EBCS-4

Fekadu wolde

Ethiopian Building Code Standard

DESIGN OF COMPOSITE STEEL AND CONCRETE STRUCTURES

Ministry of Works & Urban Development
Addis Ababa, Ethiopia
1995

FOREWORD

The Proclamation to define the powers and duties of the Central and Regional Executive Organs of the Transitional Government of Ethiopia No. 41/1993 empowers the Ministry of Works and Urban Development to prepare the Country's Building Code, issue Standards for design and construction works, and follow up and supervise the implementation of same.

In exercise of these powers and in discharge of its responsibility, the Ministry is issuing a series of **Building Code Standards** of general application.

The purpose of these standards is to serve as nationally recognized documents, the application of which is deemed to ensure compliance of buildings with the minimum requirements for design, construction and quality of materials set down by the **National Building Code**.

The major benefits to be gained in applying these standards are the harmonization of professional practice and the ensuring of appropriate levels of safety, health and economy with due consideration of the objective conditions and needs of the country.

As these standards are technical documents which, by their very nature, require periodic updating, revised editions will be issued by the Ministry from time to time as appropriate.

The Ministry welcomes comments and suggestions on all aspect of the Ethiopian Building Code Standards. All feedback received will be carefully reviewed by professional experts in the field of building construction with a view to possible incorporation of amendments in future editions.

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Minister
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1995

EBCS-4
Design of Composite Steel and Concrete Structures

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CHAPTER 1
INTRODUCTION

1.1 SCOPE

(1) This Ethiopian Building Code Standard EBCS 4 "Design of Composite Steel and Concrete Structures" applies to the design of composite structures and members for buildings and civil engineering works. The composite structures and members are made of structural steel and reinforced or prestressed concrete connected together to resist loads.

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

(3) Execution is covered in Chapter 8, and by reference to EBCS 2 and EBCS 3 to the extent that it is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Generally, the rules related to execution and workmanship are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works and methods of construction.

(4) This Code does not cover the special requirements of seismic design. Provisions related to such requirements are provided in EBCS 8 "Design of Structures for Earthquake Resistance" which complements or adapts the rules of EBCS 4 specifically for this purpose.

(5) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in EBCS 4. They are given in EBCS 1 - Basis of " Design and Actions on Structures" applicable to the various types of construction.

1.2 ASSUMPTIONS

(1) The assumptions given in EBCS 2 and EBCS 3 are applicable.

(2) The design procedures are valid only when the requirements for execution and workmanship given in Chapter 8 are also complied with.

1.3 UNITS

(1) S.I. Units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

(a) forces and loads	:	kN, kN/m, kN/m^2
(b) units mass	:	kg/m^3
(c) unit weight	:	kN/m^3
(d) stresses and strengths	:	MPa, GPa (MN/m^2, N/mm^2, kN/mm^2)
(e) moments	:	kNm.

1.4 SYMBOLS

(1) The symbols used in this Code are as follows:

A	Accident action
	Area of the equivalent composite section
A_a	The area of the structural steel section
A_c	Effective area of concrete
A_{cv}	Mean cross sectional area per unit length of beam of the concrete shear surface under consideration
A_d	Design value (specified value) of the accidental actions
A_e	Cross sectional area of reinforcement bar
A'_e	Sum of the cross sectional areas of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the shear surface under consideration including any reinforcement provided for bending of the slab
A_{f1}	Area of the front surface of a block connector
A_{f2}	Area of the front surface of a block connector enlarged at a slope of 1:5 to the rear surface of the adjacent connector
A_h	Cross sectional area of the anchor or the hoop
A_p	Cross sectional area of the profiled steel sheeting per unit length of the beam
A_s	Effective area of longitudinal slab reinforcement
A_{se}	Area of any longitudinal reinforcement in compression that is included in the calculation of the bending resistance
A_{sn}	The sum of the areas of the reinforcing bars within the depth of $2h_n$
A_{sni}	The areas of reinforcing bars within $2h_n$
a	Area of structural steel
	Area of the structural steel
	Center to center spacing between the steel beams
a_d	Geometrical design value
b	Breadth of the top flange of the steel member
	Length of an angle connector
	Width of flange of the steel member
b_c	Breadth of concrete
C_d	Design capacity for the effect of actions
c_a	Factor based on the property of the bending moment within the length L
D_d	Damage indication
d	Diameter of the studs
E_a	Modulus of elasticity for steel
E'_c	Effective modulus of concrete for long term effects
E_{cm}	Secant modulus of concrete
E_{cm}	Mean value of the secant modulus of the concrete
E_d	Design value of the particular effect of actions being considered
E_a, E_s	Elastic moduli for the structural steel and the reinforcement
$E_{d,dst}$	Design effect of the destabilizing actions
$E_{d,stb}$	Desjgn effect of the stabilizing actions
E_i	Stiffness moduli for the relevant areas

e	Eccentricity of the loading
e_{ct}	Additional eccentricity of the permanent normal force
e_i	Distance of the reinforcement bars to the relevance middle line
e_{el}	Elastic centroidal axis for short term loading
$e_{el,t}$	Elastic centroidal axis for long term loading
e_{st}	Distance from the middle line
F_c	Compressive force in the concrete flange necessary to resist the design sagging bending Moment M_{Sd}, calculated from plastic theory
F_d	Design value
F_{eQ}	Compressive force in the concrete slab at moment $M_{eQ,Rd}$
F_k	Characteristic values
F_Q	Design longitudinal force caused by composite action in the beam
F_{ser}	Service values
F_t	Design transverse force caused by composite action in the slab
F_{ten}	Design tensile force per stud
f_{ck}	Characteristic compressive strength of concrete
f_i	Design strengths of the materials for the areas
f_{sk}, f_{ck}	Characteristic strengths for the reinforcement and the concrete
f_{sk}	Characteristic tensile yield strength of reinforcement
f_u	Tensile strength of the studs
f_u	Specified ultimate tensile strength of the material of a stud, a bolt, a rivet
f_y	Nominal tensile yield strength of structural steel Yield strength of the structural steel
f_{yd}, f_{sd}, f_{cd}	Design strengths for the structural steel, the reinforcement, and the concrete
f_{yp}	Characteristic (nominal) tensile yield strength of profiled steel sheeting
G	Shear modulus for steel
G	Permanent action
G_d	Design permanent action
G_{ind}	Indirect permanent action
G_k	Characteristic permanent action
$G_{k,j}$	Characteristic values of the permanent actions
h	Greater overall dimension of the section parallel to a principal axis Height of the upstanding leg of an angle connector overall height of the stud Height of column or storey height Overall depth of the steel member
h_c	Depth of concrete
h_E	Depth to the additional point E
h_n	Depth of the neutral axis in the web
h_o	Overall height of structure
h_s	Distance center to center of flanges Distance between the shear center of the flanges of the steel member Distance to the neutral axis
I_a, I_c, I_s	Second moments of area for the considered bending plane of the structural steel, the concrete (assumed to be uncracked) and the reinforcement, respectively
I_{afz}	Second moment of area of the bottom flange about the minor axis of the steel member
I_{at}	St. Venant torsion constant o the steel section
I_{ay}, I_{az}	Second moment of area of the structural steel section about its center of area, C

I_y	Second moment of area for major axis bending
i	Radius of gyration
k	Correlation factor
k_c	Factor
k_s	Transverse stiffness per unit length of beam
k_1	Flexural stiffness of cracked concrete or composite slab in the direction transverse to the steel beam
k_2	Flexural stiffness of the steel web
L	Length of the beam between points at which the bottom flange of the steel member in laterally restrained
	Span of the composite structure
	System length
l	Buckling length
$M_{apl,Rd}$	Design plastic resistance to bending of the structural steel section alone
$M_{a,Sd}$	Moment acting in the steel section due to actions on the structural steelwork alone before the composite action becomes effective
$M_{b,Rd}$	Design buckling resistant moment of a laterally unrestrained beam
M_{cr}	Elastic critical moment for lateral-torsional buckling
$M_{E,Rd}$	Resistance moment at the additional point E
M_{el}	Value of $M_{el,Rd}$ when the γ_m factors γ_a, γ_c, and γ_s are taken as unity
$M_{el,Rd}$	Elastic resistance to bending
$M_{eQ,Rd}$	Moment that causes a stress f_y/γ_a in the extreme bottom fibre of the steel section
M_{pl}	Value of $M_{pl,Rd}$ when the γ_m factors γ_a, γ_c, and γ_s are taken as unity
$M_{pl,Rd}$	Plastic resistance moment
$M_{pl,Rd}$	Plastic design value of the resisting bending moment
$M_{pl,y,Rd}, M_{pl,z,Rd}$	Plastic resistance moment about the major and minor axis
$N_{pm,Rd}$	Compressive resistance force for the whole area of the concrete
$M_{max,Sd}$	Greatest design moment calculated by first order theory
M_{Sd}	Design value of the applied internal bending moment
	Greatest first order design bending moment
$M_{y,sd}, M_{z,sd}$	Design bending moments about the major and minor axis
N	Number of connectors provided within the same length of beam
N_{cr}	Elastic critical load
N_{cr}	Critical load for the relevant axis
$N_{E,ME}$	Normal force and bending moment at the additional point E
$N_{E,Rd}$	Resistance force at the additional point E
N_f	Number of shear connectors determined for the relevant length of beam
$N_{pl,Rd}$	Plastic resistance to compression
$N_{pm,Rd}$	Plastic resistance load for the concrete section alone
N_{sd}	Design axial force
n	Modular ratio
p	Cross sectional area of the profiled steel sheeting per unit length of the beam
$P_{pb,Rd}$	Design bearing resistance of a headed stud welded through the sheet
P_{Rd}	Design shear resistance of a composite structure connector
Q	Variable action

Q	Variable action
Q_d	Design variable action
Q_{ind}	Indirect variable action
Q_k	Characteristic variable action
$Q_{k,i}$	Characteristic values of the other variable actions
$Q_{k,1}$	Characteristic value of one of the variable actions
R_d	Design resistance of material property
R_k	Characteristic resistance of material property
r	Ratio or the lesser to the greater end moment
Sd	Design value of internal force or moment
S_E	Sum of the areas of reinforcement lying in the additional compressed region
s	Effective area of longitudinal slab reinforcement
	Longitudinal spacing of studs or rows of studs
	Steel contribution ratio
	The longitudinal spacing of studs or rows of studs
s_s	Area of any longitudinal reinforcement in compression that is included in the calculation of the bending resistance
s_1	Centre-to-centre spacing of shear connectors in the direction of compression
s_2	Distance from the edge of a compression flange to the nearest line of connectors
t	Thickness of the flange
t	Thickness of the wall of a concrete-filled hollow section
t_f	Thickness of flange
t_w	Thickness of web of the steel member
V_l	Total design longitudinal shear
V_{Sd}	Design transverse shear force
v_{Rd}	Design resistance to longitudinal shear
v_{Sd}	Design longitudinal shear per unit length
W_{pa}	Plastic section modulus for the structural steel
W_{pc}	Plastic section modulus for the concrete
W_{ps}	Plastic section modulus of the total reinforcement
$W_{pan}, W_{psn}, W_{pcn}$	Plastic section moduli for the structural steel, the reinforcement and the concrete parts of the section
X_d	Design value of material property
X_k	Characteristic value of material property
z_c	Distance between the center of area of the steel member and mid-depth of the slab
z_i	Distances to the reference axis for the calculation
α	Coefficient of linear thermal expansion
	Angle between the anchor bar or the hoop and the plane of the flanges of the beam
β	Angle in the horizontal plane between the anchor bar and the longitudinal axis of the beam for anchors set at a splay
	Factor for the determination of moments according to second-order msonary
	Equivalent moment factor

$\gamma_{G,j}$	Partial safety factor for the permanent action $G_{k,j}$
$\gamma_{GA,j}$	As $\gamma_{G,j}$ but for accidental design situations
$\gamma_{Q,i}$	Partial safety factor for the variable action $Q_{K,i}$
$\gamma_G, \gamma_Q, \gamma_o$	Partial safety factors
γ_a, γ_S	Partial safety factors for structural steel and reinforcement
γ_c	Partial safety factor for concrete
γ_{ap}	Partial safety factor for profiled steel sheeting
δ_{max}	Sagging in the final state relative to the straight line joining the supports.
δ_0	Pre-camber(hogging) of the beam in the unloaded state, (state 0).
δ_1	Variation of the deflection of the beam due to the permanent loads immediately after loading, (state 1).
δ_2	Variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load, (state 2)
$\overline{\lambda}$	Relative stiffness
$\overline{\lambda}$	Relevant slenderness
$\overline{\lambda}_{LT}$	Slenderness ratio for lateral-torsional buckling
μ_d	Bending resistance of the cross section
μ_k	Characteristic value of he bending moment of the cross section
ν	Poison's ratio
ν_a	Poison's ratio for steel
ν_{pd}	Contribution of the steel sheeting
ρ	Unit mass
τ_{Rd}	Basic shear strength
χ_{LT}	Reduction factor for lateral-torsional buckling
χ	Reduction coefficient for the relevant buckling mode
χ	Factor taking account for the influence of imperfection and slenderness
ψ_0, ψ_1, ψ_2	Load factors

CHAPTER 2
BASIS OF DESIGN

2.1 FUNDAMENTAL REQUIREMENTS

(1) A structure shall be designed and constructed in such a way that:

(a) With acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost, and
(b) with appropriate degrees of reliability, it will sustain all actions and other influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

(2) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(3) The potential damage should be limited or avoided by appropriate choice of one or more of the following:

(a) Avoiding, eliminating or reducing he hazards which the structure is to sustain
(b) Selecting a structural form which has low sensitivity to the hazards considered
(c) Selecting a structural form and design that can survive adequately the accidental removal of an individual element
(d) Tying the structure together

(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project.

2.2 LIMIT STATES

2.2.1 General

(1) A structure, or part of a structure, is considered unfit for use when it exceeds a particular state, called a limit state, beyond which it infringes one of the criteria governing its performance or use.

(2) All relevant limit states shall be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual approach will be to design on the basis of the most critical limit state and then to check that the remaining limit states will not be reached.

(3) The limit states can be placed in two categories:

(a) The **Ultimate Limit States** are those associated with collapse, or with other forms of structural failure which may endanger the safety of people. States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also treated as ultimate limit states.
(b) The **Serviceability Limit States** correspond to states beyond which specified service requirements are no longer met.

2.2.2 Ultimate Limit States

(1) The ultimate limit states which may require consideration include:

(a) Loss of equilibrium of a part or the whole of the structure considered as a rigid body.
(b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of it, including supports and foundations.

(2) Limit states may also concern only concrete or steel parts of the structure (e.g. the steel part during an erection phase), for which reference should be made to EBCS 2 and EBCS 3, respectively.

2.2.3 Serviceability Limit State

(1) Serviceability limit states which may require consideration include:

(a) Deformations or deflections which affect the appearance or effective use of the structure (including the malfunction of machines or services) or cause damage to finishes of non-structural elements.
(b) Vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.
(c) Cracking of the concrete which is likely to affect appearance, durability or water-lightness adversely.
(d) Damage to concrete because of excessive compression, which is likely to lead to loss of durability.
(e) Slip at the steel-concrete interface when it becomes large enough to invalidate design checks for other serviceability limit states in which the effects of slip are neglected.

2.3 DESIGN SITUATIONS

(1) Design situations are classified as:

(a) Persistent situations corresponding to normal conditions of use of the structure.
(b) Transient situations, for example during construction or repair.
(c) Accidental situations.

(2) For composite structures attention is drawn to the necessity of identifying and considering, when relevant, several transient design situations corresponding to the successive phases of the building process. For example, it may be necessary not only to consider the situation of the steel beam supporting the fresh concrete, but even to distinguish several situations corresponding to successive phases of pouring the concrete.

2.4 ACTIONS

2.4.1 Definitions and Principal Classification

(1) An action *F* is:

(a) a force (load) applied to the structure (direct action), or
(b) an imposed deformation (indirect action); for example, temperature effects or settlement.

(2) Actions are classified:

(a) By their variation in time:
  (i) Permanent actions (G), e.g. self-weight of structures, fittings, ancillaries and fixed equipment.
  (ii) Variable actions (Q), e.g. imposed loads or wind loads.
  (iii) Accidental actions (A), e.g. explosions or impact from vehicles.

(b) By their spatial variation:
  (i) Fixed actions, e.g. self-weight.
  (ii) Free actions, which result in different arrangements of actions, e.g. movable imposed loads and wind loads.

(3) Indirect actions are either permanent G_{ind} (e.g. settlement of support) or variable Q_{ind} (e.g. temperature) and are treated accordingly.

(4) Supplementary classifications relating to the response of the structure are given in the relevant clauses.

2.4.2 Representative Values of Actions

(1) For verification in the partial safety coefficient method, actions are introduced into the calculations by representative values, i.e. by values corresponding to certain levels of intensity. For different calculations, one may have to distinguish different representative values of an action, according to its variation in time. The complete set of representative values is as follows:

(a) Characteristic values;	F_k
(b) Service values;	F_{ser}
(c) Combination values;	$\psi_0 F_k$
(d) Frequent values;	$\psi_1 F_k$
(e) Quasi-permanent values;	$\psi_2 F_k$

The factors ψ_i are defined in Section 2.4.4(5). The above values are evaluated mainly on a statistical basis.

(2) Maximum values and minimum values, which may be zero, are defined when appropriate.

(3) Depending on the variation with time of certain actions, their representative values are sometimes subclassified as actions of long duration (or sustained actions) or of short duration (or transient actions). In special cases, certain actions have their representative values divided into sustained and transient components.

2.4.3 Representative Values of Permanent Actions

(1) The representative values of permanent actions are specified as:

(a) The characteristic values F_k specified in EBCS 1 - "Basis of Design and Actions on Structures", or
(b) by the client, or the designer in consultation with the client, provided that minimum provisions, specified in the relevant codes or by the competent authority, are observed.

(2) The other representative values are assumed to be equal to those in (1) above.

(3) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(4) The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.

2.4.4 Representative Values of Variable Actions

(1) The main representative value is the characteristic value Q_k.

(2) For variable actions the characteristic value Q_k corresponds to either:

(a) The upper value with an intended probability of not being exceeded or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
(b) the specified value.

(3) Other representative values are expressed in terms of the characteristic value Q_k by means of a factor ψ_i. These values are defined as:

(a) Combination value: $\psi_0 Q_k$
(b) Frequent value: $\psi_1 Q_k$
(c) Quasi-permanent value: $\psi_2 Q_k$

(4) Supplementary representative values are used for fatigue verification and dynamic analysis.

(5) The factors ψ_i are specified:

(a) In EBCS 1 - "Basis of Design and Actions on Structures", or
(b) by the client or the designer in conjunction with the client, provided that minimum provisions, specified in the relevant codes or by the competent public authority, are observed.

2.4.5 Representative Values of Accidental Actions

(1) The representative value of accidental actions is the characteristic value A_k (when relevant) and generally correspond to a specified unique nominal value beyond which there is no longer an assurance of a probability of survival of the structure.

(2) Their service, combination and frequent values are considered negligible or zero.

2.4.6 Design Values of Actions

(1) The design value F_d of an action is expressed in general terms as

$$F_d = \gamma_F F_k \tag{2.1}$$

Specific examples are:

$$
\begin{aligned}
G_d &= \gamma_G G_k \\
Q_d &= \gamma_Q Q_k \text{ or } \gamma_Q \psi_i Q_k \\
A_d &= \gamma_A A_k \text{ (if } A_d \text{ is not directly specified)} \\
P_d &= \gamma_P P_k
\end{aligned} \tag{2.2}
$$

where γ_F, γ_G, γ_Q, γ_A and γ_P are the partial safety factors for the action considered taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions, and uncertainties in the assessment of the limit state considered.

(2) The upper and lower design values of permanent actions are expressed as follows:

(a) Where only a single characteristic value G_k is used, then

$$
\begin{aligned}
G_{d,sup} &= \gamma_{G,sup}\, G_k \\
G_{d,inf} &= \gamma_{G,inf}\, G_k
\end{aligned}
$$

(b) Where upper and lower characteristic values of permanent actions are used, then

$$
\begin{aligned}
G_{d,sup} &= \gamma_{G,sup}\, G_{k,sup} \\
G_{d,inf} &= \gamma_{G,inf}\, G_{k,inf}
\end{aligned}
$$

where $G_{k,inf}$ is the lower characteristic value of the permanent action
$G_{k,sup}$ is the upper characteristic value of the permanent action
$\gamma_{G,inf}$ is the lower value of the partial safety factor for the permanent action
$\gamma_{G,sup}$ is the lower value of the partial safety factor for the permanent action

2.4.7 Design Values of the Effect of Actions

(1) The effects of actions are responses (e.g. internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions are determined from the design values of the actions, geometrical data and material properties when relevant.

(2) In some cases, in particular for nonlinear analysis, the effect of the randomness of the intensity of the actions and the uncertainty associated with the analytical procedures, e.g. the models used in the calculations, should be considered separately. This may be achieved by the application of a coefficient of model uncertainty, either applied to the actions or to he internal forces and moments.

2.5 MATERIALS

2.5.1 Characteristic Strength

(1) A material property is represented by a characteristic value which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

(2) In certain cases a nominal value is used as the characteristic value.

(3) Material properties for steel structures are generally represented by nominal values used as characteristic values.

(4) A material property may have two characteristic values, the upper value and the lower value. In most cases only the lower values need to be considered. However, the upper values of the yield strength, for example, should be considered in special cases where overstrength effects may produce a reduction in safety; this is for example the case for the tensile strength of concrete in the calculation of the effects of indirect actions.

2.5.2 Design Values

(1) The design value X_d of a material property is generally defined as:

$$X_d = \frac{X_k}{\gamma_M} \tag{2.3}$$

where γ_M is the partial safety factor for the material property.

(2) For composite steel and concrete structures, the design resistance R_d is generally determined directly from the characteristic values of the material properties and geometrical data:

$$R_d = \frac{R_k}{\gamma_M} \tag{2.4}$$

where γ_M is the partial safety factor for the resistance as provided in EBCS-2 and EBCS-3.

(3) The design value R_d may be determined from tests.

2.6 GEOMETRICAL DATA

(1) Geometrical data are generally represented by their nominal values:

$$a_d = a_{nom} \tag{2.5}$$

(2) In some cases the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta a \tag{2.6}$$

where Δ_a is the additive partial safety margin for geometrical data. The values of Δ_a are given in the appropriate sections.

(3) For imperfections to be adopted in the global analysis of the structure, see Sections 4.8.2.3 and 4.9.3.

2.7 LOAD ARRANGEMENTS AND LOAD CASES

(1) A load arrangement identifies the position, magnitude and direction of a free action.

(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

(3) For the relevant combinations of actions, sufficient load cases shall be considered to enable the critical design conditions to be established.

(4) Simplified load cases may be used, if based on a reasonable interpretation of the structural response.

(5) For continuous beams and slabs in buildings without cantilevers subjected to dominantly uniformly distributed loads, it will generally be sufficient to consider only the following load arrangements:

(a) Alternate spans carrying the design variable and permanent loads ($\gamma_Q Q_k + \gamma_G G_k$), other spans carrying only the design permanent load $\gamma_G G_k$.
(b) Any two adjacent spans carrying the design variable and permanent loads ($\gamma_Q Q_k + \gamma_G G_k$), all other spans carrying only the design permanent load $\gamma_G G_k$.

2.8 DESIGN REQUIREMENTS

2.8.1 General

(1) It shall be verified that no relevant limit state is exceeded.

(2) All relevant design situations and load cases shall be considered.

(3) Possible deviations from the assumed directions or positions of actions shall be considered.

(4) Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

2.8.2 Ultimate Limit States

2.8.2.1 *Verification Conditions*

(1) When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \tag{2.7}$$

where $E_{d,dst}$ is the design effect of the destabilizing actions
$E_{d,stb}$ is the design effect of the stabilizing actions.

(2) When considering a limit state of rupture or excessive deformation of a section, member or connection (fatigue excluded) it shall be verified that:

$$S_d \leq R_d \tag{2.8}$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments)
R_d is the corresponding design resistance, each taking account of the respective design values of all structural properties.

(3) When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties.

(4) When considering a limit state of stability induced by second-order effects, it shall be verified that instability does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties. In addition, sections shall be verified according to (2) above.

(5) When considering a limit state of rupture induced by fatigue, it shall be verified that the design value of the damage indicator D_d does not exceed unity, see Chapter 8.

(6) When considering effects of actions, it shall be verified that:

$$E_d \leq C_d \tag{2.9}$$

where E_d is the design value of the particular effect of actions being considered
C_d is the design capacity for the effect of actions.

2.8.2.2 *Combinations of Actions*

(1) For each load case, design values E_d for the effects of actions shall be determined from combination rules involving the design values of actions given in Table 2.1.

Table 2.1 Design Values of Actions for use in the Combination of Actions

Design situation	Permanent actions G_d	Variable actions Q_d		Accidental actions A_d
		Leading variable action	Accompanying variable actions	
Persistent and transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\psi_0 \gamma_Q Q_k$	γ
Accidental (if not specified differ-ently else where)	$\gamma_{GA} G_k$	$\psi_1 Q_k$	$\psi_2 Q_k$	$\gamma_A A_k$ (if A_d is not specified directly)

(2) The design value given in Table 2.1 shall be combined using the following rules (given in symbolic form):

(a) Persistent and transient design situations for verifications other than those relating to fatigue (fundamental combinations):

$$\Sigma \gamma_{Gj} G_{kj} + \gamma_{q,1} Q_{k,1} + \Sigma \gamma_{Q,i} \psi_{o,i} Q_{k,i} \tag{2.10}$$

(b) Accidental design situations (if not specified differently else where):

$$\Sigma \gamma_{GAj} G_{kj} + A_d + \psi_{1,1} Q_{k,1} + \Sigma \psi_{2i} Q_{k,i} \tag{2.11}$$

where
G_{kj} are the characteristic values of the permanent actions
$Q_{k,1}$ is the characteristic value of one of the variable actions
$Q_{k,i}$ are the characteristic values of the other variable actions
A_d is the design value (specified value) of the accidental actions
γ_{Gj} is the partial safety factor for the permanent action G_{kj}
γ_{GAj} is as γ_{Gj} but for accidental design situations
γ_{Qi} is the partial safety factor for the variable action $Q_{K,i}$
ψ_o, ψ_1, ψ_2 are factors defined in Section 2.4.4.

(3) Combinations for accidental design situations either involve an explicit accidental action A or refer to a situation after an accidental event ($A = 0$). Unless specified otherwise, $\gamma_{GA} = 1.0$ may be used

(4) In Eqs. (2.10) and (2.11), indirect actions shall be introduced where relevant.

(5) Simplified combinations for building structures are given in Section 2.8.3.1.

2.8.2.3 *Design Values of Permanent Actions*

(1) In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values and those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values (see Section 2.4.6(2)).

(2) Where the results of a verification may be very sensitive to variations of the magnitude of a single permanent action from place to place in the structure, this action shall be treated as consisting of separate unfavourable and favourable parts. This applies in particular to the verification of static equilibrium, (see Section 2.8.2.4).

(3) Where a single permanent action is treated as consisting of separate unfavourable and favourable parts, allowance may be made for the relationship between these parts by adopting special design values (see Section 2.8.3.1(3) for building structures).

(4) Except for the cases mentioned in (2), the whole of each permanent action should be represented throughout the structure by either its lower or its upper design value, whichever gives the more unfavourable effect.

(5) For continuous beams and frames, the same design value of the self-weight of the structure (evaluated as in Section 2.4.3(4) may be applied to all spans, except for cases involving the static equilibrium of cantilevers (see Section 2.8.2.4).

2.8.2.4 *Verification of Static Equilibrium*

(1) For the verification of static equilibrium, destabilizing (unfavourable) actions shall be represented by upper design values and stabilizing (favourable) actions by lower design values (see Section 2.8.2.1(1)).

(2) For stabilizing effects, only those actions which can reliably be assumed to be present in the situation considered shall be included in the relevant combination.

(3) Variable actions should be applied where they increase the destabilizing effects but omitted where they would increase the stabilizing effects.

(4) Account should be taken of the possibility that non-structural elements might be omitted or removed.

(5) Permanent actions shall be represented by appropriate design values, depending on whether the destabilizing and stabilizing effects result from:

(a) the unfavourable and the favourable parts of a single permanent action, see (9) below, and/or
(b) different permanent actions, see (10) below.

(6) The self-weights of any unrelated structural or non-structural elements made of different construction materials should be treated as different permanent actions.

(7) The self-weight of a homogeneous structure should be treated as a single permanent action consisting of separate unfavourable and favourable parts.

(8) The self-weights of essentially similar parts of a structure (or of essentially uniform non-structural elements) may also be treated as separate unfavourable and favourable parts of a single permanent action.

(9) For building structures, the special partial safety factors given in Section 2.8.3.1(3) apply to the unfavourable and the favourable parts of each single permanent action, as envisaged in Section 2.8.2.3(2).

(10) For building structures, the normal partial safety factors given in Section 2.8.3.1(1) apply to permanent actions other than those covered by (9) above.

(11) For closely bounded or closely controlled permanent actions, smaller ratios of partial safety factors may apply in the other parts of EBCS 3.

(12) Where uncertainty of the value of a geometrical dimension significantly affects the verification of static equilibrium, this dimension shall be represented in this verification by the most unfavourable value that it is reasonably possible for it to reach.

2.8.3 Partial Safety Factors for Ultimate Limit States

2.8.3.1 *Partial Safety Factors for Actions on Building Structures*

(1) For the persistent and transient design situations the partial safety factors given in Table 2.2 shall be used.

Table 2.2 Partial Safety Factors for Actions on Building Structures for Persistent and Transient Design Situations.

		Variable actions (γ_Q)	
	Permanent actions (γ_G)	Leading variable actions	Accompanying variable actions
Favourable effect $\gamma_{F,inf}$	1.0*	-	-
Unfavourable effect $\gamma_{F,sup}$	1.30*	1.60	1.60

* See also Section 2.8.3.1(3)

(2) For accidental design situations to which Eq.(2.10) applies, the partial safety factors for the variable actions are taken as equal to 1.0.

(3) Where, according to 2.8.2.3(2), a single permanent action needs to be considered as consisting of unfavourable and favourable parts, the favourable part may, as an alternative, be multiplied by:

$$\gamma_{G,inf} = 1.1 \tag{2.12}$$

and the unfavourable part by:

$$\gamma_{G,sup} = 1.30 \tag{2.13}$$

provided that applying $\gamma_{G,inf} = 1.0$ both to the favourable part and to the unfavourable part does not give more unfavourable effect.

(4) where the components of a vectorial effect can vary independently, favourable components (e.g. the longitudinal force) should be multiplied by a reduction factor:

(5) For building structures, as a simplification, Eq.(2.10) may be replaced by whichever of the following combinations gives the larger value:

(a) considering only the most unfavourable variable action:

$$\Sigma \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} \tag{2.14}$$

(b) considering all unfavourable variable actions:

$$\Sigma \gamma_{G,j} G_{k,j} + 0.9 \Sigma \gamma_{Q,i} Q_{k,i} \tag{2.15}$$

2.8.3.2 *Partial Safety Factors for Resistances*

(1) Partial safety factors for resistances are given in the relevant clauses in Chapters 4 and 6.

(2) For fatigue verifications see Chapter 8 of EBCS 2.

2.8.4 Serviceability Limit States

(1) It shall be verified that:

$$E_d \leq C_d \text{ or } E_d \leq R_d \tag{2.16}$$

Where C_d is a nominal value or a function of certain design properties of materials related to the design effect of actions considered, and

E_d is the design effect of actions, determined on he basis of one of the combinations defined below.

The required combination is identified in the particular clause for each serviceability verification, see Section 5.2.1(4) and Section 5.3.1(4).

(2) Three combinations of actions for serviceability limit states are defined by the following expressions:

Rare combination:

$$\Sigma G_{k,j} - Q_{k,1} + \Sigma \psi_{0,i} Q_{k,i} \tag{2.17}$$

Frequent combination:

$$\Sigma G_{k,j} + \psi_{1,1} Q_{k,1} + \Sigma \psi_{2,i} Q_{k,i} \tag{2.18}$$

Quasi-permanent combination: $$\Sigma G_{k,j} + \Sigma \psi_{2,i} Q_{k,i} \tag{2.19}$$

where the notation is defined in Section 2.8.2.2(2)

(3) Where simplified compliance rules are given in the relevant clauses dealing with serviceability limit states, detailed calculations using combinations of actions are not required.

(4) Where the design considers compliance of serviceability limit states by detailed calculations, simplified expressions may be used for building structures.

(5) For building structures, as a simplification, Eq. (2.17) for the rare combination may be replaced by whichever of the following combinations gives the larger value:

(a) Considering only the most unfavourable variable action:

$$\Sigma G_{k,j} + Q_{k,1} \tag{2.20}$$

(b) Considering all unfavourable variable actions:

$$\Sigma G_{k,j} + 0.9\, \Sigma Q_{k,i} \tag{2.21}$$

These two expressions may also be used as a substitute for Eq.(2.18) for the frequent combination.

(6) Values of γ_M shall be taken as 1.0 for all serviceability limit states, except where stated otherwise in particular clauses.

2.9 DURABILITY

(1) To ensure an adequately durable structure, the following inter-related factors shall be considered:

(a) Use of the structure
(b) Required performance criteria
(c) Expected environmental conditions
(d) Composition, properties and performance of the materials
(e) Shape of members and the structural detailing
(f) Quality of workmanship and level of control
(g) Particular protective measures
(h) Likely maintenance during the intended life.

(2) The internal and external environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

CHAPTER 3
MATERIALS

3.1 CONCRETE

3.1.1 General

(1) The strength and other data for the concrete are defined on the basis of standard tests.

3.1.2 Grades of Concrete

(1) Concrete is graded in terms of its characteristic compressive cube strength. The grade of concrete to be used in design depends on the classification of the concrete works and its intended use.

(2) Table 2.1 gives the permissible grades of concrete for the two classes of concrete works.

(3) The numbers in the grade designation denote the specified characteristic compressive strength in MPa.

Table 3.1 Grades of Concrete

Class	Permissible Grades of Concrete							
I	C5	C15	C20	C25	C30	C40	C50	C60
II	C5	C15	C20					

Grade C5 shall be used only as lean concrete

3.1.3 Characteristic Compressive Strength of Concrete

(1) For the purpose of this Code, compressive strength of concrete is determined from tests on 150mm cubes at the age of 28 days in accordance with standards issued or approved by Ethiopian Standards.

(2) The characteristic compressive strength is defined as that strength below which 5% of all possible strength measurements may be expected to fall. In practice, the concrete may be regarded as complying with the grade specified for the design if the results of the tests comply with the acceptance criteria laid down in Chapter 9 of EBCS 3.

(3) Cylindrical or cubical specimens of other sizes may also be used with conversion factors determined from a comprehensive series of tests. In the absence of such tests, the conversion factors given in Table 3.2 may be applied to obtain the equivalent characteristic strength on the basis of 150mm cubes.

(4) In Table 3.3 the characteristic cylinder compressive strength f_{ck} are given for the different grades of concrete.

Table 3.2 Conversion Factors for Strength

Size and Type of Test Specimen	Conversion Factor
Cube (200mm)	1.05
Cylinder (150mm diameter 300mm height)	1.25

Table 3.3 Grades of Concrete and Characteristic Cylinder Compressive Strength f_{ck}

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
f_{ck}	12	16	20	24	32	40	48

3.1.4 Characteristic Tensile Strength

(1) In this Code, the characteristic tensile strength refers to the axial tensile strength as determined by tests in accordance with standards issued or approved by ESA.

(2) In the absence of more accurate data, the characteristic tensile strength, may also be determined from the characteristic cylinder compressive strength according to Eq. 3.1.

$$f_{ctk} = 0.7 f_{ctm} \quad (3.1)$$

where f_{ctm} is the mean value given by Eq. 2.2.

$$f_{ctm} = 0.3 f_{ck}^{2/3} \quad (3.2)$$

(3) The corresponding values of f_{ctk} and f_{ctm} for the different grades of concrete are given in Table 2.4.

Table 3.4 Grades of Concrete and Values of f_{ctk} and f_{ctm}

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
f_{ctm}	1.6	1.9	2.2	2.5	3.0	3.5	4.0
f_{ctk}	1.1	1.3	1.5	1.7	2.1	2.5	2.8

3.1.5 Deformation Properties of Concrete

(1) The values of the material properties required for the calculation of instantaneous and time dependent deformations of concrete depend not only upon the grades of concrete but also upon the properties of the aggregates and other parameters related to the mix design and the environment. For this reason, where an accurate calculation is considered necessary, the values should be established from known data appropriate to the particular materials and conditions of use. For many calculations an approximate estimate will usually be sufficient.

3.1.5.1 *Stress-Strain Diagrams*

(1) Any idealized stress-strain diagram which results in predication of strength in substantial agreement with the results of comprehensive tests may be used.

3.1.5.2 *Modulus of Elasticity*

(1) The modulus of elasticity depends not only on the concrete grade but also on the actual properties of the aggregates used (see Section 3.5(1) above).

(2) In the absence of more accurate data, or in cases where great accuracy is not required, an estimate of the mean value of the secant modulus E_{cm} can be obtained from Table 3.5 for a given concrete grade.

Table 3.5 Values of the Secant Modulus of Elasticity E_{cm} in GPa

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
E_{cm}	26	27	29	32	35	37	39

(3) The values in Table 3.5 are based on the following equation:

$$E_{cm} = 9.5(f_{ck} + 8)^{1/3} \tag{3.3}$$

Where E_{cm} is in GPa and f_{ck} is in MPa. They relate to concrete cured under normal conditions and made with aggregates predominantly consisting of quartzite gravel. When deflections are of great importance, tests should be carried out on concrete made with the aggregate to be used in the structure. In other cases experience with a particular aggregate, backed by general test data, will often provide a reliable values for E_{cm}, but with unknown aggregates, it would be advisable to consider a range of values.

(4) As a rule, since the grade of concrete corresponds to a strength at an age of 28 days, the values of E_{cm} in Table 3.5 also relate to that same age. Where great accuracy is not required, E_{cm} can also be determined from Eq.3.3 for a concrete age t other than 28 days. In this case, f_{ck} is replaced by the actual cylinder concrete strength at time t.

3.1.5.3 *Modular Ratios*

(1) The deformation of the concrete due to creep shall be taken into account.

(2) If it is specified for the particular project that the rules for application given below are not accepted, the nominal values given in Section 3.1.5.5 should be adopted.

(3) For the design of buildings, global analyses of sway frames excepted, it is accurate enough to take account of creep by replacing in analyses concrete areas A_c by effective equivalent steel areas equal to A_c/n, where n is the nominal modular ratio, defined by $n = E_a/E'_c$,

where E_a is the elastic modulus of structural steel, given in Section 3.3.4

where E_s is the elastic modulus of structural steel, given in Section 3.3.4
E'_c is an "effective" modulus of concrete, taking in the various cases the values given below.

(4) If specified for the particular project and in any case for buildings mainly intended for storage, two nominal values E'_c should be used: One equal to E_{cm} for short term effects, the other equal to $E_{cm}/3$ for long term effects. In other cases E'_c may be taken equal to $E_{cm}/2$, E_{cm} having the value defined in Section 3.1.5.2.

3.1.5.4 *Poison's Ratio*

(1) Any value between 0 and 0.2 can be adopted for Poison's ratio. It may be assumed to be zero when concrete in tension is assumed to be cracked.

3.1.5.5 *Creep and Shrinkage*

(1) Creep and shrinkage of the concrete depend mainly on the ambient humidity, the dimensions of the element and the composition of the concrete. Creep is also influenced by the maturity of the concrete when the load is first applied and on the duration and magnitude of the loading. Any estimation of the creep coefficient $\phi_{(t,t_0)}$, and of the basic shrinkage strain, ϵ_{cs}, should take these parameters into account.

(2) In cases where great accuracy is not required, the values given in Tables 3.6 and 3.7 respectively can be considered as final creep coefficient $\phi_{(\infty,t_0)}$ and the final shrinkage strain ϵ_{cs} of a normal weight concrete subjected to a compressive stress not exceeding $0.45f_{ck}$ at the time t_o at first loading.

(3) The data given in Tables 3.6 and 3.7 apply for a range of the mean temperature of the concrete between 10 °C and 20 °C. Maximum seasonal temperature up to 40 °C can be accepted. In the same way, variations in relative humidity around the mean values given in Tables 2.6 and 2.7 between RH = 20% and RH = 100% are acceptable.

(4) Linear interpolation between the values in Tables 3.5 and 3.6 is permitted.

Table 3.6 Final Creep Coefficient $\phi_{(\infty,t_0)}$ of Normal Weight Concrete

Age at Loading t_o (days)	Notional size $2A_c/u$ (in mm)					
	50	150	600	50	150	600
	Dry atmospheric conditions (inside) (RH = 50%)			Humid atmospheric conditions (outside) (RH = 80%)		
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2.0
28	3.0	2.5	2.0	1.9	1.7	1.5
90	2.4	2.0	1.6	1.5	1.4	1.2
365	1.8	1.5	1.2	1.1	1.0	1.0

Table 3.7 **Final Shrinkage Strains $\epsilon_{cs\infty}$ (in ‰) of Normal Weight Concrete**

Location of the number	Relative humidity (%)	Notional size $2A_c/u$ (mm) ≤ 150	600
Inside	50	- 0.60	- 0.50
Outside	80	- 0.33	- 0.28

where: A_c = cross-sectional area of concrete
u = perimeter of that area

(5) The values of Table 3.6 and 3.7 apply to concrete having plastic consistency when fresh. For concrete of other consistency the values have to be multiplied by 0.70 (stiff consistency) or 0 (soft consistency)

(6) For concrete with superplasticizers, the consistency before adding the superplasticizers is used for the evaluation of the creep and shrinkage coefficients as given Tables 2.6 and 2.7.

3.1.5.6 *Coefficient of Thermal Expansion*

(1) The coefficient of thermal expansion may be taken as 10 x 10^{-6} per °C.

3.2 REINFORCING STEEL

3.2.1 Characteristic Strength of Reinforcing Steel

(1) The mechanical and technological properties of steel used for reinforced concrete shall be defined by standard and/or agrement documents or by certificates of compliance.

(2) The characteristic strength f_{yk} is defined as the 5% fractile of the proof stress f_y or 0.2% offset strength, denoted as $f_{0.2}$

(3) If the steel supplier guarantees a minimum value for f_y or $f_{0.2}$, that value may be taken as the characteristic strength.

3.2.2 Classification and Geometry of Reinforcing Steel

(1) Reinforcing steel shall be classified according to:

(a) Grade, denoting the value of the specified characteristic yield stress (f_{yk}) in MPa.
(b) Class, indicating the ductility characteristics
(c) Size
(d) Surface characteristics
(e) Weldability

(2) Each consignment shall be accompanied by a certificate containing all the information necessary for its identification with regard to (a) to (e) above, and additional information where necessary.

(3) The actual cross sectional area of the products shall not differ from their nominal cross sectional area by more than the limits specified in relevant Standards.

(4) In this Code, two classes of ductility are defined (see Section 3.2.3.2):

(a) High (Class A)
(b) Normal (Class B)

(5) In this Code two shapes of surface characteristics are defined:

(a) Ribbed bars, resulting in high bond action
(b) Plain, smooth bars, resulting in low bond action.

(6) For other types of bar, with other surface characteristics (ribs or indentations), reference should be made to relevant documents, based on test data.

(7) Welded fabric, used as reinforcing steel, shall comply with the dimensional requirements in relevant Standards.

3.2.3 Physical Properties of Reinforcing Steel

(1) The following mean values may be assumed:

(a) Density	7 850 kg/m^3
(b) Coefficient of thermal expansion	10 x 10^{-6} per °C

3.2.4 Mechanical Properties of Reinforcing Steel

3.2.4.1 *Strength*

(1) The yield stress f_{yk} and the tensile strength f_{tk} are defined respectively as the characteristic value of the yield load, and the characteristic maximum load in direct axial tension, each divided by the nominal cross sectional area.

(2) For products without a pronounced yield stress f_{yk}, the 0.2% proof stress $f_{0.2k}$ may be substituted.

3.2.4.2 *Ductility*

(1) The products shall have adequate ductility in elongation, as specified in relevant Standards.

(2) Adequate ductility in elongation may be assumed, for design purposes, if the products satisfy the following ductility requirements:

(a) High ductility:	$\epsilon_{uk} > 5\%$;	value of $(f_t / f_y)_k > 1.08$
(b) Normal ductility:	$\epsilon_{uk} > 2.5\%$;	value of $(f_t / f_y)_k > 1.05$

In which ϵ_{uk} denotes the characteristic value of the elongation at maximum load.

(3) High bond bars with diameters less than 6 mm should not be treated as having high ductility.

(4) The products shall have adequate bendability for the anticipated use.

3.2.4.3 *Stress-Strain Diagram*

(1) In the absence of more accurate information, an elasto-plastic diagram can be used for hot rolled steel or steel cold worked by drawing or rolling.

(2) For other types of production, the actual stress-strain diagrams can be replaced by bilinear, trilinear or other diagrams chosen so that the approximations are on the safe side.

3.2.4.4 *Modulus of Elasticity*

(1) The mean value of modulus of elasticity E_s may be assumed as 200 GPa.

3.2.4.5 *Fatigue*

(1) Where required, the products shall have adequate fatigue strength.

3.2.5 Technological Properties

3.2.5.1 *Bond and Anchorage*

(1) The surface characteristics of ribbed bars shall be such that adequate bond is obtained with the concrete, permitting the full force that is assumed in design, to be developed in the reinforcement.

(2) Ribbed bars, having projected ribs not satisfying the requirements for high bond bars given in relevant standards should be treated as plain bars with respect to bond.

(3) The behaviour in bond of reinforcing steels with other surface shapes should be defined in relevant Standards or technical approved documents.

(4) The strength of the welded joints along the anchorage length of welded fabric shall be adequate.

(5) The strength of the welded joint can withstand a shearing force not less than 30% of a force equivalent to the specified characteristic yield stress times the nominal cross sectional area of the anchored wire.

3.2.5.2 *Weldability*

(1) The products shall have weldability properties adequate for the anticipated use

(2) Where required, and where the weldability is unknown, tests should be requested.

(3) Ductility characteristics, as specified in Section 2.9.2, should be maintained, when necessary, at sections near to weld.

3.3 STRUCTURAL STEEL

3.3.1 Scope

(1) This Code covers the design of structures fabricated from steel material conforming to internationally accepted standards.

(2) It may also be used for other structural steels, provided that adequate data exist to justify the application of the relevant design and fabrication rules.

(3) For high strength steel refer to internationally accepted standards.

3.3.2 Material Properties for Hot Rolled Steel

3.3.2.1 *Nominal Values*

(1) The nominal values of the yield strength f_y and the ultimate tensile strength f_u for hot rolled steel are given in Table 3.8 for steel grades F_e 360, F_e 430 and F_e 510.

Table 3.8 Nominal Values of Yield Strength f_y and Ultimate Tensile Strength for Various Grades of Structural Steel

Nominal Steel Grade	Thickness t (mm)			
	$t \leq 40$mm		40mm $< t \leq$ 100mm	
	f_y (MPa)	f_u (MPa)	f_y (MPa)	f_u (MPa)
F_e 360	235	360	215	340
F_e 430	275	430	255	410
F_e 510	355	510	335	490

Note: t is the nominal thickness of the element.

(2) The nominal values in Table 3.8 may be adopted as characteristic values in calculations.

(3) Similar values may be adopted for hot finished structural hollow sections.

(4) For high strength steel refer to internationally accepted standards.

3.3.2.2 *Plastic Analysis*

(1) Plastic analysis may be utilised in the global analysis of structures or their elements provided that the steel complies with the following additional requirements:

(a) The ratio of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y satisfies:

$$f_u/f_y \tag{3.4}$$

(b) The elongation at failure on a gauge length of $5.65\sqrt{A_o}$ (where A_o is the original cross section area) is not less than 15%

(c) The stess-strain diagram shows that the ultimate strain ϵ_u corresponding to the ultimate tensile strength f_u is at least 20 times the yield strain ϵ_y corresponding to the yield strength f_y.

(2) The steel grads listed in Table 3.1 may be accepted as satisfying these requirements.

3.3.2.3 *Fracture Toughness*

(1) The material shall have sufficient fracture toughness to avoid brittle fracture at the lowest service temperature expected to occur within the intended life of the structure.

(2) For high strength steel refer to internationally accepted standards.

3.3.3 Dimensions, Mass and Tolerances

(1) The dimensions and mass of all rolled steel sections, plates and structural hollow sections, and their dimensional and mass tolerances, shall conform to internationally accepted standards.

3.3.4 Design values of Material Coefficients

(1) The material coefficients to be adopted in calculations for the steels covered by this Code shall be taken as follows:

(a) Modulus of elasticity	E	=	210 GPa
(b) Shear modulus	G	=	80 GPa
(c) Poison's ratio	ν	=	0.3
(d) Coefficient of linear thermal expansion	α	=	12×10^{-6} per °C
(e) Unit mass	ρ	=	7850 kg/m^3

3.4 CONNECTING DEVICES

3.4.1 General

(1) Connecting devices shall be suitable for their specified use.

(2) For connecting devices other than shear connectors, Section 3.3 of EBCS 3 is applicable.

3.4.2 Shear Connection

(1) The resistance of a connector is the maximum load in the direction considered (in most cases parallel to the interface between concrete flange and steel beam) that can be carried by the connector before failure. the resistance of a connector may be different for reversal in the direction of thrust. Due account shall be taken of this.

(2) The characteristic resistance P_{Rk} shall be the specified resistance below which not more than 5% of results of tests on samples of a homogeneous population may be expected to fall. When a guaranteed minimum resistance is specified this may be considered as the characteristic resistance.

(3) The design resistance P_{Rd} shall be the characteristic resistance divided by he appropriate partial safety factor γ_v. For the determination of the design resistance by testing, refer to Chapter 10.

(4) The material of the connector shall be of a quality which takes into account its required performance and the method of fixing to the structural steelwork. Where fixing is by means of welding, the quality of material shall take account of the welding technique to be used. Where anchors or hoops act as shear connectors, special care shall be taken that the material is of an appropriate weldable quality.

(5) The specified mechanical properties of the connector material shall comply with the following requirements:

(a) the ratio of he specified ultimate tensile strength f_u to the specified minimum yield strength f_y is not less than ;

(b) the elongation at failure on a gauge length of $5.65\sqrt{A_o}$ (where A_o is the original cross section are) is not less than 12%.

For studs, these material properties relate to the finished product

(6) The head of stud connectors should have a diameter of not less than $1.5d$ and a depth of not less than $0.4d$, where d is the shank diameter of the stud.

CHAPTER 4
ULTIMATE LIMIT STATES

4.1 BASIS

4.1.1 General

(1) The scope of this chapter is composite beams, columns, frames and connections, except that design of shear connection in beams and in-plane shear in a concrete flange are treated in Chapter 6. Beams with full-encased steel sections are excluded. Use of precast concrete slabs is treated in Chapter 7.

(2) Composite structures and members shall be so proportioned that the basic design requirements for the ultimate limit state given in Chapter 2 are satisfied. The relevant design requirements given in EBCS 2 and EBCS 3 shall also be satisfied.

(3) For building structures, the requirements of Section 2.8.2.4 of EBCS 3 concerning static equilibrium shall be satisfied.

(4) In analyses of composite structures, members, and cross-sections, appropriate account shall be taken of the properties of concrete and reinforcing steel as defined in EBCS 2 and of the properties of steel as defined in EBCS 3. Account shall be taken of loss of resistance or ductility associated with buckling of steel, and with cracking, crushing, or spalling of concrete.

(5) Values of γ_M for ultimate limit states are given in Section 2.8.3.1. For certain resistances where buckling of steel is relevant γ_a for structural steel is replaced by γ_{Rd}. Its value for fundamental combinations is given in relevant clauses in this Chapter. For accidental combinations, $\gamma_{Rd} = 1.0$.

(6) No consideration of temperature effects in verifications for ultimate limit states is normally necessary for composite structures for buildings.

(7) The effects of shrinkage of concrete may be neglected in verifications for ultimate limit states for composite structures for buildings. except in global analyses with members having cross-sections in Class 4 (Sections 4.3 and 4.5.3.3).

(8) The effects of creep of concrete on both global and local analyses may be allowed for in composite members and frames in building structures by the use of modular ratios. For slender columns, Section 4.8.3.6 (2) is relevant.

(9) For composite members in building structures, a fatigue check is not normally required, except for:

(a) Members supporting lifting appliances or rolling loads
(b) Members supporting vibrating machinery
(c) Members subject tow ind-induced oscillations
(d) Members subject to crowd-induced oscillations.

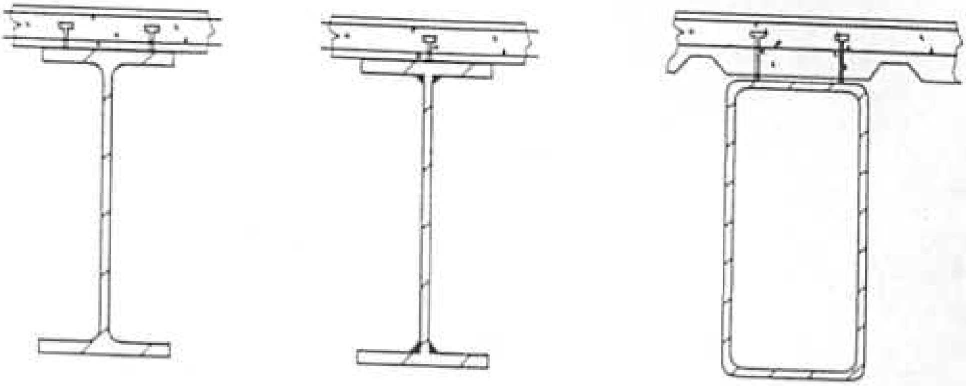


Figure 4.1 Typical Cross-Section of Composite Beams

4.1.2 Beams

(1) Typical types of cross-section are shown in Figs. 4.1 and 4.8.

(2) Any concrete encasement of a steel web should not be assumed to be part-of a concrete flange or to contribute to resistance in bending or vertical shear. Web encasement in accordance with Section 4.3.1 may be assumed to contribute to resistance to local buckling (Sections 4.3.2, 4.3.3) or lateral-torsional buckling (Section 4.6.2).

(3) Composite beams shall be checked for:

(a) Resistance of critical cross-sections (Section 4.4)
(b) Resistance to lateral-torsional buckling (Section 4.6)
(c) Resistance to shear buckling (Section 4.4.4) and web crippling (Section 4.7)
(d) Resistance to longitudinal shear (Chapter 6).

(4) Critical cross-sections include:

(a) Sections of maximum bending moment
(b) Supports
(c) Sections subjected to heavy concentrated loads or reactions
(d) Places where a sudden change of cross-section occurs, (other than a change due to cracking of concrete).

(5) For checking resistance to longitudinal shear, a critical length consists of a length of the interface between structural steel and concrete bounded by two critical cross sections. For this purpose critical cross sections also include:

(a) Free ends of cantilevers and
(b) in tapering members, sections so chosen that the ratio of the greater to the lesser second moment of area for any pair of adjacent sections does not exceed two.

(6) The concepts "full shear connection" and "partial shear connection" are applicable only to beams in which plastic theory is used for calculating bending resistances of critical cross sections. A span of a beam, or a cantilever, has full shear connection when increase in the number of shear connectors would not increase the design bending resistance of the member. Otherwise, the shear connections is partial. Limits to the use of partial shear connection are given in Section 6.1.2.

4.1.3 Composite Columns and Connections

These subjects are treated in sections 4.8 to 4.10, respectively. Section 4.2 to 4.7 (Beams) and 4.8 (Columns) apply both to isolated members and to members in composite frames.

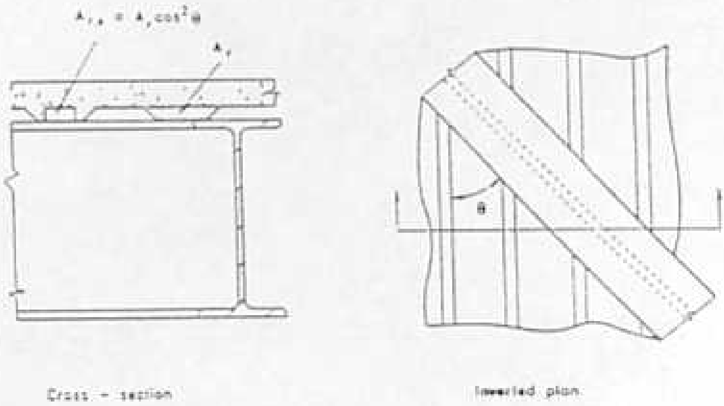


Figure 4.2 Effective Section of Rib of Composite Slab

4.2 PROPERTIES OF CROSS-SECTIONS OF BEAMS

4.2.1 Effective Section

(1) Allowance shall be made for the flexibility of concrete flange in-plane shear (shear lag) either by means of rigorous analysts, or by using an effective width of flange in accordance with Section 4.2.2.

(2) The effective section of an effective width of composite slab with its ribs running at an angle θ to the beam should be taken as the full area of the concrete above the top of the rips plus $\cos^2 \theta$ times the area of the concrete within the depth of the ribs (Fig. 4.2). Where $\theta > 60^0$, $\cos^2 \theta$ should be taken as zero.

(3) Where rigid-plastic global analysis or plastic analysis of cross sections is used, only reinforcement of high ductility, as defined in Section 2.9.2 of EBCS 2, should be included on the effective section. Welded mesh should not be included unless it has been shown to have sufficient ductility, when built into a concrete slab, to ensure that it will not fracture.

(4) Profiled steel sheeting should not be included in the effective section of a beam unless the ribs run parallel to the beam and the detail design ensures continuity of strength cross joints in the sheeting and appropriate resistance in longitudinal shear.

(5) For classification and analysis of cross-sections, a web in Class 3 may be represented by an effective web in Class 2, in accordance with Section 4.3.3.

(6) The effective cross-section properties of structural steel compression elements in Class 4, as defined in Section 4.3.1, shall be based on effective widths in accordance with Section 4.3.4 of EBCS 3.

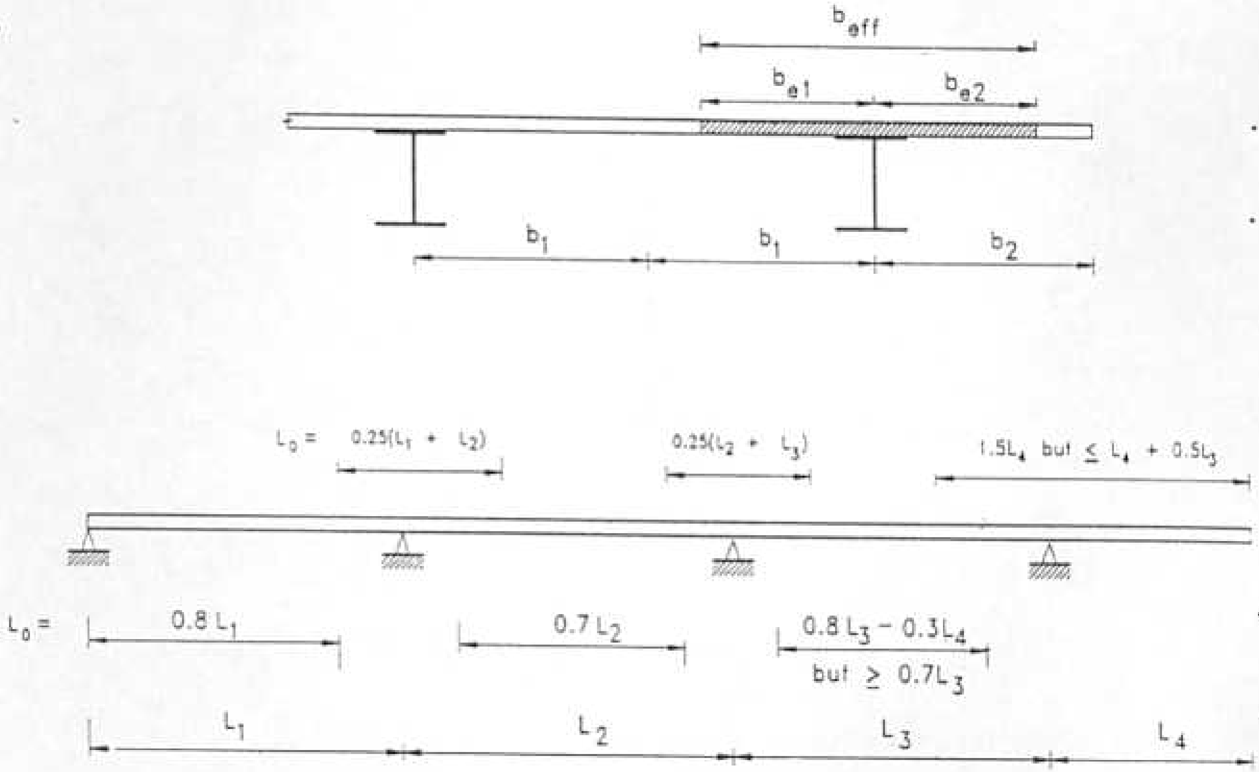


Figure 4.3 Equivalent Spans, for Effective Width of Concrete Flange

4.2.2 Effective Width of Concrete Flange for Beams in Buildings

4.2.2.1 *Effective Width for Global Analysis*

(1) A constant effective width may be assumed over the whole of each span. This value may be taken as the value at midspan, for a span supported at both ends, or the value at the support, for a cantilever.

(2) The total effective width b_{eff} of concrete flange associated with each steel web should be taken as the sum of effective widths b_e of the portion of the flange on each side of the centreline of the steel web (Figure 4.3). The effective width of each portion should be taken as $b_e = l_o/8$ but not greater than b.

(3) The actual width b of each portion should be taken as half the distance from the web to the adjacent web, measured at mid-depth of the concrete flange, except that at a free edge the actual width is the distance from the web to the free edge.

(4) The length l_o is the approximate distance between points of zero bending moment. For simply supported beams it is equal to the span. For typical continuous beams, l_o may be assumed to be as shown in Fig. 4.3 in which values at supports are shown above the beam, and midspan values below the beam.

4.2.2.2 *Effective Width for Verification of Cross-sections*

(1) For sections in sagging bending, the appropriate midspan value given by Section 4.2.2.1 should be used.

(2) For sections in hogging bending, the value at the relevant support given by Section 4.2.2.1 should be used.

4.2.3 Flexural Stiffness

(1) The elastic section properties of a composite cross-section should be expressed as those of an equivalent steel cross-section by dividing the contribution of the concrete component by a modular ratio n, as given in Section 3.1.5.3.

(2) The uncracked and cracked flexural stiffnesses of a composite cross section are defined as $E_a I_1$ and $E_a I_2$, respectively,

where: E_a is the modulus of elasticity for structural steel,
I_1 is the second moment of area of the effective equivalent steel section calculated assuming that concrete in tension is uncracked, and
I_2 is the second moment of area of the effective equivalent steel section calculated neglecting concrete in tension but including reinforcement

4.3 CLASSIFICATION OF CROSS-SECTIONS OF BEAMS

4.3.1 General

(1) The classification system defined in Sections 4.3.2 of EBCS 3 applies to cross-sections of composite beams.

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

(3) The performance of a steel compression element in Class 2,3, or 4 can be improved by attaching it to a reinforced concrete element. The restrained steel element may be placed in a higher class, provided that the relevant improvement has been established.

(4) Where relevant application rules are given, a steel compression element may be represented by an effective element in a higher class.

(5) Positions of plastic neutral axes of composite sections shall be calculated using design values of strengths of materials.

(6) For a web to be treated as "encased" in Table 4.1, the concrete that encases it shall 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.

(7) The concrete that encases a web should extend over the full width of the steel flanges. It should be reinforced by longitudinal bars and stirrups, and/or welded mesh.

(8) The concrete between the flanges may be fixed to the web by welding the stirrups to the web or by means of bars ($\varphi \geq$ 6mm) through holes and/or studs with a diameter greater than 10mm welded to the web.

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

(10) For fire design, reference should be made to recognized international standards.

4.3.2 Classification of Steel Flanges in Compression

(1) A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors in accordance with Section 6.4.1.5 may be assumed to be in Class 1.

(2) The classification of other steel flanges in compression in composite beams shall be in accordance with Table 4.1, for outstand flanges, and Table 4.3 of EBCS 3, for internal flange elements.

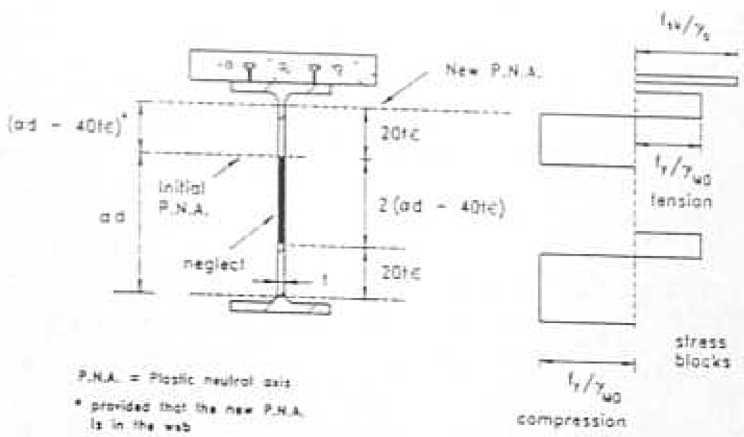


Figure 4.4 Use of an Effective Web in Class 2 for a Section in Hogging Bending with a Web in Class 3

4.3.3 Classification of Steel Webs

4.3.3.1 *Section Where the Compression Flange is in Class 1 or 2*

(1) The class of web shall be determined from Table 4.2. The plastic stress distribution for the effective composite section shall be used; except at the boundary between Classes 3 and 4, where the elastic stress distribution shall be used, as in Section 4.3.3.2.

(2) The web in Class 3 that is encased in concrete in accordance with Section 4.3.1(6) to (9) may be represented by an effective web of the same cross-section in Class 2.

(3) An encased web in Class 3 may be represented by an effective web in Class 2, by assuming that the depth of web that resists compression is limited to $20t\epsilon$ adjacent to the compression flange, and $20t\epsilon$ adjacent to the new plastic neutral axis, as shown in Fig. 4.4 for hogging bending.

4.3.3.2 *Sections where the Compression Flange is in Class 3 or 4*

(1) The class of the web shall be determined from Table 4.2, using the elastic neutral axis.

(2) In beams for buildings, the position of the elastic neutral axis should be determined for the effective concrete flange, neglecting concrete in tension, and the gross cross-section of the steel web. The modular ration for concrete in compression should be as used in the global analysis for long-term effects.

4.4 RESISTANCES OF CROSS-SECTIONS OF BEAMS

4.4.1 Bending Moment

4.4.1.1 *Basis*

(1) Section 4.4 is applicable to composite sections where the structural steel component has an axis of summery in the plane of the web, and to bending in this plane.

(2) The design bending resistance may be determined by plastic theory only where the effective composite section is in Class 1 or Class 2.

(3) Elastic analysis may be applied to cross-sections of any class.

(4) No account need be taken of the effects of longitudinal slip in composite members with full shear connection. Plane cross-sections of these members should be assumed to remain plane.

(5) Fastener holes in steel elements shall be considered, following Section 6.2.3 of EBCS 3.

(6) Small holes in steel through which reinforcing bars pass should be treated as holes for fasteners.

4.4.1.2 *Plastic Resistance Moment of a Section with Full Shear Connection*

(1) Full shear connection is defined in 4.1.2(6).

(2) The following assumptions shall be made in the calculation of $M_{pl.Rd}$:

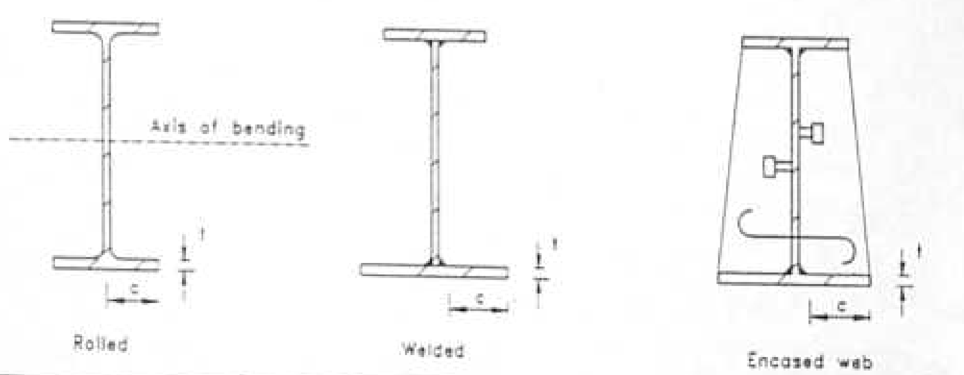
- (a) There is full interaction between structural steel, reinforcement, and concrete
- (b) The effective area of the structural steel member is stressed to its design yield strength f_y/γ_a in tension or compression
- (c) The effective areas of longitudinal reinforcement in tension and in compression are stressed to their design yield strengths f_{sk}/γ_s in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected

(3) Any profiled steel sheeting in tension included within the effective area following Section 4.2.1(4) should be assumed to be stressed to its design strength f_y/γ_{ap}.

(4) It shall be assumed that the effective area of concrete in compression resists a stress of $0.85 f_{ck}/\gamma_c$, constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete.

(5) Typical plastic stress distributions are shown in Fig. 4.5.

Table 4.1 Maximum Width-to-Thickness Ratios for Steel Outstand Flanges in Compression

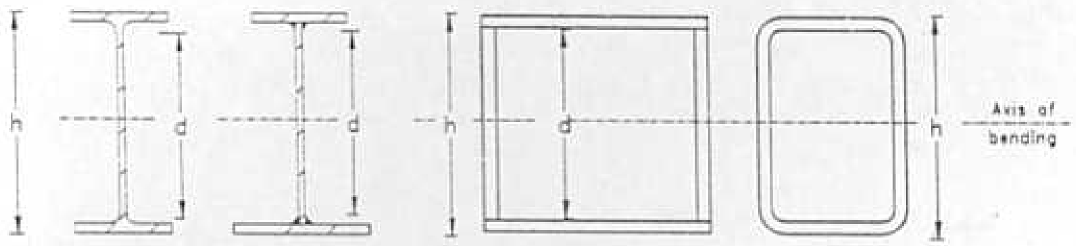


Class	Type	Web not encased	Web encased
Stress distribution (Compression positive)		+	+
1	Rolled	$c/t \leq 10\epsilon$	$c/t \leq 10\epsilon$
	Welded	$c/t \leq 9\epsilon$	$c/t \leq 9\epsilon$
2	Rolled	$c/t \leq 11\epsilon$	$c/t \leq 15\epsilon$
	Welded	$c/t \leq 10\epsilon$	$c/t \leq 14\epsilon$
3	Rolled	$c/t \leq 15\epsilon$	$c/t \leq 21\epsilon$
	Welded	$c/t \leq 14\epsilon$	$c/t \leq 20\epsilon$

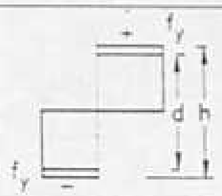
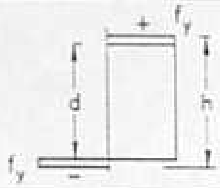
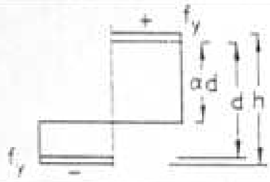
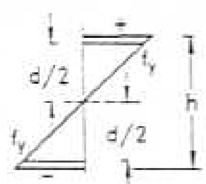
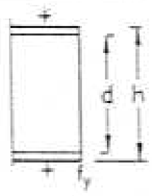
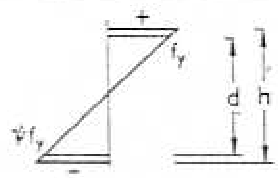
$\epsilon = \sqrt{235/f_y}$	f_y (N/mm^2)	235	275	355
	ϵ	1.0	0.92	0.81

Table 4.2 Maximum Width-to-Thickness Ratios for Steel Web

Webs: (internal elements perpendicular to axis of bending)



$d = h - 3t$

Class	Web subjected to bending	Web subjected to compression	Web subjected to bending and compression
Stress distribution (Compression positive)			
1	$d/t \leq 72\epsilon$	$d/t \leq 33\epsilon$	when $\alpha > 0.5$: $d/t \leq 396\epsilon/(13\alpha - 1)$ when $\alpha < 0.5$: $d/t \leq 36\epsilon/\alpha$
2	$d/t \leq 83\epsilon$	$d/t \leq 38\epsilon$	when $\alpha > 0.5$: $d/t \leq 456\epsilon/(13\alpha - 1)$ when $\alpha < 0.5$: $d/t \leq 41.5\epsilon/\alpha$
Stress distribution (Compression positive)			
3	$d/t \leq 124\epsilon$	$d/t \leq 42\epsilon$	when $\psi > -1$: $d/t \leq 42\epsilon/(0.67 + 0.33\psi)$ when $\psi \leq -1$: $d/t \leq 62\epsilon/(1-\psi)\sqrt{(-\psi)}$

$\epsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ϵ	1	0.92	0.81

4.4.1.3 *Plastic Resistance Moment of a Section with Partial Shear Connection*

(1) Partial shear connection in accordance with 6.2.1 may be used in composite beams for buildings for the compressive force in the concrete slab.

(2) The plastic moment of resistance of the beam should be calculated in accordance with Section 4.4.1.2, except that a reduced value of the compressive force in the concrete, F_c, determined from Section 6.2.1, should be used in place of the force given by Section 4.4.1.2(4). The location of the plastic neutral axis in the slab is determined by the new force F_c. There is a second plastic neutral axis within the steel section, which should be used for the classification of the web.

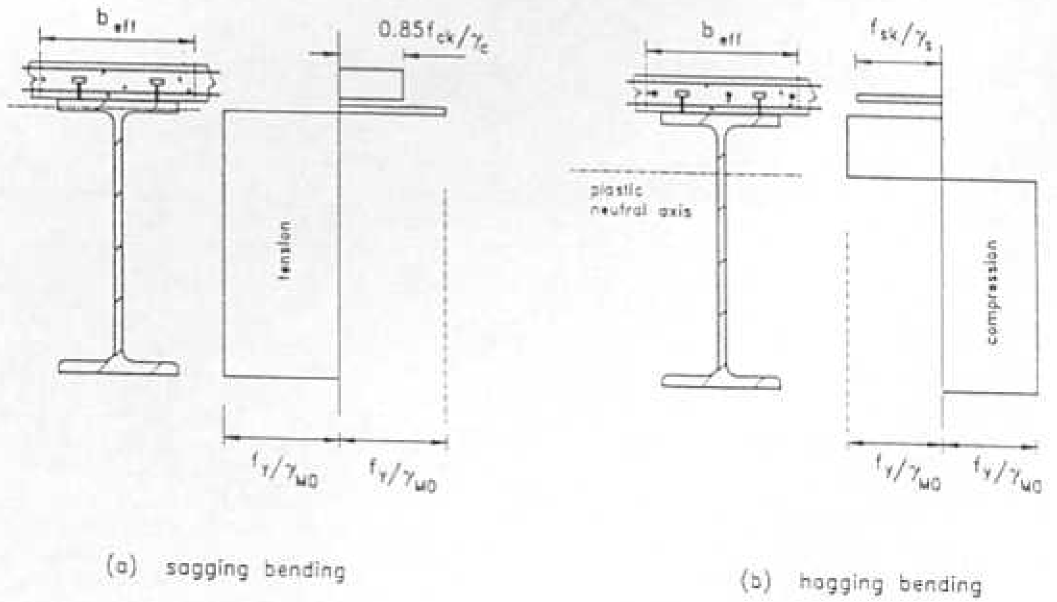


Figure 4.5 Plastic Stress Distributions for a Composite Beam with Profiled Steel Sheeting and Full Shear Connection, where the Plastic Neutral Axis is within the Steel Section

4.4.1.4 *Elastic Resistance to Bending*

(1) Stresses shall be calculated by elastic theory, using an effective cross section in accordance with Sections 4.2.1 and 4.2.2.2.

(2) Account shall be taken of creep of concrete in compression, in accordance with Section 3.1.5.4.

(3) In the calculation of $M_{el.Rd}$, the limiting bending stresses shall be taken as:

(a) $0.85 f_{ck}/\gamma_c$ in concrete in compression:
(b) f_y/γ_a in structural steel in tension, or in compression in a cross-section in Class 1, 2, or 3:
(c) f_y/γ_{Rd} in structural steel in compression in an effective cross-section in Class 4, where γ_{Rd}=1.10;
(d) f_{sk}/γ_s in reinforcement in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected.

(4) Where unpropped construction is used, stresses due to actions on the structural steel work alone shall be added to stressed due to actions on the composite member.

(5) Where unpropped construction is used, the elastic resistance to bending $M_{el.Rd}$, for a particular cross-section and a loading that causes bending moments M_a in the steel member and M_c in the composite member, shall be calculated as follows. Let r be the highest of the ratios of total bending stress (4), above to limiting stress ((3), above). Then,

$$M_{el.Rd} = (M_a + M_c)/r \qquad (4.1)$$

4.4.2 Vertical Shear

4.4.2.1 *Scope*

Sections 4.4.2 to 4.4.5 apply to composite beams with a rolled or welded structural steel section with a solid web, without longitudinal stiffeners. The web may have transverse stiffeners. In welded sections, the steel flanges are assumed to be plates of rectangular cross-section.

4.4.2.2 *Design Methods*

(1) The resistance to vertical shear shall be taken as the resistance of the structural steel section in accordance with Section 4.6.1.2 of EBCS 3, unless the value of a contribution from the reinforced concrete part of the beam has been established.

(2) The shear forces resisted by the structural steel section shall satisfy:

$$V_{Sd} \leq V_{pl.Rd} \qquad (4.2)$$

where $V_{pl.Rd}$ is the design plastic shear resistance given by:

$$V_{pl.Rd} = A_v(f_y\sqrt{3})/\gamma_a \qquad (4.3)$$

where A_v is the shear area of the structural steel member, given in Section 4.6.1.2 of EBCS 3.

(3) In addition, the shear buckling resistance of a steel web shall be verified as specified in Section 4.4.4 where:

(a) For an unstiffened and uncased web, $d/t_w > 69\epsilon$
(b) For an unstiffened web encased in accordance with Section 4.3.1,
$d/t_w > 124\epsilon$
(c) For a stiffened and uncased web,
$d/t_w > 30\epsilon\sqrt{k_\tau}$
(d) For a stiffened and encased web, d/t_w exceeds both of the two preceding limits;

where d is the depth of the web as for rolled sections and for welded sections
t_w is the thickness of the web
k_τ is the buckling factor for shear given in Section 5.6.3 of EBCS 3

$\epsilon = \sqrt{(235/f_y)}$, with f_y in MPa.

4.4.3 Bending and Vertical Shear

(1) Where the vertical shear V_{Sd} exceeds half the plastic shear resistance $V_{pl.Rd}$ given by Section 4.4.2, allowance shall be made for its effect on the resistance moment.

(2) Except where Section 4.4.2.2(3) is applicable, the following interaction criterion should be satisfied:

$$M_{Sd} \leq M_{f.Rd} + (M_{Rd} - M_{f.Rd})\,[1 - (2V_{Sd}/V_{pl.Rd} - 1)^2] \qquad (4.4)$$

where M_{Sd} and V_{Sd} are the design values,
$V_{pl.Rd}$ is given by Section 4.4.2.2(2),
M_{Rd} is the design bending resistance given by Section 4.4.1,
$M_{f.Rd}$ is the design plastic bending resistance of a cross section consisting of the flanges only, with effective sections as used in the calculation of M_{Rd}.

(3) The interaction is illustrated in Fig. 4.6.

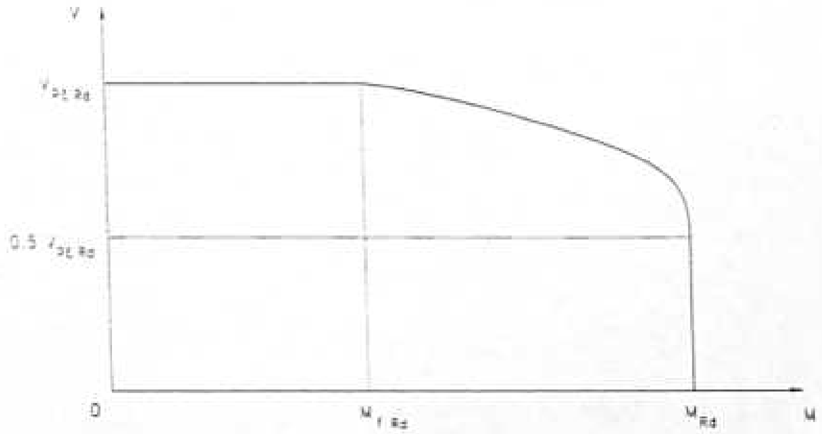


Figure 4.6 Resistance in Bending and Vertical Shear in Absence of Shear Buckling

4.4.4 Shear Buckling Resistance

(1) The principles of paragraphs Section 4.6.4.1 and of EBCS 3 are applicable.

(2) For composite beams, steel webs that shall be verified for shear buckling resistance are defined in Section 4.4.2.2(3).

(3) Webs shall be provided with transverse stiffeners at supports where:

(a) For uncased webs, $d/t_w > 69\epsilon$
(b) For webs encased in accordance with 4.3.2, $d/t_w > 124\epsilon$,

The symbols are as defined in Section 4.4.2.2(3).

(4) No contribution from web encasement to the shear resistance of a web with $d/t_w > 124\epsilon$ shall be assumed, unless verified by testing.

(5) No account shall be taken of a contribution from the concrete slab to the anchorage of a web tension field in a flange, unless the shear connection is designed for the relevant vertical force.

(6) For unstiffened webs and for webs with transverse stiffeners only, the methods given in Sections 4.6.4.1 to 4.6.4.5 of EBCS 3 are applicable, with γ_{M1} for structural steel taken as the value given in Section 5.1.1 of EBCS 3. References to flanges in these clauses are to structural steel flanges only.

(7) For simply-supported beams without intermediate stiffeners, with full shear connection, and subjected to uniformly-distributed loading, the method of Section 4.1.1 of EBCS 3 as modified by paragraphs (a) to (d) below may alternatively be used.

(a) The simple post-critical shear strength τ_{ba} should be determined as follows:

(i) for $\bar{\lambda}_w \leq 1.5$, $\tau_{ba} = f_{yw}/\sqrt{3}$ (4.5a)

(ii) for $1.5 < \bar{\lambda}_w < 3.0$, $\tau_{ba} = (f_{yw}/\sqrt{3})\,(3/\bar{\lambda}_w + 0.2\bar{\lambda}_w - 1.3)$ (4.5b)

(iii) for $3.0 \leq \bar{\lambda}_w \leq 4.0$, $\tau_{ba} = (f_{yw}/\sqrt{3})\,(0.9/\bar{\lambda}_w)$ (4.5c)

where f_{yw} is the nominal yield strength of the steel web and
$\bar{\lambda}_w$ is the web slenderness (not exceeding 4.0) defined in Section 4.6.4.2 of EBCS 3.

(b) The number N of shear connectors in each half span should be sufficient to provide full shear connection as defined in Section 4.1.2(6). Where $V_{Sd} > V_{cr}$, the N connectors should not be distributed in accordance with Section 6.1.3, but as shown in Fig. 4.7.

where $V_{cr} = dt_w\tau_{cr}$
τ_{cr} is as defined in Section 4.6.4.2 of EBCS 3
d and t_w are defined in Section 4.4.2.2(3)
$N_2 = N1 - (V_{cr}/V_{Sd})^2$
$N_1 = N - N_2$
b_{eff} is the effective flange width, defined in Section 4.2.2.1

(c) The steel end post should be designed for a uniform axial compressive force equal to the maximum design shear force V_{Sd} for the cross-section, considering stability both in and out of the plane of the web.

(d) The welds connecting the web to he end post and to a length 1.5 b_{eff} of the steel top flange should be designed for a shear force $(f_{yw}/\sqrt{3})t_w$ per unit length of web.

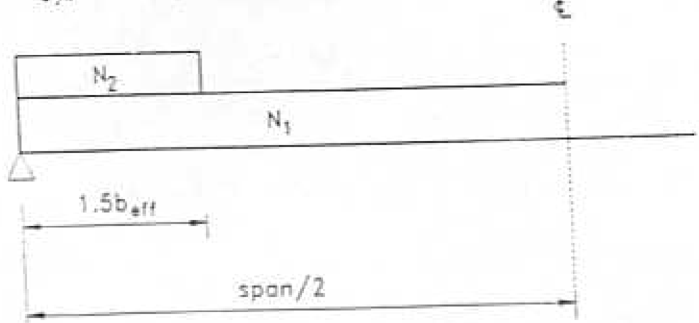


Figure 4.7 Distribution of Shear Connectors

4.4.5 Interaction Between Bending and Shear Buckling

Section 4.6.5.2(2) of EBCS 3 is applicable, with the following modifications, to composite beams in which the design axial force $N_{sd} = 0$.

(a) The word "flange' refers to both steel and composite flanges.
(b) Where the method of 4.4.4(7) is applicable, $V_{ba,Rd}$ may be taken as the design shear buckling resistance given by that method.
(c) The symbol $M_{pl,Rd}$ in Sections 4.6.5.2(2) and 4.6.5.3(5) of EBCS 3 should be replaced by M_{RD}, which is the design bending resistance of the composite section given by 4.4.1.
(d) Where the tension field method is used, Section 4.6.5.3(5) EBCS 3 may be assumed to be applicable to a composite section in which the steel member has equal flanges.

4.5 INTERNAL FORCES AND MOMENTS IN CONTINUOUS BEAMS

4.5.1 General

(1) Where bending moment is applied to a beam through a connection to a supporting column, the global analysis should be in accordance with Section 4.9.

(2) Plastic global analysis may be used for continuous beams in buildings where the requirements of Section 4.5.2 are satisfied.

(3) Elastic global analysis may be used for all continuous beams. For beams in buildings, bending moments found by elastic analysis may be redistributed in accordance with Section 4.5.3.4.

(4) The effects of slip and uplift may be neglected at interfaces between steel and concrete at which shear connection is provided in accordance with Chapter 6.

4.5.2 Plastic Analysis

4.5.2.1 *General*

(1) Plastic global analysis may be carried out using either rigid-plastic or elastic-plastic methods.

(2) The following methods of elastic-plastic analysis may be used:

(a) Elastic/perfectly-plastic
(b) Elasto-plastic.

(3) Elastic-plastic methods of analysis shall satisfy the principles of Section 4.1.1. Elasto-plastic methods of analysis shall take account of the load/slip behaviour of the shear connection. No application rules are given for these methods.

4.5.2.2 Requirements for Rigid-Plastic Analysis

(1) At each plastic hinge location:

(a) The cross-section of the structural steel component shall be symmetrical about the plane of its web
(b) The rotation capacity shall be sufficient to enable the required hinge rotation to develop

(c) The proportions and restraints of steel components shall be such that lateral-torsional buckling does not occur
(d) Lateral restraint shall be provided

(2) For composite beams in buildings, requirements (b) and (d) may be assumed to be satisfied when:

(a) 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, excluding effective webs to Section 4.3.3.1(3);
(b) Adjacent spans do not differ in length by more than 50% of the shorter span;
(c) End spans do not exceed 115% of the length of the adjacent span;
(d) In any span in which more than half of the total design load 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;
(e) The steel compression flange at a plastic hinge location is laterally restrained.

Condition (d) does not apply where it can be shown that the hinge will be the last to form in that span.

4.5.3 Elastic Analysis

4.5.3.1 *General*

(1) Elastic global analysis shall be based on the assumption that the stress-strain relationships for the materials are linear, whatever the stress level. The tensile strength of concrete may be neglected.

(2) For beams in buildings, flexural stiffnesses may be taken as the "uncracked" values $E_a I_1$ throughout the length of a beam. Alternatively, flexural stiffnesses may be taken as the "cracked" values $E_a I_2$ over 15% of the span on each side of each internal support, and as the values $E_a I_1$ elsewhere. These methods are defined as "uncracked" and "cracked" elastic analysis, respectively. The stiffnesses $E_a I_1$ and $E_a I_2$ are defined in Section 4.2.3(2).

4.5.3.2 *Sequence of Construction*

Where unpropped construction is used for structures with composite beams that have cross-sections in Class 3 or Class 4, appropriate global analyses shall be made for the separate effects of permanent actions applied to the steel member and actions applied to the composite member.

4.5.3.3 *Effects of Shrinkage of Concrete in Beams for Buildings*

(1) Account shall be taken at cross-sections in Class 4 of the bending moments due to restraints from supports of deformations caused by shrinkage of the concrete slab.

4.5.3.4 *Redistribution of Moments in Beams for Buildings*

(1) The design bending moment distribution given by an elastic analysis may be redistributed in a way that satisfies equilibrium, and takes account of the effects of cracking of concrete, inelastic behaviour of materials, and local buckling of structural steelwork.

(2) (a) The elastic bending moments for a continuous composite beam of uniform depth within each span may be modified:

(i) by reducing maximum hogging moments by amounts not exceeding the percentages given in Table 4.3; or

(ii) in beams with all cross-sections in Classes 1 or 2 only, by increasing maximum hogging moments by amounts not exceeding 10%, for "uncracked" elastic analysis, or 20%, for "cracked" elastic analysis.

(b) For each load case, the internal forces and moments after redistribution should be in equilibrium with the loads.

(c) For composite cross sections in Class 3 or 4, the Figures in Table 4.3 relate to bending moments assumed in design to be applied to the composite member. Moments applied to the steel member should not be redistributed.

Table 4.3 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 "uncracked" elastic analysis	40	30	20	10
For "cracked" elastic analysis	25	15	10	0

4.6 LATERAL-TORSIONAL BUCKLING OF COMPOSITE BEAMS FOR BUILDINGS

4.6.1 General

(1) A steel flange that is attached to a concrete or composite slab by shear connection in accordance with Chapter 6 may be assumed to be laterally stable, provided that the overall width of the slab is not less than the depth of the steel member.

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

(3) In checks for lateral stability of beams built unpropped, the bending moment at any cross section shall be taken as the sum of the moment applied to the composite member and the moment applied to its structural steel component.

4.6.2 Check without Direct Calculation

A continuous beam or a beam in a frame that is composite throughout its length may be designed without additional lateral bracing when the following conditions are satisfied.

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

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

(c) The top flange of the steel member is attached to a reinforced concrete or composite slab by shear connectors in accordance with Chapter 6.

(d) The longitudinal spacing of studs or rows of studs s is such that for uncased beams

$$s/b \leq 0.02\, d^2 h / t_w^3 \qquad (4.6)$$

where d is the diameter of the studs, and
b, h, and t_w are as shown in Fig. 4.8.

For steel members party encased in concrete in accordance with Section 4.3.2, the spacing should not exceed 50% of the maximum spacing for the uncased beam.

(e) The longitudinal spacing of connectors other than studs is such that the resistance of the connection to transverse bending is not less than that required when studs are used.

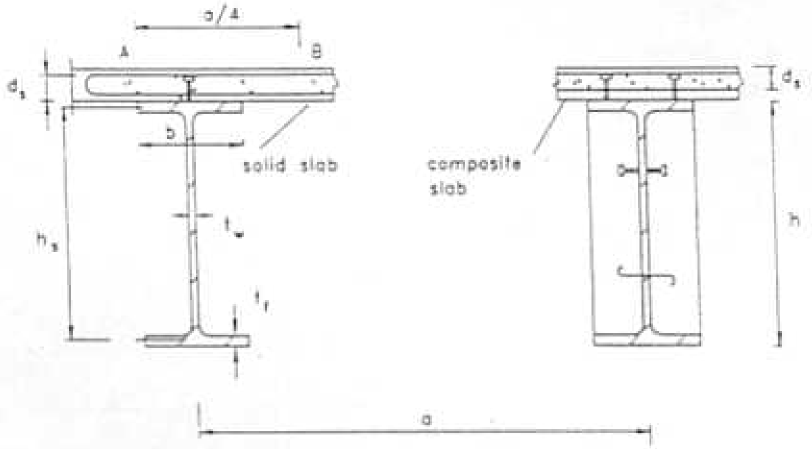


Figure 4.8 Lateral-Torsional Buckling

(f) The same slab is also attached to another supporting member approximately parallel to the composite beam considered, to form an inverted-U frame of breadth a (Fig. 4.8).

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

(h) Where the slab is simply supported at the composite beam considered, fully anchored top reinforcement extends over the length AB shown in Fig.4.8. The area of this reinforcement should be such that the resistance of the slab to hogging transverse bending, per unit length of beam, is not less than $f_y t_w^3/4\gamma_a$, where the notation is as in (d) above.

(i) At each support of the steel member, its bottom flange is laterally restrained and its web is stiffened. Elsewhere, the web is unstiffened.

(j) The bending stiffness of the solid or composite slab is such that

$$E_{cm} I_{c2} \geq 0.35\, E_a t_w^3\, a/h \qquad (4.7)$$

where $E_{cm} I_{c2}$ is the mean of the flexural stiffnesses per unit width of slab at midspan and above the steel beam considered, neglecting concrete in tension, and including transformed areas of reinforcement and any profiled sheeting that contributes to the resistance $M_{c,Rd}$ in accordance with Section 7.6.1.2;

E_{cm} is as defined in Section 3.1.5.2;

E_a is as defined in Section 3.3.4; and

t_w, a, and h are as shown in Fig.4.8.

(k) The steel member is an IPE section or an HE section or another hot-rolled section of similar shape with $A_w/A_a \leq 0.45$, the same depth h, and

$$\left[\frac{h_s}{t_w}\right]^3 \frac{t_f}{b} \leq 10^4\, \epsilon^4 \qquad (4.8)$$

where $A_w = h_i t_w$

$\epsilon = \sqrt{235/f_y}$ as in Table 4.1 and 4.2,

A_a is the area of the structural steel section, and
h_i, t_w, t_f and b are as shown in Fig. 4.8.

(l) If the steel member is not partly encased, its depth h is in accordance with Table 4.4.
(m) If the steel member is partly encased in concrete in accordance with Section 4.3.1, its depth h does not exceed the limit given in Table 4.4 by more than 200 mm.

Table 4.4 Maximum Depth h (mm) of Uncased Steel Member for which Section 4.6.2 if Applicable

Steel Member	Nominal Steel Grade		
	Fe 360	Fe 430	Fe 510
IPE or similar	≤ 600	≤ 550	≤ 400
HE or similar	≤ 800	≤ 700	≤ 650

4.6.3 Buckling Resistance Moment

(1) The design buckling resistance moment of a laterally unrestrained beam shall be taken as

$$M_{b,Rd} = \chi_{LT} M_{pl,Rd} (\gamma_a / \gamma_{Rd}) \qquad (4.9a)$$

for a Class 1 or Class 2 cross-section, with $\gamma_{Rd} = 1.10$,

$$M_{b,Rd} = \chi_{LT} M_{el,Rd} (\gamma_a / \gamma_{Rd}) \qquad (4.9b)$$

for a Class 3 cross-section, with $\gamma_{Rd} = 1.10$ and

$$M_{b,Rd} = \chi_{LT} M_{el,Rd} \qquad (4.9c)$$

for Class 4 cross-section,

where χ_{LT} is the reduction factor for lateral-torsional buckling,
$M_{pl,Rd}$ is the plastic resistance moment given by Section 4.4.1.2 or Section 4.4.1.3,
$M_{el,Rd}$ is the elastic resistance to bending given by Section 4.4.1.4.

(2) Values of χ_{LT} for the appropriate slenderness $\overline{\lambda}_{LT}$ may be obtained from Table 4.9 in EBCS 3 with $\overline{\lambda} = \overline{\lambda}_{LT}$ and $\chi = \chi_{LT}$, using:

column *a* for rolled sections
column *c* for welded beams,

or may be determined from

$$\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \overline{\lambda}_{LT}^2)^{1/2}} \quad \text{but} \quad \chi_{LT} \leq 1,$$

where $\phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - 0.2) + \overline{\lambda}^2_{LT}]$
$\alpha_{LT} = 0.21$ for rolled sections
$\alpha_{LT} = 0.49$ for welded beams,

(3) The value of $\overline{\lambda}_{LT}$ may be determined from

$\overline{\lambda}_{LT} = (M_{pl}/M_{cr})^{1/2}$ for a Class 1 or Class 2 cross-section,
$\overline{\lambda}_{LT} = (M_{el}/M_{cr})^{1/2}$ for a Class 3 or Class 4 cross-section,

where M_{pl} is the value of $M_{pl.Rd}$ when the γ_M factors γ_a, γ_c, and γ_s are taken as 1.0,
M_{el} is the value of $M_{el.Rd}$ when the γ_M factors γ_a, γ_c, and γ_s are taken as 1.0,
M_{cr} is the elastic critical moment for lateral-torsional buckling

4.6.4 Simplified Method of Calculation of the Slenderness Ratio and the Elastic Critical Moment

4.6.4.1 *Slenderness Ratio*

(1) For uncased beams that satisfy the conditions of Section 4.6.4.2(1) and have a double symmetrical steel section, the slenderness ratio $\overline{\lambda}_{LT}$ for a Class 1 or Class 2 cross-section may conservatively be taken as:

$$\overline{\lambda}_{LT} = 5.0\left[1 + \frac{t_w h_s}{4 b_f t_f}\right]\left[\left[\frac{f_y}{E_a C_4}\right]^2 \left[\frac{h_s}{t_w}\right]^3 \left[\frac{t_f}{b_f}\right]\right]^{1/4} \qquad (4.10)$$

where f_y is the yield strength of the structural steel, and the other symbols are defined in Section 4.6.4.2 or Fig. 4.9.

(2) For a cross-section in Class 3 or Class 4, the value given in (1) above should be multiplied by $(M_{el}/M_{pl})^{1/2}$, in accordance with Section 4.6.3(3).

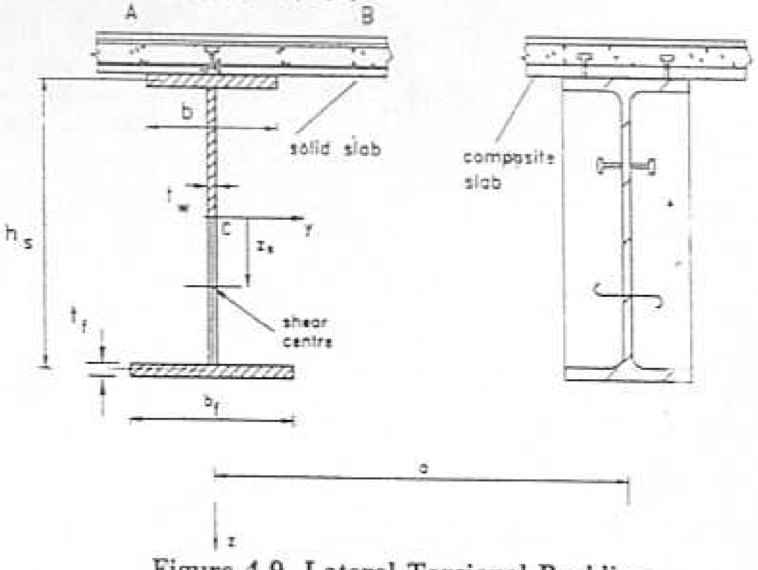


Figure 4.9 Lateral-Torsional Buckling

4.6.4.2 *Elastic Critical Moments*

(1) This Section is applicable to a composite beam with continuity at one or both ends and a restrained top flange, that satisfies conditions (c) and (f) to (j) of Section 4.6.2. The steel member should be a double symmetrical or mono-symmetrical rolled or welded I-section, uniform throughout the span considered. The shear connection should satisfy (6) and (7) below.

(2) The model for this method is the continuous inverted-U frame. It does not rely on the provision of web stiffeners, except those required by Section 4.6.2(i).

(3) No special provision need be made at internal supports to provide warping fixity or to prevent rotation on plan of the steel bottom flange.

(4) The elastic critical hogging moment M_{cr} at an internal support may be taken as:

$$M_{cr} = \frac{k_c C_4}{L} \left[(G I_{at} + k_s L^2/\pi^2) \, E_a I_{afz} \right]^{1/2} \qquad (4.11)$$

where L is the length of the beam between points at which the bottom flange of the steel member is laterally restrained,

C_4 is a property of the distribution of bending moment within length L given in Tables 4.5 to 4.7. Where the bending moments at the supports are unequal, C_4 relates to the support with the larger hogging moment.

(5) The properties of the effective cross-section in the hogging moment region are as follows:

k_c is a factor given in Section 4.6.4.3 or 4.6.4.4.

E_a and G are respectively the modulus of elasticity and the shear modulus for steel, given in Section 3.3.4;

A is the area of the equivalent composite section, as defined in Section 4.2.3(1), neglecting concrete in tension;

I_y is the second moment of area for major-axis bending of the composite section of area A;

A_a is the area of the structural steel section;

I_{ay} and I_{az} are second moments of area of the structural steel section about its centre of area, C;

$i_x^2 = (I_{ay} + I_{az})A_a$;

I_{afz} is the second moment of area of the bottom flange about the minor axis of the steel member;

I_{at} is the St. Venant torsion constant of the steel section;

k_s is a transverse stiffness per unit length of the beam, given by

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

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

$$k_1 = 4E_a I_2 / a$$

for a slab continuous across the steel beam and

$$k_1 = 2E_a I_2 / a$$

for a simply supported or cantilever slab;

$E_a I_2$ is the "cracked" flexural stiffness per unit width of the concrete or composite slab, as defined in Section 4.2.3(2);
and I_2 should be taken as the lower of:

- the value at midspan, for sagging bending, and
- the value at an internal support, for hogging bending;

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

$$k_2 = \frac{E_a t_w^3}{4(1 - \nu_a^2)h_s} \tag{4.12a}$$

for an uncased beam and as

$$k = \frac{E_a t_w b^2}{16h_s(1 + 4nt_w/b)} \tag{4.12b}$$

for a beam partly encased in concrete in accordance with Section 4.3.1(6) to (9);

n is the modular ratio E_a/E_c;

E_c is the effective modulus for concrete for long term effects, given in Section 3.1.4.2(3) or (4);

ν_a is the poison's ratio for steel;

b is the breadth of the top flange of the steel member;

h_s is the distance between the shear centres of the flanges of the steel member;

and other symbols are defined in Fig. 4.9.

(6) Except where specific account is taken of the influence of inverted-U frame action on the resistance of the shear connection, the longitudinal spacing of studs or rows of studs s should be such that

$$\frac{s}{b} \leq \frac{0.4 f_u d^2(1 - \chi_{LT}\overline{\lambda}_{LT}^2)}{k_s \chi_{LT}\overline{\lambda}_{LT}^2} \tag{4.13}$$

where d is the diameter of the studs,
f_u is the tensile strength of the studs, as defined in 6.3.2.1,
χ_{LT} and $\overline{\lambda}_{LT}$ are as given in Section 4.6.3,
k_s is as defined in Section 4.6.4.2(5),
b is as shown in Fig. 4.9.

(7) The longitudinal spacing of connectors other than studs should be such that the resistance of the connection to transverse bending is not less than that required when studs are used.

4.6.4.3 *Double Symmetrical Steel Sections*

Where the cross-section of the steel member is symmetrical about both axes, the factor k_c in Section 4.6.4.2 is given by:

$$k_c = \frac{h_s I_y / I_{ay}}{\dfrac{h_s^2/4 + i_x^2}{e} + h_s} \tag{4.14}$$

where $$e = \frac{AI_{ay}}{A_a z_c (A - A_a)}$$

z_c is the distance between the centre of area of the steel member and mid-depth of the slab.

and other symbols are defined in Section 4.6.4.2.

4.6.4.4 *Mono-Symmetrical Steel Sections*

Where the cross-section of the steel member has unequal flanges, the factor k_c in Section 4.6.4.2 is given by:

$$k_c = \frac{h_s I_y / I_{ay}}{\frac{(z_f - z_s)^2 + i_x^2}{e} + 2(z_f - z_j)} \qquad (4.15)$$

where $z_f = h_s I_{afz} / I_{az}$

$$z_j = z_s - \int_{A_s} \frac{z(y^2 + z^2)dA}{2I_{ay}}$$

and may be taken as $z_j = 0.4h_s(2\, I_{afz}/I_{az} - 1)$
when $I_{afz} > 0.5\, I_{az}$;

Z_s is the distance from the centroid of the steel section (c in Fig. 4.5) to its shear centre, positive when the shear centre and the compression flange are on the same side of the centroid;

and other symbols are defined in Sections 4.6.4.2 or 4.6.4.3

4.6.4.5 *Alternative Methods of Calculation*

(1) Where a beam does not comply with the condition of Section 4.6.4.1 to 4.6.4.4 above, the value of M_{cr} shall be determined from speciality literature, or by numerical analysis, or (conservatively) by determining M_{cr} from Section 4.6.3.2 of EBCS 3 for the steel member above.

(2) Where the slenderness $\overline{\lambda}_{LT} \leq 0.4$, no allowance for lateral-torsional buckling is necessary.

4.7 WEB CRIPPLING

4.7.1 General

(1) The principles of Section 4.6.6 of EBCS 3 are applicable to non-composite steel flanges of composite beams, and to the adjacent part of the web.

(2) The application rules of Section 4.6.6 of EBCS 3 are applicable to non-composite steel flanges of composite beams, and to the adjacent part of the web.

4.7.2 Effective Web in Class 2

At an internal support of a beam designed using an effective web in Class 2 (in accordance with 4.3.3.1(3)), transverse stiffening should be provided unless it can be shown that the unstiffened web has sufficient resistance to web crippling.

Table 4.5 Value of Factor c_4 for spans with Transverse Loading

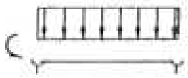
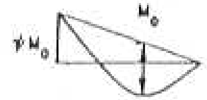
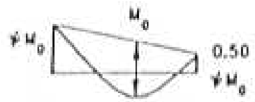
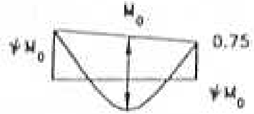
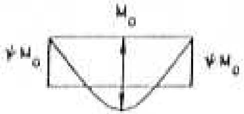
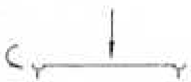
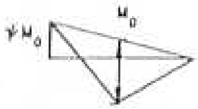
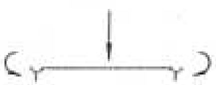
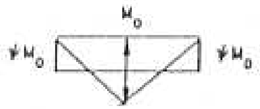
Loading and support conditions	Bending moment diagram	c_4								
		$\psi = 0.50$	$\psi = 0.75$	$\psi = 1.00$	$\psi = 1.25$	$\psi = 1.50$	$\psi = 1.75$	$\psi = 2.00$	$\psi = 2.25$	$\psi = 2.50$
		41.5	30.2	24.5	21.1	19.0	17.5	16.5	15.7	15.2
		33.9	22.7	17.3	14.1	13.0	12.0	11.4	10.9	10.6
		28.2	18.0	13.7	11.7	10.6	10.0	9.5	9.1	8.9
		21.9	13.9	11.0	9.6	8.8	8.3	8.0	7.8	7.6
		28.4	21.8	18.6	16.7	15.6	14.8	14.2	13.8	13.5
		12.7	9.8	8.6	8.0	7.7	7.4	7.2	7.1	7.0

Table 4.6 Values of Factor C_4 for Spans without Transverse Loading

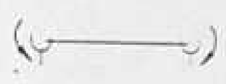
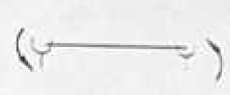
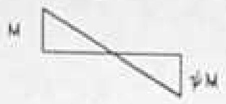
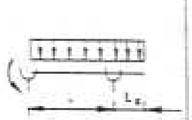
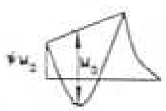
Loading and support conditions	Bending moment diagram	C_4				
		ψ = 0.00	ψ = 0.25	ψ = 0.50	ψ = 0.75	ψ = 1.00
		11.1	9.5	8.2	7.1	6.2
		11.1	12.8	14.6	16.3	18.1

Table 4.7 Values of Factor C_4 at end Supports, for Spans with a Cantilever Extension

Loading and support conditions	Bending moment diagram	L_c / L	C_4			
			ψ = 0.00	ψ = 0.50	ψ = 0.75	ψ = 1.00
		0.25	47.6	33.8	26.6	22.1
		0.50	12.5	11.0	10.2	9.3
		0.75	9.2	8.8	8.6	8.4
		1.00	7.9	7.8	7.7	7.6

4.8 COMPOSITE COLUMNS

4.8.1 Scope

(1) The steel section and the uncracked concrete section usually have the same centroid. Typical types of cross-section are shown in Fig. 4.10.

(a) Concrete encased sections (steel section completely covered by concrete as in Fig. 4.10a),
(b) Concrete filled sections (concrete completely covered by steel as in Fig. 4.10d-f),
(c) Partially encased sections (steel section partially covered by concrete as in Fig. 4.9b and c).

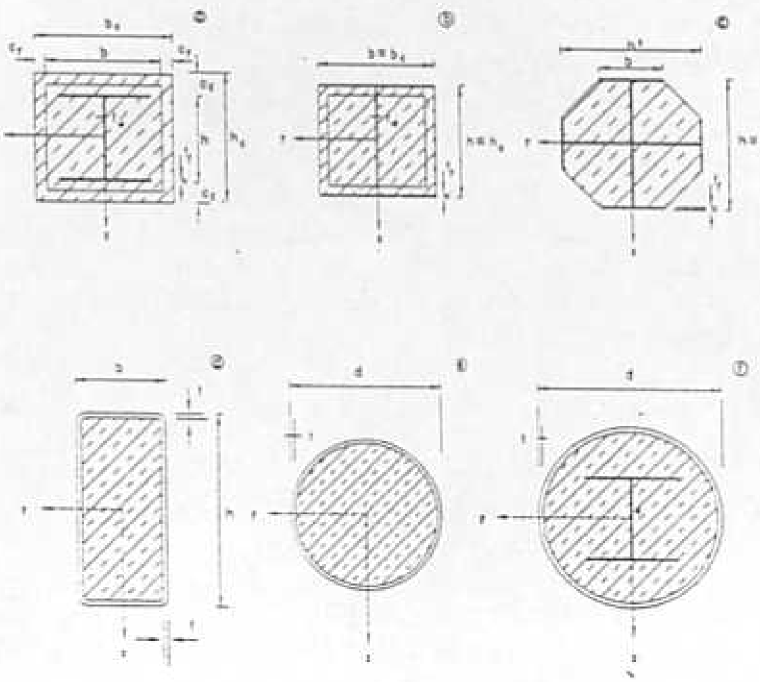


Figure 4.10 Typical Cross Section of Composite Columns, with Notation

(2) Section 4.8 applies to isolated non-sway columns. These may be:

(a) Compression members which are integral parts of a non-sway frame but which are considered to be isolated for design purposes, or
(b) Isolated compression members that satisfy the classification "non-sway" as given in Section 4.4.4.2 of EBCS 2 or Section 4.2.5.2 of EBCS 3 as appropriate.

(3) Two methods of design are given:

(a) A general method in Section 4.8.2 including columns with non-symmetrical or non-uniform cross section over the column length,

(b) A simplified method in Section 4.8.3 for columns of double symmetrical and uniform cross section over the column length using the Strut Curves in EBCS 3. Application rules for columns of mono-symmetrical section are given in Section 4.8.4.

4.8.2 General Method of Design

4.8.2.1 *General*

A composite column of any cross-section, loaded by normal forces and bending moments, shall be checked for:

(a) Resistance of member (Sections 4.8.2.2 to 4.8.2.3)
(b) Resistance to local buckling (Section 4.8.2.4)
(c) Introduction of loadings (Section 4.8.2.6)
(d) Resistance to shear (Sections 4.8.2.7 to 4.8.2.8).

4.8.2.2 *Design Procedures*

(1) Design for structural stability shall take account of second order effects including imperfections and shall ensure that, for the most unfavourable combinations of actions at the ultimate limit state, instability does not occur, and that the resistance of individual cross-sections subjected to bending and longitudinal force is not exceeded.

(2) The partial safety factors γ_M are as given in Sections 2.3.3.2(1) and 4.1.1(5), except that γ_c for concrete may be reduced where (12) below applies.

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

(4) In accordance with Section 4.4.10.3 of EBCS 2, the influence of second order effects should be considered if the increase above the first order bending moments, due to deflections within the column length, exceeds 10%. In this check, creep effects should be treated according to (9) and (10) below.

(**Note:** This check should be made by second-order elastic analysis of the column length, with its ends assumed to be pinned and subjected to the internal forces and moments determined by the global analysis, and with transverse loading, if any).

(5) Plane sections shall be assumed to remain plane. Full composite action up to failure shall be assumed between the steel and concrete components of the member.

(6) The following stress-strain relationships should be used in the (non-linear) analysis:

(a) For concrete as given in Section 3.1.5,
(b) for reinforcing steel as given in Section 3.2.4, and
(c) for structural steel as given in Section 3.3.

(7) Where second-order deformations are being calculated, the stress-strain diagram for concrete given in Section 4.2.3 of EBCS 2 should be used with f_{cd} and E_{cd} taken as:

$$f_{cd} = f_{ck}/\gamma_c$$
$$E_{cd} = E_{cm}/\gamma_c$$

(**Note:** This paragraph does not apply to calculation of resistances of cross-sections.)

(8) Shrinkage and creep effects shall be considered if they are likely to reduce the structural stability significantly.

(9) For simplification, creep effects may be ignored if the increase in the first order bending moments due to creep deformations and longitudinal force resulting from permanent loads does not exceed 10%.

(10) Creep deformations of slender compression members in non-sway frames for buildings with monolithic connections to slabs or beams at their two ends may normally be disregarded.

(11) The contribution of the tensile strength of the concrete between cracks (tension stiffening) may be taken into account.

(12) Partial safety factors for materials within precast concrete elements shall be in accordance with the appropriate parts of EBCS 2.

4.8.2.3 *Imperfections*

(1) Imperfections within the column length shall be taken into account for the calculation of the internal forces and moments.

(2) The equivalent initial bow imperfections should be related to the following buckling curves from EBCS 3:

(a) Curve *a* for concrete-filled hollow sections
(b) Curve *b* for fully or partly concrete-encased I-sections with bending about the strong axis of the steel section
(c) Curve *c* for fully or partly concrete-encased I-sections with bending about the weak axis of the steel section
(d) Curve *d* for other concrete-encased steel sections.

4.8.2.4 *Local Buckling of Steel Members*

(1) The influence of local buckling of steel members on the resistance of the column shall be considered in design.

(2) The effects of local buckling of steel members in composite columns may be neglected for steel sections fully encased according to Section 4.8.2.5 and for other types of composite columns, provided that:

(a) For circular hollow steel sections, $d/t \leq 90\epsilon^2$
(b) For rectangular hollow steel sections, $h/t \leq 52\epsilon$
(c For partly-encased I-sections, $b/t_f \leq 44\epsilon$

where, as shown in Fig. 4.10

d	is the external diameter of a circular hollow steel section,
h	is the greater overall dimension of the section parallel to a principal axis,
t	is the thickness of the wall of a circular hollow steel section,

t_f and b	are the thickness and overall breadth of the flange of a steel I-section or a similar section,
ϵ	$= \sqrt{235/f_y}$
f_y	is the yield strength of steel in MPa

(3) If the values in (2) are exceeded, the effect of local buckling should be taken into account by an appropriate experimentally confirmed method.

4.8.2.5 *Cover and Reinforcement*

(1) For fully-encased steel sections at least a minimum reinforced concrete cover shall be provided to ensure:

(a) The safe transmission of bond forces
(b) The protection of the steel against corrosion
(c) That spalling will not occur
(d) An adequate fire resistance, in accordance with international standards.

(2) The concrete cover to a flange of a fully-encased steel section should be not less than 40mm, nor less than one-sixth of the breadth b of the flange. The cover to reinforcement should be in accordance with Section 7.1.3 of EBCS 2.

(3) 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.

(4) The transverse reinforcement in concrete-encased columns should be designed according to Section 7.2.4.3 EBCS 2.

(5) For the spacing of the reinforcement Section 7.2.4 EBCS 2 applies.

(6) The clear distance between longitudinal reinforcing bars and the structural steel section may be smaller than required by (5), even zero. In this case, for bond the effective perimeter, c, of the reinforcing bar should be taken as half or one quarter of its perimeter as shown in Fig. 4.11 at (a) and (b) respectively.

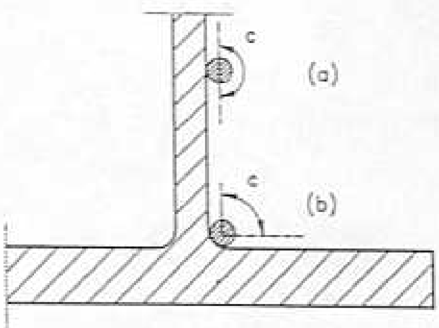


Figure 4.11 Effective Perimeter c of a Reinforcing Bar

(7) Welded mesh reinforcement may be used as links in concrete-encased columns, but should not contribute to or supply the longitudinal reinforcement.

(8) In concrete-filled hollow sections normally no longitudinal reinforcement is necessary.

4.8.2.6 *Shear Between the Steel and Concrete Components*

(1) Provision shall be made for internal forces and moments applied from members connected to the ends of a column length to be distributed between the steel and concrete components of the column, considering the shear resistance at the interface between steel and concrete according to Section 4.8.2.7.

(2) 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.

(3) The introduction length for the shear force should not be assumed to exceed twice the relevant transverse dimension.

(4) In an I-section with concrete only between the flanges the concrete should be gripped by stirrups and a clearly defined load transmission path between concrete and steel web should be identified (i.e., stirrups should pass through the web or be welded to the web, or should interlock with shear connectors).

(5) Where composite columns are subjected to significant transverse shear, as for example by local horizontal loads, provisions shall be made for the transfer of the corresponding longitudinal shear stress at the interface between steel and concrete.

(6) In absence of a more accurate method, elastic analysis of the uncracked composite section, considering the sequence of construction, should be used to estimate longitudinal shear stress due to transverse shear between the steel and concrete.

(7) Calculated resultant shear stresses at the interface between steel and concrete should nowhere be excessive according to Section 4.8.2.7.

4.8.2.7 *Resistance to Shear*

(1) The shear resistance shall be provided by bond stresses and friction at the interface or by mechanical shear connection, such that no significant slip occurs.

(2) The design shear strength due to bond and friction should be taken as:

(a) For completely concrete encased sections	0.6 MPa
(b) For concrete filled hollow sections	0.4 MPa
(c) For flanges in partially encased sections	0.2 MPa
(d) For webs in partially encased sections	zero.

(3) Alternatively it may be shown by tests that full interaction can be relied upon until failure of the member.

4.8.2.8 *Stud Connectors Attached to the Web of a Composite Column*

(1) Where stud connectors are attached to the web of a concrete-encased steel I-section (Fig. 4.12) or a similar section, the lateral expansion of the concrete against which they bear is prevented by the adjacent steel flanges. The resulting frictional forces provide resistance in longitudinal shear additional to that given by Section 6.3.2.

(2) This additional resistance may be assumed to be $\mu P_{Rd}/2$ on each flange, for each row of studs, as shown in Fig. 4.12, where P_{Rd} is the design resistance of one stud, defined in Section 6.3.2. The value of μ is the coefficient of friction which may be taken as 0.50 for steel flanges not less than 10mm thick and 0.55mm for steel flanges not less than 15mm thick, blasted with shot or grit, with loose rust removed and no pitting.

(3) In absence of better information from tests these values should only be allowed when the clear distance between the flanges as shown in Fig. 4.12 does not exceed.

(a) 300mm using one stud per row,
(b) 400mm using two studs per row,
(c) 600mm using three or more studs.

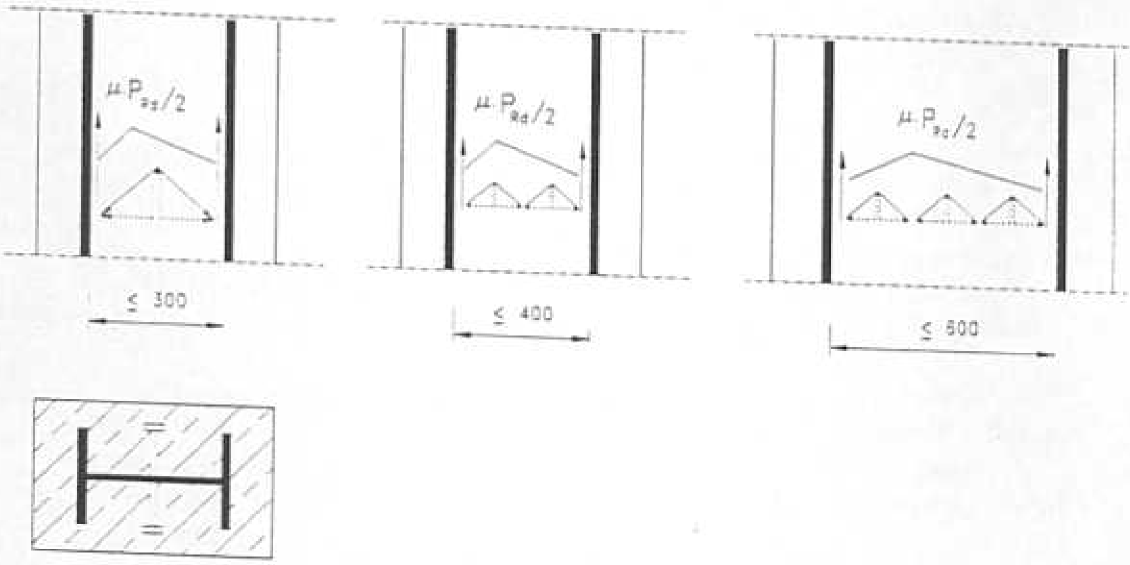


Figure 4.12 Stud Connectors in Composite Column

4.8.3 Simplified Method of Design

4.8.3.1 *Scope*

(1) The method given in Section 4.8.3, if applied, should be used as a whole in accordance with Section 4.8.1(2). If individual clauses are used as part of another method, the applicability needs to be checked.

(2) As this method takes account of imperfections within the column length, allowance need not again be made for these, but all other provisions of Section 4.8.2 apply when the method of Section 4.8.3 is used, except Section 4.8.2.2(4) and 4.8.2.2(9).

(3) The scope of this simplified method is limited as follows:

(a) The column is of double-symmetrical and uniform cross section over the column length. (Note: The centres of area of the steel section and the uncracked concrete section therefore coincide. This point is defined as the centroid of the section even when the bending moment is sufficient to cause cracking of concrete. Certain mono-symmetric sections are treated in Section 4.8.3.15).

(b) The steel contribution ratio δ as defined in Section 4.8.3.4 should lie between 0.2 and 0.9. The steel members may be rolled or welded.

(c) The non-dimensional slenderness $\overline{\lambda}$ defined in 4.8.3.7 should not exceed 2.0.

(d) For fully-encased steel sections, limits to the thickness of concrete cover that may be used in the calculations are:

(i) in the y-direction, 40mm $\leq c_y \leq 0.4b$,
(ii) in the z-direction, 40mm $\leq c_z \leq 0.3h$,
where the notation is as shown in Fig. 4.10.

Greater cover can be used but should be ignored in calculation.

(e) The cross-sectional area of longitudinal reinforcement that may be used in the calculations should not exceed 4% of the area of the concrete.

(f) If the longitudinal reinforcement is neglected in calculations for the resistance of the column, the following reinforcement may be assumed to be adequate:

(i) longitudinal bars of minimum diameter 8mm at a maximum spacing of 250mm,
(ii) links with minimum diameter of 6mm and a maximum spacing of 200mm.
(iii) for welded mesh reinforcement the minimum diameters may be reduced to 4mm.

(4) Typical cross sections and relevant notation are shown in Fig. 4.10.

(5) It may be found convenient to verify the design of a composite column in the following sequence.

(a) Check the limitations of scope given in Section 4.8.3(3).
(b) Check for local buckling (Section 4.8.2.4).
(c) Check cover and reinforcement (Section 4.8.2.5).
(d) Calculate N_{cr} and $\overline{\lambda}$ (Section 4.8.3.7) and determine γ_{Ma} from Section 4.8.3.2.
(e) Decide whether second-order analysis for bending moments is required by Section 4.8.3.10.
(f) Verify the resistance of the column following Sections 4.8.3.3, 4.8.3.8, 4.8.3.9, and 4.8.3.11 to 4.8.3.14.
(g) Verify the load introduction and longitudinal shear according to Section 4.8.2.6 to 4.8.2.8.

4.8.3.2 *Partial Safety Factors γ_{Ma}, γ_a, and γ_{Rd}*

(1) In Section 4.8.3 the partial safety factor γ_M for structural steel is written as γ_{Ma}. In accordance with Section 4.1.1(5) it has one of two values, as follows:

(a) For a column length with $\overline{\lambda} \leq 0.2$ or $N_{Sd}/N_{cr} \leq 0.1$,

$$\gamma_{Ma} = \gamma_a = 1.10,$$

where N_{Sd} is the design axial load, and
$\overline{\lambda}$ and N_{cr} are in accordance with Section 4.8.3.7.

(b) Otherwise,

$$\gamma_{Ma} = \gamma_{Rd} = 1.10$$

(2) Exceptions to (1) above are given in particular clauses.

4.8.3.3 *Resistance of Cross Sections to Axial Loads*

(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:

$$N_{pl.Rd} = A_a f_y / \gamma_{Ma} + A_c (0.85 f_{ck} / \gamma_c) + A_s f_{sk} / \gamma_s \tag{4.16}$$

where A_a, A_c and A_s are the cross-sectional areas of the structural steel, the concrete, and the reinforcement, respectively,

f_y, f_{ck} and f_{sk} are their characteristic strengths in accordance with EBCS 3 or EBCS 2.

γ_{Ma}, γ_c, and γ_s are partial safety factors at the ultimate limit states. For precast elements, γ_c and γ_s are given in the appropriate part of EBCS 2.

(2) The plastic resistance of concrete-filled hollow sections $N_{pl.Rd}$ may be calculated with $0.85 f_{ck}$ being replaced by f_{ck}.

(3) For concrete-filled tubes of circular cross section, account may be taken of the increase in strength of concrete caused by confinement provided that;

(i) the relative slenderness $\overline{\lambda}$ given by Section 4.8.3.7 does not exceed 0.5, and
(ii) the greatest design bending moment calculated by first-order theory $M_{max.Sd}$ does not exceed $N_{Sd}\, d/10$, where d is the external diameter of the column.

(4) The plastic resistance to compression may then be calculated from

$$N_{pl.Rd} = A_a \eta_2 f_y / \gamma_{Ma} + A_c (f_{ck} / \gamma_c) [1 + \eta_1 (t/d)(f_y / f_{ck})] + A_s f_{sk} / \gamma_s \tag{4.17}$$

where t is the wall thickness of the steel tube, η_1 and η_2 are coefficients defined below, and the other symbols are defined above.

(5) The eccentricity of loading e is defined as $M_{max.Sd} / N_{Sd}$. The values of η_{10} and η_{20} when $e = 0$ are given in Table 4.8 or may be taken as follows:

$$\eta_{10} = 4.9 - 18.5\,\overline{\lambda} + 17\,\overline{\lambda}^2 \quad (\text{but} \geq 0) \tag{4.18a}$$

$$\eta_{20} = 0.25(3 + 2\,\overline{\lambda}) \qquad (\text{but} \leq 1.0) \tag{4.18b}$$

Table 4.8 Values of η_{10} and η_{20} when $e = 0$

$\overline{\lambda}$	0	0.1	0.2	0.3	0.4	≥ 0.5
η_{10}	4.90	3.22	1.88	0.88	0.22	0.00
η_{20}	0.75	0.80	0.85	0.90	0.95	1.00

(6) The values of η_1 and η_2 for $0 < e \leq d/10$ are as follows:

$$\eta_1 = \eta_{10} (1 - 10e/d) \tag{4.19a}$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})(10e/d). \tag{4.19b}$$

For $e > d/10$, $\eta_1 = 0$ and $\eta_2 = 1.0$.

4.8.3.4 *Steel Contribution Ratio*

The steel contribution ratio is defined as:

$$\delta = (A_a f_y / \gamma_a) / N_{pl.Rd} \tag{4.20}$$

where $N_{pl.Rd}$ is calculated with $\gamma_{Ma} = \gamma_a$

4.8.3.5 *Effective Elastic Flexural Stiffness of Cross Sections*

(1) For short-term loading the effective elastic flexural stiffness of a cross section of a composite column, $(EI)_e$, should be calculated from.

$$(EI)_e = E_a I_a + 0.8\, E_{cd} I_c + E_s I_s \tag{4.21}$$

where	I_a, I_c and I_s	are the second moments of area of the considered bending plane of the structural steel, the concrete (assumed to be uncracked), and the reinforcement, respectively;
	E_a and E_s	are the elastic moduli for the structural steel and the reinforcement;
	$0.8 E_{cd} I_c$	is the effective stiffness of the concrete part;
	E_{cd}	$= E_{cm} / \gamma_c$
	E_{cm}	is the secant modulus of concrete according to Section 3.1.4.1;
	γ_c	= 1.35 is the safety factor for the stiffness.

(2) More accurate account should be taken to the influence of long-term loading on the effective elastic flexural stiffness where:

(a) the relative slenderness $\overline{\lambda}$ in the plane of bending being considered exceeds the limit given in Table 4.9 and
(b) $e/d < 2$,

where	e	is the eccentricity of loading as defined in Section 4.8.3.3(5),
	d	is the overall depth of the cross section in the plane of bending considered,
	δ	is as defined in Section 4.8.3.4 and
	$\overline{\lambda}$	is as defined in Section 4.8.3.7. For comparison with the limits given in Table 4.9, $\overline{\lambda}$ may be calculated without considering the influence of long-term loading on flexural stiffness. Under these conditions, the effective elastic modulus of the concrete should be reduced to the value,

$$E_c = E_{cd}(1 - 0.5 N_{G.Sd} / N_{Sd}) \tag{4.22}$$

where	N_{Sd}	is the design axial load for the column length, and
	$N_{G.Sd}$	is the part of this load that is permanent.

Table 4.9 Limiting Values of $\overline{\lambda}$ for Section 4.8.3.5(2)

	Braced non-sway frames	Sway frames and/or unbraced frames
Concrete-encased sections	0.8	0.5
Concrete-filled tubes	$0.8/(1 - \delta)$	$0.5/(1 - \delta)$

4.8.3.6 *Buckling Length of a Column*

(1) The buckling length l of an isolated non-sway composite column may conservatively be taken as equal to its system length, L.

(2) Alternatively, l may be determined using Section 4.5.2 of EBCS 3 and the following rules:

(a) Flexural stiffnesses of adjacent members attached by rigid connections should be those used in the frame analysis according to Section 4.9.6.2;
(b) Table 4.5 of EBCS 3 may be assumed to apply when the beams are composite, or of steel or reinforced concrete, and also where concrete slabs without beams are used.

(3) Where the beams are composite, Section 4.5.2.2(8) of EBCS 3 is replaced by the following rule. Where, in the global analysis for the same load case, the elastic hogging moment in a composite beam is reduced by more than 20% for "uncracked" analysis or more than 10% for "cracked" analysis, the relevant beam stiffness K_b should be taken as zero.

(4) Except where relevant rules are given in EBCS 2 or EBCS 3, paragraphs (1) to (3) above may be used for reinforced concrete and steel columns in non-sway composite frames.

4.8.3.7 *Relative Slenderness*

(1) The elastic critical load for the column length N_{cr} shall be calculated from

$$N_{cr} = \pi^2 (EI)_e / l^2 \qquad (4.23)$$

where $(EI)_e$ is given in Section 4.8.3.5, and
l is the buckling length in accordance with Section 4.8.3.6

(2) The non-dimensional slenderness for the plane of bending considered is given by

$$\overline{\lambda} = \sqrt{N_{pl,Rd}/N_{cr}} \qquad (4.24)$$

where $N_{pl,R}$ is the value of $N_{pl,Rd}$ according to Section 4.8.3.3 when the γ_M factors γ_{Ma}, γ_c, and γ_s are taken as 1.0.

4.8.3.8 *Resistance of Members in Axial Compression*

(1) The member has sufficient resistance if for both axes

$$N_{Sd} \leq \chi N_{pl.Rd} \tag{4.25}$$

where $N_{pl.Rd}$ is the resistance in accordance with Section 4.8.3.3 and
χ is the reduction coefficient for the relevant buckling made given in Section 4.5.4.3 of EBCS 3 in terms of the relevant slenderness $\overline{\lambda}$ and the relevant buckling curve.

(2) Appropriate buckling curves are:

(a) Curve *a* for concrete-filled hollow sections
(b) Curve *b* for fully or partly concrete-encased I-sections with bending about the strong axis of the steel section
(c) Curve *c* for fully or partly concrete encased I-sections with bending about the weak axis of the steel section

4.8.3.9 *Combined Compression and Bending*

(1) For each of the axes of symmetry a separate check is necessary with the relevant slenderness, bending moments and resistance in bending.

(2) For compression and uniaxial bending this check should be done according to Section 4.8.3.10 to 4.8.3.13 for the bending plane and according to Section 4.8.3.8 for the non-bending plane.

(3) For compression and biaxial bending the check is given in Section 4.8.3.14.

4.8.3.10 *Analysis for Bending Moments*

(1) Bending moments at the ends of the member should be determined assuming that the axial force acts through the centroid as defined in the Note to Section 4.8.3.1(3)(a).

(2) Column generally shall be checked for second order effects.

(3) Isolated non-sway columns need not be checked for second order effects if:

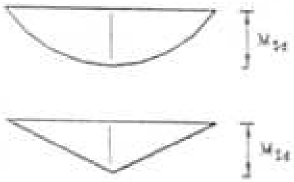
(a) $N_{Sd}/N_{cr} \leq 0.1$
where N_{cr} is defined in Section 4.8.3.7(1); or
(b) for columns with end moments, the relative slenderness does not exceed

$$\overline{\lambda}_{crit} = 0.2(2 - r) \tag{4.26}$$

where r is the ratio of the end moments according to Table 4.10. If there is any transverse loading, r should be taken as 1.0.

(4) When checking second order effects, flexural stiffness should be calculated in accordance with Section 4.8.3.5.

Table 4.10 Factors β for the Determination of Moments According to Second Order Theory

line	moment distribution	moment factors β	comment
1	first order bending moments from lateral loads in isolated non-sway column 	$\beta = 1.0$	M_{Sd} is the maximum bending moment within the column lenght due to lateral forces ignoring second order effects
2	end moments in a non-sway frame  $-1 \leq r \leq 1$	$\beta = 0.66 - 0.44r$ but $\beta \geq 0.44$	M_{Sd} and rM_{Sd} are the end moments from a frame analysis according to 4.3

(5) For simplification, second order effects in an isolated non-sway column may be allowed for by increasing the greatest first-order design bending moment M_{Sd} by a correction factor k given by

$$k = \frac{\beta}{1 - N_{Sd}/N_{cr}} \geq 1.0 \qquad (4.27)$$

where N_{cr} is the critical load for the relevant axis according to 4.8.3.7(1) with the effective length l taken as the column length, and

β is an equivalent moment factor given in Table 4.10.

In absence of more accurate calculation the value of β should not be taken less than 1.0 for combined action of end moments and moments from lateral load.

4.8.3.11 *Resistance of Cross Sections in Combined Compression and Uniaxial Bending*

(1) Points on the interaction curve of Fig. 4.13, showing resistance in combined compression and uniaxial bending, may be calculated assuming rectangular stress blocks as shown in Fig. 4.14 and taking account of the design shear force V_{Sd} according to Section 4.8.3.12.

(2) Figure 4.14 shows stress distributions corresponding to the points A to D of the interaction curve (Fig. 4.13), for a typical concrete-encased I-section with bending about the strong axis of the steel section.

(3) For concrete filled hollow sections the plastic resistances may be calculated with $0.85f_{ck}$ being replaced by f_{ck}.

(4) As a simplification, the curve may be replaced by a polygonal diagram (dashed line in Fig. 4.13). More information for the calculations for points A to D is given in Section 4.8.3.16.

(5) An additional point E should be determined approximately midway between point A and point C of Fig. 4.13 if the resistance of the column to axial compression ($\chi N_{pl,Rd}$) is greater than $N_{pm,Rd}$, where $N_{pm,Rd}$ is the plastic resistance of the concrete section alone. This is not necessary for I-profiles with bending about the strong axis of the steel section.

4.8.3.12 *Influence of Shear Forces*

(1) The design transverse shear force V_{Sd} may be assumed to act on the structural steel section alone, or may be shared between the steel and the concrete. The influence on the bending resistance of the shear force assumed to be resisted by the steel should be considered according to Section 4.4.3(1) and (2).

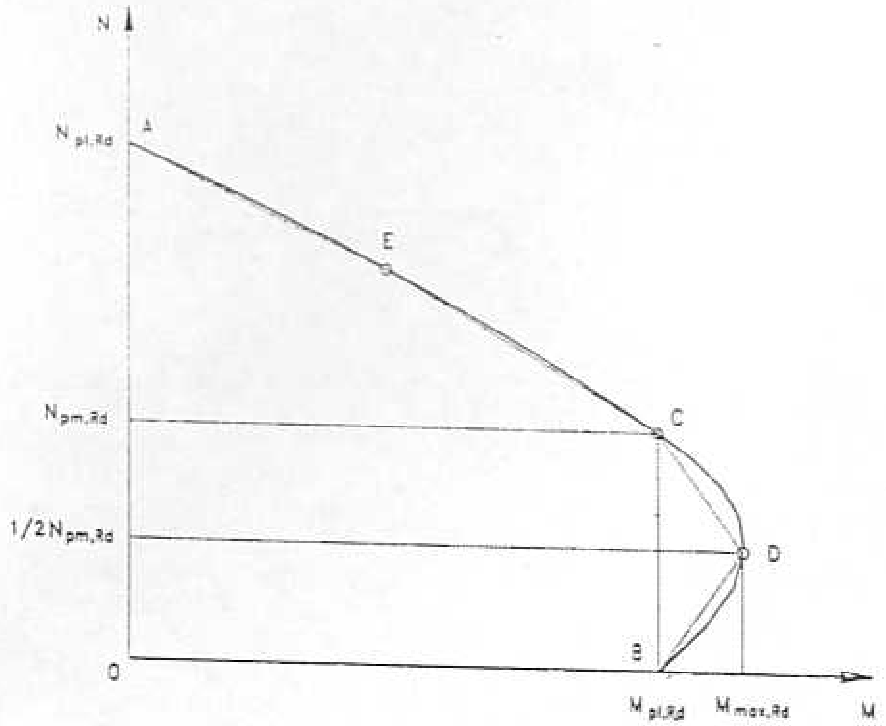


Figure 4.13 Interaction Curve for Compresion and Uniaxial Bending

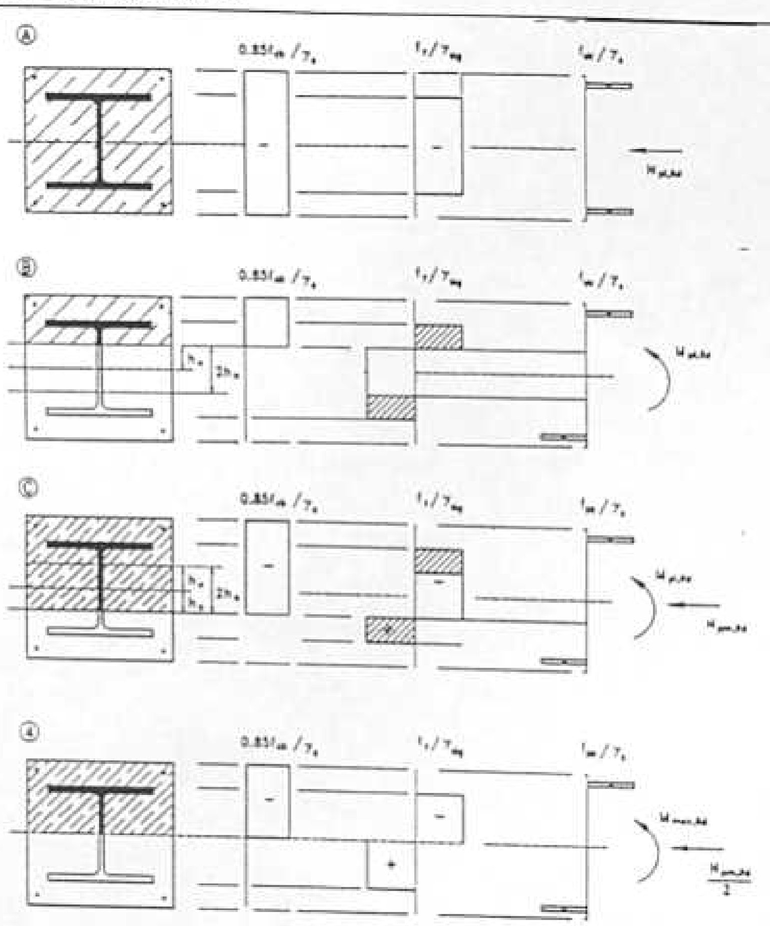


Figure 4.14 Stress Distributions Corresponding to the Interaction Curve (Fig. 4.13)

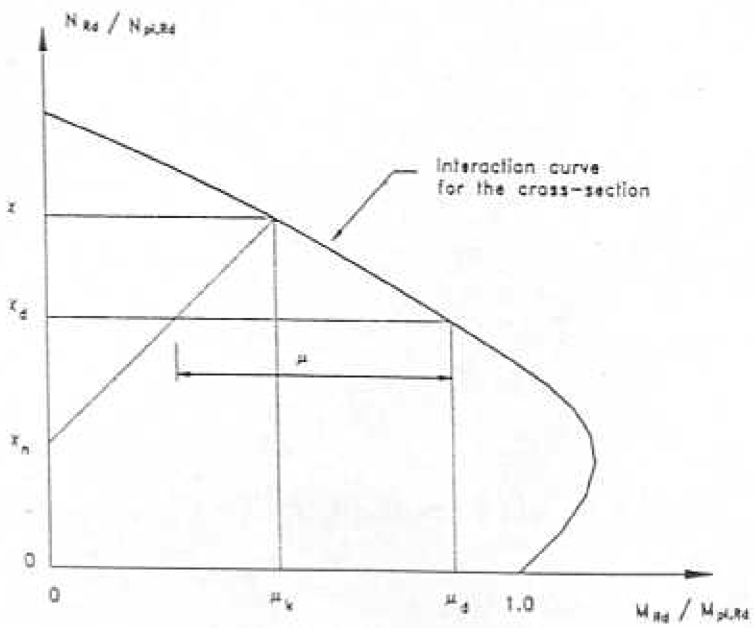


Figure 4.15 Design Procedure for Compression and Uniaxial Bending

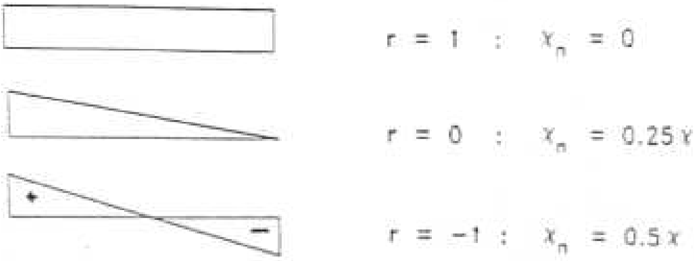


Figure 4.16 Typical Values for χ_n

4.8.3.13 *Resistance of Members in Combined Compression and Uniaxial Bending*

(1) The design procedure is given in step-by-step form, with reference to Fig. 4.15.

(2) The resistance of the member in axial compression is $\chi N_{pl.Rd}$ calculated in accordance with Section 4.8.3.8, where χ accounts for the influence of imperfections and slenderness. The corresponding value for bending of the cross section μ_k is determined from χ, as shown in Fig. 4.15.

(3) Let $\chi_d = N_{Sd} / N_{pl.Rd}$ where N_{Sd} is the design axial force, and let the corresponding bending resistance of the cross section be given by μ_d.

(4) Where the variation of bending moment along the column length is approximately linear, the ratio χ_n may be calculated from

$$\chi_n = \chi(1 - r)/4, \text{ but } \chi_n \leq \chi_d \tag{4.28}$$

where r is the ratio of the lesser to the greater end moment as shown in Fig. 4.16. Otherwise, χ_n should be taken as zero.

(5) The length μ in Fig. 4.15 is calculated from

$$\mu = \mu_d - \mu_k (\chi_d - \chi_n)/(\chi - \chi_n) \tag{4.29}$$

(6) Where $N_{Sd} < N_{pm.Rd}$ (Fig. 4.13), the increase of the bending resistance due to the normal force may be overestimated if the acting normal force N and the bending moment M are independent. This should be accounted for by reducing the partial safety factor for the favourable component N_{Sd} by 20% (see Section 3.6.1 of EBCS 2.

(7) The value of μ should not be taken as greater than 1.0, unless the bending moment M_{Sd} is due solely to the action of the eccentricity of the force N_{Sd}, e.g., in an isolated column without transverse loads acting between the column ends.

(8) The member has sufficient resistance if

$$M_{Sd} \leq 0.9 \mu M_{pl.Rd} \tag{4.30}$$

where M_{Sd} is the maximum design bending moment within the column length, calculated in accordance with Section 4.8.3.10 including second order effects if necessary; and

$M_{pl.Rd}$ is the bending moment calculated using the stress distribution shown in Fig. 4.14 (b), with γ_{Ma} in accordance with Section 4.8.3.11(3).

4.8.3.14 *Combined Compression and Biaxial Bending*

(1) Due to the different slenderness, bending moments, and resistances of bending for the two axes, in most cases a check for the biaxial behaviour is necessary.

(2) Imperfections should be considered only in the plane in which failure is expected to occur (e.g. z-axis in Fig. 4.17(a)). For the other plane of bending no imperfection needs to be considered for that plane (e.g. y-axis in Fig. 4.17 (b)). If it is not evident which plane is the more critical, checks should be made for both planes.

(3) The following design method should be used for a design axial force N_{Sd} combined with design bending moment $M_{y,Sd}$ and $M_{z,Sd}$.

(4) The values of μ for the two axes of bending, μ_y and μ_z, are found in accordance with Section 4.8.3.13.

(5) The member is strong enough if:

$$M_{y,Sd} \leq 0.9\, \mu_y M_{pl.y.Rd}, \tag{4.31a}$$

$$M_{z,Sd} \leq 0.9\, \mu_z M_{pl.z.Rd}, \tag{4.31b}$$

$$M_{y,Sd}/\mu_y M_{pl.y.Rd} + M_{z,Sd}/\mu_z M_{pl.z.Rd} \leq 1.0. \tag{4.31c}$$

with $M_{pl.y.Rd}$ and $M_{pl.z.Rd}$ according to Section 4.8.3.11, referring to the relevant axis. An example is given in Fig. 4.17(c).

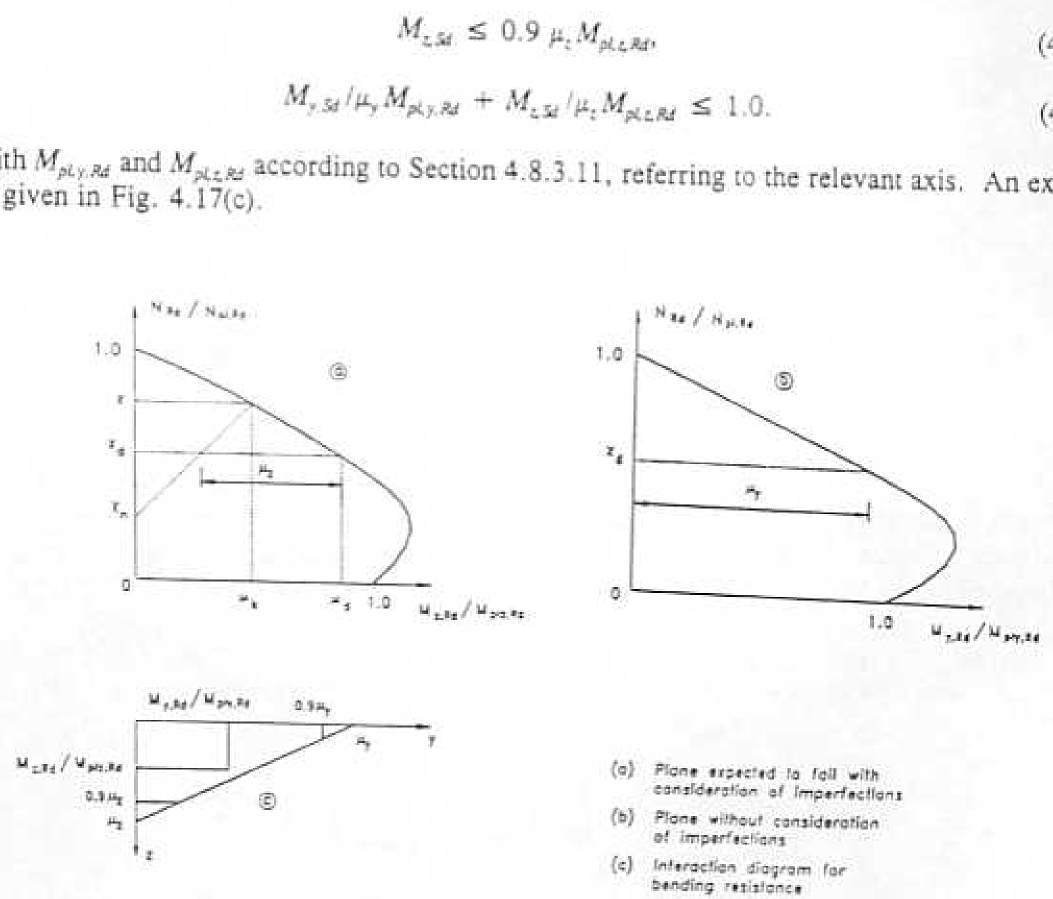


Figure 4.17 Design for Compression and Biaxial Bending

4.8.4 Design of Composite Columns with Mono-Symmetrical Cross Sections-Simplified Method

4.8.4.1 *General*

(1) For the design of composite columns with mono-symmetrical cross sections all rules of Section 4.8.3 should be observed, except those referring only to doubly symmetrical sections and/or biaxial bending. The following application rules should be observed additionally for the non-symmetrical plane of bending.

4.8.4.2 *Scope*

(1) The elastic centre of area of the uncracked composite cross section should be determined using the elastic stiffnesses with the secant modulus of elasticity for concrete according to Section 3.1.4.1.

(2) The amount of non-symmetry, determined by the distance between the axis through the centre of area and the middle line of the cross section (Fig. 4.18), should not exceed $h/10$, where h is the overall depth of the section parallel to the axis of summery.

4.8.4.3 *Design for Axial Compression*

(1) A normal force acting through the elastic centre of area is assumed to create only axial compression.

(2) The slenderness $\overline{\lambda}$ according to Section 4.8.3.7 should be determined using elastic stiffnesses according to Section 4.8.4.2(1).

(3) For design according Section to 4.8.3, the relevant buckling curves in Section 4.5.4 of EBCS 3 are:

(i) Curve *b* for concrete filled hollow sections
(ii) Curve *c* for concrete encased I-sections with bending about the strong axis of the section
(iii) Curve *d* for all other sections

4.8.4.4 *Design for Compression and Uniaxial Bending*

(1) The *M-N* interaction curve for the cross section should be calculated according to the plastic centroidal axis. This axis is defined by the centre of the strength distributions under pure compression, i.e. it is the axis about which the bending moment of the internal forces is zero when the section resists a compressive force equal to $N_{pl.Rd}$.

(2) The distance from the plastic centroidal axis to the elastic centroidal axis (e_{pl} in Fig. 4.18 is given by:

$$e_{pl} = \frac{\Sigma_i(A_i E_i z_i)}{\Sigma_i(A_i E_i)} - \frac{\Sigma(A_i f_i z_i)}{\Sigma_i(A_i f_i)} \qquad (4.32)$$

where A_i are the relevant areas
E_i are the stiffness moduli for the areas according to Section 4.8.4.2.
f_i are the design strengths of the materials for the areas, and
z_i are the distances to the reference axis for the calculation.

(3) The design rule of Section 4.8.3.13(8) changes to;

$$|M_{Sd}| + |N_{Sd}e_{pl}| \leq 0.9\mu M_{pl,Rd} \tag{4.33}$$

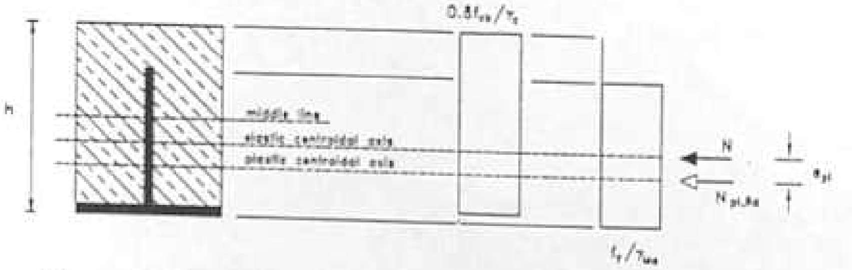


Figure 4.18 Axes of a Mono-Symmetrical Cross Section

(4) Special care should be taken when the bending moment changes its sign along the column length. Two interaction curves and two bending resistance $M_{pl,Rd}$ then have to be determined, as shown in Fig. 4.19.

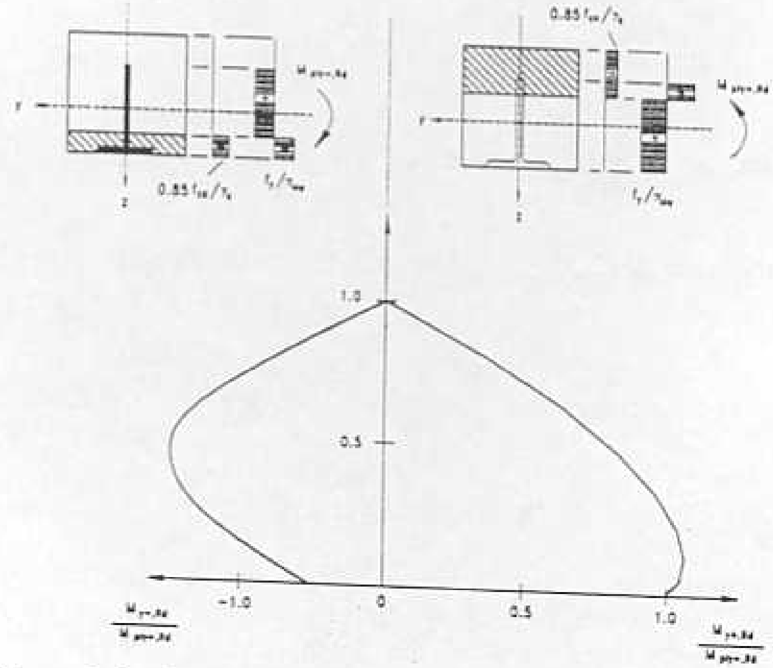


Figure 4.19 Example for the Two Interaction Curves for a Mono-Symmetrical Cross Section Related to the Same Bending Resistance $M_{pl,y+,Rd}$

4.8.4.5 *Long-Term Behaviour of Concrete*

(1) The influence of long-term loads should be taken into account if significant.

(2) The influence may be allowed for by an additional eccentricity of the permanent normal force:

$$e_{ct} = e_{el} - e_{el,t} \tag{4.34}$$

where e_{el} is the elastic centroidal axis for short term loading calculated using stiffnesses according to Section 4.8.4.2 with E_c according to Section 3.1.4.1, and

$e_{el,t}$ is the elastic centroidal axis for long term loading calculated using stiffnesses according to (2) above with $E_c = E'_c$ according to Section 3.1.4.2.

4.8.5 Simplified Calculation Method for Resistance of Doubly Symmetric Composite Cross Sections in Combined Compression and Bending

4.8.5.1 *Scope and Assumptions*

(1) This method is applicable to design in accordance with Section 4.8 of columns with cross sections that are symmetrical about both principal axes and consist of any arrangement of structural steel, concrete, and reinforcing bars. Examples are shown in Fig. 4.10.

(2) The resistance of cross sections to any combination of axial force N and bending moment M about a principal axis is represented by a curve. This Section gives methods for the calculation of the compressive resistances which define the five points A, B, C, D and E on the curve shown in Fig. 4.20. The interaction curve may be replaced by the polygonal diagram $AECDB$ through these points.

(3) Plastic analysis is used, with rectangular stress blocks for structural steel, reinforcement, and concrete in accordance with Sections 4.8.3.3 and 4.8.3.11.

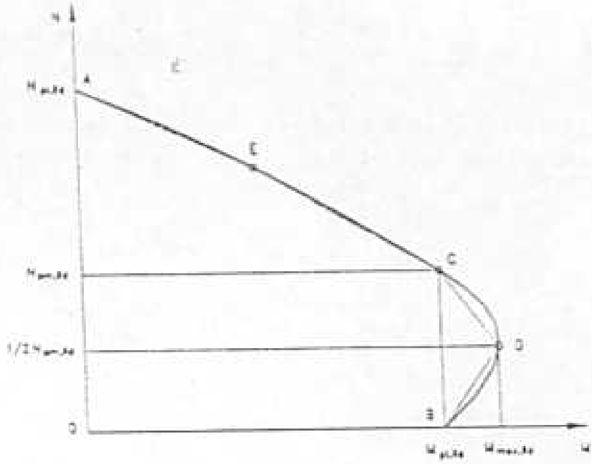


Figure 4.20 Polygonal Interaction Curve

4.8.5.2 *Compressive Resistances*

(1) The plastic resistance $N_{pl.Rd}$ is given by Section 4.8.3.3. The resistance $N_{pm.Rd}$ is calculated as follows.

(2) Figure 4.21 represents a generalised cross section of structural steel and reinforcement (cross hatched), and of concrete, symmetrical about two axes through its centre of area G. For bending only (point B) the neutral axis is line BB which defines region (1) of the cross section, within which concrete is in compression. The line CC at the same distance h_n on the other side of G is the neutral axis for point C in Fig. 4.20. This is because the areas of structural steel, concrete, and reinforcement in region (2) are all symmetrical about G, so that the changes of stress when the axis moves from BB to CC add up to the resistance $N_{pm.Rd}$ and the bending resistance is unchanged. Using subscripts 1 to 3 to indicate regions (1) to (3).

$$N_{pm.Rd} = R_{c2} + 2|R_{a2}| \tag{4.35}$$

where R_{c2} is the resistance of the concrete in region (2),
R_{a2} is the resistance of the steel in region (2).

(3) In the notation of Section 4.8.3.3

$$R_{c2} = A_{c2}(0.85\, f_{ck}/\gamma_c) \tag{4.36a}$$

or $R_{c2} = A_{c2} f_{ck}/\gamma_s$, respectively

$$R_{a2} = (A_{a2} f_y/\gamma_{Ma}) + A_{s2} f_{sk}/\gamma_s \tag{4.36b}$$

when compressive forces and strengths of materials are taken as positive.

(4) From symmetry,

$$R_{a1} = |R_{a3}| \text{ and } R_{c1} = R_{c3} \tag{4.37}$$

for neutral axis at BB, $N = 0$, so that

$$R_{a1} + R_{c1} = |R_{a2}| + |R_{a3}| \tag{4.38}$$

from Eq. 4.37 and Eq.4.38, $|R_{a2}| = R_{c1} = R_{c3}$
substituting in Eq. 4.36, $N_{pm,Rd} = R_{c2} + R_{c1} + R_{c3} = R_c$ (4.39)

where R_c is the compressive resistance of the whole area of concrete, which is easily calculated.

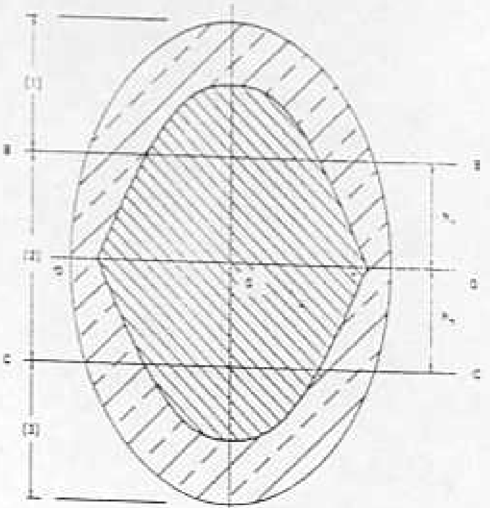


Figure 4.21 Composite Cross Section Symmetrical About two Axes

4.8.5.3 *Position of Neutral Axis*

(1) Equations for h_n depend on the axis of bending, the type of cross section and the cross section properties. The equations are derived from Eqs. 4.36 and 4.39 and are given for some cross sections in Section 4.8.6.

4.8.5.4 *Bending Resistances*

(1) The axial resistance at point D in Fig 4.20 is half that at point C, so the neutral axis for point D is line DD in Fig. 4.21.

(2) The bending resistance at point D is

$$M_{max,Rd} = W_{pa}f_{yd} + W_{ps}f_{sd} + W_{pc}f_{cd}/2 \tag{4.40}$$

where W_{pa}, W_{ps} and W_{pc} are the plastic section moduli for the structural steel, the reinforcement, and the concrete part of the section (for the calculation of W_{pc} the concrete is assumed to be uncracked), and

f_{yd}, f_{sd} and f_{cd} are the design strengths for the structural steel, the reinforcement and the concrete:

$f_{yd} = f_y/\gamma_{Ma}$
$f_{sd} = f_{sk}/\gamma_s$
$f_{cd} = f_{ck}/\gamma_c$ for concrete filled sections and
$f_{cd} = 0.85\,f_{ck}/\gamma_c$ for other sections

The bending resistance at point B is

$$M_{pl,Rd} = M_{max,Rd} + M_{n,Rd} \tag{4.41}$$

with

$$M_{n,Rd} = W_{pan}f_{yd} + W_{psn}f_{sd} + W_{pcn}f_{cd}/2 \tag{4.42}$$

where W_{pan}, W_{psn} and W_{pcn} are the plastic section moduli for the structural steel, the reinforcement and the concrete parts of the section within regime 2 of Fig. 4.21.

(4) Equations for the plastic section moduli of some cross sections are given in Section 4.8.6.

4.8.5.5 *Interaction with Transverse Shear*

(1) If the shear force to be resisted by the structural steel is considered according to Section 4.8.3.12 the appropriate areas of steel should be assumed to resist shear alone. The method of this Section can be applied using the remaining areas.

4.8.6 Neutral Axes and Plastic Section Moduli of Some Cross Sections

4.8.6.1 *General*

(1) The compressive resistance of the whole area of concrete is

$$N_{pm,Rd} = A_c f_{cd} \tag{4.43}$$

(2) The value of the plastic section modulus of the total reinforcement is given by

$$W_{ps} = \sum_{i=1}^{n} |A_{si}e_i| \tag{4.44}$$

where e_i are the distances of the reinforcement bars of area A_{si} to the relevant middle line (y-axis or z-axis).

(3) The equations for the position of the neutral axis h_n are given for selected positions in the cross sections. The resulting value h_n should lie within the limits of the assumed region.

(4) An additional point E may be found by placing the neutral axis at a significant line between line CC and the border of the section (region (3) in Fig. 4.21) and determining the resulting normal force and bending moment.

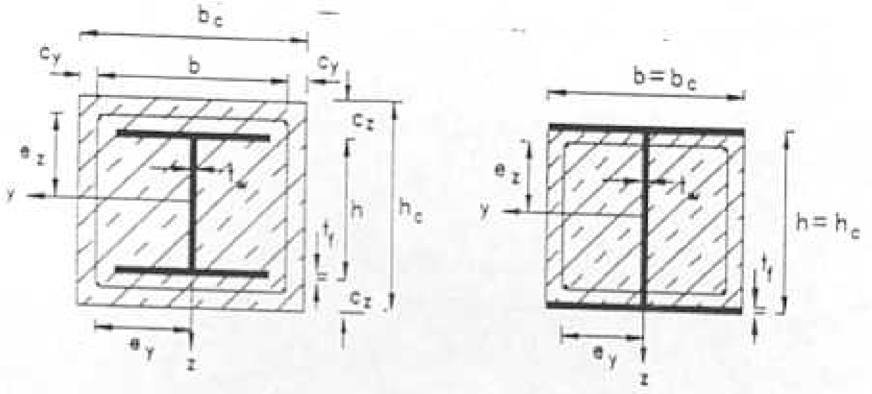


Figure 4.22 Encased I-Sections with Notation

4.8.6.2 *Major Axis Bending of Encased I-sections*

(1) The plastic section modulus of the structural steel may be taken from Tables or be calculated from:

$$W_{pa} = \frac{(h-2t_f)t_w^2}{4} + bt_f(h - t_f) \tag{4.45}$$

and

$$W_{pc} = \frac{b_c h_c^2}{4} - W_{pa} - W_{ps} \tag{4.46}$$

(2) For the different positions of the neutral axes, h_n and W_{pan} are given by:

(a) Neutral axis in the Web: $h_n \leq h/2 - t_f$

$$h_n = \frac{N_{pm.Rd} - A_{sn}(2f_{sd} - f_{cd})}{2b_c f_{cd} + 2t_w(2f_{yd} - f_{cd})} \tag{4.47}$$

$$W_{pan} = t_w h_n^2 \tag{4.48}$$

where A_{sn} is the sum of the area of reinforcing bars within the region of depth $2h_n$;

(b) Neutral axis in the flange: $h/2 - t_f < h_n < h/2$

$$h_n = \frac{N_{pm.Rd} - A_{sn}(2f_{sd}-f_{cd})+(b-t_w)(h-2t_f)(2f_{yd}-f_{cd})}{2b_c f_{cd}+2b(2f_{yd}-f_{cd})} \tag{4.49}$$

$$W_{pan} = bh_n^2 - \frac{(b - t_w)(h - 2t_f)^2}{4} \tag{4.50}$$

(c) Neutral axis outside the steel section: $h/2 \leq h_n \leq h_c/2$

$$h_n = \frac{N_{pm.Rd} + A_{sn}(2f_{sd} - f_{cd}) - A_a(2f_{yd} - f_{cd})}{2b_c f_{cd}} \tag{4.51}$$

$$W_{pan} = W_{pa} \tag{4.52}$$

(3) The plastic modulus of the concrete in the region of depth $2h_n$ then results from

$$W_{pcn} = b_c h_n^2 - W_{pan} - W_{psn} \tag{4.53}$$

$$\text{with } W_{psn} = \sum_{i=1}^{n} |A_{sni}\, e_{zi}| \tag{4.54}$$

where A_{sni} are the areas of reinforcing bars within the region of depth $2h_n$ and e_{zi} are the distances from the middle line.

4.8.6.3 *Minor Axis Bending of Encased I-sections*

(1) The notation is given in Fig. 4.22.

(2) The plastic section modulus of the structural steel may be taken from tables or be calculated from:

$$W_{pa} = \frac{(h - 2t_f)t_w^2}{4} + \frac{2t_f b^2}{4} \tag{4.55}$$

and

$$W_{pc} = \frac{h_c b_c^2}{4} - W_{pa} - W_{ps} \tag{4.56}$$

(3) For the different positions of the neutral axes, h_n and W_{pan} are given by:

(a) Neutral axis in the web: $h_n \leq t_w/2$

$$h_n = \frac{N_{pm.Rd} - A_{sn}(2f_{sd} - f_{cd})}{2h_c f_{cd} + 2h(2f_{yd} - f_{cd})} \tag{4.57}$$

$$W_{pan} - hh_n^2 \tag{4.58}$$

(b) Neutral axis in the flanges: $t_w/2 < h_n < b/2$

$$h_n = \frac{N_{pm.Rd} - A_{sn}(2f_{sd} - f_{cd}) + t_w(2t_f - h)(2f_{yd} - f_{cd})}{2h_c f_{cd} + 4t_f(2f_{yd} - f_{cd})} \tag{4.59}$$

$$W_{pan} = 2t_f h_n^2 + \frac{(h - 2t_f)t_w^2}{4} \tag{4.60}$$

(c) Neutral axis outside the steel section: $b/2 \leq h_n \leq b_c/2$

$$h_n = \frac{N_{pm.Rd} - A_{sn}(2f_{sd} - f_{cd}) - A_a(2f_{yd} - f_{cd})}{2h_c f_{cd}} \tag{4.61}$$

$$W_{pan} = W_{pa} \tag{4.62}$$

(4) The plastic modulus of the concrete in the region of depth $2h_n$ then results from

$$W_{pcn} = h_c h_n^2 - W_{pan} - W_{psn} \tag{4.63}$$

with W_{psn} according to Eq. 4.54 changing subscript z to y.

(5) For the calculation of $N_{E.Rd}$ and $M_{E.Rd}$, the resistances at the additional point E, the neutral axis should be located so that $N_{E.Rd}$ is close to the average of $N_{pm.Rd}$ and $N_{pl.Rd}$.

(6) For a neutral axis in the flanges ($t_w^2 < h_E \leq b/2$), the normal force N_E results from:

$$N_{E.Rd} = h_c(h_E - h_n)f_{cd} + 2t_f(h_E - h_n)(2f_{yd} - f_{cd}) + A_{sE}(2f_{sd} - f_{cd}) + N_{pm.Rd} \tag{4.64}$$

provided also that $t_w/2 < h_E \leq b/2$, A_{sE} is the sum of the areas of reinforcement lying in the additionally compressed region between h_E and h_n.

(7) For $t_w/2 < h_E \leq b/2$, the plastic section moduli are calculated by using Eqs. 4.61 and 4.63, substituting h_n by h_E. Eq. 4.40 then leads to the moment M_E.

4.8.6.4 *Concrete Filled Circular and Rectangular Hollow Sections*

(1) The following equations are derived for rectangular hollow sections with bending about the y-axis of the section (see Fig. 4.23). For bending about the z-axis the dimensions h and b are to be exchanged as well as the subscripts z and y. The Eqs. 4.65 to 4.70 may be used for circular hollow sections with good approximation by substituting $h = b = d$ and $r = d/2 - t$.

$$W_{pc} = \frac{(b - 2t)(h - 2t)^2}{4} - \frac{2}{3}r^3 - r^2(4 - \pi)(0.5h - t - r) - W_{ps} \tag{4.65}$$

with W_{ps} according to Eq. 4.43

(2) W_{pa} may be taken from Tables or be calculated from

$$W_{pa} = \frac{(bh^2)}{4} - \frac{2}{3}(r+t)^3 - (r+t)^2(4-\pi)(0.5h-t-r) - W_{pc} - W_{ps} \tag{4.66}$$

$$h_n = \frac{N_{pm,Rd} - A_{sn}(2f_{sd} - f_{cd})}{2bf_{cd} + 4t(2f_{yd} - f_{cd})} \tag{4.67}$$

$$W_{pcn} = (b - 2t)h_n^2 - W_{psn} \tag{4.68}$$

$$W_{pan} = bh_n^2 - W_{pcn} - W_{psn} \tag{4.69}$$

with W_{psn} according to Eq. 4.54.

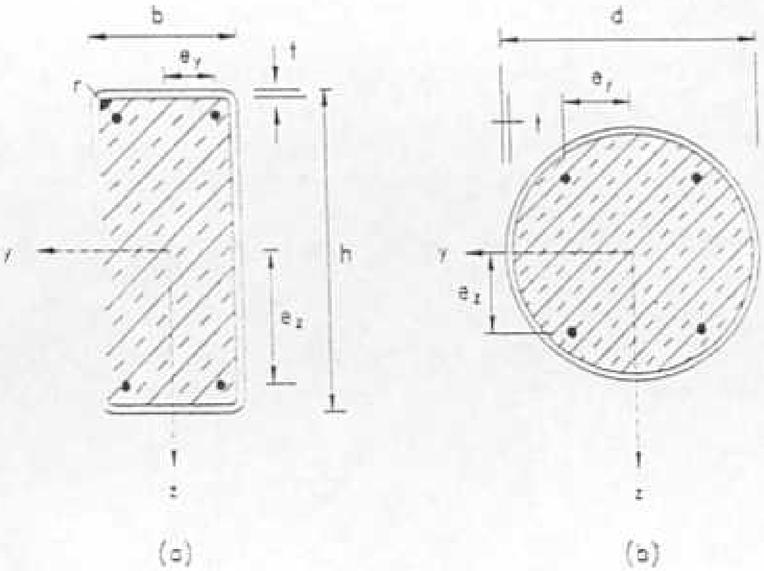


Figure 4.23 Concrete Filled Circular and Rectangular Hollow Sections with Notation

(3) For the calculation of $N_{E,Rd}$ and $M_{E,Rd}$, the resistances at the additional point E, the neutral axis is located half-way between h_n and the boarder of the section, so that $h_E = h_n/2 + h/4$.

(4) The normal force N_E results from:

$$N_{E,Rd} = b(h_E - h_n)f_{cd} + 2t(h_E - h_n)(2f_{yd} - f_{cd}) + A_{sE}(2f_{sd} - f_{cd}) + N_{pm,Rd} \tag{4.70}$$

where A_{sE} is the sum of the areas of reinforcement lying in the additionally compressed region between h_E by h_n.

(5) The plastic section moduli are calculated by using Eqs. 4.68 and 4.69 substituting h_n by h_E. Eq. 4.41 then leads to the moment $M_{E,Rd}$.

4.9 INTERNAL FORCES AND MOMENTS IN FRAMES FOR BUILDINGS

4.9.1 General

(1) It is assumed that most of the structural members and connections are either composite or of structural steel. Where the structural behaviour of the frame is essentially that of a reinforced or prestressed concrete structure, with only a few composite members, global analysis shall be generally in accordance with Chapter 4 of EBCS 2.

(2) The definitions and classifications of methods of global analysis, types of framing, and types of connections are similar to those used in Section 4.2 of EBCS 3, which is applicable to structural steel members in composite frames. The classification of frames, as braced or unbraced and sway or non-sway, is consistent with that given in Section 4.2.5 of EBCS 3.

(3) The scope of this Section excludes sway frames, as defined in Section 4.9.4.2.

(4) The general principles for plastic analysis given in Section 4.5.2.1 are applicable, but no application rules are given for elastic-plastic methods of analysis.

(5) No application rules are given for global analysis of unbraced non-sway frames, as defined in Section 4.9.4.

(6) No application rules are given for global analysis of frames with semi-rigid connections. These connections are defined in Section 4.10.5.2 and, for steel connections, in Chapter 6 of EBCS 3.

(7) It may be found convenient to verify the design of a composite braced frame in the following sequence.

(a) Define the imperfections of the frame (Section 4.9.3) and represent them by equivalent horizontal forces at nodes.
(b) Ensure that no steel connection is "semi-rigid", using Sections 4.10.5 and 6.1.5 of EBCS 3.
(c) For members of reinforced or prestressed concrete, ensure the ductility requirements.
(d) Check that the frame is braced Section 4.9.4.3.
(e) Check that the bracing substructure is non-sway Section 4.9.4.
(f) Decide whether the requirements for rigid-plastic global analysis Section 4.9.7 are satisfied.
(g) Carry out global analyses (Section 4.9.5 to 4.9.7) for relevant load combinations and arrangements and hence find design internal forces and moments at each end of each member.
(h) Verify the composite beams (Sections 4.2 to 4.4.), columns (Section 4.8), and connections (Section 4.10).
(i) Verify beams, columns, and connections of structural steel (to EBCS 3) and of concrete (to EBCS 2).
(j) Reference is made to the effective length (buckling length) of reinforced concrete and steel columns in Section 4.8.3.6(4)
(k) For reinforced concrete columns, Section 4.4 of EBCS 2 is applicable.

4.9.2 Design Assumptions

4.9.2.1 *Basis*

(1) The assumptions made in the global analysis shall be consistent with the anticipated behaviour of the connections.

(2) The assumptions made in the design of the members shall be consistent with the method used for global analysis and with the anticipated behaviour of the connections.

(3) Composite connections are classified in Section 4.10. For steel beam-to-column connections, Section 6.7 of EBCS 3 is applicable.

(4) Table 4.11 shows the types of connections for use with each type of framing, depending on he method of global analysis used.

(5) Nominally pinned connections may be used in continuous construction at points where continuity is not required, provided that the connection is designed as non-composite in accordance with Chapter 6 of EBCS 3, ignoring any reinforcement which may be provided for the control of cracking.

Table 4.11 Design Assumptions

Type of Framing (Terminology)	Method of Global Analysis	Types of Connections
Simple	Statically determinate	Nominally pinned, steel (Section 6.1.3 of EBCS 3)
Continuous	Elastic	Rigid, steel (Section 6.1.4 of EBCS 3) Nominally pinned (Section 6.1.3 of EBCS 3) Rigid, composite (Section 4.10.5.2)
	Rigid-plastic	Full-strength, steel(Section 6.1.4 of EBCS 3) Nominally pinned(Section 6.1.3 of EBCS 3) Full-strength, composite (Section 4.10.5.3)
Semi-Continuous	Rigid-plastic	As for continuous framing (above) and: partial-strength, steel(Section 6.1.5 of EBCS 3) partial-strength, composite (Section 4.10.5.3)

4.9.2.2 *Simple Framing*

In simple framing, the connections between the members may be assumed not to develop moments. In the global analysis, members may be assumed to be effectively pin connected.

4.9.2.3 *Continuous Framing*

(1) Both elastic and rigid-plastic analyses should be based on the assumption of full continuity, except where nominally pinned connections are used.

4.9.2.4 *Semi-Continuous Framing*

(1) Rigid-plastic analysis should be based on the design moment resistances of connections which have been demonstrated to have sufficient rotation capacity; see Sections 6.1.5 and 6.7.5.3 of EBCS 3.

4.9.2.5 *Effects of Deformations*

(1) Internal forces and moments in non-sway frames may generally be determined using first-order theory, using the initial geometry of the structure. Alternatively, second-order theory may be used.

4.9.3 Allowance for Imperfections

(1) The principles of Section 4.2.4 of EBCS 3 are applicable, with the following modification and additions.

(2) Section 4.2.4.2(4) of EBCS 3 applies only to steel columns. For the composite and reinforced concrete columns, the effects of imperfections within the member may be neglected in a global analysis for a frame within the scope of this Section.

(3) For braced frames, the effects of frame imperfections shall be included in the global analysis of the bracing.

(4) The application rules of Section 4.2.4 of EBCS 3 are applicable.

4.9.4 Sway Resistance

4.9.4.1 *General*

(1) The principles of Section 4.2.5 of EBCS 3 are applicable with the following modifications, to non-sway composite frames, whether braced or unbraced, in which most of the columns are composite or of structural steel.

(2) Where a composite frame is classified as braced and the bracing system is not composite, that system shall be designed to the relevant Code, and shall satisfy the requirements for resistance and stiffness given in Section 4.2.5.3 of EBCS 3.

4.9.4.2 *Classification as Sway or Non-Sway*

(1) The criteria of Section 4.2.5 of EBCS 3 shall be used to classify a composite frame as non-sway. Consideration shall be given to the effects of cracking and creep of concrete.

(2) A braced frame shall be treated as a non-sway frame.

4.9.4.3 *Classification as Braced or Unbraced*

(1) The criteria of Section 4.2.5.3 of EBCS 3 shall be used to classify a composite frame as braced. Consideration shall be given to the effects of cracking and creep of concrete.

(2) A composite frame may be treated as braced if the bracing system reduces its horizontal displacements by at least 80%, when account is taken in both analyses of the effects of cracking of concrete and, where necessary, of creep.

(3) Where the analyses are based on uncracked cross-sections of composite beams, the limit of 80% may also be used.

(4) The application rules of Section 4.2.5.3 of EBCS 3 are applicable to composite bracing systems.

4.9.5 Methods of Global Analysis

(1) The internal forces and moments in a statically determinate structure shall be obtained using statics.

(2) The internal forces and moments in a statically indeterminate structure may generally be determined using either.

(a) Elastic global analysis in accordance with Section 4.9.6, or
(b) plastic global analysis in accordance with Section 4.9.7.

(3) Where the global analysis is carried out by applying the loads in a series of increments, it may be assumed to be sufficient, in the case of building structures, to adopt simultaneous proportional increases of all loads.

4.9.6 Elastic Global Analysis

4.9.6.1 *General*

(1) Elastic global analysis shall be based on the assumption that the stress-strain relationships for the materials are linear, whatever the stress level. Concrete in tension may be included or neglected. When it is included, reinforcement in tension may be neglected. Reinforcement in compression may normally be neglected.

(2) The effects of slip and uplift may be neglected at interfaces between steel and concrete at which shear connection is provided in accordance with Chapter 6.

(3) The principles of Sections 4.5.3.2 (sequence of construction) and 4.5.3.3 (shrinkage of concrete) are applicable.

(4) Elastic global analysis should be used only where all connections are either rigid or nominally pinned.

4.9.6.2 *Flexural Stiffness*

(1) Creep effects shall be considered if they are likely to reduce the structural stability significantly.

(2) For composite beams in braced frames, Section 4.5.3.1(2) is applicable.

(3) Creep in columns may be ignored if the increase in the first order bending moments resulting from permanent loads and due to creep deformations and longitudinal force does not exceed 10%.

(4) For first order analysis, the elastic stiffness of a composite column should be taken as $E_a I_1$ where E_a is the modulus of elasticity of structural steel and I_1 is the "uncracked" second moment of area, defined in Section 4.2.3.

4.9.6.3 *Redistribution of Moments*

(1) The bending moment distribution given by an elastic global analysis may be redistributed in a way that satisfies equilibrium, and takes account of the effects of cracking of concrete, inelastic behaviour of materials, and all types of buckling.

(2) Bending moments from a first-order elastic analysis may be redistributed;.

(a) In steel members in accordance with Section 4.2.1.3(3) of EBCS 3; but for unpropped construction, subject to Section 4.5.3.4(2)(c).

(b) In concrete members subject mainly to flexure, in accordance with Section 2.5.3.4.2 of EBCS 2;

(c) In spans of composite beams in braced frames with rigid full-strength connections at their ends, or with one rigid full-strength connection and one nominally pinned connection, in accordance with Section 4.5.3.4(2);

(d) but elastic moments may not be reduced in concrete or composite columns. Where beam-to-column connections are rigid and midspan moments are redistributed to supports, column end moments should be increased, according to the relative stiffness of the members. For columns, stiffness should be based on the length between centres of restraint.

4.9.7 Rigid-Plastic Global Analysis

4.9.7.1 *General*

(1) Rigid-plastic global analysis shall not be used unless:

(a) The frame is non-sway in accordance with Section 4.9.4
(b) The frame, if unbraced in accordance with Section 4.9.4, is of two storeys of less
(c) All the members of the frame are steel or composite
(d) The cross-sections of steel members satisfy the principles of Sections 4.2.7 and 4.3.3 of EBCS 3
(e) Composite beams satisfy the principles of Section 4.5.2.2(1)
(f) The steel material satisfies Section 3.2.2.2 of EBCS 3

(2) Where rigid-plastic global analysis is used, the connections in the frame should be steel or composite and either:

(a) Have been shown to have sufficient rotation capacity, or
(b) have a design moment resistance at least 1.2 times the design plastic moment resistance of the connected beam (Section 4.10.5.3(2)).

(3) In rigid-plastic analysis, elastic deformations of the members, connections, and foundations are neglected and plastic deformations are assumed to be concentrated at plastic hinge locations.

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

4.9.7.2 *Plastic Hinges*

(1) At each plastic hinge location:

(a) The cross section of the structural steel member or component shall be symmetrical about a plane parallel to the plane of the web or webs
(b) The proportions and restraints of steel components shall be such that lateral-torsional buckling does not occur
(c) Lateral restraint to the compression flange shall be provided at all hinge locations at which plastic rotation may occur under any load case
(d) The rotation capacity shall be sufficient, when account is taken of any axial compression in the member, to enable the required hinge rotation to develop

(2) Where rotation requirements are not calculated, all members containing plastic hinges shall have effective cross-sections at plastic hinge locations that are in Class 1 in accordance with Section 4.3 of EBCS 3.

(3) For composite beams, all other effective cross-sections should be in Class 1 or Class 2.

(4) For individual spans of composite beams that contain a sagging moment hinge that is not the last to form, Section 4.5.2.2(2)(d) should be satisfied.

(5) Design should ensure that plastic hinges do not occur in composite columns.

(6) Where plastic hinges occur in steel columns, Sections 4.2.7(3) and (4) of EBCS 3 should be satisfied.

(7) Where the cross-section of a steel member varies along its length, Section 4.3.3(5) of EBCS 3 is applicable.

(8) Where restraint is required by Section 4.9.7.2(1)(d), 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 component or member.

4.10 COMPOSITE CONNECTIONS IN BRACED FRAMES FOR BUILDINGS

4.10.1 General

(1) Other connections in composite frames shall be designed in accordance with EBCS 2 or EBCS 3, as appropriate.

(2) This Section 4.10 is intended for use with Chapter 6 of EBCS 3. It supplements or modifies that Chapter.

(3) In this Section 4.10, the term 'connection' refers to composite connections.

(4) The internal forces and moments applied to connections for the ultimate limit state shall be determined by global analysis conforming with Section 4.9.

(5) The resistance of a connection shall be determined on the basis of the resistances of the individual components.

(6) Connections may be designed by distributing the internal forces and moments in the best rational way, provided that the distribution is in accordance with Chapter 6 of EBCS 3. In addition, the deformations implied by the distribution shall be within the deformation capacity of the reinforcement and any concrete assumed to resist compression.

(7) Ease of fabrication and assembly shall be considered in the design of all joints and splices. Section 8.5.5 of EBCS 2 and Section 6.1.1 of EBCS 3 are applicable.

4.10.2 Classification of Connections

(1) Section 6 of EBCS 3 is applicable, with the reference to Table 4 and Section 4.2.2 of EBCS 3 replaced by reference to Table 4.11 and Section 4.9.2 of EBCS 4.

4.10.3 Connections Made with Bolts, Rivets or Pins

4.10.3.1 *General*

(1) Section 6.2 of EBCS 3 is applicable, with the modifications given below.

4.10.3.2 *Distribution of Forces Between Fasteners*

(1) Due account shall be taken of the forces in the reinforcement and concrete components of the connection, except as allowed in Section 4.9.2.1(5).

4.10.3.3 *Pin Connections*

(1) No provision is made for the use of pin connections to Section 6.3 of EBCS 3 as part of a composite connection.

4.10.4 Splices in Composite Members

(1) Section 6.4 of EBCS 3 is applicable provided that due account is taken of the forces in the reinforcement and concrete when designing the splice between the structural steel components.

4.10.5 Beam-to-Column Connections

4.10.5.1 *General*

(1) Section 6.4 of EBCS 3 is applicable to composite connections, with the modifications given below.

4.10.5.2 *Classification by Rotational Stiffness*

(1) Section 6.7.2 of EBCS 3 is applicable, except that semi-rigid connections and unbraced frames are outside the scope of Section 4.10.

(2) For classification of a connection, the value taken for the flexural rigidity of the connected beam should be consistent with that taken for a section adjacent to the connection in global analysis of the frame.

4.10.5.3 *Classification by Moment Resistance*

(1) Section 6.7.3 of EBCS 3 is applicable.

(2) If the connected beam is a composite member, the plastic moment resistance $M_{pl.Rd}$ should be that of the beam's cross-section immediately adjacent to the connection, calculated in accordance with Sections 4.4.1.2 or 4.4.1.3.

4.10.5.4 *Classification of Moment-Rotation Characteristics*

(1) The classification boundary for braced frames given in Fig. 6.18 of EBCS 3 is applicable.

(2) For composite beams, the plastic moment resistance and flexural rigidity are defined in Sections 4.10.5.3 and 4.10.5.2 respectively.

4.10.5.5 *Calculated Properties*

(1) Section 6.7.5 of EBCS 3 is applicable provided that due account is taken of the forces in the reinforcement and in concrete components of the connection.

(2) The criteria for the tension zone shall include yielding of the reinforcement of the connection.

(3) The buckling resistance of the column web can be improved by encasement in reinforced concrete. Account may be taken of such improvement where it has been established by testing.

4.10.5.6 *Application Rules*

The detailed rules given in of EBCS 3 may be applied to components of composite connections, if appropriate.

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CHAPTER 5
SERVICEABILITY LIMIT STATES

5.1 GENERAL

(1) This chapter covers the common serviceability limit states. These are:

(a) Deflection control, and
(b) crack control.

Other limit states (such as vibration) may be of importance in particular structures but these are not covered in this Part of EBCS 4.

(2) Calculation of stresses and deformations at the serviceability limit state shall take into account the effects of:

(a) Shear lag
(b) Increased flexibility resulting from significant incomplete interaction, due to slip and/or uplift;
(c) Cracking and tension stiffening of concrete in hogging moment regions
(d) Creep and shrinkage of concrete
(e) Yielding of steel, if any, especially when unpropped construction is used
(f) Yielding of reinforcement, if any, in hogging moment regions

These effects shall be established by test or analysis, where practicable.

(3) In the absence of a more rigorous analysis, the effects of creep may be taken into account by using modular ratios, as given in Section 3.1.5.3, for the calculation of flexural stiffnesses.

(4) Serviceability limit states for floor with precast concrete slabs are covered in Chapter 7

5.2 DEFORMATIONS

5.2.1 General

(1) Deformations shall not adversely affect the use, efficiency, or appearance of the structure. Composite members shall be so proportioned that deflections of beams and sidesway of unbraced frames are within acceptable limits. Appropriate limits depend on the properties of any non-structural components, such as partitions in buildings, and on the intended use and occupancy of the structure.

(2) In buildings it will normally be satisfactory to consider the deflections under the rare combination of loading.

(3) For buildings the recommended limits for horizontal deflections at the tops of the columns are as given in Section 5.2.2(4) of EBCS 3.

(4) For floor and roof construction in buildings, the deflection limits given in Section 5.2.2 of EBCS 3 are applicable. The sagging vertical deflection δ_{max} for unpropped beams should be determined for the underside of the beam, only where the deflection can impair the appearance of the building. In all other cases the reference level is the upper side of the beam.

5.2.2 Calculation of Maximum Deflections of Beams

(1) Deflections due to loading applied to the steel member alone shall be calculated in accordance with EBCS 3.

(2) Deflections due to loading applied to the composite member shall be calculated using elastic analysis with corrections for the effects given in Section 5.1(2).

(3) The influence of shear lag on deflections can usually be ignored. For members where breadth b of the concrete flange exceeds one-eighth of the span, shear lag may be allowed for by using the effective area of concrete flange given in Section 4.2.2.1 when calculating stiffness.

(4) The effects of incomplete interaction may be ignored in spans or cantilevers where one or more of the critical cross sections is in Class 3 or 4.

(5) The effects of incomplete interaction may be ignored in unpropped construction provided that:

(a) the design of the shear connection is in accordance with Chapter 6;
(b) either not less shear connectors are used than half the number for full shear connection, or the forces on the shear connectors do not exceed 0.7 P_{Rk} in Section 3.5.2;
(c) in case of a ribbed slab with ribs transverse to the beam, the height of the ribs does not exceed 80mm.

(6) If the conditions in (4) above are not met, but $N/N_f \geq 0.4$, then in lieu of testing or accurate analysis, the increased deflection arising form incomplete interaction may be determined from:

(a) for propped construction;

$$\frac{\delta}{\delta_c} = 1 + 0.5\left[1 - \frac{N}{N_f}\right]\left[\frac{\delta_a}{\delta_c} - 1\right] \tag{5.1}$$

(b) for unpropped construction;

$$\frac{\delta}{\delta_c} = 1 + 0.3\left[1 - \frac{N}{N_f}\right]\left[\frac{\delta_a}{\delta_c} - 1\right] \tag{5.2}$$

where δ_a is the deflection for the steel beam acting alone;
δ_c is the deflection for the composite beam with complete interaction;
N/N_f is the degree of shear connection as given in Section 6.1.2.

(7) The effect of cracking of concrete in hogging moment regions may be taken into account by adopting one of the following methods of analysis:

(a) The hogging bending moment at each internal support and the resulting top-fibre tensile stress in the concrete, σ_{ct}, are first calculated using the flexural stiffnesses $E_a I_1$. For each support at which σ_{ct} exceeds $0.15 f_{ctk}$, the stiffness should be reduced to the value $E_a I_2$ over 15% of the length of the span on each side of the support. A new distribution of bending moments is then determined by re-analyzing the beam. At every support where stiffnesses $E_a I_2$ are used for a particular loading, they should be used for all other loadings.

Flexural stiffnesses $E_a I_1$ and $E_a I_2$ are defined in Section 4.2.3.

(b) For beams with critical sections in Classes 1, 2 or 3 the following method may be used. At every support where σ_{ct} exceeds $0.15 f_{ct}$, the bending moment is multiplied by the reduction factor f_1 given in Fig. 5.1, and corresponding increases are made to the bending moments in adjacent spans.

Curve A may be used 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.

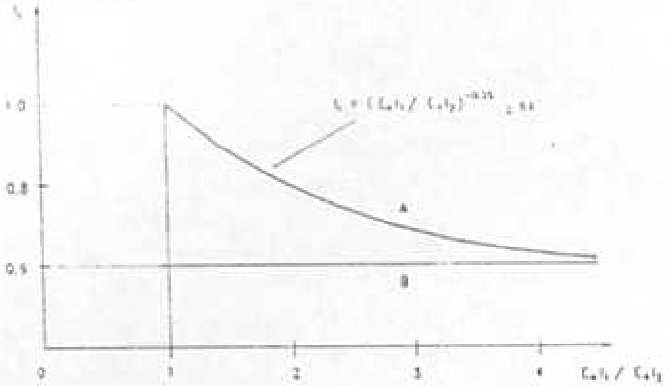

Figure 5.1 Reduction Factor for the Bending Moment at Supports

(8) In unpropped beams in buildings, 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 according to the methods given in this clause, with an additional reduction factor as follows:

$f_2 = 0.5$ if f_y is reached before the concrete slab has hardened;
$f_2 = 0.7$ if f_y is caused by the loading after concrete has hardened.

(9) In statically-determinate beams in buildings, the effect of curvature due to shrinkage of concrete should be included when the ratio of span to overall depth of the beam exceeds 20 and the predicted free shrinkage strain of the concrete exceeds 400×10^{-6}.

5.3 CRACKING OF CONCRETE IN BEAMS

5.3.1 General

(1) Cracking shall be limited to a level that will not be expected to impair the proper functioning of the structure or cause its appearance to be unacceptable.

(2) Cracking is almost inevitable where reinforced concrete elements of composite beams are subject to tension resulting from either direct loading or restraint of imposed deformations.

(3) Where cracks are avoided by measures such as the provision of joints that can accommodate the movement, the measures taken shall not impair the proper functioning of the structure or cause its appearance to be unacceptable.

(4) Where the exposure is Class 1, crack width has no influence on durability, and flexural cracks may be permitted to form without any attempt to control their width. In accordance with (1) above:

(a) Their appearance shall be acceptable, if they are visible, and
(b) any finish to the surface of the concrete shall not be brittle.

(5) Where a composite beam is subjected to hogging bending, and no attempt is made to control the width of cracks in the concrete of its top flange, the longitudinal reinforcement provided within the effective width of that flange should be not less than:

(a) 0.4% of the area of concrete, for propped construction, or
(b) 0.2% of the area of concrete, for unpropped construction.

The reinforcement should extend over a length span/4 each side of an internal support, or length/2 for a cantilever. The effective width should be as given in Section 4.2.2.2. No account should be taken of any profiled steel sheeting. The maximum spacing of the bars should be in accordance with Section 7.1.4 of EBCS 2.

(6) Appropriate limits to design crack widths, taking account of the proposed function and nature of the structure and the costs of limiting cracking, shall be determined.

(7) Design crack-width limits should be agreed with the client.

(8) Limitation of cracks to acceptable widths, and the avoidance of uncontrolled cracking between widely-spaced bars, are achieved by ensuring:

(a) That, at all sections likely to be subjected to significant tension due to restraint of imposed deformations, whether or not the restraint is combined with direct loading, a minimum amount of bonded reinforcement is present, sufficient to ensure that the reinforcement will remain elastic when cracking first occurs, and
(b) that bar spacings and diameters are limited.

(9) In the absence of specific requirements (e.g. watertightness), it may be assumed that for exposure Classes 2 to 4, limitation of the design crack width to about 0.3mm will generally be satisfactory for reinforced concrete elements of composite beams in buildings, in respect of appearance and durability.

(10) Special measures to limit crack widths may be necessary for members subjected to exposure Class 5. The choice of measures will depend on the nature of the aggressive chemical involved.

(11) Application rules are given in Sections 5.3.2 and 5.3.4 for design crack widths w_k of 0.3mm for general use except where the exposure is Class 5; and of 0.5mm which may be appropriate where the exposure is Class 1. It is assumed that the reinforcing bars have high-bond action.

(12) It may be found convenient to consider cracking of concrete in a composite beam in a building as follows:

(a) Determine those regions where concrete may be subjected to longitudinal tension due to loading and/or to restraint of imposed deformation, and determine the area of reinforcement required for the ultimate limit states.
(b) Decide the exposure class, and the crack-width limit (if any). Use (5) above if applicable.
(c) In regions where only minimum reinforcement is required, and crack widths are influenced more by imposed deformations than by loading, use Section 5.3.2. This gives the minimum area of tensile reinforcement and the maximum diameter for the reinforcing bars.
(d) In other regions, use Section 5.3.3 to determine internal forces and moment. Then use Section 5.3.4 if the crack-width limit is 0.3mm or 0.5mm. Otherwise, use Section 5.3.5. Section 5.3.4 gives the maximum spacing for reinforcing bars. The required areas are known (paragraph (a) above), so bar diameters can be calculated.)

5.3.2 Minimum Reinforcement

(1) In determining the minimum area of reinforcement required to ensure that the reinforcement remains elastic when cracking first occurs, account shall be taken of the different types of restraint distinguished in Section 5.3.2 of EBCS 2 and of the stress distribution in the concrete just before it cracks.

(2) Where crack widths are to be controlled in a concrete flange of a composite beam, (and unless more rigorous calculation shows a lesser area to be adequate), the cross-sectional area A_s of reinforcement within the effective area A_c of the concrete flange should satisfy.

$$A_s \geq kk_c f_{cte} A_c / \sigma_{st} \qquad (5.3)$$

where f_{cte} is the effective tensile strength of the concrete at the time when cracks may first be expected to occur. Values of f_{cte} may be obtained by taking as the class the strength at the time cracking is expected to occur, and using the value f_{ctm} given in Table 3.1. When the age of the concrete at cracking cannot be established with confidence as being less than 28 days, it is suggested that a minimum tensile strength of 3 MPa be adopted;

σ_{st} is the maximum stress permitted in the reinforcement immediately after cracking. It depends on the chosen bar size, as given in Table 5.1, and should not exceed the characteristic yield strength of the reinforcement;

k is defined in Section 5.3.2(3) of EBCS 2, and should be taken as 0.8;

k_c is a coefficient that may conservatively be taken as 0.9. It takes account of self-equilibrating stresses and the stress distribution in the slab prior to cracking, and is more accurately given by;

$$k_c = \frac{1}{1 + (h_c / 2z_o)} \geq 0.7 \qquad (5.4)$$

where h_c is the thickness of the concrete flange, excluding any haunch or ribs, and

z_o is the vertical distance between the centroids of the uncracked unreinforced concrete flange and the uncracked unreinforced composite section, calculated using the modular ratio for short term effects, E_a / E_{cm}.

At least half of the required minimum reinforcement shall be placed between mid-depth of the slab and the face subjected to the greater tensile strain.

(3) Minimum longitudinal reinforcement for the concrete encasement of the web of a steel I-section shall be determined from Eq. (5.1) with $k = 0.8$, $k_c = 0.4$, and $\sigma_{st} = f_{sk}$.

Table 5.1 Maximum Steel Stress for Minimum Reinforcement, High Bond Bars

Maximum Bar Size, mm	6	8	10	12	16	20	25	32
Design crack width	Maximum Steel Stress σ_s or σ_{st}, MPa							
w_k = 0.3 mm w_k = 0.3 mm	450 500	400 500	360 500	320 450	280 380	240 340	200 300	160 260

(4) For restraint cracking, but not for load-induced cracking, the maximum bar diameter may be modified to a value φ_s, where;

$$\varphi_s = \varphi'_s \frac{f_{cte}}{2.5} \tag{5.5}$$

where φ'_s is the bar diameter that relates to stress σ_{st}, in accordance with Table 5.1;
f_{cte} is as defined in Section 5.3.2(2)

5.3.3 Analysis of the Structure for the Control of Cracking

(1) Internal forces and moments shall be determined by elastic global analysis. The principles of Section 4.5.3 are applicable.

(2) The quasi-permanent combination of actions, defined in Section 2.8.4(2), should normally be used.

(3) The applicable, except that the limits to the redistribution of moments given in Table 4.3 are replaced by the following:

(a) For "cracked" elastic analysis, zero for sections of any class
(b) For "uncracked" elastic analysis:
  (i) 15% for hogging regions with cross-sections in Class 1 or 2,
  (ii) 10% for other regions of hogging bending moment.

5.3.4 Control of Cracking due to Direct Loading without Calculation of Crack Widths

(1) This Section is applicable in regions where the quantity of tensile reinforcement required to provide resistance to bending at ultimate limit states exceeds the minimum reinforcement required by Section 5.3.2.

(2) Tensile stresses in reinforcement shall be determined by elastic analysis of cross sections. The effect of tension stiffening in a composite section increase the tensile stress that is relevant to control of cracking to a value σ_s. Equation 5.6 may be used for reinforcement in a concrete flange of a composite beam:

$$\sigma_s = \sigma_{se} + \frac{0.4 f_{ctm} A_c}{\alpha A_s} \tag{5.6}$$

where σ_{se} is the stress in the reinforcement closest to the relevant concrete surface, calculated neglecting concrete in tension and in accordance with Sections 5.3.3 and 4.4.1.4(1), (2), and (4);
A_c is the area of the concrete flange within the effective width;
A_s is the total area of all layers of longitudinal reinforcement within the effective width;
f_{ctm} is the mean tensile strength of the concrete, from Table 3.4;
α is given by;

$$\alpha = AI/A_a I_a \tag{5.7}$$

where A and I are area and second moment of area, respectively, of the composite section neglecting concrete in tension and profiled sheeting, if any; and A_a an I_a are the corresponding properties of the structural steel section.

(3) In beams for buildings, σ_{st} may be calculated neglecting the effects of shrinkage of concrete, except as required by Section 4.5.3.3.

(4) If the stress σ_s is found to exceed the design yield strength for the reinforcement f_{sk} the section should be re-designed. This is not necessary if the maximum calculated stress in the structural steel exceeds its yield strength f_y, but σ_s does not exceed f_{sk}.

(5) Where the steel stress σ_s is within the range available in Table 5.2, the maximum spacing of the reinforcing bars should be determined from that Table.

Table 5.2 Maximum Bar Spacing for High Bond Bars

Steel Stress σ_s MPa		≤ 160	200	240	280	320	360	400
Maximum bar spacing (mm)	w_k = 0.3mm	250	200	160	110	use Table 5.1		
	w_k = 0.5mm	250	250	250	250	200	140	80

(6) Where Table 5.2 is not applicable, the maximum diameter of the reinforcing bars should be determined from Table 5.1, for the relevant values of σ_s taken as $0.5 f_{sk}$.

5.3.5 Control of Cracking by Calculation of Crack Widths

(1) The crack width to be compared with the design value w_k shall be calculated in accordance with Section 5.3.4 of EBCS 2.

(2) Tensile stresses in reinforcement shall be calculated taking account of tension stiffening in cracked concrete. In the absence of a more accurate method, σ_s may be calculated as given in Section 5.3.4.

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CHAPTER 6
SHEAR CONNECTION IN BEAMS FOR BUILDINGS

6.1 GENERAL

6.1.1 Basis of Design

(1) Shear connectors and transverse reinforcement shall be provided throughout the length of the beam to transmit the longitudinal shear force between the concrete slab and the steel beam for the ultimate limit state, ignoring the effect of natural bond between the two.

(2) The number of connectors shall be at least equal to the design shear force, determined according to 6.2, divided by the design resistance of a connector P_{Rd} determined according to Sections 6.3 or 6.5.

(3) If all cross-sections are in Class 1 or Class 2, partial shear connection may be used, if the design ultimate loading is less than that which could be carried by the member if full shear connection were provided. The number of connectors shall then be determined by a partial connection theory taking into account the deformation capacity of the shear connectors.

(4) Shear connectors shall be capable of providing resistance to uplift of the concrete slab.

(5) To prevent uplift of the slab, shear connectors should be designed for a nominal tensile force, perpendicular to the plane of the steel flange, of at least 0.1 times the design shear resistance of the connectors. If necessary they should be supplemented by anchoring devices.

(6) Headed stud shear connectors in accordance with Section 6.3.2 and 6.4.2 or 6.3.3 and 6.4.3 may be assumed to provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension.

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

(8) If the detailing of the shear connection is in accordance with 6.4 and the transverse reinforcement is in accordance with 6.6, it may be assumed that longitudinal shear failure and splitting is prevented.

(9) Methods of interconnection, other than the shear connectors covered in this Chapter, may be used to effect the transfer of longitudinal forces between a steel member and the slab, provided the adequacy with regard to behaviour and strength is demonstrated by tests and supported by a conceptual model. In such cases, the design of the composite beam shall conform to the design of a similar member employing either studs or other shear connectors as included in this code, in so far as practicable.

6.1.2 Deformation Capacity of Shear Connectors

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

(2) Headed studs with an overall length after welding not less than 4 times the diameter, and with a shank of diameter not less than 16mm and not exceeding 22mm, may be considered as ductile within the following limits for the degree of shear connection, which is defined by the ratio N/N_f.

(a) For steel sections with equal flanges:

$$L \leq 5 \quad \frac{N}{N_f} \geq 0.4 \tag{6.1a}$$

$$5 < L \leq 25 \quad \frac{N}{N_f} \geq 0.25 + 0.03L \tag{6.1b}$$

$$L > 25 \quad \frac{N}{N_f} \geq 1.0 \tag{6.1c}$$

(b) For steel sections having a bottom flange with an area not exceeding 3 times the area of the upper flange:

$$L \leq 20 \quad \frac{N}{N_f} \geq 0.4 + 0.03L \tag{6.2a}$$

$$L > 20 \quad \frac{N}{N_f} \geq 1.0 \tag{6.2b}$$

where L is the span in metres,
N_f is the number of shear connectors determined for the relevant length of beam in accordance with Section 6.2.1.1, and
N is the number of shear connectors provided within the same length of beam.

(3) Headed stud connectors may be considered as ductile over a wider range of spans than given in (2) above where:

(a) The studs have an overall length after welding not less than 76mm and a shank of diameter not less than 19mm and not exceeding 20mm
(b) The steel section is a rolled I or H with equal flanges
(c) The concrete slab is composite with profiled steel sheeting that spans perpendicular to the beam and is continuous across it
(d) There is one stud per rib of sheering, placed centrally within the rib
(e) For the sheeting, $b_o/h_p \geq 2$ and $h_p \leq 60$mm where the notation is as in Section 6.3.3.1
(f) The force F_c is calculated by the method of Section 6.2.1.2(3)

where these conditions are satisfied, the ratio N/N_f should satisfy:

$$L \leq 10 \quad \frac{N}{N_f} \geq 0.4 \tag{6.3a}$$

$$10 < L \leq 25 \quad \frac{N}{N_f} \geq 0.04L \tag{6.3b}$$

$$L > 25 \quad \frac{N}{N_f} \geq 1.0$$

where L, N and N_f are as defined in Section 6.1.2(2)

(4) Other shear connections having a characteristic slip capacity of not less than 6mm at the characteristic resistance, determined from push tests may be considered as having the same deformation capacity as headed studs with the dimensions given in (2) above.

6.1.3 Spacing of Shear Connectors

(1) The shear connectors shall be spaced along the beam so as to transmit longitudinal shear and to prevent separation between the concrete slab and the steel beam, considering an appropriate distribution of design longitudinal shear force.

(2) In cantilevers and negative moment regions of continuous beams, the shear connectors shall be spaced to suit the curtailment of tension reinforcement, ignoring the anchorage length of curtailed bars.

(3) Stud connectors in accordance with Sections 6.3.2 and 6.3.3 may be spaced uniformly over a length L_{cr}, between adjacent critical cross-sections as defined in Section 4.1.2 provided that:

- (a) All critical sections in the span considered are in Class 1 or Class 2
- (b) N/N_f satisfies the limit given by Section 6.1.2, when L is replaced by L_{cr}, and
- (c) the plastic resistance moment of the composite section does not exceed 2.5 times the plastic resistance moment of the steel member alone.

(4) 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.

(5) 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, unless the method of Section 4.4.4(7) for shear buckling resistance of a web is used.

6.2 LONGITUDINAL SHEAR FORCE

6.2.1 Beams in which Plastic Theory is Used for Resistance of Cross-Section

6.2.1.1 *Full Shear Connection*

(1) For full shear connection, the total design longitudinal shear V_l to be resisted by shear connectors spaced in accordance with 6.1.3 between the point of maximum sagging bending moment and a simple end support shall be:

$$V_l = F_{cf} \tag{6.4}$$

where $F_{cf} = \dfrac{A_a f_y}{\gamma_a}$

or $F_{cf} = \dfrac{0.85 A_c f_{ck}}{\gamma_c} + \dfrac{A_{se} f_{sk}}{\gamma_s}$

which ever is the smaller, and

A_a is the area of structural steel,

A_c is the effective area of concrete, defined in Section 4.2.1 and Section 4.2.2 excluding any web encasement,

A_{se} is the area of any longitudinal reinforcement in compression that is included in the calculation of the bending resistance,

and these areas relate to the cross-section at the point of maximum sagging bending moment.

(2) For full shear connection, the total design longitudinal shear V_l to be resisted by shear connectors spaced in accordance with Section 6.1.3 between the point of maximum sagging bending moment and an intermediate support or a restrained end support shall be:

$$V_l = F_{cf} + \frac{A_s f_{sk}}{\gamma_s} + \frac{A_{ap} f_{yp}}{\gamma_{ap}} \tag{6.5}$$

where A_s is the effective area of longitudinal slab reinforcement

A_{ap} is the effective area of any profiled steel sheeting, used in accordance with Section 4.2.1(4).

and these areas relate to the cross section at the support, and F_{cf} is as defined in (1) above, and is taken as zero for a cantilever.

6.2.1.2 *Partial Shear Connection with Ductile Connectors*

(1) If the connectors are ductile as defined in Section 6.1.2, it may be assumed that sufficient slip can occur at the ultimate limit state for moments of resistance at critical sections to be calculated from plastic theory in accordance with Section 4.4.1.3.

(2) In the absence of a more rigorous calculation the longitudinal shear V_l may be taken as:

$$V_l = F_c \tag{6.6}$$

between the considered cross-section with a sagging bending moment and a simple end support; and

$$V_l = F_c + \frac{A_s f_{sk}}{\gamma_s} + \frac{A_{ap} f_{yp}}{\gamma_{ap}} \tag{6.7}$$

between the considered cross-section with a sagging bending moment and an intermediate support or a restrained end support;

where F_c is the compressive force in the concrete flange necessary to resist the design sagging bending moment M_{Sd} calculated from plastic theory in accordance with Section 4.4.1.3, and the other symbols are as in Section 6.2.1.1.

The relation between F_c and M_{Sd} is qualitatively given by the curve ABC in Fig. 6.1.

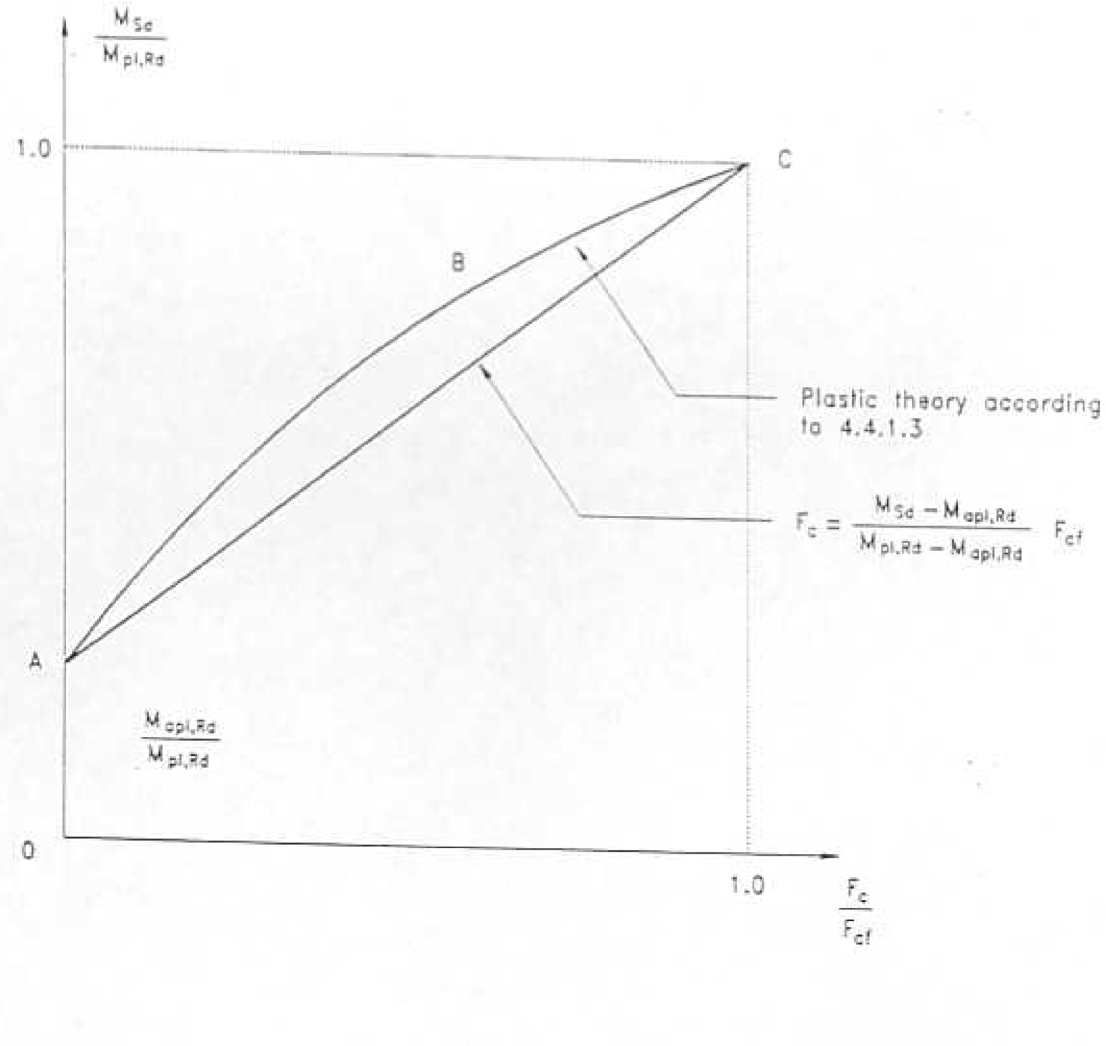


Figure 6.1 Relation Between F_c and M_{Sd}

(3) For the method given in (2) above a conservative value for F_c may alternatively be determined by the straight line AC in Fig. 6.1.

$$F_c = \frac{M_{Sd} - M_{apl,Rd}}{M_{pl,Rd} - M_{apl,Rd}} F_{cf} \qquad (6.8)$$

where $M_{apl,Rd}$ is the design plastic resistance to bending of the structural steel section alone.

6.2.1.3 *Partial Shear Connection with Non-Ductile Connectors*

(1) If the shear connectors are not ductile as defined in 6.1.2 the longitudinal shear shall be determined from stress distributions at the critical cross-sections based on full continuity at the interface between steel and concrete.

(2) The total design longitudinal shear V_l may be determined with the simplified method given in Section 6.2.1.2 except that F_c is determined from:

$$F_c \geq \frac{M_{Sd} - M_{a,Sd}}{M_{el,Rd} - M_{a,Sd}} F_{el} \qquad \text{for } M_{Sd} \leq M_{el,Rd} \tag{6.9}$$

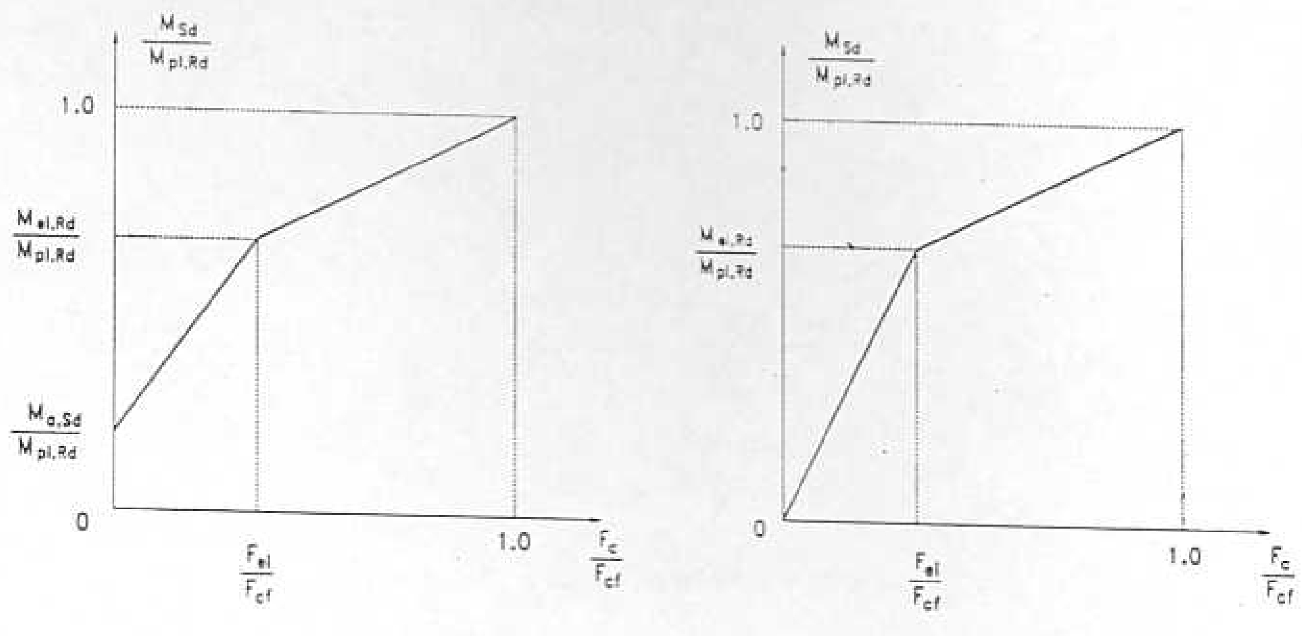
$$F_c \geq F_{el} + \frac{M_{Sd} - M_{el,Rd}}{M_{pl,Rd} - M_{el,Rd}} (F_{cf} - F_{el}) \quad \text{for } M_{el,Rd} \leq M_{Sd} < M_{pl,Rd} \tag{6.10}$$

where $M_{el,Rd}$ is the moment that causes a stress f_y/γ_a in the extreme bottom fibre of the steel section; where unpropped construction is used Section 4.4.1.4(4) is applicable,

$M_{a,Sd}$ is the moment acting in the steel section due to actions on the structural steelwork alone before the composite action becomes effective,

F_{el} is the compressive force in the concrete slab at moment $M_{el,Rd}$.

The relations between F_c and M_{Sd} are qualitatively given in Fig. 6.2.



a) Structures which are not fully supported during construction

b) Structures which are fully supported during construction

Figure 6.2 Relations Between F_c and M_{Sd}

6.2.2 Beams in which Elastic Theory is used for Resistances of One or More Cross Sections

(1) If elastic theory is applied to cross sections in accordance with Section 4.4.1.4, the longitudinal shear per unit length shall be calculated by elastic theory from the vertical shear force added after the shear connection has become effective. The elastic properties of the cross section shall be those used in the calculation of the longitudinal stresses.

6.3 DESIGN RESISTANCE OF SHEAR CONNECTORS

6.3.1 General

(1) Where the concrete slab is unhaunched, or the haunch satisfies Sections 6.3.3.1 or 6.4.1.4, the design resistance of shear connectors embedded in normal-density or lightweight-aggregate concrete (density greater than 1750 kg/m^3) should be calculated from the equations given in this Section.

(2) Where the concrete density or haunch dimensions do not satisfy the conditions in (1) above, or where other types of shear connectors are employed than covered in this Section, the design resistance should be determined in accordance with Section 3.4.2 using the characteristic resistance.

(Note: All references to the length of the stud refer to the length after welding.)

6.3.2 Stud Connectors in Solid Slabs

6.3.2.1 *Headed Studs - Shear Resistance*

(1) The design shear resistance P_{Rd} of an automatically welded headed stud with a normal weld collar, should be determined from;

$$P_{Rd} = 0.8\, f_u (\pi d^2/4) \gamma_v \tag{6.11}$$

or

$$P_{Rd} = 0.29 \alpha d^2 \sqrt{(f_{ck} E_{cm})}\, / \gamma_v \tag{6.12}$$

whichever is smaller,

where 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 500 MPa;

f_{ck} is the characteristic cylinder strength of the concrete at the age considered;

E_{cm} is the mean value of the secant modulus of the concrete in accordance with Section 3.1.5.2;

α $= 0.2[(h/d) + 1]$ for $3 \leq h/d \leq 4$;

α $= 1$ for $h/d > 4$, and

h is the overall height of the stud.

The partial safety factor γ_v should be taken as 1.25 for the ultimate limit state.

(2) These formula may not be used for studs of diameter greater than 22mm.

(3) A normal weld collar should comply with the following requirements:

(a) The weld should have a regular form and be fused to the shank of the stud.
(b) The diameter should be not less than $1.25d$.
(c) The mean height should be not less than $0.20d$ and the minimum height.
(d) Not less than $0.15d$.

6.3.2.2 *Influence of Tension on Shear Resistance*

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

If $F_{ten} \leq 0.1\ P_{Rd}$, the tensile force may be neglected.
If $F_{ten} > 0.1\ P_{Rd}$, the connection is not within the scope of this Code.

where P_{Rd} is the design shear resistance defined in Section 6.3.2.1.

6.3.2.3 *Studs Without Head - Shear Resistance*

(1) Equations (6.11) and (6.12) may be used for studs without heads, provided uplift of the slab is prevented. The ties which resist uplift should be designed for the ultimate limit state in accordance with Section 6.1.1(5).

6.3.3 Headed Studs Used with Profiled Steel Sheeting

6.3.3.1 *Sheeting with Ribs Parallel to the Supporting Beams*

(1) The studs are located within a region of concrete that has the shape of a haunch (Fig. 6.3). Where the sheeting is continuous across the beam, the width of the haunch b_o is equal to the width of the trough as given in Fig. 6.3a. Where the sheeting is not continuous, b_o is defined in a similar way as given in Fig. 6.3b. The depth of the haunch should be taken as h_p, the overall depth of the sheeting excluding embossments.

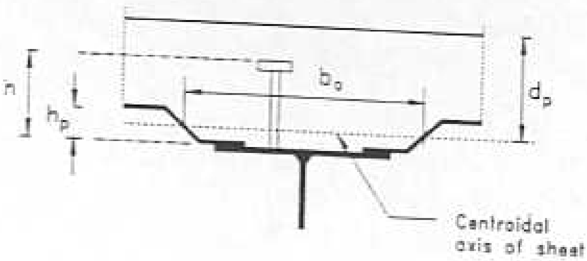


Figure 6.3a Beam with Profiled Steel Sheeting Continuous Across the Beam

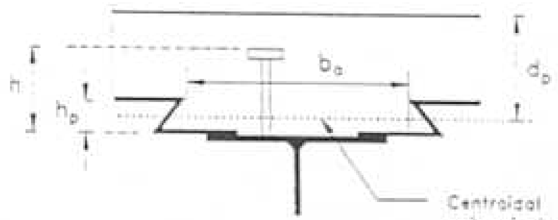


Figure 6.3b Beam with Profiled Steel Sheeting not Continuous Across the Beam

The design shear resistance should be taken as their resistance in a solid slab (see Section 6.3.2.1) multiplied by the reduction factor k_t given by the following expression:

$$k_l = 0.6\,(b_o/h_p)\,[(h/h_p) - 1] \leq 1.0 \qquad (6.13)$$

where h is the overall height of the stud, but not greater than h_p + 75mm.

6.3.3.2 *Sheeting with Ribs Transverse to the Supporting Beams*

(1) Where the studs are placed in ribs with a height h_p not exceeding 85mm and a width b_o not less than h_p, the design shear resistance should be taken as their resistance in a solid slab (see Section 6.3.2.1) multiplied by the reduction factor k_t given by the following expression:

$$k_t = (0.7/\sqrt{N_r})(b_o/h_p)\,[(h/h_p) - 1] \leq 1.0 \qquad (6.14)$$

where N_r is the number of stud connectors in one rib at a beam intersection, not to exceed 2 in computations,
and other symbols are as defined in Section 6.3.3.1.

(2) For the other cases, not within the scope of (1) above, the design resistance should be determined from tests.

6.3.3.3 *Biaxial Loading of Shear Connectors*

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

$$\frac{F_l^2}{P_{l,Rd}^2} + \frac{F_t^2}{P_{t,Rd}^2} < 1 \qquad (6.15)$$

where F_l is the design longitudinal force caused by composite action in the beam, and
F_t is the design transverse force caused by composite action in the slab.

6.3.4 Block Connectors in Solid Slabs

(1) Connectors may be designed as block connectors if the front is not wedge shaped and is so stiff that it can reasonably by assumed that at failure the pressure on the concrete in front of the connector is uniformly distributed.

(2) Bar, T-, [and horseshoe connectors may be designed as block connectors if the detailing provisions in Section 6.4.4 are satisfied.

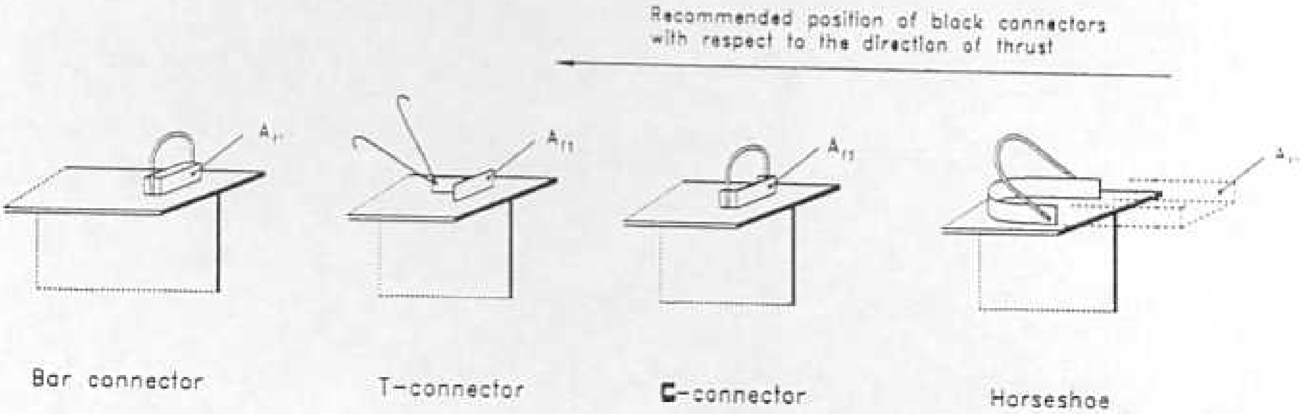


Figure 6.4 Block Connectors

(3) The design resistance of a block connector should be determined from

$$P_{Rd} = \eta A_{f1} f_{ck} / \gamma_c \qquad (6.16)$$

where A_{f1} is the area of the front surface, as shown in Fig. 6.4;

η is equal to $\sqrt{(A_{f2}/A_{f1})}$ but not greater than 2.5 for normal density concrete of 2.0 for lightweight aggregate concrete;

A_{f2} is the area of the front surface of the connector enlarged at a slope of 1:5 to the rear surface of the adjacent connector (Fig. 6.5). Only the parts of A_{f2} falling within the concrete section may be taken into account;

γ_c is the partial safety factor for concrete in accordance with Section 3.5.3 of EBCS 2.

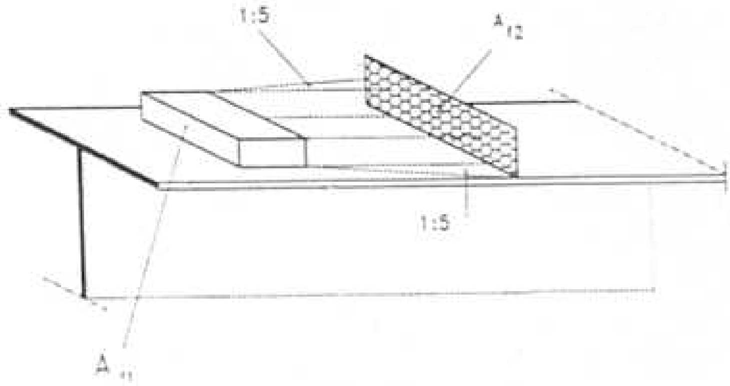


Figure 6.5 Definition of A_{f2}

(4) In the design of the welds fastening the block connector to the steel beam the eccentricity of the force shall be taken into account.

(5) The welds should be designed in accordance with Section 6.5. of EBCS 3 for 1.2 P_{Rd}.

(6) Ties to prevent uplift shall be designed in accordance with Section 6.1.1.

6.3.5 Anchors and Hoops in solid Slabs

(1) The design resistance to longitudinal shear for each leg of anchors and hoops should be determined from

$$P_{Rd} = \frac{A_h f_{yd}}{\sqrt{(1 + \sin^2\alpha)}} \cos\beta \tag{6.17}$$

where A_h is the cross-sectional area of the anchor or the hoop,
α is the angle between the anchor bar or the hoop and the plane of the flange of the beam,
β is the angle in the horizontal plane between the anchor bar and the longitudinal axis of the beam for anchors set at a splay,
f_{yd} is the design strength of the material of the bar, to be taken as f_y/γ_a or f_{sk}/γ_s whichever is applicable,
γ_a, γ_s are the partial safety factors for structural steel and reinforcement in accordance with Section 3.5.3 of EBCS 2.

(2) Anchor and hoop, and also a combination of block connection with anchor and hoop, may be used to provide shear connection between a steel member and concrete slab forming a composite beam. Examples are shown in Figs. 6.6 and 6.7.

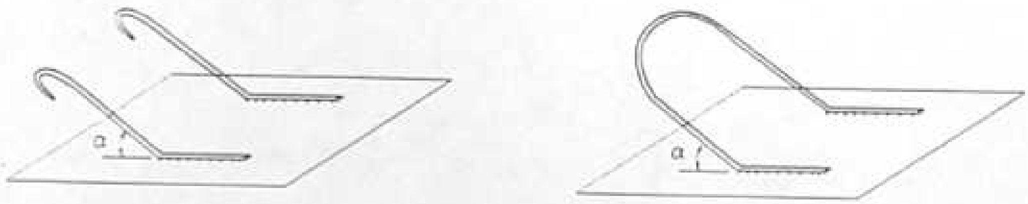


Figure 6.6 Example of Anchor and Hoop

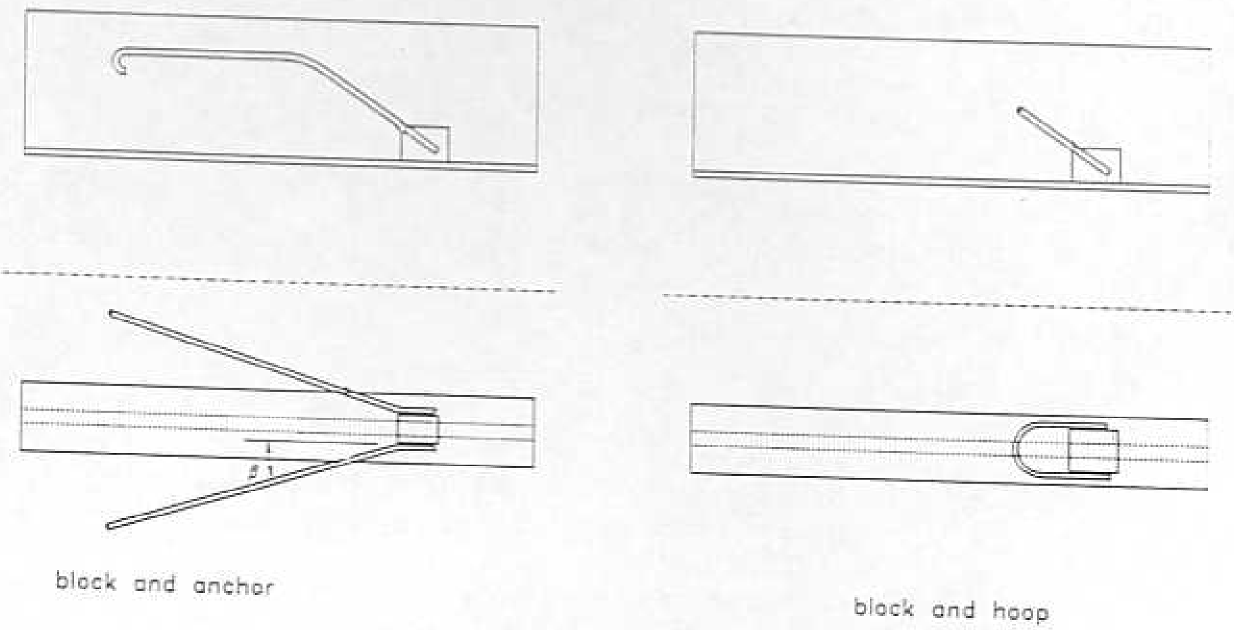


Figure 6.7 Example of Combination of Block Connector with Anchor and Hoop

6.3.6 Block Connectors with Anchors or Hoops in Solid Slabs

(1) Block connectors may be assumed to share load with anchors or hoops, provided due account is taken of the differences of stiffness of the block connector and the anchors or the hoop.

(2) In the absence of more accurate calculations or tests, the design resistance of the combination should be determined from one of the following expressions, whichever is applicable.

$$P_{Rd}\ \text{comb} = P_{Rd}\ \text{block} + 0.5\ P_{Rd}\ \text{anchors} \tag{6.18}$$

$$P_{Rd}\ \text{comb} = P_{Rd}\ \text{block} + 0.7\ P_{Rd}\ \text{hoop} \tag{6.19}$$

(3) The welds fastening the block connector with anchors or hoop to the steel beam should be designed for 1.2 P_{Rd} for the block plus P_{Rd} for each anchor or the hoop.

6.3.7 Angle Connectors in Solid Slabs

(1) The design resistance of an angle connector welded to the steel beam as shown in Fig. 6.8 should be determined from

$$P_{Rd} = 10\, bh^{3/4} f_{ck}^{2/3} / \gamma_v \tag{6.20}$$

where P_{Rd} is in Newtons,
b is the length of the angle in mm,
h is the height of the upstanding leg of the angle in mm,
f_{ck} is the characteristic strength of concrete in MPa.

The partial safety factor γ_v should be taken as 1.25 for the ultimate limit state.

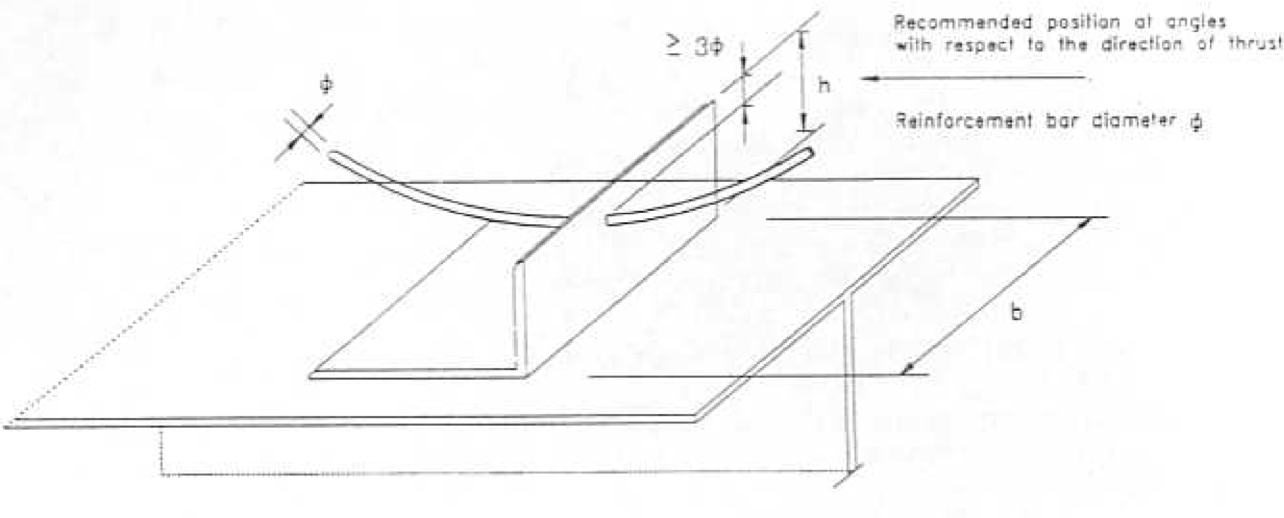


Figure 6.8 Angle Connector

(2) In the design of the welds fastening the angle to the steel beam the eccentricity of the force should be taken as $e = h/4$.

(3) The welds should be designed in accordance with Section 6.5 of EBCS 3 for 1.2 P_{Rd}.

(4) The reinforcement used to prevent uplift should be determined from:

$$A_s f_{sk} / \gamma_s \geq 0.1\ P_{Rd} \tag{6.21}$$

where A_s is the cross-sectional area of the bar, $\pi\varphi^2/4$
f_{sk} is the characteristic yield strength of the reinforcement,
γ_s is the partial safety factor for reinforcement in accordance with Section 3.5.3 of EBCS 2.

6.4 DETAILING OF THE SHEAR CONNECTION

6.4.1 General Recommendations

6.4.1.1 *Resistance to Separation*

The surface of a connector that resists separation forces (that is the inside of a hoop or the underside of the head of a stud) shall extend not less than 30mm clear above the bottom reinforcement.

6.4.1.2 *Cover and Compaction of Concrete*

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

(2) If cover over the connector is required, it should be not less than 20mm.

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

6.4.1.3 *Local Reinforcement in the Slab*

(1) Where the shear connection is adjacent to a longitudinal edge of a concrete slab, transverse reinforcement provided in accordance with 6.6 shall be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors (see Section 6.6.5).

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

6.4.1.4 *Haunches Other than Formed by Profiled Steel Sheeting*

(1) Where a concrete haunch is used between the steel girder 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 (Fig. 6.9).

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

(3) Transverse reinforcing bars sufficient to satisfy the requirements of 6.6 should be provided in the haunch at least 40mm clear below the surface of the connector that resists uplift.

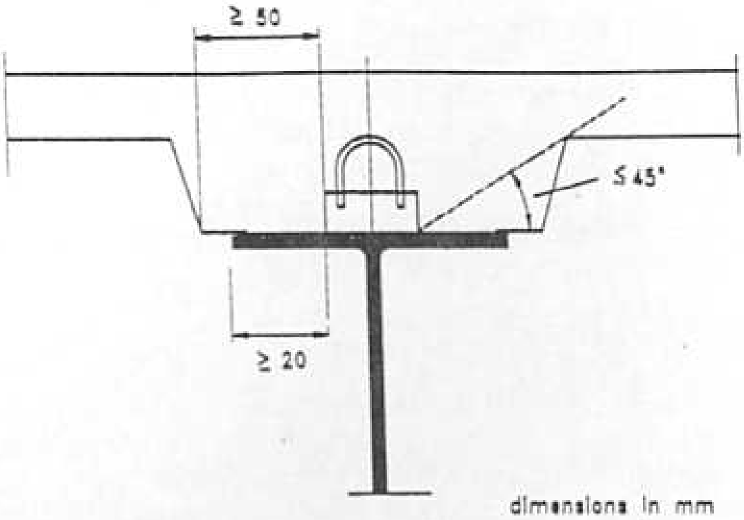


Figure 6.9 Dimensions of Haunches

6.4.1.5 *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 a lower class, is assumed to be in Class 1 or Class 2 because of restraint from shear connectors, the centre-to-centre spacing of the shear connectors in the direction of compression should not exceed the following limits:

where the slab is in contact over the full length(e.g. solid slab)

$$S_1 \leq 22\ t\ \sqrt{235/f_y} \qquad (6.22)$$

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

$$S_1 \leq 15\ t\ \sqrt{235/f_y} \qquad (6.23)$$

The clear distance from the edge of the compression flange to the nearest line of shear connectors should not exceed

$$S_2 \leq 9\ t\ \sqrt{235/f_y} \qquad (6.24)$$

where t is the thickness of the flange, and
f_y is the nominal yield strength of the flange in MPa

(3) The maximum longitudinal centre-to-centre spacing of shear connectors should not exceed 6 times the total slab thickness nor 800mm.

(4) Alternatively, connectors may be placed in groups, with the spacing of groups greater than that specified for individual connectors, provided that consideration is given in design to the non-uniform flow of longitudinal shear, to the greater possibility of slip and vertical separation between the slab and the steel member, and to buckling of the steel flange.

6.4.1.6 *Dimensions of the Steel Flange*

(1) The thickness of the steel plate or flange to which a connector is welded shall be sufficient to allow proper welding and proper transfer of load from the connector to the plate without local failure or excessive deformation. (For studs see 6.4.2(4)).

(2) The distance between the edge of a connector and the edge of the flange of the beam to which it is welded should be not less than 20mm (Fig. 6.9).

6.4.2 Stud Connectors

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

(2) A stud connector should have a head in accordance with Section 3.4.2(6) or be provided with ties to resist separation forces in accordance with Section 6.1.1.

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

(4) Except when the studs are located directly over the web, the diameter of a welded stud should not exceed 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 shear connector.

6.4.3 Headed Studs Used with Profiled Steel Sheeting

6.4.3.1 *General*

(1) Studs may be welded through the steel sheeting provided that it is shown by procedure trails that the quality can be consistently achieved. Otherwise, holes for placing studs shall be made through the sheets as necessary.

(2) It is possible to weld through a profiled steel sheet overlapping an edge trim. The sheets should be in close contact and the total thickness of the sheeting should not exceed 1.25mm if galvanised or 1.5mm if not galvanised. The maximum thickness of galvanising should not exceed 30 microns on each sheet face.

(**Note:** Welding through two galvanised profiled steel sheets is not recommended).

(3) After installation, the connectors should extend not less than $2d$ above the top of the steel deck, where d is the diameter of the shank.

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

6.4.3.2 *Sheeting with Ribs Transverse to the Supporting Beams*

(1) The profiled seel sheeting should be anchored in each trough to each steel beam designed for composite action. Such anchorage may be provided by stud connectors, a combination of stud connectors and are spot (puddle) welds, or other devices specified by the designer.

6.4.4 Block Connectors

(1) The height of a bar connector should not exceed 4 times the thickness.

(2) A T-connector should be a hot rolled section of part of it with a flange width not exceeding 10 times the flange thickness.

(3) The height of a T-connector should not exceed 10 times the flange thickness nor 150mm.

(4) A [connector should be a hot rolled section with a web width not exceeding 25 times the web thickness.

(5) The height of a [connector should not exceed 15 times the web thickness nor 150mm.

(6) The height of a horseshoe should not exceed 20 times the web thickness nor 150mm.

6.4.5 Anchors and Hoops

(1) The anchorage length and the concrete cover shall be in accordance with EBCS 2.

(2) A hoop may be assumed to be sufficiently anchored when the following conditions are met:

$r > 7.5\,\phi$, $l > 4r$, and concrete cover $> 3\,\phi$,

where the symbols are as shown in Fig. 6.10.

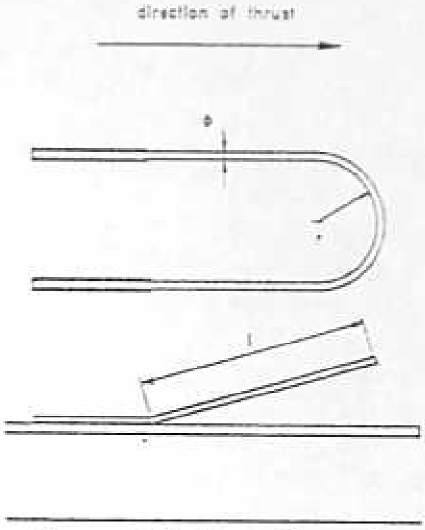


Figure 6.10 Hoop Connector

(3) The anchors and hoops designed for longitudinal shear should point in the direction of thrust. Where thrust can occur in both directions, connectors pointing in both directions should be provided.

6.4.6 Angle Connectors

(1) The height h of the upstanding leg of an angle connector should not exceed 10 times the thickness nor 150mm.

(2) The length b of an angle connector should not exceed 300mm unless the resistance is determined by testing.

6.5 FRICTION GRIP BOLTS

6.5.1 General

(1) High strength friction grip bolts may be used to provide a shear connection between a steel member and a precast concrete slab forming a composite beam. An example is shown in Fig. 6.11.

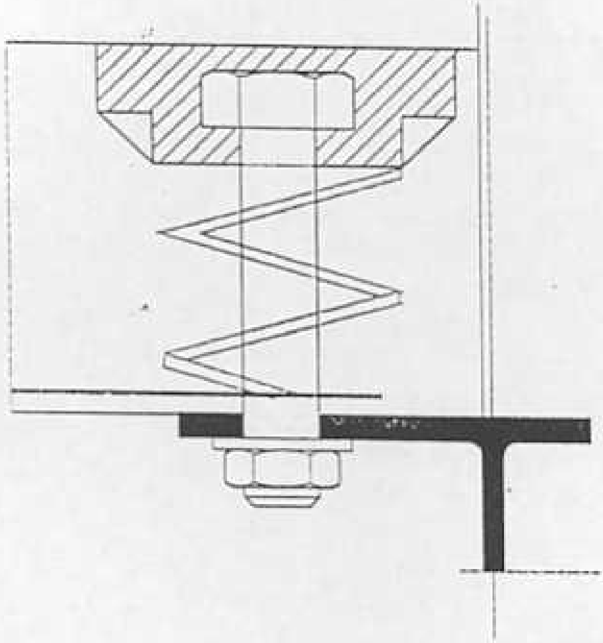


Figure 6.11 Example for Shear Connections with Friction-Grip Bolts

6.5.2 Ultimate Limit State

6.5.2.1 *Design Friction Resistance*

(1) The design friction resistance per bolt should be determined using accepted procedures. Section 6.2.5 of EBCS 3 has to be seen.

6.5.2.2 *Design Resistance of a Bolt in Shear and Bearing*

Where the shear resistance is assumed to be developed by the resistance of the bolts alone in shear and bearing, the maximum design longitudinal shear per bolt should not exceed the design shear resistance of a bolt, determined in accordance with Section 6.2.4 of EBCS 3, nor the bearing resistance, which may be taken equal to P_{Rd} determined with Eq. 6.12.

6.5.2.3 *Combined Resistance*

Where the shear resistance is assumed to be developed by a combination of friction and shear, the combined shear resistance should be established by suitable testing.

6.5.2.4 *Effects of Slip*

The effects of slip may be neglected for verifications at the ultimate limit state in beams with cross-sections in Class 1 and 2 and holes with a clearance not exceeding 3mm

6.5.3 Serviceability Limit State

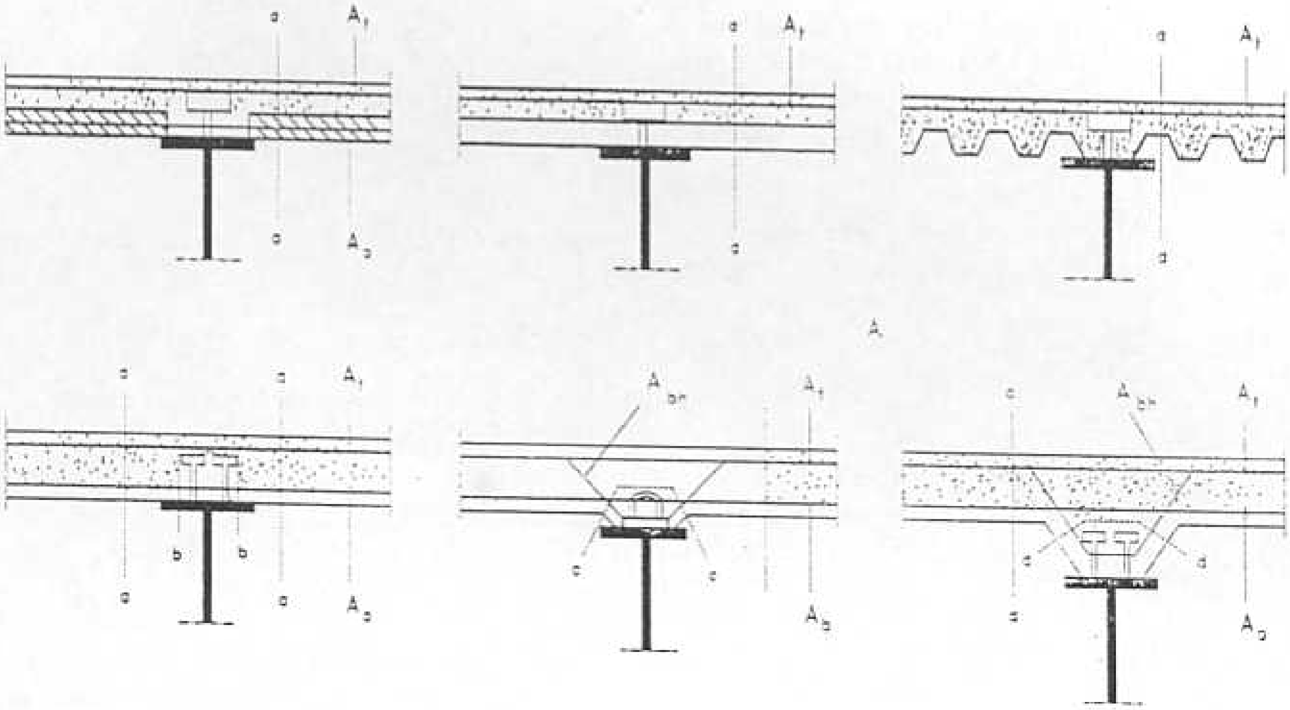
(1) Slip shall be limited to a level such that the Principles of Chapter 5 are fulfilled.

6.6 TRANSVERSE REINFORCEMENT

6.6.1 Longitudinal Shear in the Slab

(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 per unit length for any potential surface of longitudinal shear failure within the slab V_{Sd} shall not exceed the design resistance to longitudinal shear V_{Rd} of the shear surface considered.



type	A_e
a - a	$(A_b + A_t)$
b - b	$2A_b$
c - c	$2(A_b - A_{bh})$
d - d	A_{bh}
e - e	A_t

Figure 6.12 Typical Potential Surfaces of Shear Failure

(3) The length of the shear surface *b-b* shown in Fig. 6.12 should be taken as equal to $2h$ plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to

$2h + s_t$ plus the head diameter for stud shear connectors arranged in pairs, where h is the height of the studs and s_t is the transverse spacing centre-to-centre of the studs.

(4) Where profiled steel sheeting is used transverse to the beam it is not necessary to consider shear surfaces of type *b-b*, provided that the design resistances of the studs are determined using the appropriate reduction factor k_t as given in Section 6.3.3.2.

(5) The design longitudinal shear per unit length of beam in a shear surface V_{Sd} shall be determined in accordance with 6.2 and be consistent with the design of the shear connectors for the ultimate limit state.

(6) In determining V_{Sd} account may be taken of the variation of longitudinal shear across the width of the concrete flange.

6.6.2 Design Resistance to Longitudinal Shear

(1) The design resistance of the concrete flange (shear planes *a-a* illustrated in Fig. 6.12) shall be determined in accordance with the principles in Section 4.5 of EBCS 2. Profiled steel sheeting with ribs transverse to the steel beam may be assumed to contribute to resistance to longitudinal shear, if it is continuous across the top flange of the steel beam or if it is welded to the steel beam by stud shear connectors.

(2) In the absence of a more accurate calculation the design resistance of any surface of potential shear failure in the flange or a haunch should be determined from:

$$V_{Rd} = 2.5\, A_{cv}\, \eta\, \tau_{Rd} + A_e f_{sk} / \gamma_s + V_{pd} \tag{6.25}$$

or

$$V_{Rd} = 0.2\, A_{cv}\, \eta\, f_{ck} / \gamma_c + V_{pd} / \sqrt{3} \tag{6.26}$$

whichever is smaller,

where τ_{Rd} is the basic shear strength to be taken as $0.25\, f_{ctk\,0.05} / \gamma_c$.
f_{ck} is the characteristic cylinder strength of the concrete in MPa units,
f_{sk} is the characteristic yield strength of the reinforcement,
$\eta =$ 1 for normal-weight concrete,
$\eta =$ $0.3 + 0.7(\rho/24)$ for lightweight-aggregate concrete of unit weight ρ in kN/m_3,
A_{cv} is the mean cross-sectional area per unit length of beam of the concrete shear surface under consideration,
A_e is the sum of the cross-sectional areas of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the shear surface under consideration (Fig. 6.12) including any reinforcement provided for bending of the slab,
V_{pd} is the contribution of the steel sheeting, if applicable, as given in Section 6.6.3.

(3) For a ribbed slab the area of concrete shear surface A_{cv} should be determined taking into account of the effect of the ribs. Where the ribs run transverse to the span of the beam, the concrete within the depth of the ribs may be included in the value of A_{cv} in Eq. 6.25; but for potential shear surfaces of type *e-e* in Fig. 6.12, it should not be included in A_{cv} in Eq. 6.26.

(4) Transverse reinforcement taken into account for resistance to longitudinal shear shall be anchored so as to develop its yield strength in accordance with EBCS 2.

(5) Anchorage may be provided by means of U-bars looped around the shear connectors.

6.6.3 Contribution of Profiled Steel Sheeting

(1) Where the profiled steel sheets are continuous across the top flange of the steel beam, the contribution of profiled steel sheeting with ribs transverse to the beam should be taken as

$$v_{pd} = \frac{A_p f_{yp}}{\gamma_{ap}} \tag{6.27}$$

where v_{pd} is per unit length of the beam for each intersection of the shear surface by the sheeting,
A_p is the cross-sectional area of the profiled steel sheeting per unit length of the beam, and,
f_{yp} is its yield strength, given in Section 3.3.2.

(2) 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 contribution of the profiled steel sheeting should be taken as

$$v_{pd} = \frac{P_{pb,Rd}}{s_1} \quad \text{but} \quad \leq \frac{A_p f_{yp}}{\gamma_{ap}} \tag{6.28}$$

where $P_{pb,Rd}$ is the design bearing resistance of a headed stud welded through the sheet, according to Eq. 6.29, and
s_1 is the longitudinal spacing centre-to-centre of the studs.

$$P_{pb,Rd} = k_\varphi d_{do} t f_{yp} / \gamma_{ap} \tag{6.29}$$

where $k_\varphi = 1 + a/d_{do} \leq 4.0$
d_{do} is the diameter of weld collar which may be taken as 1.1 times the diameter of the shank of the stud,
a is the distance from the centre of the stud to the end of the sheeting to be not less than $2d_{do}$, and
t is the thickness of the sheeting.

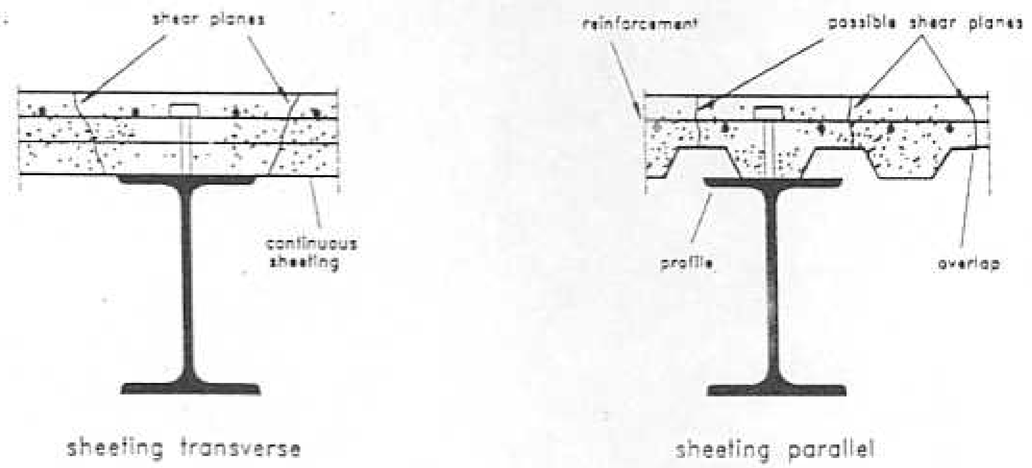


Figure 6.13 Potential Shear Surfaces in a Slab with Profiled Steel Sheeting

6.6.4 Minimum Transverse Reinforcement

6.6.4.1 *Solid Slabs*

The area of reinforcement in a solid slab should be not less than 0.002 times the concrete area being reinforced and should be uniformly distributed.

6.6.4.2 *Ribbed Slabs*

(1). Where the ribs are parallel to the beam span, the area of transverse reinforcement should be not less than 0.002 times the concrete cover slab area in the longitudinal direction and should be uniformly distributed.

(2) Where the ribs are transverse to the beam span the area of transverse reinforcement should be not less than 0.002 times the concrete slab area in the longitudinal direction and should be uniformly distributed. Profiled steel sheets continuous across the top flange of the steel beam may be assumed to contribute to this requirement.

6.6.5 Longitudinal Splitting

(1) To prevent longitudinal splitting of the concrete flange caused by the shear connectors, the following additional recommendations should be applied in all composite beams where the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300mm:

(a) Transverse reinforcement should be supplied by U-bars passing around the shear connectors
(b) Where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than 6d, where d is the nominal diameter of the stud, and the U-bars should be not less than $0.5d$ in diameter
(c) The U-bars should be placed as low as possible while still providing sufficient bottom cover

(Note: These conditions normally apply to edge beams but may also occur adjacent to large openings).

CHAPTER 7
FLOORS WITH PRECAST CONCRETE SLABS FOR BUILDINGS

7.1 GENERAL

(1) This Chapter deals with reinforced or prestressed precast concrete slabs or planks, used either as floors spanning between steel beams or as permanent formwork for insitu concrete.

(2) The precast elements shall be designed in accordance with the relevant chapters of EBCS 2 and also for composite action with the steel beams.

7.2 ACTIONS

(1) Attention shall be given to the local effects of heavy concentrated loads applied above joints between precast elements.

(2) The following loads shall be taken into account in calculations for precast elements as permanent form work:

(a) Weight of insitu concrete and precast elements
(b) Construction loads, including local heaping of concrete during construction, and storage load
(c) Ponding effect (increased depth of insitu concrete due to deflection of the precast elements)

7.3 PARTIAL SAFETY FACTORS FOR MATERIALS

(1) For the structural steel, any reinforcement that is embedded in insitu concrete, and the insitu concrete, the safety factors given in Section 2.8.3 shall be used.

(2) Partial safety factors for materials within precast concrete elements shall be in accordance with the appropriate Parts of EBCS 1.

7.4 DESIGN, ANALYSIS, AND DETAILING OF THE FLOOR SYSTEM

7.4.1 Support Arrangements

(1) The precast floor elements may be designed as simply supported or as continuous. The connections between elements at their support should be designed and detailed accordingly.

(2) The top reinforcement of continuous or cantilevering precast floors should be anchored in the precast elements or in a structural topping layer.

7.4.2 Joints Between Precast Elements

(1) When the floor is considered as monolithic, joints between precast elements shall be designed for all internal forces and moments that are required to be transferred from one element to another.

(2) Vertical shear force may be transferred between adjacent elements by the lapping of projecting reinforcement, or by other shear transferring connections, e.g. by shaping the joints as in Fig. 7.1.

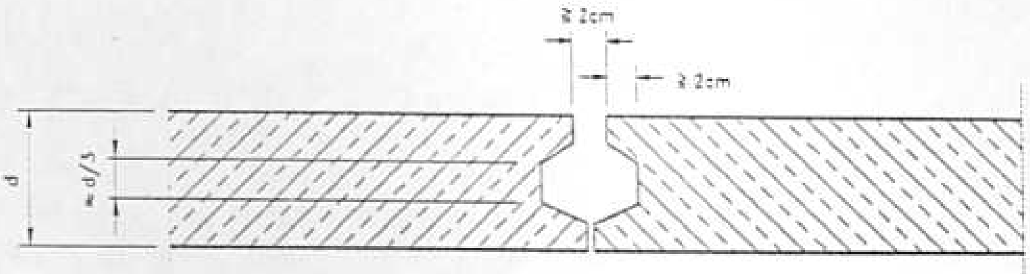


Figure 7.1 Joints Between Precast Floor Elements

7.4.3 Interfaces

(1) Interfaces between insitu concrete and precast elements used as permanent formwork should be detailed and constructed to enable the completed floor to be considered as monolithic in design.

7.5 JOINT BETWEEN STEEL BEAMS AND CONCRETE SLAB

7.5.1 Bedding and Tolerances

(1) Where a precast slab is supported on steel beams with or without bedding, the thickness of any bedding used and the vertical tolerances of the bearing surfaces shall be such that local stresses in the concrete slab are not excessive.

(2) Particular care should be taken when friction-grip bolting according to Section 6.5 is used.

7.5.2 Corrosion

(1) The protection of the steel top flange against corrosion throughout the life of the structure shall be considered.

7.5.3 Shear Connection and Transverse Reinforcement

(1) The shear connection and the transverse reinforcement shall be designed in accordance with Section 7.4.3 and the relevant clauses in Chapter 6.

(2) If shear connectors welded to the steel beam project into recesses within slabs or joints between slabs, which are filled with concrete or mortar after erection, the detailing shall be such that the infill can be fully compacted.

(3) In the absence of relevant experience, the minimum thickness of infill around each shear connector should be at least 25mm.

(4) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the precast or the insitu concrete.

(5) Where a joint between precast elements is parallel to and above the steel beam, continuous transverse reinforcement need not be provided for horizontal shear if the recommendations of Sections 6.4.1.3 and 6.6 are followed for each of the two slabs independently.

7.6 CONCRETE FLOOR DESIGNED FOR HORIZONTAL LOADING

(1) If a concrete floor is designed as a beam or a diaphragm for horizontal loading (for example, from wind), account shall be taken of any interaction between the resulting shear forces and those due to composite action, as these may add up in joints between the concrete elements. The resulting tensile forces may also require additional reinforcement in slabs or across joints.

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(3) When unpropped construction is used, measures should be taken to limit additional thickness of the floor slabs resulting from deflections of the steel beams, unless extra thickness of concrete be taken into account in the final design.

8.4.2 Compaction of Concrete

(1) Special attention should be paid to the achievement of satisfactory compaction around shear connectors and in concrete-filled steel tubes.

8.4.3 Shear Connection in Beams and Columns

8.4.3.1 *Headed Studs in Structures for Buildings*

(1) The proper duration of the welding period and the strength of the current shall be determined on the basis of trial weldings under site conditions, and tests in accordance with the appropriate standards in force.

(2) The quality of the stud welding shall be checked by visual inspection. Special attention shall be given to the weld collar and the length of the stud. Any studs with defective welding shall be replaced. In addition, a number of studs specified and selected as specified in these documents shall be bent, until the head of each such stud is displaced laterally from its original position a distance of approximately one quarter of the height of the stud. The stud weld shall not show any signs of cracking. The satisfactory studs shall be left in the bent position.

(3) Studs should not be welded to contaminated steel surfaces (e.g. water, moisture, grease etc.)

8.4.3.2 *Anchors, Hoops, Block Connectors*

(1) The welding of anchors, hoops and block connectors shall be in accordance with the relevant clauses of EBCS 3.

(2) Anchors and hoops that shall be welded should comply with the conditions of weldability given in EBCS 2. They may be either butt welded or bent and fillet welded. When fillet welding is used, the bend adjacent to the weld should be made in the red hot condition.

8.4.3.3 *Friction Grip Bolts*

(1) The interface between the steel member and the concrete flange shall be free of paint or other applied finishes, oil, dirt, rust, loose mill scale, burrs and other defects which would prevent a uniform sealing between the two elements, or would interfere with the development of friction between them.

(2) The method should be in accordance with the relevant Sections of Chapter 7 of EBCS 2.

8.4.3.4 *Corrosion Protection in the Interface*

(1) Steel parts of composite beams in buildings in general need not be protected against corrosion unless particular corrosion action has to be taken into account. If the steel parts must be protected against corrosion by painting, the painting may also be applied in the interface and to the shear connectors.

(2) Where protection from corrosion is required without the interface and shear connectors being fully painted, the protection should extend at least 30mm into the interface.

CHAPTER 8
EXECUTION

8.1 GENERAL

(1) This chapter specifies the minimum standards of workmanship required during execution to ensure that the design assumptions of this Code are satisfied and hence that the intended level of safety can be attained.

(2) This chapter gives specific recommendations related to the design of composite structures. In addition, the relevant clauses of appropriate Parts of EBCS 2 and EBCS 3 are applicable to composite structures.

(3) This chapter is neither intended, nor extensive enough, for a contract document.

(4) This chapter defines what is to be provided, irrespective of the persons who will have the responsibility for providing it.

8.2 SEQUENCE OF CONSTRUCTION

(1) The sequence of construction shall be compatible with the design (for example, because of its influences on stresses, shear connection and deflections). All information necessary to ensure this compatibility shall be clearly indicated and described on the final drawings and specifications.

(2) These shall include instructions for control measurements in the different phases construction, if appropriate.

(3) The speed and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations.

8.3 STABILITY

(1) The stability of the steelwork shall be ensured during construction, particularly before the development of composite action.

(2) It shall not be assumed that permanent or temporary formwork provides restraint to steel members susceptible to buckling unless it has been demonstrated that the formwork and its fixings are capable of transferring sufficient restraining forces from the supports to the steel member.

8.4 ACCURACY DURING CONSTRUCTION AND QUALITY CONTROL

8.4.1 Static Deflection During and After Concreting

(1) Section 5.2 and the following provisions are applicable.

(2) The shuttering and the supporting structure should be such that they can follow without deterioration the deflections of the steel beams that are assumed to occur during concreting.

(3) When unpropped construction is used, measures should be taken to limit additional thickness of the floor slabs resulting from deflections of the steel beams, unless extra thickness of concrete be taken into account in the final design.

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