may be taken as equal to infinity.

A drawing of a beam

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**Figure A.1 — Joints with bending moments**

Table A.1 — Stiffness coefficient *k*s,r

|  |  |  |
| --- | --- | --- |
| Configuration | Loading | Stiffness coefficient |
| Single‑sided |  |  |
| Double‑sided |  |  |
|  | For the joint with: |
|  |
| Wit |
|  |
| For the joint with : |
|  |
| *A*sr is the cross‑sectional area of the longitudinal reinforcement in row *r* within the effective width of the concrete flange determined for the cross‑section at the connection in accordance with 7.4.1.2; | | |
| *M*Ed,j is the design bending moment applied to a joint *j* by a connected beam, see Figure A.1; | | |
| *h* is the depth of the ’olumn's steel section, see Figure 8.24; | | |
| *β*  is the transformation parameter given in FprEN 1993‑1‑8:2023, 7.2. | | |
| NOTE The stiffness coefficient for *M*Ed,1 = *M*Ed,2 is applicable to a double‑sided beam‑to‑beam joint configuration under the same loading condition, provided that the breadth of the flange of the supporting primary beam replaces the depth *h* of the column section. | | |

* + 1. Other components in composite joints
       1. Column web panel in shear

(1) For an unstiffened panel in a joint with a steel contact plate connection, the stiffness coefficient *k*wp may be taken as 0,87 times the value given in FprEN 1993‑1‑8:2023, A.2.2.

* + - 1. Column web in transverse compression

(1) For an un‑stiffened web and a contact plate connection, the stiffness coefficient *k*c,wc may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (A.1) |

where:

|  |  |  |
| --- | --- | --- |
|  | *b*eff,c,wc | is the effective width of the column web in compression, see 11.4.3; |
|  | *d*wc | is the straight height of the column web, as defined by FprEN 1993-1-8:2023, A.3.1.1(7); |
|  | *t*wc | is the thickness of the column web. |

* + 1. Reinforced components
       1. Column web panel in shear

(1) Where the steel column web is encased in concrete, see Figure 8.24 b, the stiffness of the panel may be increased to allow for the encasement. The addition *k*wp,c to the stiffness coefficient *k*wp may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (A.2) |

where:

|  |  |  |
| --- | --- | --- |
|  | *E*cm | is the modulus of elasticity of the concrete for short-term loading in accordance with 5.1 (8); |
|  | *z* | is the lever arm, see FprEN 1993‑1‑8:2023, Table B.1; |
|  | *k*wp | is the stiffness coefficient for a steel column web given by FprEN 1993-1-8:2023, A.2.2(1); |
|  | *b*c, *h*c | See Figure 8.24. |

* + - 1. Column web in transverse compression

(1) Where the steel column web is encased in concrete, see Figure 8.24b, the stiffness of the column web in compression may be increased to allow for the encasement.

(2) For a contact plate connection, the addition *k*c,wc,c to the stiffness coefficient *k*c,wc may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (A.3) |

where:

|  |  |  |
| --- | --- | --- |
|  | *t*eff,c | is the effective length of concrete, see 11.4.4.2(2); |
|  | *b*c, *h*c | See Figure 8.24. |

(3) For an end plate connection, the addition *k*c,wc,c may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (A.4) |

* 1. Deformation of the shear connection

(1) Unless account is taken of deformation of the shear connection by a more exact method, the influence of slip on the stiffness of the joint may be determined by (2) to (4) below.

(2) The stiffness coefficient *k*s,r, see A.2.1.1, may be multiplied by the reduction factor, *k*slip determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (A.5) |

with:

|  |  |  |
| --- | --- | --- |
|  |  | (A.6) |
|  |  | (A.7) |
|  |  | (A.8) |

where:

|  |  |  |
| --- | --- | --- |
|  | *h*st | is the distance between the longitudinal reinforcing bars in tension and the centre of compression; see FprEN 1993‑1-8:2023, Table B.1 for the centre of compression; |
|  | *d*sb | is the distance between the longitudinal reinforcing bars in tension and the centroid of the beam's steel section; |
|  | *I*a | is the second moment of area of the beam's steel section; |
|  | *L* | is the length of the beam in hogging bending adjacent to the joint, which in a braced system may be taken as 15% of the length of the span; |
|  | Ls | is the number of shear connectors distributed over the length; |
|  | *k*sc | is the stiffness of one shear connector. |

NOTE An evaluation of *k*sc is given in Annex B.

(3) For a concrete slab in which the reduction factor *k*t is unity, see 8.6.9.2, the following approximate values may be assumed for *k*sc:

* for 19 mm diameter headed studs: 100 kN/mm
* for cold‑formed angles of 80 mm to 100 mm height: 70 kN/mm.

(4) For a composite joint with more than a single layer of reinforcement considered effective in tension, (2) above is applicable provided that the layers are represented by a single layer of equivalent cross‑sectional area and equivalent distances from the centre of compression and the centroid of the beam’s steel section.

1. (normative)  
     
   Standard tests
   1. General

(1) In this Standard rules are given for:

1. tests on shear connectors in B.2 and
2. testing of composite floor slabs in B.3.
   1. Test on shear connectors
      1. General

(1) As an alternative to the design rules in 8.6.8 and 8.6.9 and for cases where the rules are not applicable, the design may be based on tests, carried out in a way that provides information on the properties of the shear connection required for design in accordance with this Standard.

(2) The variables to be investigated include the geometry and the mechanical properties of the concrete slab, the shear connectors and the reinforcement.

(3) The resistance to loading, other than fatigue, may be determined by push tests in accordance with the requirements in this Annex.

(4) For fatigue tests, the specimen should also be prepared in accordance with this Annex.

* + 1. Testing arrangements
       1. General

(1) Where the shear connectors are used in T‑beams with a concrete slab of uniform thickness, or with haunches complying with 8.6.10.4, standard push tests should be used (B2.2.2). In other cases, such as slabs with profiled steel sheeting, specific push tests may be used (B2.2.3).

(2) For each variable to be investigated, tests on no fewer than three nominally identical specimens within a series should be undertaken to determine the characteristic and design resistance of the shear connectors.

(3) A load *P* should progressively be applied to the top of the steel section as shown in Figure B.1 and Figure B.2. For push tests where no recess, or alternative methods, have been provided at the base of the specimen (see Figure B.1 and Figure B.2), the loads from the tests should be reduced by 5% when establishing the properties of the connector.

* + - 1. Standard push tests

(1) For standard push tests the dimensions of the test specimen, the steel section and the reinforcement should be as given in Figure B.1.

(2) Where the purpose of the test is to determine values to be used in the design of floor beams, where both the permanent and variable actions will be applied normal to the face of the slab in the downward direction, the tie in Figure B.1 should be used to eliminate tensile forces in the shear connectors located at the bottom level.

A diagram of a rectangular object with numbers and letters

Description automatically generated with medium confidence

**Key**

|  |  |
| --- | --- |
| 1 | cover 15 mm |
| 2 | bedded mortar, gypsum or similar |
| 3 | recess |
| 4 | reinforcement: ribbed bars, *Φ* 10 mm resulting in a high bond with 450 ≤ *f*sk ≤ 550 N/mm2 |
| 5 | steel section: HE260B or 254 × 254 × 89 kg/m. UC |
| 6 | steel profile with external tension ties, or similar |

Figure B.1 — Test specimen for standard push test

* + - 1. Specific push tests

(1) Specific push tests may be used to determine the behaviour of shear connectors in concrete slabs of uniform thickness, haunches not complying with 8.6.10.4, or shear connectors welded within the rib of profiled steel sheeting.

(2) Specific push tests should be carried out on a test specimen generally as shown in Figure B.2. In addition to B.2.2.1 and Figure B.1:

1. the length *L* of each slab should be related to the longitudinal spacing of the connectors in the composite structure;
2. the slabs and the reinforcement should be suitably dimensioned in comparison with the beams for which the test is designed;
3. the thickness *h*cs of each slab should not exceed the minimum thickness of the slab in the beam;
4. the width *b*c of each slab should not exceed the effective width of the slab of the beam;
5. the slab should have the same location of reinforcement as the slab in the beam;
6. the amount of reinforcement and the thickness of the sheeting in the slab should not exceed that provided to the slab in the beam;
7. when two shear connectors per rib are provided, the distance between the centres of the outstand shear connectors *b*0i should not exceed that provided in the slab in the beam;
8. 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 *L*;
9. for open profile sheeting, a shear connector should not be welded within the last rib to the test slab at the top of the push test;
10. where a haunch in the beam does not comply with 8.6.10.4, the slabs of the push specimen should have the same haunch and reinforcement as the beam.

(3) Where the purpose of the test is to determine values to be used in the design of floor beams, and where both the permanent and variable actions will be applied normal to the face of the slab in the downward direction, a normal force *K*p *P* (Figure B.2) may be applied as uniformly as practicable to the external faces of the slabs, as an alternative to the optional tie given in Figure B.1. The dimension of the test specimen and the steel section should be as given in Figure B.2.

NOTE The value *K*p for the normal force is 0,1 unless the National Annex gives a different value.

(4) When the design resistance of shear connectors is based on push tests with transverse loading, the results should be limited to applications where 8.6.8.2(2) is satisfied.

A diagram of a machine

Description automatically generated with medium confidence

**Key**

|  |  |
| --- | --- |
| 1 | cover 15 mm |
| 2 | bedded mortar, gypsum or similar |
| 3 | recess |
| 4 | reinforcement |
| 5 | steel section: HE260B or 254 × 254 × 89 kg/m. UC |

Figure B.2 — Specific push-tests

* + 1. Preparation of specimens

(1) Both concrete slabs should be cast in the horizontal position. The concrete for all push specimens in a series to investigate one variable should be of the same mix and should be air-cured. The final arrangement should be symmetrical.

NOTE To minimise the variation in the concrete strength, the slabs can be cast at the same time by splitting the steel section into two halves and rejoining them after the concrete has hardened.

(2) Bond at the interface between flanges of the steel beam and the concrete should be prevented by greasing the flange or by other suitable means.

(3) For each series of nominally identical specimens that will be tested within 48 hours, a minimum of four concrete specimens (cylinders or cubes) for the determination of the cylinder strength should be prepared at the time of casting the push specimens. These concrete specimens should be cured alongside the push specimens. The concrete strength *f*cm,t of each series should be taken as the mean value, when the deviation of each specimen from the mean value does not exceed 10%. When the deviation of the compressive strength from the mean value exceeds 10%, the concrete strength should be taken as the maximum observed value.

(4) The compressive strength *f*cm,t of the concrete at the time of testing should not be greater than 1,25 *f*ck, where *f*ck is the specified strength of the concrete in the beams for which the test is designed. This requirement can be met by using concrete of the specified grade, but testing earlier than 28 days after casting of the specimens.

(5) If profiled steel sheeting is used for the slabs, the tensile strength and the yield strength of the profiled steel sheet should be obtained from coupon tests on specimens cut from the sheets as used in the push tests.

(6) The yield strength, the actual tensile strength *f*ut and the maximum elongation of a representative sample of the shear connector material used in the test specimens should be determined.

* + 1. Testing procedure

(1) The load *P* should first be applied progressively in increments up to 0,4 *P*e and then cycled 25 times between 0,05 *P*e and 0,4 *P*e, where *P*e is the expected failure load. If three tests are used, one of the three test specimens may be subjected to just the static test without cyclic loading in order to determine the level of the cyclic load for the other two.

(2) Subsequent load increments *P* should then be imposed progressively such that failure does not occur in less than 15 minutes.

(3) The longitudinal slip between each concrete slab and the steel section should be measured continuously during loading or at each load increment. The slip should be measured at least until the load has dropped to 20% below the maximum load.

(4) As close as possible to each group of connectors, the transverse separation between the steel section and each slab should be measured.

(5) The sum of the tie forces should not be greater than 0,1 *P*. Where a normal force is applied uniformly to the face of the slab, the value should not be greater than (*K*p *P*).

(6) When long-term effects are not directly taken into consideration in the design of a composite element, the load should be applied in 1,0 mm slip increments and paused for a duration of not less than 5 minutes to enable the load relaxation to be measured. The load-slip curve as shown in Figure B.3 should be plotted using the smallest load for each increment of slip.

NOTE If there is no direct check of short-term relaxation in the tests, the test load is to be reduced by 10%.

* + 1. Test evaluation

(1) If three tests on nominally identical specimens with headed stud shear connectors are carried out, and the deviation of any individual test result from the mean value obtained from all tests does not exceed 10%, the design resistance may be determined as follows:

- the characteristic resistance *P*Rk may be taken as the minimum failure load (divided by the number of connectors) reduced by 10 %;

- the design resistance *P*Rd may be taken as the smaller of Formula (B.1) and Formula (B.2):

|  |  |
| --- | --- |
|  | (B.1) |
|  | (B.2) |

where:

|  |  |  |
| --- | --- | --- |
|  | *f*u | is the minimum specified ultimate tensile strength of the material of the stud; |
|  | *f*ut | is the measured ultimate tensile strength of the material of the stud in the test specimen; |
|  | *f*cm | is the mean value of the concrete cylinder compressive strength obtained from EN 1992-1-1; |
|  | *f*cm,t | is the measured mean value of the concrete cylinder compressive strength in the test specimen; and |
|  | *γ*V | is the partial factor for shear connection, given in 4.4.1.2(5). |

(2) If the deviation from the mean exceeds 10%, at least three more tests of the same kind should be made. The test evaluation should then be carried out in accordance with EN 1990:2023, Annex D.

(3) For shear connectors other than headed studs, the test evaluation should be carried out in accordance with EN 1990:2023, Annex D.

(4) Where the connector is composed of two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the steel beam, the ties which resist separation should be sufficiently stiff and strong so that separation in push tests, measured when the connectors are subjected to 80 % of their ultimate load, is less than half of the longitudinal movement of the slab relative to the beam.

(5) The slip capacity of a specimen *δ*u should be taken as the maximum slip measured at the characteristic resistance *P*Rk, as shown in Figure B.3. The characteristic slip capacity *δ*uk should be taken as the minimum test value of *δ*u reduced by 10% or determined by statistical evaluation from all the test results. In the latter case, the characteristic slip capacity should be determined in accordance with EN 1990:2023, Annex D.

(6) The stiffness of a shear connector *k*sc should be taken as 0,7 *P*Rk/*s*e, where *s*e is the slip at a load of 0,7 *P*Rk, as shown in Figure B.3. The slip when the characteristic resistance *P*Rk is first reached *δ*e should be determined from:

|  |  |
| --- | --- |
|  | (B.3) |

A diagram of a function

Description automatically generated

**Figure B.3 — Determination of slip capacity *δ*u and stiffness *k*sc**

(7) If three tests on nominally identical specimens with head stud connectors are carried out, and the deviation of any individual test result from the mean value obtained from all tests does not exceed 10%, the characteristic value of the elastic slip *δ*ek should be taken as the minimum test value of *δ*e reduced by 10%, or determined by statistical evaluation from all the test results. In the latter case, the characteristic value of the elastic slip should be determined in accordance with EN 1990:2023, Annex D.

* 1. Testing of composite floor slabs
     1. General

(1) Tests in accordance with this section should be used for the determination of the value of *τ*u,Rd to be used for the verification of the resistance to longitudinal shear as given in Clause 10.

(2) From the load-deflection curves, the longitudinal shear behaviour is to be classified as brittle or ductile. The behaviour is deemed to be ductile if it is in accordance with 10.7.2(3). Otherwise the behaviour is classified as brittle.

(3) The variables to be considered within a complete investigation include the thickness and the type of steel sheeting, the steel grade, the coating of the steel sheet, the density and grade of concrete, the slab thickness and the shear span length *L*s.

(4) To reduce the number of tests as required for a complete investigation, the values of the shear strength *τ*u,Rd obtained from a test series may be used also for other values of variables as follows:

* for thickness of the steel sheeting *t*p larger than tested;
* for concrete with specified strength *f*ck not less than 0,8 *f*cm,t, where *f*cm,t is the mean value of the concrete strength in the tests;
* for steel sheeting having a nominal yield strength *f*yp not larger than the nominal yield strength of the material used in the test.

In addition the measured mean value of the yield strength of the steel used in the tests (*f*ypm,t) should not exceed 1,25 *f*yp where *f*yp is the nominal yield strength of the material.

* + 1. Testing arrangement

(1) Tests should be carried out on simply supported slabs.

(2) The test set-up should be as shown in Figure B.4 or equivalent.

(3) Two equal concentrated line loads, placed symmetrically at *L*/4 and 3*L*/4 on the span, should be applied to the specimen.

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Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | neoprene pad or equivalent ≤ 100 mm x *b*c |
| 2 | support bearing plate ≤ 100 mm x *b*c x 10 mm (min) (typical for all bearing plates) |

Figure B.4 — Test set‑up

(4) The distance between the centre line of the supports and the end of the slab should not exceed 100 mm.

(5) The width of the bearing plates and the line loads should not exceed 100 mm.

(6) For each variable to be tested, one group of a minimum of four tests should be carried out on specimens of same thickness *h*cs without additional reinforcement or end anchorage. In at least three of the tests, the shear span should be as long as possible, while still providing failure in longitudinal shear. In the remaining one test, the shear span should be as short as possible while still providing failure in longitudinal shear, but not less than 3 *h*cs in length. The one test with the short shear span should only be used for classifying the behaviour in accordance with B.3.1(2).

* + 1. Preparation of specimens

(1) The surface of the profiled steel sheet should be in the 'as-rolled' condition, no attempt being made to improve the bond by degreasing the surface.

(2) The shape and embossment of the profiled sheet should accurately represent the sheets to be used in practice. The measured spacing and depth of the embossments should not deviate from the nominal values by more than 5% and 10%, respectively.

(3) In the tension zone of the slabs, crack inducers should be placed across the full width of the test slab under the applied loads. The crack inducers should extend at least to the depth of the sheeting. Crack inducers are placed to better define the shear span length, *L*s, and to eliminate the tensile strength of concrete.

(4) It is permitted to restrain exterior webs of the deck so that they act as they would act in wider slabs.

(5) The width *b*c of test slabs should not be less than three times the overall depth, 600 mm and the cover width of the profiled sheet.

(6) Specimens should be cast in the fully supported condition. This is the most unfavourable situation for the shear bond mode of failure.

(7) Mesh reinforcement may be placed in the slab, for example to reinforce the slab during transportation, against shrinkage, etc. If placed, it must be located such that it acts in compression under sagging moment.

(8) The concrete for all specimens in a series to investigate one variable should be of the same mix and cured under the same conditions.

(9) For each group of slabs that will be tested within 48 hours, a minimum of four concrete specimens, for the determination of the cylinder or cube strength, should be prepared at the time of casting the test slabs. The concrete strength *f*cm,t of each group should be taken as the mean value, when the deviation of each specimen from the mean value does not exceed 10%. When the deviation of the compressive strength from the mean value exceeds 10%, the concrete strength should be taken as the maximum observed value.

(10) The tensile strength and yield strength of the profiled steel sheet should be obtained from coupon tests on specimens cut from each of the sheets used to form the test slabs.

* + 1. Test loading procedure

(1) The test loading procedure is intended to represent loading applied over a period of time. It is in two parts consisting of an initial phase, where the slab is subjected to cyclic loading; this is followed by a subsequent phase, where the slab is loaded to failure under an increasing load. One of the longer test specimens from each group may be subject to just the static test without cyclic loading, in order to determine the level of cyclic load for the remaining specimens, which should not be less than two.

(2) Initial phase: the slab should be subjected to an imposed cyclic load, which varies between a lower value not greater than *G* + 0,2 (*W*t – *G*) and an upper value not less than 0,6 *W*t, where *G* is the weight of the slab and *W*t is the measured failure load of the preliminary static test in accordance with (1).

(3) The loading should be applied for 25 cycles.

(4) Subsequent phase: on completion of the initial phase, the slab should be subjected to a static test where the imposed load is increased progressively, such that failure does not occur in less than 1 hour. The failure load *W*t is the maximum load imposed on the slab at failure plus the weight of the composite slab and spreader beams.

(5) In the subsequent phase, the load may be applied either as force-controlled or deflection-controlled.

* + 1. Determination of the design values for *τ*u,Rd

(1) The partial interaction diagram as shown in Figure B.5 should be determined using the measured dimensions and strengths of the concrete and the steel sheet. For the concrete strength the mean value *f*cm,t of a group as specified in B.3.3(9) may be used.

In Figure B.5:

|  |  |  |
| --- | --- | --- |
|  | *M*pl,Rm | is the plastic moment resistance of the composite slab with full shear connection, using measured values; |
|  | *N*c,fm | is the compressive normal force in the concrete flange at moment *M*pl,Rm; |
|  | *N*c,m | is the compressive normal force in the concrete flange with partial connection; |
|  | *f*ypm,t | is the mean value of the measured strength of the profiled steel sheeting. |

(2) Points on the resistance curve in Figure B.5 may be determined as follows.

Assume a value for the degree of shear connection, *η*. The depth of the concrete stress block for *η*=1 *z*pl,m is determined from:

|  |  |
| --- | --- |
|  | (B.4) |

where *b*c is the width of the concrete slab in the test specimen.

The lever arm *z* is determined from:

|  |  |
| --- | --- |
|  | (B.5) |

where *e* and *e*p are defined in 10.7.2 (9).

The reduced plastic moment *M*pr,m is determined from:

|  |  |
| --- | --- |
|  | (B.6) |

*M*pa,m is the plastic resistance moment of the effective cross-section of the profiled steel sheeting, based on measured values. The bending resistance *M* is determined from:

|  |  |
| --- | --- |
|  | (B.7) |

where *M*pl,Rm is the value of *M* for *η* = 1.

(3) From the maximum applied loads, the bending moment *M* at the cross-section under the point load due to the applied load, dead weight of the slab and spreader beams should be determined. The path A 🡪 B --> C in Figure B.5 then gives a value *η*test for each test, and a value *τ*u determined from:

|  |  |
| --- | --- |
|  | (B.8) |

where:

|  |  |  |
| --- | --- | --- |
|  | *L*s | is the shear span length as defined by Figure B.5; |
|  | *L*o | is the length of the overhang. |

A picture containing diagram, sketch, technical drawing, plan

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Figure B.5 — Determination of the degree of shear connection from *M*test

(4) If in design, the additional longitudinal shear resistance caused by the support reaction is taken into account in accordance with 10.7.3(2), *τ*u should be determined from:

|  |  |
| --- | --- |
|  | (B.9) |

where:

|  |  |  |
| --- | --- | --- |
|  | *µ*m | is the mean value of the coefficient of friction, which may be taken as 0,75 or established from tests. |
|  | *V*t | is the support reaction under the ultimate test load. |

(5) The characteristic shear strength*τ*u,Rk should be calculated from the test values as the 5% fractile using an appropriate statistical model in accordance with EN 1990:2023, Annex D. If the deviation of any individual test result from the mean value obtained from at least three tests within a group does not exceed 10%, the characteristic shear strength *τ*u,Rk may be taken as the minimum value of *τ*u reduced by 10 %.

(6) The design shear strength *τ*u,Rd is the characteristic strength*τ*u,Rk divided by the partial factor *γ*Vs, given in 4.4.1.2(6).

1. (informative)  
     
   Shrinkage of concrete for composite structures for buildings

(1) Unless accurate control of the profile during execution is essential, or where shrinkage is expected to take exceptional values, the nominal value of the total final free shrinkage strain may be taken as follows in calculations for the effects of shrinkage:

* in dry environments (whether outside or within buildings but excluding concrete‑filled members):

325 × 10‑6 for normal concrete;

500 × 10‑6 for lightweight concrete;

* in other environments and in filled members:

200 × 10‑6 for normal concrete;

300 × 10‑6 for lightweight concrete.

1. (normative)  
     
   Composite beams with web-openings
   1. Scope
      1. General

(1) This Annex extends the application of EN 1994-1-1 and EN 1993-1-13 to composite beams with web-openings and states the means by which composite action may be included in the design of the beam at opening positions. This Annex only applies to buildings. Advanced design and calculations methods may be used as an alternative to the rules in this Annex.

(2) The rules for steel beams with web openings are presented in EN 1993-1-13 and should be followed in the design of composite beams with web openings.

(3) The shapes of openings given in FprEN 1993-1-13:2023, 1.2.2 are covered. For *Vierendeel* bending, these opening shapes may be represented by an equivalent rectangle, as given in FprEN 1993-1-13:2023, 8.4.

(4) This Annex covers openings located in the sagging moment region of composite beams. The hogging moment region of continuous beams, cantilevers and columns are outside the scope of application.

(5) This Annex applies to composite beams with shear connectors which have a Ductility Category D2 or D3 in accordance with Table 5.1.

(6) This Annex covers composite beams with web openings where the flexural stiffness of the concrete slab above the hole can be neglected. Supplementary rules are given in Annex E for beams where the flexural stiffness is significant and where additional modes of failure are considered.

NOTE Criterion for composite beams where the flexural stiffness of the concrete slab is significant is given in E.1(3). The flexural stiffness of the concrete slab in composite beams in accordance with E.1(4) can always be neglected.

* + 1. Dimensional limits of openings

(1) Geometric limits for the opening and edge to edge spacing of web openings are defined in FprEN 1993-1-13:2023, 8.1.2 and 8.1.3.

(2) Other limits for composite beams are given in the relevant clauses.

(3) For widely spaced openings with maximum dimension less than 30% of the steel section depth or 200 mm whichever is the smaller, and with eccentricity of its centre line not exceeding 10% of the section depth, the verification of global bending in D.4.1.2 and of shear in D.4.1.3 is sufficient to satisfy the other checks in D.4.1.1(1), provided that the web slenderness does not exceed 72 . For circular openings, the maximum diameter may be increased to 40% of the section depth.

(4) Circular or hexagonal openings in composite beams should be considered as closely spaced when their edge to edge spacing is less than *h*o, where *h*o is the height of the openings.

(5) Rectangular or elongated circular openings in composite beams should be considered as closely spaced when their edge to edge spacing is less than the larger of *a*eff or 2 *h*o, where *a*eff is the effective opening length given in FprEN 1993-1-13:2023, 7.5(4). Where the adjacent openings are not identical the average values of *a*eff and *h*o may be used.

(6) The rules for closely spaced openings in D.3.4 are only applicable where the openings are nominally identical.

(7) For composite beams with closely spaced openings, supplementary rules given in D.4.2 should be applied.

(8) Two adjacent openings which do not fulfil the conditions given in (4) or (5) are widely spaced.

* 1. Method of design
     1. General

(1) For *Vierendeel* bending, a local bending resistance is developed due to composite action over the opening.

(2) The modes of failure of a composite beam at and between closely spaced openings are shown in Figure D.1. Additional failure modes should be considered for beams covered by Annex E.

(3) For composite beams with closely spaced openings, the in-plane moment acting in the web-post should be calculated in accordance with D.3.4.

(4) Alternative methods based on FprEN 1993-1-13:2023, 8.9 to 8.11 may also be used.

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Key

|  |  |
| --- | --- |
| 1 | Bottom Tee tension |
| 2 | Top Tee compression |
| 3 | Concrete compression |
| 4 | Shear at opening |
| 5 | Vierendeel bending |
| 6 | Web buckling adjacent to an opening |
| 7 | Web-post moment |
| 8 | Web-post shear |
| 9 | Web-post buckling |

Figure D.1 — Failure modes of composite beams at web openings

* + 1. Equivalent length and depth of openings

(1) For *Vierendeel* bending calculations for composite beams, the equivalent opening length, *a*eq, should be as given in FprEN 1993-1-13:2023, Table 8.3.

(2) For section classification and deflection calculations for composite beams, the effective length of openings, *a*eff, should be as given in FprEN 1993-1-13:2023, 7.5(4).

(3) The classification of the web outstand for *Vierendeel* bending should be in accordance with FprEN 1993-1-13:2023, 7.5(3), (5) and (6).

* + 1. Effective width of concrete slab at an opening

(1) For global bending resistance at an opening, the effective width of the concrete slab should be taken as given in 7.4.1.2.

(2) For the local bending resistance of composite Tee sections and the concrete slab, the effective width of the concrete slab at the opening, *b*eff,b, should be determined from:

|  |  |
| --- | --- |
|  | (D.1) |

where

|  |  |
| --- | --- |
| *d*y | is the width of load introduction perpendicular to the beam axis, defined by:  *d*y is equal to the distance between outer longitudinal rows of shear connectors (see Figure D.2): *d*y = *b*0i;  *d*y = 7,5 **t for single row of shear connector. |
| **t | is the diameter of the transverse reinforcement below the top of the shear connector at the high moment end of the opening; |
| *a*0 | is the length of the opening. |

NOTE Where steel sheeting is transverse to the beam, the resistance and ductility of more than two rows of shear studs is outside the scope of EN 1994-1-1.

Diagram of a metal beam with measurements

Description automatically generated

Figure D.2 — Effective widths

(3) For vertical shear resistance of the concrete slab, the effective width, *b*eff,V, should be determined from:

|  |  |
| --- | --- |
|  | (D.2) |

where *d*s,c,ten is the distance of the transverse reinforcement in the tension region to the upper edge of the slab.

(4) For composite slabs, the steel sheeting should be ignored in (2) and (3).

* 1. Analysis
     1. General

(1) The distribution of internal forces at opening locations is required for the verification of the beam design.

(2) Any distribution may be used provided it satisfies equilibrium and is consistently applied across all design checks. The distribution of internal forces given in D.3.2 to D.3.4 may be used.

* + 1. Axial forces at the centre of openings

(1) The global bending at the centre of the opening is resisted by axial forces in the steel Tees and concrete slab. These forces are initially calculated assuming there is no force in the steel Tee above the opening. If these forces, acting as a couple, are not sufficient to resist the applied moment, a force is then assumed to be present in the top Tee. As indicated in Figure D.3, a compressive force in the top Tee is considered to be positive.

NOTE The global moment is resisted by the axial forces alone, it is assumed that there is no moment in the steel Tees at the centre of an opening.

(2) The design tensile force in the bottom Tee, *N*bT,Ed, and the design compression force in the slab, *N*oc,Ed, may be determined from:

|  |  |
| --- | --- |
|  | (D.3) |

provided that:

|  |  |
| --- | --- |
| and | (D.4) |

where:

|  |  |
| --- | --- |
| *M*o,Ed | is the design value of the global applied moment taken at the centre-line of the opening; |
| *N*oc,Rd | is the compression resistance of concrete slab in accordance with D.4.1.2(6); |
| *N*bT,Rd | is the tension resistance of bottom Tee in accordance with D.4.1.2(3); |
| *d*c | is the distance of the centroid of the compressed part of the slab from the upper face of the steel profile, given by: |
| *z*c | is the depth of concrete in compression, with *z*c ≤ *h*c – see Formula (D.5).  (D.5) |
| *z*bT | is the distance of centroid of the bottom Tee from the bottom of the section; |
| *b*eff | is the effective width of the slab at the centre of the opening, see D.2.3(1). |

NOTE It is conservative to assume that *d*c = *h*p + 0,5 *h*c.

(3) The design compression force in the top Tee, *N*tT,Ed, should be taken equal to 0 when Formula (D.4) is fulfilled.

(4) Where the value of *N*oc,Ed from Formula (D.3) exceeds the resistance given in D.4.1.2(6), the axial forces in the members should be obtained by:

|  |  |
| --- | --- |
|  | (D.6) |
|  | (D.7) |
|  | (D.8) |

where:

|  |  |
| --- | --- |
| *z*tT | is the distance of centroid of the top Tee from the top of the steel section. |

(5) Where the value of *N*bT,Ed from Formula (D.3) exceeds the resistance given in D.4.1.2(3), the axial forces in the members should be obtained by:

|  |  |
| --- | --- |
|  | (D.9) |
|  | (D.10) |
|  | (D.11) |

A diagram of a rectangular object with a square and a square with a square and a square with a square with a square with a square with a square with a square with a square with a square

Description automatically generated

Figure D.3 — Axial forces at an opening in pure bending

* + 1. Distribution of shear forces

(1) The applied shear will normally vary over the length of the opening. The design shear at the centre of the opening *V*o,Ed may be taken as the maximum value over the length *a*eq.

(2) The global design shear force *V*o,Ed at the centre of an opening should be distributed between the slab, *V*oc,Ed, and the steel profile, *V*oa,Ed, (Figure D.4) with:

*V*o,Ed = *V*oc,Ed + *V*oa,Ed (D.12)

(3) It may be assumed that the concrete slab resists as much shear as possible. In this case:

*V*oc,Ed = min(*V*o,Ed, *V*oc,Rd)

*V*oa,Ed = *V*o,Ed – *V*oc,Ed

NOTE 1 This minimises the shear in the steel section and any effect on the *Vierendeel* moment resistance.

NOTE 2 To simplify calculations, the shear force in the slab can be neglected.

(4) The shear forces in the top and bottom Tees may be obtained assuming that the shear force in the profile is distributed in the Tees in accordance with their plastic shear resistance:

|  |  |
| --- | --- |
|  | (D.13) |
|  | (D.14) |

where *A*v,tT and *A*v,bT are the shear areas of the top and bottom Tees respectively, defined in EN 1993-1-1:2022, 8.2.6(3).

(5) As an alternative to (4), the shear forces *V*bT,Ed and *V*tT,Ed may be distributed between the Tees in order to minimise *M*wp,Ed as obtained in D.3.4(2).

A diagram of a diagram of a circuit

Description automatically generated

Figure D.4 — Shear forces at an opening

* + 1. Forces and moments in web-posts between closely spaced openings

(1) The forces acting on the web-post between closely spaced openings in a composite beam are shown in Figure D.5. The horizontal shear force, *V*wp,Ed, acting on the web‑post of a composite beam is given by the difference of the axial forces in the bottom Tees. *V*wp,Ed may be obtained by the larger of:

|  |  |
| --- | --- |
|  | (D.15) |
|  | (D.16) |

where:

|  |  |
| --- | --- |
| Δ*N*oc,Ed | is the compression force developed by the shear connectors placed between the centre-lines of adjacent openings, which may be obtained by: |
| *n*r/sx | is the number of shear connectors per unit length along the beam over the opening; |
| *s*o | is the spacing between the centre-lines of the adjacent openings; |
| *V*av,Ed | is the average value of the shear forces at the centre of the adjacent openings. |



Key

|  |  |
| --- | --- |
| 1 | Centre-lines of openings |
| 2 | Compression in web-post |

Figure D.5 — Forces and moment in the web-post between rectangular openings

(2) The in-plane moment, *M*wp,Ed, acting at the centre-line of the web post, may be obtained by:

|  |  |
| --- | --- |
|  | (D.17) |

where:

|  |  |
| --- | --- |
| *e*o | is the eccentricity of the openings to the centre of the steel section |
| *V*bT,Ed | is the shear force in the bottom Tee taken as the average of the shear in the adjacent openings. |

NOTE See D.1.2 (4), (5) and (6) for the scope of application of Formulas (D.15) to (D.17).

* + 1. Classification for global bending

(1) For global bending resistance, the cross-sections should be classified at each web-opening. At the opening, the flange and the web outstand, treated as an “outstand flange”, should be classified in accordance with 7.5.2.

(2) Rules defined in FprEN 1993-1-13:2023, 7.4(2) to (4) may be applied.

* + 1. Classification for *Vierendeel* bending

(1) FprEN 1993-1-13:2023, 7.5 applies.

* 1. Ultimate Limit States
     1. Design rules
        1. General

(1) The overall design of composite beams with web openings should be verified in accordance with Clause 8. Additional verifications for composite beams with large web openings should be considered in the region affected by the openings for:

1. Global bending resistance (see D.4.1.2);
2. Global shear resistance (see D.4.1.3);
3. Resistance to Vierendeel bending including the composite *Vierendeel* bending resistance (see D.4.1.4);
4. Web buckling resistance in the case of widely spaced openings (see D.4.1.5);
5. Web-post buckling, shear and bending resistance in the case of closely spaced openings (see D.4.2);

(2) The rules relevant to steel sections with web openings in EN 1993-1-13 should be applied taking account of the distribution of internal forces in composite beams in accordance with D.3.

(3) The spacing of the shear connectors in the region of the openings should not exceed 0,5 *a*o for rectangular or elongated openings or 0,7 *h*o for circular and hexagonal openings, to ensure effective composite action for *Vierendeel* bending. If this is not satisfied, the *Vierendeel* bending resistance due to composite action should be ignored.

NOTE The rules for the minimum spacing of connectors are given in 8.6.10.7(4).

(4) Supplementary rules are given in Annex E for the case where the bending resistance of the concrete slab adds to the Vierendeel bending resistance due to composite action.

(5) For beams with widely spaced circular openings or for other shapes of openings, the degree of shear connection should be calculated for the solid web composite beam and should satisfy the requirements of 8.6.3.3. For the application of Formula (8.13), the utilisation ratio *ρ*m of a composite beam with web openings should be obtained by:

|  |  |  |
| --- | --- | --- |
|  | and 0,8 ≤ m ≤ 1 | (D.18) |

where:

|  |  |
| --- | --- |
|  | for gross cross-sections as in 8.6.3.3(3). |
| , , | as defined in D.4.1.2(3), (4) and (6) respectively for cross-sections at opening centres. |

* + - 1. Global bending resistance

(1) The verification of global bending resistance in paragraphs (3) to (8) is based on verification of axial forces in the elements of the cross-section.

(2) For small openings as defined in D.1.2(3), the global bending resistance may be verified in accordance with 8.2.1, by excluding the area of the opening.

(3) The resistance of the bottom Tee to tension is adequate provided that:

|  |  |
| --- | --- |
|  | (D.19) |

where:

|  |  |
| --- | --- |
| *N*bT,Ed | is the tensile design force in the bottom Tee, obtained in accordance with D.3.2; |
| *N*bT,Rd | is the tensile design resistance, given by:  (D.20) |
| *A*a,bT | is the area of the gross cross-section of the bottom Tee. |

A diagram of a beam

Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | *N*c,Ed < *N*c,Rd and *N*tT,Ed = 0 |
| 2 | *N*c,Ed = *N*c,Rd and *N*tT,Ed > 0 |

Figure D.6 — Plastic distribution of stresses at an opening in pure global bending

(4) The resistance of the top Tee to axial force is adequate provided that:

|  |  |
| --- | --- |
|  | (D.21) |

where:

|  |  |
| --- | --- |
| *N*tT,Ed | is the design axial force in the bottom Tee, obtained in accordance with D.3.2; |
| *N*tT,Rd | is the design axial resistance, given by:  (D.22) |
| *A*a,tT | is the gross area for Class 1 to 3 cross-sections or the effective area for Class 4 cross-section, where the Class of the Tee cross-section is defined in accordance with D.3.5. |

(5) In the presence of high shear force in the steel profile, the moment-shear interaction should be taken into account in the calculation of *N*bT,Rd and *N*tT,Rd in accordance with D.4.1.3(5).

(6) The resistance of the slab to compression is adequate provided that:

|  |  |
| --- | --- |
|  | (D.23) |

where:

|  |  |
| --- | --- |
| *N*oc,Ed | is the compression design force in the slab, obtained in accordance with D.3.2; |
| *N*oc,Rd | is the compression design resistance, given by:  (D.24) |
| *N*sl,Rd | is the cumulated shear resistance of the connectors placed between the nearer support and the centre-line of the opening; |
| *b*eff | is the effective width of the slab determined at the centre-line of the opening. |

(7) The resistance of the slab to compression at the high-moment end of the opening should be verified by the following:

|  |  |
| --- | --- |
|  | (D.25) |

where:

|  |  |
| --- | --- |
| *b*hm,eff | is the effective width of the slab determined at the high-moment end of the opening; |
| *n*r | is the number of rows of shear connectors; |
| *s*x | is the longitudinal spacing of the rows above the opening. |

(8) Checks on the stability of the composite top Tee in the plane of the web may be omitted provided the opening length satisfies:

|  |  |
| --- | --- |
| *a*eff ≤ 12 *h*tT | for unstiffened openings; |
| *a*eff ≤ 16 *h*tT | for longitudinally stiffened openings. |

where

|  |  |
| --- | --- |
| *a*eff | is the effective opening length defined in D.2.2(2); |
| *h*tT | is the depth of the top Tee. |

* + - 1. Shear resistance of composite beam at web opening

(1) The shear resistance of the top Tee, bottom Tee and concrete slab should be checked using the assumed distribution of internal forces in accordance with D.3.

(2) The resistance to shear forces in the Tees should be checked by:

|  |  |
| --- | --- |
| *V*tT,Ed ≤ *V*tT,Rd and *V*bT,Ed ≤ *V*bT,Rd | (D.26) |

where:

|  |  |
| --- | --- |
| *V*tT,Ed and *V*bT,Ed | in accordance with D.3.3; |
| *V*tT,Rd and *V*bT,Rd | are the plastic shear resistance of the top and bottom Tees respectively, given by:  and  (D.27) |

(3) The resistance of flexible slabs to shear forces should be checked by:

|  |  |
| --- | --- |
|  | (D.28) |

where:

|  |  |
| --- | --- |
| *V*oc,Rd | is the shear resistance of the concrete slab at the opening position, given by:  (D.29) |
| *V*c,Rd | is the shear resistance of the concrete slab as defined in (4); |
|  | see D.3.2 (2). |

NOTE Flexible slabs are slabs where the flexural stiffness can be neglected, see D.1.1(6).

(4) The shear resistance of the concrete slab may be obtained in accordance with EN 1992-1-1:2023, 8.2, using the section width *b*eff,V defined by Formula (D.2). Alternatively a strut and tie model in accordance with EN 1992-1-1:2023, 8.5 may be used. The minimum value of *V*c,Rd at either end of the opening may be taken.

(5) For each Tee cross-section, when *V*iT,Ed > *η*v*V*iT,Rd, the effect of high vertical shear force on the resistance to moment and axial force should be taken into account by reducing the yield strength of the shear area of the Tee in accordance with EN 1993-1-1:2022, 2.10(3).

NOTE The value of *η*V is 0,5 unless the National Annexes provide an other value.

* + - 1. Shear resistance for Vierendeel bending

(1) The shear resistance due to *Vierendeel* bending at openings with locally flexible slabs [see NOTE in D.4.1.3(3)] should satisfy:

|  |  |
| --- | --- |
|  | (D.30) |

(2) The shear resistance due to *Vierendeel* bending is given by:

|  |  |
| --- | --- |
|  | (D.31) |

where:

|  |  |
| --- | --- |
| *M*NV,bT,Rd and *M*NV,tT,Rd | are the bending resistances of bottom and top Tees respectively, reduced by the effects of shear in accordance with FprEN 1993-1-13:2023, 8.2(2) or (3) and of tension or compression in accordance with FprEN 1993-1-13:2023, 8.4(6). |
| *a*eq | is the equivalent opening length for *Vierendeel* bending given in D.2.2(1). |
| *k*a | is a reduction factor to the *Vierendeel* bending resistance due to local composite action to take account of effects due to deformation across the opening and tensile forces in the shear connectors, and is given in (5) and (6). |

(3) When the condition given in D.4.1.1(3) is fulfilled, the contribution to *Vierendeel* bending resistance from composite action of the top Tee with the concrete slab is given by:

|  |  |
| --- | --- |
|  | (D.32) |

(4) For short unstiffened openings in which *a*eff ≤ 5 *h*tT or short stiffened openings in which *a*eff  ≤ 7 *h*tT, the factor *k*a in (2) may be taken as 1,0.

Where *a*eff is the effective opening length defined in D.2.2(2).

(5) For un–stiffened openings, the reduction factor *k*a is given by:

|  |  |
| --- | --- |
|  | (D.33) |

(6) For beams with longitudinally stiffened openings, in which the total cross-sectional area of the stiffeners satisfies the limits in FprEN 1993-1-13:2023, 8.1.3(1), *k*a is given by:

|  |  |
| --- | --- |
|  | (D.34) |

* + - 1. Web buckling next to widely spaced openings

(1) For widely spaced openings, the web buckling resistance next to the openings should be verified in accordance with FprEN 1993-1-13:2023, 8.5.2, where the compressive force in the web, *N*w,Ed, is taken as the larger of *V*tT,Ed + *V*oc,Ed and *V*bT,Ed.

(2) Web buckling next to web openings does not need to be checked for composite beams if:

|  |  |
| --- | --- |
| *h*o  ≤ 30 *t*w ԑ | for circular, hexagonal and elongated openings; |
| *h*o ≤ 18 *t*w ԑ | for rectangular openings. |

* + 1. Rules for closely spaced openings
       1. General

(1) These rules supplement the general rules given in D.4.1 and apply to composite beams with closely spaced web openings as defined in D.1.2(4) and (5).

* + - 1. Minimum degree of shear connection

(1) For composite beams with regular closely spaced circular web openings, Formula (8.13) in 8.6.3.3(2) may be replaced by:

|  |  |
| --- | --- |
|  | (D.35) |

where:

|  |  |
| --- | --- |
|  | see 8.6.3.3(3); |
| ρm | see D.4.1.1(5); |
| *A*atT/*A*a,bT | is the ratio of cross-sectional areas of the bottom and top Tees. |

* + - 1. Shear and bending resistance between openings

(1) The resistance to web-post bending should be checked in accordance with FprEN 1993-1-13:2023, 8.6.2(2) for circular, hexagonal or elongated openings and in accordance with FprEN 1993-1-13:2023, 8.6.2(3) for rectangular openings.

(2) The resistance to shear force should be checked in accordance with FprEN 1993-1-13:2023, 8.6.4.

(3) For beams composed of two rolled steel sections, the web-post shear and bending resistance should be obtained using the minimum web thickness.

(4) The design shear force *V*wp,Ed and the design moment *M*wp,Ed used in (1) and (2) should be obtained in accordance with D.3.

* + - 1. Web post buckling

(1) The web post buckling resistance should be checked in accordance with FprEN 1993-1-13:2023, 8.6.3, using the shear force *V*wp,Ed and the moment *M*wp,Ed as given in D.3.4.

* 1. Serviceability Limit States

(1) Web openings lead to additional bending and shear deflections, which should be calculated by a suitable method based on elastic principles including composite action at an opening. The calculation of the additional deflections should consider the reduction in the second moment of area due to the opening, *Vierendeel* bending of the Tee sections above and below the opening, and the flexibility of the web-posts and end posts.

(2) For simply supported uniformly loaded composite beams, the combined additional bending and *Vierendeel* bending deflection at mid-span due to a single opening at position, *x*o, from the nearer support may be taken as:

|  |  |  |  |
| --- | --- | --- | --- |
|  | for |  | (D.39) |





where:

|  |  |
| --- | --- |
| *w*add | is the additional mid-span deflection of the beam due to the opening; |
| *w*b | is the pure bending deflection of the composite beam calculated using *I*1,gross; |
| *I*1,gross | is the second moment of area of the equivalent composite section with a solid web; |
| *I*1,net | is the second moment of area of the equivalent composite section at the centre of the opening; |
| *I*bT | is the second moment of area of the bottom Tee; |
| *I*tT1 | is the second moment of area of the equivalent composite top Tee with an effective slab width *b*eff,b given in D.2.3(2); |
| *a*eff | is the effective opening length defined in D.2.2(2); |
| *L* | is the span length. |

The formulae for *r*w1 and *r*w2 do not include the effect of the flexibility of the web-post between openings. They may be used to calculate the total additional deflection due to more than one opening provided they are widely spaced.

NOTE Widely spaced openings are defined in D.1.2(8).

(3) For composite beams with multiple regularly spaced circular or hexagonal openings subject to uniformly distributed loading, the additional mid-span deflection relative to the pure bending deflection of the composite beam with a solid web may be estimated using the following formulae, provided that the ratio of the span length to the total depth is between 15 and 25 and the ratio of the depth of the slab to that of the steel beam is between 0,15 and 0,3:

|  |  |  |
| --- | --- | --- |
|  | for *w*p ≥ 0,5 *h*o | (D.40) |
|  | for *w*p < 0,5 *h*o | (D.41) |

where: *n*bo is the number of openings along the beam.

(4) For openings in which *a*eff exceeds 2 *h*o, the relative deflection due to *Vierendeel* bending across the opening may be determined from:

|  |  |
| --- | --- |
|  | (D.42) |

where:

|  |  |
| --- | --- |
| *V*o,ser,Ed | is the shear force at the centre-line of the opening at the serviceability limit state; |
| *I*tT1 | is the second moment of area of the equivalent composite top Tee with an effective slab width *b*eff,b given in D.2.3(2); |
| *I*bT | is the second moment of area of the bottom Tee. |

(5) When *a*eff exceeds 2*h*o, wv,add should not exceed *a*eff/150. For slabs where crack control is required, *w*v,add should not exceed *a*eff/200.

1. (informative)  
     
   Composite beams with web-openings and stiff slabs
   1. Scope

(1) This Annex supplements Annex D for widely spaced web openings with stiff slabs. The rules allow for additional *Vierendeel* resistance and provide checks for brittle failure modes due to slab shear and tension on the shear connectors.

(2) All the design rules given in Annex D should be applied to web openings with stiff slabs, except as given in E.3.

(3) At an opening location, the slab should be considered to be locally stiff when both of the following conditions apply:

* *h*c ≥ *h*ts,lim = *h*tT
* the longitudinal reinforcement ratio is greater than ρst,lim = 0,3%, based on the cross-sectional area of the slab over the effective width, *b*eff,b, given in D.2.3(2).

(4) For composite beams with regular closely spaced circular openings, slabs may always be considered to be locally flexible.

(5) Composite beams not covered by (4) with adjacent openings where *s*o < 2 *a*eq and a locally stiff slab in accordance with (3) are outside the scope of this Annex.

* 1. Analysis

(1) The flexural stiffness of the slab over the opening leads to significant moments in the slab (Figure E.1). At the low moment end of the opening, the moment in the slab, *M*A,Ed, leads to compression in the bottom of the slab. At the high moment end, the moment, *M*B,Ed, leads to compression at the top of the slab. The calculation of the axial forces at the openings should take this into account.

A diagram of a house

Description automatically generated

**Figure E.1 — Local shear force and bending moment acting in a locally stiff slab  
 around a rectangular web opening**

(2) The moments in the slab lead to tension in the shear connectors adjacent to the high moment end of the opening and significant shear in the slab. As the modes of failure due to the tension and shear are brittle, a conservative estimate of the slab moments should be assumed. In the absence of advanced analysis, the following (3) to (7) may be assumed for the analysis.

(3) The calculation of the axial forces at the centre of the openings should be in accordance with D.3.2 but the centre of compression in the slab should be assumed to be at mid height and the distance *d*c taken as *h*p + 0,5 *h*c. When calculating the additional force in the concrete due to shear connectors over the opening, those connectors within 0,5 *h*c of the high moment end should be neglected.

(4) The moment resistances of the slab at the ends of the openings, *M*A,Rd and *M*B,Rd, should be calculated in accordance with EN 1992-1-1 about the mid-height of the slab taking into account the presence of axial force.

(5) The maximum possible shear in the slab at the centre of the opening should be calculated. This may be taken as the shear on the composite beam, *V*o,Ed, less that which is associated with the elastic *Vierendeel* resistance of the steel Tees and that due to local composite action over the opening. The shear associated with bending in the slab may therefore be taken as that determined from:

|  |  |
| --- | --- |
|  | (E.1) |

Where:

|  |  |
| --- | --- |
| *M*elN,tT | is the moment required to cause yielding in the top steel Tee at the high moment end of the opening taking into account any axial force; |
| *M*elN,bT | is the moment required to cause yielding in the bottom steel Tee at the low moment end of the opening taking into account the axial force; |
| *d*c/*s*x | should not be taken as more than 0,5 with *d*c as given in (3). |

(6) Alternatively when the sequence of construction is known, the shear in the slab may be taken as the total shear less any shear applied to the steel during construction.

(7) The applied moments in the slab may be assumed to be proportional to the shear in the slab:

|  |  |
| --- | --- |
|  | (E.2) |
|  | (E.3) |

(8) The moment in the slab at the lower moment end of the opening, *M*A,Ed, causes a vertical compression force, *P*comp,Ed, to act on the slab. At ultimate limit state, *P*comp,Ed may be obtained by:

|  |  |
| --- | --- |
|  | (E.4) |

where *a*eqA is the distance from the lower moment end of the opening to the point where the local moment in the slab is equal to 0 (see Figure E.1). *a*eqA may be obtained by:

|  |  |
| --- | --- |
|  | (E.5) |

(9) The vertical tension force acting on the group of shear connectors at the higher moment side of the opening may be obtained by:

|  |  |
| --- | --- |
|  | (E.6) |

where *a*eqB is the distance from the higher moment end of the opening to the point where the local moment in the slab is equal to 0. *a*eqB may be obtained by:

|  |  |
| --- | --- |
|  | (E.7) |

(10) The tension force in the shear connectors may be taken as acting over a length equal to 2 (*h*c + *h*p), which should include a maximum group of two lines of shear connectors.

NOTE The acting length extends from 0,5 (*h*c + *h*p) inside the actual opening to 1,5 (*h*c + *h*p) outside the opening, see Figure E.2.

* 1. Additional checks at ultimate limit states for widely spaced openings with locally stiff slabs
     1. General

(1) The overall design of composite beams with web openings should be verified in accordance with Clause 8 and Annex D. Additional verifications for composite beams with large web openings and stiff slabs should be considered in the region affected by the openings for:

1. Global bending resistance (see E.3.2);
2. Global shear resistance (see E.3.3);
3. Resistance to *Vierendeel* bending including the composite *Vierendeel* bending resistance (see E.3.4);
4. Resistance of the connection at the opening edges (see E.3.5);
5. Web buckling resistance (see E.3.6);
6. Resistance of transverse reinforcement to local loads (see E.3.7).
   * 1. Global bending resistance

(1) The verification of global bending resistance should be carried out in accordance with D.4.1.2 using the forces calculated in E.2.

* + 1. Global shear resistance

(1) The applied shear in the concrete slab should be determined from:

|  |  |
| --- | --- |
|  | (E.8) |

In Formula (E.8) the ratio *d*c/*s*x should not be taken as more than 0,5 with *d*c as given in E.2(3).

(2) *V*oc,Ed should be less than the shear resistance of the concrete slab, *V*oc,Rd, given in D.4.1.3(3).

(3) Alternatively the shear resistance of the slab may be calculated using the strut and tie provisions of EN 1992-1-1:2023, 8.5.

(4) The difference between the total shear applied to the beam, *V*o,Ed, and that applied to the slab should be able to be resisted by the steel Tees in accordance with D.4.1.3(2).

(5) The requirements of EN 1992-4:2018, Annex A.2 should be met.

* + 1. Resistance to *Vierendeel* bending

(1) For rectangular or elongated openings with an edge to edge spacing, *s*o ≥ 2 *a*eq, and where the Class of the Tees in accordance with D.3.6 is 1 or 2, the contribution of the bending moments in a locally stiff slab, as shown in Figure E.2, to the overall *Vierendeel* resistance may be considered.

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Figure E.2 — Shear and tension forces acting in a solid concrete slab   
at a rectangular web opening

(2) The resistance to shear by *Vierendeel* bending should satisfy Forumla (E.9):

|  |  |
| --- | --- |
|  | (E.9) |

(3) The resistance *V*Vier,Rd to shear by Vierendeel bending, including the moments acting in the solid slab, is determined from:

|  |  |
| --- | --- |
|  | (E.10) |

Where:

|  |  |
| --- | --- |
| *M*c,Rd | is the bending resistance of the slab for the Vierendeel effect, given by:  *M*c,Rd = *M*A,Ed + *M*B,Ed (E.11) |
| *M*A,Ed and *M*B,Ed | are the bending moments in the slab calculated in E.2. |

NOTE The values of moment used can be less than the resistances of the sections. These are used because they are consistent with the calculated applied shear in the slab.

(4) When the condition in D.4.1.1(3) is satisfied, the contribution from composite action of the top Tee with the concrete slab *M*vc,Rd is determined from:

|  |  |
| --- | --- |
|  | (E.12) |

In Formula (E.12), the ratio *d*c/*s*x should not be taken as more than 0,5, with *d*c as given in E.2(3).

* + 1. Resistance of the shear connection at the opening edges

(1) The resistance of the shear connectors to the tension force acting on the group of shear connectors at the higher moment end of the opening is satisfied if:

|  |  |
| --- | --- |
|  | (E.13) |
|  | (E.14) |

where:

|  |  |
| --- | --- |
| *n*t | is the number of shear connectors in the group acting in tension defined in E.2(10). |
| *P*s,Ed | is the longitudinal shear force acting on a shear connector over the opening that is used to develop the *Vierendeel* bending resistance due to composite action, which may be taken as:  (E.15) |
| *P*ten,Rd | is the tension resistance of a shear connector in a slab – for headed studs see Annex H, where *ψ*ec,N in Formula (H.11) may be assumed equal to 1,0. |
| *P*Rd  *ρ*m | is the shear resistance of a shear connector – for headed studs, see 8.6.8 and 8.6.9.  is as given by Formula D.18 but can be less than 0.8. |

* + 1. Resistance to web buckling

(1) For widely spaced openings with a locally stiff slab, the web buckling resistance next to the openings should be verified in accordance with FprEN 1993-1-13:2023, 8.5.2, where the compressive force in the web, *N*w,Ed, is taken as the larger of *V*tT,Ed + *V*oc,Ed + *P*comp,Ed and *V*bT,Ed.

* + 1. Resistance of transverse reinforcement to local loads

(1) The resistance of the bottom transverse reinforcement should be verified using the following:

|  |  |
| --- | --- |
|  | (E.16) |

(2) *F*tr,Rd is the resistance of transverse reinforcement to local loads. *F*tr,Rd may be obtained using the simplified Formula (E.17).

|  |  |
| --- | --- |
|  | (E.17) |

where

|  |  |
| --- | --- |
| *h*c and *d*s,c,b | are defined in mm and *F*tr,Rd is in kN. |
| *d*s,c,b | is the distance of the centroid of the bottom reinforcement to the top of the slab; |
| ρt | is the transverse reinforcement ratio; |
| ρmin | is the minimum reinforcement ratio, equal to 0,2 %. |

Adequately anchored sheeting may be accounted for in the calculation of ρt.

1. (normative)  
     
   Headed studs that cause splitting forces in the direction of the slab thickness
   1. Design resistance and detailing

(1) The design longitudinal shear resistance of a headed stud in accordance with 8.6.8.1, that causes splitting forces in the direction of the slab thickness, see Figure F.1, should be the lowest value given by Formula (F.1), and Formulas (8.23) and (8.24). This resistance should be used for ultimate limit states other than fatigue.

|  |  |
| --- | --- |
|  | (F.1) |

where:

|  |  |
| --- | --- |
| *a*rp | is the effective edge distance, given by: *a*rp =*a*r - *c*v - *ϕ*s/2 ≥ 50 mm ; |
| *a*r | is the distance between the axis of the stud and the closest concrete surface; |
| *k*v | = 1,0 for shear connection in an edge position (see Figure F.1);  = 1,14 for shear connection in an internal position; |
| *γV* | is the partial factor for shear connectors – see 4.4.1.2(5); |
| *f*ck | is the characteristic cylinder strength of the concrete in N/mm2; |
| *d* | is the diameter of the shank of the stud with 19 ≤ *d* ≤ 25 mm; |
| *h*sc | is the length after welding of the headed stud with *h*sc/*d* ≥ 4; |
| *s*x | is the longitudinal spacing of studs with 110 ≤ *s*x ≤ 440 mm; |
| *s* | is the spacing of the stirrups with *s*x /2 ≤ *s* ≤ *s*x and *s*/*a*rp ≤ 3; |
|  | is the diameter of the stirrups with ; |
|  | is the diameter of the longitudinal reinforcement with ; |
| *c*v | is the nominal concrete cover in accordance with Figure F.1 in mm. |

|  |
| --- |
| A diagram of a beam with symbols  Description automatically generated with medium confidence |
| a) |
|  |
| b) |
|  |
| c) |

**Key**

|  |  |
| --- | --- |
| 1 | transverse reinforcement |
| a | internal position |
| b | edge position |
| c | section A-A |

Figure F.1 — Position and geometrical parameters of headed studs positioned close to a concrete surface

(2) A failure by pull-out of the stud at the edge of the slab should be prevented by fulfilling the following conditions:

Uncracked concrete:  or 

Cracked concrete:  or 

with *v* and  as shown in Figure F.1.

Alternatively, the pull-out of the stud may be verified in accordance with EN 1992-4.

NOTE 1 If tension forces in the studs occur from global loading, their influence is to be taken into account e.g. by verification in accordance with EN 1992-4.

NOTE 2 Annex H specifies the tensile resistance of headed studs under certain conditions.

(3) The influence of vertical shear on the design resistance of a stud connector supporting a slab edge should be taken into account. The interaction may be verified by the following:

|  |  |
| --- | --- |
|  | (F.2) |

where

|  |  |
| --- | --- |
| *F*L,Ed | is the design longitudinal shear force; |
| *F*V,Ed | is the design vertical shear force; |
| *P*V,Rd | is the design vertical shear resistance, determined from: |

|  |  |
| --- | --- |
|  | (F.3) |

where:

|  |  |
| --- | --- |
| *a*rp,o | is the relevant effective edge distance, with: *a*rp,o = *a*r,o - *c*v - *ϕ*s/2 ≥ 50 mm ; |
| *a*r,o | is the distance between the axis of the stud and the slab surface where the concrete cone failure would appear. |

The following conditions should also be satisfied:

*hsc* ≥ 100 mm; 110 ≤ *s*x ≤ 250 mm; *ϕ*s ≤ 12 mm ; *ϕ*l ≤ 16 mm;

For vertical shear in an internal position, *k*v may be taken as 1,25.

(4) Where, as shown in Figure F.2, there are two rows of studs, the vertical shear resistance should be calculated for each row separately and the sum of these resistances multiplied by the factor *η*r given in Formula (F.4), with

|  |  |  |
| --- | --- | --- |
|  |  |  |
|  |  | (F.4) |
|  |  |  |

|  |  |
| --- | --- |
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| a) | b) |

**Key**

|  |  |
| --- | --- |
| a | section A-A |
| b | edge position |

Figure F.2 — Double row of headed studs

(5) The splitting force in the direction of the slab thickness should be resisted by stirrups, which should be designed for a tensile force determined from:

|  |  |
| --- | --- |
|  | (F.5) |

where:

|  |  |
| --- | --- |
|  | is the design longitudinal shear force; |
|  | is the design vertical shear force. |

* 1. Fatigue strength

(1) The fatigue strength curve of headed studs causing splitting forces under longitudinal and vertical shear is given for normal-weight concrete by the lower of the values from prEN 1994-2:2024, 8.7 and the following:

|  |  |
| --- | --- |
|  | (F.6) |

where:

|  |  |
| --- | --- |
| Δ*P*R | is the fatigue strength based on the range of shear force per stud; |
| Δ*P*c | is the reference value of fatigue strength at *N*cyc = 2 x 106 in accordance with Table F.1; |
| *m* | is the slope of the fatigue strength curve with ; |
| *N*cyc,f | is the number of force range cycles. |

In Table F.1, *a*rp is the effective edge distance according Figure F.1 and Clause F.1(1).

**Table F.1 — Fatigue strength Δ*P*c for headed studs close to a surface**

|  |  |  |
| --- | --- | --- |
|  | Longitudinal shear | Vertical shear |
| *a*rp  mm | kN | kN |
| 50 | 24,9 | 8,9\* |
| 100 | 34,2 | 27,7\* |
| ≥ 125 | 34,2 | 34,2 |
| NOTE Intermediate values is to be determined by linear interpolation  ⃰ Where longitudinal tension causes cracking of the concrete, the fatigue strength for vertical shear should be halved | | |

Where the concrete strength *f*ck < 30 N/mm2, the fatigue strength Δ*P*c should be multiplied by *f*ck / 30. The values in Table F.1 are valid for stud diameter greater or equal to 22 mm. For 19 mm diameter studs, the fatigue strength should be reduced to 75% of the values in Table F.1.

(2) For the maximum fatigue strength per connector prEN 1994-2:2024, 8.7.1(3) applies.

(3) An interaction of longitudinal and vertical shear should be taken into account for the edge position by applying the following rule:

|  |  |
| --- | --- |
|  | (F.7) |

where:

|  |  |
| --- | --- |
| *ΔP*L,R and *ΔP*V,R | can be determined equivalent to prEN 1994-2:2024, 8.7.6.2. |
|  | is the reference value of fatigue strength for the range of longitudinal shear force per stud at *N*cyc = 2x106 in accordance with Table F.1; |
|  | is the reference value of fatigue strength for the range of vertical shear force per stud at *N*cyc = 2x106 in accordance with Table F.1; |
|  | is the fatigue strength based on the range of longitudinal shear force per stud; |
|  | is the fatigue strength based on the range of vertical shear force per stud; |
|  | see prEN 1994-2:2024, 4.4.1.2(3); |
|  | partial factor for fatigue actions – see EN 1990:2023, 8.3.3.6(1). |

(4) For the stirrups, a fatigue verification as given in EN 1992-1-1:2023, Clause 10 should be carried out. The mandrel diameter should be considered.

1. (informative)  
     
   Design resistance of headed studs used with open trough profiled steel sheeting in buildings with ribs transverse to the supporting beams
   1. Scope

(1) When the embedment depth *h*A satisfies 8.6.10.8(1) but is less than 2,7*d*, or the distance from the edge of the concrete rib on the higher moment side to the centre-line of the nearest stud connector *e*k satisfies 25 mm ≤ *e*k ≤ 60 mm (Figure 8.19), the values for *P*Rd given by G.2 apply, provided that all the following conditions are satisfied:

* the dimension for *e*k is more than 25mm;
* the nominal thickness *t*p of the steel sheeting is not less than 0,70 mm;
* the nominal concrete cover above the stud satisfies 8.6.10.2;
* the height of the profiled sheeting *h*p does not exceed a value of 135 mm;
* 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;
* concrete strength Class is not greater than C50/60 and density not less than 2200 kg/m3.

For other cases, specific tests as specified in B.2 should be used to determine the design shear resistance.

(2) The Ductility Category of the shear connector should be verified by tests in accordance with Annex B.

* 1. Shear resistance

(1) The design shear resistance should be taken as the smaller of the values from Formula (G.1) and Formula (G.2).

|  |  |  |
| --- | --- | --- |
|  |  | (G.1) |
| or |  | (G.2) |

with

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
|  |  | but | |  | (G.3) |
|  |  | | | | (G.4) |
|  | for  or studs in a staggered position, see Figure G.1; | | | | (G.5) |
|  |  | | for other cases | | (G.6) |
|  |  | | | | (G.7) |

where:

|  |  |
| --- | --- |
| γV | is the partial factor in accordance with 4.4.1.2(5); |
| *d* | is the diameter of the shank of the stud; |
| *f*u | is the specified ultimate tensile strength of the material of the stud, but not greater than 450 N/mm2; |
| *k*cc | is a reduction value taking into account the effect from concrete relaxation and sustained loading, in accordance with 8.6.8.1(1); |
| *k*u | is a correction factor, taken from Table G.1; |
| *f*ctk,0,05 | is the characteristic value of tensile strength of concrete in accordance with EN 1992-1-1:2023, Table 5.1; |
| *h*p | is the overall height of the profiled steel sheeting, excluding the height of any top re-entrant stiffener, provided that the geometry satisfies 8.6.10.8(3); |
| *n*r | is the number of stud connectors in one rib at the beam intersection, not to exceed 2; |
| *h*sc | is the length after welding of the stud in accordance with EN ISO 13918:2018, Table 10, which may be assumed to apply for through-deck welded studs;  NOTE For through-deck welded studs, the shorter length after welding is accounted for in the values of the correction factor *k*u. |
| *s*y | is the transverse spacing between the studs in a rib, see Figure G.1; if only one row of centered studs is present, *s*y is 0; |
| *b*top | is the width of the top of the concrete rib (see Figure G.1). |

**Table G.1 — Values of the correction factor *k*u**

|  |  |  |  |
| --- | --- | --- | --- |
| Stud position, Figure G.1 | Profiled sheeting with pre-punched holes | Through-deck welded studs | |
| *t*p < 1,00 mm | *t*p ≥ 1,00 mm |
| Centred or staggered position | 1,0 | 1,0 | 1,25 |

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Key

|  |  |
| --- | --- |
| 1 | centred position () |
| 2 | centred position () |
| 3 | staggered position () |

Figure G.1 — Definition of the shear stud positions

1. (normative)  
     
   Design tension resistance of headed studs
   1. Scope

(1) The tension resistance of headed studs within the scope defined in H.2 have been derived from EN 1992-4 using European Technical Approvals of the most commonly used products.

(2) The tension resistance may be applied for headed studs in solid slabs with a design shear resistance *P*Rd in accordance with 8.6.8.1, and studs in steel sheeting with a design shear resistance *P*Rd in accordance with 8.6.9.

(3) The requirements of EN 1992-4:2018, A.2 should be fulfilled.

(4) The headed studs should be in accordance with EN ISO 13918 and should be welded in accordance with EN ISO 4063:2010, method 783.

* 1. Design tension resistance for headed studs

(1) The design tension resistance, *P*ten,Rd may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.1) |

where:

|  |  |
| --- | --- |
| *N*s,Rd | is the resistance of the steel stud to tension, which may be obtained from (2); |
| *N*p,Rd | is the resistance of the headed stud to concrete pull-out failure, which may be obtained from (3); |
| *N*c,Rd | is the resistance of the headed stud to concrete cone failure, which should be calculated from EN 1992-4:2018, 7.2.1.4, using equations in (4). |

(2) The resistance of the steel stud to tension may be determined from:

|  |  |
| --- | --- |
|  | (H.2) |

where:

|  |  |
| --- | --- |
| *N*s,Rk | is the characteristic tension resistance of the steel stud given by Table H.1; |
| *d* | is the nominal diameter of the shank of the stud; |
| *γ*Ms | is the partial factor for the tension resistance of steel. |

NOTE The value of γMs is 1,55 unless the National Annex gives a different value.

**Table H.1 — Characteristic tension resistance of headed steel studs**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *d* [mm] | 16 | 19 | 22 | 25 |
| *N*s,Rk [kN] | 90 | 128 | 171 | 221 |

(3) The resistance of the headed stud to concrete pull-out failure may be determined from:

|  |  |
| --- | --- |
|  | (H.3) |

where:

|  |  |
| --- | --- |
| *N*p,Rk | is the characteristic resistance of the headed stud to concrete pull-out failure, given by Table H.2; |
| ψp | is the modification factor for the characteristic resistance, given in Table H.3 for the concrete Class; |
| γMp | is the partial factor for pull-out failure. |

NOTE The value of *γ*Mp is 1,5 unless the National Annex gives a different value.

**Table H.2 — Characteristic resistances of headed studs to concrete pull-out failure**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *d* [mm] | 16 | 19 | 22 | 25 |
| *N*p,Rk [kN] | 901) | 75 | 85 | 115 |
| 1) the head diameter is assumed to be 32 mm | | | | |

**Table H.3 — Modification factor *ψ*p**

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Concrete Class | C25/30 | C30/37 | C35/45 | C40/50 | C45/55 | C50/60 |
| *ψ*p | 1,20 | 1,48 | 1,80 | 2,00 | 2,20 | 2,40 |

(4) The resistance of a headed stud to concrete cone failure should be calculated in accordance with EN 1992-4:2018, 7.2.1.4, using the following:

|  |  |
| --- | --- |
|  | (H.4) |

where:

|  |  |
| --- | --- |
| *N*c,R,k | is the characteristic resistance of a headed stud to concrete cone failure:  (H.5) |
|  | is the characteristic resistance of a single headed stud placed in the slab and not influenced by adjacent studs or edges of the slab, which is defined in (5); |
|  | is a ratio taking into account the geometric effect of stud spacing and edge distance on the characteristic resistance *N*c,Rk– see (6); |
|  | is a factor taking account of the effect of a slab edge - see (7); |
|  | is the shell spalling factor - see (8); |
|  | is a factor taking into account the group effect when different tension loads act on the individual fasteners in a group, for concrete cone failure; see (9) and Figure H.1; |
|  |  |
|  | is the partial factor for concrete cone failure. |

NOTE The value of *γ*Mc is 1,5 unless the National Annex gives a different value.

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Figure H.1 — Geometry of concrete cones in composite members

(5) The characteristic resistance  in N should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.6) |

where:

|  |  |
| --- | --- |
| *k*1 | is a factor that take into account the load transfer mechanism; for the tension force acting on the slab at the high moment end of a large web opening, cracked concrete should be considered, with: *k*1 = *k*cr = 8,5; |
| *f*ck | is the characteristic resistance of concrete to compression given in N/mm2; |
| *h*ef | is the effective embedment depth of the stud, in mm, obtained by:  (H.7) |
| *h*sc | is the length after welding of the stud (= *h*n in EN 1992-4); |
| *t*n | is the thickness of the stud head. |

(6) For a transverse row of *n*s regularly spaced headed studs, the ratio *A*c,N/*A*0c,N may be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.8) |

where:

|  |  |
| --- | --- |
| *b*x | = *b*bot for a row of studs located at the centre-line of a transverse rib, where *b*bot is the bottom width of the rib;  = *s*x for studs in a solid slab, where *s*x is the longitudinal spacing between regularly spaced transverse rows of studs. |
| *s*y | is the transverse spacing between studs (Figure H.2). |

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Figure H.2 — Row of regularly spaced headed studs in a transverse rib

(7) The factor  should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.9) |

where:

|  |  |
| --- | --- |
| *c* | is the distance of the headed stud from the closest edge, which may be the bottom edge of the rib in case of a composite slab (see Figure H.3). |
| *c*cr,N | is the characteristic edge distance, given by *c*cr,N = 1,5 *h*ef . |

NOTE For internal composite beams with a solid slab, ψs,N = 1,0.



Figure H.3 — Edge distances defined by the sheeting for headed studs in composite slabs

(8) The shell spalling factor *ψ*re,N applies when *h*ef < 100 mm and accounts for the effect of dense reinforcement between which the studs are installed. It should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.10) |

The factor ψre,N may be taken as 1,0 in the following cases:

1. reinforcement (of any diameter) is present at a spacing not less than 150 mm, or
2. reinforcement with a diameter not greater than 10 mm is present at a spacing not less than 100 mm.

(9) The factor *ψ*ec,N should be determined from:

|  |  |  |
| --- | --- | --- |
|  |  | (H.11) |

where:

|  |  |
| --- | --- |
| *e*N | is the eccentricity of the resultant tension force in a group of connectors with respect to the centre of gravity of the group; |
|  | is the characteristic spacing of studs to ensure the characteristic resistance of the stud in case of concrete cone failure under tension load, which for headed studs may be obtained by: . |

1. (normative)  
     
   Additional rules for shallow floor beams
   1. General

(1) This Annex gives rules for composite shallow floor beams. Unless otherwise stated in this Annex all other parts of this standard also apply.

(2) A composite shallow floor beam consists of a steel section partially-encased in a concrete slab and acting compositely with it. The slab is supported on the bottom flange of the beam. The concrete slab may be in-situ reinforced concrete, precast concrete, or composite. The steel-section may be hot rolled or welded from plates, with an open or a closed cross-section. Typical examples of shallow floor cross-sections are given in Figure I.1.

|  |
| --- |
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| a) Open cross-sections |
| A picture containing sketch, diagram, line, screenshot  Description automatically generated |
| b) Closed cross-sections |

Key

|  |  |
| --- | --- |
| 1 | hollow core |
| 2 | profiled steel sheeting |

NOTE The beams, slabs and reinforcement shown are indicative only.

Figure I.1 — Typical examples of shallow floor cross-sections

(3) This Annex applies to shallow floor beams where all the elements of the steel cross-section are Class 1, Class 2 or effective Class 2 (7.5.2(3) or 7.5.3(3)).

(4) Where the conditions in (3) are not fulfilled more advanced methods in accordance with 8.2.1.5 and 8.6.2 are required.

(5) The design of the concrete slab, including the effect of any interaction between the slab and beam, is outside the scope of this Annex.

NOTE 1 The flexibility of the support offered by a beam to precast slabs can affect the resistance of the slabs.

NOTE 2 Tolerances in erection and fabrication can affect the bearing length of the slab and can affect the design.

(6) Beams with slabs using lightweight concrete are outside the scope of this Annex.

(7) The design rules in this Annex are for beams subjected to static loading only.

(8) The design rules in this Annex are for the beam in its final composite state. Verification for the construction stage, including any torsional loads resulting from the slab construction sequence, should be in accordance with EN 1993-1-1.

(9) Shallow floor beams in accordance with this Annex may be assigned in one of the typologies in Table I.1.

**Table I.1 — Typologies for shallow floor beams (examples of cross-section)**

|  |
| --- |
| **Type I** |
| **A diagram of a table  Description automatically generated** |
| Shallow floor sections not classified into Type II or III where the local flexural stiffness of the concrete flange is not significant when compared to the total flexural stiffness of the composite beam. |
| **Type II** |
| **A diagram of a table  Description automatically generated** |
| Shallow floor sections not classified into Type III where the local flexural stiffness of the concrete flange is significant when compared to the total flexural stiffness of the composite beam. |
| **Type III** |
| **A grey rectangular object with black trim  Description automatically generated** |
| Shallow floor sections where the local flexural stiffness of the concrete flange is significant when compared to the total flexural stiffness of the composite beam: where ha < 0,6 h. |

* 1. Structural Analysis
     1. Global analysis for buildings

(1) When applying rigid plastic global analysis and the location of any plastic hinge occurs in a shallow floor beam, the cross-section of the shallow floor beam shall be in Class 1. In addition, sufficient rotation capacity at the location of the plastic hinge shall be verified.

NOTE The section classification is of the steel elements alone, and does not guarantee the rotation capacity of the composite section.

(2) When the verification in accordance with (1) is not provided, the calculation of moments in continuous beams in braced structures fulfilling the requirements of 7.4.4(4) should take into account the stiffness over the beam length considering non-linear material behaviour for steel and concrete and the flexibility of the shear connectors when significant.

* For Type I sections (see Table I.1) the condition in 7.4.4(6) Class 2 applies to all steel grades where it can be demonstrated that the cross-section reaches the plastic moment resistance for sagging and hogging. In this case the tension reinforcement in the span direction of the beam should fulfill the requirements of Class B reinforcement in accordance with EN 1992-1-1:2023, 5.2.
* For Type II and Type III sections (see Table I.1) the redistribution of the moment should be determined in accordance with EN 1992-1-1:2023, 7.3.2(3), although the redistribution should not exceed the values provided in 7.4.4(6) for Class 3 sections.

(3) The effective width *b*eff of the concrete flange for the determination of the action effects may be taken in accordance with 7.4.1.2.

* + 1. Classification of shallow floor beam cross-sections

(1) A compression flange may be assumed to be Class 2, when it is covered by reinforced structural concrete and the cover above the steel top flange is greater than 50 mm and greater than one-sixth of the flange width (*b*f/6), see Figure I.2a and I.2d. The minimum cover to any reinforcement should be in accordance with EN 1992-1-1:2023, 6.4.

(2) Open sections with concrete encasement between flange outstands and extending the full width of the outstand, may be classified in accordance with 7.5.3. If the section is encased on both sides of the web, then the web may be assumed to be Class 2 (see Figure I.2b). If the concrete encasement is on one side only, and the concrete between the flanges is fixed to the web in accordance with 7.5.3(2), then the web may be assumed to be in Class 2.

(3) When the encasement conditions in (2) are not met, the elements of the cross-section should be classified in accordance with EN 1993-1-1:2022, Table 7.3.

(4) For open sections in hogging bending, where the compression flange comprises a plate welded to the flange of a rolled profile (see Figure I.2 b), the outstands of the rolled profile may be classified in accordance with (2). The internal and outstand elements of the plate should be classified in accordance with EN 1993-1-1:2022, Table 7.3.

(5) For closed sections without concrete infill (see Figure I.2 c), the webs, internal parts of the flanges and flange outstands may be classified in accordance with EN 1993-1-1:2022, Table 7.3.

(6) The outstand elements of closed sections with concrete infill (see Figure I.2 d) may be classified in accordance with 7.5.3.

|  |  |
| --- | --- |
| Diagram of a diagram of a building  Description automatically generated with medium confidence | A diagram of a house  Description automatically generated |
| a) Example of a Class 2 compression flange (compression positive) | b) Example of a Class 2 web |
| A diagram of a beam  Description automatically generated | A diagram of a beam  Description automatically generated |
| c) Closed section without concrete infill | d) Closed section with concrete infill |

Key

|  |  |
| --- | --- |
| 1 | Class 2 compression flange |
| 2 |  |
| 3 | Class 2 web |
| 4 | Classification in accordance with EN 1993-1-1:2022, Table 7.3 |
| 5 | No concrete infill |
| 6 | Class 2 flange if the concrete cover complies with (1) otherwise Classification in accordance with Table I.2 |
| 7 | Hollow core |
| 8 | Profiled steel sheeting |
| 9 | Concrete infill |

Figure I.2 — Cross-section classification of composite shallow floor beams

(7) The webs of closed sections infilled with concrete may be classified in accordance with Table. I.1.

**Table I.2 — Classification limits for internal elements of sections with concrete infill**

|  |  |
| --- | --- |
| Class | Limit |
| 2 |  |
| 3 |  |

* 1. Ultimate limit states
     1. Bending Resistance
        1. General

(1) The rules in 8.2.1, or, when the reinforcement in tension between the flanges is considered, in 8.3.1 , should be used to calculate the bending resistance taking into account the effect of transverse bending and torsional effects as set out in I.3.1.2 and I.3.1.3 respectively.

* + - 1. Effect of transverse bending

(1) Local effects on the composite shallow floor cross-section (e.g. transverse bending in the plate supporting the slab) shall be taken into account when determining the resistance in bending.

(2) The interaction between longitudinal stresses and stresses due to transverse bending in the bottom flange or plate may be taken into account by using a reduced yield strength *f*y,red for the bottom flange or plate:

|  |  |
| --- | --- |
|  | (I.1) |

where:

|  |  |
| --- | --- |
|  | is the ratio *m*ybt,Ed / *m*ybt,Rd; |
| *m*ybt,Ed | is the maximum transverse moment per unit length; |
| *m*ybt,Rd | is the bending resistance of the plate taken as 1,2 times the elastic resistance. |

* + - 1. Torsional effects

(1) Torsional effects shall be taken into account in the design of edge beams.

(2) Torsional resistance and stiffness should be provided e.g. by torsional anchorage (Figure I.3 b). The adequacy of the anchorage should be assured.

NOTE Types of torsional anchorage other than those presented in Figure I.3 b can also be used.

|  |  |
| --- | --- |
| A black and white image of a shield  Description automatically generated | A diagram of a rectangular object  Description automatically generated |
| a) Beam with a closed cross-section | b) Torsional anchoring of an edge beam with an open cross-section |

Key

|  |  |
| --- | --- |
| 1 | Anchor bolt |

Figure I.3 — Typical examples of shallow cross-sections suitable for edge beams

* + 1. Resistance to vertical shear

(1) The plastic resistance to vertical shear *V*pl,Rd of a composite shallow floor beam of Type I (see Table I.1) with a solid web may be taken as the resistance *V*pl,a,Rd of the structural steel section calculated in accordance with EN 1993-1-1:2022, 8.2.6 or in accordance with 8.3.3(1) and (2). For the contribution of concrete to the shear resistance, the need for stirrups and/or transfer of forces should be taken into account.

(2) For the resistance to vertical shear *V*pl,Rd of a composite shallow floor beam of Type II and Type III (see Table I.1) the possible contribution of the concrete to the shear resistance should be considered according to (3) to (5).

(3) The contribution of concrete to the shear resistance for Type II and Type III section should be considered according to EN 1992-1-1 and may be calculated in accordance with 8.3.3 taking account of the need for stirrups and/or transfer of forces. For the determination of the design shear resistance of the concrete slab in accordance with EN 1992-1-1:2023, 8.2, the width *bw* may be determined from:

|  |  |
| --- | --- |
|  | (I.2) |

where:

|  |  |
| --- | --- |
| *b*0 | is the width of the bottom plate of the steel section; |
| *d* | is the distance between the centroid of the slab reinforcement in tension and the outer fibre of concrete in compression. |

NOTE For the determination of vertical shear resistance of the concrete contribution in accordance with EN 1992-1-1 the inner lever arm *z* can be determined as the lever arm between the centroid of the longitudinal reinforcement of the slab parallel to the beam and the centroid of the concrete compression zone.

(4) In order to prevent punching failure or failure of the concrete slab under vertical shear, for shallow floor beams of Type II and Type III where the main load transfer at the support is realised by steel joints, unless advanced calculations are carried out, at least 60% of the overall vertical shear load should be considered to be transferred by the concrete slab for Type II and 100% for Type III. The overall vertical shear load should not exceed the resistance *V*pl,a,Rd of the structural steel section.

NOTE For composite shallow floor section of Type III specific rules for the distribution of the vertical shear forces are given in CEN/TS 1994-1-102.

(5) If the load transfer at the support is partly directly into the concrete slab and this load transfer is verified according to EN 1992-1-1, the vertical shear load verified against the shear resistance *V*pl,a,Rd of the structural steel section may be reduced accordingly.

(6) The verification of a composite shallow floor beam for bending and vertical shear should be carried out in accordance with 8.2.2.5 or 8.3.4.

* + 1. Shear connection
       1. General

(1) Examples for typical shear connections for shallow floor beams are given in Figure I.4.

(2) The resistance of headed studs should be calculated in accordance with 8.6.

NOTE For situation shown on the left of Figure I.4 where horizontal headed stud connectors on the web do not project beyond the flanges, splitting forces need not be considered.

(3) The resistance of transverse bars should be calculated in accordance with I.3.3.3.

(4) Requirements for the determination of the longitudinal shear forces and the arrangements of the shear connectors are given in Clause 8.6.

* + - 1. Partial shear connection

(1) Partial shear connection may be used in accordance with 8.6.3 when all conditions provided by 8.6.3 are fulfilled.

A picture containing diagram, line, sketch, white

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | Headed studs |
| 2 | Transverse bar |

Figure I.4 — Examples of typical shear connectors for composite shallow floor beams

(2) Limits to the use of partial shear connection are given in 8.6.3.2 and 8.6.3.3. For steel sections of Type I and II having a bottom flange with an area larger than three times that of the top flange area, the ratio *η* = *n* / *n*f should satisfy the following condition:

|  |  |
| --- | --- |
| for *L* > 18m: | (I.3) |

NOTE For hybrid steel sections (steel sections composed of different steel grades) the value *f*y of the highest steel grade is to be used.

(2) Alternatively the slip capacity of the shear connector may be verified by advanced calculation in accordance with 8.6.2 or testing.

* + - 1. Transverse bars

(1) Transverse reinforcement bars of Class B or Class C (see EN 1992-1-1:2023, Table C.2) used as shear connectors should have a diameter not less than 12 mm and not greater than 20 mm. Unless results from push-tests are available the connectors should be considered as Ductility Category D1.

NOTE Classification of the shear connector into the Ductility Category can be provided in accordance with the requirements given in Table 5.1.

(2) The centre-to-centre spacing of transverse bars should be not less than 125 mm.

(3) The design resistance per shear plane *P*Rd of a transverse bar should be taken as its plastic shear resistance:

|  |  |
| --- | --- |
|  | (I.4) |

where:

|  |  |
| --- | --- |
|  | is the diameter of the transverse bar, |
| *f*sk | is the yield strength of the transverse bar, |
|  | is the partial factor – see 4.4.1.2(5). |

(4) As an alternatively to (3) the design resistance may be obtained by testing (in accordance with Annex B) or by an advanced method.

(5) Transverse bars acting as shear connectors in accordance with (3) shall not be used for crack control or as slab reinforcement, e.g. to transfer hogging moments. They may be used to resist longitudinal shear in the composite beam.

NOTE Additional reinforcement above the upper flange of the steel section can be necessary for crack control or as shear reinforcement, see reinforcement *A*t in Figure I.5.

(6) Web-openings to accommodate the transverse bars should have a diameter at least two times the diameter of the transverse bar.

(7) Transverse bars should be adequately anchored in the concrete slab beyond potential surfaces of shear failure, as shown in Figure 8.22 and Figure I.5 of this document.

(8) Longitudinal shear in concrete slabs supported on composite shallow floor beams should be verified in accordance with 8.6.11. In addition to the potential surfaces for shear failure given in Figure 8.22 shear plane e-e illustrated in Figure I.5 should be verified.

|  |  |
| --- | --- |
| A diagram of a beam  Description automatically generated | A diagram of a beam  Description automatically generated |
| a) Transverse bar, Ab crossing failure plane | b) Shear reinforcement, Ab + Ab\* crossing failure plane |

Figure I.5 — Additional potential surfaces for shear failure

* 1. Serviceability limit states

(1) In addition to Clause 9, the stress at any point in the steel section, under the characteristic load combination, taking into account the effects of transverse bending, should not exceed the material yield strength.

(2) The deformation of shallow floor beams Type I (see Table I.1) may be determined in accordance with 9.3.1.

NOTE Modular ratio method can be used for the determination of the time dependent cross-section moments of area, similar to partial encased composite beams thereby the effects of concrete cracking are to be considered.

(3) For single span composite shallow floor beams of Type II and III (see Table I.1) the flexural stiffness of the concrete slab should be considered for the determination of deformation. Therefore, the impact of concrete cracking on the flexural stiffness should be considered.

In the absence of more detailed analysis, the second moment of area for the composite shallow floor section, may be taken into account as given in Formula (I.5) considering the effective width of the bending state of the concrete slab for composite shallow floor beams where the the following conditions are fulfilled:

* span length of the composite shallow floor beam 4,0 m ≤ *L* ≤ 9,0 m;
* concrete slab width 2,0 m ≤ *b*c ≤ 8,0 m;
* concrete slab thickness 16 cm ≤ *h*c ≤ 34 cm; and
* amount of reinforcement in the slab for each top and bottom layer 0 ≤ *a*s ≤ 18cm2/m.

|  |  |
| --- | --- |
| *I*i,eff,L = *I*a + *α*c ⋅ (*I*c,L + *S*i,L ⋅ *a*St) | (I. 5) |

where

|  |  |
| --- | --- |
| αc = αV ⋅ αMat ⋅ αQS ⋅ αM | (I.6) |
| for Type II: αV = 0,4 | (I.7) |
| for Type III: αV = 3.7    for Type II and Type III: | (I.8) |
|  | (I.9) |
|  | (I.10) |

where

|  |  |
| --- | --- |
| *f*ck | characteristic concrete compressive strength; |
| *f*c,0 | reference value for *f*ck (30 N/mm2); |
| *fct,k* | characteristic concrete tensile strength; |
| *fct,0* | reference value for *f*ct,k (1,0 N/mm2); |
| *fyk* | characteristic yield strength of structural steel; |
| *f*y,0 | reference value for *f*yk (355 N/mm2); |
| *As,u* | area of reinforced in tensile zone; |
| *Aa* | area of structural steel section; |
| *L* | subscript in accordance with 7.4.2.2(2); |
| *A*I,L | time dependant area of composite section taking into account the modular ratio; |
| *I*c,L | time dependant second moment of area for the concrete section of width *b*c taking into account the modular ratio; |
| *I*I,L | time dependant second moment of area for the concrete section taking into account the modular ratio; |
| *S*i,L | time dependant static moment of composite section taking into account the modular ratio; |
| *a*st | distance between centroids of concrete and steel section; |
| *h*c | height of concrete slab; |
| *b*c | full geometric width of the concrete section; |
| *L* | beam length; |
| *M*crack | moment at beginning of cracks under tension, i.e. elastic bending moment of composite section when reaching *f*ct,k at the concrete tension fibre considering the second moment of area *I*i,0 for short-term loading; |
| *M*Ek | total characteristic moment in serviceability limit state. |

1. (informative)  
     
   Other flooring types using precast concrete slabs in buildings
   1. Scope

(1) This Annex gives additional rules for the use of precast concrete slabs in composite beams. Unless otherwise stated in this Annex all other parts of this standard also apply.

(2) Typical types of cross-section are shown in Figure J.1 with either a hollow core slab, composite hollow core slab or a solid composite slab. The rules may also be applied to beams with solid precast concrete slabs.

(3) The scope of the rules for composite beams with precast slabs is limited to uniform steel members of doubly symmetrical or singly symmetrical cross-section. The steel members may be hot-rolled, cold-formed or formed from welded plates.

(4) The rules are limited to beams in buildings that are designed as simply-supported.

(5) The rules are limited to slabs with normal weight concrete of Class C20/25 to C60/75.

(6) The rules are limited to precast concrete hollow core slabs manufactured in accordance with EN 1168, and solid composite slabs with or without lattice girders in accordance with EN 13747.

(7) The design of the concrete slab, including the effect of any interaction between the slab and beam on the design of the concrete, is outside the scope of this Annex.

NOTE 1 The flexibility of the support offered by the beam to precast slabs can affect the resistance of the slab. For prestressed hollow core slabs, see EN 1168.

NOTE 2 Composite interaction with the steel beam induces stresses in the precast slabs.

NOTE 3 Deviations in the position and dimension of the steel beam can affect the bearing length of the precast slabs.

(8) The rules are limited to beams subjected to static loading only.

(9) The rules cover design of the beam in its final composite state. Design for the construction stage, including allowance for any torsional loads resulting from the construction sequence, should be in accordance with EN 1993-1-1.

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| |  |  | | --- | --- | |  |  | | a) Composite beams with square ended hollow core slabs at filled core | b) Composite beams with chamfered ended hollow core slabs at filled core | |  |  | | c) Composite beams with solid composite slabs | d) Composite Shallow Floor beams. | |

Figure J.1 — Typical cross-sections of composite beams with precast slabs

* 1. Materials
     1. Precast concrete slabs in buildings

(1) Properties should be obtained by reference to EN 1168 and EN 13747.

* 1. Structural analysis
     1. Effective width of flanges using precast concrete slabs.

(1) For floors with hollow core slabs or composite hollow core slabs, the tops of alternate cores in each unit should be broken out and infilled with in-situ concrete. The infill should have embedded transverse reinforcement which should extend to the end of the infill length. The effective width of the flange may be determined using 7.4.1.2, but for ULS should not be taken as more than the sum of the gap between the slabs (*b*g) and the infill lengths (*L*f) on either side of the beam (See Figure J.2). The depth of the concrete flange should only include the depth of joint where compression transfer is possible (this depth is the dimension (*h* – *h*j) in Figure J.3).

(2) For floors with hollow core slabs, the effective width of the flange at the serviceability limit state may be determined using 7.4.1.2.

(3) For floors with solid composite slabs, the effective width of the flange may be determined using 7.4.1.2. The depth of the concrete flange should only include the depth of joint where compression transfer is possible (this depth is the dimension (*h* – *h*j) in Figure J.3).

|  |  |
| --- | --- |
| A diagram of a floor  Description automatically generated | |
| a) Cross-section at filled core | |
| A diagram of a concrete slab  Description automatically generated | A diagram of a graph  Description automatically generated with medium confidence |
| b) Cross-section at unfilled core | c) Longitudinal joint between hollow core slabs |
|  | |

Key

|  |  |
| --- | --- |
| 1 | Topping when composite hollow core slabs are used |
| 2 | Square end |
| 3 | Chamfered end |

Figure J.2 — Typical cross-sections of composite beams with precast hollow core slabs, showing a filled core with embedded transverse reinforcement and an unfilled core

* 1. Ultimate limit states

(1) Composite beams with precast concrete slabs should be verified for:

* moment resistance of critical cross-sections (see 8.2 and 8.3);
* resistance to lateral-torsional buckling (see 8.4);
* resistance to transverse forces on webs (see 8.5);
* resistance to longitudinal shear (see 8.6).

(2) For a composite hollow core slab or a solid composite slabs, the in-situ topping should be at least 40 mm thick.

(3) The strength of the concrete slab should be taken as the strength of the in-situ infill concrete.

* 1. Design resistance of shear connectors used with precast floors in buildings
     1. General

(1) The shear connection should satisfy the requirements given in 8.6.1.

(2) For headed stud shear connectors the design resistance should be determined in accordance with J.5.2 or J.5.3. When the minimum dimensions given in J.5.2 or J.5.3 are not satisfied, the design shear resistance and characteristic slip capacity of the headed studs should be evaluated from specific tests in accordance with J.8.

(3) For non-preloaded bolts used as shear connectors, the design resistance should be determined in accordance with J.5.4. When the requirements given in J.5.2 or J.5.3 are not satisfied, the design shear resistance and characteristic slip capacity of the shear connectors should be evaluated from specific tests in accordance with J.8.

(4) For shear connectors other than headed studs or non-preloaded bolts, the design resistance and characteristic slip capacity should be evaluated from specific tests in accordance with J.8.

(5) The gap between the precast slabs should be such that there is adequate space for the concrete to be compacted adjacent to the shear connectors. The as-built gap should be not less than the stud diameter plus twice the aggregate size (*D*upper) plus 5 mm. Where 19 mm diameter shear studs are used, a minimum gap of 65 mm may be used for studs installed before the slabs. The nominal gap should include an allowance for construction deviations.

(6) The gap between the precast slabs should be such that if the shear connectors are placed after the slabs there is adequate space for installation of the connectors. An allowance for construction deviations should be added to these values.

* + 1. Headed studs in solid composite slabs

(1) The nominal thickness of the precast floor elements *h*pc should not exceed 100 mm and the transverse reinforcement bars should be positioned in accordance with 8.6.10.1(1).

(2) The diameter of the shank of the headed stud d should be not less than 16 mm, or greater than 25 mm. The connectors should be provided in accordance with 8.6.10.7.

(3) The design shear resistance of headed stud connectors should be taken as the resistance in a solid slab, see 8.6.8.1. Classification into ductility category D2 may be assumed in accordance with Table 5.1.

|  |  |
| --- | --- |
| A diagram of a beam  Description automatically generated | A diagram of a metal beam  Description automatically generated with medium confidence |

Key

|  |  |
| --- | --- |
| 1 | Longitudinal joint between precast floor plates |

Figure J.3 — Detailing dimensions for solid composite slabs

* + 1. Headed studs in precast hollow core slabs

(1) The nominal thickness of the hollow core slab excluding topping *h*pc should be not greater than 265 mm.

(2) The ends of the hollow core slab may be square or chamfered. For hollow core slabs with chamfered ends, the depth of the chamfer *a*h should not exceed 85 mm and the breadth of the chamfer *a*b should not exceed 235 mm (see Figure J.4).

(3) The diameter of the shank of the headed stud d should be not less than 19 mm, or greater than 22 mm. The headed stud connectors should be provided in accordance with 8.6.10.7. When the diameter of the transverse reinforcement is at least 12 mm, the shear connector may be assumed to be Ductility Category D2 in accordance with Table 5.1.

(4) The design shear resistance of headed stud connectors should be taken as the resistance in a solid slab, calculated as given by 8.6.8.1, multiplied by the reduction factor *k* given by:

|  |  |
| --- | --- |
|  | (J.1) |

with

|  |  |
| --- | --- |
|  | for |
|  | for |

where *b*g is the nominal distance between the ends of the hollow core slabs (mm) and  is the diameter of the transverse reinforcing bars (mm).

(5) Open cores should be provided to receive transverse reinforcing bars, which should be positioned below the heads of the studs in accordance with 8.6.10.1 (see Figure J.5).

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Description automatically generated

Figure J.4 — Detailing dimensions for precast hollow core slabs

|  |
| --- |
| A black and white image of an oval  Description automatically generated |
| a) Longitudinal view of filled cores with embedded transverse reinforcement and chamfered ended hollow core slabs |
| A grey rectangular object with black letters  Description automatically generated |
| b) Longitudinal view of shear connectors |

Key

|  |  |
| --- | --- |
| 1 | Transverse reinforcement |

Figure J.5 — Longitudinal dimensions for precast hollow core slabs

* + 1. Non-preloaded bolts
       1. General

(1) Non-preloaded bolts with embedded nuts and placed in holes with nominal clearances not exceeding those for normal holes as specified in EN 1090-2 may be used as shear connectors (see Figure J.6).

(2) Bolts conforming to the requirements given in EN 14399 (all parts) may be used, provided that any preloading is only applied within the grip length between the embedded and outer nut (see Figure J.6) and that the preloading force within the grip length is not higher than *F*p,C in accordance with FprEN 1993-1-8:2023, 5.9.1.

A drawing of bolts and nuts

Description automatically generated

**Key**

|  |  |
| --- | --- |
| 1 | Grip length |

Figure J.6 — Dimensions of shear connection with non-preloaded bolts

(3) Design shear resistance and characteristic slip capacity in accordance with J.5.4.2 and J.5.4.3 are valid for bolt diameters not less than 12 mm and not greater than 24 mm, Class 8.8 or 10.9, in accordance with EN 1993-1-8, with one or two embedded nuts.

(4) The spacing of non-preloaded bolts in the direction of the shear force should comply with 8.6.10.7 (4). The spacing in the direction transverse to the shear force should not be less than 5 *d*.

(5) Shear connectors based on non-preloaded bolts with embedded nuts and placed in holes can be classified into Ductility Category D2 if the slip capacity δuk in accordance with Formula (J.6) is at least 6 mm.

* + - 1. Design shear resistance

(1) The design shear resistance of a non-preloaded bolt may be taken as the lesser of:

|  |  |
| --- | --- |
|  | (J.2) |

or

|  |  |
| --- | --- |
|  | (J.3) |

with:

|  |  |
| --- | --- |
|  | (J.4) |
|  | (J.5) |

where:

|  |  |
| --- | --- |
| *P*b,Rd | is the design value of shear resistance based on bolt failure [N]; |
| *P*c,Rd | is the design value of shear resistance based on concrete failure [N]; |
| *d* | is the bolt diameter in mm, 12 mm ≤ *d* ≤ 24 mm; |
| *A*s | is the tensile stress area of the bolt in mm2; |
| *h*sc | is the overall nominal height of the bolt above the flange in mm; |
| *f*ub | is the ultimate tensile strength in N/mm2; |
| *f*ck | is the characteristic cylinder compressive strength of the concrete in N/mm2, that is cast in-situ around the shear connector; |
| *γ*V | is the partial factor in accordance with 4.4.1.2(5). |

* + - 1. Slip capacity

(1) The characteristic slip capacity (mm) for non-preloaded bolts may be determined from:

|  |  |
| --- | --- |
|  | (J.6) |

* 1. Longitudinal shear in precast concrete slabs

(1) The design longitudinal shear and the shear resistance should be determined in accordance with 8.6.11. Figure J.7 shows potential surfaces of shear failure.

(2) For precast concrete floors using hollow core slabs or composite hollow core slabs, the transverse reinforcement should be placed in each open core and between adjacent units if necessary to comply with the minimum bar spacing. The centre-to-centre bar spacing should be not more than twice the core spacing (see Figure J.5). The transverse reinforcement should be provided in accordance with 8.6.11 and fully anchored, with the length of the concrete infill *L*f not being less than 500 mm (see Figure J.2). Ends of cores without transverse reinforcement bars should be filled with concrete to a nominal distance equal to at least the depth of the cores *h*core (see Figure J.2).

A diagram of a beam

Description automatically generated

Figure J.7 — Typical potential surfaces of shear failure where precast slabs are used

* 1. Detailing of the precast slabs
     1. Support arrangements

(1) The precast floor elements may be designed as simply-supported. The connection between opposing elements at a support should be designed and detailed accordingly.

(2) The bearing length should allow correct positioning of the precast elements, taking into account deviations in geometry of the precast elements and the supporting steel beams. The nominal bearing length *L*bc as indicated in Figure J.3, Figure J.4 and Figure J.5 should not be less than 55 mm.

* + 1. Edge beams

(1) The local reinforcement requirements given in 8.6.10.3 should be satisfied. For precast flooring using hollow core slabs, the local reinforcement should be positioned in accordance with Figure J.8.

A diagram of a beam

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | Filled core |
| 2 | U-bar |

Figure J.8 — Detailing of local reinforcement for hollow core slabs on edge beams

* 1. Tests on shear connectors
     1. Specific push tests for headed studs in hollow core slabs or composite hollow core slabs of uniform thickness

(1) This subclause gives rules that supplement those in Annex B for tests on headed studs in hollow core slabs, or composite hollow core slabs of uniform thickness.

(2) Specific push tests should be carried out with slabs and reinforcement suitably dimensioned in comparison with the beams for which the test is designed. In particular:

1. the width *b*e of each hollow core element should be related to the longitudinal spacing of the connectors *s* in the beam (e.g. see Figure J.5);
2. the lengths of the hollow core elements should be such that the width of the specimen *b*c is equal to the nominal distance between the ends of the hollow core elements, plus the length *L*f of the in-situ infill to either side of the beam;
3. the thickness *h* of each precast element should not exceed the minimum thickness of the slab in the beam for which the test is designed;
4. the diameter of the transverse reinforcement bars *φ* and their spacing *s*f should be the same as in the beam;
5. when the ends of the hollow core elements are chamfered, the breadth and depth of the chamfer should be *a*b and *a*h, respectively.

Bibliography

**References contained in recommendations (i.e. “should” clauses)**

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative documents could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1090-1, *Execution of steel structures and aluminium structures – Part 1: Requirements for conformity assessment of structural components*

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

EN 1090-4*, Execution of steel structures and aluminium structures - Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications*

EN 1168, *Precast concrete products - Hollow core slabs*

FprEN 1991-1-1, *Eurocode 1 — Actions on structures — Part 1-1: Specific weight of materials, self-weight of construction works and imposed loads on buildings*

prEN 1991-1-6:2024, *Eurocode 1 — Actions on structures — Part 1-6: Actions during execution*

EN 1992-4:2018, *Eurocode 2 — Design of concrete structures — Part 4: Design of fastenings for use in concrete*

EN 1993 (all parts), *Eurocode 3 — Design of steel structures*

FprEN 1993‑1‑3:2023, *Eurocode 3 — Design of steel structures — Part 1-3: Cold‑formed members and sheeting buildings*

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prEN 1994-2:2024, *Eurocode 4 — Design of composite steel and concrete structures — Part 2: Bridges*

EN 1997 (all parts), *Eurocode 7 — Geotechnical design*

EN 1998 (all parts), *Eurocode 8 — Design of structures for earthquake resistance*

EN ISO 4063:2010, *Welding and allied processes — Nomenclature of processes and reference numbers*

EN 10025 *(all parts), Hot rolled products of structural steels*

EN 10149-2, *Hot-rolled flat products made of high yield strength steels for cold forming - Part 2: technical delivery conditions for thermomechanically rolled steels*

EN 10149-3, *Hot-rolled flat products made of high yield strength steels for cold forming - Part 3: technical delivery conditions for normalized or normalized rolled steels*

EN 10169, *Continuously organic coated (coil coated) steel flat products – Technical delivery conditions*

EN 10346, *Continuously hot-dip coated steel flat products for cold forming — Technical delivery conditions*

EN 13670, *Execution of concrete structures*

EN 13747, *Precast concrete products - Floor plates for floor systems*

EN ISO 13918:2018, *Welding Studs and ceramic ferrules for arc stud welding*

EN ISO 14555, *Welding — Arc stud welding of metallic materials*

**References contained in permissions (i.e. “may” clauses)**

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated refer-ences, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 14399 *(all parts), High-strength structural bolting assemblies for preloading*

**References contained in possibilities (i.e. “can” clauses) and notes**

EN 1337-1, *Structural bearings — Part 1: General*

prCEN/TS 1994-1-102, *Eurocode 4 — Design of composite steel and concrete structures – Part 1-102: Composite Dowels*

1. As impacted by EN 1990:2023/prA1:2024. [↑](#footnote-ref-2)