A **cement** and **concrete** industry publication

Concise Eurocode 2 for Bridges

For the design of concrete bridges to BS EN 1992-1-1 and BS EN 1992-2 and their National Annexes

O Brooker BEng CEng MICE MIStructE
P A Jackson BSc(Hons) PhD CEng FICE FIStructE
S W Salim BEng(Hons) PhD CEng MICE

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Foreword

The introduction of European standards to UK construction is a significant event as for the first time all design and construction codes within the EU will be harmonised. The ten design standards, known as the Eurocodes, will affect all design and construction activities as current British Standards for structural design are due to be withdrawn in March 2010.

The cement and concrete industry recognised the need to enable UK design professionals to use *Eurocode 2, Design of concrete structures*, quickly efficiently and with confidence. Supported by government, consultants and relevant industry bodies, the Concrete Industry Eurocode 2 Group (CIEG) was formed in 1999 and this group has provided the guidance for a coordinated and collaborative approach to the introduction of Eurocode 2.

As a result, a range of resources is being developed and made available through The Concrete Centre (see www.eurocode2.info). One of those resources, *Concise Eurocode 2*, published in 2006, is targeted at structural engineers designing concrete framed buildings. Whilst there are many similarities in the design of buildings and bridges, there are also significant differences and hence Eurocode 2 has a distinct part for the design of bridges. This publication is based on the style of *Concise Eurocode 2*, but has been completely revised and rewritten to suit the requirements of Eurocode 2, Part 2 and the current design practices for concrete bridge design.

Relevant extracts have been incorporated from *Precast Eurocode 2: Design manual* published by British Precast, which is a similar document for designers of precast concrete. The authors are grateful for the permission granted by British Precast.

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Riverside House, 4 Meadows Business Park, Station Approach, Blackwater, Camberley, Surrey GU17 9AB
Tel: +44 (0)1276 606800 **Fax:** +44 (0)1276 606801 **www.concretecentre.com**

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Concise Eurocode 2 for Bridges

Contents

Symbols and abbreviations used in this publication

Symbol	Definition
$\lvert x \rvert$	Absolute value of x
$1/r$	Curvature at a particular section
A	Cross-sectional area; Accidental action
A, B, C	Variables used in the determination of λ_{lim}
A_c	Cross-sectional area of concrete
$A_{c,eff}$	Effective area of concrete in tension
A_{ct}	Area of concrete in that part of the section that is calculated to be in tension just before the formation of the first crack
A_d	Design value of an accidental action
A_k	Area enclosed by the centre lines of connecting walls including the inner hollow area (torsion)
A_p	Area of prestressing tendon or tendons
A_p'	Area of prestressing tendons within $A_{c,eff}$
A_s	Cross-sectional area of reinforcement
$A_{s,min}$	Minimum cross-sectional area of reinforcement
$A_{s,prov}$	Area of steel provided
$A_{s,req}$	Area of steel required
A_{s1}	Area of reinforcing steel in layer 1
A_{s2}	Area of compression steel (in layer 2)
A_{sl}	Area of the tensile reinforcement extending at least $l_{bd} + d$ beyond the section considered
A_{sM} (A_{sN})	Total area of reinforcement required in symmetrical, rectangular columns to resist moment (axial load) using simplified calculation method
A_{st}	Cross-sectional area of transverse steel (at laps)
A_{sw}	Cross-sectional area of shear reinforcement
A_{sw}	Area of punching shear reinforcement in one perimeter around the column
$A_{sw,min}$	Minimum cross-sectional area of shear reinforcement
$A_{sw,min}$	Minimum area of punching shear reinforcement in one perimeter around the column
a	Distance, allowance at supports
a	Geometric data
Δa	Deviation for geometrical data
a	An exponent (in considering biaxial bending of columns)
a	Projection of the footing from the face of the column or wall
a_b	Half the centre-to-centre spacing of bars (perpendicular to the plane of the bend)
a_l	Distance by which the location where a bar is no longer required for bending moment is displaced to allow for the forces from the truss model for shear. ('Shift' distance for curtailment)
a_v	Distance between bearings or face of support and face of load
a_1, b_1	Dimensions of the control perimeter around an elongated support (punching shear)
b	Overall width of a cross-section, or flange width in a T- or L-beam
b_0	Width of the bottom flange of the section
b_e	Effective width of a flat slab (adjacent to perimeter column)
b_{eff}	Effective width of a flange
b_{eq} (h_{eq})	Equivalent width (height) of column = b (h) for rectangular sections
b_{min}	Minimum width of web on T-, I- or L- beams
b_t	Mean width of the tension zone. For a T-beam with the flange in compression, only the width of the web is taken into account

Symbol	Definition
b_w	Width of the web on T-, I- or L- beams. Minimum width between tension and compression chords
b_y, b_z	Dimensions of the control perimeter (punching shear)
Δc	Permitted deviation from c_{nom} (BS EN 13760)
$\Delta c_{,dev}$	Allowance made in design for deviation
c_{min}	Minimum cover (due to the requirements for bond, $c_{min,b}$ or durability $c_{min,dur}$)
c_{nom}	Nominal cover (minimum cover plus allowance for deviations)
c_y, c_x	Column dimensions in plan
c_1, c_2	Dimensions of a rectangular column. For edge columns, c_1 is measured perpendicular to the free edge (punching shear)
D	Diameter of a circular column; Diameter of mandrel; Diameter
D_{Ed}	Fatigue damage factor
d	Effective depth to tension steel
d_2	Effective depth to compression steel
d_c	Effective depth of concrete in compression
d_{eff}	Effective depth of the slab taken as the average of the effective depths in two orthogonal directions (punching shear)
d_g	Largest maximum aggregate size
dl	A short length of a perimeter (punching shear)
E	Effect of action; Elastic modulus
E_c, $E_{c(t)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at time, t, days
$E_{c,eff}$	Effective modulus of elasticity of concrete
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
E_d	Design value of the effect of actions
EI	Bending stiffness
E_p	Design value of elasticity of prestressing steel
E_s	Design value of modulus of elasticity of reinforcing steel
Exp.	Expression
EQU	Static equilibrium
e	Eccentricity
e_2	Deflection (used in assessing M_2 in slender columns)
e_i	Eccentricity due to imperfections
e_{par}	Eccentricity parallel to the slab edge resulting from a moment about an axis perpendicular to the slab edge (punching shear)
e_y, e_z	Eccentricity, M_{Ed}/V_{Ed} along y and z axes respectively (punching shear)
F	Action
F_{bt}	Tensile force in the bar at the start of the bend caused by ultimate loads
F_c (F_s)	Force in concrete (steel)
F_{cd}	Design value of the concrete compression force in the direction of the longitudinal member axis
F_{cr}	Absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$
F_d	Design value of an action
F_E	Tensile force in reinforcement to be anchored
F_{Ed}	Compressive force, design value of support reaction
F_k	Characteristic value of an action

Symbol	Definition
F_{rep}	Representative action (= ψF_k where ψ = factor to convert characteristic to representative action)
F_{Rs}	Resisting tensile force in steel
F_s	Tensile force in the bar
F_{td}	Design value of the tensile force in longitudinal reinforcement
ΔF_{td}	Additional tensile force in longitudinal reinforcement due to the truss shear model
F_{wd}	Design shear strength of weld, design value of the force in stirrups in corbels
F_{Wk}	Characteristic value of wind force (Annex A2, BS EN 1990)
f	Frequency
f_{bd}	Ultimate bond stress
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength
$f_{cd,fat}$	Design fatigue strength of concrete
$f_{cd,pl}$	Design compressive strength of plain concrete
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
$f_{ck}(t_0)$	Characteristic concrete compressive strength at time of loading
$f_{ck,\,cube}$	Characteristic compressive cube strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
$f_{ct,d}$	Design tensile strength of concrete ($\alpha_{ct} f_{ct,k}/\gamma_C$)
$f_{ct,eff}$	Mean tensile strength of concrete effective at the time cracks may be first expected to occur. $f_{ct,eff} = f_{ctm}$ at the appropriate age
$f_{ct,k}$	Characteristic axial tensile strength of concrete
f_{ctb}	Tensile strength prior to cracking in biaxial state of stress
f_{ctm}	Mean value of axial tensile strength of concrete
f_{ctx}	Appropriate tensile strength for evaluation of cracking bending moment
$f_{ctk,0.05}$	5% fractile value of axial tensile strength of concrete
$f_{ctk,0.95}$	95% fractile value of axial tensile strength of concrete
f_{cvd}	Concrete design strength in shear and compression (plain concrete)
f_p	Tensile strength of prestressing steel
$f_{p0.1}$	0.1% proof-stress of prestressing steel
$f_{p0.1k}$	Characteristic 0.1% proof-stress of prestressing steel
$f_{p0.2k}$	Characteristic 0.2% proof-stress of prestressing steel
f_{pk}	Characteristic tensile strength of prestressing steel
f_{sc}	Compressive stress in compression reinforcement at ULS
f_t	Tensile strength of reinforcement
$f_{t,k}$	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of longitudinal reinforcement, A_{sl}
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield strength of the shear reinforcement
$f_{ywd,ef}$	Effective design strength of punching shear reinforcement
f_{ywk}	Characteristic yield strength of shear reinforcement
G_k	Characteristic value of a permanent action
g_k	Characteristic value of a permanent action per unit length or area
H_i	Horizontal action applied at a level
h	Overall depth of a cross-section; Height

Symbol	Definition
h_0	Notional size of cross-section
h_f	Depth of footing; Thickness of flange
h_H	Vertical height of a drop or column head below soffit of a slab (punching shear)
h_s	Depth of slab
I	Second moment of area of concrete section
i	Radius of gyration
J	Creep function
K	M_{Ed}/bd^2f_{ck}. A measure of the relative compressive stress in a member in flexure
K	Factor to account for structural system (deflection)
K'	Value of K above which compression reinforcement is required
K_c	Factor for cracking and creep effects
K_r	Correction factor for curvature depending on axial load
K_s	Factor for reinforcement contribution
$K\varphi$	Factor for taking account of creep
k	Coefficient or factor
k	Unintentional angular displacement for internal tendons
k_c	Coefficient allowing for the nature of the stress distribution within the section immediately prior to cracking and for the change of the lever arm as a result of cracking (minimum areas)
k_t	Factor in crack width calculations which depends on the duration of loading
l	Clear height of column between end restraints
l	Height of the structure in metres
l (or L)	Length; Span
l_0	Effective length (of columns)
l_0	Distance between points of zero moment
l_0	Design lap length
l_{bd}	Design anchorage length
$l_{b,eq}$	Equivalent anchorage length
$l_{b,min}$	Minimum anchorage length
$l_{b,rqd}$	Basic anchorage length
l_{eff}	Effective span
l_H	Horizontal distance from column face to edge of a drop or column head below soffit of a slab (punching shear)
l_n	Clear distance between the faces of supports
l_s	Floor to ceiling height
l_x, l_y	Spans of a two-way slab in the x and y directions
M	Bending moment. Moment from first order analysis
M'	Moment resistance of a singly reinforced section (above which compression reinforcement is required)
$M_{0,Eqp}$	First order bending moment in quasi-permanent load combination (SLS)
M_{01}, M_{02}	First order end moments at ULS *including* allowances for imperfections
M_{0Ed}	Equivalent first order moment including the effect of imperfections (at about mid height)
M_2	Nominal second order moment in slender columns
M_{Ed}	Design value of the applied internal bending moment
M_{Edy}, M_{Edz}	Design moment in the respective direction
M_{freq}	Applied bending moment due to frequent combination
M_{Rdy}, M_{Rdz}	Moment resistance in the respective direction

Symbol	Definition
$M_{Rd,max}$	Maximum transverse moment resistance
M_{rep}	Cracking bending moment
m	Number of vertical members contributing to an effect
m	Mass; Slab components
N	Axial force
NA	National Annex
N_a, N_b	Longitudinal forces contributing to H_i
N_{Ed}	Design value of axial force (tension or compression) at ULS
NDP	Nationally Determined Parameter(s) as published in a country's National Annex
n	Axial stress at ULS
n	Ultimate action (load) per unit length (or area)
n	Plate components
n	Number of bars
n_b	Number of bars in the bundle
P	Prestressing force
P_0	Initial force at the active end of the tendon immediately after stressing
Q_c	Characteristic construction load
Q_{fat}	Characteristic fatigue load
Q_k	Characteristic value of a variable action
Q_{k1} (Q_{ki})	Characteristic value of a leading variable action (Characteristic value of an accompanying variable action)
$Q_{Sn,k}$	Characteristic value of snow load
q_k	Characteristic value of a variable action per unit length or area
q_{ud}	Maximum value of combination reached in non-linear analysis
R	Resistance
R/A'	Vertical bearing resistance per unit area (foundations)
R_d	Design value of the resistance to an action
RH	Relative humidity
r	Radius; Correcting factor for prestress
r_{cont}	The distance from the centroid of a column to the control section outside the column head
r_{inf}, r_{sup}	Allowance in serviceability and fatigue calculations for possible variations in prestress
r_m	Ratio of first order end moments in columns at ULS
S	Internal forces and moments; First moment of area
S, N, R	Cement types
SLS	Serviceability limit state(s) – corresponding to conditions beyond which specified service requirements are no longer met
s	Spacing of the stirrups; Spacing between cracks
s_r	Radial spacing of perimeters of shear reinforcement
$s_{r,max}$	Maximum final crack spacing
s_t	Tangential spacing shear reinforcement along perimeters of shear reinforcement
T	Torsional moment; Tensile force
T_{Ed}	Design value of the applied torsional moment
T_k	Characteristic value of thermal actions
T_{Rd}	Design torsional resistance moment
$T_{Rd,max}$	Maximum design torsional resistance moment resistance
t	Thickness; Time being considered; Breadth of support; Time after tensioning

Symbol	Definition
t_0	The age of concrete at the time of loading
$t_{0,T}$	Temperature adjusted age of concrete at loading in days
$t_{ef,i}$	Effective wall thickness (torsion)
t_{inf}	Thickness of the bottom flange of the section
ULS	Ultimate limit state(s) – associated with collapse or other forms of structural failure
u	Perimeter of concrete cross-section, having area A_c
u	Perimeter of that part which is exposed to drying
u	Circumference of outer edge of effective cross-section (torsion)
u	Component of the displacement of a point
u_0	Perimeter adjacent to columns (punching shear)
u_1	Basic control perimeter, (at $2d$ from face of load) (punching shear)
u_{1*}	Reduced control perimeter at perimeter columns (at $2d$ from face of load) (punching shear)
u_i	Length of the control perimeter under consideration (punching shear)
u_k	Perimeter of the area A_k (torsion)
u_{out}	Perimeter at which shear reinforcement is no longer required
V	Shear force
V_{Ed}	Design value of the applied shear force
$V_{Ed,red}$	Applied shear force reduced by the force due to soil pressure less self weight of base (punching shear, foundations)
$V_{Rd,c}$	Shear resistance of a member without shear reinforcement
$V_{Rd,max}$	Shear resistance of a member limited by the crushing of compression struts
$V_{Rd,s}$	Shear resistance of a member governed by the yielding of shear reinforcement
v	Transverse shear or component of the displacement of a point
v_{Ed}	Punching shear stress
v_{Ed}	Shear stress for sections *without* shear reinforcement (= $V_{Ed}/b_w d$)
$v_{Ed,z}$	Shear stress for sections *with* shear reinforcement (= $V_{Ed}/b_w z = V_{Ed}/b_w 0.9d$)
$v_{Rd,c}$	Design shear resistance of concrete without shear reinforcement expressed as a stress
$v_{Rd,cs}$	Design punching shear resistance of concrete *with* shear reinforcement expressed as a stress (punching shear)
$v_{Rd,max}$	Resistance of concrete struts expressed as a stress
W_1	Factor corresponding to a distribution of shear (punching shear)
w	Component of the displacement of a point
w_k	Crack width
w_{max}	Limiting calculated crack width
X	Advisory limit of percentage of coupled tendons at a section
X0, XA, XC, XD, XF, XS	Concrete exposure classes
x	Neutral axis depth
x	Distance of the section being considered from the centre line of the support
x, y, z	Co-ordinates; Planes under consideration
x_c	Depth of the compression zone
x_u	Depth of the neutral axis at the ultimate limit state after redistribution
z	Lever arm of internal forces
z_{cp}	Distance between centre of gravity of concrete section and tendons
z_s	Lever arm for prestress
α	Angle; Angle of shear links to the longitudinal axis; Ratio
α	Long term effects coefficient

Symbol	Definition
α	Ratio between principal stresses
α	Deformation parameter
α_1, α_2, α_3 α_4, α_5, α_6	Factors dealing with anchorage and laps of bars
α_1, α_2, α_3	Factors used in creep calculations
α_{cc} (α_{ct})	A coefficient taking into account long term effects of compressive (tensile) load and the way load is applied
α_{cw}	Coefficient taking account of the state of stress in the compression chord
α_e	Effective modular ratio
α_h	Reduction factor for θ
β	Angle; Ratio; Coefficient
β	Factor dealing with eccentricity (punching shear)
$\beta(f_{cm})$	Factor to allow for the effect of concrete strength on the notional creep coefficient
β_H	Coefficent depending on the relative humidity and the notional member size
$\beta(t_0)$	Factor to allow for the effect of concrete age at loading on the notional creep coefficient
$\beta(t,t_0)$	Coefficient to describe the development of creep with time after loading
γ	Partial factor
γ_A	Partial factor for accidental actions, A
γ_C	Partial factor for concrete
$\gamma_{C,fat}$	Partial factor for fatigue of concrete
γ_F	Partial factor for actions, F
$\gamma_{F,fat}$	Partial factor for fatigue actions
γ_f	Partial factor for actions without taking account of model uncertainties
γ_g	Partial factor for permanent actions without taking account of model uncertainties
γ_G	Partial factor for permanent actions, G
γ_M	Partial factor for a material property, taking account of uncertainties in the material property itself, in geometric deviation and in the design model used
γ_P	Partial factor for actions associated with prestressing, P
γ_Q	Partial factor for variable actions, Q
γ_S	Partial factor for reinforcing steel or prestressing steel
$\gamma_{S,fat}$	Partial factor for reinforcing or prestressing steel under fatigue loading
γ_{SH}	Partial factor for shrinkage
δ	Ratio of the redistributed moment to the elastic bending moment.
ε_c	Compressive strain in concrete
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{c2}	Compressive strain limit in concrete for concrete in pure axial compression or strain in concrete at reaching maximum strength assuming use of the parabolic-rectangular relationship
ε_{c3}	Compressive strain limit in concrete for concrete in pure axial compression or strain in concrete at reaching maximum strength assuming use of the bilinear stress-strain relationship
ε_{ca}	Autogenous shrinkage strain
ε_{cc}	Creep strain
ε_{cd}	Drying shrinkage strain
ε_{cm}	Mean strain in concrete between cracks
ε_{cs}	Total shrinkage strain
ε_{cu}	Ultimate compressive strain in the concrete
ε_{cu2}	Ultimate compressive strain limit in concrete which is not fully in pure axial compression assuming use of the parabolic-rectangular stress-strain relationship (numerically $\varepsilon_{cu2} = \varepsilon_{cu3}$)

Symbol	Definition
ε_{cu3}	Ultimate compressive strain limit in concrete which is not fully in pure axial compression assuming use of the bilinear stress-strain relationship
$\varepsilon_{p(0)}$	Initial strain in prestressing steel
$\Delta \varepsilon_p$	Change in strain in prestressing steel
ε_s	Strain in reinforcing steel
ε_{sm}	Mean strain in reinforcement
ε_u	Strain of reinforcement or prestressing steel at maximum load
ε_{ud}	Design limit for strain for reinforcing steel in tension = $0.9\ \varepsilon_{uk}$
ε_{uk}	Characteristic strain of reinforcement (or prestressing steel) at maximum load
ε_y	Reinforcement yield strain
η	Factor defining effective strength (= 1 for ≤C50/60)
η_1	Coefficient for bond conditions
η_2	Coefficient for bar diameter
η_{p1}	Coefficient that takes into account the type of tendon and the bond situation at release
θ	Angle; Angle of compression struts (shear)
θ_{fat}	Inclination of compressive struts
θ_i	Inclination used to represent imperfections
λ	Slenderness ratio
λ	Damage equivalent factors in fatigue
λ	Factor defining the height of the compression zone (= 0.8 for ≤C50/60)
λ_{lim}	Limiting slenderness ratio (of columns)
μ	Coefficient of friction between the tendons and their ducts
μ	Characteristic value of the tensile strength of prestressing steel
ν	Poisson's ratio
ν_1	Strength reduction factor for concrete cracked in shear
ξ	Creep redistribution function
ξ	Bond strength ratio
ξ_1	Adjusted area of bond strength
ρ	Required tension reinforcement ratio. Assume A_s/bd
ρ	Oven dry density of concrete in kg/m^3
ρ'	Reinforcement ratio for required compression reinforcement, A_{s2}/bd
ρ_0	Reference reinforcement ratio $f_{ck}^{0.5} \times 10^{-3}$
ρ_1	Percentage of reinforcement lapped within $0.65l_0$ from the centre line of the lap being considered
ρ_{1000}	Value of relaxation loss (in %) at 1000 hours after tensioning and at a mean temperature of 20°C
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete
σ_{cp}	Compressive stress in the concrete from axial load or prestressing
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ε_{cu}
σ_{gd}	Design value of the ground pressure
σ_{pi}	The absolute value of initial prestress
σ_{pm0}	The absolute value of initial prestress during post-tensioning
$\sigma_{Rd,max}$	Design strength of concrete strut
σ_s	Stress in reinforcement at SLS
σ_s	Absolute value of the maximum stress permitted in the reinforcement immediately after the formation of the crack

Symbol	Definition
σ_{sc} (σ_{st})	Stress in compression (and tension) reinforcement
σ_{sd}	Design stress in the bar at the ultimate limit state
σ_{sr}	Stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking
τ	Torsional shear stress
Φ	Dynamic factor according to BS EN 1991-2
φ_0	Notional creep coefficient
$\varphi(\infty,t_0)$	Final value of creep coefficient
φ_{ef}	Effective creep factor
φ_{fat}	Damage equivalent impact factor in fatigue
$\varphi_{nl}(\infty,t_0)$	Non-linear notional creep coefficient
φ_p	Equivalent diameter of tendon
$\varphi(t,t_0)$	Creep coefficient, defining creep between times t and t_0, related to elastic deformation at 28 days
φ_{RH}	Factor to allow for the effect of relative humidity on the notional creep coefficient
ϕ	Bar diameter; Diameter of prestressing duct
ϕ_{eq}	Equivalent bar diameter
ϕ_m	Mandrel diameter
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
χ	Ageing coefficient
ψ	Factors defining representative values of variable actions
ψ_0	Combination value of a variable action (e.g. used when considering ULS)
ψ_1	Frequent value of a variable action (e.g. used when considering whether section will have cracked or not)
ψ_2	Quasi-permanent value of a variable action (e.g. used when considering deformation)
ω	Mechanical reinforcement ratio = $A_s f_{yd}/A_c f_{cd} \leq 1$

1 Introduction

BS EN 1992-1-1 (Eurocode 2: *Design of concrete structures* Part 1-1[1]) sets out general rules for the design of concrete structures and rules for the design of buildings. BS EN 1992-2 (Eurocode 2 - Part 2 Concrete bridges - Design and detailing rules)[2] provides additional or amended guidance to Part 1-1 for bridge structures.

The aim of this *Concise Eurocode 2 for Bridges* is to distil from all relevant parts of BS EN 1992 and the UK National Annexes material that will be commonly used in the design of normal bridge structures. Each country can publish non-contradictory, complementary information, and for concrete bridge design, PD 6687, Part 2[3] gives useful guidance. Material from this document is also included where appropriate, and presented on a pale yellow background to distinguish it from the main text.

As far as possible, the Eurocode clauses are repeated verbatim. One of the objectives is to embed the UK National Annex values into the document for ease of use. Due to the way Nationally Determined Parameters (NDPs) are introduced in the Eurocodes it has been necessary in some places to modify the text for clarity of reading. It has not been the intention to modify the meaning of the text and clearly if there is any doubt as to the meaning then the original Eurocode version should be adopted.

Further, some of the original text has been modified to reduce its length, while keeping the same meaning. In this case the text has been given a grey background to draw the reader's attention to the fact that the text is not strictly from the Code. Likewise other text, derived formulae, tables and illustrations that are provided to assist the designers but that are not taken directly from the original have been given a grey background. As before, it is intended to convey the original meaning and where any doubt exists the meaning of Eurocode 2 should be adopted.

The NDPs are recognition that each Member State of the EU is responsible for determining matters such as safety and current practice and allow individual countries to set their own values. As noted above, the UK values have been adopted throughout, but have been highlighted with a green background so that it is clear to the reader what NDP value has been used.

Guide to presentation

Grey shaded text, tables and figures	Modified Eurocode 2 text and additional text, derived formulae, tables and illustrations **not** from Eurocode 2
Yellow shaded text, tables and figures	Additional text from PD 6687[6] or PD 6687-2[3]
BS EN 1992-1-1 6.4.4	Relevant code and clauses or figure numbers
BS EN 1992-1-1 NA	From the relevant UK National Annex
BS EN 1992-1-1 6.4.4 & NA	From both Eurocode 2-1-1 and UK National Annex
Section 5.2	Relevant parts of this publication
1.0	Nationally Determined Parameter. UK values have been used throughout

For ease of reference, this guide is repeated on the inside back cover.

1.1 Scope

This publication is intended to cover the design of a typical bridge; there are particular types of bridges such as suspension bridges and segmental bridges that should not be designed without reference to the Code itself. It should also be noted that not every method presented in the Code is given here. Generally, the simplest and more conservative methods have been included and therefore the reader may find there are benefits to be gained by using other methods and should consult the Code on these occasions.

This publication does not cover the method of designing concrete elements using membrane rules. Membrane elements may be used for the design of two-dimensional concrete elements subject to a combination of internal forces evaluated by means of a linear finite element analysis. The reader should refer to BS EN 1992-2 Annex LL in conjunction with Annex F and Cl. 6.109 for detailed guidance. For the design or verification of shell elements subject to bending alone (i.e. with zero membrane forces) the approaches given by Wood Armer[4] and Denton & Burgoyne[5] may generally be used.

2 Basis of design

2.1 General

BS EN 1992-1-1[1] and BS EN 1992-2[2] should be used in conjunction with BS EN 1990: *Basis of structural design*[7], which:

BS EN 1992-1-1 1.1.1(3)

BS EN 1990 1.1.1(1)

- Establishes principles and requirements for the safety, serviceability and durability of structures.
- Describes the basis for their design and verification.
- Gives guidelines for related aspects of structural reliability.

2.2 Basic requirements

2.2.1 General

A structure should be designed and executed (constructed) in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way:

BS EN 1990 2.1(1), (2) & (4)

- Sustain all actions and influences likely to occur during execution and use.
- Meet the specified serviceability requirements for a structure or a structural member.

It should be designed to have adequate structural resistance, serviceability and durability.

A structure should be designed and executed in such a way that it will not be damaged by events such as explosion, impact and the consequences of human errors, to an extent disproportionate to the original cause.

2.2.2 Avoidance of damage

Potential damage should be avoided or limited by appropriate choice of one or more of the following:

BS EN 1990 2.1(5)

- Avoiding, eliminating or reducing the hazards to which the structure can be subjected.
- Selecting a structural form which has low sensitivity to the hazards considered.
- Selecting a structural form and design that can survive adequately the accidental removal of an individual structural member or a limited part of the structure or the occurrence of localised damage.
- Avoiding as far as possible structural systems that can collapse without warning.
- Tying the structural members together.

2.2.3 Limit states principles

BS EN 1990 implies that the design should be verified using limit states principles.

BS EN 1990 3.1(1)

An indicative value of 120 years is given in the UK National Annex for the design working life of bridges. It can generally be assumed that the guidance given in BS 8500[8] for at least a 100-year 'intended working life' will be appropriate for an 'indicative design working life' of 120 years.

BS EN 1990 2.3 & NA

2.3 Limit state design

BS EN 1990
3.1 & 3.4

Limit states are states beyond which the structure no longer fulfils the relevant design criteria:

- Ultimate limit states (ULS) are associated with collapse or other forms of structural failure.
- Serviceability limit states (SLS) correspond to conditions beyond which specified service requirements are no longer met.

Limit states should be verified in all relevant design situations selected, taking into account the circumstances under which the structure is required to fulfil its function.

BS EN 1990
3.2

2.3.1 Design situations

Normally, in non-seismic zones, the following design situations should be considered:

- Persistent situations which refer to the conditions of normal use.
- Transient situations which refer to temporary conditions, such as during execution or repair.
- Accidental situations which refer to exceptional conditions applicable to the structure or to its exposure e.g. fire, explosion, impact or the consequences of localised failure.

2.3.2 Actions

BS EN 1990
1.5.3.1

Actions refer to a set of forces (loads) applied to the structure (direct action), or to a set of imposed deformations or accelerations caused, for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

BS EN 1990
1.5.3.3

BS EN 1990
1.5.3.4

BS EN 1990
1.5.3.5

- Permanent actions refer to actions for which the variation in magnitude with time is negligible.
- Variable actions are actions for which the variation in magnitude with time is not negligible.
- Accidental actions are actions of short duration but of significant magnitude that are unlikely to occur on a given structure during the design working life.

BS EN 1990
4.1.2(1)

The characteristic value, F_k, of an action is its main representative value and shall be specified by:

- A mean value – generally used for permanent actions.
- An upper value with an intended probability of not being exceeded or lower value with an intended probability of being achieved – normally used for variable actions with known statistical distributions, such as wind or snow.
- A nominal value – used for some variable and accidental actions.

The values of actions given in the various parts of BS EN 1991: *Actions on structures*[9] are taken as characteristic values.

2.3.3 Verification

BS EN 1990

BS EN 1991

Verification, using the partial factor method, is detailed in BS EN 1990[4]. In this method it is verified that, in all relevant design situations, no relevant limit state is exceeded when design values for actions and resistances are used in the design models.

2.3.4 Design values of actions

The design value, F_d, of an action, F, can be expressed in general terms as $F_d = \gamma_F \psi F_k$

BS EN 1990 6.3.1

where

γ_F = partial factor for the action which takes account of the possibility of unfavourable deviations of the action values from the representative values.

ψ = a factor for the action

ψ can have the value 1.0, ψ_0 or ψ_1 or ψ_2 which is used to obtain the characteristic, combination, frequent and quasi-permanent values respectively. It adjusts the value of the action to account for the joint probability of the actions occurring simultaneously. See Tables 2.1 to 2.3 which are derived from BS EN 1990 and its National Annex [7a].

F_k = characteristic value of an action.

Table 2.1

Values of ψ factors for road bridges

BS EN 1990 NA table NA. A.2.1

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads (see BS EN 1991-2, table 4.4)	gr1a[a]	TS	0.75	0.75	0
		UDL	0.75	0.75	0
		Pedestrian and cycle-track loads [b]	0.40	0.40	0
	gr1b[a]	Single axle	0	0.75	0
	gr2	Horizontal forces	0	0	0
	gr3	Pedestrian loads	0	0.40	0
	gr4	Crowd loading	0	0.75	0
	gr5[c]	Vertical forces from SV and SOV Vehicles	0	1.0	0
Wind forces	F_{Wk}	Persistent design situations	0.50	0.20	0
	F_{Wk}	During execution	0.80	–	0
	F^*_W	During execution	1.0	–	0
Thermal actions	T_k		0.60 [d]	0.60	0.50
Snow loads	$Q_{Sn,k}$ (during execution)		0.80	–	–
Construction loads	Q_c		1.0	–	1.0

Key

a The values of ψ_0 , ψ_1 and ψ_2 for gr1a and gr1b are given for road traffic corresponding to adjusting factors α_{Qi}, α_{qi}, α_{qr} and $\beta_Q = 1$

b The value of the pedestrian and cycle-track load, given in Table 4.4a of EN 1991-2, is a 'reduced' value accompanying the characteristic value of LM1 and should not be factored again by ψ_1. However, when gr1a is combined with leading non-traffic actions, this value should be factored by ψ_0

c In accordance with BS EN 1991-2 Cl. 4.5.2, the frequent value of gr5 does not need to be considered.

d The ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO

Table 2.2

Values of ψ factors for footbridges

BS EN 1990 NA table NA. A.2.2

Action	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads	gr1	0.4	0.4	0
	Q_{fwk}	0	0	0
	gr2	0	0	0
Wind forces	F_{Wk}	0.3	0.2	0
Thermal actions	T_k	0.6 [a]	0.6	0.5
Snow loads	$Q_{Sn,k}$ (during execution)	0.8	–	0
Construction loads	Q_c	1.0	–	1.0

Key

a The ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO

BS EN 1990
table A.2.3

Table 2.3
Values of ψ factors for railway bridges

Actions			ψ_0	ψ_1	ψ_2 [a]
Individual components of traffic actions [b]	LM71		0.80	[c]	0
	SW/0		0.80	[c]	0
	SW/2		0	1.00	0
	Unloaded train		1.00	–	–
	HSLM		1.00	1.00	0
	Tracking and braking Centrifugal forces Interaction forces due to deformation under vertical traffic loads		Individual components of traffic actions in design situations where the traffic loads are considered as a single (multi-directional) leading action and not as groups of loads should use the same values of ψ factors as those adopted for the associated vertical loads		
	Nosing forces		1.00	0.80	0
	Non-public footpaths loads		0.80	0.50	0
	Real trains		1.00	1.00	0
	Horizontal earth pressure due to traffic load surcharge		0.80	[c]	0
	Aerodynamic effects		0.80	0.50	0
Main traffic actions (groups of loads)	gr11 (LM71+SW/0)	Max. vertical 1 with max. longitudinal	0.80	0.80	0
	gr12 (LM71+SW/0)	Max. vertical 2 with max. transverse			
	gr13 (Braking/traction)	Max. longitudinal			
	gr14 (Centrifugal/nosing)	Max. lateral			
	gr15 (Unloaded train)	Lateral stability with 'unloaded train'			
	gr16 (SW/2)	SW/2 with max. longitudinal			
	gr17 (SW/2)	SW/2 with max. transverse			
	gr21 (LM71+SW/0)	Max. vertical 1 with max. longitudinal	0.80	0.70	0
	gr22 (LM71+SW/0)	Max. vertical 2 with max. transverse			
	gr23 (Braking/traction)	Max. longitudinal			
	gr24 (Centrifugal/nosing)	Max. lateral			
	gr26 (SW/2)	SW/2 with max. longitudinal			
	gr27 (SW/2)	SW/2 with max. transverse			
	gr31 (LM71+SW/0)	Additional load cases	0.80	0.60	0
Other operating actions	Aerodynamic effects		0.80	0.50	0
	General maintenance loading for non-public footpaths		0.80	0.50	0
Wind forces [d]	F_{Wk}		0.75	0.50	0
	F^{**}_{W}		1.00	0	0
Thermal actions [e]	T_K		0.60	0.60	0.50
Snow loads	$Q_{Sn,k}$ (during execution)		0.80	–	0
Construction loads	Q_c		1.00	–	1.00

Key

a If deformation is being considered for persistent and transient design situations, ψ_2 should be taken equal to 1.00 for rail traffic actions. For seismic design situations, see Table NA A2.5 of BS EN 1990 NA

b Minimum coexistant favourable vertical load with individual components of rail traffic actions (e.g. centrifugal, traction or braking) is 0.5LM71, etc.

c 0.8 if 1 track only is loaded
0.7 if 2 tracks are simultaneously loaded
0.6 if 3 or more tracks are simultaneously loaded

d When wind forces act simultaneously with traffic actions, the wind force $\psi_0 F_{Wk}$ should be taken as no greater than F^{**}_{W} (see BS EN 1991-1-4). See A2.2.4(4) of BS EN 1990

e See BS EN 1991-1-5

2.3.5 Combinations of actions

Ultimate limit states

The following ultimate limit states shall be verified as relevant:

EQU Loss of static equilibrium of the structure or any part of it considered as a rigid body.
STR Internal failure or excessive deformation of the structure or structural members.
GEO Failure or excessive deformation of the structure where the strengths of soil or rock are significant in providing resistance.
FAT Fatigue failure of the structure or structural members.

BS EN 1990 6.4.1

The partial factors and combinations of actions for these limit states are given in Tables 2.4 to 2.7.

BS EN 1990 tables NA.A.2.4(A), (B) & (C)

Table 2.4
Recommended partial factors

Action	EQU (Set A)		STR/GEO (Set B)		STR/GEO (Set C)	
Permanent actions	$\gamma_{G,sup}$	$\gamma_{G,inf}$	$\gamma_{G,sup}$	$\gamma_{G,inf}$	$\gamma_{G,sup}$	$\gamma_{G,inf}$
Concrete self-weight	1.05	0.95	1.35	0.95	1.00	1.00
Steel self-weight	1.05	0.95	1.20	0.95	1.00	1.00
Superimposed dead [a]	1.05	0.95	1.20	0.95	1.00	1.00
Road surfacing [a]	1.05	0.95	1.20	0.95	1.00	1.00
Weight of soil	1.05	0.95	1.35	0.95	1.00	1.00
Self-weight of other materials listed in BS EN 1991-1-1, tables A.1-A.6	1.05	0.95	1.35	0.95	1.00	1.00
Creep and shrinkage	–	–	1.20	0	1.00	0
Settlement (linear structural analysis)	–	–	1.20	0	1.00	0
Settlement (non-linear structural analysis)	–	–	1.35	0	1.00	0
Variable actions (γ_Q)	**Unfavourable**	**Favourable**	**Unfavourable**	**Favourable**	**Unfavourable**	**Favourable**
Road traffic actions (gr1a, gr1b, gr2, gr5, gr6)	1.35	0	1.35	0	1.15	0
Pedestrian actions (gr3, gr4)	1.35	0	1.35	0	1.15	0
Rail traffic actions (LM71, SW/0, HSLM)	1.45	0	1.45	0	1.25	0
Rail traffic actions (SW/2 and other load models representing controlled exceptional traffic)	1.40	0	1.40	0	1.20	0
Rail traffic actions (real trains)	1.70	0	1.70	0	1.45	0
Wind actions	1.70	0	1.70	0	1.45	0
Thermal actions	1.55	0	1.55	0	1.30	0

Key
a See Table NA.1 of BS EN 1991-1-1[9] for guidance on thicknesses for ballast, waterproofing, surfaces and other coatings.
Note
For design values for self-weight of water, ground water pressure and earth pressures refer to BS EN 1997-1.

BS EN 1990 tables NA.A.2.4(A), (B) & (C)

Table 2.5
Combinations of actions for EQU, STR and GEO limit states

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action	Accompanying variable actions
	Unfavourable	Favourable			
Exp. (6.10)	$\gamma_{G,sup}\ G_{kj,sup}$	$\gamma_{G,inf}\ G_{kj,inf}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$	$\gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$

Note
For partial factors see Table 2.4 (except see Table 2.9 for γ_P)

BS EN 1990 table A2.5

Table 2.6
Combinations for accidental situations

Accidental design situation	Permanent actions		Prestress	Accidental action	Accompanying variable actions	
	Unfavourable	Favourable			Main	Others
Exp. (6.11a/b)	$G_{k,j,sup}$	$G_{k,j,inf}$	P	A_d	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$

BS EN 1992-1-1 6.8.3

Table 2.7
Combinations of fatigue actions

Action	Permanent actions		Prestress	Leading variable action	Accompanying variable actions	Fatigue action
	Favourable	Unfavourable				
Non-cyclic	$G_{k,j,inf}$	$G_{k,j,sup}$	P	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$	–
Cyclic	$G_{k,j,inf}$	$G_{k,j,sup}$	P	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$	Q_{fat}

Serviceability limit states

The combinations of actions for the serviceability limit state are given in Table 2.8.

BS EN 1990 table A2.6

Table 2.8
Combinations of actions for the serviceability limit state

Combination	Permanent actions		Prestress	Variable actions	
	Favourable	Unfavourable		Leading	Others
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$Q_{k,1}$	$\psi_{0,i}\ Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{2,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$

2.3.6 Actions to consider

BS EN 1992-1-1 2.3.1.2, 2.3.1.3 & 2.3.2.2

Thermal effects, differential settlements/movements, creep and shrinkage should be taken into account when checking serviceability limit states.

Thermal effects, differential settlements/movements, creep and shrinkage should be considered for ultimate limit states only where they are significant (e.g. fatigue conditions, in the verification of stability where second order effects are of importance, etc.). In other cases they need not be considered, provided that the ductility and rotation capacity of the elements are sufficient.

BS EN 1992-1-1 2.3.1.2(3)

Where thermal effects are taken into account they should be considered as variable actions and applied with a partial factor and ψ factor, which can be determined from BS EN 1990 Annex A2.

Differential settlements/movements of the structure due to soil subsidence should be classified as a permanent action, G_{set} which is introduced as such in combinations of actions. A partial safety factor for settlement effects should be applied.

BS EN 1992-1-1 2.3.1.3(1) & (4)

When creep is taken into account its design effects should be evaluated under the quasi-permanent combination of actions irrespective of the design situation considered i.e. persistent, transient or accidental. In most cases the effects of creep may be evaluated under permanent loads and the mean value of prestress.

BS EN 1992-1-1 2.3.2.2(3)

The design values for vehicle impacts on supporting structures and substructures are given in BS EN 1991-1-1[9] section 4.3. The deisgn values for vehicle impacts on parapets are given in BS EN 1991-2[9] section 4.8.

Appropriate values for partial actions are given in Table 2.9

Table 2.9
Values for partial factors applied to actions

Action	Ultimate limit state					Servicability limit state	
	STR/GEO		EQU		FAT	Favourable	Unfavourable
	Favourable	Unfavourable	Favourable	Unfavourable			
Shrinkage	$\gamma_{SH} = 0$	$\gamma_{SH} = 1.0$	–	–	–	–	–
Prestress effects	$\gamma_{P,fav} = 0.9$	$\gamma_{P,unfav} = 1.1$	$\gamma_{P,fav} = 0.9$	$\gamma_{P,unfav} = 1.2$	–	$r_{inf} = 1.0$	$r_{sup} = 1.0$
Fatigue	–	–	–	–	$\gamma_{F,fat} = 1.0$	–	–

2.3.7 Material properties

Material properties are specified in terms of their characteristic values, which in general correspond to a defined fractile of an assumed statistical distribution of the property considered (usually the lower 5% fractile).

BS EN 1992-1-1 2.4.2.4(1) & NA

The values of γ_C and γ_S, partial factors for materials, are indicated in Table 2.10.

BS EN 1992-1-1 table 2.1 N & NA

Table 2.10
Partial factors for materials

Design situation	γ_C – concrete	γ_S – reinforcing steel	γ_S – prestressing steel
ULS – Persistent and transient	1.50	1.15	1.15
Accidental	1.20	1.00	1.00
Fatigue	1.50	1.15	1.15
SLS	1.00	1.00	1.00

2.4 Assumptions

BS EN 1992-1-1 1.3

In addition to the assumptions in BS EN 1990, it is assumed that:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided.
- Construction is carried out by personnel having the appropriate skill and experience.
- Materials and products will be used as specified in Eurocode 2 or in the relevant material or product specifications.
- The structure will be adequately maintained and will be used in accordance with the design brief.
- The requirements for execution and workmanship given in ENV 13670[10] are complied with.

At the time of writing BS EN 13670[11] is expected to be published in late 2009, and will replace ENV 13670. Once published it is anticipated that standard specifications will be updated to refer to it. In the interim existing specifications should be adapted.

2.5 Foundation design

BS EN 1997
2.4.6.4

The design of concrete foundations is subject to Eurocode 7[12] for the geotechnical aspects and to Eurocode 2 for the structural concrete design. Further guidance on the geotechnical design can be found in PD 6694-1[13].

BS EN 1997
2.1(4)

Eurocode 7 is wide ranging and provides all the requirements for geotechnical design. It states that no limit state e.g. equilibrium, stability, strength or serviceability, as defined by BS EN 1990, shall be exceeded. The requirements for ULS and SLS design may be accomplished by using, in an appropriate manner, the following alone or in combination:

- Calculations.
- Prescriptive measures.
- Testing.
- Observational methods.

The foundation design and the derivation of design resistance are covered by the Geotechnical Design Report. For simple structures, this report can be combined with the ground investigation report but it is still a distinct requirement. Both the ULS and SLS conditions must be met but the definition of the SLS criteria is not possible without the liaison with the bridge designer and a full Eurocode 7 compatible design cannot be carried out by a party in isolation from the rest of the structure design team.

3 Materials

3.1 Concrete

3.1.1 Strength and other properties

BS EN 1992-1-1 3.1.2(1)

The compressive strength is denoted by concrete strength classes which relate to the characteristic (5%) cylinder strength f_{ck}, or the cube strength $f_{ck,cube}$, in accordance with BS EN 206-1 *Concrete: Specification, performance, production and conformity*[14].

BS EN 1992-1-1 4.4.1.2(5) & NA

In the UK, BS 8500[8] complements BS EN 206-1 and the guidance given in the former should be followed.

BS EN 1992-1-1 table 3.1

Concrete strength classes and properties are shown in Table 3.1. In the notation used for compressive strength class, 'C' refers to normal weight concrete, the first number refers to the cylinder strength f_{ck} and the second to cube strength $f_{ck,cube}$.

BS EN 1992-2 3.1.2 (102) & NA

BS EN 1992-1-1 3.1.2 (2) & NA

The strength classes (C) in BS EN 1992-2 are denoted by the characteristic cylinder strength f_{ck} determined at 28 days with a minimum value of C25/30 and a maximum value of C70/85. The shear strength of concrete classes higher than C50/60 should be determined by tests or limited to that of C50/60.

BS EN 1992-1-1 table 3.1

Table 3.1
Concrete strength classes and properties

Property	Strength class (MPa)									
	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85	C28/35[a]	C32/40[a]
f_{ck}	25	30	35	40	45	50	55	60	70	28	32
$f_{ck, cube}$	30	37	45	50	55	60	67	75	85	35	40
f_{cm}	33	38	43	48	53	58	63	68	78	36	40
f_{ctm}	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	2.8	3.0
$f_{ctk,0.05}$	1.8	2.0	2.2	2.5	2.7	2.9	3.0	3.1	3.2	1.9	2.1
$f_{ctk,0.95}$	3.3	3.8	4.2	4.6	4.9	5.3	5.5	5.7	6.0	3.6	3.9
$E_{cm,}$(GPa)	31	33	34	35	36	37	38	39	41	32	33

Key
a Derived data

BS EN 1992-1-1 3.1.6(1) & NA

The value of the design compressive strength of concrete, f_{cd}, is defined as:

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C$

where

f_{ck} = characteristic compressive cylinder strength of concrete at 28 days
γ_C = partial factor for concrete (See Table 2.9)
α_{cc} = a coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied. In the UK α_{cc} is either 1.0 or 0.85 depending on the situation. The correct values are used in the appropriate clauses below. Generally it is 1.0 for shear and 0.85 in other circumstances

The value of concrete design tensile strength f_{ctd} is defined as:

$f_{ctd} = 1.0\ f_{ctk,0.05}/\gamma_C$

where

$f_{ctk, 0.05}$ = 5% fractile value of axial tensile strength of concrete

BS EN 1992-1-1 3.1.3(4)

Poisson's ratio may be taken equal to 0.2 for uncracked concrete and 0 for cracked concrete.

BS EN 1992-1-1 3.1.3(5)

Unless more accurate information is available, the linear coefficient of thermal expansion may be taken equal to 10 x 10^{-6} K^{-1}.

3.1.2 Creep

BS EN 1992-1-1 3.1.4(2)

The creep coefficient, $\varphi(t,t_0)$ is related to E_c, the tangent modulus, which may be taken as $1.05\ E_{cm}$.

BS EN 1992-1-1 B.1(1)

The creep coefficient $\varphi(t,t_0)$ may be obtained from Figure 3.1 of BS EN 1992-1-1, or calculated from:

$\varphi(t,t_0) = \varphi_0\ \beta_c(t,t_0)$

where

φ_0 = notional creep coefficient and may be estimated from:

$= \varphi_{RH}\ \beta(f_{cm})\ \beta(t_0)$

where

φ_{RH} = factor to allow for the effect of relative humidity on the notional creep coefficient:

$= 1 + \dfrac{1 - RH/100}{0.1\ h_0^{1/3}}$ for $f_{cm} \leq 35$ MPa

$= \left[1 + \dfrac{\alpha_1(1 - RH/100)}{0.1\ h_0^{1/3}}\right] \alpha_2$ for $f_{cm} > 35$ MPa

RH = relative humidity of the ambient environment in %

$\beta(f_{cm})$ = factor to allow for the effect of concrete strength on the notional creep coefficient

$= 16.8 / f_{cm}^{0.5}$

f_{cm} = mean compressive strength of concrete in MPa at the age of 28 days

$\beta(t_0)$ = factor to allow for the effect of concrete age at loading on the notional creep coefficient

$= 1/(0.1 + t_0^{\ 0.20})$

h_0 = notional size of the member in mm

$= 2\ A_c/u$

A_c = cross-sectional area

u = perimeter of the member in contact with the atmosphere

$\beta_c(t, t_0)$ = coefficient to describe the development of creep with time after loading

$= [(t - t_0)/(\beta_H + t - t_0)]^{0.3}$

where

t = age of concrete in days at the moment considered

t_0 = age of concrete at loading in days

$t - t_0$ = non-adjusted duration of loading in days

β_H = coefficient depending on the relative humidity (RH in %) and the notional member size (h_0 in mm)

$= 1.5\ [1 + (0.012\ RH)^{18}]\ h_0 + 250 \leq 1500$ for $f_{cm} \leq 35$ MPa

$= 1.5\ [1 + (0.012\ RH)^{18}]\ h_0 + 250\ \alpha_3 \leq 1500\ \alpha_3$ for $f_{cm} > 35$ MPa

$\alpha_1 = (35/f_{cm})^{0.7}$

$\alpha_2 = (35/f_{cm})^{0.2}$

$\alpha_3 = (35/f_{cm})^{0.5}$

BS EN 1992-1-1 B.1(2)

The effect of type of cement on the creep coefficient of concrete may be taken into account by modifying the age of loading t_0 according to the following Expression:

$t_0 = t_{0,T}\ (9 / (2 + t_{0,T}^{\ 1.2}) + 1)^{\alpha} \geq 0.5$

where

$t_{0,T}$ = temperature adjusted age of concrete at loading in days (see below)

α = power which depends on type of cement

= -1 for cement Class S (cement Class CEM 32.5N)

= 0 for cement Class N (cement Classes 32.5R & CEM 42.5N)

= 1 for cement Class R (cement Classes CEM 42.5R, CEM 52.5N & CEM 52.5R)

Cement classes can be specified using BS EN 197-1[15]. Where the cement Class is not known, generally Class R may be assumed. Where the ground granulated blastfurnace slag (ggbs) content exceeds 35% or the fly ash content exceeds 20% of the cement combination, Class N may be assumed. Where ggbs exceeds 65% or pfa exceeds 35%, Class S maybe assumed.

The effect of elevated or reduced temperatures within the range 0–80°C on the maturity of concrete may be taken into account by adjusting the concrete age according to the following Expression:

BS EN 1992-1-1 B.1(3)

$$t_T = \sum_{i=1}^{n} e^{-(4000/[273 + T(\Delta t_i)]-13.65)} \Delta t$$

where

t_T = temperature-adjusted concrete age which replaces t in the corresponding equations
$T(\Delta t_i)$ = temperature in °C during the time period Δt_i
Δt_i = number of days where a temperature T prevails

The mean coefficient of variation of the above predicted creep data, deduced from a computerised data bank of laboratory test results, is of the order of 20%.

The creep deformation of concrete $\varepsilon_{cc}(\infty,t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0, is given by:

BS EN 1992-1-1 3.1.4(3)

$$\varepsilon_{cc}(\infty,t_0) = \varphi(\infty,t_0)\ (\sigma_c / E_c)$$

When the compressive stress of concrete at an age t_0 exceeds the value 0.45 $f_{ck}(t_0)$ then creep non-linearity should be considered. Such a high stress can occur as a result of pretensioning, e.g. in precast concrete members at tendon level. In such cases the non-linear notional creep coefficient should be obtained as follows:

BS EN 1992-1-1 3.1.4(4)

$$\varphi_{nl}(\infty, t_0) = \varphi(\infty, t_0) \exp[1.5\ (k_\sigma - 0.45)]$$

where

$\varphi_{nl}(\infty, t_0)$ = non-linear notional creep coefficient, which replaces $\varphi(\infty, t_0)$
k_σ = stress–strength ratio $\sigma_c/f_{ck}(t_0)$
σ_c = compressive stress
$f_{ck}(t_0)$ = characteristic concrete compressive strength at the time of loading

BS EN 1992-1-1 3.1.4(6)

3.1.3 Shrinkage

The total shrinkage strain is composed of two components, the drying shrinkage strain and the autogenous shrinkage strain. The drying shrinkage strain develops slowly, since it is a function of the migration of the water through the hardened concrete. The autogenous shrinkage strain develops during hardening of the concrete: the major part therefore develops in the early days after casting. Autogenous shrinkage is a linear function of the concrete strength. It should be considered specifically when new concrete is cast against hardened concrete.

Hence the values of the total shrinkage strain follow from:

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

where

ε_{cs} = total shrinkage strain
ε_{cd} = drying shrinkage strain (see Table 3.2)
ε_{ca} = autogenous shrinkage strain (see Table 3.2)

3.2 Steel reinforcement

The properties of steel reinforcement to BS 4449: 2005[16] are shown in Table 3.3. This British Standard complements BS EN 10080[17] and Annex C of BS EN 1992-1-1.

BS EN 1992-1-1 3.2

Annex C allows for a strength range between 400 and 600 MPa. BS 4449: 2005 adopts 500 MPa.

Table 3.2
Long-term (70-year) shrinkage strains

Concrete strength at 28 days (f_{ck})	Strain due to drying shrinkage (x 10^3)	Strain due to autogenous shrinkage (x 10^3)	Total shrinkage strains (x 10^3)
20	0.517	0.025	0.542
25	0.489	0.038	0.527
30	0.463	0.050	0.513
35	0.438	0.063	0.501
40	0.415	0.075	0.490
45	0.393	0.088	0.481
50	0.372	0.100	0.472
60	0.333	0.125	0.458
70	0.298	0.150	0.448

Notes
1 The values shown assume Class R cement (Class N and Class S will have lower values).
2 The drying shrinkage values assume a notional member size, h_0, of 150 mm. For h_0 of 300 mm multiply values by 0.81 and for h_0 of 500 mm or greater, multiply values by 0.75.

Table 3.3
Properties of reinforcement

BS EN 1992-1-1 table C1

Property	Class		
	A	B	C
Characteristic yield strength f_{yk} or $f_{0.2k}$ (MPa)	500	500	500
Minimum value of $k = (f_t/f_y)_k$	≥ 1.05	≥ 1.08	≥ 1.15 < 1.35
Characteristic strain at maximum force ε_{uk} (%)	≥ 2.5	≥ 5.0	≥ 7.5

Note
Table derived from BS EN 1992-1-1 Annex C, BS 4449: 2005 and BS EN 10080. The nomenclature used in BS 4449: 2005 differs from that used in Annex C and used here.

BS EN 1992-2 3.2.4 & NA

Class B or Class C reinforcement should be used. For steel fabric reinforcement, Class A may also be used provided it is not taken into account in the evaluation of the ultimate resistance.

3.3 Prestressing steel

Typical prestressing strand properties are shown in Table 3.4. The manufacturer's technical data sheets should be referred for detailed information.

BS EN 1992-1-1 3.3.2(1)

Properties of prestressing steels should be in accordance with EN 10138. However, until this standard is published BS 5896[18] may be used.

BS EN 1992-1-1 3.3.3(1)

The 0.1% proof stress ($f_{p0.1k}$) and the specified value of tensile strength (f_{pk}) are used to define the characteristic values of the prestressing steels.

The design values for the prestressing steel are derived by dividing the characteristic values by the partial safety factor for prestressing steel γ_S. For ultimate limit state verification, γ_S = 1.15 for persistent and transient design situations and 1.0 for accidental design situations.

BS EN 1992-1-1 3.3.6(2) & (3)

The modulus of elasticity E_p can be assumed equal to 205 GPa for wires and bars and 195 GPa for strand.

Table 3.4
Dimensions and properties of low relaxation strand and wire for prestressed concrete

Type of strand	Nominal diameter (mm)	Nominal tensile strength (MPa)	Steel area (mm^2)	Nominal mass (g/m)	Characteristic breaking load (kN)	Characteristic 0.1% proof load (kN)	Characteristic load at 1% elongation (kN)
7 wire standard	15.2	1860	139	1090	259	220	228
	15.2	1670	139	1090	232	197	204
	12.5	1860	93	730	173	147	152
	12.5	1770	93	730	164	139	144
	11.0	1770	71	557	125	106	110
	9.3	1860	52	408	97	82	85
	9.3	1770	52	408	92	78	81
7 wire super	15.7	1860	150	1180	279	237	246
	15.7	1770	150	1180	265	225	233
	12.9	1860	100	785	186	158	163
	11.3	1860	75	590	139	118	122
	9.6	1860	55	432	102	87	90
	8.0	1860	38	298	70	59	61
7 wire drawn	18.0	1770	223	1750	380	323	334
	15.2	1820	165	1295	300	255	264
	12.7	1860	112	890	209	178	184

3.3.1 Relaxation of prestressing steel

BS EN 1992-1-1 defines three classes of relaxation.

BS EN 1992-1-1 3.3.2(4)

Class 1: ordinary wire and strand
Class 2: low relaxation wire and strand
Class 3: hot rolled and processed bars

BS EN 1992-1-1 3.3.2(5)

The design calculation of the losses due to relaxation of the prestressing steel should be based on the loss at 1000 hr (ρ_{1000}) after tensioning at a mean temperature of 20°C and initial prestress of 70% (0.7 f_{pk}). The values of ρ_{1000} can be taken from the certificate or assumed to be:

BS EN 1992-1-1 3.3.2(6)

Class 1 – 8%
Class 2 – 2.5%
Class 3 – 4%

BS EN 1992-1-1 3.3.2(7)

The relaxation loss may be obtained from manufacturer's test certificates or they can be calculated, based upon the following equations (see also Table 3.5):

Class 1 $$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 5.39\,\rho_{1000}\,e^{6.7\mu}\left(\frac{t}{1000}\right)^{0.75\,(1-\mu)}10^{-5}$$

Class 2 $$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 0.66\,\rho_{1000}\,e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75\,(1-\mu)}10^{-5}$$

Class 3 $$\frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 1.98\,\rho_{1000}\,e^{8\mu}\left(\frac{t}{1000}\right)^{0.75\,(1-\mu)}10^{-5}$$

where

$\Delta\sigma_{pr}$ = absolute value of the relaxation losses of the prestress

σ_{pi} = absolute value of the initial prestress

= σ_{pm0} for **post-tensioning** (see Section 11.3.2)

= maximum tensile stress applied to the tendon minus the immediate losses occurring during the stressing process for **pre-tensioning** (see Section 11.3.3)

t = time after tensioning (in hours)

μ = σ_{pi} / f_{pk}, where f_{pk} is the characteristic value of the tensile strength of the prestressing steel

ρ_{1000} = value of relaxation loss (%), at 1000 hours after tensioning and at a mean temperature of 20°C

BS EN 1992-1-1 3.3.2(8)

The long-term (final) values of the relaxation losses may be estimated from t = 500,000 hours.

BS EN 1992-1-1 3.3.2(9)

Relaxation losses are very sensitive to temperature of the steel where heat treatment is applied (e.g. by steam) (see BS EN 1992-1-1 Cl. 10.3.2.1). Otherwise where the temperature is greater than 50°C, the relaxation losses should be verified.

Table 3.5
Relaxation losses based on using Expressions (3.28 to 3.30) in BS EN 1992-1-1

Class	ρ_{1000} **(%)**	$\mu = \sigma_{pi} / f_{pk}$	***t* (hours)**	$\Delta\sigma_{pr} / \sigma_{pi}$ **%**
1	8	0.80	500,000	23.3
		0.76		21.5
		0.72		19.8
		0.68		18.2
		0.64		16.8
		0.60		15.5
2	2.5	0.80		6.1
		0.76		5.1
		0.72		4.3
		0.68		3.6
		0.66		3.0
		0.60		2.5
3	4	0.80		12.1
		0.76		10.6
		0.72		9.3
		0.68		8.1
		0.64		7.1
		0.60		6.2

Relaxation losses are sensitive to variations in stress levels over time and can therefore be reduced by taking into consideration of other time-dependent losses occurring within the structure at the same time (such as creep and shrinkage). Previous UK practice is to base design on the relaxation loss of 1000 hours without considering the interaction with creep and shrinkage.

4 Durability and cover

4.1 General

A durable structure shall meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance.

BS EN 1992-1-1 4.1

In order to achieve the required design working life of the structure, adequate measures shall be taken to protect each structural element against the relevant environmental actions. Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to mechanical actions.

BS EN 1992-1-1 4.3(1), 4.2(1)

Requirements of durability should be considered at all stages of design and construction, including the selection of materials, construction details, execution and quality control.

BS EN 1992-1-1 4.3(2)

Half-joints should not be used in bridges unless there are adequate provisions for inspection and maintenance. Water penetration or the possibility of leakage from the carriageway into the inside of voided structures should be considered in the design. For concrete surfaces protected by waterproofing the exposure class is XC3 .

BS EN 1992-2 4.2(104), (105) & NA, PD 6687-2 5.1

Where de-icing salt is used, all exposed concrete surfaces within 10 m of the carriageway horizontally or within 5 m above the carriageway should be considered as being directly affected by de-icing salts. Top surfaces of supports under expansion joints should also be considered as being directly affected by de-icing salts. The exposure classes for surfaces directly affected by de-icing slats are XD3 and XF2 or XF4 as appropriate.

BS EN 1992-2 4.2(106) & NA

Adequate cover is required to ensure safe transmission of bond forces (see Section 4.2) and protection of steel against corrosion (see Sections 4.3 and 4.4).

BS EN 1992-1-1 4.4.1.2(1)

The concrete cover to reinforcement is the distance from the outer surface of the reinforcement to the nearest concrete surface. Drawings should specify the nominal cover. As illustrated in Figure 4.1, the nominal cover should satisfy the minimum requirements in respect of bond and durability, and allow for the deviation to be expected in execution (see Section 4.5).

BS EN 1992-1-1 4.4.1.3(3)

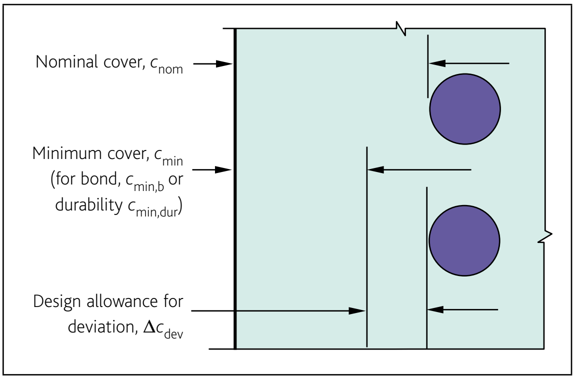


Figure 4.1 **Determination of cover**

4.2 Cover for bond, $c_{min,b}$

BS EN 1992-1-1 4.4.1.2(3) & NA

In order to transmit bond forces safely and to ensure adequate compaction, the minimum cover should not be less than $c_{min,\,b}$ in Table 4.1.

BS EN 1992-1-1 table 4.2

Table 4.1
Minimum cover, $c_{min,b}$, requirements for bond

Reinforcement type and arrangement	$c_{min,b}$
Individual bars	Diameter of bar, ϕ
Bundled bars	Equivalent diameter of bars, ϕ_n
Post-tensioned circular ducts	Diameter of duct or 80 mm whichever is smaller
Post-tensioned rectangular ducts	Smaller dimension or half greater dimension whichever is greater, but not more than 80 mm
Pre-tensioned strand or wire	1.5 times the diameter
Pre-tensioned indented wire	2.5 times the diameter

4.3 Cover for durability, $c_{min,dur}$

BS EN 1992-1-1 4.4.1.2(5) & NA

Environmental conditions are classified according to Table 4.2, which is based on BS 8500. Concrete composition and minimum covers required for durability in different environmental conditions are set out in Table 4.3, derived from BS 8500[8]. These tables give recommendations for normal weight concrete using maximum aggregate size of 20 mm for selected exposure classes and cover to reinforcement.

In accordance with BS 8500, special attention should be given to the concrete composition and aggregates, when considering freeze/thaw attack, chemical attack or abrasion resistance.

BS 8500 table A.1

Table 4.2
Exposure Classes

Class	Class description	Informative example applicable to the United Kingdom
No risk of corrosion or attack (X0 class)		
X0	For concrete without reinforcement or embedded metal all exposures except where there is freeze/thaw, abrasion or chemical attack.	Unreinforced concrete surfaces inside structures. Unreinforced concrete completely buried in soil classed as AC–1 and with hydraulic gradient not greater than 5. Unreinforced concrete permanently submerged in non-aggressive water. Unreinforced concrete in cyclic wet and dry conditions not subject to abrasion, freezing or chemical attack. **Note:** For reinforced concrete, use at least XC1.
Corrosion induced by carbonation (XC classes)[a] *(Where concrete containing reinforcement or other embedded metal is exposed to air and moisture)*		
XC1	Dry or permanently wet.	Reinforced and prestressed concrete surfaces inside enclosed structures except areas of structures with high humidity. Reinforced and prestressed concrete surfaces permanently submerged in non-aggressive water.
XC2	Wet, rarely dry.	Reinforced and prestressed concrete completely buried in soil classed as AC–1 and with a hydraulic gradient not greater than 5.
XC3 & **XC4**	Moderate humidity or cyclic wet and dry.	External reinforced and prestressed concrete surfaces sheltered from, or exposed to, direct rain. Reinforced and prestressed concrete surfaces inside structures with high humidity (e.g. poorly ventilated, bathrooms, kitchens). Reinforced and prestressed concrete surfaces exposed to alternate wetting and drying. Interior concrete surfaces of pedestrian subways not subject to de-icing salts, voided superstructures or cellular abutments. Reinforced or prestressed concrete beneath waterproofing.

Table 4.2
Exposure Classes (continued)

Class	Class description	Informative example applicable to the United Kingdom
Corrosion induced by chlorides other than from sea water (XD classes)[a] ***(Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water)***		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides. Reinforced and prestressed concrete wall and structure supports more than 10 m horizontally from a carriageway. Bridge deck soffits more than 5 m vertically above the carriageway. Parts of structures exposed to occasional or slight chloride conditions.
XD2	Wet, rarely dry.	Reinforced and prestressed concrete surfaces totally immersed in water containing chlorides[b]. Buried highway structures more than 1 m below adjacent carriageway.
XD3	Cyclic wet and dry.	Reinforced and prestressed concrete surfaces directly affected by de-icing salts or spray containing de-icing salts (e.g. walls; abutments and columns within 10 m of the carriageway; parapet edge beams and buried structures less than 1 m below carriageway level, pavements and car park slabs).
Corrosion induced by chlorides from sea water (XS classes)[a] ***(Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water)***		
XS1	Exposed to airborne salt but not in direct contact with sea water.	External reinforced and prestressed concrete surfaces in coastal areas.
XS2	Permanently submerged.	Reinforced and prestressed concrete completely submerged and remaining saturated, e.g. concrete below mid-tide level[b].
XS3	Tidal, splash and spray zones.	Reinforced and prestressed concrete surfaces in the upper tidal zones and the splash and spray zones[c].
Freeze/thaw attack (XF classes) ***(Where concrete is exposed to significant attack from freeze/thaw cycles whilst wet)***		
XF1	Moderate water saturation without de-icing agent.	Vertical concrete surfaces such as facades and columns exposed to rain and freezing. Non-vertical concrete surfaces not highly saturated, but exposed to freezing and to rain or water.
XF2	Moderate water saturation with de-icing agent.	Concrete surfaces such as parts of bridges, which would otherwise be classified as XF1 but which are exposed to de-icing salts either directly or as spray or run-off.
XF3	High water saturation without de-icing agent.	Horizontal concrete surfaces, such as parts of buildings, where water accumulates and which are exposed to freezing. Concrete surfaces subjected to frequent splashing with water and exposed to freezing.
XF4	High water saturation with de-icing agent or sea water[d].	Horizontal concrete surfaces, such as roads and pavements, exposed to freezing and to de-icing salts either directly or as spray or run-off. Concrete surfaces subjected to frequent splashing with water containing de-icing agents and exposed to freezing.
Chemical attack (ACEC classes) ***(Where concrete is exposed to chemical attack. Note: BS 8500-1 refers to ACEC classes rather than XA classes used in BS EN 206-1)***		
Refer to Section 4.4		

Key

a The moisture condition relates to that in the concrete cover to reinforcement or other embedded metal but, in many cases, conditions in the concrete cover can be taken as being that of the surrounding environment. This might not be the case if there is a barrier between the concrete and its environment.

b Reinforced and prestressed concrete elements, where one surface is immersed in water containing chlorides and another is exposed to air, are potentially a more severe condition, especially where the dry side is at a high ambient temperature. Specialist advice should be sought where necessary, to develop a specification that is appropriate to the actual conditions likely to be encountered.

c Exposure XS3 covers a range of conditions. The most extreme conditions are in the spray zone. The least extreme is in the tidal zone where conditions can be similar to those in XS2. The recommendations given take into account the most extreme UK conditions within this class.

d It is not normally necessary to classify in the XF4 exposure class those parts of structures located in the United Kingdom which are in frequent contact with the sea.

Table 4.3

Selected[a] recommendations for normal-weight reinforced concrete quality for combined exposure classes and cover to reinforcement

Exposure conditions			Cement/ combination designations[b]	Minimum strength class[c], maximum w/c ratio, minimum cement or combination						
				At least 50-year working life						
				Nominal cover to reinforcement[d]						
Typical example	Primary	Secondary		15 + Δc_{dev}	20 + Δc_{dev}	25 + Δc_{dev}	30 + Δc_{dev}	35 + Δc_{dev}	40 + Δc_{dev}	45 + Δc_{dev}
Internal elements or permanently wet elements	XC1	–	All	C20/25, 0.70, 240 or RC20/25	<<<	<<<	<<<	<<<	<<<	<<<
Buried concrete in AC–1 ground conditions[e]	XC2	AC–1	All	–	–	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<	<<<
Vertical surface protected from direct rainfall	XC3/4	–	All except IVB-V	–	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	C25/30, 0.65, 260 or RC25/30	<<<	<<<
Vertical surface exposed to rain and freezing		XF1	All except IVB-V	–	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	<<<	<<<	<<<
Exposed horizontal surfaces		XF3	All except IVB-V	–	C40/50, 0.45, 340[g] or RC40/50XF[g]	<<<	<<<	<<<	<<<	<<<
		XF3 (air entrained)	All except IVB-V	–	–	C30/37, 0.55, 300[g,h]	C28/35, 0.60, 280[g,h] or PAV2	C25/30, 0.60, 280[g,h,j] or PAV1	<<<	<<<
Elements subject to airborne chlorides	XD1[f]	XF1	All	–	–	C40/50, 0.45, 360	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<	<<<
Exposed vertical surfaces near coast	XS1[f]	XF1	CEM I, IIA, IIB-S, SRPC	–	–	–	See BS 8500	C35/45, 0.45, 360	C32/40, 0.50, 340	<<<
			IIB-V, IIIA	–	–	–	See BS 8500	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320
		–	IIIB	–	–	–	C32/40, 0.40, 380	C25/30, 0.50, 340	C25/30, 0.50, 340	C25/30, 0.55, 320
Exposed horiz. surfaces near coast		XF3 or XF4	CEM I, IIA, IIB-S, SRPC	–	–	–	See BS 8500	C40/50, 0.45, 360[g]	<<<	<<<
Elements submerged in water containing chlorides	XD2 or XS2[f]	–	CEM I, IIA, IIB-S, SRPC	–	–	–	C40/50, 0.40, 380	C32/40, 0.50, 340	C28/35, 0.55, 320	<<<
			IIB-V, IIIA	–	–	–	C35/45, 0.40, 380	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<
			IIIB, IVB-V	–	–	–	C32/40, 0.40, 380	C25/30, 0.50, 340	C20/25, 0.55, 320	<<<
Elements subject to moderate water saturation with de-icing agent and freezing	XD3[f]	XF2	IIB-V, IIIA	–	–	–	–	–	C35/45, 0.40, 380	C32/40, 0.45, 360
			CEM I, IIA, IIB-S, SRPC	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380
			IIIB, IVB-V	–	–	–	–	–	C32/40, 0.40, 380	C32/40 0.45, 360
Elements subject to water saturation with de-icing agent and freezing		XF4[k]	IIB-V, IIIA	–	–	–	–	–	C40/50, 0.45, 380[g]	C40/50, 0.45, 360[g]
		XF4 (air entrained)	CEM I, IIA, IIB-S, SRPC	–	–	–	–	–	See BS 8500	C28/35, 0.40, 380[g]
			IIB-V, IIIA	–	–	–	–	–	C28/35, 0.40,380[g,h]	C28/35 0.45, 360[g,h]
Elements in tidal, splash and spray zones	XS3	–	CEM I, IIA, IIB-S, SRPC	–	–	–	–	–	–	See BS 8500
			IIB-V, IIIA	–	–	–	–	–	C35/45, 0.40, 380	C32/40, 0.45, 360
			IIIB, IVB-V	–	–	–	–	–	C32/40, 0.40, 380	C28/35, 0.45, 360

Key

a This table comprises a selection of common exposure class combinations. Requirements for other sets of exposure classes, e.g. XD2, XS2 and XS3 should be derived from BS 8500–1: 2006, Annex A.

b See Table 4.4 (CEM I is Portland cement, IIA to IVB are cement/combinations.)

c For prestressed concrete the minimum strength class should be C28/35.

d Δc_{dev} is an allowance for deviations.

for either at least a 50-year or 100-year intended working life and 20 mm maximum aggregate size

content (kg/m^3), and equivalent designated concrete where applicable											
	At least 100-year working life										
	Nominal cover to reinforcement[d]										
$50 +\Delta c_{dev}$	$15 +\Delta c_{dev}$	$25 +\Delta c_{dev}$	$30 +\Delta c_{dev}$	$35 +\Delta c_{dev}$	$40 +\Delta c_{dev}$	$45 +\Delta c_{dev}$	$50 +\Delta c_{dev}$	$55 +\Delta c_{dev}$	$60 +\Delta c_{dev}$	$65 +\Delta c_{dev}$
<<<	C20/25, 0.70, 240 RC20/25	<<<	<<<	<<<	<<<	<<<	<<<	<<<	<<<	<<<
<<<	–	C25/30, 0.65, 260 RC25/30	<<<	<<<	<<<	<<<	<<<	<<<	<<<	<<<
<<<	–	–	C40/50, 0.45, 340 RC40/50	C35/45, 0.50, 320 RC35/45	C30/37, 0.55, 300 RC30/37	C28/35, 0.60, 280 RC28/35	C25/30, 0.65, 260 RC25/30	<<<	<<<	<<<
<<<	–	–	C40/50, 0.45, 340 RC40/50	C35/45, 0.50, 320 RC35/45	C30/37, 0.55, 300 RC30/37	C28/35, 0.60, 280 RC28/35	<<<	<<<	<<<	<<<
<<<	–	–	C40/50, 0.45, 340[g] RC40/50	<<<	<<<	<<<	<<<	<<<	<<<	<<<
<<<	–	–	–	C35/45, 0.50, 320[g,h]	C30/37, 0.55, 300[g,h]	C28/35, 0.60, 280[g,h] or PAV2	C25/30 0.60, 280[g,h,j] or PAV1	<<<	<<<	<<<
<<<	–	–	C45/55, 0.40, 380	C40/50, 0.45, 360	C35/45, 0.50, 340	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<	<<<	<<<
<<<	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360	<<<	<<<
<<<	–	–	–	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<	<<<	<<<
<<<	–	–	–	C35/45, 0.45, 360	C30/37, 0.50, 340	C28/35, 0.55, 320	C25/30, 0.55, 320	<<<	<<<	<<<
<<<	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380[g]	<<<	<<<	<<<
<<<	–	–	–	–	C35/45, 0.45, 360	C32/40, 0.50, 340	C28/35, 0.55, 320	<<<	<<<	<<<
<<<	–	–	–	–	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<	<<<	<<<
<<<	–	–	–	–	C28/35, 0.45, 360	C25/30, 0.50, 340	C20/25, 0.55, 320	<<<	<<<	<<<
C32/40, 0.50, 340	–	–	–	–	–	See BS 8500	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55 320
C35/45, 0.45, 360	–	–	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
C32/40, 0.50, 340	–	–	–	–	–	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340	<<<	<<<
C40/50, 0.45, 340	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380[g]	C40/50, 0.45, 360[g]	C40/50, 0.45, 340[g]	C40/50, 0.45, 340[g]
C28/35, 0.45, 360[g]	–	–	–	–	–	–	–	See BS 8500	C28/35, 0.40, 380[g]	C28/35, 0.45, 360[g]
C28/35, 0.50, 340[g,h]	–	–	–	–	–	See BS 8500	C28/35, 0.40, 380[g,h]	C28/35, 0.45, 360[g,h]	C28/35, 0.50, 340[g,h]	C28/35, 0.55, 320[g,h]
C40/50, 0.40, 380	–	–	–	–	–	–	–	–	See BS 8500	C40/50, 0.40, 380
C28/35, 0.50, 340	–	–	–	–	–	See BS 8500	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320
C25/30, 0.50, 340	–	–	–	–	–	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340	<<<	<<<

e For sections less than 140 mm thick refer to BS 8500.

f Also adequate for exposure class XC3/4.

g Freeze/thaw resisting aggregates should be specified.

h Air entrained concrete is required.

j This option may not be suitable for areas subject to severe abrasion.

k Not recommended for pavements and hardstandings – see BS 8500-1, A.4.3.

– Not recommended

<<< Indicates that concrete quality in cell to the left should not be reduced

BS 8500 table A.6

Table 4.4
Cement and combination type[a]

Broad designation[b]	Composition	Cement/combination types (BS 8500)
CEM I	Portland cement	CEM I
SRPC	Sulfate-resisting Portland cement	SRPC
IIA	Portland cement with 6–20% fly ash, ground granulated blastfurnace slag, limestone, or 6–10% silica fume[c]	CEM II/A-L, CEM II/A-LL, CIIA-L, CIIA-LL, CEM II/A-S, CIIA-S, CEM II/A-V, CIIA-V, CEM II/A–D
IIB-S	Portland cement with 21–35% ground granulated blastfurnace slag	CEM II/B-S, CIIB-S
IIB-V	Portland cement with 21–35% fly ash	CEM II/B-V, CIIB-V
IIB+SR	Portland cement with 25–35% fly ash	CEM II/B-V+SR, CIIB-V+SR
IIIA[d, e]	Portland cement with 36–65% ground granulated blastfurnace slag	CEM III/A, CIIIA
IIIA+SR[e]	Portland cement with 36–65% ground granulated blastfurnace slag with additional requirements that enhance sulfate resistance	CEM III/A+SR[f], CIII/A+SR[f], CIIIA+SR
IIIB[e, g]	Portland cement with 66–80% ground granulated blastfurnace slag	CEM III/B, CIIIB
IIIB+SR[e]	Portland cement with 66–80% ground granulated blastfurnace slag with additional requirements that enhance sulfate resistance	CEM III/B+SR[f], CIIIB+SR[f]
IVB-V	Portland cement with 36–55% fly ash	CEM IV/B(V), CIVB

Key

a There are a number of cements and combinations not listed in this table that may be specified for certain specialist applications. See BRE Special Digest[19] for the sulfate-resisting characteristics of other cements and combinations.

b The use of these broad designations is sufficient for most applications. Where a more limited range of cement or combinations types is required, select from the notations given in BS 8500–2: 2006, Table 1.

c When IIA or IIA–D is specified, CEM I and silica fume may be combined in the concrete mixer using the k-value concept; see BS EN 206–1:2000, Cl. 5.2.5.2.3.

d Where IIIA is specified, IIIA+SR may be used.

e Inclusive of low early strength option (see BS EN 197–4 and the 'L' classes in BS 8500–2: 2006, Table A.1).

f '+SR' indicates additional restrictions related to sulfate resistance. See BS 8500–2: 2006, Table 1, footnote D.

g Where IIIB is specified, IIIB+SR may be used.

4.4 Chemical attack

Where plain or reinforced concrete is in contact with the ground, further checks are required to achieve durability. An aggressive chemical environment for concrete class (ACEC class) should be assessed for the site. *BRE Special Digest 1*[19] gives guidance on the assessment of the ACEC class and this is normally carried out as part of the interpretive reporting for a ground investigation. Knowing the ACEC class, a design chemical class (DC class) can be obtained from Table 4.5. In general, fully buried concrete in the UK need not be designed to be freeze-thaw resisting.

For designated concretes, an appropriate foundation concrete (FND designation) can be selected using Table 4.6. An FND concrete has the strength class of C25/30. Where a higher strength is required, either for its strength or where the foundation is classified as XD2 or XD3, a designed concrete should be specified. For designed concretes, the concrete producer should be advised of the DC–class.

4.5 Δc_{dev} and other allowances

To calculate the nominal cover, c_{nom}, an addition to the minimum cover shall be made in design to allow for the deviation (Δc_{dev}). In the UK the recommended value is 10 mm. In certain situations the accepted deviation and hence allowance Δc_{dev} may be reduced:

BS EN 1992-1-1 4.4.1.3(1)&(3) & NA

- Where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation Δc_{dev} may be reduced: 10 mm ≥ Δc_{dev} ≥ 5 mm
- Where it can be assured that a very accurate measurement device is used for monitoring, and non-conforming members are rejected (e.g. precast elements), the allowance in design for deviation Δc_{dev} may be reduced: 10 mm ≥ Δc_{dev} ≥ 0 mm

Δc_{dev} is recognised in BS 8500 as Δc.

The minimum cover for concrete cast on prepared ground (including blinding) is 40 mm and that for concrete cast directly against soil is 65 mm.

BS EN 1992-1-1 4.4.1.3(4) & NA

For uneven surfaces (e.g. exposed aggregate) the minimum cover should be increased by at least 5 mm.

BS EN 1992-1-1 4.4.1.2(11)

Table 4.5
Selection of the DC–class and the number of additional protection measures (APMs) where the hydrostatic head of groundwater is not more than five times the section width [a, b, c, d, e]

BS 8500 table A.9

ACEC-class (aggressive chemical environment for concrete class)	Intended working life	
	At least 50 years	At least 100 years
AC–1s, AC–1	DC–1	DC–1
AC–2s, AC–Z	DC–2	DC–2
AC–2z	DC–2z	DC–2z
AC–3s	DC–3	DC–3
AC–3z	DC–3z	DC–3z
AC–3	DC–3	Refer to BS 8500
AC–4s	DC–4	DC–4
AC–4z	DC–4z	DC–4z
AC–4	DC–4	Refer to BS 8500
AC–4ms	DC–4m	DC4m
AC–4m	DC–4m	Refer to BS 8500
AC–5	DC–4[f]	DC–4[f]
AC–5z	DC–4z[f]	DC–4z/1[f]
AC–5m	DC–4m[f]	DC–4m[f]

Key
a Where the hydrostatic head of groundwater is greater than five times the section width, refer to BS 8500.
b For guidance on precast products see Special Digest 1[19].
c For structural performance outside these values refer to BS 8500.
d For section widths <140 mm refer to BS 8500.
e Where any surface attack is not acceptable e.g. with friction piles, refer to BS 8500.
f This should include APM3 (surface protection), where practicable, as one of the APMs; refer to BS 8500.

BS 8500
table A.1

Table 4.6
Guidance on selecting designated concrete for reinforced concrete foundations

DC-Class	Appropriate designated concrete
DC–1	RC 25/30
DC–2	FND2
DC–2z	FND2z
DC–3	FND3
DC–3z	FND3z
DC–4	FND4
DC–4z	FND4z
DC–4m	FND4m

Note
Strength class for all FND concrete is C25/30.

BS EN 1992-2
4.4.1.2(114)

Bare concrete decks of road bridges, without waterproofing or surfacing, should be classified as Abrasion Class XM2.

BS EN 1992-2
4.4.1.2(115)

Where a concrete surface is subject to abrasion caused by ice or solid transportation in running water, the cover should be increased by a minimum of 10 mm.

5 Structural analysis

5.1 General

The purpose of structural analysis is to establish the distribution of either internal forces and moments or stresses, strains and displacements over the whole or part of a structure. Additional local analysis shall be carried out where necessary.

BS EN 1992-1-1 5.1.1(1)

When designing a bridge deck slab of box beam or beam and slab construction, it is necessary to consider, in addition to overall global effects, the local effects induced in the top slab by wheel loads. Current practice is to use elastic analysis and often assume full fixity at the slab and web junctions and use either Pucher's influence surfaces, Westergaard's equations, or FE analysis.

5.2 Idealisation of the structure

5.2.1 Definitions

For bridge structures the following can be applied:

BS EN 1992-1-1 5.3.1

- A beam is a member for which the span is not less than three times its depth. If not, it is a deep beam.
- A slab is a member for which the minimum panel dimension is not less than five times the overall thickness.
- A one-way spanning slab has either two approximately parallel unsupported edges or, when supported on four edges, the ratio of the longer to shorter span exceeds 2.0.
- A column is a member for which the section depth does not exceed four times its width and the height is at least three times the section depth. If not, it is a wall.

5.2.2 Effective flange width

The effective width of a flange, b_{eff}, should be based on the distance, l_0, between points of zero moments as shown in Figure 5.1 and defined in Figure 5.2.

BS EN 1992-1-1 5.3.2.1(2)&(3)

$b_{eff} = b_w + b_{eff,1} + b_{eff,2}$

where

BS EN 1992-1-1 fig. 5.2

$b_{eff,1}$ = $(0.2b_1 + 0.1l_0)$ but $\le 0.2l_0$ and $\le b_1$
$b_{eff,2}$ = to be calculated in a similar manner to $b_{eff,1}$ but b_2 should be substituted for b_1 in the above

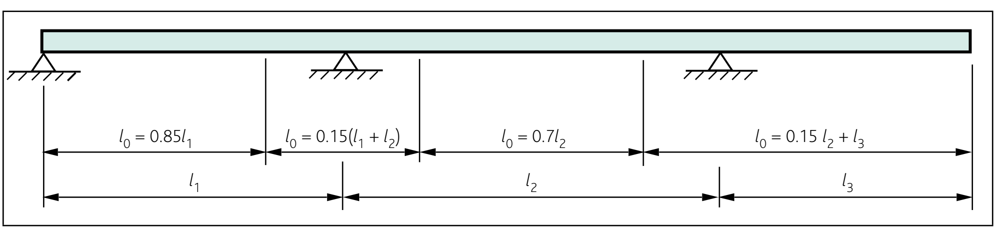


Figure 5.1
Elevation showing definition of l0 for calculation of flange width

5.2.3 Effective span

BS EN 1992-1-1 5.3.2.2(1)

The effective span, l_{eff}, of a member should be calculated as follows:

$l_{eff} = l_n + a_1 + a_2$

where

l_n = clear distance between the faces of the supports; values for a_1 and a_2, at each end of the span, may be determined from the appropriate a_i values in Figure 5.3 where t is the width of the supporting element as shown.

BS EN 1992-1-1 fig. 5.3

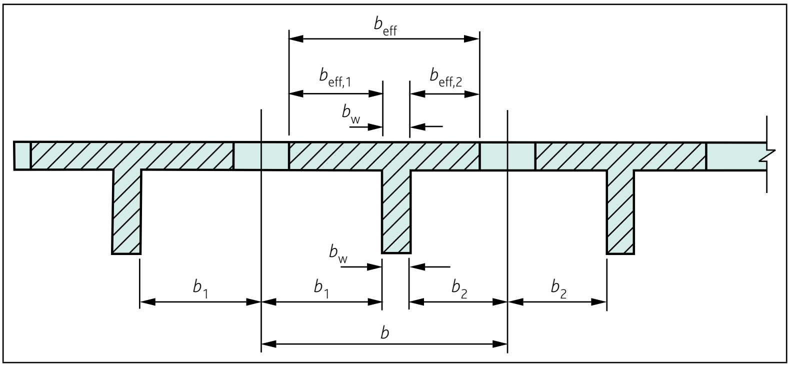


Figure 5.2
Section showing effective flange width parameters

BS EN 1992-1-1 fig. 5.4

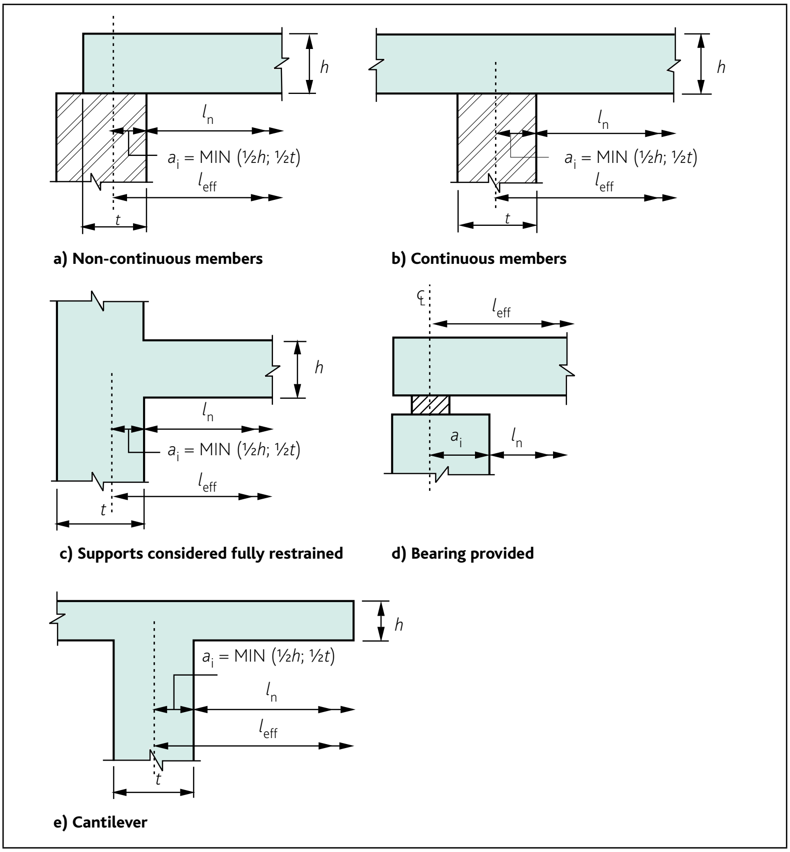


Figure 5.3
Effective span, l_{eff}, for different support conditions

5.3 Methods of analysis

5.3.1 Ultimate limit states (ULS)

The type of analysis should be appropriate to the problem being considered. The following are commonly used: linear elastic analysis, linear elastic analysis with limited redistribution, and plastic analysis.

BS EN 1992-1-1 5.1.1(6)

For determination of the action effects, linear elastic analysis may be carried out assuming:

BS EN 1992-1-1 5.4(2)

- Uncracked cross-sections.
- Linear stress–strain relationships.
- The use of mean values of elastic modulus.

If a linear elastic analysis with limited redistribution is undertaken, shears and reactions used in the design should be taken as those either prior to redistribution or after redistribution, whichever is greater.

PD 6687-2 6.4

In continuous beams or slabs which:

BS EN 1992-2 5.5(104) & NA

- Are predominantly subject to flexure
- Have the ratio of lengths of adjacent spans in the range of 0.5 to 2.0

redistribution of bending moment may be carried out without explicit check on the rotation capacity provided that:

$\delta \geq 0.44 + 1.25(0.6 + 0.0014/\varepsilon_{cu2})\ x_u/d \quad$ for $f_{ck} \leq 50$ MPa
$\delta \geq 0.54 + 1.25(0.6 + 0.0014/\varepsilon_{cu2})\ x_u/d \quad$ for $f_{ck} > 50$ MPa
$\delta \geq 0.85$ where Class B and Class C reinforcement is used

where

δ = the ratio of the redistributed moment to the moment in the linear elastic analysis
x_u = the depth of the neutral axis at the ultimate limit state after redistribution
d = the effective depth of the section

Redistribution should not be carried out in circumstances where the rotation capacity cannot be defined with confidence (e.g. in curved and or skewed bridges).

BS EN 1992-2 5.5 (105)

It is recommended that moment redistribution is not used for sections deeper than 1.2 m unless a rigorous analysis of rotation capacity is undertaken.

PD 6687-2 6.6

The following rules may be used for solid concrete slabs with $f_{ck} \leq 50$ MPa.

$\delta \geq 0.4 + 1.0\ x_u/d \geq 0.7$ where the reinforcement is Class B or Class C

BS EN 1992-2 5.5(104)

No redistribution is allowed for Class A reinforcement.

Where used, plastic analysis should be based either on static (lower bound) or kinematic (upper bound) methods. The ductility of the critical sections should be sufficient for the envisaged mechanism to be formed. The required ductility may be deemed to be satisfied if all of the following are fulfilled:

BS EN 1992-1-1 5.6.1(3) & (2)

BS EN 1992-2 5.6.2(102)

- $x_u/d \leq 0.15$ for $f_{ck} \leq 50$ MPa
- $x_u/d \leq 0.10$ for $f_{ck} \geq 50$ MPa
- Reinforcement is either Class B or C; and
- Ratio of the moments at internal supports to the moments in the span is between 0.5 and 2.0.

For solid slabs $x_u/d \leq 0.25$ may be used when $f_{ck} \leq 50$ MPa.

BS EN 1992-2
5.7 (105) & NA

Non-linear analysis should be undertaken using model factors and material models, which give results that err on the safe side. Typically, this may be achieved by using design material properties and applying design actions. However, in some situations, underestimating stiffness through the use of design properties can lead to unsafe results. Such situations can include cases where indirect actions such as imposed deformations are significant, cases where the failure load is associated with a local brittle failure mode, and cases where the effect of tension stiffening is unfavourable. In such situations, sensitivity analyses should be undertaken to investigate the effect of variations in material properties, including spatial variations, to provide confidence that the results of the analysis do err on the safe side.

For non-linear analysis that considers only direct and flexural effects, reference may be made to BS EN 1992-2, Cl. 5.8.6. Such analysis should account for the effects of long-term loading. Effects not considered directly in the analysis should be considered separately in accordance with Sections 6 to 11.

Non-linear analysis that determines shear and torsional strength directly has not yet reached a stage where it can be fully codified. Particular analyses may be used when they have been shown by comparison with tests to give reliable results, with the agreement of the National Authority.

5.3.2 Serviceability limit states (SLS)

BS EN 1992-1-1
5.4

Linear elastic analysis may be carried out assuming:

- Uncracked cross-sections.
- Linear stress–strain relationships.
- The use of mean values of elastic modulus.

The moments derived from elastic analysis should not be redistributed but a gradual evolution of cracking should be considered, which requires a non-linear analysis allowing for the effect of cracking and tension stiffening. Un-cracked global analysis in accordance with BS EN 1992-1-1 Cl. 5.4 (2) may always be used as a conservative alternative to such considerations.

5.3.3 General note

Regardless of the method of analysis used, the following apply.

BS EN 1992-1-1
5.3.2.2(3)

BS EN 1992-1-1
5.3.2.2(104)

- Where a beam or slab is monolithic with its supports, the critical design moment at the support may be taken as that at the face of the support. The design moment and reaction transferred to the supporting element (e.g. column, wall, etc.) should be generally taken as the greater of the elastic or redistributed values. The moment at the face of the support should not be taken as less than 65% of the full fixed end moment.
- Where a beam or slab is continuous over a support which is considered to provide no restraint to rotation (e.g. over walls) and the analysis assumes point support, the design support moment calculated on the basis of a span equal to the centre-to-centre distance between supports, may be reduced by an amount ΔM_{Ed} as follows:

  $\Delta M_{Ed} = F_{Ed,sup}\ t/8$

  where

  $F_{Ed,sup}$ = design support reaction

  t = breadth of bearing

5.4 Loading

5.4.1 Combinations of actions

BS EN 1990
6.4.3

The following combinations of actions should be used where appropriate.

Ultimate Limit State

Persistent or transient design situations:

$\gamma_G\ G_k + \gamma_p\ P + \gamma_{Q,1}\ Q_{k,1} + \Sigma\ \gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$ $\quad i > 1$

Accidental design situations:

$G_k + P + A_d + \psi_{1,1}\ Q_{k,1} + \Sigma\ \psi_{2,i}\ Q_{k,i}$ $\quad i > 1$

Note: $\psi_{2,1}$ may be substituted for $\psi_{1,1}$ depending on the accidental design situation.

Serviceability Limit State

Characteristic combination of actions:

$G_k + P + Q_{k,1} + \Sigma\ \psi_{0,i}\ Q_{k,i}$ $\quad i > 1$

To be used for stress limitation check for concrete and steel.

Frequent combination of actions:

$G_k + P + \psi_{1,1}\ Q_{k,1} + \Sigma\ \psi_{2,i}\ Q_{k,i}$ $\quad i > 1$

To be used for decompression check or crack width check for prestressed concrete with bonded tendons.

Quasi-permanent combination of actions:

$G_k + P + \Sigma\ \psi_{2,i}\ Q_{k,i}$ $\quad i > 1$

To be used for crack width check for reinforced concrete and prestressed concrete without bonded tendons and deformation check.

Terms

Combination value of a variable action: $\psi_0\ Q$
Frequent value of a variable action: $\psi_1\ Q$
Quasi-permanent value of a variable action: $\psi_2\ Q$

Load cases

In considering the combinations of actions (see Section 6 and Annex A2 of BS EN 1990) the relevant load cases shall be considered to enable the critical design conditions to be established at all sections, within the structure or part of the structure considered.

BS EN 1992-2
5.1.3

5.5 Geometrical imperfections

5.5.1 General

The unfavourable effects of possible deviations in the geometry of the structure and the position of loads shall be taken into account in the analysis of members and structures.

BS EN 1992-1-1
5.2(1)

Imperfections shall be taken into account in ultimate limit states in persistent and accidental design situations.

BS EN 1992-1-1
5.2(2)

5.5.2 Imperfections and global analysis of structures

BS EN 1992-2 5.2(105)

Imperfections may be represented by an inclination θ_i given by:

θ_1 = (1/200) α_h

where

α_h = $2/l^{0.5} \leq 1.0$
l = length or height in metres

BS EN 1992-1-1 5.2(7)

For isolated members the effect of the imperfections may be represented as an eccentricity or by a transverse force.

BS EN 1992-1-1 Exp. (5.2)

The eccentricity, e_i, given by
$e_i = \theta_i\, l_0 / 2$ where l_0 is the effective length

The transverse force is applied in the position that gives maximum moment.

$H_i = \theta_1 N k$

where

H_i = action applied at that level
N = axial load
k = 1.0 for unbraced members (see Figure 5.4a)
= 2.0 for braced members (see Figure 5.4b)

BS EN 1992-2 5.2(106)

For arch bridges, the shape of imperfections in the horizontal and vertical planes should be based on the shape of the first horizontal and vertical buckling mode shape respectively. Each mode shape may be idealised by a sinusoidal profile. The amplitude should be taken as $a = \theta_1 l / 2$, where l is the half wavelength.

PD 6687-2 6.2

The disposition of imperfections used in analysis should reflect the behaviour and function of the structure and its elements. The shape of imperfection should be based on the anticipated mode of buckling of the member. For example, in the case of bridge piers, an overall lean imperfection should be used where buckling will be in a sway mode ("unbraced" conditions), while a local eccentricity within the member should be used where both ends of the member are held in position ("braced" conditions).

In using Expression (5.2) of Part 1-1 the eccentricity, e_i, derived should be taken as the amplitude of imperfection over the half wavelength of buckling; Figure 5.5 shows the imperfection suitable for a pier rigidly built in for moment at each end. A lean imperfection should however be considered in the design of the positional restraints for braced members. Further guidance and background are given in Hendy and Smith. [20]

BS EN 1992-1-1 fig. 5.1a

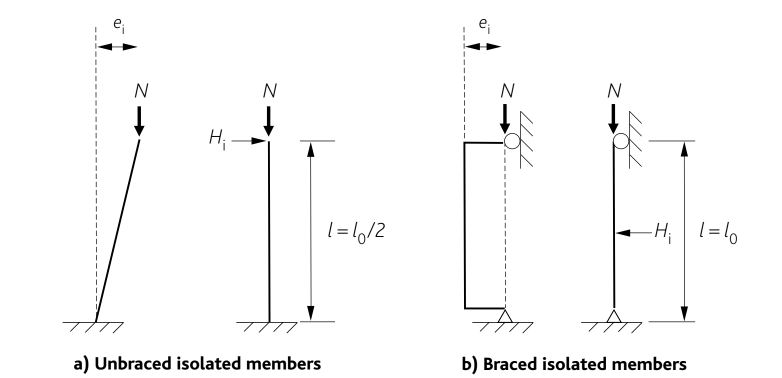


Figure 5.4
Examples of the effects of geometric imperfections

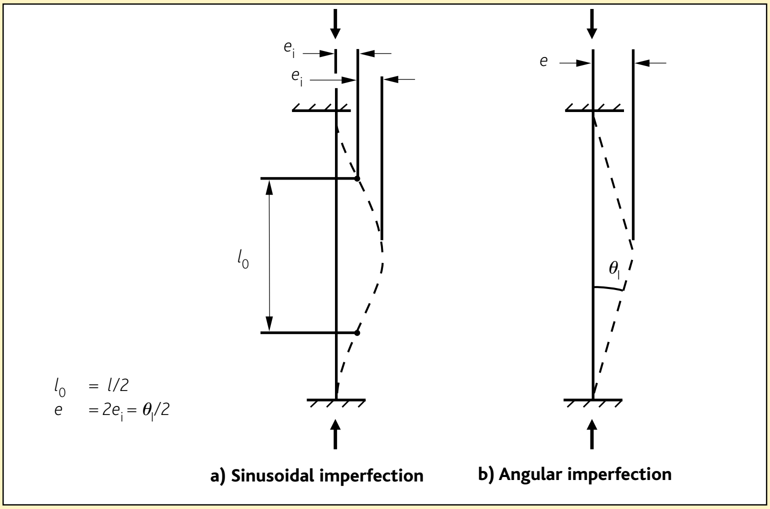


Figure 5.5
Imperfections for pier built in at both ends

PD 6687-2 fig. 1

5.6 Design moments in columns

5.6.1 Definitions

Bracing members

Bracing members are members that contribute to the overall stability of the structure, whereas braced members do **not** contribute to the overall stability of the structure.

BS EN 1992-1-1 5.8.1

Effective length l_0

BS EN 1992-1-1 5.8.3.2(3)

For braced members:

$l_0 = 0.5l[(1 + k_1/(0.45 + k_1))\ (1 + k_2/(0.45 + k_2))]^{0.5}$

For unbraced members l_0 is the larger of either:

$l_0 = l[1 + 10k_1k_2/(k_1 + k_2)]^{0.5}$
or
$l_0 = l[1 + k_1/(1.0 + k_1)]\ [1 + k_2/(1.0 + k_2)]$

where
l = clear height of the column between the end restraints
k_1, k_2 = relative flexibilities of rotational restraints at ends 1 and 2 respectively

Examples of different buckling modes and corresponding effective length factors for isolated members are shown in Figure 5.6. PD 6687-2[3] notes that the effective lengths shown do not cover typical bridge cases and provides additional examples, which are given in Table 5.1.

BS EN 1992-1-1 5.8.3.2(2)
PD 6687-2, 6.71

BS EN 1992-1-1 fig. 5.7

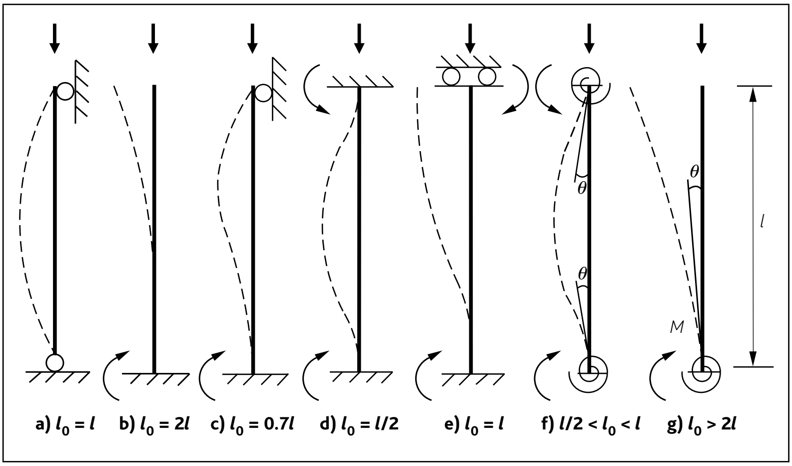


Figure 5.6
Examples of different buckling modes and corresponding effective lengths for isolated members

BS EN 1992-1-1 5.8.3.2(1)

Slenderness ratio, λ

Slenderness ratio $\lambda = l_0/i$

where
i = the radius of gyration of the uncracked concrete section

Ignoring reinforcement:

$\lambda = 3.46\ l_0/h$ for rectangular sections
$= 4.0\ l_0/d$ for circular sections

where
h = the depth in the direction under consideration
d = the diameter

Limiting slenderness ratio λ_{lim}

The limiting slenderness ratio, λ_{lim}, above which second order effects should be considered, is given by

BS EN 1992-1-1 5.8.3.1(1) & NA

$\lambda_{lim} = 20ABC/n^{0.5}$

where
$A = 1/(1 + 0.2\varphi_{ef})$ (if φ_{ef} is not known A may be taken as 0.7)
where

BS EN 1992-1-1 5.8.4

BS EN 1992-1-1 3.1.4(4), 3.1.4(5)

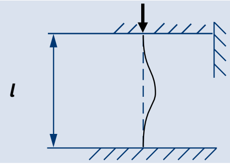
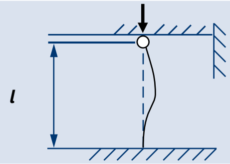
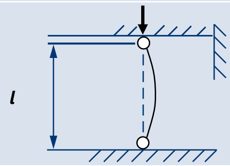
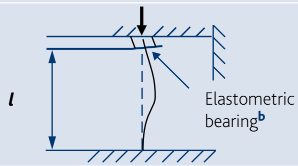
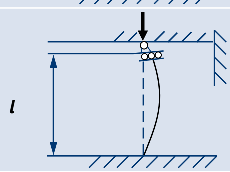
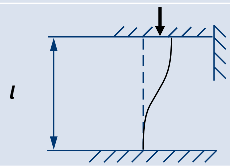
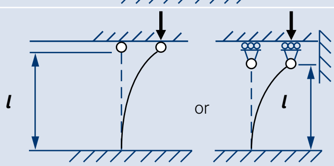
φ_{ef} = effective creep factor = $\varphi(\infty, t_0)M_{0Eqp}/M_{0Ed}$
$\varphi(\infty, t_0)$ = final creep coefficient (see Section 3.1.2)
M_{0Eqp} = the first order bending moment in the quasi-permanent load combination (SLS)
M_{0Ed} = the first order bending moment in design load combination (ULS)

$B = (1 + 2\omega)^{0.5}$ (if ω is not known B may be taken as 1.1)
where
ω = mechanical reinforcement ratio
$= (A_s/A_c)(f_{yd}/f_{cd})$
A_s = total area of longitudinal reinforcement

$C = 1.7 - r_m$ (If r_m is not known, C may be taken as 0.7
where
r_m = M_{01}/M_{02}, where M_{01} and M_{02} are the first order end moments at ULS with M_{02} numerically larger than M_{01}. If M_{01} and M_{02} give tension on the same side then r_m is positive (and C < 1.7), see Figure 5.7
r_m = 1.0 for unbraced members and braced members in which the first order moments are caused largely by imperfections or transverse loading

PD 6687-2 table 1

Table 5.1
Effective height, l_0 for columns

Case	Idealized column and buckling mode	Restraints			Effective height l_0
		Location	Position	Rotation	
1		Top Bottom	Full Full	Full[a] Full[a]	0.70l
2		Top Bottom	Full Full	None Full[a]	0.85l
3		Top Bottom	Full Full	None None	1.0l
4		Top Bottom	None[c] Full	None[c] Full[a]	1.3l
5		Top Bottom	None Full	None Full[a]	1.4l
6		Top Bottom	None Full	Full[a] Full[a]	1.5l
7		Top Bottom	None Full	None Full[d]	2.3l

Notes
The height of the bearings is negligible compared with that of the column.

Key
a Rotational restraint is at least 4(EI)/l, where (EI) is the flexural rigidity of the column
b May also be used with roller bearings provided that the rollers are held in place by an effective means, such as racks
c Lateral and rotational rigidity of elastomeric bearings are negligible
d Rotational restraint is at least 8(EI)/l, where (EI) is the flexural rigidity of the column

$n = N_{Ed}/(A_c f_{cd})$
where
N_{Ed} = design value of axial force

BS EN 1992-1-1 5.8.3.1(1) & NA

BS EN 1992-1-1 5.8.4(4)

Note: φ_{ef} may be taken as 0 if all the following conditions are met:
- $\varphi(\infty, t_0) \leq 2.0$;
- $\lambda \leq 75$; and
- $M_{0Ed}/N_{Ed} \geq h$, the depth of the cross-section in the relevant direction.

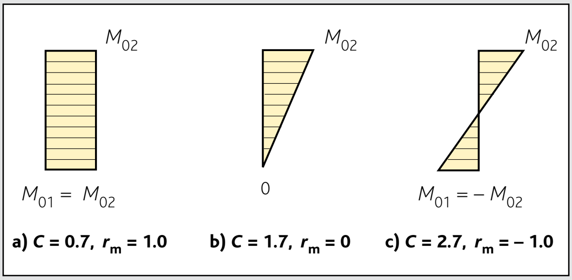


Figure 5.7
Values of *C* for different values of r_m

5.6.2 Design bending moments

Non-slender columns

BS EN 1992-1-1 5.8.3.1(1)

When $\lambda \leq \lambda_{lim}$ i.e. when non-slender, the design bending moment in a column is

$M_{Ed} = M_{02}$

where
- M_{Ed} = design moment
- M_{02}, M_{01} = first order end moments at ULS including allowances for imperfections. M_{02} is numerically larger than M_{01}. Attention should be paid to the sign of the bending moments. If they give tension on the same side, M_{01} and M_{02} should have the same sign
- M_{02} = $M + e_i N_{Ed}$

  where
  - M = largest moment from first order analysis (elastic moments without redistribution)and ignoring effect of imperfections
  - N_{Ed} = design value of axial force
  - e_i = eccentricity due to imperfections = $\theta_i\, l_0/2$

BS EN 1992-1-1 5.2.7

BS EN 1992-1-1 6.1(4)

BS EN 1992-1-1 5.8.3.2(3)

For columns in braced systems $e_i = l_0/400$ (i.e. $\theta_i = l/200$ for most braced columns). The design eccentricity should be at least (h/30) but not less than 20 mm.

where
- θ = inclination used to represent imperfections
- l_0 = effective length of column
- h = depth of the section in the relevant direction

Slender columns (nominal curvature method)

BS EN 1992-1-1 5.8.8.1(1)

This method is primarily suitable for isolated members with constant normal force and a defined effective length l_0. The method gives a nominal second order moment based on a deflection, which in turn is based on the effective length and an estimated maximum curvature. Other methods are available in BS EN 1992-1-1.

BS EN 1992-1-1 5.8.8.2(1)

The design moment is:
$M_{Ed} = M_{0Ed} + M_2$
where

M_{0Ed} = first order moment, including the effect of imperfections (and may be taken as M_{0E})
M_2 = nominal second order moment

The maximum value of M_{Ed} is given by the distributions of M_{0Ed} and M_2; the latter may be taken as parabolic or sinusoidal over the effective length.

For braced structures (see Figure 5.8):

$M_{Ed} = \text{MAX}\{M_{0e} + M_2; M_{02}; M_{01} + 0.5\ M_2\}$

For unbraced structures:

$M_{Ed} = M_{02} + M_2$

PD 6687-1 2.11

For moments without loads applied between their ends, differing first order end moments M_{01}

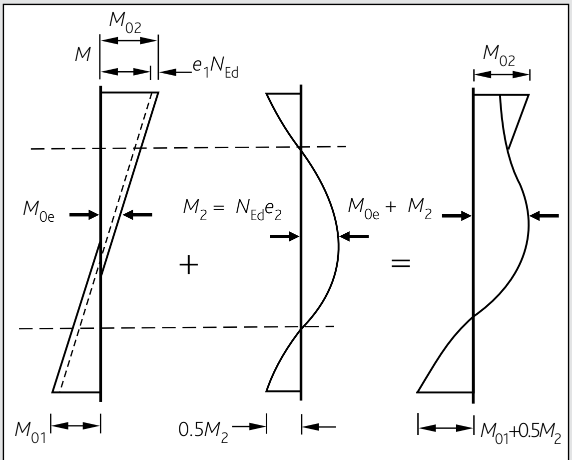


Figure 5.8
Moments in braced columns

and M_{02} (see above) may be replaced by an equivalent first order end moment M_{0e}:
$M_{0e} = 0.6\ M_{02} + 0.4\ M_{01} \geq 0.4\ M_{02}$

BS EN 1992-1-1 5.8.8.2(2)

The nominal second order moment M_2 is

$M_2 = N_{Ed}\ e_2$

BS EN 1992-1-1 5.8.8.2(3)

where

N_{Ed} = design value of axial force
e_2 = deflection
= $(1/r)\ l_o^2/c$
$1/r$ = curvature = $K_r K_\varphi (f_{yd}\ /(E_s 0.45d))$
K_r = $(n_u - n)\ /\ (n_u - n_{bal}) \leq 1.0$

BS EN 1992-1-1 5.8.8.2(4)

BS EN 1992-1-1 5.8.8.3

Note: K_r may be derived from column charts.

n_u = $1 + \omega$
ω = mechanical reinforcement ratio
= $(A_s/A_c)(f_{yd}/f_{cd})$ as in Section 5.6.1 above
n = $N_{Ed}/A_c f_{cd}$ as defined in Section 5.6.1 above
n_{bal} = value of n at maximum moment of resistance and may be taken as 0.4
K_φ = $1 + \beta\varphi_{ef} \geq 1.0$
β = $0.35 + (f_{ck}/200) - (\lambda/150)$
λ = slenderness ratio l_0/i

i	=	radius of gyration of the uncracked concrete section
φ_{ef}	=	effective creep coefficient as defined in Section 5.6.1
l_0	=	effective length of column
E_s	=	200 GPa
c	=	factor depending on the curvature distribution (see below)

BS EN 1992-1-1 3.2.7(4)

BS EN 1992-1-1 5.8.8.2(4)

For constant cross section, $c = 10$ ($\approx \pi^2$) is normally used. If the first order moment is constant, a lower value should be considered (8 is a lower limit, corresponding to constant total moment).

5.6.3 Biaxial bending

BS EN 1992-1-1 5.8.9(2)

BS EN 1992-1-1 5.8.9(3)

Separate design in each principal direction, disregarding biaxial bending, may be undertaken as a first step. No further check is necessary if:

$0.5 \leq \lambda_y / \lambda_z \leq 2.0$ and, for rectangular sections, and
$0.2 \geq (e_y/h_{eq})/(e_z/b_{eq}) \geq 5.0$

where

λ_y, λ_z	=	slenderness ratios l_0/i with respect to the y- and z-axes
e_y	=	M_{Edy}/N_{Ed}
h_{eq}	=	$3.46 i_z$ (= h for rectangular sections)
e_z	=	M_{Edz}/N_{Ed}
b_{eq}	=	$3.46 i_y$ (= b for rectangular sections)

where

N_{Ed}	=	design value of axial force
M_{Edy}, M_{Edz}	=	design moment in the respective direction. (Moments due to imperfections need be included only in the direction where they have the most unfavourable effect.)

BS EN 1992-1-1 5.8.9(2)

Note: for square columns $(e_y/h_{eq})/(e_z/b_{eq}) = M_{Edy}/M_{Edz}$

BS EN 1992-1-1 5.8.9(4)

If this condition is not satisfied, and in the absence of an accurate cross-section design for biaxial bending, the following simplified criterion may be used:

$$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$$

where

M_{Rdy}, M_{Rdz}	=	design moment around the respective axis including second order moment in the direction where it will give the most unfavourable effect.
a	=	an exponent: for circular or elliptical sections, $a = 2.0$, for rectangular sections, interpolate between
	=	1.0 for $N_{Ed}/N_{Rd} = 0.1$
	=	1.5 for $N_{Ed}/N_{Rd} = 0.7$
	=	2.0 for $N_{Ed}/N_{Rd} = 1.0$

5.7 Corbels

5.7.1 Definition

Corbels are short cantilevers projecting from columns or walls with the ratio of shear span (i.e. the distance between the face of the applied load and the face of the support) to the depth of the corbel in the range 0.5 to 2.0.

5.7.2 Analysis

Corbels ($a_c < z_0$) may be designed using strut and tie models (see Figure 5.9) or as short beams designed for bending and shear.

The inclination of the strut is limited by $1.0 \leq \tan\theta \leq 2.5$.

If $a_c \leq 0.5h_c$ closed horizontal or inclined links with $A_{s,lnk} \geq 0.5A_{s,main}$ should be provided in addition to the main tension reinforcement (see Figure 5.10a).

If $a_c > 0.5h_c$ and $F_{Ed} > V_{Rd,c}$ (see Section 7.2.1) closed vertical links with $A_{s,lnk} \geq 0.5F_{Ed}/f_{yd}$ should be provided in addition to the main tension reinforcement (see Figure 5.10b).

The main tension reinforcement should be anchored at both ends. It should be anchored in the supporting element on the far face and the anchorage length should be measured from the location of the vertical reinforcement in the near face. The reinforcement should be anchored in the corbel and the anchorage length should be measured from the inner face of the loading plate.

If there are special requirements for crack limitation, inclined stirrups at the re-entrant opening can be effective.

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The check of the compression strut can effectively be made by limiting the shear stress such that $F_{Ed} \leq 0.5b_w d\nu' f_{cd}$

where

$\nu' = 1 - (f_{ck}/250)$

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C$

$\alpha_{cc} = 0.85$

BS EN 1992-1-1 6.5.2(2)

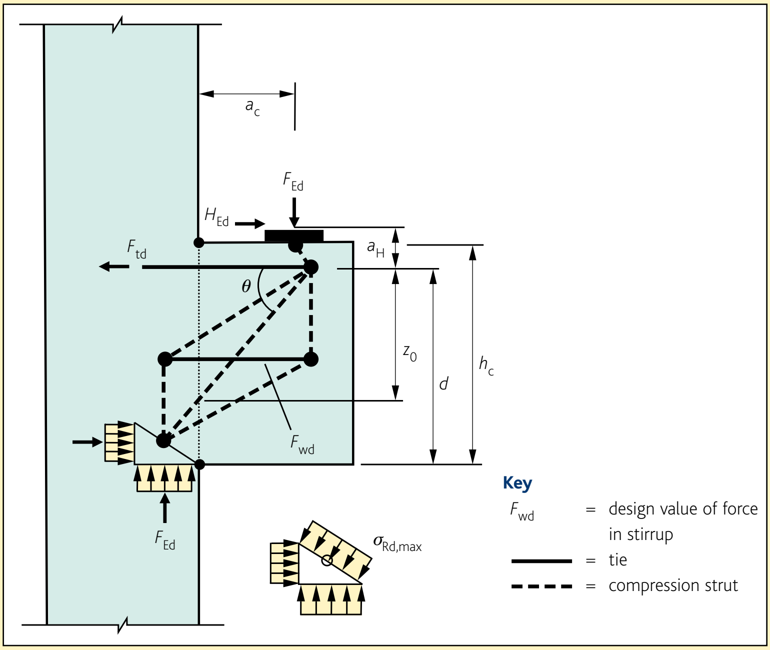


Figure 5.9
Corbel strut-and-tie model

PD 6687 fig. B4

PD 6687
fig. B5

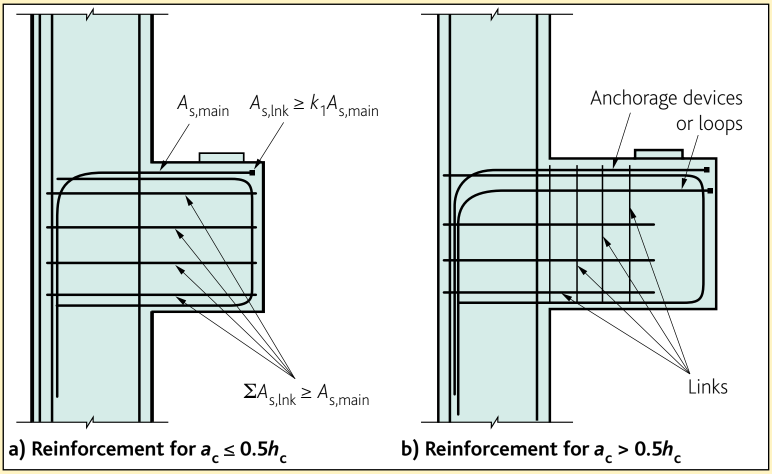


Figure 5.10
Corbel detailing

5.8 Lateral instability of slender beams

BS EN 1992-1-1
5.9

Lateral instability of slender beams shall be taken into account where necessary, e.g. for precast beams during transport and erection, for beams without sufficient lateral bracing in the finished structure etc. Geometric imperfections shall be taken into account. A lateral deflection of l / 300 should be assumed as a geometric imperfection in the verification of beams in unbraced conditions, with l = total length of beam. In finished structures, bracing from connected members may be taken into account

Second order effects in connection with lateral instability may be ignored if the following conditions are fulfilled:

persistent situations: $l_{0t} /b \leq 50/(h/b)^{1/3}$ and $h/b \leq 2.5$
transient situations: $l_{0t} /b \leq 70/(h/b)^{1/3}$ and $h/b \leq 3.5$

where
- l_{0t} = distance between torsional restraints
- h = total depth of beam in central part of l_{0t}
- b = width of compression flange

Torsion associated with lateral instability should be taken into account in the design of supporting structures.

6 Bending and axial force

6.1 Assumptions

In determining the resistance of sections, the following assumptions are made.

BS EN 1992-1-1 6.1(2)

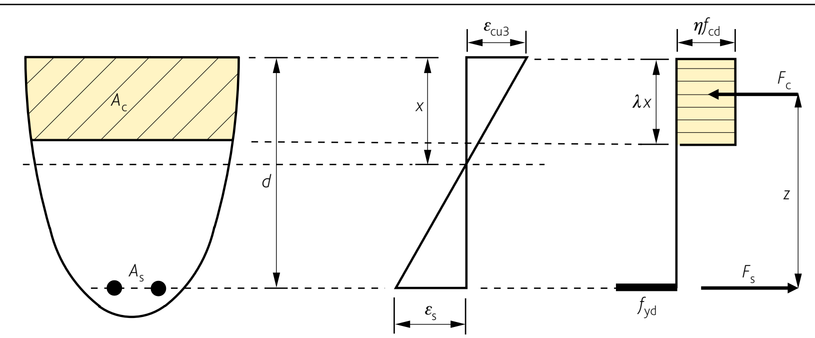
- Plane sections remain plane.
- Strain in the bonded reinforcement, whether in tension or compression, is the same as that in the surrounding concrete.
- Tensile strength of the concrete is ignored.
- Rectangular stress distribution in the section is as shown in Figure 6.1. Other stress–strain relationships are given in BS EN 1992-1-1 Cl. 3.1.7.
- Stresses in reinforcement are derived from Figure 6.2. The inclined branch of the design line may be used when strain limits are checked.
- For sections not fully in compression, the compressive strain in concrete should be limited to ε_{cu2} (see Figure 6.3).
- For sections in pure axial compression, the compressive strain in concrete should be limited to ε_{c2} (see Figure 6.3).
- For situations intermediate between these two conditions, the strain profile is defined by assuming that the strain is ε_{c2} at half the depth of the section (see Figure 6.3).

BS EN 1992-1-1 3.1.7(3), fig. 3.5

BS EN 1992-1-1 fig. 6.1

BS EN 1992-1-1 fig. 3.5

Expressions derived from Figures 6.1 to 6.3 are provided in Section 17.



For f_{ck} ≤ 50 MPa, $\varepsilon_{cu3} = 0.0035$, η = 1 and $\lambda = 0.8$. For other concrete classes see Table 6.1

x = neutral axis depth

ε_{cu3} = ultimate compressive strain in concrete

ε_s = tensile strain in reinforcement

$F_c(F_s)$ = force in concrete (steel)

d = effective depth

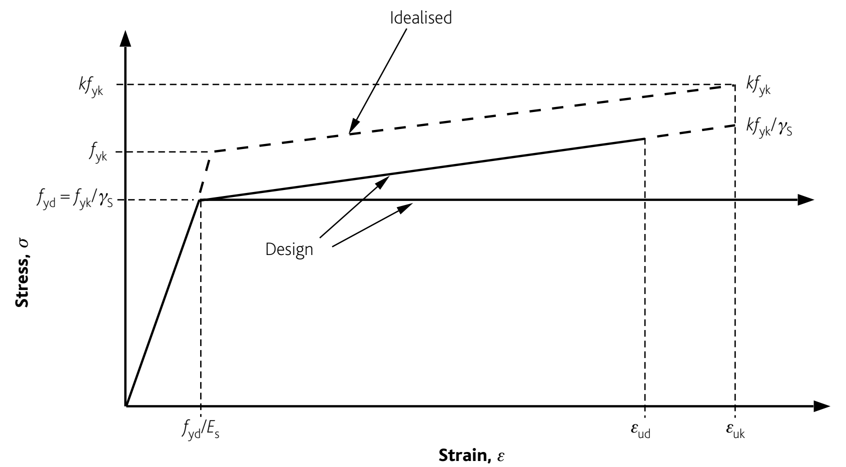
z = lever arm

Figure 6.1
Rectangular stress distribution

Table 6.1
Values for η and λ when $f_{ck} > 50$

f_{ck}	ε_{cu3} [a]	η [b]	λ [c]
55	0.00313	0.98	0.79
60	0.00288	0.95	0.78
70	0.00266	0.90	0.75

Key
a $\varepsilon_{cu3}(\%) = 2.6 + 35\,[(90 - f_{ck})/100]^4$
b $\eta = 1 - (f_{ck} - 50)/200$
c $\lambda = 0.8 - (f_{ck} - 50)/400$



Where

k = $(f_t / f_y)_k$ (See Table 3.3)

f_{yk} = characteristic yield strength of reinforcement

f_{yd} = design yield strength of reinforcement

f_t = tensile strength of reinforcement

γ_S = partial factor for reinforcing steel = 1.15

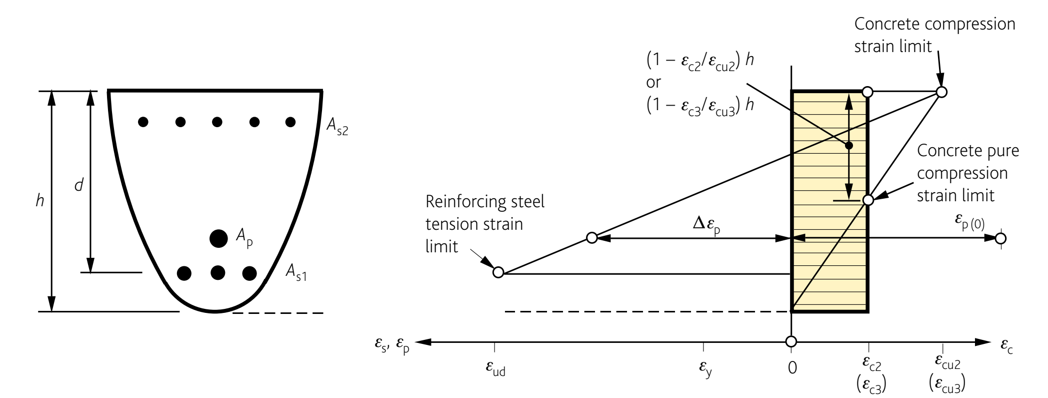
ε_{ud} = design strain limit for reinforcing steel = $0.9\ \varepsilon_{uk}$

ε_{uk} = characteristic strain of reinforcement at maximum force (see Table 3.3)

BS EN 1992-1-1 fig. 3.8

Figure 6.2
Idealised and design stress–strain diagrams for reinforcing steel (for tension and compression)

BS EN 1992-1-1 fig. 6.1



a) Section

b) Strain

Where

A_{s1} = area of reinforcing steel in layer 1

A_p = area of prestressing steel

d = effective depth

h = overall depth of section

ε_c = compressive strain in concrete

ε_s = strain in reinforcing steel

$\varepsilon_{p(0)}$ = initial strain in prestressing steel

$\Delta\varepsilon_p$ = change in strain in prestressing steel

ε_{ud} = design strain limit for reinforcing steel in tension

ε_{c3} = compressive strain limit in concrete for concrete in pure axial compression assuming bilinear stress–strain relationship (see Figure 3.4 of BS EN 1992-1-1).

ε_{c3} (‰) = 1.75 for $f_{ck} \leq 50$ MPa

ε_{c3} (‰) = $1.75 + 0.55\ [(f_{ck} - 50)/40]$ for $f_{ck} > 50$ MPa

ε_{cu2} = (= ε_{cu3}) compressive strain limit in concrete not fully in pure axial compression

ε_{cu3} (‰) = 3.5 for $f_{ck} \leq 50$ MPa.

ε_{cu3} (‰) = $2.6 + 0.35\ [(90 - f_{ck})/100]^4$ for $f_{ck} > 50$ MPa

ε_y = reinforcement yield strain

Figure 6.3
Possible strain distributions in the ultimate limit state

7 Shear

7.1 General

7.1.1 Definitions

BS EN 1992-1-1 6.2.1(1)

For the verification of the shear resistance the following symbols are defined:

$V_{Rd,c}$ = design shear resistance of a member without shear reinforcement
$V_{Rd,s}$ = design value of the shear force that can be sustained by the yielding shear reinforcement
$V_{Rd,max}$ = design value of the maximum shear force that can be sustained by the member, limited by crushing of the compression struts

These resist the applied shear force, V_{Ed}.

7.1.2 Requirements for shear reinforcement

BS EN 1992-1-1 6.2.1(3)

If $V_{Ed} \leq V_{Rd,c}$, no calculated shear reinforcement is necessary. However, minimum shear reinforcement should still be provided (see Section 15.2.6) except in:

BS EN 1992-1-1 6.2.1(4)

- Slabs, where actions can be redistributed transversely.
- Members of minor importance, which do not contribute significantly to the overall resistance and stability of the structure (e.g. lintels with a span of less than 2 m).

BS EN 1992-1-1 6.2.1(5)

If $V_{Ed} > V_{Rd,c}$ shear reinforcement is required such that $V_{Rd,s} > V_{Ed}$. The resistance of the concrete to act as a strut should also be checked.

BS EN 1992-1-1 6.2.1(8)

In members subject predominantly to uniformly distributed loading, the following apply:

- Shear at the support should not exceed $V_{Rd,max}$.
- Required shear reinforcement should be calculated at a distance d from the face of the support and continued to the support.

BS EN 1992-1-1 6.2.1(7)

The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear (see Section 15.2.2).

BS EN 1992-2 3.1.2(2) & NA

The UK NA limits shear strength of concrete higher than C50/60 to that of C50/60.

7.1.3 Shear and transverse bending

BS EN 1992-2 6.2.106 (101)

Due to the presence of compressive stress fields arising from shear and bending, the interaction between longitudinal shear and transverse bending in the webs of box girder sections should be considered in the design. When $V_{Ed}/V_{Rd,max} < 0.2$ or $M_{Ed}/M_{Rd,max} < 0.1$, this interaction can be disregarded (where $V_{Rd,max}$ and $M_{Rd,max}$ represent respectively the maximum web capacity for longitudinal shear and transverse bending).

Further information on the interaction between shear and transverse bending may be found in Annex MM of BS EN 1992-2.

7.2 Resistance of members not requiring shear reinforcement

7.2.1 General situation

BS EN 1992-2 6.2.2(101) & NA

The design value for the shear resistance is given by:

$$V_{Rd,c} = [(0.18/\gamma_C)k(100\,\rho_l\,f_{ck})^{1/3} + 0.15\,\sigma_{cp}]\,b_w\,d$$
$$\geq (0.035k^{1.5}\,f_{ck}^{0.5} + 0.15\,\sigma_{cp})\,b_w\,d$$

where

$k = 1 + (200/d)^{0.5} \leq 2.0$ (d in mm; see Table 7.1)

$\gamma_C = 0.15$

$\rho_l = A_{sl}/(b_w d) \leq 0.02$

where

A_{sl} = area of the tensile reinforcement extending at least $l_{bd} + d$ beyond the section considered (see Figure 7.1); the area of bonded prestressing steel may be included in the calculation of A_{sl}. In this case a weighted mean value of d may be used.

l_{bd} = design anchorage length

b_w = smallest width of the cross-section in the tensile area

$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$ (MPa)

where

N_{Ed} = axial force in the cross-section due to loading or prestressing in newtons ($N_{Ed} > 0$ for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C = area of concrete cross-section (mm^2)

Table 7.1

$V_{Rd,c}$, shear resistance without shear reinforcement where $\sigma_{cp} = 0$ (MPa)

$\rho_l = A_s/(bd)$	Effective depth d (mm)										
	≤200	225	250	275	300	350	400	450	500	600	750
0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.69	0.68	0.65	0.64	0.62	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.74	0.73	0.71	0.69	0.67	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.79	0.78	0.75	0.73	0.71	0.70	0.67	0.65
1.75%	0.90	0.87	0.85	0.83	0.82	0.79	0.77	0.75	0.73	0.71	0.68
≥ **2.00%**	0.94	0.91	0.89	0.87	0.85	0.82	0.80	0.78	0.77	0.74	0.71
k	2.000	1.943	1.894	1.853	1.816	1.756	1.707	1.667	1.632	1.577	1.516

Notes

1 Table derived from BS EN 1992-1-1 and the UK National Annex.

2 Table created for f_{ck} = 30 MPa assuming vertical links.

3 For $\rho_l \geq 0.4\%$ and f_{ck} = 25 MPa, apply factor of 0.94; f_{ck} = 35 MPa, apply factor of 1.05; f_{ck} = 40 MPa, apply factor of 1.10; f_{ck} = 45 MPa, apply factor of 1.14; $f_{ck} \geq$ 50 MPa, apply factor of 1.19

7.2.2 Uncracked regions in prestressed members

BS EN 1992-1-1 6.2.2(2)

In prestressed single-span members without shear reinforcement, the shear resistance of the regions cracked in bending may be calculated using the Expression in Section 7.2.1. In regions uncracked in bending (where the flexural tensile stress is smaller than $f_{ctk,0.05}/\gamma_C$) the shear resistance should be limited by the tensile strength of the concrete. In these regions the shear resistance is given by:

$$V_{Rd,c} = (I\, b_w/S)\, (f_{ctd}{}^2 + \alpha_l\, \sigma_{cp}\, f_{ctd})^{0.5}$$

where

I = second moment of area

b_w = width of the cross-section at the centroidal axis, allowing for the presence of ducts (see below)

S = first moment of area above and about the centroidal axis

α_l = $l_x/l_{pt2} \leq 1.0$ for pretensioned tendons

= 1.0 for other types of prestressing

l_x = distance of section considered from the starting point of the transmission length

l_{pt2} = upper bound value of the transmission length of the prestressing (See section 14.6)

σ_{cp} = concrete compressive stress at the centroidal axis due to axial loading and/or prestressing ($\sigma_{cp} = N_{Ed}/A_c$ in MPa, $N_{Ed} > 0$ in compression)

PD 6687-2 7.2.2

Alternatively , where there is no axial force, including prestressing
$V_{Rd,c} = b_w\, d\, v_{Rd,c}$ with $v_{Rd,c}$ available from Table 7.1

In most practical cases if $v_{Ed} < v_{Rd,c}$ shear reinforcement will not be required

where

v_{Ed} = shear stress for sections without shear reinforcement = $V_{Ed}/b_w d$.

$v_{Rd,c}$ may be interpolated from Table 7.1.

7.2.3 Short span shear enhancement

The approach in BS EN 1992-1-1 of applying a reduction factor to the load is not suitable for cases with multiple, indirect or distributed loads. Therefore, in the NA to BS EN 1992-2, the expression for $C_{Rd,c}$ has been modified from the recommended value so that the effects of shear enhancement are taken into account through increasing the shear resistance of members near to supports. See Figure 7.2.

For members with actions applied at a distance α_v where $0.5d \leq \alpha_v \leq 2d$:

$$V_{Rd,c} = \left[\left(\frac{0.18}{\gamma_C}\right)\left(\frac{2d}{\alpha_v}\right) k\,(100\rho_1 f_{ck})^{1/3} + 0.15\sigma_{cp} \right] b_w\, d$$

BS EN 1992-2 NA 6.2.2(101)

provided that the shear force, V_{Ed}, is not multiplied by β (BS EN 1992-1-1, Cl. 6.2.2(6)) and the longitudinal reinforcement is fully anchored at the support.

BS EN 1992-2 fig. 6.3

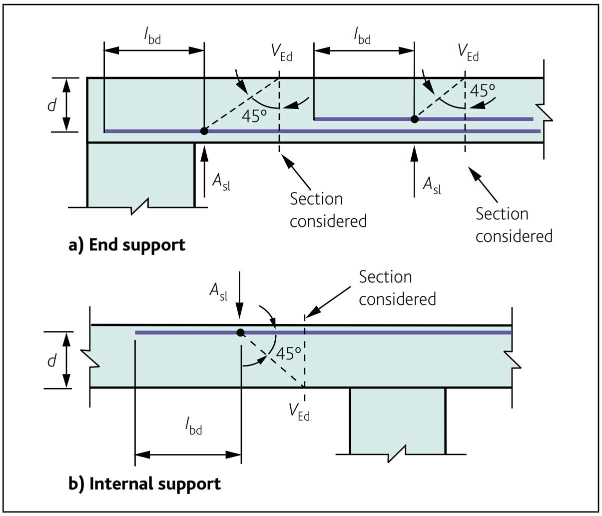


Figure 7.1
Definition of A_{sl}

BS EN 1992-1-1
fig. 6.4

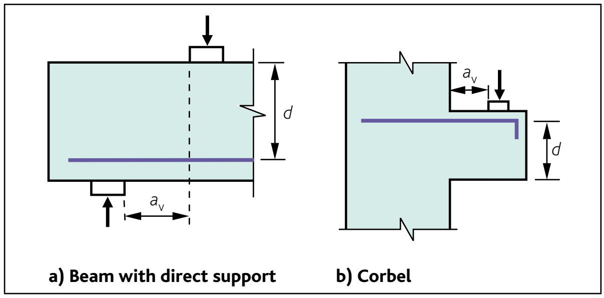


Figure 7.2
Loads near supports

7.3 Resistance of members requiring shear reinforcement

BS EN 1992-1-1
6.2.3

7.3.1 Basis

The design of members with shear reinforcement is based on the truss model shown in Figure 7.3. A simplified version of this diagram is shown in Figure 7.4.

BS EN 1992-2
fig. 6.5

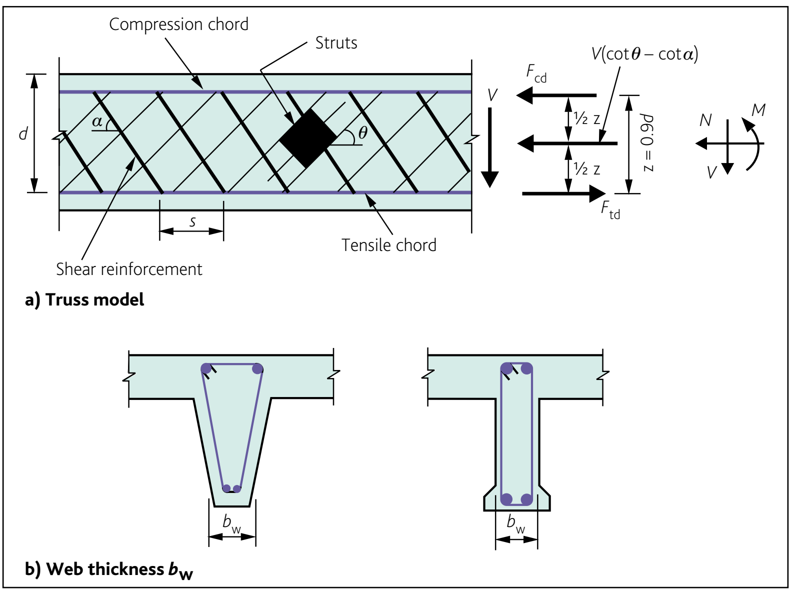


Figure 7.3
Truss model and notation for shear reinforced members

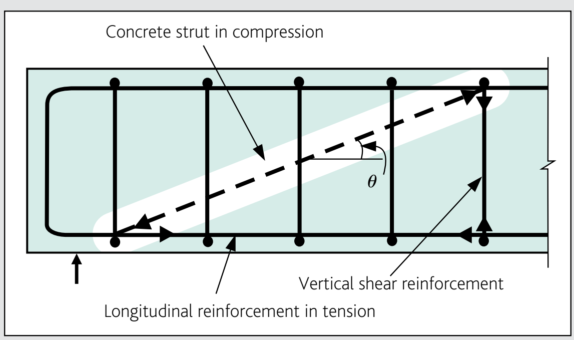


Figure 7.4
Variable strut angle, θ

7.3.2 Shear resistance check

The resistance of the concrete section to act as a strut $V_{Rd,max}$ should be checked to ensure that it equals or exceeds the design shear force, V_{Ed} i.e. ensure that:

BS EN 1992-1-1 Exp. (6.9) Exp. (6.14) & NA

$$V_{Rd,max} = \alpha_{cw}\, b_w\, z \nu_1\, f_{cd}(\cot\theta + \tan\theta) \geq V_{Ed} \text{ with vertical links}$$
$$= \alpha_{cw}\, b_w\, z \nu_1\, f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta) \geq V_{Ed} \text{ with inclined links}$$

where

z = lever arm (See Section 7.3.3)

BS EN 1992-2 6.2.3(103) & NA

ν_1 = $0.6\,[1 - (f_{ck}/250)]\,(1 - 0.5\cos\alpha)$

f_{cd} = $1.0\, f_{ck}/1.5$

θ = angle of inclination of the strut, such that $\cot\theta$ lies between 1.0 and 2.5.

α = angle of inclination of the links to the longitudinal axis For vertical links $\cot\alpha = 0$.

α_{cw} = coefficient taking account of the state of the stress in the compression chord (see Table 7.2)

= 1 for non-prestressed structures

= $(1 + \sigma_{cp}/f_{cd})$ for $0 < \sigma_{cp} \leq 0.25\, f_{cd}$

= 1.25 for $0.25\, f_{cd} < \sigma_{cp} \leq 0.5\, f_{cd}$

= $2.5\,(1 - \sigma_{cp}/f_{cd})$ for $0.5\, f_{cd} < \sigma_{cp} < 1.0\, f_{cd}$

BS EN 1992-2 6.2.3(103) & NA

where

σ_{cp} = mean compressive stress, measured positive, in the concrete due to the design axial force.

σ_{cp} should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than $0.5d \cot\theta$ from the edge of the support.

The values of ν_1 and α_{cw} should not give rise to a value of $V_{Rd,max}$ greater than $200 b_w^2$ at sections more than a distance d from the edge of a support. For this purpose, the value of b_w does not need to be reduced for ducts.

Table 7.2
Values for α_{cw}

f_{ck}	f_{cd}	Mean compressive stress, σ_{cp}									
		0	0.5	1	2	3	4	5	10	20	30
20	13.3	1	1.04	1.08	1.15	1.23	1.25	1.25	0.63	—	—
25	16.7	1	1.03	1.06	1.12	1.18	1.24	1.25	1.00	—	—
30	20.0	1	1.03	1.05	1.10	1.15	1.20	1.25	1.25	—	—
35	23.3	1	1.02	1.04	1.09	1.13	1.17	1.21	1.25	0.36	—
40	26.7	1	1.02	1.04	1.08	1.11	1.15	1.19	1.25	0.63	—
45	30.0	1	1.02	1.03	1.07	1.10	1.13	1.17	1.25	0.83	—
50	33.3	1	1.02	1.03	1.06	1.09	1.12	1.15	1.25	1.00	0.25
60	40.0	1	1.01	1.03	1.05	1.08	1.10	1.13	1.25	1.25	0.63
70	46.7	1	1.01	1.02	1.04	1.06	1.09	1.11	1.21	1.25	0.89

In the case of straight tendons, a high level of prestress ($\sigma_{cp}/f_{cd} > 0.5$) and thin webs, if the tension and the compression chords are able to carry the whole prestressing force and blocks are provided at the extremity of beams to disperse the prestressing force (see Figure 7.5), it may be assumed that the prestressing force is distributed between the chords. In these circumstances, the compression field due to shear only should be considered in the web ($\alpha_{cw} = 1$).

BS EN 1992-2 fig. 6.101

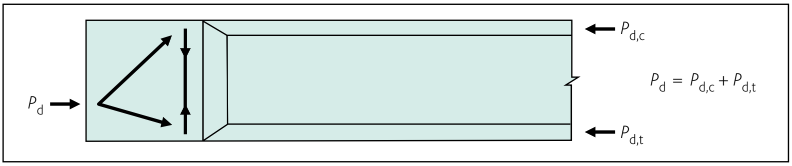


Figure 7.5
Dispersion of prestressing by end blocks within the chords

BS EN 1992-1-1
Exp. (6.15)

The maximum effective cross-sectional area of the shear reinforcement $A_{sw,max}$ for cot $\theta = 1$ is given by:

$$\frac{A_{sw,max}\, f_{ywd}}{b_w\, s} \leq \tfrac{1}{2}\alpha_{cw}\nu_1 f_{cd}$$

7.3.3 Lever arm, *z*

BS EN 1992-1-1
6.2.3(1)

In the shear analysis of reinforced concrete, the approximate value $z = 0.9d$ may normally be used. This is not appropriate when:

PD 6687-2
7.2.4.1

- There is an axial force or prestress, or
- The width at the centroid is greater than the minimum cross-section width in compression, or
- The cross-section has a tension flange but no compression flange

In such cases the lever arm should be determined based on an analysis of the section.

7.3.4 Shear reinforcement required, A_{sw}/s

The cross-sectional area of the shear reinforcement required is calculated using the shear resistance:

BS EN 1992-1-1
Exp. (6.13)

$V_{Rd,s} = (A_{sw}/s) z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \leq V_{Rd,max}$

where

A_{sw} = cross-sectional area of the shear reinforcement. (For $A_{sw,min}$ see Section 15.2.6)
s = spacing of the stirrups
z = lever arm (See Section 7.3.3)
$f_{ywd} = f_{ywk}/\gamma_S$ = design yield strength of the shear reinforcement
α = angle of the links to the longitudinal axis

For vertical links, cot $\alpha = 0$ and sin $\alpha = 1.0$, and:

BS EN 1992-1-1
Exp. (6.8)

$A_{sw}/s \geq V_{Ed}/(z f_{ywd} \cot\theta)$

7.3.5 Webs containing metal ducts

BS EN 1992-1-1
6.2.3(6)

Where the web contains grouted metal ducts with a diameter $\phi > b_w/8$ the shear resistance $V_{Rd,max}$ should be calculated on the basis of a nominal web thickness given by:

BS EN 1992-1-1
Exp. (6.16)

$b_{w,nom} = b_w - 0.5\Sigma\phi$ where ϕ is the outer diameter of the duct and $\Sigma\phi$ is determined for the most unfavourable level.

For grouted metal ducts with $\phi \leq b_w/8$, $b_{w,nom} = b_w$

For non-grouted ducts, grouted plastic ducts and unbonded tendons the nominal web thickness is:

BS EN 1992-1-1
Exp. (6.17)

$b_{w,nom} = b_w - 1.2\,\Sigma\phi$

The value 1.2 is introduced to take account of splitting of the concrete struts due to transverse tension. If adequate transverse reinforcement is provided this value may be reduced to 1.0.

7.3.6 Additional tensile forces

BS EN 1992-1-1
Exp. (6.18)

The additional tensile force, ΔF_{td}, in the longitudinal reinforcement due to shear V_{Ed} may be calculated from:

$\Delta F_{td} = 0.5\, V_{Ed}\,(\cot\theta - \cot\alpha)$
$M_{Ed}/z + \Delta F_{td}$ should be taken as not greater than $M_{Ed,max}/z$

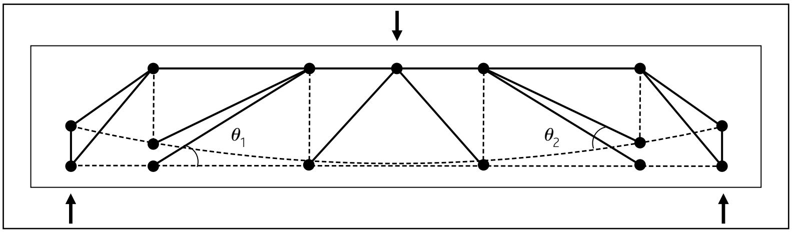
where

$M_{Ed,max}$ = maximum moment along the beam

This additional tensile force gives rise to the 'shift' rule for the curtailment of reinforcement (see Section 15.2).

In the case of bonded prestressing, located within the tensile chord, the resisting effect of prestressing may be taken into account for carrying the total longitudinal tensile force. In the case of inclined bonded prestressing tendons in combination with other longitudinal reinforcement/tendons the shear strength may be evaluated, by a simplification, superimposing two different truss models with different geometry (Figure 7.6); a weighted mean value between θ_1 and θ_2 may be used for concrete stress field verification with Expression (6.9).

BS EN 1992-2 6.2.3(107)



BS EN 1992-2 fig. 6.102N

Figure 7.6
Superimposed resisting model for shear

7.3.7 Members with actions applied near bottom of section

Where load is applied near the bottom of a section, sufficient shear reinforcement to carry the load to the top of the section should be provided in addition to any shear reinforcement required to resist shear.

BS EN 1992-1-1 6.2.1(9)

7.3.8 Shear between web and flanges of T-sections

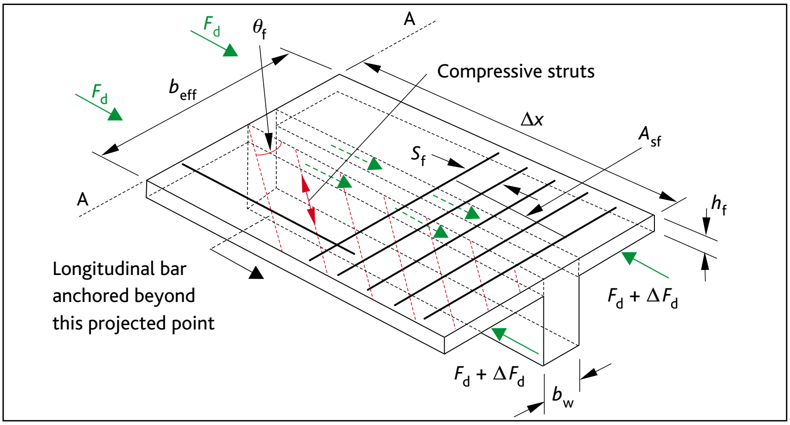
The longitudinal shear stress, v_{Ed}, at the junction between one side of a flange and the web is determined by the change of the normal (longitudinal) force in the part of the flange considered, according to:

BS EN 1992-2 6.2.4(103)

$v_{Ed} = \Delta F_d/(h_f\, \Delta x)$

where

h_f = thickness of flange at the junctions
Δx = length under consideration, see Figure 7.7
ΔF_d = change of the normal force in the flange over the length Δx



BS EN 1992-1-1 fig. 6.7

Figure 7.7
Notations for the connection between flange and web

The maximum value that may be assumed for Δx is half the distance between the section where the moment is 0 and the section where the moment is maximum. Where point loads are applied the length Δx should not exceed the distance between point loads.

Alternatively, considering a length Δx of the beam, the shear transmitted from the web to the flange is $v_{Ed}\,\Delta x/z$ and is divided into three parts: one remaining within the web breadth and the other two going out to the flange outstands. It should be generally assumed that the proportion of the force remaining within the web is the fraction b_w/b_{eff} of the total force. A greater proportion may be assumed if the full effective flange breadth is not required to resist the bending moment. In this case a check for cracks opening at SLS may be necessary.

The rate of change of the flange forces can be underestimated if the effects of web shear on the flange forces are not included, particularly for compression flanges. The chord forces as determined for the design of the web reinforcement should therefore be used.

PD 6687-2, 7.2.5

For compression flanges, F_d may be calculated as follows:

$F_d = (b_{eff} - b_w)\,F_{cd} / (2\,b_{eff})$ when $h_f > x_c$
$F_d = (b_{eff} - b_w)\,f_{cd}\,h_f / 2$ when $h_f \le x_c$

where

x_c = depth of the compression zone
F_{cd} = total force in the compression chord of the shear truss in the web

For tension flanges, F_d may be calculated by determining the proportion of the total tension chord force, F_{td}, carried in each side of the flange.

The transverse reinforcement per unit length A_{sf}/s_f may be determined as follows:

$(A_{sf}\,f_{yd}/s_f) \ge v_{Ed}\,h_f / \cot\theta_f$

BS EN 1992-1-1 6.2.4(4)

To prevent crushing of the compression struts in the flange, the following condition should be satisfied:

$V_{Ed} \le 0.6(1 - f_{ck}/250)f_{cd} \sin\theta_f \cos\theta_f$

where

$f_{cd} = 1.0\,f_{ck}/1.5$

The permitted range of the values for cot θ_f are:

BS EN 1992-1-1 6.2.4(6)

$1.0 \le \cot\theta_f \le 2.0$ for compression flanges ($45° \ge \theta_f \ge 26.5°$)
$1.0 \le \cot\theta_f \le 1.25$ for tension flanges ($45° \ge \theta_f \ge 38.6°$)

If v_{Ed} is less than or equal to $0.4\,f_{ctd}$ no extra reinforcement above that for flexure is required.

Longitudinal tension reinforcement in the flange should be anchored beyond the strut required to transmit the force back to the web at the section where this reinforcement is required (See Section (A – A) of Figure 7.7).

7.3.9 Interface shear between concretes placed at different times

BS EN 1992-1-1 6.2.5

When composite action is assumed between two layers of concrete cast at different times, it is necessary to prevent slip between the two layers. This will result in shear stress at the interface, which should be verified. The force at the interface is the flange force in the new concrete caused by the bending moment on the composite section.

The shear stress at the interface between concrete cast at different times should satisfy the following.

$v_{Edi} \le v_{Rdi}$

where

v_{Edi} = design value of the shear stress in the interface and is given by:

$v_{Edi} = \beta V_{Ed}/(zb_i)$

where

V_{Ed} = transverse shear force

z = lever arm of the composite section

b_i = width of the interface (see Figure 7.8)

β = ratio of the longitudinal force in the new concrete and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

$$v_{Rdi} = cf_{ctd} + \mu\,\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \le 0.5\ 0.6(1 - f_{ck}/250)\ f_{cd}$$

where

the values of c and μ depend on the interface surface and are shown in Table 7.3

$f_{ctd} = f_{ctk,0.05}/1.5$

$f_{cd} = 1.0\ f_{ck}/1.5$

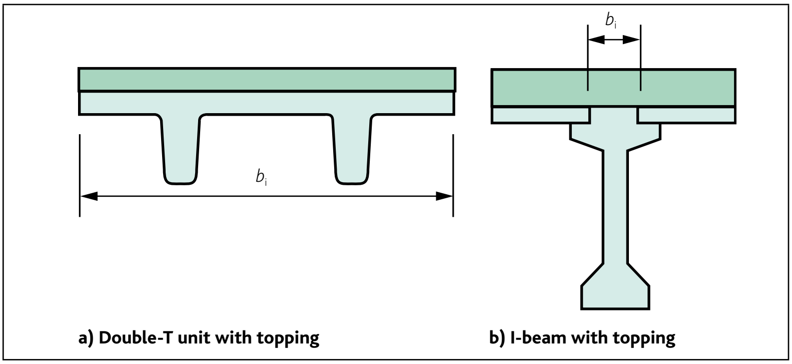
σ_n = stress normal to the interface not to be taken greater than $0.6f_{cd}$

$\rho = A_s/A_i$

A_s = area of reinforcement crossing the interface including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface

A_i = area of the joint

α = angle of reinforcement crossing the interface (45° ≤ α ≤ 90°)



BS EN 1992-1-1 fig. 6.8

Figure 7.8
Examples of interfaces for shear

Table 7.3
Values of μ and c for various interfaces

Classification of interface surfaces	Friction coefficient, μ	Shear coefficient, c
Very smooth: surface cast against steel, plastic or smooth wooden moulds	0.5	0.10
Smooth: slipformed or extruded surface or face untreated after vibration	0.6	0.20
Rough: a surface with at least 3 mm roughness at 40 mm spacing	0.7	0.40

BS EN 1992-1-1 6.2.5(2)

Where a stepped distribution of transverse reinforcement is used, the total resistance within any band of reinforcement should be not less than the total longitudinal shear in the same length and the longitudinal shear stress evaluated at any point should not exceed the resistance evaluated locally by more than 10%.

PD 6687-2 7.2.6

8 Punching shear

8.1 General

8.1.1 Basis of design

BS EN 1992-1-1
6.4.1

Punching shear is a local shear failure around concentrated loads on slabs and the most common situations where the punching shear rules are relevant in bridge applications are in the design of pile caps, pad footings, and deck slabs which are subjected to wheel loads and around supports in slabs supported on discrete bearings or columns. The resulting stresses are verified along defined control perimeters around the loaded area. The shear force acts over an area ud_{eff}, where u is the length of the perimeter and d_{eff} is the effective depth of the slab taken as the average of the effective depths in two orthogonal directions.

8.1.2 Design procedure

BS EN 1992-1-1
6.4.3(2)

At the column perimeter, or the perimeter of the loaded area, the maximum punching shear stress should not be exceeded, $v_{Ed} \leq v_{Rd,max}$.

Punching shear reinforcement is not necessary if

$v_{Ed} \leq v_{Rd,c}$

Where v_{Ed} exceeds $v_{Rd,c}$ for the control section considered, punching shear reinforcement should be provided according to Section 8.5

BS EN 1992-1-1
6.4.3(1)

where

v_{Ed} = design value of the applied shear stress (see Sections 8.2 and 8.3)

$v_{Rd,max}$ = design value of the maximum punching shear resistance (see Section 8.6) along the control section considered

$v_{Rd,c}$ = design value of punching shear resistance of a slab **without** punching shear reinforcement, along the control section considered (see Section 8.4)

$v_{Rd,cs}$ = design value of punching shear resistance of a slab **with** punching shear reinforcement, along the control section considered (see Section 8.5)

8.2 Applied shear stress

8.2.1 General

Where the support reaction is eccentric with regard to the control perimeter, the maximum shear stress should be taken as:

BS EN 1992-1-1
6.4.3(3)

$v_{Ed} = \beta V_{Ed}/(u_i d)$

where

d = mean effective depth

= $(d_y + d_z)/2$

u_i = length of the control perimeter under consideration (see Section 8.3)

V_{Ed} = design value of applied shear force

β = factor dealing with eccentricity

d_y = effective depth in y-direction

d_z = effective depth in z-direction

8.2.2 Values of β (conservative values from diagram)

For braced structures, where adjacent spans do not differ by more than 25%, the values of β shown in Figure 8.1 may be used.

BS EN 1992-1-1 6.4.3(6)

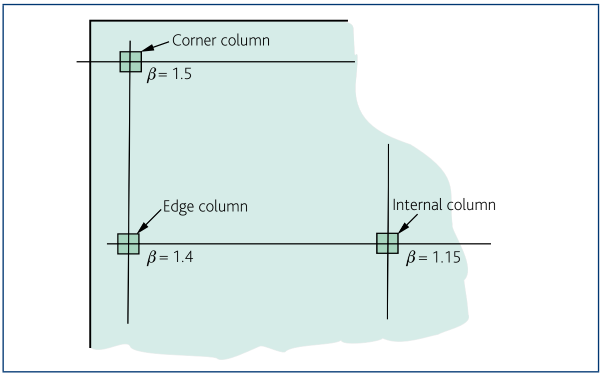


Figure 8.1 **Recommended values for β**

BS EN 1992-1-1 fig. 6.21N & NA

8.2.3 Values of β (using calculation method)

As an alternative to Section 8.2.2, the values of β can be obtained using the following methods.

Internal columns

- For internal rectangular columns with loading eccentric to *one* axis:
  $\beta = 1 + (kM_{Ed}/V_{Ed})(u_1/W_1)$

BS EN 1992-1-1 Exp. (6.39)

where

k = coefficient depending on the ratio of the column dimensions c_1 and c_2 as shown in Figure 8.2 (see Table 8.1)
M_{Ed} = design value of the applied internal bending moment
u_1 = length of the basic control perimeter (see Figure 8.3)
W_1 = a distribution of shear as illustrated in Figure 8.2 and is a function of u_1
W_1 = $c_1^2/2 + c_1c_2 + 4c_2d + 16d^2 + 2\pi dc_1$ for a rectangular column
= $\int_0^{u_i} |e|\, dl$ for the general case

BS EN 1992-1-1 Exp. (6.40)

where

$|\ |$ = the absolute value
e = the distance of dl from the axis about which M_{Ed} acts
dl = a length increment of the perimeter

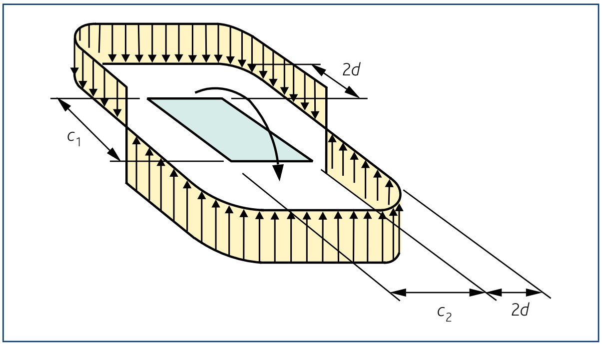


Figure 8.2 **Shear distribution due to an unbalanced moment at a slab/ internal column connection**

BS EN 1992-1-1 fig. 6.19

BS EN 1992-1-1 table 6.1

Table 8.1
Values for *k* for rectangular loaded areas

c_1/c_2	≤ 0.5	1.0	2.0	≥ 3.0
k	0.45	0.60	0.70	0.80

BS EN 1992-1-1 Exp. (6.43)

- For internal rectangular columns with loading eccentric to *both* axes:
  $\beta = 1 + 1.8[(e_y/b_z)^2 + (e_z/b_y)^2]^{0.5}$

  where
  e_y and e_z = M_{Ed}/V_{Ed} along y and z axes respectively
  b_y and b_z = the dimensions of the control perimeter (see Figure 8.3)

BS EN 1992-1-1 Exp. (6.42)

- For internal circular columns:
  $\beta = 1 + 0.6\pi e/(D + 4d)$

  where
  D = diameter of the circular column
  e = M_{Ed}/V_{Ed}

BS EN 1992-1-1 fig. 6.13

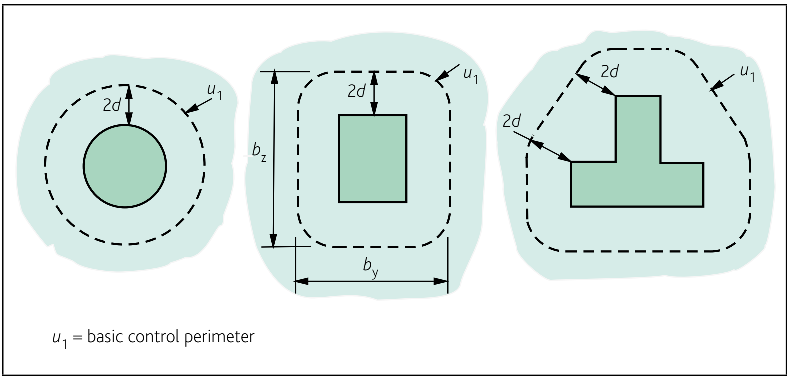


Figure 8.3
Typical basic control perimeters around loaded areas

Edge columns

BS EN 1992-1-1 6.4.3(4)

- For edge columns, with loading eccentricity perpendicular and interior to the slab edge,
  $\beta = u_1/u_{1*}$

  where
  u_1 = basic control perimeter (see Figure 8.4)
  u_{1*} = reduced control perimeter (see Figure 8.5)

BS EN 1992-1-1 Exp. (6.44)

- For edge columns, with eccentricity to both axes and interior to the slab edge
  $\beta = u_1/u_{1*} + ke_{par}u_1/W_1$

  where
  k = coefficient depending on the ratio of the column dimensions c_1 and c_2 as shown in Figure 8.5 (see Table 8.2)
  e_{par} = eccentricity parallel to the slab edge resulting from a moment about an axis perpendicular to the slab edge
  W_1 = $c_2^2/4 + c_1c_2 + 4c_1d + 8d^2 + \pi dc_2$

  where
  c_1 and c_2 are as in Figure 8.5

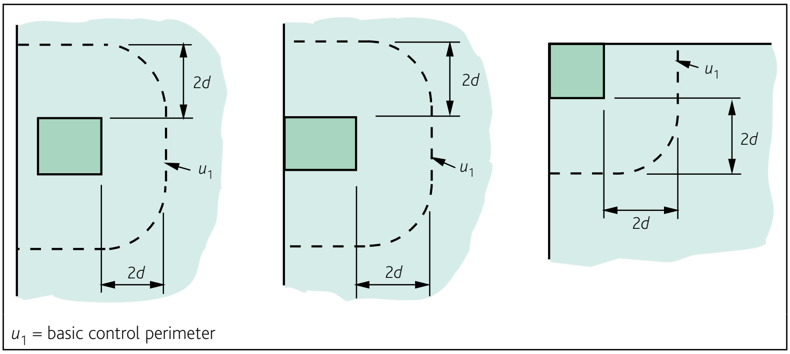


Figure 8.4
Control perimeters for loaded areas at or close to an edge or corner

BS EN 1992-1-1 fig. 6.15

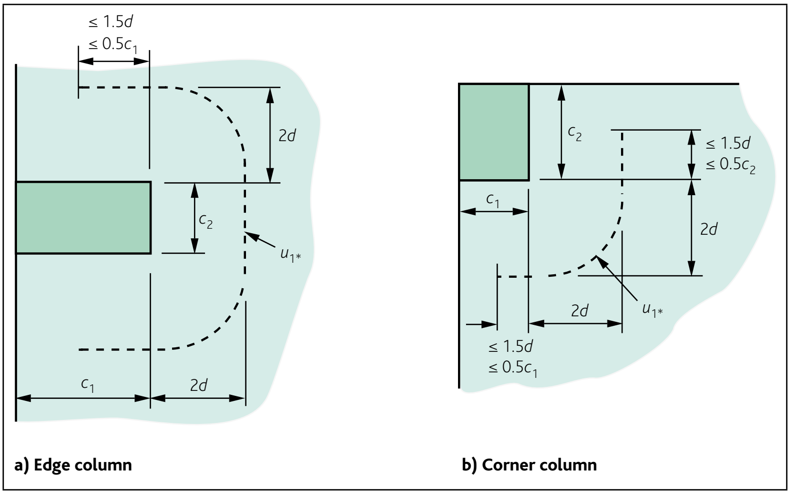


Figure 8.5
Equivalent control perimeter u_{1*}

BS EN 1992-1-1 fig. 6.20

Table 8.2
Values for k for rectangular loaded areas at edge of slabs and subject to eccentric loading in both axes

$c_1/2c_2$*	≤ 0.5	1.0	2.0	≥ 3.0
k	0.45	0.60	0.70	0.80
Note * differs from Table 8.1				

Corner columns

BS EN 1992-1-1 6.4.3(5)

For corner columns with eccentricity towards interior of the slab

BS EN 1992-1-1 Exp. (6.46)

$\beta = u_1/u_{1*}$

where

u_1 = basic control perimeter (see Figure 8.4)
u_{1*} = reduced control perimeter (see Figure 8.5)

Perimeter columns where eccentricity is exterior to slab

BS EN 1992-1-1
6.4.3(4)
6.4.3(5)

For edge and corner columns, where eccentricity is exterior to the slab the expression

$\beta = 1 + (kM_{Ed}/V_{Ed})\,(u_1/W_1\text{i}$

applies as for internal columns above. However, M_{Ed}/V_{Ed} (= e, eccentricity) is measured from the centroid of the control perimeter.

8.3 Control perimeters

8.3.1 Basic control perimeter u_1 (internal columns)

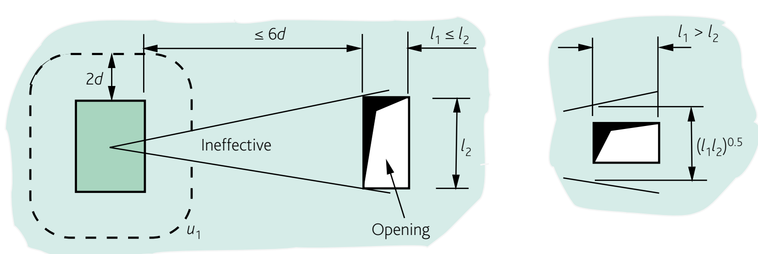
BS EN 1992-1-1
6.4.2

The basic control perimeter u_1 may be taken to be at a distance of 2.0d from the face of the loaded area, constructed so as to minimise its length (see in Figure 8.3).

8.3.2 Openings

BS EN 1992-1-1
fig. 6.14

Where openings in the slab exist within 6d from the face of the loaded area, part of the control perimeter will be ineffective as indicated in Figure 8.6.



a) Where l_1 l_2 **b) Where $l_1 > l_2$**

Figure 8.6
Control perimeter near an opening

8.3.3 Perimeter columns

BS EN 1992-1-1
6.4.2(5)

For edge or corner columns (or loaded areas), the basic control perimeter u_1 shown in Figure 8.4 may be used for concentric loading. This perimeter must not be greater than the perimeter obtained for internal columns from using Figure 8.3 (see Section 8.3.1).

Where eccentricity of loads is interior to the slab, the reduced control perimeter, u_{1*} shown in Figure 8.5 should be used as indicated in Section 8.2.3.

8.3.4 Column heads

BS EN 1992-1-1
6.4.2(9)

Where column heads are provided, distinction should be made between cases where $l_H > 2h_H$ and where $l_H < 2h_H$

where

l_H = projection of head from the column
h_H = height of head below soffit of slab

Where $l_H < 2h_H$ punching shear needs to be checked only in the control section outside the column head (see Figure 8.7). Where $l_H > 2h_H$ the critical sections both within the head and slab should be checked (see Figure 8.8).

BS EN 1992-1-1 fig. 6.17

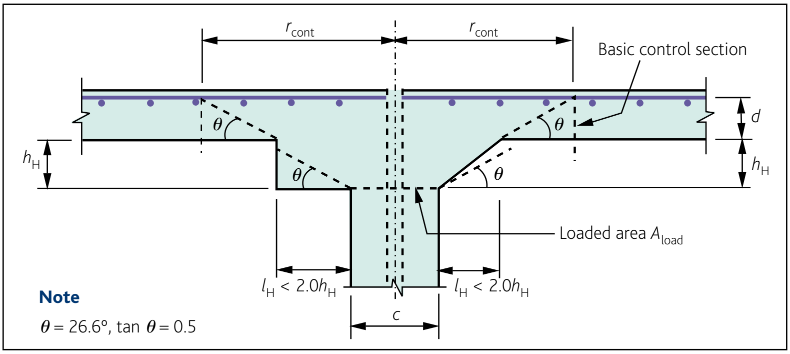


Figure 8.7
Slab with enlarged column head where $l_H < 2.0\ h_H$

BS EN 1992-1-1 fig. 6.18

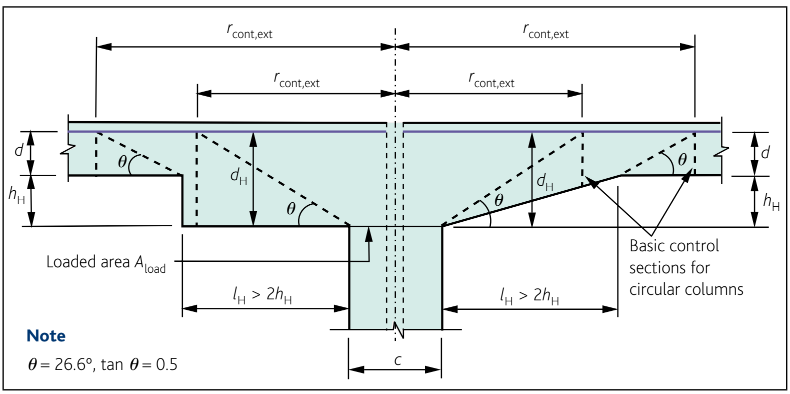


Figure 8.8
Slab with enlarged column head where $l_H > 2h_H$

8.4 Punching shear resistance without shear reinforcement

BS EN 1992-1-1 6.4.4

The punching shear resistance of a slab should be assessed for the basic control section according to Section 8.3. The design punching shear resistance may be calculated as follows:

The design value for the shear resistance is given by:

BS EN 1992-1-1 Exp. (6.47) & NA

$$v_{Rd,c} = (0.18/\gamma_C)k(100\ \rho_l\ f_{ck})^{1/3} + 0.15\ \sigma_{cp}$$
$$\geq\ 0.035k^{1.5}\ f_{ck}^{\ 0.5} + 0.15\,\sigma_{cp}$$

where

$k\ =\ 1 + (200/d)^{0.5} \leq 2.0$ (d in mm)

$\gamma_C = 1.5$

$\rho_l\ =\ (\rho_{ly,}\ \rho_{lz})^{0.5} \leq 0.02$

where

ρ_{ly}, ρ_{lz} = reinforcement ratios in the y and z directions respectively. The values should be calculated as mean values, taking into account a slab width equal to the column width plus $3d$ each side

$\sigma_{cp}\ =\ (\sigma_{cy} + \sigma_{cz})/2$

$\sigma_{cy}\ =\ N_{Ed,y}/A_{cy}$

$\sigma_{cz}\ =\ N_{Ed,z}/A_{cz}$

where

$N_{Ed,y}$, $N_{Ed,z}$ = longitudinal forces across the full bay for internal columns and the longitudinal forces across the control section for edge columns. The force may be from a load or prestressing action

A_C = area of concrete cross-section according to the definition of N_{Ed} (mm^2)

8.5 Punching shear resistance with shear reinforcement

BS EN 1992-1-1 6.4.5

Where shear reinforcement is required it should be calculated in accordance with the Expression below:

BS EN 1992-1-1 Exp. (6.52)

$v_{Rd,cs} = 0.75\ v_{Rd,c} + 1.5\ (d/s_r)A_{sw}\ f_{ywd,ef}\ (1/u_1 d)\sin\ \alpha$

where

A_{sw} = area of one perimeter of shear reinforcement around the column (mm^2) (for $A_{sw,min}$ see Section 15.4.3)

s_r = radial spacing of perimeters of shear reinforcement (mm)

$f_{ywd,ef}$ = effective design strength of reinforcement $(250 + 0.25d) \leq f_{ywd}$

d = mean effective depth in the two orthogonal directions (mm)

u_1 = basic control perimeter at $2d$ from the loaded area (see Figure 8.3)

$\sin\ \alpha$ = 1.0 for vertical shear reinforcement

α = angle between the shear reinforcement and plane of the slab

Assuming vertical reinforcement
$A_{sw} = (v_{Ed} - 0.75\ v_{Rd,c})s_r\ u_1/(1.5\ f_{ywd,ef})$ per perimeter

8.6 Punching shear resistance adjacent to columns

Adjacent to the column the punching shear resistance is limited to a maximum of:

BS EN 1992-1-1 6.4.5(3)

$v_{Ed} = \beta V_{Ed}/u_0 d \leq v_{Rd,max}$

where

β = factor dealing with eccentricity (see Section 8.2)

V_{Ed} = design value of applied shear force

d = mean effective depth

u_0 = length of column periphery for an interior column

= $2(c_1 + c_2)$ for interior columns

= $c_2 + 3d \leq c_2 + 2c_1$ for edge columns

= $3d \leq c_1 + c_2$ for corner columns

where

c_1 = column depth (for edge columns, measured perpendicular to the free edge)

c_2 = column width as illustrated in Figure 8.5

$v_{Rd,max} = 0.5\nu f_{cd}$

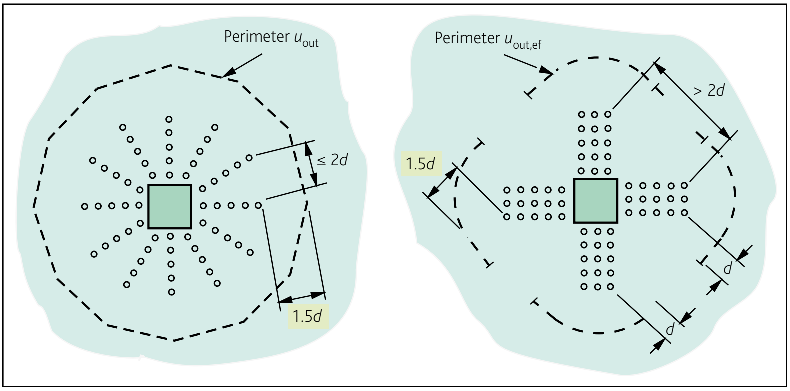
where

ν = $0.6\ [1 - (f_{ck}/250)]$

8.7 Control perimeter where shear reinforcement is no longer required, u_{out}

BS EN 1992-1-1 6.4.5(4) & NA

Shear reinforcement is not required at a perimeter where the shear stress due to the effective shear force does not exceed $v_{Rd,c}$. The outermost perimeter of shear reinforcement should be placed at a distance not greater than $1.5d$ within the perimeter where reinforcement is no longer required. See Figures 8.9, 15.6 and 15.7.



BS EN 1992-1-1 fig. 6.22

Figure 8.9
Control perimeters at internal columns

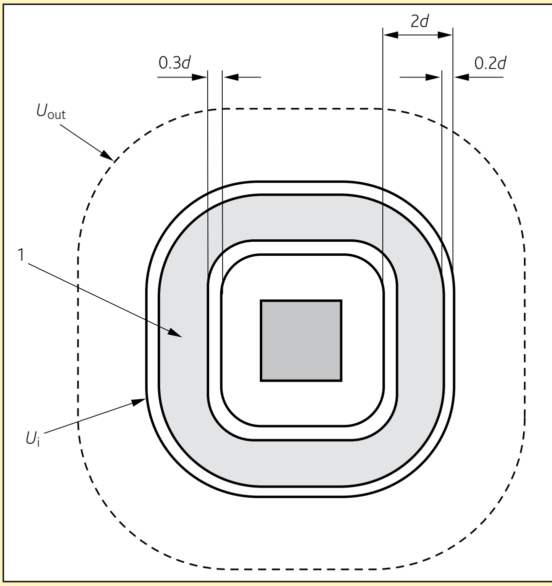
8.8 Distribution of shear reinforcement

The Expression given in Section 8.5 assumes a constant area of shear reinforcement on each perimeter moving away from the loaded area as shown in Figure 8.9. In cases where the reinforcement area varies on successive perimeters, the required shear reinforcement may be determined by checking successive perimeters, u_i between the basic control perimeter and the perimeter u_{out}, to ensure that the shear reinforcement of area ΣA_{sw} satisfies the following expression:

$$\Sigma A_{sw} = \frac{(v_{Ed} - 0.75\, v_{Rd,\,c})\; u_i\, d}{f_{ywd,\,ef}\sin \alpha}$$

PD 6687-2 7.3.2

where ΣA_{sw} is the total shear reinforcement as shown in Figure 8.10, placed within an area enclosed between the control perimeter u_i chosen and one $2d$ inside it, except that shear reinforcement within a distance of $0.3d$ from the inner perimeter and $0.2d$ from the control perimeter should be ignored. Further guidance and background are given by Hendy & Smith [20]



Notes

1 u_i is between u_1 and u_{out}

2 Reinforcement ΣA_{sw} to include include for check on perimeter u_i

PD 6687-2 fig. 4

Figure 8.10
Reinforcement to include in punching shear check

8.9 Punching shear resistance of foundation bases

BS EN 1992-1-1
6.4.2(6)

In addition to the verification at the basic control perimeter at $2d$ from the face of the column, perimeters within the basic perimeter should also be checked for punching resistance. In cases where the depth of the base varies, the effective depth of the base may be assumed to be that at the perimeter of the loaded area. See Figure 8.11.

For concentric loading the net applied force is

BS EN 1992-1-1
6.4.4(2)

$$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$$

where

V_{Ed} = applied shear force

ΔV_{Ed} = net upward force within the perimeter considered, i.e. upward pressure from soil minus self-weight of base.

When a column transmits an axial load V_{Ed} and a moment M_{Ed}, the punching shear stress is given by the following expression:

BS EN 1992-1-1
Exp. (6.51)

$$v_{Ed} = (V_{Ed,red}/ud)[1 + kM_{Ed}u/(V_{Ed,red}W)]$$

where

u = the perimeter being considered

k = a coefficient depending on the ratio of the column dimensions shown in Figure 8.2 and Table 8.1

W = W_1

For a square column and a general perimeter at r_i with length u_i:

PD 6687-2
7.3.1

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 2c_2 r_i + 4r_i^2 + \pi r_i c_1$$

The punching shear resistance $v_{Rd,c}$ given in Section 8.4 may be enhanced for column bases by multiplying the Expressions by $2d/a$, where a is the distance of the perimeter considered from the periphery of the column.

BS EN 1992-1-1
fig. 6.16

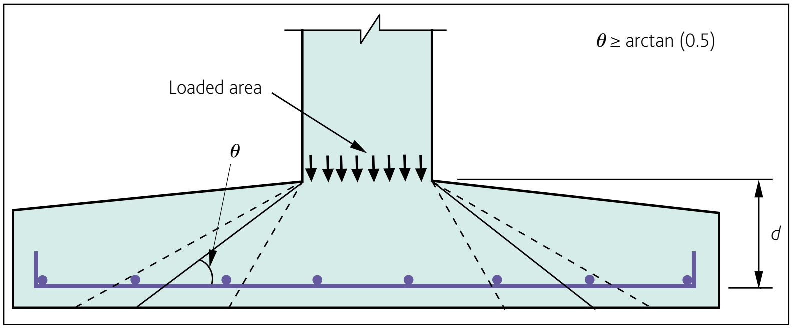


Figure 8.11
Depth of control section in a footing with variable depth

9 Torsion

9.1 General

Torsional resistance should be verified in elements that rely on torsion for static equilibrium. In statically indeterminate building structures in which torsion arises from consideration of compatibility and the structure is not dependent on torsion for stability, it will normally be sufficient to rely on detailing rules for minimum reinforcement to safeguard against excessive cracking, without the explicit consideration of torsion at ULS.

BS EN 1992-1-1
6.3.1

In Eurocode 2, torsional resistance is calculated by modelling all sections as equivalent thin-walled sections. Complex sections, such as T-sections are divided into a series of sub-sections and the total resistance is taken as the sum of the resistances of the individual thin-walled sub-sections.

BS EN 1992-1-1
6.3.2

The same strut inclination θ should be used for modelling shear and torsion. The limits for cot θ noted in Section 7 for shear also apply to torsion.

9.2 Torsional resistances

BS EN 1992-2
6.3.2 (104)

The design torsional resistance moment

$T_{Rd,max} = 2 \nu \alpha_{cw} f_{cd} A_k t_{ef,i} \sin \theta \cos \theta$

where

ν = $0.6 [1 - (f_{ck}/250)]$

A_k = area enclosed by the centre lines of connecting walls including the inner hollow area (see Figure 9.1)

$t_{ef,i}$ = effective wall thickness (see Figure 9.1). It may be taken as A/u but should not be taken as less than twice the distance between edge (the outside face of the member) and centre of the longitudinal reinforcement. For hollow sections the real thickness is an upper limit

θ = angle of the compression strut

α_{cw} = coefficient taking account of the state of stress in the compression chord (see Table 7.2)

= 1 for non-prestressed structures

= $(1 + \sigma_{cp}/f_{cd})$ for $0 < \sigma_{cp} \leq 0.25 f_{cd}$

= 1.25 for $0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd}$

= $2.5 (1 - \sigma_{cp}/f_{cd})$ for $0.5 f_{cd} < \sigma_{cp} < 1.0 f_{cd}$

BS EN 1992-2
6.2.3(103) & NA

where

σ_{cp} = mean compressive stress, measured positive, in the concrete due to the design axial force.

σ_{cp} should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than 0.5d cot θ from the edge of the support.

BS EN 1992-1-1 fig. 6.11

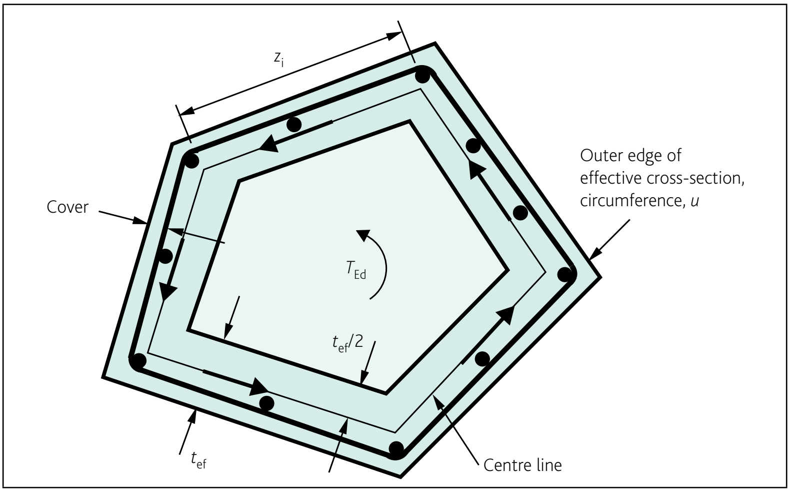


Figure 9.1
Notations used in Section 9

The torsional resistance of a solid rectangular section with shear reinforcement on the outer periphery $T_{Rd,max}$ may be deduced from the general expression, assuming $t_{eff} = A/u$:

$$T_{Rd,max} = 2\nu\alpha_{cw} f_{cd}\, k_2 b^3 \sin\theta \cos\theta$$

where

k_2 = coefficient obtained from Table 9.1
b = breadth of the section (< h, depth of section)

Table 9.1
Values of k_2

h/b	1	2	3	4
k_2	0.141	0.367	0.624	0.864

ENV 1992-1-1 Exp. (4.43)[21]

Torsional resistance governed by the area of closed links is given by:

$$T_{Rd} = A_{sw}\, 2A_k \cot\theta\, f_{ywd}/s$$

where

A_{sw} = area of link reinforcement
$f_{yw,d}$ = design strength of the link reinforcement
s = spacing of links

BS EN 1992-2 Exp. (6.28)

The required area of longitudinal reinforcement for torsion, ΣA_{sl}, may be calculated from:

$$\Sigma A_{sl} f_{yd} = T_{Ed} u_k \cot\theta/(2A_k)$$

where

T_{Ed} = applied design torsion
u_k = perimeter of the area A_k

BS EN 1992-2 6.3.2(103)

In compressive chords, the longitudinal reinforcement may be reduced in proportion to the available compressive force. In tensile chords, the longitudinal reinforcement for torsion should be added to the other reinforcement. The longitudinal reinforcement should generally be distributed over the length of side z_i, (side length of wall i defined by the distance between the intersection points with the adjacent walls), but for smaller sections it may be concentrated at the ends of this length.

Bonded prestressing tendons can be taken into account limiting their stress increase to $\Delta\sigma_p \leq 500$ MPa. In that case, $\Sigma A_{sl} f_{yd}$ in Expression (6.28) is replaced by $\Sigma A_{sl} f_{yd} + A_p\, \Delta\sigma_p$.

9.3 Combined torsion and shear

In solid sections the following relationship should be satisfied:

BS EN 1992-2 Exp. (6.29)

$(T_{Ed} / T_{Rd,max}) + (V_{Ed}/V_{Rd,\ max}) \leq 1.0$

where

$T_{Rd,max}$ = design torsional resistance moment
= $2 \nu \alpha_{cw} f_{cd} A_k t_{ef,i} \sin\theta \cos\theta$ as in Section 9.2.

$V_{Rd,max}$ = maximum design shear resistance
= $\alpha_{cw} b_w z \nu_1 f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$ as in Section 7.3.2.

The effects of torsion and shear for both hollow and solid members may be superimposed, assuming the same value for the strut inclination θ. The limits for θ given in Section 7.3.2 are also fully applicable for the case of combined shear and torsion.

BS EN 1992-2 6.3.2 (102)

For box sections, each wall should be verified separately, for the combination of shear forces derived from shear and torsion (Figure 9.2).

BS EN 1992-2 fig. 6.104

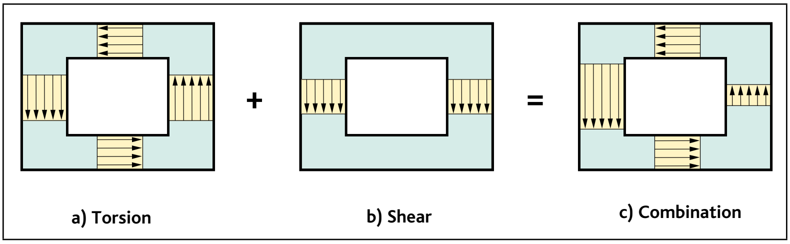


Figure 9.2
Internal actions combination within the different walls of a box section

10 Strut-and-tie models, bearing zones and partially loaded areas

10.1 Design with strut-and-tie models

10.1.1 Struts

BS EN 1992-1-1
6.5.2

The design strength for a concrete strut in a region with transverse compressive stress or no transverse stress may be calculated from the following Expression (see Figure 10.1).

$\sigma_{Rd,max} = f_{cd} = 0.85\, f_{ck} / 1.5$

BS EN 1992-1-1
fig. 6.23

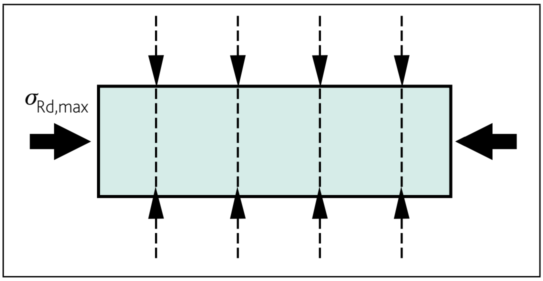


Figure 10.1
Design strength of concrete struts without transverse tension

BS EN 1992-1-1
fig. 6.24

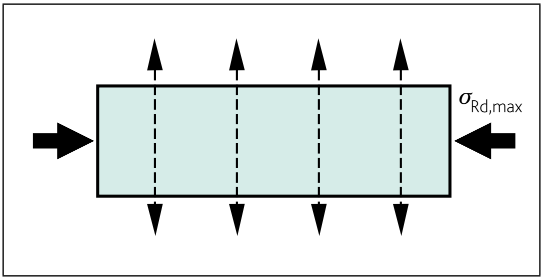


Figure 10.2
Design strength of concrete struts with transverse tension

The design strength for concrete struts should be reduced in cracked compression zones and, unless a more rigorous approach is used, may be calculated from the following Expression (see Figure 10.2).

$\sigma_{Rd,max} = 0.6\,(1 - f_{ck}/250)\, f_{cd}$

where

$f_{cd} = 1.0\ f_{ck} / 1.5$

10.1.2 Ties

BS EN 1992-1-1
6.5.3

Reinforcement required to resist the forces at the concentrated nodes may be smeared over a length (see Figure 10.3). When the reinforcement in the node area extends over a considerable length of an element, the reinforcement should be distributed over the length where the compression trajectories are curved (ties and struts). The tensile force, T, may be obtained by:

- For partial discontinuity regions ($b \leq H/2$)
  $T = (b - a)F/4b$
- For full discontinuity regions ($b > H/2$)
  $T = (1 - 0.7a/h)\,F/4$

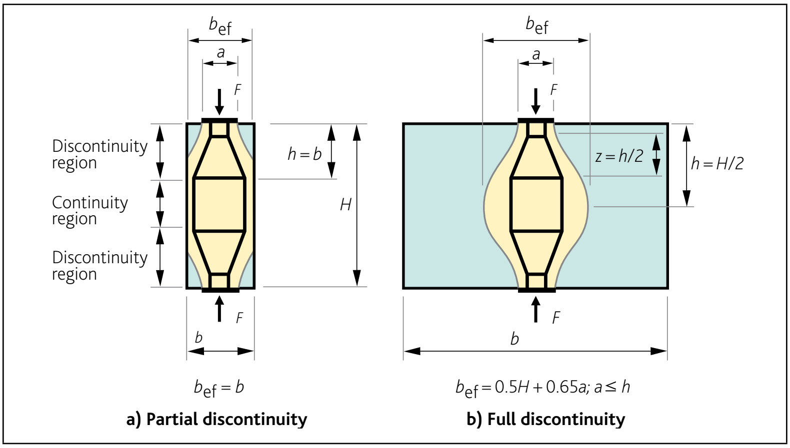


Figure 10.3
Parameters for the determination of transverse tensile forces in a compression field with smeared reinforcement

BS EN 1992-1-1 fig 6.25

10.1.3 Nodes

BS EN 1992-1-1 6.5.4

The rules for nodes also apply to regions where concentrated forces are transferred in a member and which are not designed by the strut-and-tie method. The forces acting at nodes should be in equilibrium. Transverse tensile forces perpendicular to an inplane node should be considered.

The dimensioning and detailing of concentrated nodes are critical in determining their loadbearing resistance. Concentrated nodes may develop, for example, where point loads are applied, at supports, in anchorage zones with concentration of reinforcement or prestressing tendons, at bends in reinforcing bars, and at connections and corners of members.

The design values for the compressive stresses within nodes may be determined in three ways:

- In compression nodes where no ties are anchored at the node (see Figure 10.4)

$\sigma_{Rd,max} = 1.0 \;\; (1 - f_{ck}/250)\, f_{cd}$

where

$\sigma_{Rd,max}$ = maximum stress which can be applied at the edges of the node
f_{cd} = $0.85\; f_{ck}/1.5$

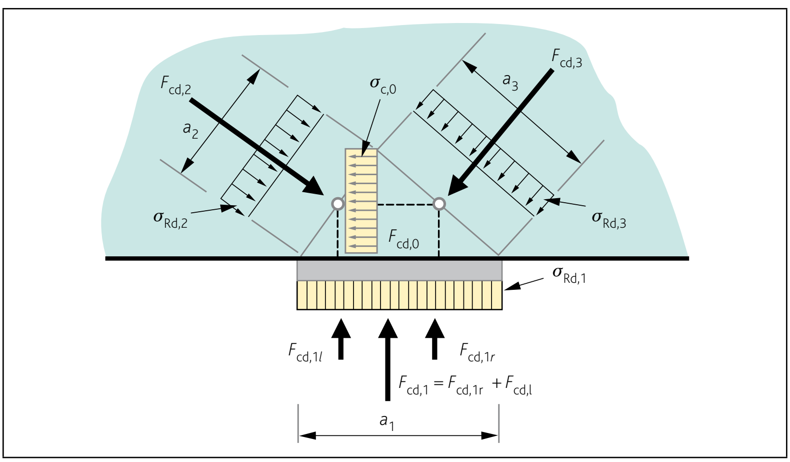


Figure 10.4
Compression node without ties

BS EN 1992-1-1 fig. 6.26

- In compression–tension nodes with anchored ties provided in one direction (see Figure 10.5)

  $\sigma_{Rd,max} = 0.85 \ (1 - f_{ck}/250) \ f_{cd}$

  where

  $\sigma_{Rd,max}$ is the maximum of $\sigma_{Rd,1}$ and $\sigma_{Rd,2}$

  $f_{cd} = 0.85 \ f_{ck}/1.5$

BS EN 1992-1-1 fig. 6.27

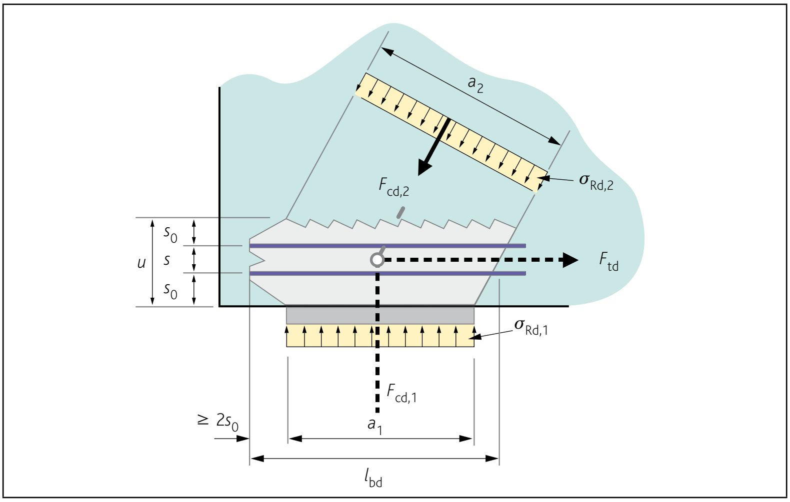


Figure 10.5
Compression–tension node with reinforcement provided in one direction

- In compression–tension nodes with anchored ties provided in more than one direction (see Figure 10.6),

  $\sigma_{Rd,max} = 0.75 \ (1 - f_{ck}/250) \ f_{cd}$

  where

  $f_{cd} = 0.85 \ f_{ck}/1.5$

BS EN 1992-1-1 fig. 6.28

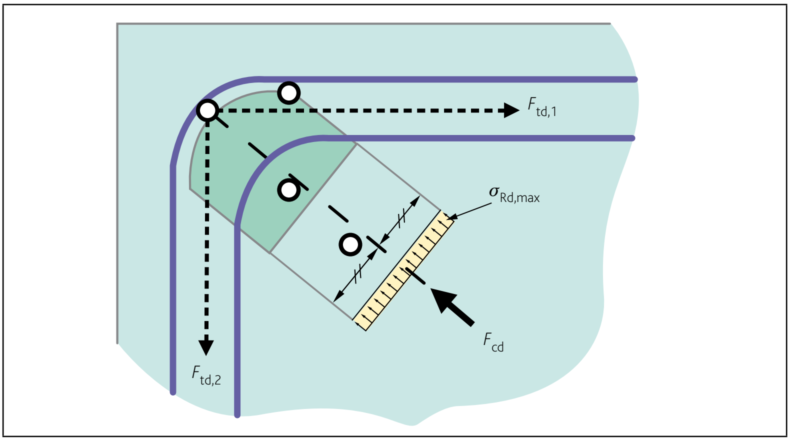


Figure 10.6
Compression–tension node with reinforcement provided in two directions

The anchorage of the reinforcement in compression-tension nodes starts at the beginning of the node; in the case of a support anchorage, starting at its inner face (see Figure 10.5). The anchorage length should extend over the entire node length. In certain cases, the reinforcement may also be anchored behind the node.

10.2 Partially loaded areas

BS EN 1992-1-1
6.7

For a uniform distribution of load on an area A_{c0} (see Figure 10.7) the concentrated resistance force may be determined as follows:

$$F_{Rdu} = A_{c0} f_{cd} (A_{c1} / A_{c0})^{0.5} \leq 3.0 f_{cd} A_{c0}$$

where

A_{c0} = loaded area,
A_{c1} = maximum design distribution area with a similar shape to A_{c0}
f_{cd} = 0.85 f_{ck} / 1.5

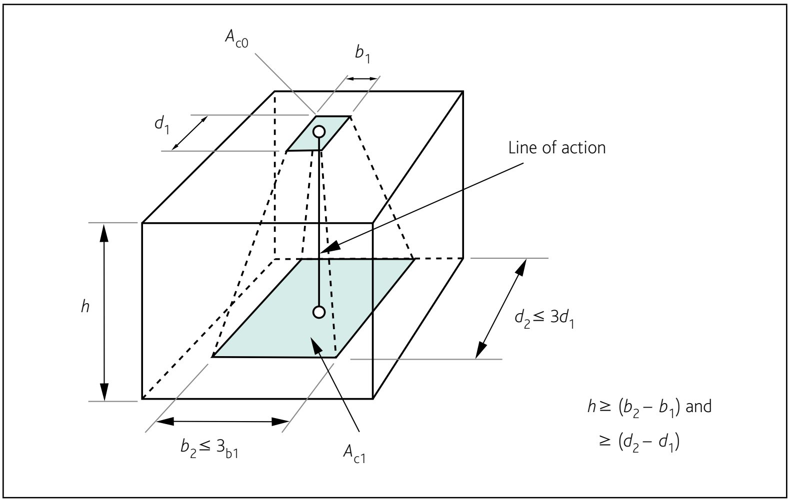
The design distribution area A_{c1} required for the resistance force F_{Rdu} should correspond to the following conditions:

- The height for the load distribution in the load direction should correspond to the conditions given in Figure 10.7.
- The centre of the design distribution area A_{c1} should be on the line of action passing through the centre of the load area A_{c0}.
- If there is more than one compression force acting on the concrete cross-section, the designed distribution areas should not overlap.

Reinforcement should be provided to resist the tensile forces due to the effect of actions, the rules for strut and tie may be used.

Transverse tensile forces can arise in the localised area near a concentrated load and also from any further spread of load outside the localised area. The strut-and-tie model used should account for both of these potential sources of transverse tensile forces Further guidance and background are given by Hendy & Smith[20].

PD 6687-2
7.5



BS EN 1992-1-1
fig. 6.29

Figure 10.7
Design distribution for partially loaded areas

10.3 Bearing zones of bridges

For bearing zones of bridges, Section 10.2 for partially loaded areas should be used, noting the following.

BS EN 1992-2 J.104.1 (103)

For concrete classes equal to or higher than C55/67, the concentrated resistance force should be calculated using the following Expression:

$$F_{Rdu} = A_{c0} f_{cd} (0.46 f_{ck}^{\ 2/3}) / (1 + 0.1 f_{ck}) \times (A_{c1} / A_{c0})^{0.5} \leq 3.0 A_{c0} f_{cd} (0.46 f_{ck}) / (1 + 0.1 f_{ck}^{\ 2/3})$$

BS EN 1992-2 J.104.1 (102)

The distance from the edge of the loaded area to the free edge of the concrete section should not be less than 1/6 of the corresponding dimension of the loaded area measured in the same direction. In no case should the distance to the free edge be taken as less than 50 mm.

BS EN 1992-2 J.104.1 (104)

In order to avoid edge sliding, uniformly distributed reinforcement parallel to the loaded face should be provided to the point at which local compressive stresses are dispersed. This point is determined by drawing a line at an angle, θ (30°), to the direction of load application from the edge of the section to intersect with the opposite edge of the loaded surface, as shown in Figure 10.8. The reinforcement provided to avoid edge sliding should be adequately anchored.

BS EN 1992-2 fig. J.107

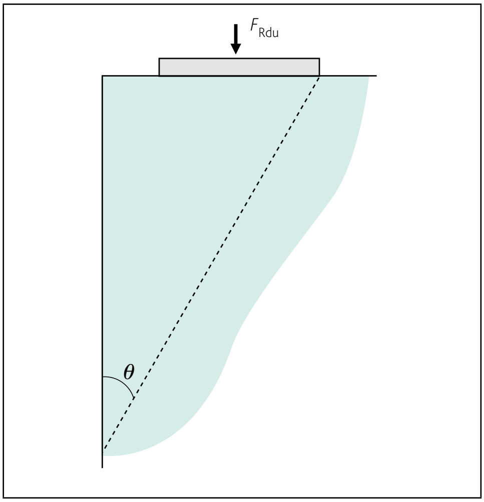


Figure 10.8
Edge sliding mechanism

The reinforcement provided in order to avoid edge sliding (A_r) should be calculated in accordance with the expression $A_r f_{yd} \geq F_{Rdu}/2$.

11 Prestressed members and structures

11.1 General

The section provides guidance that is specific to prestressed concrete using stressed tendons. In general, prestress is introduced in the action combinations defined in BS EN 1990 as part of the loading cases and its effects should be included in the applied internal moment and axial force.

The contribution of the prestressing tendons to the resistance of the section should be limited to their additional strength beyond prestressing. This requirement is most simply achieved by treating the secondary effects of prestress as an applied action (i.e. a load), whilst omitting the primary effects of the design prestressing force. This may be calculated assuming that the origin of the stress–strain relationship of the tendons is displaced by the effects of prestressing.

11.2 Brittle fracture

Brittle failure of the member caused by failure of prestressing tendons should be avoided by using one of three methods given below.

BS EN 1992-2 6.1(109)

11.2.1 Verification

Verify the load capacity using a reduced area of prestress as follows:

- Calculate the applied bending moment due to the frequent combination of actions M_{freq}.
- Determine the reduced area of prestress, $A_{p,Red}$, that results in the tensile stress reaching f_{ctm} at the extreme tension fibre when the section is subject to the bending moment calculated above.

$$A_{p,Red} = (M_{freq} / Z - f_{ctm}) / [\sigma_p (1/A_c + e / Z)]$$

where

Z = section modulus

f_{ctm} = mean value of concrete cylinder compressive strength

σ_p = stress in tendons just prior to cracking

A_c = area of concrete

e = eccentricity of the tendons

Further guidance is given by Hendy & Smith[20]

- Using this reduced area of prestress, calculate the ultimate flexural capacity. It should be ensured that this exceeds the bending moment due to the frequent combination. Redistribution of internal actions within the structure may be taken into account for this verification and the ultimate bending resistance should be calculated using the material partial factors for accidental design situations given in Table 2.9.

Assuming there is no conventional reinforcement then:

$$M_{freq} \geq (f_{ctm}\, z_s) / [(z_s / Z) - [\sigma_p (1/A_c + e / Z)/ f_{p0.1k}]]$$

where

z_s = lever arm for the prestress, where $A_{p,Red}$ has been used to determine the accidental ultimate limit state.

f_{ctm} = mean value of concrete cylinder compressive strength

σ_p = stress in tendons just prior to cracking

A_c = area of concrete
e = eccentricity of the tendons

Further guidance is given by Hendy & Smith[20]

11.2.2 Minimum provision

The minimum reinforcement area, $A_{s,min}$, is given below. Reinforcing steel provided for other purposes may be included in $A_{s,min}$.

BS EN 1992-2
Exp. (6.101a)

$$A_{s,min} = M_{rep} / (z_s f_{yk})$$

where

M_{rep} = cracking bending moment calculated using an appropriate tensile strength, $f_{ctk,0.05}$ at the extreme tension fibre of the section, ignoring any effect of prestressing. At the joint of segmental precast elements M_{rep} should be assumed to be zero

z_s = lever arm at the ultimate limit state related to the reinforcing steel

BS EN 1992-2
6.1(110)

This minimum reinforcement steel area should be provided in regions where tensile stresses occur in the concrete under the characteristic combination of actions. In this check the secondary effects of prestressing should be considered and the primary effects should be ignored.

For pretensioned members, Expression (6.101a) should be applied using one of the alternative approaches described below:

- Tendons with concrete cover at least 1.0 times the minimum specified in Figure 4.1 are considered as effective in $A_{s,min}$. A value of z_s based on the effective strands is used in the expression and f_{yk} is replaced with $f_{p0.1k}$.
- Tendons subject to stresses lower than 0.6 f_{pk} after losses under the characteristic combination of actions are considered as fully active. In this case the following requirement should be met.

BS EN 1992-2
Exp. (6.101b)

$$A_{s,min}\, f_{yk} + A_p\, \Delta\sigma_p \geq M_{rep} / z$$

where

$\Delta\sigma_p$ = smaller of 0.4 f_{ptk} and 500 MPa

- To ensure adequate ductility the minimum reinforcing steel area $A_{s,min}$, in continuous beams should extend to the intermediate support of the span considered. However this extension is not necessary if, at the ultimate limit state, the resisting tensile capacity provided by reinforcing and prestressing steel above the supports, calculated with the characteristic strength f_{yk} and $f_{p0.1k}$ respectively, is less than the resisting compressive capacity of the bottom flange. This means that the failure of the compressive zone is not likely to occur, and can be assessed as follows:

BS EN 1992-2
Exp. (6.102)

$$A_s f_{yk} + 1.0\, A_p f_{p0,1k} < t_{inf}\, b_0\, 1.0\, f_{ck}$$

where

t_{inf} = the thickness of the bottom flange of the section. In case of T-sections, t_{inf} is taken as equal to b_0
b_0 = the width of the bottom flange of the section
A_s = the area of reinforced steel in the tensile zone at the ultimate limit state
A_p = the area of prestressing steel in the tensile zone at the ultimate limit state

11.2.3 Inspection

Agreement of an appropriate inspection regime with the relevant national authority on the basis of satisfactory evidence.

11.3 Prestressing force during tensioning

11.3.1 Maximum stressing force

The force applied to a tendon, P_{max} (i.e. the force at the active end during tensioning) shall not exceed the following value:

$P_{max} = A_p \; \sigma_{p,max}$

where

A_p = cross-sectional area of the tendon

$\sigma_{p,max}$ = maximum stress applied to the tendon
= MIN{$0.8\, f_{pk}$; $0.9\, f_{p0,1k}$}

BS EN 1992-1-1 5.10.2.1

Overstressing is permitted if the force in the jack can be measured to an accuracy of ± 5 % of the final value of the prestressing force. In such cases the maximum prestressing force P_{max} may be increased to $0.95\, f_{p0.1k}\, A_p$ (e.g. for the occurrence of unexpectedly high friction in long-line pretensioning). For pre-tensioning this relaxation is intended to be used only where there are unforeseen problems during construction.

The concrete compressive stress in the structure resulting from the prestressing force and other loads acting at the time of tensioning or release of prestress, should be limited to:

$\sigma_c \leq 0.6\, f_{ck}(t)$

where

$f_{ck}(t)$ = characteristic compressive strength of the concrete at time t when it is subjected to the prestressing force

BS EN 1992-1-1 5.10.2.2(5)

For pretensioned elements the stress at the time of transfer of prestress may be increased to $0.7\, f_{ck}(t)$, if it can be justified by tests or experience that longitudinal cracking is prevented.

If the compressive stress permanently exceeds $0.45\, f_{ck}(t)$ the non-linearity of creep should be taken into account.

11.3.2 Prestress force

The value of the initial prestress force $P_{m0}(x)$ (at time $t = t_0$) applied to the concrete immediately after tensioning and anchoring (post-tensioning) or after transfer of prestressing (pre-tensioning) is obtained by subtracting from the force at tensioning P_{max} the immediate losses $\Delta P_i(x)$ and should not exceed the following value:

$P_{m0}(x) = A_p \; \sigma_{pm0}(x)$

where

$\sigma_{pm0}(x)$ = stress in the tendon immediately after tensioning or transfer
= MIN{$0.75\, f_{pk}$; $0.85\, f_{p0,1k}$}

BS EN 1992-1-1 5.10.3(2)

At a given time t and distance x (or arc length) from the active end of the tendon the mean prestress force $P_{m,t}(x)$ is equal to the maximum force P_{max} imposed at the active end, minus the immediate losses and the time dependent losses:

$P_{m,t}(x) = P_{m0}(x) - \Delta P_{c+s+r}(x)$.

where

$P_{m,t}(x)$ = mean value of the prestress force at the time $t > t_0$ and should be determined with respect to the prestressing method

$\Delta P_{c+s+r}(x)$ = change in prestress due to the result of creep, shrinkage of the concrete and the long term relaxation of the prestressing steel

BS EN 1992-1-1 5.10.3(1)&(4)

Absolute values are considered for all the losses.

BS EN 1992-1-1
5.10.3(3)

11.3.3 Immediate losses

When determining the immediate losses $\Delta P_i(x)$ the following immediate influences should be considered for pre-tensioning and post-tensioning where relevant:

- Losses due to elastic deformation of concrete, ΔP_{el}.

For pre-tensioning:

$$\Delta P_{el}(x) = A_p E_p \sigma_c(x) / E_{cm}(t)$$

where

E_p = modulus of elasticity of prestressing steel
$E_{cm}(t)$ = modulus of elasticity of concrete at time, t
$\sigma_c(x)$ = stress in the concrete adjacent to the tendon at transfer

BS EN 1992-1-1
5.10.5.1(2)

For post-tensioning:

$$\Delta P_{el} = A_p E_p \Sigma (j \Delta\sigma_c(t) / E_{cm}(t))$$

where

$\Delta\sigma_c(t)$ = variation of stress in the concrete at the centre of gravity of the tendons applied at time t
j = $(n-1)/2n$ when considering losses before stressing. As an approximation j may be taken as 1/2
= 1 for the variations due to permanent actions applied after prestressing
n = number of identical tendons successively prestressed.

- Losses due to short term relaxation, ΔP_r (e.g. loss due to relaxation of the prestressing during the period which elapses between the tensioning of the tendons and prestressing of the concrete).

BS EN 1992-1-1
5.10.5.2(1)

- Losses due to friction, $\Delta P_\mu(x)$, in post-tensioned tendons may be estimated from:

$$\Delta P_\mu(x) = P_{max}(1 - e^{-\mu(\theta + kx)})$$

where

θ = sum of the angular displacements over a distance x (irrespective of direction or sign)
μ = coefficient of friction between the tendon and its duct
k = unintentional angular displacement for internal tendons (per unit length)
x = distance along the tendon from the point where the prestressing force is equal to P_{max} (the force at the active end during tensioning)

BS EN 1992-1-1
5.10.5.2(2) & (3)

The values μ and k are given in the relevant European Technical Approval. In the absence of data given in a European Technical Approval the values for μ given in Table 11.1 may be assumed. Unintended angular displacements for internal tendons will generally be in the range $0.005 < k < 0.01$ per metre.

BS EN 1992-1-1
table 5.1

Table 11.1
Coefficients of friction, μ, for post-tensioned internal tendons and external unbonded tendons

Tendon type	Internal tendons[a]	External unbonded tendons			
		Steel duct: non-lubricated	HDPE duct: non-lubricated	Steel duct: lubricated	HDPE duct: lubricated
Cold drawn wire	0.17	0.25	0.14	0.18	0.12
Strand	0.19	0.24	0.12	0.16	0.10
Deformed bar	0.65	–	–	–	–
Smooth round bar	0.33	–	–	–	–

Key
a For tendons which fill about half of the duct

- Losses due to anchorage slip, ΔP_{sl}, which are available from the European Technical Approval.
- Losses due to friction at the bends (in the case of curved wires or strands).

11.3.4 Time-dependent losses of prestress

BS EN 1992-1-1 5.10.6

A simplified method to evaluate time dependent losses at location x under the permanent loads is as follows:

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \dfrac{E_p}{E_{cm}} \varphi(t,t_0)\sigma_{c,QP}}{1 + \dfrac{E_p A_p}{E_{cm} A_c} \left(1 + \dfrac{A_c}{I_c} z_{cp}{}^2\right)\left[1 + 0.8\,\varphi(t,t_0)\right]}$$

where

A_p = area of all the prestressing tendons at the location x
$\Delta\sigma_{p,c+s+r}$ = absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t
ε_{cs} = estimated shrinkage strain, in absolute value
E_p = modulus of elasticity for the prestressing steel
E_{cm} = modulus of elasticity for the concrete
$\Delta\sigma_{pr}$ = absolute value of the variation of stress in the tendons at location x, at time t, due to the relaxation of the prestressing steel. It is determined for a stress of $\sigma_p = \sigma_p(G + P_{m0} + \psi_2 Q)$, which is the initial stress in the tendons due to initial prestress and quasi-permanent actions
$\varphi(t,t_0)$ = creep coefficient at a time t and load application at time t_0
$\sigma_{c,QP}$ = stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant. The value of $\sigma_{c,QP}$ may be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of actions ($\sigma_c(G + P_{m0} + \psi_2 Q)$), depending on the stage of construction considered
A_c = area of the concrete section
I_c = second moment of area of the concrete section
z_{cp} = distance between the centre of gravity of the concrete section and the tendons

Compressive stresses and the corresponding strains should be used with a positive sign. This Expression applies for bonded tendons when local values of stresses are used and for unbonded tendons when mean values of stresses are used. The mean values should be calculated between straight sections limited by the idealised deviation points for external tendons or along the entire length in case of internal tendons.

11.3.5 Effects of prestressing at ultimate limit state

BS EN 1992-1-1 5.10.8 & NA

In general the design value of the prestressing force may be determined from $P_{d,t}(x) = \gamma_{P'} P_{m,t}(x)$

For prestressed members with permanently unbonded tendons, it is generally necessary to take the deformation of the whole member into account when calculating the increase of the stress in the prestressing steel. If no detailed calculation is made, it may be assumed that the increase of the stress from the effective prestress to the stress in the ultimate limit state is

100 MPa unless the tendon is outwith β_d from the tension face, in which case $\Delta\sigma_{p,ULS} = 0$.
β = 0.1 for $d \geq 1000$ mm
= 0.25 for $d \leq 500$ mm
The value of β may be interpolated for the values of d between 500 mm and 1000 mm.

BS EN 1992-2 5.10.8 (103)

If the stress increase in external tendons is calculated using the deformation state of the overall member, non-linear analysis should be used.

12 Fatigue

12.1 Verification conditions

PD 6687-2
7.6.1

Because of the high live load to dead load ratio, deck slabs are likely to be amongst the elements most affected by fatigue calculations. However, tests show that the actual stress ranges in the reinforcement in these are much lower than the conventional elastic calculations suggest. Because of this, the NA to BS EN 1992-2 identifies cases where fatigue assessment is not required and provides conservative rules.

BS EN 1992-2
6.8.1(102)

A fatigue verification is generally not necessary for the following structures and structural elements:

- Footbridges, with the exception of structural components very sensitive to wind action.
- Buried arch and frame structures with a minimum earth cover of 1 m and 1.5 m for road and railway bridges.
- Foundations.
- Piers and columns which are not rigidly connected to superstructures.
- Retaining walls of embankments for roads and railways.
- Abutments of road and railway bridges which are not rigidly connected to superstructures, except the slabs of hollow abutments.
- Prestressing and reinforcing steel, in regions where, under the frequent load combination of actions and P_k only compressive stresses occur at the extreme concrete fibres.

BS EN 1992-2
NA 6.8.1(102)

Fatigue verification for road bridges is not necessary for the local effects of wheel loads applied directly to a slab spanning between beams or webs provided that:

- The slab does not contain welded reinforcement or reinforcement couplers.
- The clear span to overall depth ratio of the slab does not exceed 18.
- The slab acts compositely with its supporting beams or webs.
- Either:
  - the slab also acts compositely with transverse diaphragms; or
  - the width of the slab perpendicular to its span exceeds three times its clear span.

BS EN 1992-1-1
6.8.2

12.2 Internal forces and stresses for fatigue verification

The stress calculation shall be based on the assumption of cracked cross-sections neglecting the tensile strength of concrete but satisfying compatibility of strains.

The effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account by increasing the stress range in the reinforcing steel calculated under the assumption of perfect bond by the factor, η, given by

$$\eta = \frac{A_s + A_p}{A_s + A_p \sqrt{\xi\left(\phi_s/\phi_p\right)}}$$

where:

A_s = area of reinforcing steel
A_p = area of prestressing tendon or tendons
ϕ_s = largest diameter of reinforcement
ϕ_p = diameter or equivalent diameter of prestressing steel
= $1.6 \sqrt{A_p}$ for bundles
= 1.75 ϕ_{wire} for single 7-wire strands where ϕ_{wire} is the wire diameter
= 1.20 ϕ_{wire} for single 3-wire strands where ϕ_{wire} is the wire diameter
ξ = ratio of bond strength between bonded tendons and ribbed steel in concrete. The value is subject to the relevant European Technical Approval. In the absence of this the values given in Table 12.1 may be used.

Table 12.1
Ratio of bond strength, ξ, between tendons and reinforcing steel

BS EN 1992-1-1 table 6.2

Prestressing steel	ξ		
	Pre-tensioned	Bonded, post-tensioned	
		≤ C50/60	≥ C70/85
Smooth bars and wires	Not applicable	0.3	0.15
Strands	0.6	0.5	0.25
Indented wires	0.7	0.6	0.30
Ribbed bars	0.8	0.7	0.35

Note
For intermediate values between C50/60 and C70/85 interpolation may be used.

In the design of the shear reinforcement the inclination of the compressive struts θ_{fat} may be calculated using a strut-and-tie model or in accordance with

where

θ = angle of compression struts to the beam axis assumed in ULS design (see Section 7.3.2).

12.3 Verification of concrete under compression or shear

PD 6687-2 7.6.2

S–N curves required to undertake a fatigue verification of concrete under compression or shear are unlikely to be available from National Authorities. In the absence of such data, the simplified approach given in BS EN 1992-2, Annex NN may be used for railway bridges, but no such option exists for highway bridges.

BS EN 1992-1-1 6.8.7(2)

The fatigue verification for concrete under compression may be assumed to be met if the following condition is satisfied:

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.5 + 0.45\,\frac{\sigma_{c,min}}{f_{cd,fat}}$$

where

$\sigma_{c,max}$ = maximum compressive stress at a fibre under the frequent load combination (compression measured positive)

$\sigma_{c,min}$ = minimum compressive stress at the same fibre where $\sigma_{c,max}$ occurs. If $\sigma_{c,min}$ is a tensile stress, then $\sigma_{c,min}$ should be taken as 0

$f_{cd,fat}$ = design fatigue strength of concrete

$$= 0.85\ \beta_{cc}(t_0)\, f_{cd}\left(1 - \frac{f_{ck}}{250}\right)$$

f_{cd} = $1.0\ f_{ck} / 1.5$

$\beta_{cc}(t_0)$ = coefficient for concrete strength at first load application

$$= \exp\left\{s\left[1 - \left(\frac{28}{t_0}\right)^{0.5}\right]\right\}$$

where

t_0 = time of the start of the cyclic loading on concrete in days

s = 0.2 for cement of strength Classes CEM 42.5R, CEM 52.5N and CEM 52.5R (Class R)

= 0.25 for cement of strength Classes CEM 32.5R, CEM 42.5 (Class N)

= 0.38 for cement of strength Class CEM 32.5 (Class S)

The maximum value for the ratio $\sigma_{c,max} / f_{cd,fat}$ is given in Table 12.2.

Table 12.2
Values for $\sigma_{c,max}/f_{cd,fat}$

Concrete strength	$\sigma_{c,max}/f_{cd,fat}$
$f_{ck} \leq 50$ MPa	≤ 0.9
$f_{ck} > 50$ MPa	≤ 0.8

BS EN 1992-1-1 6.8.7(4)

For members not requiring design shear reinforcement for the ultimate limit state it may be assumed that the concrete resists fatigue due to shear effects where the following apply:

for $\dfrac{V_{Ed,min}}{V_{Ed,max}} \geq 0$:

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 + 0.45\frac{|V_{Ed,min}|}{|V_{Rd,c}|} \begin{cases} \leq 0.9 \text{ up to C50/60} \\ \leq 0.8 \text{ greater than C55/67} \end{cases}$$

for $\dfrac{V_{Ed,min}}{V_{Ed,max}} < 0$:

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5\frac{|V_{Ed,min}|}{|V_{Rd,c}|}$$

where

$V_{Ed,max}$ = design value of the maximum applied shear force under frequent load combination

$V_{Ed,min}$ = is the design value of the minimum applied shear force under frequent load combination in the cross-section where $V_{Ed,max}$ occurs

$V_{Rd,c}$ = design value for shear resistance according to Section 7.2.1.

12.4 Limiting stress range for reinforcement under tension

BS EN 1992-1-1 6.8.6(1)

Adequate fatigue resistance may be assumed for reinforcing bars under tension if the stress range under the frequent cyclic load combined with the basic combination does not exceed 70 MPa for unwelded bars and 35 MPa for welded bars.

PD 6687-2 7.6.3

For UK highway bridges, the values in Tables 12.3 and 12.4 may be used for straight reinforcement. These are based on bars conforming to BS 4449. For bars not conforming to BS 4449, the rules for bars > 16 mm diameter should be used for all sizes unless the ranges for bars ≤ 16 mm diameter can be justified.

PD 6687-2 table 2A

Table 12.3
Limiting stress ranges – longitudinal bending for unwelded reinforcing bars in road bridges, MPa

Span m	Adjacent spans loaded		Alternate spans loaded	
	Bars ≤ 16 mm	Bars > 16 mm	Bars ≤ 16 mm	Bars > 16 mm
≤ 3.5	150	115	210	160
5	125	95	175	135
10	110	85	175	135
20	110	85	140	110
30 to 50	90	70	110	85
100	115	90	135	105
≥ 200	190	145	200	155

Notes

1 Intermediate values may be obtained by linear interpolation.

2 This table applies to slabs but need only be applied to those slabs that do not conform to the criteria given in Section 12.1.

Table 12.4
Limiting stress ranges – transverse bending for unwelded reinforcing bars in road bridges, MPa

Span m	Bars ≤ 16 mm	Bars > 16 mm
≤ **3.5**	210	160
5	120	90
≥ **10**	70	55

Notes
1 Intermediate values may be obtained by linear interpolation.
2 This table applies to slabs but need only be applied to those slabs that do not conform to the criteria given in Section 12.1.

PD 6687-2 table 2B

If the stress range limits exceed the values given in Tables 12.3 and 12.4 (e.g. for reinforcement over the pier), full fatigue checks are to be carried out using the 'damage equivalent stress range' method, following the guidance given in Annex NN of BS EN 1992-2. The stress ranges are calculated using 'Fatigue Load Model 3', which represents a four-axle vehicle with an all up weight of 48 tonnes. Annex NN further increases this weight to 84 tonnes for intermediate supports and 67 tonnes for other areas.

13 Serviceability

13.1 General

BS EN 1992-1-1
7.1

The common serviceability limit states considered are:

- Stress limitation.
- Crack control.
- Deflection control.

In the calculation of stresses and deflections, cross-sections should be assumed to be uncracked provided that the flexural tensile stress does not exceed $f_{ct,eff}$. The value of $f_{ct,eff}$ may be taken as f_{ctm} or $f_{ctm,fl}$ provided that the calculation for minimum tension reinforcement is also based on the same value. For the purposes of calculating crack widths and tension stiffening f_{ctm} should be used.

13.2 Stress limitation

BS EN 1992-2
7.2(102) & NA

PD 6687-2
8.1.1

Longitudinal cracks may occur if the stress level under the characteristic combination of loads exceeds a critical value. Such cracking may lead to a reduction of durability. In the absence of other measures, such as an increase in the cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it is recommended that the compressive stress is limited to a value 0.6 f_{ck} in areas exposed to environments of exposure classes XD, XF and XS (see Table 4.2). In the presence of confinement the maximum stress limit is 0.66 f_{ck}.

BS EN 1992-1-1
7.2(3)

If the stress in the concrete under the quasi-permanent loads is less than 0.45 f_{ck}, linear creep may be assumed. If the stress in concrete exceeds 0.45 f_{ck}, non-linear creep should be considered (see Section 3.1.2).

BS EN 1992-1-1
7.2(5), NA

PD 6687-2
8.1.2

For the appearance unacceptable cracking or deformation may be assumed to be avoided if, under the characteristic combination of loads, the tensile stress in the reinforcement does not exceed 0.8 f_{yk}. Where the stress is caused by an imposed deformation, the tensile stress should not exceed 1.0 f_{yk}. The mean value of the stress in prestressing tendons should not exceed 0.75 f_{pk}. Crack control (Section 13.4) and deflection (Section 13.6) should also be checked.

13.3 Calculation of crack widths

BS EN 1992-1-1
7.3.4

The crack width, w_k, may be calculated from following Expression:

$$w_k = s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm})$$

where

$s_{r,max}$ = maximum crack spacing

ε_{sm} = mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered

ε_{cm} = mean strain in the concrete between cracks

$\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from the Expression:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{ct,eff}}\,(1 + \alpha_e \rho_{p,eff})}{E_s} \qquad 0.6\,\frac{\sigma_s}{E_s}$$

where

σ_s = stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\sigma_p$, which is the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level

k_t = factor dependent on the duration of the load
= 0.6 for short term loading
= 0.4 for long term loading

α_e = ratio E_s/E_{cm}

$\rho_{p,eff}$ = $(A_s + \xi_1^2 A_p')/A_{c,eff}$

where

A_p' = area of pre- or post-tensioned tendons within $A_{c,eff}$.

$A_{c,eff}$ = effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth, $h_{c,ef}$, where $h_{c,ef}$ is the lesser of 2.5($h – d$), ($h – x$)/3 or h/2 (see Figure 13.3).

ξ_1 = adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel:
= $(\xi\ \phi_s / \phi_p)^{0.5}$

ξ = ratio of bond strength of prestressing and reinforcing steel, according to Table 12.1
If only prestressing steel is used to control cracking, $\xi_1 = \xi^{0.5}$

ϕ_s = largest bar diameter of reinforcing steel

ϕ_p = equivalent diameter of tendon according to Section 12.2

In situations where bonded reinforcement is fixed at reasonably close centres within the tension zone (spacing ≤ 5(c + ϕ/2), the maximum final crack spacing may be calculated from following Expression (see Figure 13.1):

$$s_{r,max} = k_3 c + k_1 k_2 k_4 \phi / \rho_{p,eff}$$

BS EN 1992-1-1 Exp. (7.11)

where

ϕ = bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, ϕ_{eq}, should be used

where

ϕ_{eq} = $(n_1\phi_1^2 + n_2\phi_2) / (n_1\phi_1 + n_2\phi_2)$

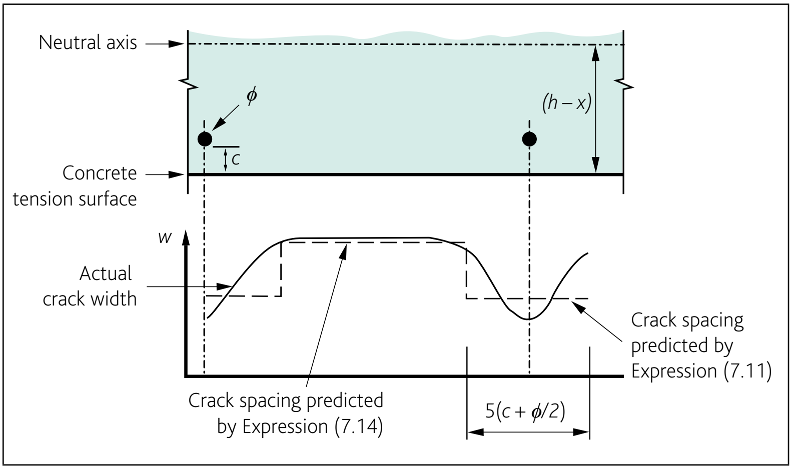
n_1 = number of bars of diameter ϕ_1

n_2 = number of bars of diameter ϕ_2

k_3 = 3.4

c = cover to the longitudinal reinforcement. The nominal cover, c_{nom}, may be used.

k_1 = coefficient which takes account of the bond properties of the bonded reinforcement
= 0.8 for high bond bars
= 1.6 for bars with an effectively plain surface (e.g. prestressing tendons)



BS EN 1992-1-1 fig. 7.2

Figure 13.1
Crack width, *w*, at concrete surface relative to distance from bar

k_4 = 0.425

k_2 = coefficient which takes account of the distribution of strain
= 0.5 for bending
= 1.0 for pure tension
For cases of eccentric tension or for local areas, intermediate values of k_2 should be used which may be calculated from the relation:

k_2 = $(\varepsilon_1 + \varepsilon_2)/(2\varepsilon_1)$

ε_1 = greater tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section

ε_2 = lesser tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section

Where the spacing of the bonded reinforcement exceeds $5(c + \phi/2)$ (see Figure 13.1) or where there is no bonded reinforcement within the tension zone, an upper bound to the crack width may be found by assuming a maximum crack spacing:

BS EN 1992-1-1 Exp. (7.14)

$s_{r,max} = 1.3\ (h - x)$

Where the angle between the axes of principal stress and the direction of the reinforcement, for members reinforced in two orthogonal directions, is significant (>15°), then the crack spacing $s_{r,max}$ may be calculated from the following expression:

$s_{r,max} = 1\ /\ (\cos\theta\ /\ s_{r,max,y} + \sin\theta\ /\ s_{r,max,z})$

where

θ = angle between the reinforcement in the y direction and the direction of the principal tensile stress

$s_{r,max,y}$ = crack spacing calculated in the y-direction

$s_{r,max,z}$ = crack spacing calculated in the z-direction

BS EN 1992-2 7.3.1(105) & NA

A limiting calculated crack width w_{max} , taking account of the proposed function and nature of the structure and the costs of limiting cracking, should be established. Due to the random nature of the cracking phenomenon, actual crack widths cannot be predicted. However, if the crack widths calculated in accordance with the models given in BS EN 1992-2 are limited to the values given in Table 13.1, the performance of the structure is unlikely to be impaired.

BS EN 1992-2 NA. NA.2.2

The decompression limit requires that all concrete within a certain distance of bonded tendons or their ducts should remain in compression under the specified loading. The distance within which all concrete should remain in compression should be taken as the value of $c_{min,dur}$. Where the most tensile face of a section is not subject to XD or XS exposure but another face is, the decompression limit should require all tendons within 100 mm of a surface subject to XD or XS exposure to have a depth $c_{min,dur}$ of concrete in compression between them and surfaces subject to XD or XS exposure.

PD 6687-2 8.2.1b)

Since the decompression limit of 100 mm is likely to be greater than the cover required for durability, this introduces an anomaly. For example, only 50 mm of concrete in compression might be deemed adequate to protect the tendons, whereas 60 mm of concrete in compression plus 40 mm not in compression would not, which is clearly illogical. Changing the distance from the recommended value of 100 mm to the cover required for durability, $c_{min,dur,}$ is more logical.

PD 6687-2 8.2.2

In some situations, such as structures cast against the ground, c_{nom} will be significantly greater than the cover required for durability. Where there are no appearance requirements it is reasonable to determine the crack width at the cover required for durability and verify that it does not exceed the relevant maximum crack width. This may be done by multiplying the crack width determined at the surface by $(c_{min,dur,} + \Delta c_{dev})/c_{nom}$ to give the crack width at the cover required for durability, and verifying that it is not greater than w_{max}. This approach assumes that the crack width varies linearly from zero at the bar. Given the accuracy of crack calculation methods this simplification is considered reasonable.

Table 13.1
Recommended values of w_{max} and relevant combination rules

Exposure class[a]	Reinforced members and prestressed members without bonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination[b] (mm)	Frequent load combination[b] (mm)
X0, XC1	0.3[c]	0.2
XC2, XC3, XC4	0.3	0.2[d]
XD1, XD2, XD3 XS1, XS2, XS3	0.3	0.2[e] and decompression

Key

a The exposure class considered, including at transfer, applies to the most severe exposure the surface will be subject to in service.

b For the crack width checks under combinations which include temperature distribution, the resulting member forces should be calculated using gross section concrete properties and self-equilibrating thermal stresses within a section may be ignored.

c For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

d For these exposure classes, in addition, decompression should be checked under the quasipermanent combination of loads.

e 0.2 applies to the parts of the member that do not have to be checked for decompression.

BS EN 1992-2 & NA, table NA.2

13.4 Control of cracking

BS EN 1992-1-1 7.3.3(2)

Where the minimum reinforcement given by Section 13.5 is provided, crack widths are unlikely to be excessive if:

- For cracking caused predominantly by restraint, the bar sizes given in Table 13.2 are not exceeded where the steel stress is the value obtained immediately after cracking (i.e. σ_s in Section 13.5).
- For cracks caused mainly by loading, either the provisions of Table 13.2 or the provisions of Table 13.3 are complied with. The steel stress should be calculated on the basis of a cracked section under the relevant combination of actions.

For pre-tensioned concrete, where crack control is mainly provided by tendons with direct bond, Tables 13.2 and 13.3 may be used with a stress equal to the total stress minus prestress. For post-tensioned concrete, where crack control is provided mainly by ordinary reinforcement, the tables may be used with the stress in this reinforcement calculated with the effect of prestressing forces included.

The maximum bar diameter should be modified as follows:

Bending (at least part of section in compression):

$$\phi_s = \phi^*_s \,(f_{ct,eff}/2.9)\,\frac{k_c h_{cr}}{2\,(h-d)}$$

Tension (uniform axial tension):

$$\phi_s = \phi^*_s \,(f_{ct,eff}/2.9)\,h_{cr}/(8(h-d))$$

where

ϕ_s = adjusted maximum bar diameter
ϕ_s = maximum bar size given in the Table 13.2
h = overall depth of the section
h_{cr} = depth of the tensile zone immediately prior to cracking, considering the characteristic values of prestress and axial forces under the quasi-permanent combination of actions
d = effective depth to the centroid of the outer layer of reinforcement

Where all the section is under tension $h - d$ is the minimum distance from the centroid of the layer of reinforcement to the face of the concrete (consider each face where the bar is not placed symmetrically).

BS EN 1992-1-1 table 7.2N

Table 13.2
Maximum bar diameters for crack control

Steel stress (MPa)	Maximum bar size (mm) for crack widths of		
	0.4 mm	0.3 mm	0.2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	—

BS EN 1992-1-1 table 7.3N

Table 13.3
Maximum bar spacing for crack control

Steel stress (MPa)	Maximum bar spacing (mm) for maximum crack widths of		
	0.4 mm	0.3 mm	0.2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	—
360	100	50	—

13.5 Minimum reinforcement areas of main bars

BS EN 1992-1-1 7.3.2(1)

BS EN 1992-2 7.3.2(102)

BS EN 1992-1-1 Exp. (7.1)

If crack control is required, a minimum amount of bonded reinforcement is required to control cracking in areas where tension is expected. The required minimum areas of reinforcement $A_{s,min}$ may be calculated as follows. In profiled cross-sections like T-beams and box girders, minimum reinforcement should be determined for the individual parts of the section (webs, flanges).

$$A_{s,min} = k_c \, k \, f_{ct,eff} \, A_{ct} \, / \, \sigma_s$$

where

A_{ct} = area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

σ_s = absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack width limits according to the maximum bar size or spacing indicated in Tables 13.2 and 13.3.

BS EN 1992-2 7.3.2(105)

$f_{ct,eff}$ = mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur.

= f_{ctm} or lower ($f_{ctm}(t)$) if cracking is expected earlier than 28 days

A minimum value of 2.9 MPa should be taken

k = coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces

= 1.0 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm, intermediate values may be interpolated

= 0.65 for webs with $h \geq 800$ mm or flanges with widths greater than 800 mm, intermediate values may be interpolated

k_c = coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:
= 1.0 for pure tension.
= 0.4 for pure bending.
= $0.4\,[1 - (\sigma_c / (k_1(h/h^*)\,f_{ct,eff}))] \leq 1$ for rectangular sections and webs of box sections and T-sections.
= $0.9\,F_{cr} / (A_{ct}\,f_{ct,eff}) \geq 0.5$ for flanges of box sections and T-sections.

σ_c = mean stress of the concrete acting on the part of the section under consideration.
= N_{Ed} / bh

N_{Ed} = axial force at the serviceability limit state acting on the part of the cross-section under consideration (compressive force positive). N_{Ed} should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions

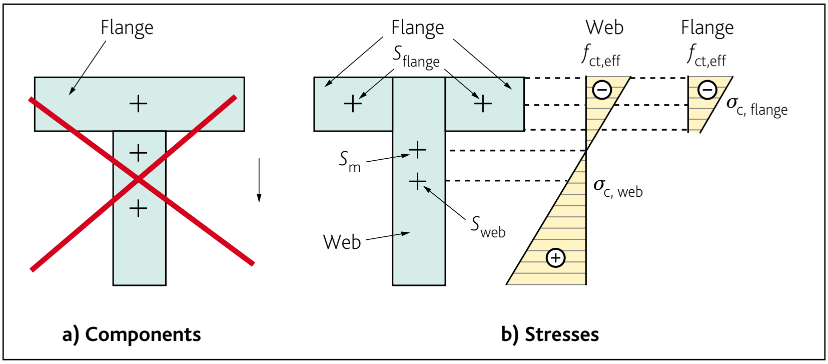
h^* = MIN{h ;1.0}

k_1 = coefficient considering the effects of axial forces on the stress distribution:
= 1.5 if N_{Ed} is a compressive force
= $2h^*/3h$ if N_{Ed} is a tensile force

F_{cr} = absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$

See also Section 15.2.1 for a simplified method of calculating minimum area of steel.

In flanged cross-sections like T-beams and box girders, the division into parts should be as indicated in Figure 13.2.



BS EN 1992-2 fig. 7.101

Figure 13.2
Example for a division of a flanged cross-section for analysis of cracking

BS EN 1992-1-1 7.3.2(3)

Bonded tendons in the tension zone may be assumed to contribute to crack control within a distance ≤ 150 mm from the centre of the tendon. This may be taken into account by including the term $\xi_1 A_p{}' \Delta_p$ in the Expression above to give:

$$A_{s,min} = (k_c\,k\,f_{ct,eff}\,A_{ct} - \xi_1 A_p{}'\,\Delta\sigma_p) / \sigma_s$$

where

$A_p{}'$ = area of pre- or post-tensioned tendons within $A_{c,eff}$

$A_{c,eff}$ = effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth, $h_{c,ef}$

$h_{c,ef}$ = MIN {$2.5(h - d)$; $(h - x)/3$; $h/2$} (see Figure 13.3)

ξ_1 = adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel
= $(\xi\,\phi_s / \phi_p)^{0.5}$
= $\xi^{\,0.5}$ if only prestressing steel is used to control cracking

ξ = ratio of bond strength of prestressing and reinforcing steel, according to Table 13.4

ϕ_s = largest bar diameter of reinforcing steel
ϕ_p = equivalent diameter of tendon
= $1.6\ A_p^{0.5}$ for bundles
= $1.75\ \phi_{wire}$ for single 7 wire strands where ϕ_{wire} is the wire diameter
= $1.20\ \phi_{wire}$ for single 3 wire strands where ϕ_{wire} is the wire diameter
$\Delta\sigma_p$ = stress variation in prestressing tendons from the state of zero strain of the concrete at the same level

BS EN 1992-1-1 Fig 7.1

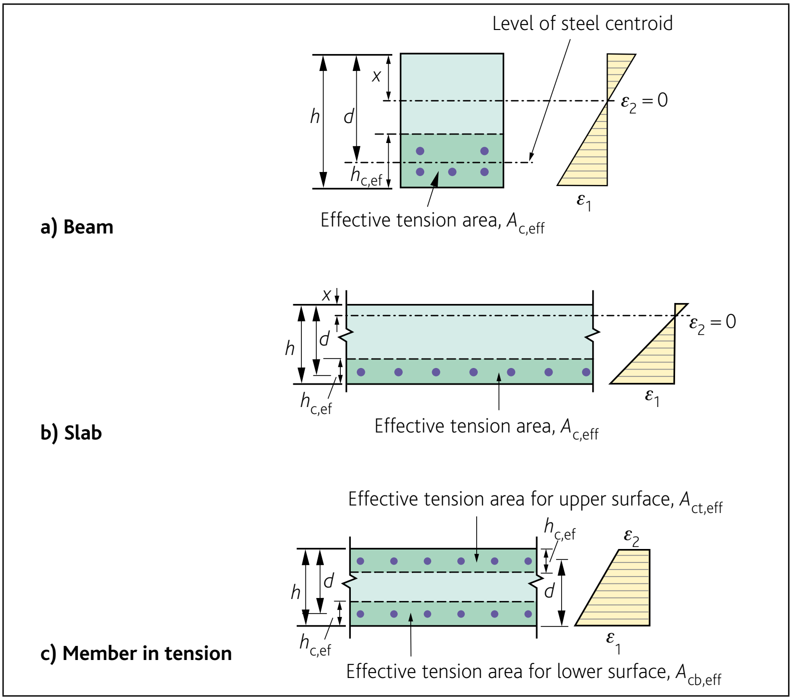


Figure 13.3
Effective tension area (typical cases)

BS EN 1992-1-1 7.3.2 (4)

In prestressed members no minimum reinforcement is required in sections where, under the characteristic combination of loads and the characteristic value of prestress, the concrete is compressed or the absolute value of the tensile stress in the concrete is below $f_{ct,eff}$.

Table 13.4
Ratio of bond strength, ξ, between tendons and reinforcing steel

BS EN 1992-1-1 table 6.2

Prestressing steel	ξ		
	Pre-tensioned	Bonded, post-tensioned	
		≤ C50/60	≥ C70/85
Smooth bars and wires	Not applicable	0.3	0.15
Strands	0.6	0.5	0.25
Indented wires	0.7	0.6	0.30
Ribbed bars	0.8	0.7	0.35
Note For intermediate values between C50/60 and C70/85 interpolation may be used.			

13.6 Control of deflection

The calculation method adopted shall represent the true behaviour of the structure under relevant actions to an accuracy appropriate to the objectives of the calculation.

BS EN 1992-1-1 7.4.3(2)

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Members which are expected to crack, but may not be fully cracked, will behave in a manner intermediate between the uncracked and fully cracked conditions and, for members subjected mainly to flexure, an adequate prediction of behaviour is given in the following expression:

BS EN 1992-1-1 7.4.3(3)

$\alpha = \zeta\ \alpha_{II} + (1 - \zeta)\ \alpha_{I}$

BS EN 1992-1-1 Exp. (7.18)

where

- α = deformation parameter considered which may be, for example, a strain, a curvature, or a rotation. (As a simplification, a may also be taken as a deflection – see below)
- α_I , α_{II} = the values of the parameter calculated for the uncracked and fully cracked conditions respectively
- ζ = distribution coefficient (allowing for tensioning stiffening at a section)
  - = $1 - \beta\ (\sigma_{sr} / \sigma_s)^2$
  - = 0 for uncracked sections
- β = coefficient taking account of the influence of the duration of the loading or of repeated loading on the average strain
  - = 1.0 for a single short-term loading
  - = 0.5 for sustained loads or many cycles of repeated loading
- σ_s = stress in the tension reinforcement calculated on the basis of a cracked section
- σ_{sr} = stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking

Note: σ_{sr}/σ_s may be replaced by M_{cr}/M for flexure or N_{cr}/N for pure tension, where M_{cr} is the cracking moment and N_{cr} is the cracking force.

Deformations due to loading may be assessed using the tensile strength and the effective modulus of elasticity of the concrete.

BS EN 1992-1-1 7.4.3(4)

Table 3.1 indicates the range of likely values for tensile strength. In general, the best estimate of the behaviour will be obtained if f_{ctm} is used. Where it can be shown that there are no axial tensile stresses (e.g. those caused by shrinkage or thermal effects) the flexural tensile strength, $f_{ctm,fl}$, may be used.

$f_{ctm,fl} = \mathrm{MAX}\{(1.6 - h/1000)f_{ctm}; f_{ctm}\}$

BS EN 1992-1-1 3.1.8

where

- h = total member depth in mm
- f_{ctm} = mean axial tensile strength from Table 3.1

For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete according to the following Expression

BS EN 1992-1-1 7.4.3(5)

$E_{c,eff} = E_{cm} / (1 + \varphi\ (\infty,t_0))$

where

- $\varphi\ (\infty,t_0)$ = creep coefficient relevant for the load and time interval (see Section 3.1.2)

Shrinkage curvatures may be assessed from the following:

BS EN 1992-1-1 7.4.3(6)

$1/r_{cs} = \varepsilon_{as}\ \alpha_e\ S / I$

where

- $1/r_{cs}$ = curvature due to shrinkage

ε_{cs} = free shrinkage strain (see Section 3.1.3)
S = first moment of area of the reinforcement about the centroid of the section
I = second moment of area of the section
α_e = effective modular ratio
= E_s / $E_{c,eff}$

S and *I* should be calculated for the uncracked condition and the fully cracked condition, the final curvature being assessed using the Expression for α above.

BS EN 1992-1-1
7.4.3(7)

The most rigorous method of assessing deflections using the method given above is to compute the curvatures at frequent sections along the member and then calculate the deflection by numerical integration. In most cases it will be acceptable to compute the deflection twice, assuming the whole member to be in the uncracked and fully cracked condition in turn, and then interpolate using the Expression for α above.

14 Detailing – general requirements

14.1 General

The rules given in this section apply to ribbed reinforcement, welded mesh and prestressing tendons used in structures subject predominantly to static loading.

BS EN 1992-1-1 8.1(1)

Unless otherwise stated, the rules for individual bars also apply for bundles of bars for which an equivalent diameter $\phi_n = \phi(n_b)^{0.5}$ should be used in the calculations. In this Expression, n_b is the number of bars in the bundle. A value for n_b should be limited to four vertical bars in compression and in lapped joints, and to three in all other cases. The value of ϕ_n should be less than or equal to 55 mm.

BS EN 1992-1-1 8.9.1(1) & (2)

The clear distance between (and the cover to) bundled bars should be measured from the actual external contour of the bundled bars. Bars are allowed to touch one another at laps and they need not be treated as bundled bars under these conditions.

BS EN 1992-1-1 8.9.1(3) & (4)

14.2 Spacing of bars

BS EN 1992-1-1 8.2

The spacing of bars should be such that concrete can be placed and compacted satisfactorily for the development of bond.

The clear distance between individual parallel bars or between horizontal layers of parallel bars should not be less than the bar diameter, the aggregate size + 5 mm, or 20 mm, whichever is the greatest.

Where bars are positioned in separate horizontal layers, the bars in each layer should be located vertically above each other. There should be sufficient space between the resulting columns of bars to allow access for vibrators to give good compaction of the concrete.

14.3 Mandrel sizes for bent bars

BS EN 1992-1-1 8.3(1) & (2)

The diameter to which a bar is bent should be such as to avoid damage to the reinforcement and crushing of concrete inside the bend of the bar. To avoid damage to reinforcement the mandrel size is as follows:

- 4ϕ, for bar diameter $\phi \leq 16$ mm
- 7ϕ, for bar diameter $\phi > 16$ mm
- 20ϕ, for mesh bent after welding where transverse bar is on or within 4ϕ of the bend.

Otherwise 4ϕ, or 7ϕ, as above. Welding must comply with ISO/FDIS 17660-2[22].

BS EN 1992-1-1 table 8.1N & NA

The mandrel diameter ϕ_m to avoid crushing of concrete inside the bend need not be checked if:

- Anchorage of the bar does not require a length more than 5ϕ past the end of the bend; and
- The bar is not positioned at an edge and there is a cross bar (of diameter $\geq \phi$) inside the bend, and
- Diameters noted above are used.

BS EN 1992-1-1 8.3(3)

Otherwise the following minimum mandrel diameter $\phi_{m,min}$ should be used:

$$\phi_{m,min} \geq F_{bt}\ ((1/a_b) + 1/(2\phi))/\ f_{cd}$$

BS EN 1992-1-1 Exp. (8.1)

where

F_{bt} = tensile force in the bar at the start of the bend caused by ultimate loads

a_b = half the centre to centre spacing of bars (perpendicular to the plane of the bend). For bars adjacent to the face of the member, a_b = cover + 0.5ϕ

f_{cd} = $0.85\ f_{ck}\ /\ 1.5$

BS EN 1992-1-1 3.1.6(1) & NA to BS EN 1992-2

The value of f_{cd} should not be taken greater than that for concrete class C55/67

where

f_{ck} = characteristic cylinder strength

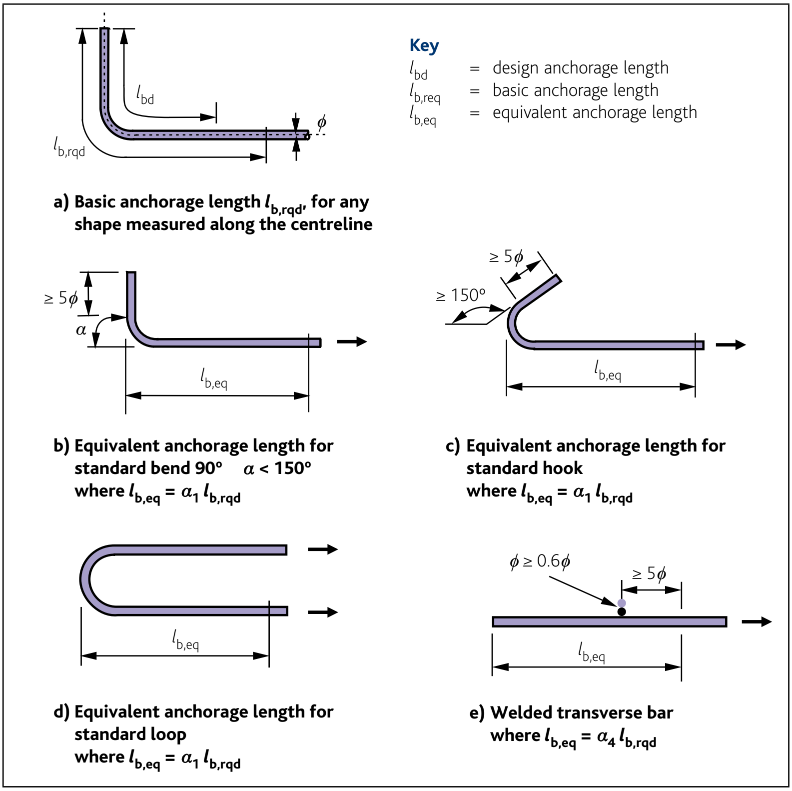
14.4 Anchorage of bars

14.4.1 General

BS EN 1992-1-1
8.4(1) & (2)

All reinforcement should be so anchored that the forces in them are safely transmitted to the surrounding concrete by bond without causing cracks or spalling. The common methods of anchorage of longitudinal bars and links are shown in Figures 14.1 and 14.2.

BS EN 1992-1-1
fig. 8.1



a) Basic anchorage length $l_{b,rqd}$, for any shape measured along the centreline

b) Equivalent anchorage length for standard bend 90° α < 150° where $l_{b,eq} = \alpha_1 \, l_{b,rqd}$

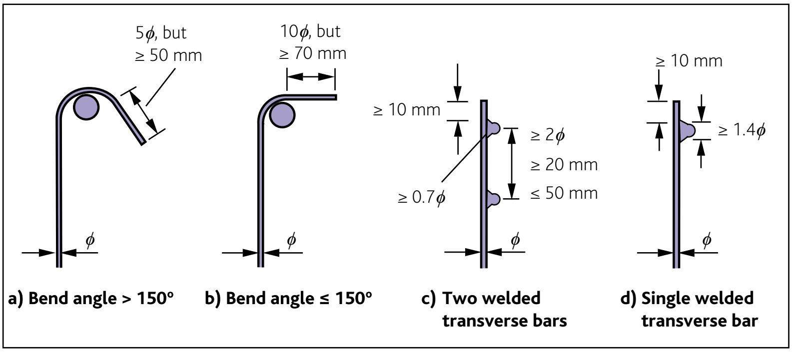
c) Equivalent anchorage length for standard hook where $l_{b,eq} = \alpha_1 \, l_{b,rqd}$

d) Equivalent anchorage length for standard loop where $l_{b,eq} = \alpha_1 \, l_{b,rqd}$

e) Welded transverse bar where $l_{b,eq} = \alpha_4 \, l_{b,rqd}$

Figure 14.1
Methods of anchorage other than by a straight bar

BS EN 1992-1-1
fig. 8.5



a) Bend angle > 150° **b) Bend angle ≤ 150°** **c) Two welded transverse bars** **d) Single welded transverse bar**

Figure 14.2
Anchorage of links

14.4.2 Design anchorage length l_{bd}

BS EN 1992-1-1 8.4.4(1) & table 8.2

The design anchorage length l_{bd} is

$l_{bd} = (\alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5) l_{b,rqd} \geq l_{b,min}$

where

α_1 = factor dealing with the form of bar assuming adequate cover

= 0.7 for bent bars in tension where $c_d > 3\phi$, where c_d is defined in Figure 14.3
= 1.0 otherwise for bars in tension
= 1.0 for bars in compression

α_2 = factor dealing with effect of concrete minimum cover

= $1 - 0.15(c_d - \phi)/\phi \geq 0.7$ for straight bars in tension but ≤ 1.0
= $1 - 0.15(c_d - 3\phi)/\phi \geq 0.7$ for bent bars in tension but ≤ 1.0
= 1.0 otherwise for bars in tension
= 1.0 for bars in compression

α_3 = factor dealing with effect of confinement by transverse reinforcement

= 1.0 generally

α_4 = factor dealing with the effect of influence of welded transverse bars

= 0.7 for a welded transverse bar conforming with Figure 14.1e)
= 1.0 otherwise

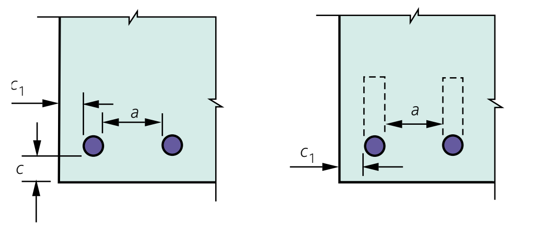
α_5 = factor dealing with the effect of pressure transverse to the plane of splitting along the design anchorage length

= 1.0 generally

$l_{b,rqd}$ = basic anchorage length (see Section 14.4.3)
$l_{b,min}$ = the minimum anchorage length
$\geq$ MAX{$0.3 l_{b,rqd}$; 10ϕ; 100 mm} in tension bars; and
$\geq$ MAX{$0.6 l_{b,rqd}$; 10ϕ; 100 mm} in compression bars.

The product $(\alpha_2 \alpha_3 \alpha_5) \geq 0.7$

BS EN 1992-1-1 fig. 8.3



a) Straight bars
c_d = minimum of a/2, c or c_1

b) Bent or hooked bars
c_d = minimum of a/2 or c_1

Note
c and c_1 are taken to be nominal covers

Figure 14.3
Values of c_d for beams and slabs

14.4.3 Basic anchorage length $l_{b,rqd}$

BS EN 1992-1-1 8.4.3

$l_{b,rqd}$ = basic anchorage length required = $(\phi/4)\ (\sigma_{sd}/f_{bd})$

where

ϕ = diameter of the bar
σ_{sd} = design stress in the bar at the position from where the anchorage is measured
f_{bd} = ultimate bond stress (see Section 14.5)

The anchorage length should be measured along the centre line of the bar in bent bars.

14.4.4 Equivalent anchorage length $l_{b,eq}$

BS EN 1992-1-1 8.4.4(2)

As a simplification

- For the shapes shown in Figure 14.1b) to d) an equivalent anchorage length $l_{b,eq}$ may be used where $l_{b,eq} = \alpha_1 l_{b,rqd}$.
- For the arrangement shown in Figure 14.1e) $l_{b,eq} = \alpha_4 l_{b,req}$.

14.5 Ultimate bond stress

BS EN 1992-1-1 8.4.2(2)

The ultimate bond stress, f_{bd}, for ribbed bars may be taken as (see also Table 14.1)

$$f_{bd} = 2.25\ \eta_1\ \eta_2\ f_{ctd}$$

where

η_1 = coefficient related to the quality of the bond condition and the position of the bar during concreting
= 1.0 for 'good' bond conditions (see Figure 14.4 for definition)
= 0.7 for all other cases and for bars in structural elements built using slipforms

η_2 = coefficient related to bar diameter
= 1.0 for bar diameter ≤ 32 mm
= $(132 - \phi)/100$ for bar diameter >32 mm

BS EN 1992-2 3.1.6(102) & NA

f_{ctd} = $(\alpha_{ct} f_{ctk,0.05}/\gamma_C)$ is the design value of tensile strength using the value of $f_{ctk,0.05}$ obtained from Table 3.1, α_{ct} = 1.0 and γ_C = 1.5. Due to the brittleness of higher strength concrete $f_{ctk,0.05}$ should be limited here to the value for C60/75, unless it can be verified that the average bond strength increases above this limit

BS EN 1992-1-1 fig. 8.2

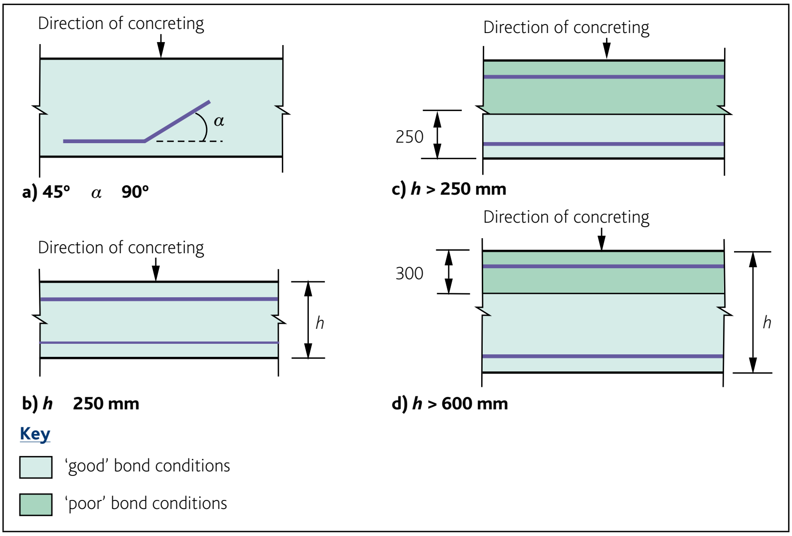


Figure 14.4
Description of bond conditions

Table 14.1
Design bond stress for different bond conditions

	Design bond stress f_{bd} (N/mm²)							
	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75
Good bond & $\phi \le 32$ mm	2.7	3.0	3.3	3.8	4.1	4.4	4.5	4.7
Good bond & $\phi = 40$ mm	2.5	2.8	3.0	3.5	3.7	4.0	4.1	4.3
Poor bond & $\phi \le 32$ mm	1.9	2.1	2.3	2.6	2.8	3.0	3.2	3.3
Poor bond & $\phi = 40$ mm	1.7	1.9	2.1	2.4	2.6	2.8	2.9	3.0

14.6 Anchorage of tendons at ULS

BS EN 1992-1-1 8.10.2.3

The anchorage of tendons should be checked in sections where the concrete tensile stress exceeds $f_{ctk,0.05}$ ($f_{ctk,0.05}$ should not exceed the value for class C60/75 concrete). The total anchorage length for a tendon with a stress σ_{pd} is:

BS EN 1992-1-1 Exp. (8.21)

$$l_{bpd} = l_{pt2} + (\alpha_2\, \phi\, (\sigma_{pd} - \sigma_{pm,\infty})/f_{bpd})$$

where

$$l_{pt2} = 1.2\, (\alpha_1 \alpha_2\, \phi\, \sigma_{pm,0} / f_{bpt})$$

where

α_1 = 1.0 for gradual release of prestress and 1.25 for sudden release
$\sigma_{pm,0}$ = stress in the tendon just after release
f_{bpt} = $\eta_{p1}\eta_1 f_{ctd}(t)$
η_{p1} = 2.7 for indented wires and 3.2 for 3- and 7-wire strands
η_1 = 1.0 for good bond conditions and 0.7 otherwise
$f_{ctd}(t)$ = design tensile value of strength at the time of release
= $1.0 \times 0.7 f_{ctm}(t)/1.5$
$f_{ctm}(t)$ = $(\beta_{cc}(t))^{\alpha} f_{ctm}$
$\beta_{cc}(t)$ = coefficient which depends on the age of loading
= $\exp\left\{s\left[1-\left(\frac{28}{t}\right)^{0.5}\right]\right\}$
s = coefficient which depends on type of cement (see Section 3.1.2)
= 0.20 for cement strength classes CEM 42.5R, CEM 52.5N and CEM 52.5R (Class R)
= 0.25 for cement strength classes CEM 32.5R, CEM 42.5N (Class N)
= 0.38 for cement strength class CEM 32.5 (Class S)
t = age of concrete in days
f_{ctm} = mean value of axial tensile strength of concrete (see Table 3.1)
α = 1 for $t < 28$
= 2/3 for $t \ge 28$
α_2 = 0.25 for circular tendons and 0.19 for 3- and 7-wire strands
ϕ = nominal diameter of the tendon
σ_{pd} = tendon stress corresponding to the force calculated for a cracked section
$\sigma_{pm,\infty}$ = prestress after all losses
f_{bpd} = $\eta_{p2}\eta_1 f_{ctd}$, where η_{p2} = 1.4 for indented wires and 1.2 for 7-wire strands. η_1 is described in Section 14.5

BS EN 1992-1-1
8.10.2.2(2)

14.7 Anchorage of tendons at transfer of prestress

BS EN 1992-1-1
Exp. (8.16)

The basic value of the transmission length at the release of tendons, l_{pt}, is given by:

$l_{pt} = \alpha_1 \alpha_2 \, \phi \, \sigma_{pm,0} \, / \, f_{bpt}$

where

α_1 = 1.0 for gradual release
= 1.25 for sudden release

α_2 = 0.25 for tendons with circular cross-section
= 0.19 for 3- and 7-wire strands

ϕ = nominal diameter of tendon

$\sigma_{pm,0}$ = tendon stress just after release

f_{bpt} = bond stress
= $\eta_{p1} \eta_1 f_{ctd}(t)$

where

η_{p1} = coefficient that takes into account the type of tendon and the bond situation at release
= 2.7 for indented wires
= 3.2 for 3- and 7-wire strands

η_1 = 1.0 for good bond conditions
= 0.7 otherwise, unless a higher value can be justified with regard to special circumstances in execution

$f_{ctd}(t)$ = design tensile value of strength at time of release (see Section 14.6)

The design value of the transmission length should be taken as the less favourable of two values, depending on the design situation:

$l_{pt1} = 0.8 \, l_{pt}$

or

$l_{pt2} = 1.2 \, l_{pt}$

Normally the lower value is used for verifications of local stresses at release, the higher value for ultimate limit states (shear, anchorage etc.).

14.8 Laps

14.8.1 General

BS EN 1992-1-1
8.7

Forces are transmitted from one bar to another by lapping, welding or using mechanical devices.

Laps between bars should normally be staggered and not located in areas of high moments/forces. All bars in compression and secondary reinforcement may be lapped at one place.

14.8.2 Lapping bars

BS EN 1992-1-1
8.7.2(2) & 8.7.3

Laps of bars should be arranged as shown in Figure 14.5.

The design lap length l_0 is:

BS EN 1992-1-1
Exp. (8.10)

$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$

where

α_6 = $(\rho_1/25)^{0.5}$. $1 < \alpha_6 < 1.5$ (see Table 14.2)

ρ_1 = percentage of reinforcement lapped within $0.65 l_0$ from the centre line of the lap being considered

$l_{0,min} \geq$ MAX$\{0.3 \, \alpha_6 \, l_{b,rqd}$; 15φ; 200 mm$\}$

α_1, α_2, α_3, α_4 and α_5 are described in Section 14.4.2

Table 14.2
Values of coefficient α_6

ρ_1, % lapped bars relative to total cross-sectional area	< 25%	33%	50%	> 50%
α_6	1.0	1.15	1.4	1.5

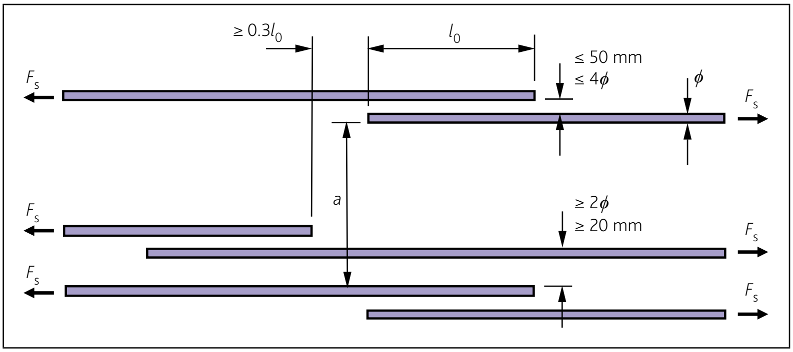


Figure 14.5
Arranging adjacent lapping bars

BS EN 1992-1-1 fig. 8.7

14.8.3 Lapping fabric

BS EN 1992-1-1 8.7.5

Laps of fabric should be arranged as shown in Figure 14.6.

When fabric reinforcement is lapped by layering, the following should be noted:

- Calculated stress in the lapped reinforcement should not be more than 80% of the design strength; if not, the moment of resistance should be based on the effective depth to the layer furthest from the tension face and the calculated steel stress should be increased by 25% for the purposes of crack control.
- Permissible percentage of fabric main reinforcement that may be lapped in any section is:
  100% if $(A_s/s) \leq 1200$ mm^2/m (where s is the spacing of bars)
  60% if $A_s/s > 1200$ mm^2/m.
- All secondary reinforcement may be lapped at the same location and the minimum lap length $l_{0,min}$ for layered fabric is as follows:

  ≥ 150 mm for $\phi \leq 6$ mm
  ≥ 250 mm for 6 mm < ϕ < 8.5 mm
  ≥ 350 mm for 8.5 mm < ϕ < 12 mm

There should generally be at least two bar pitches within the lap length. This could be reduced to one bar pitch for $\phi \leq 6$ mm.

BS EN 1992-1-1 8.7.5.1(7) 8.7.5.2(1)

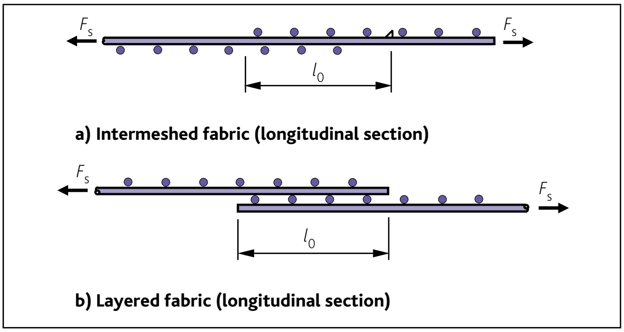


Figure 14.6
Lapping of welded fabric

BS EN 1992-1-1 fig. 8.10

BS EN 1992-1-1
8.7.4

14.8.4 Transverse reinforcement

Transverse reinforcement is required in the lap zone to resist transverse tension forces.

Where the diameter of the lapped bar is less than 20 mm or the percentage of reinforcement lapped at any section is less than 25%, then any transverse reinforcement or links necessary for other purposes may be deemed sufficient for the transverse tensile forces without further justification.

When the above conditions do not apply, transverse reinforcement should be provided as shown in Figure 14.7. Where more than 50% of bars are lapped at one section and the spacing between adjacent laps (dimension *a* in Figure 14.5) < 10ϕ, the transverse reinforcement should be in the form of links or U bars anchored into the body of the section.

In Figure 14.7, the total area of transverse reinforcement at laps $\Sigma A_{st} > A_s$ of one lapped bar.

BS EN 1992-1-1
fig. 8.9

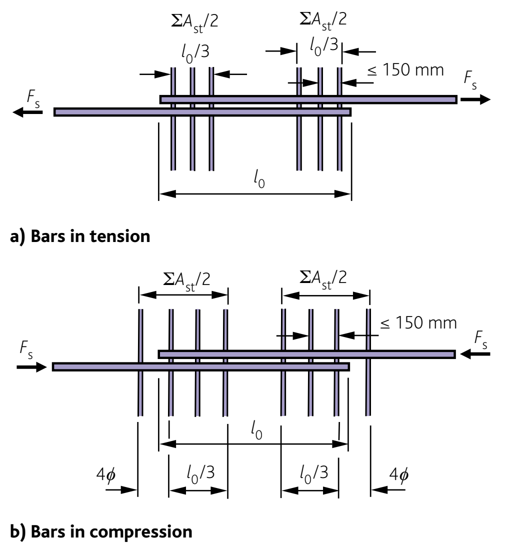


Figure 14.7
Transverse reinforcement for lapped splices

14.8.5 Lapping large bars

BS EN 1992-1-1
8.8

For bars larger than 40 mm in diameter the following additional requirements apply:

- Bars should be anchored using mechanical devices. As an alternative they may be anchored as straight bars, but links should be provided as confining reinforcement.
- Bars should not be lapped except in sections with a minimum dimension of 1 m or where the stress is not greater than 80% of the ultimate strength.
- In the absence of transverse compression, transverse reinforcement, in addition to that required for other purposes, should be provided in the anchorage zone at spacing not exceeding 5 times the diameter of the longitudinal bar. The arrangement should comply with Figure 14.8.

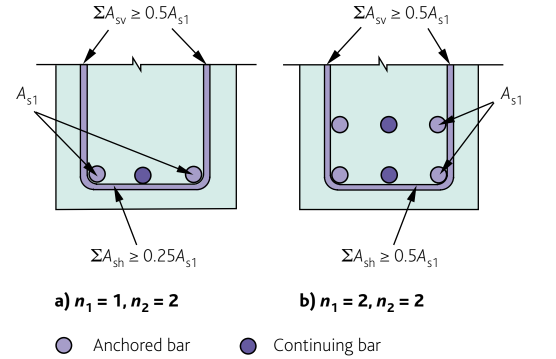


Figure 14.8 **Additional reinforcement in an anchorage for large diameter bars where there is no transverse compression**

Note

n_1 = number of layers, n_2 = number of bars

BS EN 1992-1-1 fig. 8.11

15 Detailing – particular requirements

15.1 General

BS EN 1992-1-1
9.1

This section gives particular requirements for detailing of structural elements. These are in addition to those outlined in Sections 13 and 14.

15.2 Beams

BS EN 1992-1-1
9.2.1.1

15.2.1 Longitudinal bars

The area of longitudinal reinforcement shall not be taken as less than $A_{s,min}$

$A_{s,min} = 0.26(f_{ctm}/f_{yk})\, b_t d \geq 0.0013 b_t d$

where

f_{ctm} = mean axial tensile strength of concrete (see Table 3.1)
f_{yk} = characteristic yield strength of reinforcement
b_t = mean width of the tension zone; for a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value b_t
d = effective depth

Values of $A_{s,min}$ for various strength classes are provided in Table 15.1

The cross-sectional area of tension or compression reinforcement is $0.04A_c$ outside lap locations.

BS EN 1992-1-1
9.2.1.2(3)

Any compression longitudinal reinforcement which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than 15 times the diameter of the longitudinal bar.

PD 6687-2
10.2

Where the design of a section has included the contribution of any longitudinal compression reinforcement in the resistance calculation, such longitudinal compression reinforcement should be effectively restrained by transverse reinforcement. Effective restraint may be achieved by satisfying all of the following conditions.

- Links should be so arranged that every corner and alternate bar or group in an outer layer of compression reinforcement is held in place by a link anchored in accordance with Figure 14.2 a) or b).
- All other compression reinforcement should be within 150 mm of a bar held in place by a link.
- The minimum size of the transverse reinforcement and links, where necessary, should be not less than 6 mm or one quarter of the diameter of the longitudinal bars, whichever is greater.

Table 15.1
Minimum area of longitudinal reinforcement as a proportion of $b_t d$

Strength class	C25/30	C28/35	C30/37	C32/40	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85
$A_{s,min}$ as a % of $b_t d$	0.13	0.14	0.15	0.16	0.17	0.18	0.20	0.21	0.22	0.23	0.24

BS EN 1992-1-1
9.2.1.1(4)

For members prestressed with permanently unbonded tendons or with external prestressing cables, it should be verified that the ultimate bending capacity is larger than the flexural cracking moment. A capacity of 1.15 times the cracking moment is sufficient.

15.2.2 Curtailment

Sufficient reinforcement should be provided at all sections to resist the envelope of the acting tensile force. The resistance of bars within their anchorage lengths may be taken into account assuming linear variation of force.

BS EN 1992-1-1 9.2.1.3

The longitudinal tensile forces in the bars include those arising from bending moments and those from the truss model for shear. As may be seen from Figure 15.1, those forces from the truss model for shear may be accommodated by displacing the location where a bar is no longer required for bending moment by a distance of a_l where

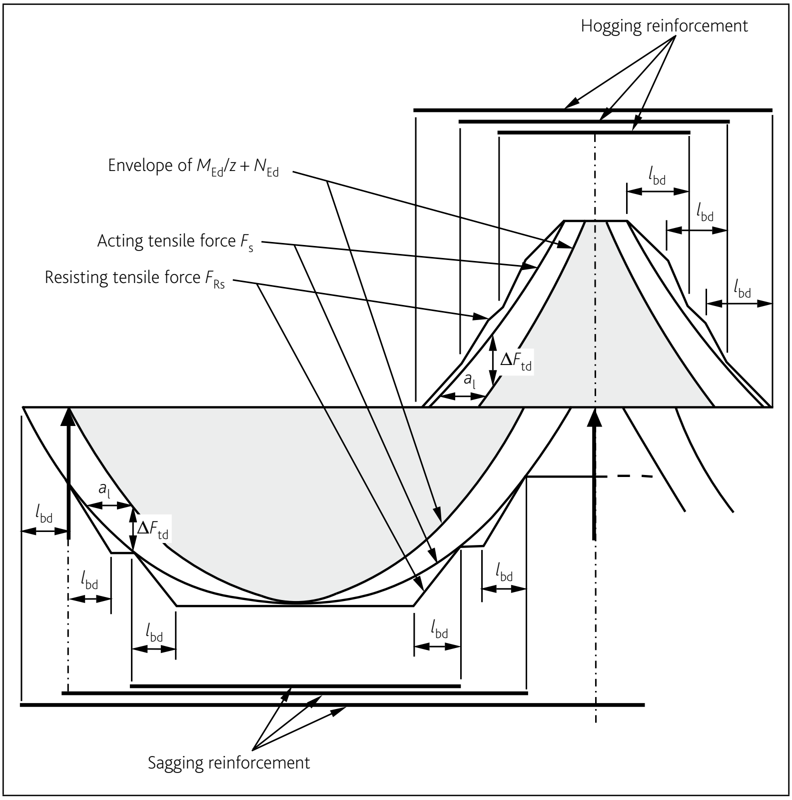
$a_l = z(\cot\theta - \cot\alpha)/2$

where

θ = strut angle used for shear calculations (see Figure 7.3)

α = angle of the shear reinforcement to the longitudinal axis (see Figure 7.3)

For all but high shear $\cot\theta = 2.5$; for vertical links $\cot\alpha = 0$; so generally $a_l = 1.25z$.



BS EN 1992-1-1 fig. 9.2

Figure 15.1
Illustration of the curtailment of longitudinal reinforcement, taking into account the effect of inclined cracks and the resistance of reinforcement within anchorage lengths

15.2.3 Top reinforcement in end supports

In monolithic construction, (even when simple supports have been assumed in design) the section at supports should be designed for bending moment arising from partial fixity of at least 25% of the maximum bending design moment in span.

BS EN 1992-1-1 9.2.1.2(1) & NA

15.2.4 Bottom reinforcement in end supports

The area of bottom reinforcement provided at ends with little or no fixity assumed in the design should be at least 25% of the area of the steel provided in the span. The bars should be anchored to resist a force, F_{Ed}, of

$$F_{Ed} = (|V_{Ed}|a_l/z) + N_{Ed}$$

where

$|V_{Ed}|$ = absolute design value of shear force
N_{Ed} = axial force,
a_l, is as defined in Section 15.2.2

The anchorage is measured from the line of contact between the beam and the support.

15.2.5 Intermediate supports

BS EN 1992-1-1 9.2.1.2(2)

At intermediate supports of continuous beams, the total area of tension reinforcement, A_s, of a flanged cross-section should be spread over the effective width of the flange (as defined in Figure 15.2). Part of it may be concentrated over the web width.

BS EN 1992-1-1 fig. 9.1

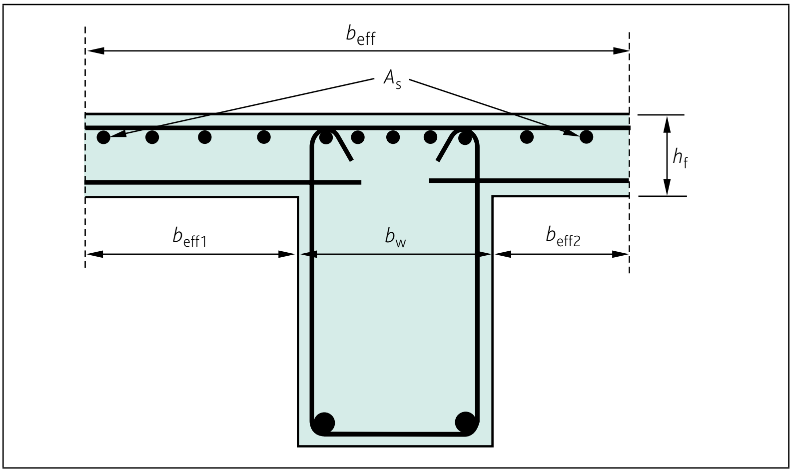


Figure 15.2
Placing of tension reinforcement in flanged cross-section

15.2.6 Shear reinforcement

BS EN 1992-1-1 9.2.2(4),(6)&(7)

Where a combination of links and bent up bars is used as shear reinforcement, at least 50% of the reinforcement required should be in the form of links. The longitudinal spacing of shear assemblies should not exceed $0.75d(1 + \cot \alpha)$, and spacing of bent-up bars should not exceed $0.6d(1 + \cot \alpha)$ where α is the inclination of the shear reinforcement to the longitudinal axis of the beam. The transverse spacing of the legs of shear links should not exceed $0.75d \leq 600$ mm.

A minimum area of shear reinforcement should be provided to satisfy the following Expression:

BS EN 1992-1-1 9.2.2(5) & NA

$$A_{sw}/(s\, b_w \sin \alpha) \geq 0.08\, f_{ck}^{0.5}/f_{yk}$$

where

s = longitudinal spacing of the shear reinforcement
b_w = breadth of the web member
α = angle of the shear reinforcement to the longitudinal axis of the member. For vertical links $\sin \alpha = 1.0$.

15.2.7 Torsion reinforcement

Where links are required for torsion, they should comply with the anchorage shown in Figure 15.3. The maximum longitudinal spacing of the torsion links $s_{l,max}$ should be:

$s_{l,max} \leq \text{MIN}\{u/8;\ 0.75d\ (1 + \cot\ \alpha);\ h;\ b\}$

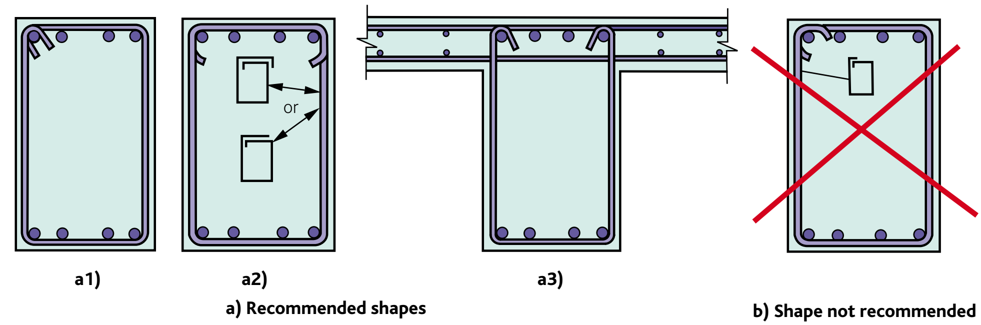
where

u = circumference of outer edge of effective cross-section (see Figure 9.1)
d = effective depth of beam
h = height of beam
b = breadth of beam

BS EN 1992-1-1 9.2.3

The longitudinal bars required for torsion should be arranged such that there is at least one bar at each corner with the others being distributed uniformly around the inner periphery of the links at a spacing not exceeding 350 mm.

BS EN 1992-1-1 fig. 9.6



Note
The second alternative for a2) should have a full lap length along the top

Figure 15.3
Examples of shapes for torsion links

15.2.8 Indirect supports

Where a beam is supported by a beam, instead of a wall or column, reinforcement should be provided to resist the mutual reaction. This reinforcement is in addition to that required for other reasons. The supporting reinforcement between two beams should consist of links surrounding the principal reinforcement of the supporting member. Some of these links may be distributed outside the volume of concrete common to the two beams. See Figure 15.4.

BS EN 1992-1-1 9.25

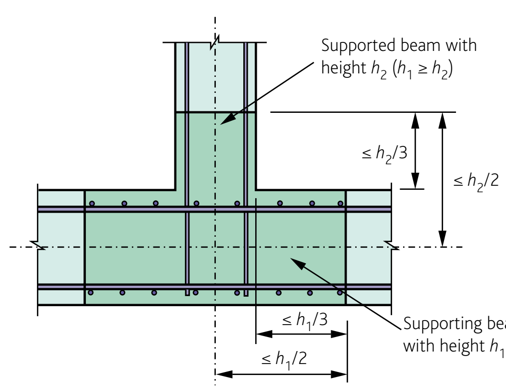


Figure 15.4
Plan section showing supporting reinforcement in the intersection zone of two beams

BS EN 1992-1-1 fig. 9.7

BS EN 1992-1-1
9.3

15.3 One-way and two-way spanning slabs

15.3.1 Main (principal) reinforcement

BS EN 1992-1-1
9.3.1.1(1)

The minimum and maximum steel percentages in the main direction in Section 15.2.1 apply.

BS EN 1992-1-1
9.3.1.1(3) & NA

The spacing of main reinforcement should not exceed $3h$ (but not greater than 400 mm), where h is the total depth of the slab. In areas with concentrated loads or areas of maximum moment these provisions become respectively $2h$ and ≤ 250 mm.

15.3.2 Secondary (distribution) reinforcement

Secondary reinforcement of not less than 20% of the principal reinforcement should be provided in one-way slabs.

BS EN 1992-1-1
9.3.1.1(3) & NA

The spacing of secondary reinforcement should not exceed $3.5h$ (but not greater than 450 mm). In areas with concentrated loads, or in areas of maximum moment, these provisions become respectively $3h$ (but not greater than 400 mm).

15.3.3 Reinforcement in slabs near supports

BS EN 1992-1-1
9.3.1.2

In simply supported slabs, half the calculated span reinforcement should continue up to the support and be anchored therein in accordance with Section 14.4.2.

Where partial fixity occurs along an edge of a slab but is not taken into account in the analysis, the top reinforcement should be capable of resisting at least 25% of the maximum moment in the adjacent span. This reinforcement should extend at least 0.2 times the length of the adjacent span measured from the face of the support. At an end support the moment to be resisted may be reduced to 15% of the maximum moment in the adjacent span.

BS EN 1992-1-1
9.3.2

15.3.4 Shear reinforcement

A slab in which shear reinforcement is provided should have a depth of at least 200 mm.

Where shear reinforcement is provided the rules for beams may be followed.

15.4 Flat slabs

15.4.1 Details at internal columns

At internal columns, unless rigorous serviceability calculations are carried out, top reinforcement with an area of $0.5A_t$ should be placed over the column in a width equal to the sum of 0.125 times the panel width on either side of the column. A_t represents the area of reinforcement required to resist full negative moment from the sum of the two half panels on each side of the column. Bottom bars (≥ 2 bars) in each orthogonal direction should be provided at internal columns and this reinforcement should pass through the column.

15.4.2 Details at edge and corner columns

Reinforcement perpendicular to a free edge required to transmit bending moments from the slab to an edge or corner column should be placed within the effective width b_e shown in Figure 15.5.

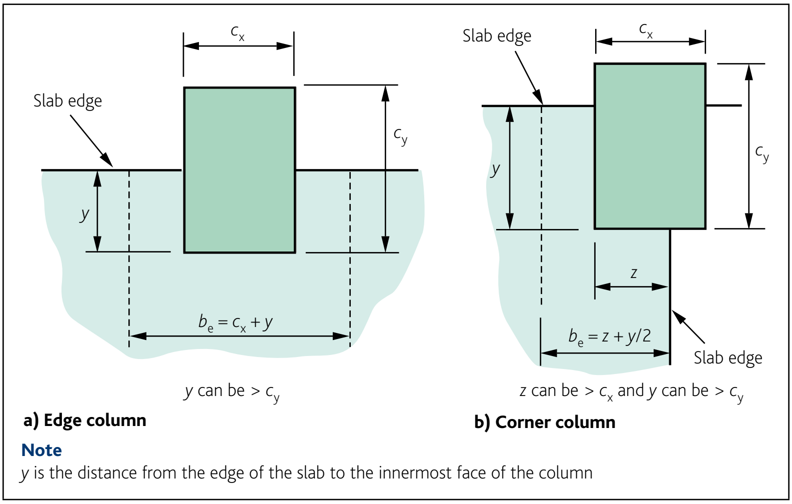


Figure 15.5
Effective width, b_e, of a flat slab

BS EN 1992-1-1 9.4.1(2) & (3)

BS EN 1992-1-1 9.4.2(1)

15.4.3 Punching shear reinforcement

Where punching shear reinforcement is required, it should be placed between the loaded area /column and 1.5*d* inside the control perimeter at which reinforcement is no longer required.

BS EN 1992-1-1 fig. 9.9

The spacing of link legs around a perimeter should not exceed 1.5*d* within the first control perimeter (2*d* from the loaded area) and should not exceed 2*d* for perimeters outside the first control perimeter (see Figure 15.6).

BS EN 1992-1-1 9.4.3

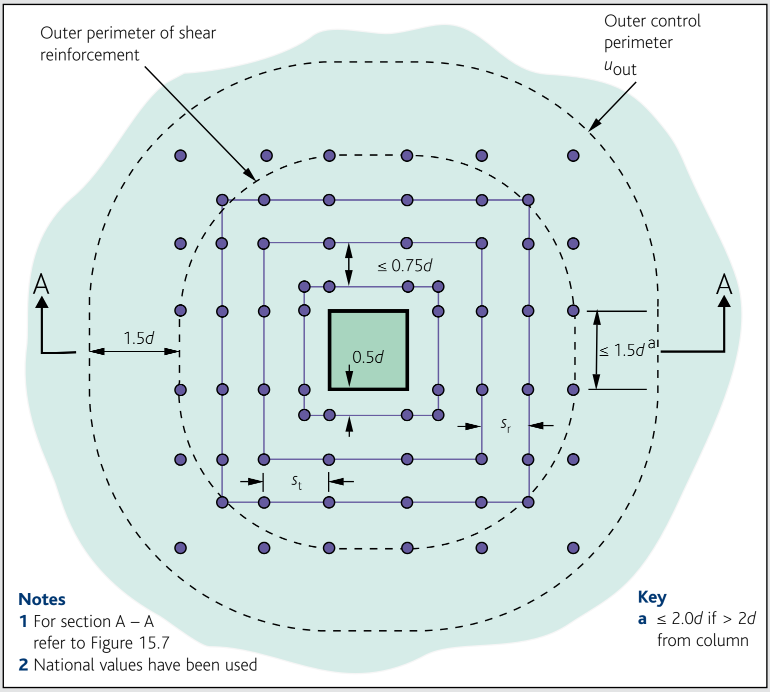


Figure 15.6
Layout of flat slab shear reinforcement

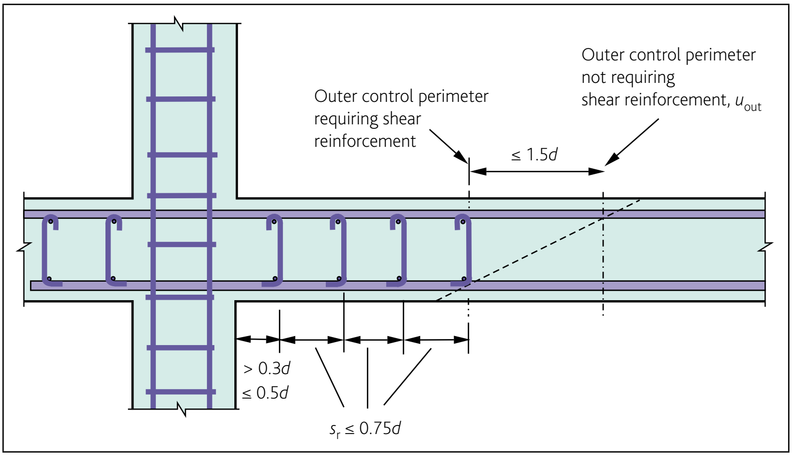


Figure 15.7
Section A – A from Figure 15.6: spacing of punching shear reinforcing links

The intention is to provide an even distribution/density of punching shear reinforcement within the zone where it is required (see Section 8.8).

BS EN 1992-1-1 fig. 9.10

BS EN 1992-1-1 6.4.5(4) & NA

The control perimeter at which shear reinforcement is not required, U_{out}, should be calculated from the following Expression:

$U_{out} = \beta V_{Ed}/(V_{Rd,c}\ d)$

The outermost perimeter of shear reinforcement should be placed not greater than 1.5 d within U_{out}.

BS EN 1992-1-1 9.4.3(2)

Where shear reinforcement is required, the area of a link leg, $A_{sw,min}$ is given by the following Expression:

$A_{sw,min}\ (1.5 \sin \alpha + \cos \alpha)/(s_r\ s_t) \geq 0.08\ f_{ck}^{0.5}/f_{yk}$

where

s_r and s_t = spacing of shear reinforcement in radial and tangential directions respectively (see Figure 15.6)

15.5 Columns

15.5.1 Longitudinal reinforcement

BS EN 1992-1-1 9.5.2 & NA

Longitudinal bars should have a diameter of not less than 12 mm.

The total amount of longitudinal reinforcement should not be less than $A_{s,min}$:

$A_{s,min} \geq \text{MAX}\{0.1N_{Ed}/f_{yd};\ 0.002A_c\}$

where

N_{Ed} = design axial compression force
f_{yd} = design yield strength of the reinforcement
A_c = cross-sectional area of concrete

The area of longitudinal reinforcement should not exceed $0.04A_c$ outside lap locations. This limit should be increased to $0.08A_c$ at laps.

15.5.2 Transverse reinforcement (links)

BS EN 1992-1-1 9.5.3(1) & NA

The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the diameter of the longitudinal bars, whichever is greater.

The spacing of the transverse reinforcement along the column should not exceed:

- 20 times the diameter of the longitudinal bar, or
- the lesser dimension of the column, or
- 400 mm.

BS EN 1992-1-1 9.5.3(3) & (4)

The maximum spacing required above should be reduced by a factor of 0.6:

- In sections within a distance equal to the larger dimension of the column cross-section above and below a beam or slab.
- Near lapped joints, if the maximum diameter of the longitudinal bars is greater than 14 mm. A minimum of 3 bars evenly placed in the lap length is required.

Where the direction of the longitudinal bars changes, the spacing of transverse reinforcement should be calculated taking into account the transverse forces involved. These effects may be ignored if the change of direction is less than or equal to 1 in 12.

BS EN 1992-1-1 9.5.3(5)

Where the design of a section has included the contribution of any longitudinal compression reinforcement in the resistance calculation, such longitudinal compression reinforcement should be effectively restrained by transverse reinforcement. Effective restraint may be achieved by satisfying all of the following conditions:

- Links should be so arranged that every corner and alternate bar or group in an outer layer of compression reinforcement is held in place by a link anchored in accordance with Figure 14.2 a) or b).

PD 6687-2 10.2

- All other compression reinforcement should be within 150 mm of a bar held in place by a link.
- The minimum size of the transverse reinforcement and links, where necessary, should be not less than 6 mm or one quarter of the diameter of the longitudinal bars, whichever is greater.

For circular columns, where the longitudinal reinforcement is located round the periphery adequate lateral support is provided by a circular tie passing round the bars or groups.

15.6 Walls

15.6.1 Vertical reinforcement

The area of vertical reinforcement should lie between $0.002A_c$ and $0.04A_c$ outside laps locations. This limit may be doubled at laps.

BS EN 1992-1-1 9.6.2 & NA

The distance between two adjacent bars should not exceed 3 times the wall thickness or 400 mm, whichever is the lesser.

15.6.2 Horizontal reinforcement

Horizontal reinforcement running parallel to the faces of the wall should be provided at each surface. It should not be less than either 25% of the vertical reinforcement or $0.001A_c$, whichever is greater.

BS EN 1992-1-1 9.6.3 & NA

The spacing between two adjacent horizontal bars should not be greater than 400 mm.

15.6.3 Transverse reinforcement

In any part of a wall where the total area of the vertical reinforcement in the two faces exceeds $0.02A_c$, transverse reinforcement in the form of links should be provided in accordance with the rules for columns.

BS EN 1992-1-1 9.6.4

Where the main reinforcement is placed nearest to the wall faces, transverse reinforcement should also be provided in the form of links with at least 4 per m^2 of wall area. Transverse reinforcement need not be provided where welded mesh and bars of diameter $\phi \leq 16$ mm are used with concrete cover larger than 2ϕ.

15.7 Pile caps

BS EN 1992-2
9.8.1 (103) & NA

The distance from the outer edge of the pile to the edge of the pile cap should be such that the tie forces can be properly anchored. The expected deviation of the pile on site should be taken into account.

Reinforcement in a pile cap should be calculated either by using strut-and-tie or flexural methods as appropriate. The main tensile reinforcement to resist the action effects should be concentrated in stress zones between the tops of the piles. The minimum diameter of bars should be 12 mm.

PD 6687-2
10.4

Where the distance between the edge of a pile and a pier is less than $2d$, some of the shear force in the pile cap will be transmitted directly between the pier and the pile via a strutting action. The basic punching perimeter cannot be constructed without encompassing a part of the support as shown for the $2d$ perimeter in Figure 15.8. In such cases, it is recommended that the tension reinforcement is provided with a full anchorage beyond the line of the pile centres and that the shear design takes into consideration of the following in addition to other verifications required by BS EN 1992-2: 2005.

- **Flexural shear on plane passing across the full width of the pile cap**. Flexural shear should be checked on planes passing across the full width of the pile cap, such as the flexural shear plane in Figure 15.8. Shear enhancement should be taken into account by an increase to the concrete resistance and not by a reduction in the shear force. Where the spacing of the pile centres is less than or equal to 3 pile diameters, the short shear span enhancement may be applied over the whole section. Where the spacing is greater than this, the enhancement may only be applied on strips of width 3 pile diameters centred on each pile. The shear span α_v should be taken as the distance between the face of the column or wall and the nearer edge of the piles plus 20% of the pile diameter.
- **Maximum permissible shear stress for punching**. The maximum permissible shear stress at the face of the piles and piers should be checked in accordance with Section 8.6. The shear perimeter for corner piles should be the pile perimeter, or a perimeter passing partially around the pile and extending out to the free edges of the pile cap, whichever is less.
- **Punching resistance of corner piles.** Corner piles should be checked for punching resistance at a $2d$ perimeter (without support enhancement) ignoring the presence of the pier or support and any vertical reinforcement within it.

Further guidance is given in Hendy & Smith [20].

PD 6687-2
fig. 6

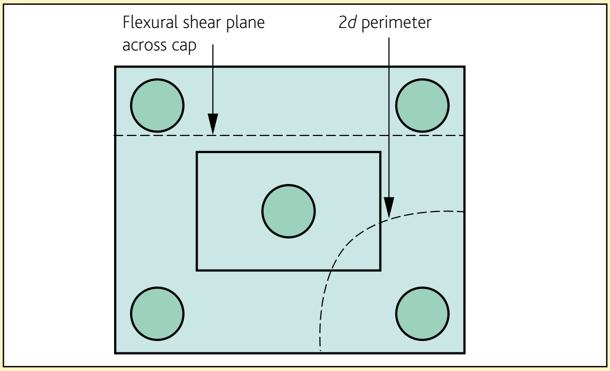


Figure 15.8
Corner piles within 2*d* of a column base

15.8 Bored piles

Bored piles should have the minimum reinforcement shown in Table 15.2. A minimum of six longitudinal bars with diameter of at least 16 mm should be provided with a maximum spacing of 200 mm around the periphery of the pile. The detailing should comply with BS EN 1536[23].

BS EN 1992-1-1
9.85

Table 15.2
Recommended minimum longitudinal reinforcement in cast-in-place bored piles

Area of cross-section of the pile (A_c)	$A_c \leq 0.5$ m²	0.5 m² ≤ 1.0 m²	$A_c > 1.0$ m²
Minimum area of longitudinal reinforcement ($A_{s,bpmin}$)	$\geq 0.005A_c$	≥ 25 cm²	$\geq 0.0025A_c$

BS EN 1992-1-1
table 9.6.N

15.9 Requirements for voided slabs

PD 6687-2 gives the following guidance for the design of voided slab bridge decks cast in situ.

15.9.1 Transverse shear

The effects of cell distortion due to transverse shear should be considered. In particular:

PD 6687-2
10.5.2

- The increased stresses in the transverse reinforcement and shear links due to cell distortion resulting from transverse shear should be calculated by an appropriate analysis (e.g. an analysis based on the assumption that the transverse action acts in a manner similar to a Vierendeel frame).
- The resistance of the flanges and webs to the local moments produced by the transverse shear effects should be verified.

The top and bottom flanges should be designed as solid slabs, each to carry a part of the global transverse shear force proportional to the flange thickness.

15.9.2 Longitudinal shear

The longitudinal ribs between the voids should be designed as beams to resist the shear forces in the longitudinal direction, including any shear due to torsional effects.

PD 6687-2
10.5.3

15.9.3 Punching

Punching of wheel loads through the top flange of decks with circular voids will generally need to be considered for unusually thin flanges, typically those with void diameter to slab depth ratios of greater than 0.75.

PD 6687-2
10.5.4

15.9.4 Transverse reinforcement

In the absence of a detailed analysis, the minimum transverse reinforcement provided should be as follows:

PD 6687-2
10.5.5

- In the predominantly tensile flange either 1500 mm²/m or 1% of the minimum flange section, whichever is the lesser.
- In the predominantly compressive flange either 1000 mm²/m or 0.7% of the minimum flange section, whichever is the lesser.

The spacing of the transverse reinforcement should not exceed twice the minimum flange thickness.

In skew voided slabs, it is preferable for the transverse steel to be placed perpendicular to the voids and the longitudinal steel to be placed parallel to the voids.

15.10 Prestressing

15.10.1 Arrangement of pre-tensioned tendons

BS EN 1992-1-1 8.10.1.2(1)

The minimum clear horizontal and vertical spacing of individual pre-tensioned tendons should be in accordance with that shown in Figure 15.9.

BS EN 1992-1-1 fig. 8.14

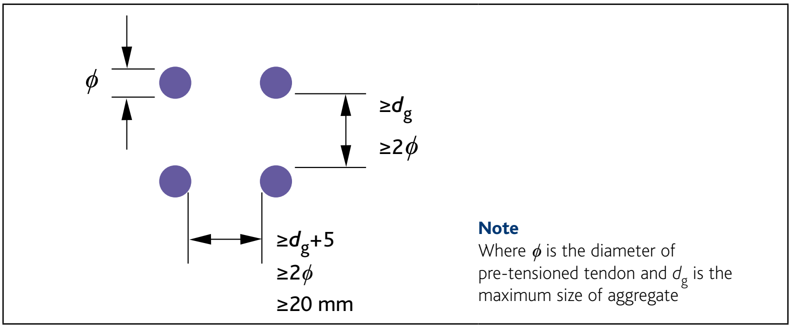


Figure 15.9
Minimum clear spacing between pre-tensioned tendons

15.10.2 Arrangement of post-tensioned ducts

BS EN 1992-1-1 8.10.1.3 (3)

The minimum clear spacing of between ducts should be in accordance with that shown in Figure 15.10.

PD 6687-2 9.2

In the absence of the provision of reinforcement between ducts to prevent splitting of the concrete, designed in accordance with the strut and tie rules (see Section 10), the minimum centre to centre duct spacings should be as given in Table 15.3. These spacings are given in BS 5400-4: 1990[24].

BS EN 1992-1-1 fig. 8.15

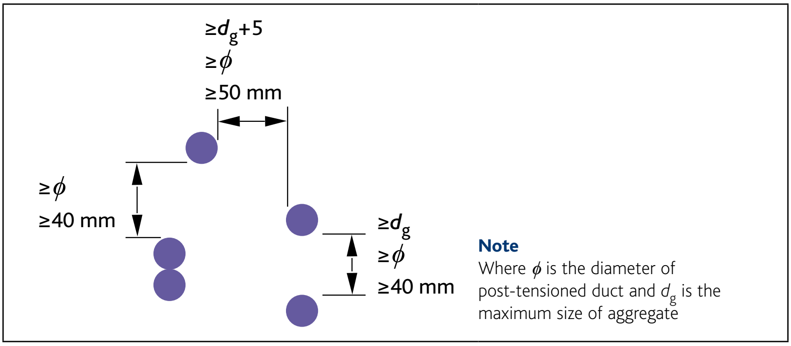


Figure 15.10
Minimum clear spacing between post-tensioned ducts

PD 6687-2 table 3

Table 15.3
Minimum spacing of post-tensioning ducts (mm)

Radius of curvature of duct (m)	Duct internal diameter (mm)															
	19	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
	Tendon force (kN)															
	296	387	960	1337	1920	2640	3360	4320	5183	6019	7200	8640	9424	10336	11248	13200
2	110	140	350	485	700	960										
4	55	70	175	245	350	480	610	785	940					Radii not normally used		
6	38	60	120	165	235	320	410	525	630	730	870	1045				
8			90	125	175	240	305	395	470	545	655	785	855	940		
10			80	100	140	195	245	315	375	440	525	630	685	750	815	
12						160	205	265	315	365	435	525	570	625	680	800
14						140	175	225	270	315	375	450	490	535	585	785
16							160	195	235	275	330	395	430	470	510	600
18								180	210	245	290	350	380	420	455	535
20									200	220	265	315	345	375	410	480
22											240	285	310	340	370	435
24												265	285	315	340	400
26												260	280	300	320	370
28																345
30																340
40	38	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340

Notes

1 The tendon force shown is the maximum normally available for the given size of duct (taken as 80% of the characteristic strength of the tendon).

2 Values less than 2 × duct internal diameter are not included.

15.11 Connections

15.11.1 Connections transmitting compression

BS EN 1992-1-1 10.9.4.3(3)

Connections without bedding material (dry connections) should only be used where an appropriate quality of workmanship can be achieved. The average bearing stress should not exceed $0.3f_{cd}$ where

$f_{cd} = 0.85\, f_{ck}/1.5$

In the absence of other specifications the following value can be used for the bearing strength of other connections

$f_{Rd} = f_{bed} \leq 0.85\, f_{cd}$

where

BS EN 1992-1-1 10.9.5.2(2)

f_{cd} = lower of the design strengths for supported and supporting member

$= 0.85\, f_{ck}/1.5$

f_{bed} = design strength of the bedding material

15.12 Bearings

BS EN 1992-1-1 10.9.5.1(4)

Bearings shall be designed and detailed to ensure correct positioning taking into account the production and assembling deviations.

BS EN 1992-1-1 10.9.5.2(1)

The nominal length, a, of a simple bearing as shown in Figure 15.11 may be calculated as:

$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^{\,2} + \Delta a_3^{\,2}}$$

where

a_1 = net bearing length with regard to bearing stress $a_1 = F_{Ed}/(b_1 f_{Rd})$ but not less than minimum values in Table 15.4
F_{Ed} = design value of support reaction
b_1 = net bearing width (see below)
f_{Rd} = design value of bearing strength (see Section 15.11.1)
a_2 = distance assumed ineffective beyond outer end of supporting member, see Figure 15.11 and Table 15.5
a_3 = similar distance for supported member, see Figure15.11 and Table 15.6
Δa_2 = allowance for deviations for the distance between supporting members, see Table 15.7
Δa_3 = allowance for deviations for the length of supporting member
= $l_n/2500$
l_n = length of member

BS EN 1992-1-1 10.9.4.7(1)

The effective bearing length a_1 is controlled by a distance, d, (see Figure 15.12) from the edge of the respective elements

where

d_i = $c_i + \Delta a_i$ with horizontal loops or otherwise end-anchored bars
d_i = $c_i + \Delta a_i + r_i$ with vertically bent bars

where

c_i = concrete cover
Δa_i = deviation
r_i = bend radius

BS EN 1992-1-1 10.9.5.2(3)

If measures are taken to obtain a uniform distribution of the bearing pressure, e.g. with mortar, neoprene or similar pads, the design bearing width b_1 may be taken as the actual width of the bearing. Otherwise, and in the absence of a more accurate analysis, b_1 should not be greater than to 600 mm.

BS EN 1992-1-1 fig. 10.6

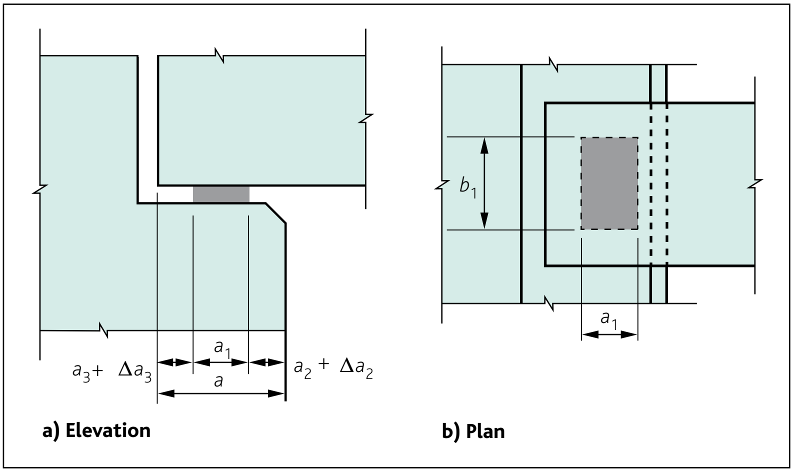


Figure 15.11
Example of bearing with definitions

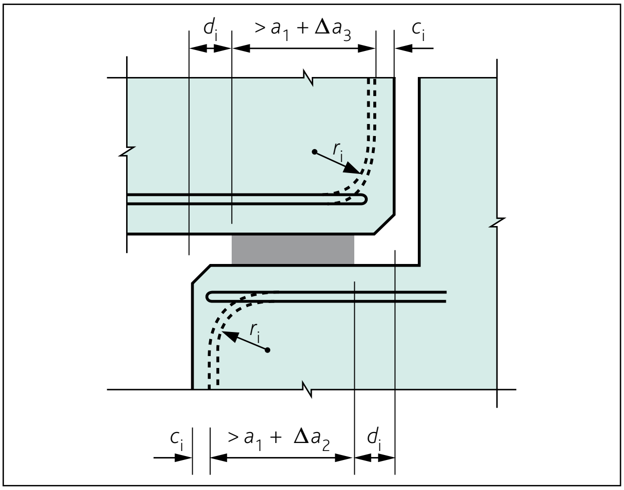


Figure 15.12
Example of detailing of reinforcement in support

BS EN 1992-1-1 fig.10.5

Table 15.4
Minimum value of a_1 (mm)

Relative bearing stress, σ_{Ed} / f_{cd}	≤0.15	0.15 – 0.4	>0.4
Line supports (floors, roofs)	25	30	40
Ribbed floors and purlins	55	70	80
Concentrated supports (beams)	90	110	140

BS EN 1992-1-1 table 10.2

Table 15.5
Distance a_2 (mm) assumed ineffective from outer end of supporting member

Support material and type		Relative bearing stress, σ_{Ed} / f_{cd}		
		≤0.15	0.15 – 0.4	>0.4
Steel	line	0	0	10
	concentrated	5	10	15
Reinforced concrete ≥ C30	line	0	10	15
	concentrated	10	15	25
Plain concrete and reinforced concrete < C30	line	10	15	25
	concentrated	15	25	35
Brickwork	line	10	15	**a**
	concentrated	20	25	**a**

Key
a Concrete padstone should be used in these cases

BS EN 1992-1-1 table 10.3

Table 15.6
Distance a_3 (mm) assumed ineffective beyond outer end of supported member

Detailing of reinforcement	Support	
	Line	Concentrated
Continuous bars over support (restrained or not)	0	0
Straight bars, horizontal, close to end of member	5	15, but not less than end cover
Tendons or straight bars exposed at end of member	5	15
Vertical loop reinforcement	15	End cover + inner radius of bending

BS EN 1992-1-1 table 10.4

BS EN 1992-1-1
table 10.5

Table 15.7
Allowance Δa_2 for tolerances for the clear distance between the faces of the supports

Support material	Δa_2
Steel or precast concrete	$10 \leq l/1200 \leq 30$ mm
Brickwork or cast in-situ concrete	$15 \leq l/1200 + 5 \leq 40$ mm

Note
l = span length

16 Design for the execution stages

For bridges built in stages, account of the construction procedure should be considered at serviceability and ultimate limit states.

BS EN 1992-2 113

Serviceability criteria for the completed structure need not be applied to intermediate execution stages, provided that durability and final appearance of the completed structure are not affected (e.g. deformations). Even for bridges or elements of bridges in which the limit-state of decompression is checked under the quasi-permanent or frequent combination of actions on the completed structure, tensile stresses less than 1.0 $f_{ctm}(t)$ under the quasi-permanent combination of actions during execution are permitted. For bridges or elements of bridges in which the limit-state of cracking is checked under frequent combination on the completed structure, the limit state of cracking should be verified under the quasi-permanent combination of actions during execution.

BS EN 1992-2 113.3.2

17 Design aids

The following text, tables and figures have been derived from Eurocode 2 and are provided as an aid to designers in the UK.

17.1 Design for bending

- Determine whether $K \leq K'$ or not (i.e. whether under-reinforced or not).

  where

  $K = M_{Ed}/(bd^2 f_{ck})$

  where

  d = effective depth = h – cover – ϕ_{link} – $\phi/2$

  b = width of section

  Section 4

  For rectangular sections K' may be determined from Table 17.1 or, for slabs only, Table 17.2 may be used . K' is dependent on the concrete strength and the redistribution ratio used.

For non-rectangular section x/d limits given in Table 17.1 may be used.

- If $K \leq K'$, section is **under-reinforced.**

  For rectangular sections:

  $A_{s1} = M_{Ed}/f_{yd}z$

  where

  A_{s1} = area of tensile reinforcement

  M_{Ed} = design moment

  $f_{yd} = f_{yk}/\gamma_S = 500/1.15 = 434.8$ MPa

  $z = d[0.5 + 0.5(1 - 3.53K/\eta)^{0.5}] \leq 0.95d$

  η = factor defining effective strength

  $= 1.0 - (f_{ck} - 50)/200$ (see Table 6.1)

  For flanged beams where $x < 1.25h_f$,

  $A_{s1} = M_{Ed}/f_{yd}z$

  x = depth to neutral axis

  h_f = thickness of flange

  For flanged beams where $x \geq 1.25h_f$, refer to *How to design concrete structures using Eurocode 2: Beams*[25]

- If $K > K'$, section is **over-reinforced** and requires compression reinforcement.

  $A_{s2} = (M_{Ed} - M'_{Ed})/f_{sc}(d - d_2)$

  where

  A_{s2} = compression reinforcement

  $M'_{Ed} = K'bd^2f_{ck}$

  $f_{sc} = 700(x_u - d_2)/x_u \leq f_{yd}$

  where

  d_2 = effective depth to compression reinforcement

  $x_u = (\delta - \lambda/z)d$

  δ = redistribution ratio

  When $f_{yk} = 500$ MPa, $f_{sc} = f_{yd}$ unless $d_2/x_u \geq 0.379$.

  Total area of tension steel $A_{s1} = M'_{Ed}/(f_{yd}z) + A_{s2}\,f_{sc}/f_{yd}$

Table 17.1
Limiting values of K' and x_u/d

Percentage redistribution	δ (Redistribution ratio)	f_{ck}	50	55	60	70
		η	1.000	0.975	0.950	0.900
		λ	0.800	0.788	0.775	0.750
		ε_{cu2}	0.0035	0.0031	0.0029	0.0027
		k_1 or k_3	0.44	0.54	0.54	0.54
		$k_2 = k_4$	1.251	1.310	1.357	1.409
0	1.00	K'	0.167	0.132	0.123	0.110
		x_u/d	0.448	0.351	0.339	0.326
5	0.95	K'	0.155	0.119	0.111	0.099
		x_u/d	0.408	0.313	0.302	0.291
10	0.90	K'	0.142	0.107	0.099	0.088
		x_u/d	0.368	0.275	0.265	0.256
15	0.85	K'	0.129	0.093	0.087	0.077
		x_u/d	0.328	0.237	0.228	0.220

Table 17.2
Limiting values of K' and x_u/d for slabs

Percentage redistribution	δ (Redistribution ratio)	f_{ck}	50	55	60	70
		η	1.000	0.975	0.950	0.900
		λ	0.800	0.788	0.775	0.750
		ε_{cu2}	0.0035	0.0031	0.0029	0.0027
		k_1 or k_3	0.4	0.4	0.4	0.4
		$k_2 = k_4$	1.000	1.048	1.086	1.127
0	1.00	K'	0.207	0.193	0.181	0.163
		x_u/d	0.600	0.573	0.553	0.532
5	0.95	K'	0.194	0.181	0.170	0.152
		x_u/d	0.550	0.525	0.507	0.488
10	0.90	K'	0.181	0.169	0.158	0.141
		x_u/d	0.500	0.477	0.461	0.444
15	0.85	K'	0.167	0.155	0.145	0.130
		x_u/d	0.450	0.429	0.415	0.399
20	0.80	K'	0.152	0.141	0.132	0.118
		x_u/d	0.400	0.382	0.368	0.355
25	0.75	K'	0.136	0.126	0.118	0.105
		x_u/d	0.350	0.334	0.322	0.311
30	0.70	K'	0.120	0.111	0.103	0.092
		x_u/d	0.300	0.286	0.276	0.266

Section 7.2

17.2 Design for beam shear

17.2.1 Requirement for shear reinforcement

If $v_{Ed} > v_{Rd,c}$ then shear reinforcement is required

where

v_{Ed} = $V_{Ed}/b_w d$, for sections *without* shear reinforcement (i.e. slabs)

$v_{Rd,c}$ = shear resistance without shear reinforcement, from Table 17.3

Table 17.3

Shear resistance without shear reinforcement, $v_{Rd,c}$ (MPa)

$\rho_l = A_{sl}/b_w d$	Effective depth d (mm)										
	≤ 200	225	250	275	300	350	400	450	500	600	750
0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.69	0.68	0.65	0.64	0.62	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.74	0.73	0.71	0.69	0.67	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.79	0.78	0.75	0.73	0.71	0.70	0.67	0.65
1.75%	0.90	0.87	0.85	0.83	0.82	0.79	0.77	0.75	0.73	0.71	0.68
≥ 2.00%	0.94	0.91	0.89	0.87	0.85	0.82	0.80	0.78	0.77	0.74	0.71

Notes

1 Table derived from BS EN 1992-1-1 and UK National Annex.

2 Table created for f_{ck} = 30 MPa assuming vertical links.

3 For $\rho_l \geq 0.4\%$ and
f_{ck} = 25 MPa, apply factor of 0.94
f_{ck} = 35 MPa, apply factor of 1.05
f_{ck} = 40 MPa, apply factor of 1.10
f_{ck} = 45 MPa, apply factor of 1.14
f_{ck} ≥ 50 MPa, apply factor of 1.19

Section 7.3.2

17.2.2 Section capacity check

If $v_{Ed,z} > v_{Rd,max}$ then section size is inadequate

where

$v_{Ed,z}$ = $V_{Ed}/b_w z = V_{Ed}/b_w 0.9d$, for sections *with* shear reinforcement

$v_{Rd,max}$ = capacity of concrete struts expressed as a stress in the vertical plane

= $V_{Rd,max}/b_w z$

= $V_{Rd,max}/b_w 0.9d$

$v_{Rd,max}$ can be determined from Table 17.4, initially checking at $\cot \theta = 2.5$. Should it be required, a greater resistance may be assumed by using a larger strut angle, θ.

17.2.3 Shear reinforcement design

Section 7.3.3

$A_{sw}/s \geq v_{Ed,z} b_w / f_{ywd} \cot \theta$

where

A_{sw} = area of shear reinforcement (vertical links assumed)

s = spacing of shear reinforcement

$v_{Ed,z}$ = $V_{Ed}/b_w z$, as before

b_w = breadth of the web

f_{ywd} = f_{ywk}/γ_S = design yield strength of shear reinforcement

Alternatively, A_{sw}/s per metre width of b_w may be determined from Figure 17.1a) or 17.1b) as indicated by the blue arrows in Figure 17.1a). These figures may also be used to estimate the value of $\cot \theta$.

Beams are subject to a minimum shear link provision. Assuming vertical links, $A_{sw,min}/sb_w \geq 0.08\, f_{ck}^{0.5}/f_{yk}$ (see Table 17.5).

Table 17.4
Capacity of concrete struts expressed as a stress, $v_{Rd,max}$, where $z = 0.9d$

f_{ck}		$v_{Rd,max}$ (MPa)						ν
	cot θ	2.50	2.14	1.73	1.43	1.19	1.00	
	θ	21.8°	25°	30°	35°	40°	45°	
25		3.10	3.45	3.90	4.23	4.43	4.50	0.540
30		3.64	4.04	4.57	4.96	5.20	5.28	0.528
35		4.15	4.61	5.21	5.66	5.93	6.02	0.516
40		4.63	5.15	5.82	6.31	6.62	6.72	0.504
45		5.09	5.65	6.39	6.93	7.27	7.38	0.492
≥50		5.52	6.13	6.93	7.52	7.88	8.00	0.480

Notes
1 Table derived from BS EN 1992-1-1 and UK National Annex assuming vertical links, i.e. cot $\alpha = 0$
2 $\nu = 0.6[1 - (f_{ck}/250)]$
3 $v_{Rd,max} = \nu f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$

Section 15.2.6

Table 17.5
Values of $A_{sw,min}/sb_w \times 10^3$ for beams for vertical links and f_{yk} = 500 MPa

Concrete class	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	≥C50/60
$A_{sw,min}/sb_w \times 10^3$	0.72	0.80	0.88	0.95	1.01	1.07	1.13

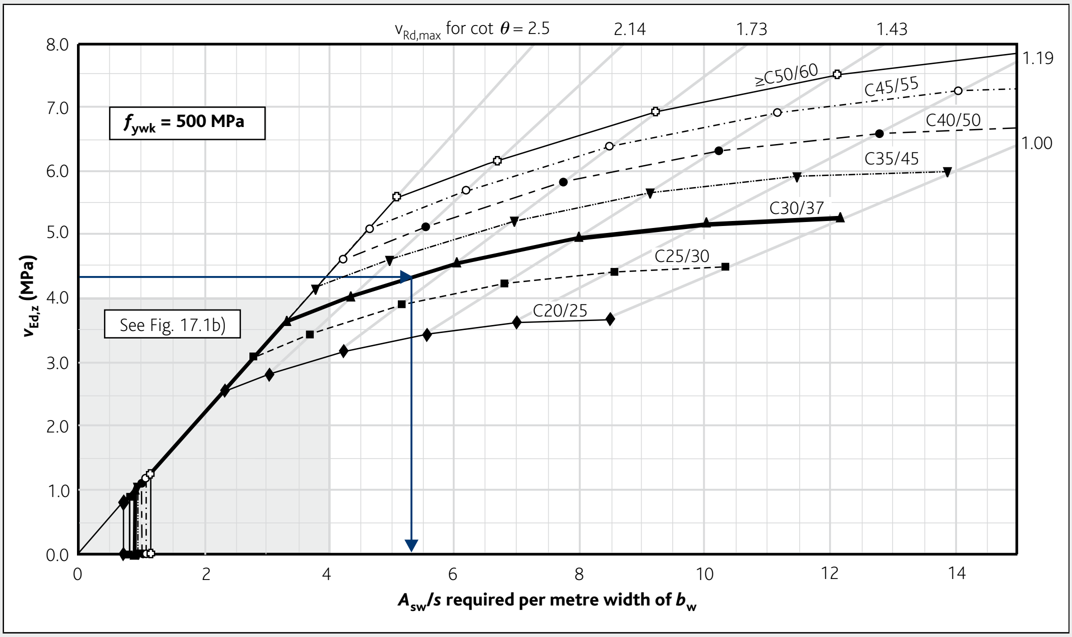


Figure 17.1a)
Diagram to determine A_{sw}/s required (for beams with high shear stress)

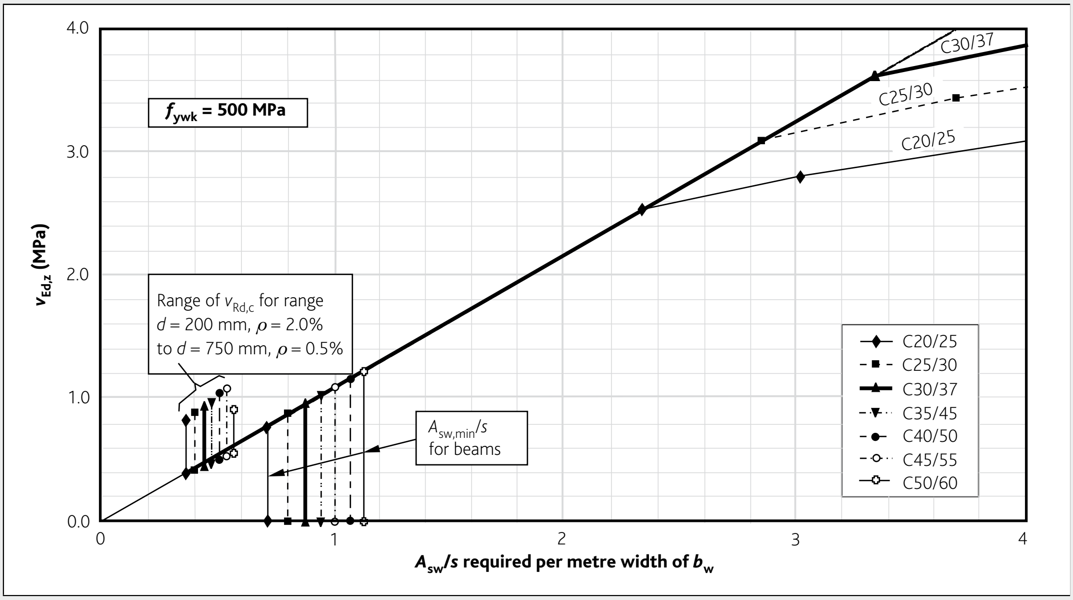


Figure 17.1b)
Diagram to determine A_{sw}/s required (for slabs and beams with low shear stress)

17.3 Design for punching shear

Section 8.21

Determine if punching shear reinforcement is required, initially at u_1, then if necessary at subsequent perimeters, u_i.

If $v_{Ed} > v_{Rd,c}$ then punching shear reinforcement is required

where

$v_{Ed} = \beta V_{Ed}/u_i d$

where

β = factor dealing with eccentricity (see Section 8.2)
V_{Ed} = design value of applied shear force
u_i = length of the perimeter under consideration (see Sections 8.3, 8.7 and 15.4.3)
d = mean effective depth
$v_{Rd,c}$ = shear resistance without shear reinforcement (see Table 17.3)

For vertical shear reinforcement

Section 8.5

$(A_{sw}/s_r) = u_1(v_{Ed} - 0.75\ v_{Rd,c})/(1.5\ f_{ywd,ef})$

where

Section 15.4.3

A_{sw} = area of shear reinforcement in one perimeter around the column. For $A_{sw,min}$ see Section 15.4.3
s_r = radial spacing of perimeters of shear reinforcement
u_1 = basic control perimeter (see Figures 8.3 and 8.4)
$f_{ywd,ef}$ = effective design strength of reinforcement = $(250 + 0.25d) \leq f_{ywd}$. For Class 500 shear reinforcement see Table 17.6

Table 17.6
Values of $f_{ywd,ef}$ for Class 500 reinforcement

d	150	200	250	300	350	400	450
$f_{ywd,ef}$	287.5	300	312.5	325	337.5	350	362.5

17.4 Design for axial load and bending

17.4.1 General

In columns, design moments M_{Ed} and design applied axial force N_{Ed} should be derived from analysis, consideration of imperfections and, where necessary, 2nd order effects (see Section 5.6).

17.4.2 Design by calculation

Section 5.6

Assuming two layers of reinforcement, A_{s1} and A_{s2}, the total area of steel required in a column, A_s, may be calculated as shown below.

Section 6.2.2

- For axial load

$$A_{sN}/2 = (N_{Ed} - \alpha_{cc}\eta f_{ck} b d_c/\gamma_C)/(\sigma_{sc} - \sigma_{st})$$

where

A_{sN} = total area of reinforcement required to resist axial load using this method.
A_{sN} = $A_{s1} + A_{s2}$ and $A_{s1} = A_{s2}$

where

$A_{s1}(A_{s2})$ = area of reinforcement in layer 1 (layer 2) (see Figure 6.3)
N_{Ed} = design value of axial force
α_{cc} = 0.85
η = 1 for ≤ C50/60 (see Table 6.1 when f_{ck} > 50)
b = breadth of section
d_c = effective depth of concrete in compression = $\lambda x \leq h$

where

λ = 0.8 for ≤ C50/60 (see Table 6.1 when f_{ck} > 50)
x = depth to neutral axis
h = height of section
γ_C = 1.5

σ_{sc}, (σ_{st}) = stress in compression (and tension) reinforcement

- For moment

$$\frac{A_{sM}}{2} = \frac{M_{Ed} - \alpha_{cc}\eta f_{ck} b d_c\,(h/2 - d_c/2)/\,\gamma_C}{(h/2 - d_2)(\sigma_{sc} + \sigma_{st})}$$

where

A_{sM} = total area of reinforcement required to resist moment using this method
A_{sM} = $A_{s1} + A_{s2}$ and $A_{s1} = A_{s2}$

- Solution: iterate x such that $A_{sN} = A_{sM}$.

17.4.3 Column charts

Alternatively A_s may be estimated from column charts.

Figures 17.2a) to e) give non-dimensional design charts for symmetrically reinforced rectangular columns where reinforcement is assumed to be concentrated in the corners.

In these charts:
α_{cc} = 0.85
f_{ck} ≤ 50 MPa

Simplified stress block assumed.

A_s = total area of reinforcement required
= $(A_s f_{yk}/bh\,f_{ck})bh\,f_{ck}/f_{yk}$ where
$(A_s f_{yk}/bh\,f_{ck})$ is derived from the appropriate design chart interpolating as necessary between charts for the value of d_2/h for the section.

Where reinforcement is not concentrated in the corners, a conservative approach is to calculate an effective value of d_2 as illustrated in Figures 17a) to e).
d_2 = effective depth to steel in layer 2.

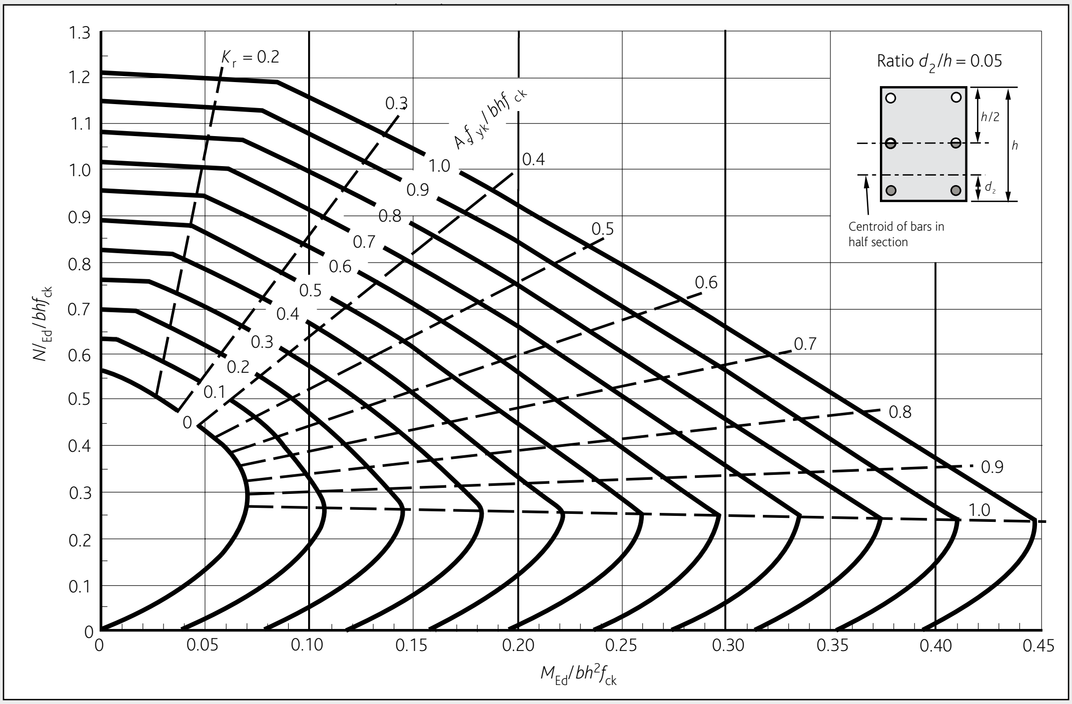


Figure 17.2a)
Rectangular columns (f_{ck} ≤ 50 MPa, f_{yk} = 500 MPa) d_2/h = 0.05

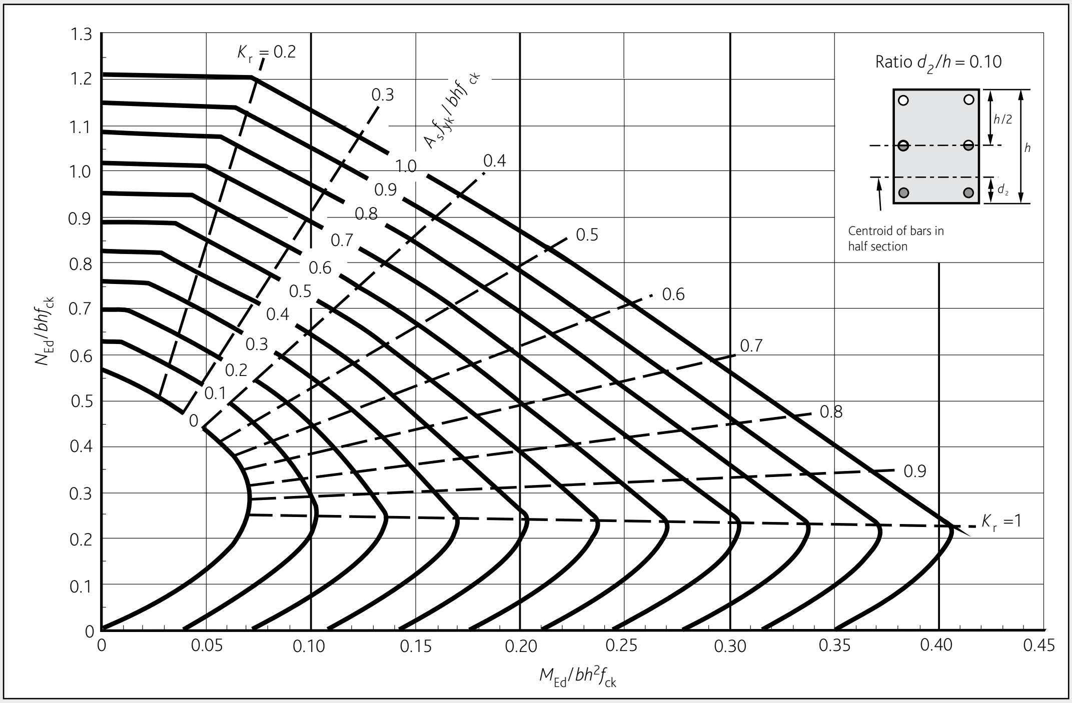


Figure 17.2b)
Rectangular columns ($f_{ck} \leq 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.10$

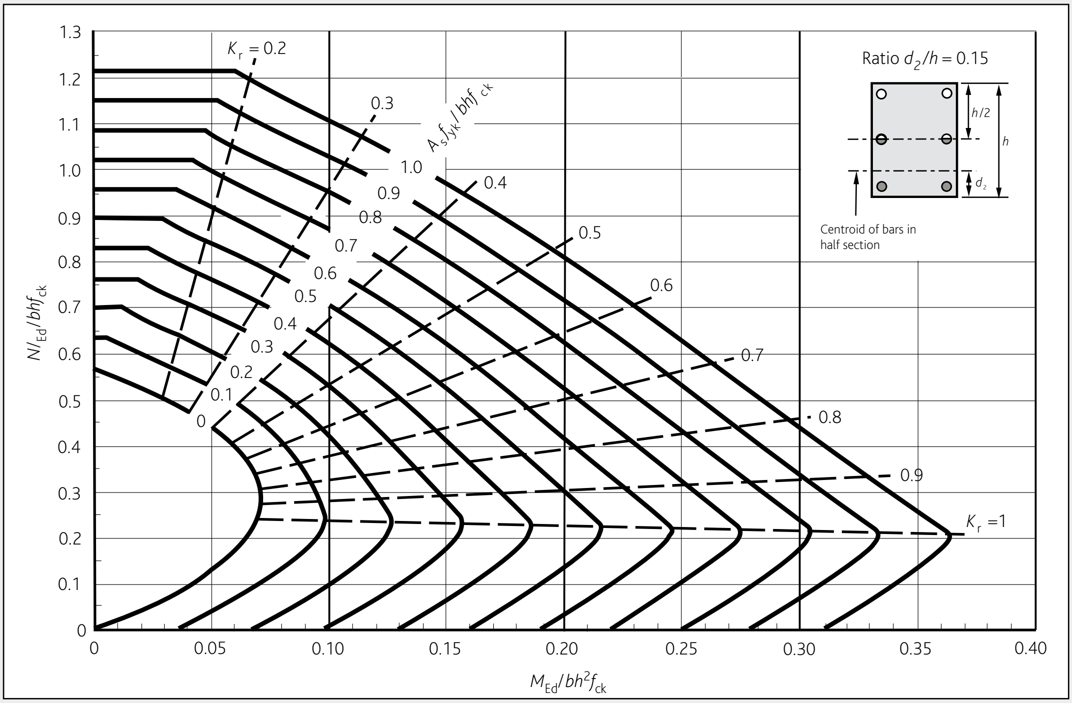


Figure 17.2c)
Rectangular columns ($f_{ck} \leq 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.15$

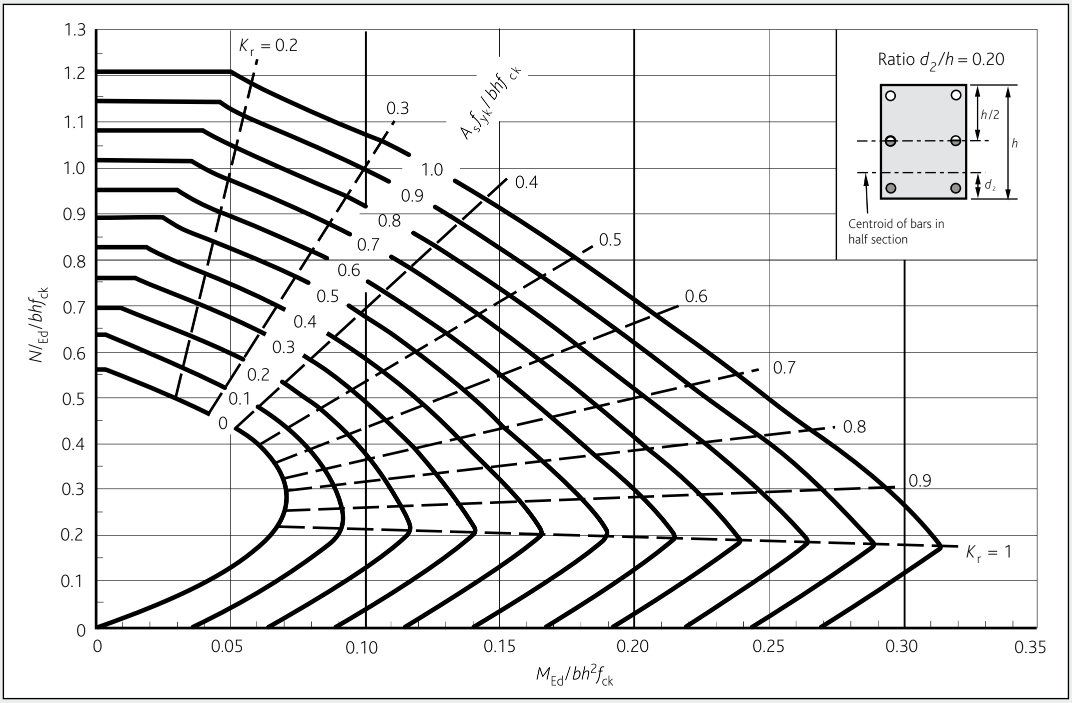


Figure 17.2d)
Rectangular columns ($f_{ck} \leq 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.20$

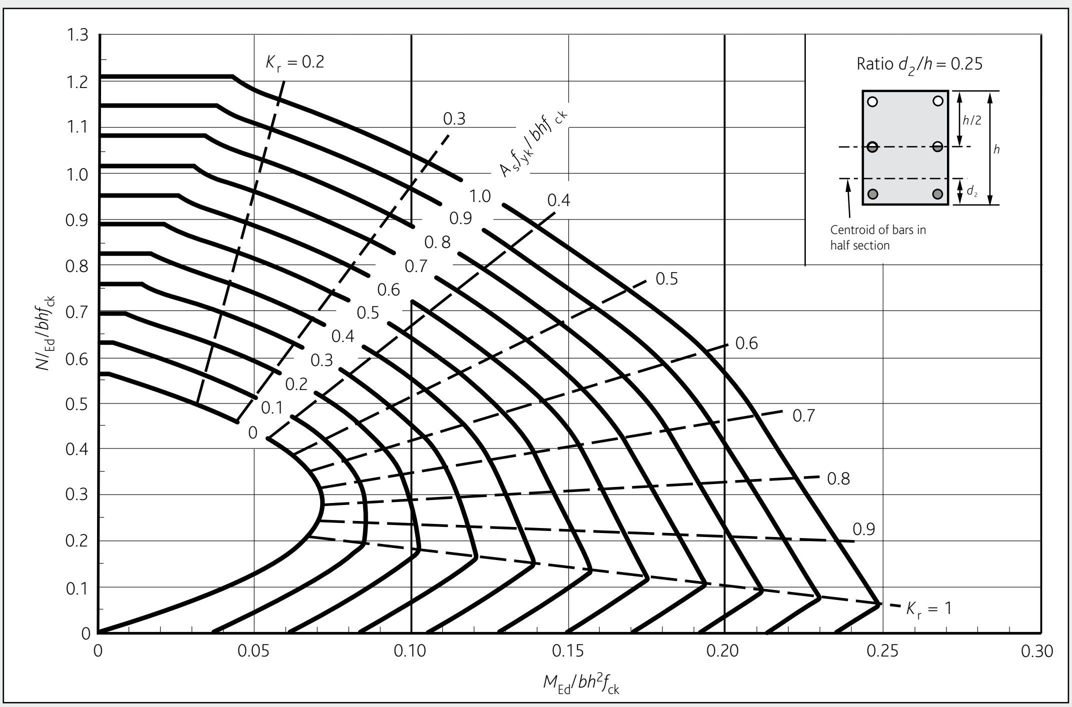


Figure 17.2e)
Rectangular columns ($f_{ck} \leq 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.25$

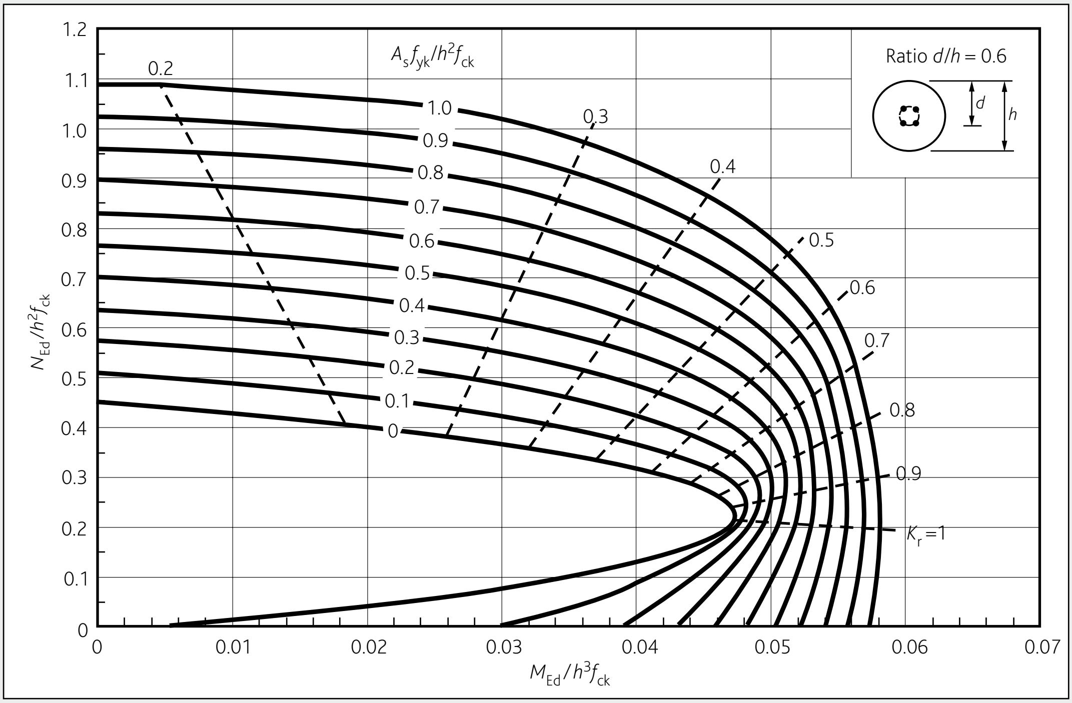


Figure 17.3a)
Rectangular columns (f_{ck} ≤ 50 MPa, f_{yk} = 500 MPa) d_2/h = 0.6

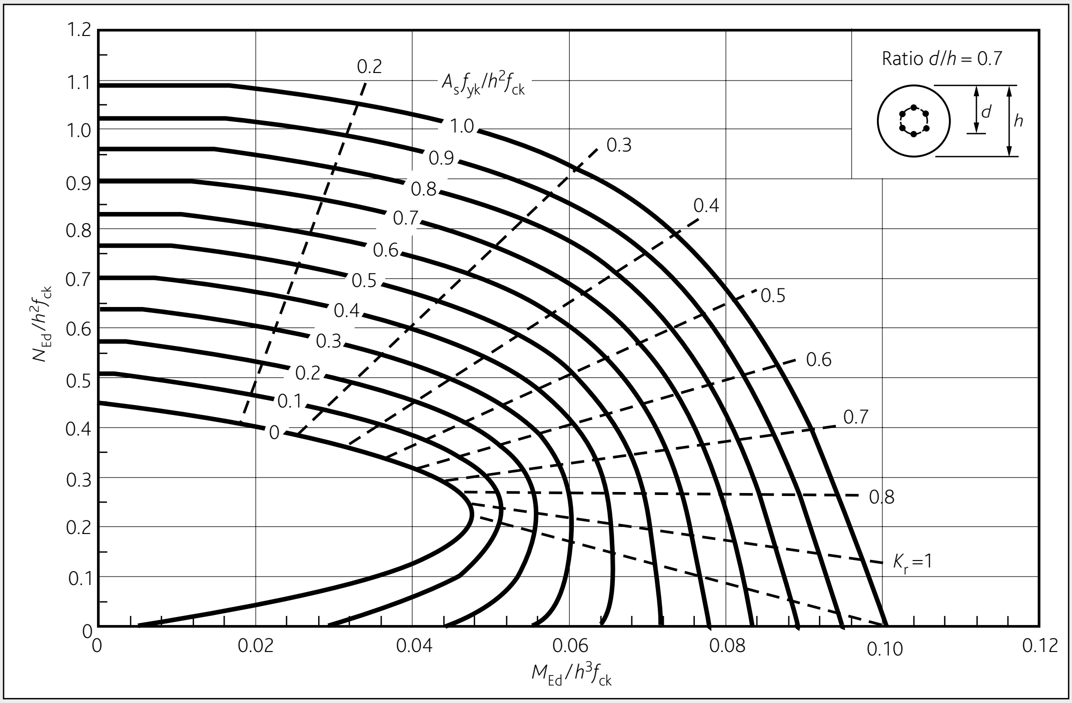


Figure 17.3b)
Rectangular columns (f_{ck} ≤ 50 MPa, f_{yk} = 500 MPa) d_2/h = 0.7

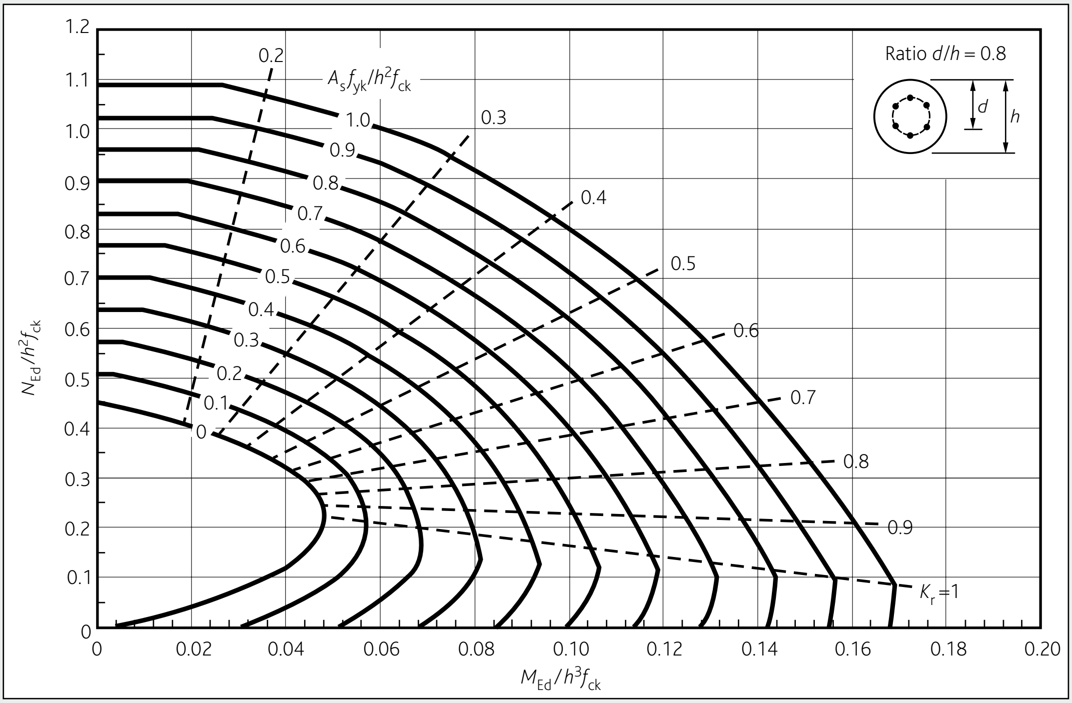


Figure 17.3c)
Rectangular columns ($f_{ck} \le 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.8$

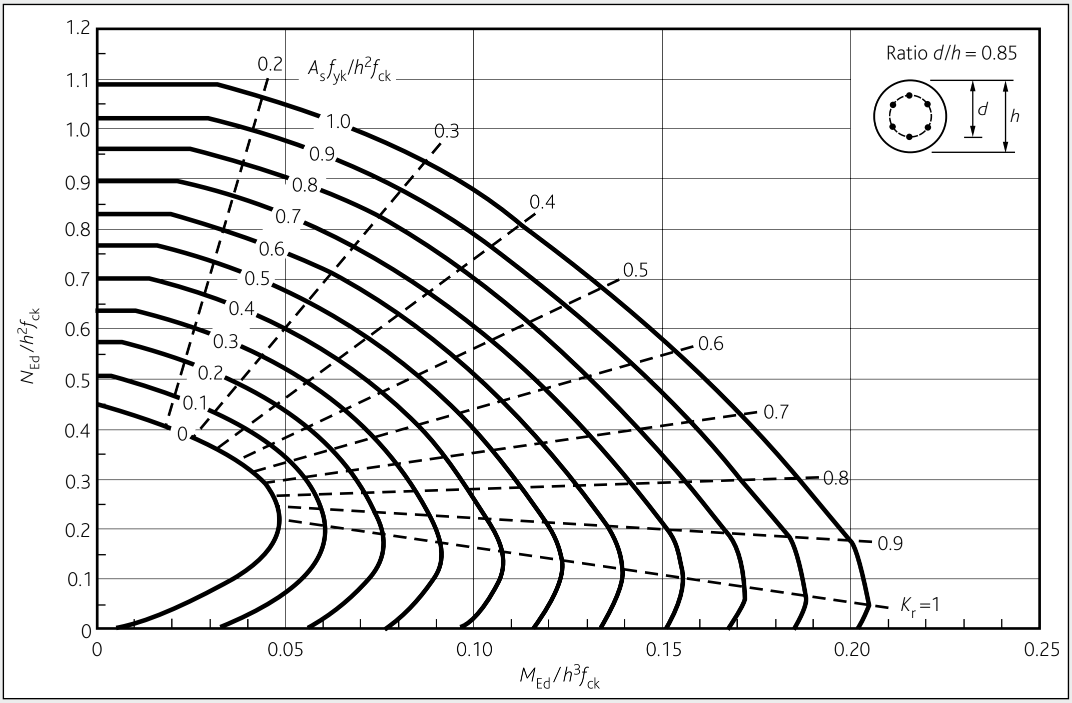


Figure 17.3d)
Rectangular columns ($f_{ck} \le 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.85$

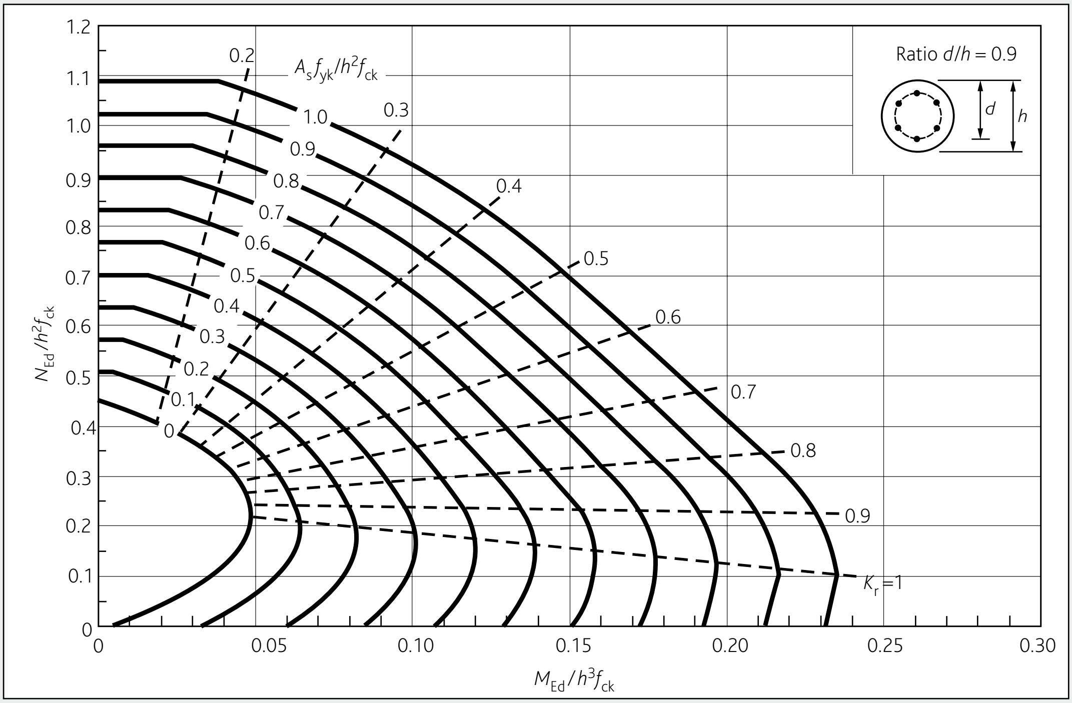


Figure 17.3e)

Rectangular columns ($f_{ck} \leq 50$ MPa, $f_{yk} = 500$ MPa) $d_2/h = 0.9$

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Guide to presentation

Grey shaded text, tables and figures	Modified Eurocode 2 text and additional text, derived formulae, tables and illustrations **not** from Eurocode 2
Yellow shaded text, tables and figures	Additional text from PD 6687[6] or PD 6687-2[3]
BS EN 1992-1-1 6.4.4	Relevant code and clauses or figure numbers
BS EN 1992-1-1 NA	From the relevant UK National Annex
BS EN 1992-1-1 6.4.4 & NA	From both Eurocode 2-1-1 and UK National Annex
Section 5.2	Relevant parts of this publication
1.0	Nationally Determined Parameter. UK values have been used throughout

Concise Eurocode 2 for Bridges

This publication summarises the material that will be commonly used in the design of reinforced and prestressed concrete bridges using Eurocode 2.

With extensive clause referencing, readers are guided through Eurocode 2, other relevant European standards and non-contradictory complementary information. The publication, which includes design aids, aims to help designers with the transition to design using Eurocodes.

Concise Eurocode 2 for Bridges is part of a range of resources available from the cement and concrete industry to assist engineers with the design of a variety of concrete bridges. For more information visit www.concretecentre.com

Owen Brooker is senior structural engineer for The Concrete Centre. He is author of several publications, including the well received series *How to design concrete structures using Eurocode 2*. He regularly lectures and provides training to structural engineers, particularly on the application of Eurocode 2.

Paul Jackson is a technical director of Gifford, which he joined from BCA in 1988. He has worked on bridge design, assessment, construction, strengthening and research. He has contributed extensively to codes of practice and is currently a member of the BSI committee for bridges and convenor of its working group for concrete bridges. He served on the project team for EN 1992-2.

Stephen Salim is a senior engineer with Gifford. He has worked on bridge assessment, feasibility studies, design, construction supervision and research. He was also involved in the calibration work of BS EN 1992-2; some of the proposals arising from this work were adopted in the UK National Annex.

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Riverside House, 4 Meadows Business Park,
Station Approach, Blackwater, Camberley, Surrey, GU17 9AB
Tel: +44 (0)1276 606800 **Fax:** +44 (0)1276 606801
www.concretecentre.com