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Concise Eurocode 2

This publication summarises the material that will be commonly used in the design of reinforced concrete framed buildings to Eurocode 2.

With extensive clause referencing, readers are guided through Eurocode 2 and other relevant Eurocodes. The publication, which includes design aids, aims to help designers with the transition to design to Eurocodes.

Concise Eurocode 2 is part of a range of resources available from The Concrete Centre to assist engineers with design to Eurocodes. For more information visit www.eurocode2.info.

R S Narayanan, the main author of this publication, was the Chairman of CEN/TC 250/SC2, the committee responsible for structural Eurocodes on concrete. He is consultant to Clark Smith Partnership, consulting engineers.

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CCIP-005 Concise Eurocode 2

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

Concise Eurocode 2

For the design of in-situ concrete framed buildings to BS EN 1992-1-1: 2004 and its UK National Annex: 2005

R S Narayanan FREng
C H Goodchild BSc CEng MCIOB MIStructE

BCA

The **Concrete** Centre™

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will affect all design and construction activities as current British Standards for structural design are due to be withdrawn in 2010.

The cement and concrete industry recognised the need to enable UK design professionals to use *Eurocode 2, Design of concrete structures*, quickly, effectively, 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 co-ordinated and collaborative approach to the introduction of Eurocode 2.

As a result, a range of resources are being delivered by the concrete sector (see www.eurocode2.info). The aim of this publication, *Concise Eurocode 2*, is to distil from Eurocode 2 and other Eurocodes the material that is commonly used in the design of concrete framed buildings.

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Concise Eurocode 2

Contents

Symbols and abbreviations used in this publication

Symbol	Definition
$\|x\|$	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_{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 steel
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	Axis distance from the concrete surface to the centre of the bar (fire)
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_m	Average axis distance (fire)
a_{sd}	Axis distance (in fire) from the lateral surface of a member to the centre of the bar
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_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
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)
c_{min}	Minimum cover, (due to the requirements for bond, $c_{min,b}$ or durability $c_{min,dur}$)
c_{nom}	Nominal cover. Nominal cover should satisfy the minimum requirements of bond, durability and fire
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
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)
dl	A short length of a perimeter (punching shear)
E	Effect of action; Integrity (in fire); Elastic modulus
E_c, $E_{c(28)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at 28 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

Symbol	Definition
EI	Bending stiffness
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)
F1	Factor to account for flanged sections (deflection)
F2	Factor to account for brittle partitions in association with long spans (deflection)
F3	Factor to account for service stress in tensile reinforcement (deflection)
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_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
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
$F_{tie,col}$	Horizontal tie force, column to floor or roof (kN)
$F_{tie,fac}$	Horizontal tie force, wall to floor or roof (kN/m)
$F_{tie,int}$	Internal tie tensile force
$F_{tie,per}$	Peripheral tie tensile force
F_{wd}	Design shear strength of weld, design value of the force in stirrups in corbels
f_{bd}	Ultimate bond stress
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength

Symbol	Definition
$f_{cd,pl}$	Design compressive strength of plain concrete
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
$f_{ck, cube}$	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_{ctm}	Mean value of axial tensile strength of concrete
$f_{ct,0.05}$	5% fractile value of axial tensile strength of concrete
$f_{ct,0.95}$	95% fractile value of axial tensile strength of concrete
f_{cvd}	Concrete design strength in shear and compression (plain concrete)
f_{sc}	Compressive stress in compression reinforcement at ULS
f_t	Tensile strength of reinforcement
$f_{t,k}$	Characteristic tensile 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
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_0	Notional size of cross section
h_s	Depth of slab
I	Second moment of area of concrete section
I	Insulation (in fire)
i	Radius of gyration
K	$M_{Ed}/bd^2 f_{ck}$. A measure of the relative compressive stress in a member in flexure
K	Factor to account for structural system (deflection)

Symbol	Definition
K'	Value of K above which compression reinforcement is required
K_r	Correction factor for curvature depending on axial load
K_φ	Factor for taking account of creep
k	Coefficient or factor
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)
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
$l_{0,fi}$	Effective length under fire conditions
l_b	Basic anchorage 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 capacity 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)
$M_{0Ed,fi}$	First order moment under fire conditions
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_{Rdy}, M_{Rdz}	Moment resistance in the respective direction
m	Number of vertical members contributing to an effect
m	Mass
N	Axial force
N	Basic span-to-effective-depth ratio, l/d, for K = 1.0 (used in Section 15)
$N_{0Ed,fi}$	Axial load under fire conditions
NA	National Annex
N_a, N_b	Longitudinal forces contributing to H_i
N_{Ed}	Design value of the applied axial force (tension or compression) at ULS
NDP	Nationally Determined Parameter(s) as published in a country's National Annex
n	Load level at normal temperatures. Conservatively n = 0.7 (fire)
n	Axial stress at ULS
n	Ultimate action (load) per unit length (or area)
n_0	Number of storeys
n_b	Number of bars in the bundle
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_k	Characteristic value of a variable action per unit length or area
R	Resistance; Mechanical resistance (in fire)
R/A'	Vertical bearing resistance per unit area (foundations)
R_d	Design value of the resistance to an action
RH	Relative humidity
r	Radius
r_{cont}	The distance from the centroid of a column to the control section outside the column head
r_m	Ratio of first order end moments in columns at ULS
S, N, R	Cement types
SLS	Serviceability limit state(s) – corresponding to conditions beyond which specified service requirements are no longer met
s	Spacing
s_r	Radial spacing of perimeters of shear reinforcement
s_t	Tangential spacing shear reinforcement along perimeters of shear reinforcement
T	Torsional moment
T_{Ed}	Design value of the applied torsional moment
T_{Rd}	Design torsional resistance moment
$T_{Rd,max}$	Maximum design torsional resistance moment capacity

Symbol	Definition
t	Thickness; Time being considered; Breadth of support
t_0	The age of concrete at the time of loading
$t_{ef,i}$	Effective wall thickness (torsion)
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_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_{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)
$v_{Rd,max}$	Capacity of concrete struts expressed as a stress
W_1	Factor corresponding to a distribution of shear (punching shear)
w_k	Crack width
w_{max}	Limiting calculated crack width
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_u	Depth of the neutral axis at the ultimate limit state after redistribution
z	Lever arm of internal forces
α	Angle; Angle of shear links to the longitudinal axis; Ratio
$\alpha_1, \alpha_2, \alpha_3$ $\alpha_4, \alpha_5, \alpha_6$	Factors dealing with anchorage and laps of bars
α_{cc} (α_{ct})	A coefficient taking into account long term effects of compressive (tensile) load and the way load is applied
β	Angle; Ratio; Coefficient
β	Factor dealing with eccentricity (punching shear)
γ	Partial factor
γ_A	Partial factor for accidental actions, A
γ_C	Partial factor for concrete
γ_F	Partial factor for actions, F
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_Q	Partial factor for variable actions, Q
γ_s	Partial factor for reinforcing steel
δ	Ratio of the redistributed moment to the elastic bending moment. Redistribution ratio (1- % redistribution)
Δ_c	Allowance for deviation made in design, e.g. to allow for workmanship (BS EN 13760)
$\Delta_{c,dev}$	Allowance made in design for deviation
$\Delta_{\varepsilon p}$	Change in strain in prestressing steel
ΔF_{td}	Additional tensile force in longitudinal reinforcement due to the truss shear model
ε_c	Compressive strain in concrete
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 bilinear stress-strain relationship

Symbol	Definition
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_{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 $\varepsilon_{cu2} = \varepsilon_{cu3}$)
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
ε_{s}	Strain in reinforcing steel
ε_{u}	Strain of reinforcement or prestressing steel at maximum load
ε_{ud}	Design limit for strain for reinforcing steel in tension = 0.9 ε_{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
θ	Angle; Angle of compression struts (shear)
θ_i	Inclination used to represent imperfections
λ	Slenderness ratio
λ	Factor defining the height of the compression zone (= 0.8 for ≤C50/60)
λ_{fi}	Slenderness in fire
λ_{lim}	Limiting slenderness ratio (of columns)
μ_{fi}	Ratio of the design axial load under fire conditions to the design resistance of the column at normal temperature but with an eccentricity applicable to fire conditions
ν	Strength reduction factor for concrete cracked in shear
ξ	Reduction factor/distribution coefficient. Factor applied to G_k in BS EN 1990 Exp. (6.10b)

Symbol	Definition
ρ	Required tension reinforcement ratio
ρ'	Reinforcement ratio for required compression reinforcement, A_{s2}/bd
ρ_1	Percentage of reinforcement lapped within 0.65l_0 from the centre line of the lap being considered
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_0	Reference reinforcement ratio $f_{ck}^{0.5}$ x 10^{-3}
σ_{gd}	Design value of the ground pressure
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
σ_{sc} (σ_{st})	Stress in compression (and tension) reinforcement
σ_{sd}	Design stress in the bar at the ultimate limit state
σ_{su}	Estimate of stress in reinforcement at SLS (deflection)
τ	Torsional shear stress
$\varphi_{(\infty,t0)}$	Final value of creep coefficient
φ_{ef}	Effective creep factor
$\varphi_{(t,t0)}$	Creep coefficient, defining creep between times t and t_0, related to elastic deformation at 28 days
ϕ	Bar diameter
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
ϕ_m	Mandrel diameter
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. It necessarily covers a wide spectrum of structures and, therefore, may be seen as unduly complex for routine design work.

The aim of this Concise Eurocode 2 is to distil from all relevant parts of BS EN 1992 and the UK National Annexes[1-4] material that will be commonly used in the design of normal building structures. Thus this publication limits itself to concrete grades up to and including C50/60 and does not cover prestressed or lightweight concrete. Even with these restrictions, it is believed that the vast majority of day-to-day designs will fall within its scope.

As far as possible, the Eurocode clauses are repeated verbatim – unless clarity demands rewording or, in some cases (as identified by grey shading) additional text, derived formulae, tables, charts and illustrations. This applies particularly to Section 15 and the Appendix. If relevant, other European Codes and/or British Standards are cited. Cross references to clause numbers in Eurocode 2 or other Eurocodes are signposted in arrow boxes for quick identification.

Recognising each member state's responsibility for determining matters such as safety and current practice. The Eurocode system permits individual countries to set their own values for some construction parameters used within their jurisdiction. These values are referred to as Nationally Determined Parameters (NDPs) and are published as part of that country's National Annex. This Concise Eurocode 2 uses the NDPs in the UK National Annex to BS EN 1992-1-1[1a], and these too are highlighted in arrow boxes.

Generally the flow of information is presented in the same order as in Eurocode 2. However, all structural design is required to comply with BS EN 1990 (Eurocode: *Basis of structural design*[5]) which provides information applicable to construction in any material. The relevant basic information from BS EN 1990 is presented in Section 2. Also, some of the commonly needed design charts and tables derived for UK practice are provided in Section 15.

Based on the latest available information, this publication is all that engineers will need for the majority of concrete structures.

Guide to presentation

Grey shaded text, tables and figures	Additional text, derived formulae, tables and illustrations ***not*** from Eurocode 2
6.4.4	Relevant clauses or figure numbers from Eurocode 2-1-1 (if the reference is to other parts, other Eurocodes or other documents this will be indicated)
NA	From the relevant UK National Annex (generally to Eurocode 2-1-1)
6.4.4 NA	From both Eurocode 2-1-1 and UK National Annex
Section 5.2	Relevant parts of this publication

2 Basis of design

2.1 General

BS EN 1992-1-1[1] should be used in conjunction with BS EN 1990: *Basis of structural design*[5], which:

- 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

BS EN 1990[5]: 2.1

A structure shall 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:

- Sustain all actions and influences likely to occur during execution and use.
- Remain fit for the use for which it is required.

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

In the case of fire, the structural resistance shall be adequate for the required period of time.

A structure shall 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 shall 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: 2.2 (1)

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

BS EN 1990: 2.3

An indicative value of 50 years is given for the design working life of building structures and other common structures.

2.3 Limit state design

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

BS EN 1990: 3.1

- 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.

2.3.1 Design situations

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

BS EN 1990: 3.2

- 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

Actions refer to loads applied to the structure directly or to imposed deformations, such as uneven settlements or temperature effects, which induce internal forces in the structure.

BS EN 1990: 1.5

- 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: 4.1.1

The characteristic value of an action is defined by one of the following three alternatives.

BS EN 1990: 4.1.2

- Its 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 action with known statistical distributions, such as wind or snow.
- A nominal value – used for some variable and accidental actions.

The values of action given in the various parts of BS EN 1991: *Actions on structures*[6] are taken as characteristic values.

2.3.3 Verification

Verification, using the partial factor method, is detailed in BS EN 1990[5]. 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.

BS EN 1991[6]

2.3.4 Design values of actions

The design value of an action is $\gamma_F \psi' F_k$

where

ψ' = a factor that converts the characteristic value of an action into a representative value. It adjusts the value of the action to account for the joint probability of the actions occurring simultaneously and can assume the values equal to:

1.0 for a permanent action

ψ'_0 or ψ'_1 or ψ'_2 for a variable action when it occurs simultaneously with other variable actions. See Tables 2.1 and 2.2 which are derived from BS EN 1990 and its National Annex [5a].

γ_F = partial factor for the action (see Table 2.2)

$\psi' F_k$ may be considered as the representative action, F_{rep}, appropriate to the limit state being considered.

BS EN 1990: A1.2.2 & NA

Table 2.2 indicates the partial factors to be used for the combinations of representative actions in building structures. Table 2.1 shows how characteristic values of variable actions are converted into representative values.

BS EN 1990: A1.3.1(1) & NA

For the ULS of strength, the designer may choose between using Exp. (6.10) or the less favourable of Exps. (6.10a) and (6.10b). Exp. (6.10) leads to the use of $\gamma_F = \gamma_G = 1.35$ for permanent actions and $\gamma_F = \gamma_Q = 1.50$ for variable actions (γ_G for permanent actions is intended to be constant across all spans). Exp. (6.10) is always equal to or more conservative than the least favourable of Exps. (6.10a) and (6.10b).

Except in the case of concrete structures supporting storage loads where $\psi'_0 = 1.0$, or for mixed use, Exp. (6.10b) will usually apply. Thus $\gamma_F = \gamma_G = 1.25$ for permanent actions and $\gamma_F = \gamma_Q = 1.50$ for variable actions will be applicable to most concrete structures. In other words, for members supporting vertical actions **$1.25G_k + 1.5Q_k$ will be appropriate for most situations at ULS.**

Table 2.1
Values of ψ factors

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area vehicle weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area 30 kN < vehicle weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs[a]	0.7	0.0	0.0
Snow loads where altitude ≤ 1000 m above sea level[a]	0.5	0.2	0.0
Wind loads[a]	0.5	0.2	0.0
Temperature effects (non-fire)[a]	0.6	0.5	0.0

Note
The numerical values given above are in accordance with BS EN 1990 and its UK National Annex

Key
a See also BS EN 1991

Table 2.2
Partial factors (γ_F) for use in verification of limit states in persistent and transient design situations

Limit state	Permanent actions (G_k)	Leading variable action ($Q_{k,1}$)	Accompanying variable actions ($Q_{k,i}$)	Reference
a) Equilibrium (EQU) Set A				
	$\gamma_{G,sup}$ ($\gamma_{G,inf}$)[a]	$\gamma_{Q,1}$	$\gamma_{Q,i}\psi_{0,i}$	BS EN 1990 Table A1.2(A) (Set A)
UK values	1.10 (0.9)[a]	1.50 (0.0)[a]	$1.50\psi_{0,i}$ (0.0)[a]	NA to BS EN 1990
b) Strength at ULS (STR/GEO) not involving geotechnical actions Set B				
Either				
Exp. (6.10)	γ_G	γ_Q	$\psi_0\gamma_Q$	BS EN 1990 Exp. (6.10) & Table A1.2(B)
UK values	1.35 (1.0)[a]	1.5	$1.5\psi_0$	NA to BS EN 1990
or worst case of				
Exp. (6.10a)	γ_G	$\psi_{0,1}\gamma_Q$	$\psi_{0,i}\gamma_Q$	BS EN 1990 Exp. (6.10a) & Table A1.2(B)
UK values	1.35 (1.0)[a]	$1.5\psi_0$	$1.5\psi_0$	NA to BS EN 1990
and				
Exp. (6.10b)	$\xi\gamma_G$	γ_Q	$\psi_0\gamma_Q$	BS EN 1990 Exp. (6.10b) & Table A1.2(B)
UK values	0.925 x 1.35 = 1.25 (1.0)[a]	1.5	$1.5\psi_0$	NA to BS EN 1990
c) Strength at ULS with geotechnical actions (STR/GEO)				
Worst case of				
Set B	γ_{G1}	γ_{Q1}		BS EN 1990 Table A1.2(B)
UK values	1.35 (1.0)[a]	1.5 (0.0)[a]		NA to BS EN 1990
or				
Set C	γ_{G2}	γ_{Q2}		BS EN 1990 Table A1.2(C)
UK values	1.0	1.3		NA to BS EN 1990
d) Serviceability				
Characteristic	1.00	1.00	$1.00\psi_{0,i}$	BS EN 1990 Table A1.4
Frequent	1.00	$1.00\psi_{1,1}$	$1.00\psi_{2,i}$	
Quasi-permanent	1.00	$1.00\psi_{2,1}$	$1.00\psi_{2,i}$	
e) Accidental design situations				
Exp. (6.11a)	$\gamma_{G,sup}$ or ($\gamma_{G,inf}$)	$\psi_{1,1}$[b]	$\psi_{2,i}$	BS EN 1990 Exp. (6.11a)
UK values	1.00	$\psi_{1,1}$[b]	$\psi_{2,i}$	NA to BS EN 1990

Key
a Value if favourable
b Leading *accidental* action (A_d)

Notes
The values of ψ are given in Table 2.1.
Where the variation between $G_{k,sup}$ and $G_{k,inf}$ is not great, say < 10%, G_k is taken to represent permanent action.
Geotechnical actions given in the table are based on Design Approach 1 in Clause A1.3.1(5) of BS EN 1990, which is recommended in the National Annex for BS EN 1990.

Variable actions may be subjected to reduction factors: α_A, according to area supported (m^2), A, and/or α_n according to number of storeys supported, n.

BS EN 1991-1-1 6.3.1.2 (10), 6.3.1.2 (11) & NA

$$\alpha_A = 1.0 - A/1000 \geq 0.75$$
$$\alpha_n = 1.1 - n/10 \text{ for } 1 \leq n \leq 5$$
$$= 0.6 \text{ for } 5 \leq n \leq 10 \text{ and}$$
$$= 0.5 \text{ for } n > 10$$

2.3.5 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).

2.4.2.4(1) & NA

The values of γ_c and γ_s, partial factors for materials, are indicated in Table 2.3.

Table 2.1 N & NA

Table 2.3
Partial factors for materials

Design situation	γ_c – concrete	γ_s – reinforcing steel
ULS – Persistent and transient	1.50	1.15
Accidental – Non-fire	1.20	1.00
Accidental – Fire	1.00	1.00
SLS	1.00	1.00

2.4 Assumptions

1.3

In addition to the assumptions in BS EN 1990, Eurocode 2 assumes that:
- Design and construction will be undertaken by appropriately qualified and experienced personnel.
- Adequate supervision and quality control will be provided.
- Materials and products will be used as specified.
- 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 are complied with.

PD 6687[7]

ENV 13670[8] is currently available but without its National Application Document. For building structures in the UK, the background document PD 6687[7] considers the provisions of the National Structural Concrete Specification (NSCS)[9] to be equivalent to those in ENV 13670 for tolerance class 1. When published, EN 13670[10] and the corresponding National Annex, will take precedence.

2.5 Foundation design

BS EN 1997: 2.4.6.4

The design of concrete foundations is subject to Eurocode 7[11] for the geotechnical aspects and to Eurocode 2 for the structural concrete design.

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.

It is anticipated that, within *prescriptive measures,* the current UK practice of checking characteristic loads (γ_{Gk} = 1.0, γ_{Qk} = 1.0) against presumed allowable bearing pressures will prevail until Eurocode 7 is fully implemented. In this case it will be for the writer of the site/ground investigation report to ensure that the presumed bearing pressures provide designs consistent with both the ULS and SLS requirements of Eurocode 7.

Further guidance on the design of simple foundations to Eurocode 7 may be found in the Appendix to this publication.

3 Materials

3.1 Concrete

Concrete should comply with BS EN 206-1 *Concrete: Specification, performance, production and conformity*[12]. In the UK, BS 8500[13] complements BS EN 206-1 and the guidance given in the former should be followed. 3.1.2

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 $f_{ck,cube}$. N.B. This notation was adopted in Amendment 3 to BS 8110: 1997[14]). Table 3.1

Table 3.1
Concrete strength classes and properties

Property	Strength class (MPa)										
	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C28/35 a	C32/40 a
f_{ck}	12.0	16.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0	28.0	32.0
$f_{ck,cube}$	15.0	20.0	25.0	30.0	37.0	45.0	50.0	55.0	60.0	35.0	40.0
f_{cm}	20.0	24.0	28.0	33.0	38.0	43.0	48.0	53.0	58.0	36.0	40.0
f_{ctm}	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	2.8	3.0
$f_{ctk,0.05}$	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	1.9	2.1
$f_{ctk,0.95}$	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	3.6	3.9
E_{cm} (GPa)	27.0	29.0	30.0	31.0	32.0	34.0	35.0	36.0	37.0	32.0	33.0

Note
a Derived data

All expressions in Eurocode 2[1-4] refer back to cylinder strength f_{ck}. It should be noted that the scope of this publication is limited to concrete in compression strength classes up to and including C50/60. 3.1.2 (2) & NA

The design strength of concrete f_{cd} should be taken as: 3.1.6 (1) & NA

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

where

f_{ck} = characteristic concrete strength
γ_c = partial factor for concrete
α_{cc} = a coefficient. In the UK α_{cc} = to 0.85 for flexure and axial loading but may be taken as 1.0 for all other phenomena (e.g. shear).

The design value of concrete tensile strength f_{ctd} should be taken as $f_{ctk,0.05}/\gamma_c$.

3.2 Steel reinforcement

3.2

The properties of steel reinforcement to BS 4449: 2005[15] are shown in Table 3.2. This British Standard complements BS EN 10080[16] and Annex C of BS EN 1992-1-1[1].

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

Table 3.2
Properties of reinforcement

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

Notes
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.

4 Durability and cover

4.1 General

A durable structure shall meet the requirements of serviceability, strength and stability throughout its intended working life, without significant loss of utility or excessive maintenance. 4.1

In order to achieve the required working life of the structure, adequate measures shall be taken to protect each structural element against the relevant environmental actions. Exposure conditions include both chemical and physical conditions to which the structure is exposed in addition to mechanical actions. 4.3, 4.2

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

Adequate cover is required to ensure:

a) Safe transmission of bond forces (see Section 4.2);
b) Protection of steel against corrosion (see Sections 4.3 and 4.4); and 4.4.1.2(1)
c) Adequate fire resistance (note that the requirements for fire resistance are given as axis distances measured from surface of the concrete to centre of the bar). (See Section 4.6.)

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 a) to c) above *and*, in the cases of a) and b), allow for the deviation to be expected in execution (see Section 4.5). 4.4.1.3(3)

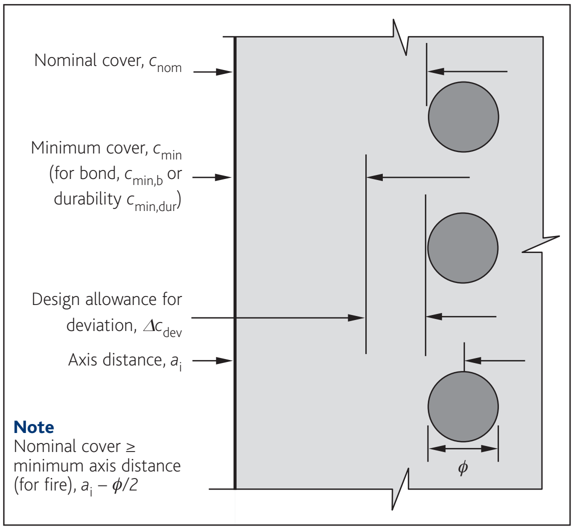


Figure 4.1
Cover

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

In order to transmit bond forces safely and to ensure adequate compaction, the minimum cover should not be less than the diameter of the bar (or the equivalent diameter of bundled bars). This minimum should be increased by 5 mm if the nominal maximum aggregate size exceeds 32 mm. 4.4.1.2(3) & NA

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

Environmental conditions are classified according to Table 4.1, which is based on BS EN 206-1[12]. Concrete composition and minimum covers required for durability in different environmental conditions are set out in Tables 4.2 and 4.3, derived from BS 8500[13]. These tables give recommendations for normal weight concrete using maximum aggregate size of 20 mm for selected exposure classes and cover to reinforcement. For each applicable exposure class, the minimum covers and required strength class or equivalent designated concrete should be determined from Tables 4.2 or 4.3 as appropriate and the worst case taken for use.

Table 4.1

Table 4.1
Exposure classes related to environmental conditions

Class	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement: all exposures except where there is freeze/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations (often combined with appropriate Aggressive Chemical Environment for Concrete (ACEC) class)
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Concrete, totally immersed in water containing chlorides e.g. swimming pools Concrete exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements, car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze/thaw attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6 Chemical attack		
XA1[a]	Slightly aggressive chemical environment	Natural soils and groundwater
XA2[a]	Moderately aggressive chemical environment	Natural soils and groundwater
XA3[a]	Highly aggressive chemical environment	Natural soils and groundwater

Key
a Whilst exposure conditions XA1, XA2 and XA3 are in accordance with BS EN 206-1, they are not appropriate according to BS 8500. See Section 4.4

Table 4.2
Concrete quality and cover to reinforcement for durability for an intended working life of at least 50 years

Class	Exposure conditions	Cement combination types[a]	Strength class, maximum w/c ratio, minimum cement or combination content (kg/m³) or equivalent designated concrete							
			Nominal cover to reinforcement (including pre-stressing steel)							
			15 + Δc[b]	20 + Δc	25 + Δc	30 + Δc	35 + Δc	40 + Δc	45 + Δc	50 + Δc
1 No risk of corrosion or attack										
X0	Completely dry	All	Recommended that this exposure is not applied to reinforced concrete							
2 Corrosion induced by carbonation										
XC1	Dry or permanently wet	All	C20/25, 0.70, 240 or RC25	<<<<<[d]	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<
XC2	Wet, rarely dry	All	—[c]	—	C25/30, 0.65, 260 or RC30	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<
XC3	Moderate humidity	All except IVB	—	C40/50, 0.45, 340 or RC50	C32/40, 0.55, 300 or RC40	C28/35, 0.60, 280 or RC35	C25/30, 0.65, 260 or RC30	<<<<<	<<<<<	<<<<<
XC4	Cyclic wet and dry									
3 Corrosion induced by chlorides excluding chlorides from sea water										
XD1	Moderate humidity	All	—	—	C40/50, 0.45, 360	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<<<	<<<<<	<<<<<
XD2	Wet, rarely dry	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	—	—	—	C32/40, 0.40, 380	C25/30, 0.50, 340	C20/25, 0.55, 320	<<<<<	<<<<<
XD3	Cyclic wet and dry	CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	C45/55, 0.35, 380	C40/50, 0.40, 380	C35/45, 0.45, 360
		IIB-V, IIIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C38/35, 0.50, 340
		IIIB, IVB	—	—	—	—	—	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340

Continues overleaf

Key

a Cement or combination types:

CEM I = Portland cement
IIA = Portland cement with 6 – 20% fly ash, ggbs or 20% limestone
IIIA = Portland cement with 36 – 65% ggbs
IVB = Portland cement with 36 – 55% fly ash
–S = slag i.e. ground granulated blastfurnace slag (ggbs)
IIB = Portland cement with 21 – 35% fly ash or ggbs
IIIB = Portland cement with 66 – 80% ggbs
SRPC = Sulphate resisting Portland cement
–V = fly ash (pfa)

b Δc = an allowance for deviations (see Section 4.5)

c — = not recommended, use greater cover

d <<<<< = quality of concrete given in the previous column should not be reduced

Table 4.2
Continued

Class	Exposure conditions	Cement combination types[a]	Strength class, maximum w/c ratio, minimum cement or combination content (kg/m^3) or equivalent designated concrete							
			Nominal cover to reinforcement (including pre-stressing steel)							
			15 + Δc[b]	20 + Δc	25 + Δc	30 + Δc	35 + Δc	40 + Δc	45 + Δc	50 + Δc
4 Corrosion induced by chlorides from sea water										
XS1	Airborne salts but no direct contact	CEM I, IIA, IIB-S, SRPC	— [c]	—	—	C50/60, 0.35, 380	C40/50, 0.45, 360	C35/45, 0.50, 340	<<<<<[d]	<<<<<
		IIB-V, IIIA	—	—	—	C45/55, 0.35, 380	C35/45, 0.45, 360	C32/40, 0.50, 340	<<<<<	<<<<<
		IIIB, IVB	—	—	—	C35/45, 0.40, 380	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<<<	<<<<<
XS2	Wet, rarely dry	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	—	—	—	C32/40, 0.40, 380	C25/30, 0.50, 340	C20/25, 0.55, 320	<<<<<	<<<<<
XS3	Tidal, splash and spray zones	CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	—	C45/55, 0.35, 380	C40/50, 040, 380
		IIB-V, IIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340
		IIB, IVB	—	—	—	—	—	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340

BS 8500

In accordance with BS 8500[13], special attention should be given to the concrete composition and aggregates, when considering freeze/thaw attack, chemical attack or abrasion resistance.

Exposure conditions, XF, only affect concrete quality and do not directly affect the corrosion of reinforcement. However:

- XF1 is likely to coexist with XC3 and XC4; the requirements for XC3 and XC4 satisfy those for XF1 but, in such cases, a minimum of strength class C28/35 should be used.
- XF2 is likely to coexist with XD3; the requirements for XD3 satisfy those for XF2.
- XF3 is likely to coexist with XC3 and XC4; the requirements for XC3 and XC4 satisfy those for XF3 providing air entrainment and freeze/thaw resisting aggregates are used or alternatively a minimum strength class C40/50 with freeze/thaw resisting aggregates is used.
- XF4 is likely to coexist with XD3. Using cement combination types I, IIA, IIB-S and SRPC, the requirements for XD3 satisfy those for XF4 provided freeze/thaw resisting aggregates and a minimum strength class of C40/50 are used. Alternatively, using cement/combination type IIIB, the requirements for XD3 satisfy those of XF4 provided air entrainment and minimum strength class of C28/35 are used.

Table 4.3

Concrete quality and cover to reinforcement for durability for an intended working life of 100 years

Class	Exposure conditions	Cement combination types[a]	Strength class, maximum w/c ratio, minimum cement or combination content (kg/m^3) or equivalent designated concrete							
			Nominal cover to reinforcement (including pre-stressing steel)							
			15 + Δc[b]	20 + Δc	25 + Δc	30 + Δc	35 + Δc	40 + Δc	45 + Δc	50 + Δc
1 No risk of corrosion or attack										
X0	Completely dry	All	Recommended that this exposure is not applied to reinforced concrete							
2 Corrosion induced by carbonation										
XC1	Dry or permanently wet	All	C20/25, 0.70, 240 or RC25	<<<<<[d]	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<
XC2	Wet, rarely dry	All	—[c]	—	C25/30, 0.65, 260 or RC30	<<<<<	<<<<<	<<<<<	<<<<<	<<<<<
XC3	Moderate humidity	All except IVB	—	—	—	C40/50, 0.45, 340 or RC50	C35/45, 0.50, 320 or RC45	C32/40, 0.55, 300 or RC40	C28/35, 0.60, 280 or RC35	<<<<<
XC4	Cyclic wet and dry									

Key

a Cement or combination types:

CEM I = Portland cement

IIA = Portland cement with 6 – 20% fly ash, ggbs or 20% limestone

IIIA = Portland cement with 36 – 65% ggbs

IVB = Portland cement with 36 – 55% fly ash

–S = slag i.e. ground granulated blastfurnace slag (ggbs)

IIB = Portland cement with 21 – 35% fly ash or ggbs

IIIB = Portland cement with 66 – 80% ggbs

SRPC = Sulphate resisting Portland cement

–V = fly ash (pfa)

b Δc = an allowance for deviations (see Section 4.5)

c — = not recommended, use greater cover

d <<<<< = quality of concrete given in the previous column should not be reduced

4.4 Chemical attack

For foundations, an aggressive chemical environment for concrete (ACEC) class should be assessed for the site. BS 8500-1 refers to BRE Special Digest 1[17] which identifies ACEC classes rather than XA classes.

Knowing the ACEC class for sections with a thickness of at least 140 mm and an intended working life of either 50 or 100 years, a design chemical (DC) class can be obtained and an appropriate designated concrete (e.g. FND designation) selected.

For designed concrete, the concrete producer should be advised of the DC class. Alternatively, a designated FND concrete, which has a minimum strength class of C28/35, can be specified. Additional protective measures may be necessary, see BS 8500[13].

4.5 Δc_{dev} and other allowances

4.4.1.3(1) & NA

The minimum covers for bond in Section 4.2 and for durability in Tables 4.2 and 4.3 should be increased by an allowance in design, Δc_{dev} to allow for likely deviations during execution as follows.:

- 10 mm generally.
- Between 5 and 10 mm, where a QA system operates and concrete cover is measured.
- Between 0 and 10 mm, where non-conforming members are rejected on the basis of accurate measurement of cover (e.g. precast elements).

Δc_{dev} is recognised in BS 8500[13] as Δc and in prEN 13670[10] as $\Delta c_{(minus)}$. In terms of execution tolerances $\Delta c_{(minus)}$ and $\Delta c_{(plus)}$ are subject to prEN 13670 and/or the project's specification.

4.4.1.3(4) & NA

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.

4.4.1.2(11)

Additional cover should be considered for textured or profiled surfaces. The minimum covers in Tables 4.2 and 4.3 should, in these situations, be increased by at least 5 mm.

4.4.1.2(6) & NA PD 6687

For prestressing steel, subject to XD exposure conditions, an additional allowance, $\Delta c_{,dur\gamma}$, of 10 mm should be allowed for intended working lives of 50 and 100 years.

4.6 Cover for fire resistance

4.6.1 General

BS EN 1992-1-2: Sections 4.2, 4.3 & 5

Minimum sizes of members and axis distance to reinforcement for achieving fire resistance are defined in Figures 4.2 and 4.3 and given in Tables 4.4 to 4.10. These are based on the tabulated data in BS EN 1992-1-2[2] and its UK National Annex[2a] and indicate whether the resistance relates to fire resistance for actions, *R*, integrity, *E*, and/or insulation, *I*. Fire engineering methods are available; these are introduced in Section 4.6.10.

Axis distances for prestressing bars are generally 10 mm greater and for prestressing wires and strands 15 mm greater.

BS EN 1992-1-2: Fig. 5.2

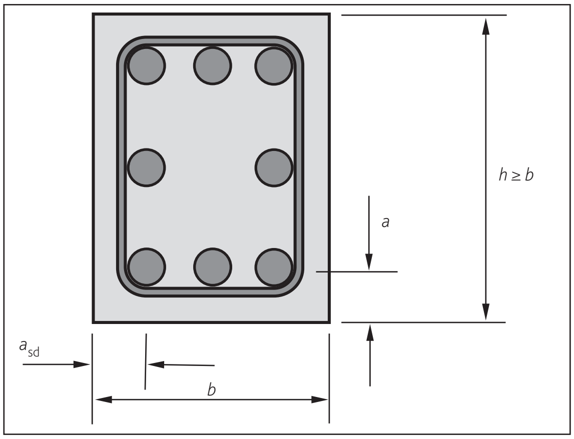
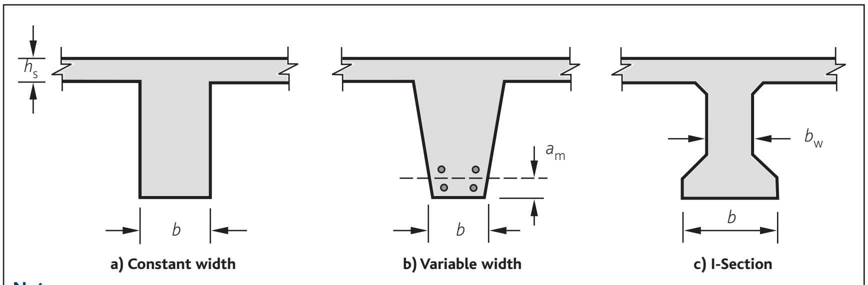


Figure 4.2
Section through a structural member showing nominal axis distance *a*, and axis distance to side of section a_{sd}

BS EN 1992-1-2: Fig. 5.4



Note
a_m = average axis distance, h_s = depth of slab

Figure 4.3
Definition of dimensions for different types of beam section

4.6.2 Columns

The fire resistance of braced columns may be assessed using Method A or Method B. Essentially:

BS EN 1992-1-2: 5.3

- When eccentricity $e < 0.15b$, Method A may be used (see Table 4.4A).
- When $0.15b < e < 0.25b$ or 100 mm, Method B may be used (see Table 4.4B).
- When $0.25b < e < 0.5b$, the further information on Method B given in BS EN 1992-1-2 Annex C may be used.
- When $e > 0.5b$, at low levels of axial load, the member may be considered to be similar to a flexural member (i.e. a beam). As an alternative, moments could be redistributed beyond normal limits such that the eccentricity falls within the limits of the tables, provided the connected beams are designed to increase span moments and have adequate ductility.

Table 4.4A is valid under the following conditions:

- The first order of eccentricity under fire conditions should be $\leq 0.15b$ (or h). The eccentricity under fire conditions may be taken as that in normal temperature design.
- The effective length of the column under fire conditions $l_{0,fi} \leq 3$ m. The value of $l_{0,fi}$ may be taken as 50% of the actual length for intermediate floors and between 50% and 70% of the actual length for the upper floor.
- The reinforcement area does not exceed 4% of the concrete cross section.

Table 4.4A
Fire resistance: columns with rectangular or circular section – Method A

BS EN 1992-1-2: Table 5.2a

Standard fire resistance	Minimum dimensions (mm) Column width b_{min}/axis distance a of main bars		
	Column exposed on more than one side		Column exposed on one side
	μ_{fi}[a] = 0.5	μ_{fi} = 0.7	μ_{fi} = 0.7
R 60	200/36 300/31	250/46 350/40	155/25
R 90	300/45 400/38	350/53 450/40 [b]	155/25
R 120	350/45 [b] 450/40 [b]	350/57 [b] 450/51	175/35
R 240	450/75 [b]	— [c]	295/70

Key
a μ_{fi} = ratio of the design axial load under fire conditions to the design resistance of the column at normal temperature
b = minimum 8 bars
c Method B indicates 600/70 for R240 and $n = 0.5$

Table 4.4B is valid where:

BS EN 1992-1-2: 5.3.3

- Eccentricity in fire, $e = M_{0Ed,fi}/N_{0Ed,fi} \leq 0.25h \leq 100$ mm.

  where
  $M_{0Ed,fi}$ and $N_{0Ed,fi}$ = first order moment and axial load under fire conditions.
  $N_{0Ed,fi}$ = $0.7\ N_{0Ed}$. Eccentricity under fire conditions may be taken as that for the normal temperature design.
- Slenderness in fire, $\lambda_{fi} = l_{0,fi}/i \leq 30$

  where
  $l_{0,fi}$ = effective length under fire conditions and
  i = radius of gyration (see Section 5.6.1);
- Amount of reinforcement $\omega = A_s f_{yd}/A_c f_{cd} \leq 1$.

  (For f_{yk} = 500 MPa, A_s/A_c = 1% and f_{ck} = 30 MPa, ω = 0.22.
  For f_{yk} = 500 MPa, A_s/A_c = 1% and f_{ck} = 50 MPa, ω = 0.13)

BS EN 1992-1-2: 5.3.3

Table 4.4B
Fire resistance: columns with rectangular or circular section – Method B

Standard fire resistance	ω	Minimum dimensions (mm) Column width b_{min}/axis distance a of main bars			
		n = 0.15	n = 0.3	n = 0.5	n = 0.7
R 60	0.1	150/30 to 200/25[a]	200/40 to 300/25[a]	300/40 to 500/25[a]	500/25[a]
	0.5	150/25[a]	150/35 to 200/25[a]	250/35 to 350/25[a]	350/40 to 550/25[a]
	1.0	150/25[a]	150/30 to 200/25[a]	250/40 to 400/25[a]	300/50 to 600/30
R 90	0.1	200/40 to 250/25[a]	300/40 to 400/25[a]	500/50 to 550/25[a]	550/40 to 600/25[a]
	0.5	150/35 to 200/25[a]	200/45 to 300/25[a]	300/45 to 550/25[a]	550/50 to 600/40
	1.0	200/25[a]	200/40 to 300/25[a]	250/40 to 550/25[a]	500/50 to 600/45
R 120	0.1	250/50 to 350/25[a]	400/50 to 550/25[a]	550/25[a]	550/60 to 600/45
	0.5	200/45 to 300/25[a]	300/45 to 550/25[a]	450/50 to 600/25[a]	500/60 to 600/50
	1.0	200/40 to 250/25[a]	250/50 to 400/25[a]	450/45 to 600/30	600/60
R 240	0.1	500/60 to 550/25[a]	550/40 to 600/25[a]	600/75	[b]
	0.5	450/45 to 500/25[a]	550/55 to 600/25[a]	600/70	[b]
	1.0	400/45 to 500/25[a]	500/40 to 600/30	600/60	[b]

Key

a Normally the requirements of BS EN 1992-1-1 will control the cover.

b Requires width greater than 600 mm. Particular assessment for buckling is required.

ω = mechanical reinforcement ratio. $\omega = A_s f_{yd} / A_c f_{cd} \le 1$.

n = load level = $N_{0Ed,fi}/0.7(A_c f_{cd} + A_s f_{yd})$. Conservatively n = 0.7.

4.6.3 Walls

Reference should be made to Table 4.5, where:

- Wall thickness given in the table may be reduced by 10% if calcareous aggregates are used.
- The ratio of the height of the wall to its thickness should not exceed 40.
- μ_{fi} is the ratio of the design axial load under fire conditions to the design resistance of the column at normal temperature conditions but with an eccentricity applicable to fire conditions. It may be conservatively taken as 0.7.

BS EN 1992-1-2: Table 5.4

Table 4.5
Fire resistance: walls

Standard fire resistance, R, integrity, E, insulation, I	Minimum dimensions (mm) Wall thickness/axis distance for			
	μ_{fi} = 0.35		μ_{fi} = 0.7	
	Wall exposed on one side	Wall exposed on two sides	Wall exposed on one side	Wall exposed on two sides
REI 60	110/10	120/10	130/10	140/10
REI 90	120/20	140/10	140/25	170/25
REI 120	150/25	160/25	160/35	220/35
REI 240	230/55	250/55	270/60	350/60

4.6.4 Beams

Reference should be made to Table 4.6, where:

- In the table, a is the axis distance and b_{min} is the width of the beam.
- The table is valid only if the detailing requirements are observed (see Sections 11 and 12) and, in the normal temperature design of continuous beams, redistribution of bending moments does not exceed 15%.
- For continuous beams for fire resistance of R90 and above, for a distance of $0.3l_{eff}$ from the centre line of each intermediate support the area of top reinforcement should not be less than the following:

  $A_{s,req}(x) = A_{s,req}(0)\ (1- 2.5x/l_{eff})$

  where

  x = distance of the section being considered from the centre line of the support

  $A_{s,req}(0)$ = area of reinforcement required for normal temperature design

  $A_{s,req}(x)$ = minimum area of reinforcement required in fire at the section being considered but not less than that required for normal temperature design

  l_{eff} = the greater of the effective lengths of the two adjacent spans

- For fire resistances R120 – R240, the width of the beam at the first intermediate support should be at least 250 mm for R120 or 480 mm for R240 if both the following conditions exist:
  a) there is no fixity at the end support; and
  b) the acting shear at normal temperature $V_{Ed} > 0.67\ V_{Rd,max}$
  where $V_{Rd,max}$ is the shear resistance controlled by the failure of compression struts.
- For beams exposed on all sides, refer to BS EN 1992-1-2 Cl 5.6.4.

Table 4.6
Fire resistance: beams

BS EN 1990-1-2: Tables 5.5 & 5.6

Standard fire resistance	Possible combination of minimum dimensions a and b_{min} (mm)								
	Dim.	Simply supported beams				Continuous beams			
		a	a	a		a	a		
R 60	b_{min} = a =	120 40	160 35	200 30	300 25	120 25	200 12		
R 90	b_{min} = a =	150 55	200 45	300 40	400 35	150 35	250 25		
R 120	b_{min} = a =	200 65	240 60	300 55	500 50	200 45	300 35	450 35	500 30
R 240	b_{min} = a =	280 90	350 80	500 75	700 70	280 75	500 60	650 60	700 50

Key
a Where beam width = b_{min} and there is only one layer of bottom reinforcement, $a_{sd} = a + 10$ mm

4.6.5 Solid slabs

Reference should be made to Table 4.7, where:

- The slab thickness h_s is the sum of the slab thickness and the thickness of any non-combustible flooring.
- Dimensions given for continuous one-way and two-way slabs apply when redistribution does not exceed 15%. Otherwise each span should be regarded as simply supported.
- l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer span.
- In two-way slabs axis distance refers to the axis distance of the lower layer of reinforcement.
- The axis distance *a* for two-way slabs relate to slabs supported at all four edges. Otherwise, they should be treated as one-way spanning slab.
- The following additional rule applies to continuous solid slabs: a minimum negative reinforcement $A_s \geq 0.005A_c$ should be provided over intermediate supports if:
  a) cold worked reinforcement is used; or
  b) there is no fixity over the end supports in a two span slab; or
  c) where transverse redistribution of load effects cannot be achieved.

BS EN 1992-1-2: Table 5.8

Table 4.7
Fire resistance: one-way and two-way solid slabs

Standard fire resistance, R, integrity, E, insulation, I	Minimum dimensions (mm)				
	Slab thickness h_s	Axis distance, *a* (simply supported)			Axis distance, *a* (continuous)
		One-way	Two-way		
			$l_y/l_x \leq 1.5$	$1.5 < l_y/l_x \leq 2$	
REI 60	80	20	10	15	10
REI 90	100	30	15	20	15
REI 120	120	40	20	25	20
REI 240	175	65	40	50	40

4.6.6 Solid flat slabs

Reference should be made to Table 4.8, where:

- Slab thickness h_s refers to the thickness of the structural slab excluding any finishes.
- Dimensions given in the table apply when redistribution in the normal temperature design does not exceed 15%. Otherwise the axis distance for one-way slabs in Table 4.7 should be used.
- Axis distance refers to the axis distance of the reinforcement in the bottom layer.
- For fire resistances REI 90 and above, continuous top reinforcement should be provided over the full span in the column strips. The area of such reinforcement should be at least 20% of the total top reinforcement required for the normal temperature design over intermediate supports.

BS EN 1992-1-2: Table 5.9

Table 4.8
Fire resistance: reinforced concrete solid flat slabs

Standard fire resistance, R, integrity, E, insulation, I	Minimum dimensions (mm)	
	Slab thickness h_s	Axis distance *a*
REI 60	180	15
REI 90	200	25
REI 120	200	35
REI 240	200	50

4.6.7 One-way ribbed slabs

For simply supported one-way ribbed slabs, reference should be made to Table 4.9.

- The table applies to slabs subjected predominantly to uniformly distributed loading.
- The axis distance measured to the lateral surface of the rib, a_{sd}, should be at least a + 10 mm.

For continuous ribbed slabs reference should be made to Table 4.9 for the flanges, but ribs should be treated as beams (see Section 4.6.4).

BS EN 1992-1-2: 5.7.5(1)

Table 4.9
Fire resistance: one-way spanning, simply supported ribbed slabs in reinforced concrete

BS EN 1992-1-2: Table 5.10

Standard fire resistance, R, integrity, E, insulation, I	Minimum dimensions (mm)				
	Possible combinations of width of ribs b_{min} and axis distance a				Slab thickness h_s and axis distance a in flange
REI 60	b_{min} = a =	100 35	120 25	≥ 200 15	h_s = 80 a = 10
REI 90	b_{min} = a =	120 45	160 40	≥ 250 30	h_s = 100 a = 15
REI 120	b_{min} = a =	160 60	190 55	≥ 300 40	h_s = 120 a = 20
REI 240	b_{min} = a =	280 90	350 75	≥ 500 70	h_s = 175 a = 40

4.6.8 Two-way continuous ribbed slabs

Reference should be made to Table 4.10, where:

- The table applies to slabs with at least one restrained edge.
- The table applies to slabs subjected predominantly to uniformly distributed loading.
- The top reinforcement should be placed in the upper half of the flange.
- The axis distance measured to the lateral surface of the rib, a_{sd}, should be at least a + 10 mm.
- For R90 and above a distance of $0.3l_{eff}$ from the centre line of each intermediate support the area of top reinforcement should not be less than the following: $A_{s,req}(x) = A_{s,req}(0)(1 - 2.5x/l_{eff})$ as given in Section 4.6.4. If this detailing requirement is not fulfilled the slab should be treated as simply supported.

Table 4.10
Fire resistance: two-way continuous ribbed slabs with at least one restrained edge

BS EN 1992-1-2: Table 5.11

Standard fire resistance, R, integrity, E, insulation, I	Minimum dimensions (mm)				
	Possible combinations of width of ribs b_{min} and axis distance a				Slab thickness h_s and axis distance a in flange
REI 60	b_{min} = a =	100 25	120 15	≥ 200 10	h_s = 80 a = 10
REI 90	b_{min} = a =	120 35	160 25	≥ 250 15	h_s = 100 a = 15
REI 120	b_{min} = a =	160 45	190 40	≥ 300 30	h_s = 120 a = 20
REI 240	b_{min} = a =	450 70	700 60		h_s = 175 a = 40

4.6.9 Covers for fire resistance when using >15% redistribution

Tables 4.6, 4.7 and 4.8 are restricted in their use to where, in the normal temperature design, redistribution of bending moments does not exceed 15%.

For beams (and continuous ribbed slabs), where redistribution exceeds 15%, the beam should be treated as simply supported or the rotational capacity at the supports should be checked, for example, by using BS EN 1992-1-2 Annex E.

For continuous solid slabs and continuous solid flat slabs, provided:
a) the all-spans-loaded case is used (see Section 5.4.2);
b) redistribution is restricted to 20%;
c) $Q_k > 0.5G_k$; and
d) bar diameter, $\phi \geq 12$ mm;
then Tables 4.7 and 4.8 may be used for fire ratings up to R120. However, if nominal cover, $c_{nom} < 25$ mm, bar diameter, ϕ, should be ≥ 16 mm and for $\phi = 16$ mm only, $A_{s,prov}/A_{s,req}$ should be ≥ 1.11[19].

4.6.10 Fire engineering

BS EN 1992-1-2: 4.2, 4.3

BS EN 1992-1-2 allows for simplified and advanced calculation methods to determine the capacities of sections in fire. Fire design is based on verifying that the effects of actions in fire are not greater than the resistance in fire after time, t, i.e. that $E_{d,fi} \leq R_{d,fi}(t)$. In that assessment:

- Actions (loads) are taken from BS EN 1991-1-2[6]
- Member analysis is based on the equation $E_{d,fi} = \eta_{fi} E_d$

where
E_d = design value of the corresponding force or moment for normal temperature design
η_{fi} = reduction factor for the design load level for the fire incident

Simplified calculation methods include the 500°C isotherm method, the zone method and checking buckling effects in columns. These methods and advanced calculation methods, used for very complex structures, are beyond the scope of this publication.

5 Structural analysis

5.1 General

The primary purpose of structural analysis in building structures is to establish the distribution of internal forces and moments over the whole or part of a structure and to identify the critical design conditions at all sections. 5.1.1

The geometry is commonly idealised by considering the structure to be made up of linear elements and plane two-dimensional elements.

5.2 Idealisation of the structure

5.2.1 Definitions

For building structures the following apply: 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.
- For the purposes of analysis, ribbed and waffled slabs need not be treated as discrete beams when the following are satisfied:
  - the rib spacing does not exceed 1500 mm;
  - the depth of the rib below the flange is not greater than four times its average width;
  - the depth of the flange exceeds the greater of either 10% of the clear distance between the ribs or 50 mm (40 mm where permanent blocks are incorporated); and
  - transverse ribs are provided at a clear spacing of ≤ 10 times the overall depth.
- 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. 5.3.2.1

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

where

$b_{eff,1}$ = $(0.2b_1 + 0.1l_0)$ but $\leq 0.2l_0$ and $\leq b_1$

$b_{eff,2}$ = to be calculated in a similar manner to $b_{eff,1}$ but b_2 should be substituted for b_1 in the above

Fig. 5.2

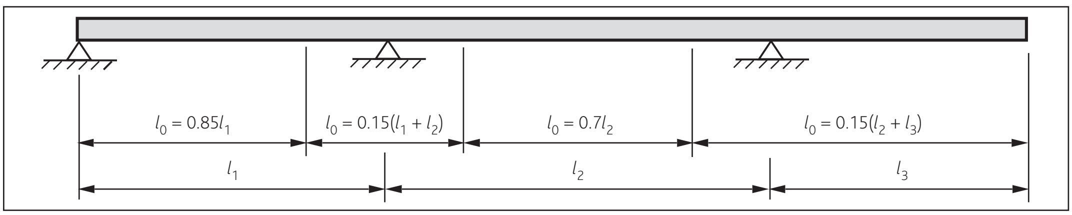


Figure 5.1
Elevation showing definition of l_0 for calculation of flange width

Fig. 5.3

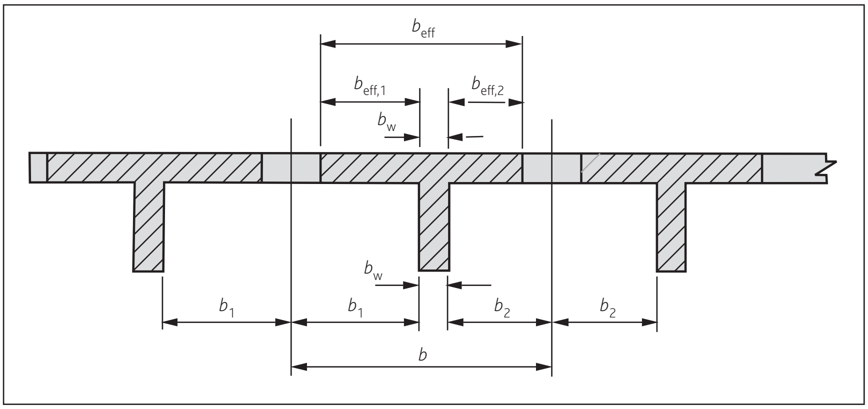


Figure 5.2
Section showing effective flange width parameters

5.2.3 Effective span

5.3.2.2

The effective span, l_{eff}, is the sum of the clear distance between the faces of supports, l_n, and an allowance *'a'* at each support as indicated in Figure 5.3.

5.3 Methods of analysis

5.3.1 Ultimate limit states (ULS)

5.1.1(7)

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.

Linear elastic analysis may be carried out assuming:

- Cross sections are uncracked and remain plane (i.e. may be based on concrete gross sections).
- Linear stress-strain relationships.
- The use of mean values of elastic modulus.

For ULS, the moments derived from elastic analysis may be redistributed provided that the resulting distribution of moments remains in equilibrium with the applied actions. In continuous beams or slabs, when the ratio of the lengths of adjacent spans is in the range of 0.5 to 2.0 and they are subjected predominantly to flexure, the following rules may be used for concrete with $f_{ck} \leq 50$ MPa.

5.5(4) & NA

$\delta \geq 0.4 + x_u/d \geq 0.7$ where the reinforcement is Class B or Class C
$\delta \geq 0.4 + x_u/d \geq 0.8$ where the reinforcement is Class A

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

5.5(6)

The design of columns should be based on elastic moments without redistribution.

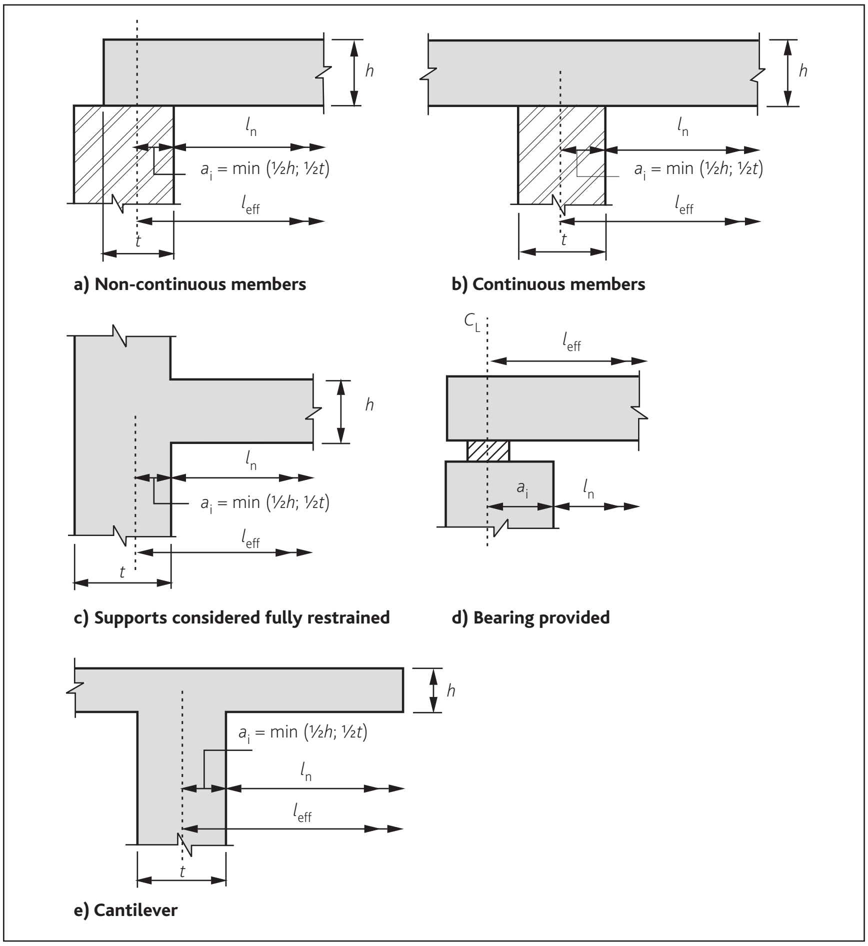


Fig. 5.4

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

Where used, plastic analysis should be based either on static or kinematic methods. The ductility of the critical sections should be sufficient for the envisaged mechanism to be formed. Plastic analysis of slabs may be carried out without check on rotation capacity provided that: 5.6.2

a) $x_u/d \leq 0.25$;
b) reinforcement is either Class B or C; and
c) ratio of the moments at internal supports to those in the span is in the range 0.5 to 2.0.

5.3.2 Serviceability limit states (SLS)

Linear elastic analysis may be carried out assuming: 5.4

- Cross sections are uncracked and remain plane (i.e. may be based on concrete gross 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.

5.3.3 General note

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

5.3.2.2(3)

- 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, which should not be taken as less than 65% of the full fixed end moment.

5.3.2.2(4)

- Where a beam or slab is continuous over a support which is considered not to provide rotational restraint, the moment calculated at the centre line of the support may be reduced by $F_{Ed,sup}t/8$, where $F_{Ed,sup}$ is the support reaction and t is the breadth of the support.

5.4 Loading

5.4.1 Load cases and load combinations

Load cases are compatible variable load arrangements that are considered simultaneously with permanent actions. Load combinations refer to the values of actions that occur in a load case.

5.1.3 & NA

5.4.2 Load arrangements

In building structures, any of the following sets of simplified load arrangements may be used at ULS and SLS (see Figure 5.4).

- The more critical of:
  a) alternate spans carrying $\gamma_G G_k + \gamma_Q Q_k$ with other spans loaded with $\gamma_G G_k$; and
  b) any two adjacent spans carrying $\gamma_G G_k + \gamma_Q Q_k$ with other spans loaded with $\gamma_G G_k$;
- Or the more critical of:
  a) alternate spans carrying $\gamma_G G_k + \gamma_Q Q_k$ with other spans loaded with $\gamma_G G_k$; and
  b) all spans carrying $\gamma_G G_k + \gamma_Q Q_k$;
- Or for slabs only, all spans carrying $\gamma_G G_k + \gamma_Q Q_k$, provided the following conditions are met:
  - in a one-way spanning slab the area of each bay exceeds 30 m^2 (a bay is defined as a strip across the full width of a structure bounded on the other sides by lines of support);
  - ratio of the variable action, Q_k, to the permanent action, G_k, does not exceed 1.25; and
  - magnitude of the variable action excluding partitions does not exceed 5 kN/m^2.

  Where analysis is carried out for the single load case of all spans loaded, the resulting moments, except those at cantilevers, should be reduced by 20%, with a consequential increase in the span moments.

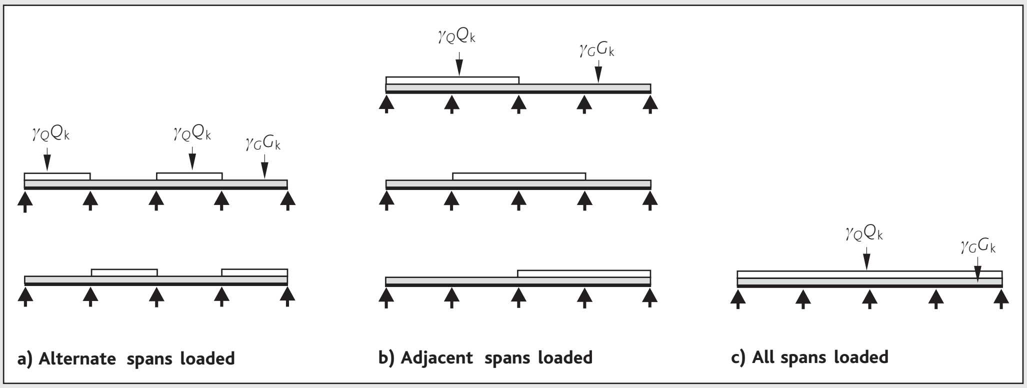


Figure 5.4
Load arrangements for beams and slabs according to UK National Annex
(The magnitude of loads indicated are those for Exp. (6.10) of BS EN 1990)

5.4.3 Load factors

For the numerical values of the factors to be used in a load case see Section 2.3.4.

N.B. γ_G is constant throughout.

5.5 Geometrical imperfections

5.5.1 General

For ULS, the unfavourable effects of possible deviations in the geometry of the structure and the position of actions shall be taken into account when verifying stability. These are in addition to other destabilising forces applied to the structure (e.g. wind actions). 5.2

5.5.2 Imperfections and global analysis of structures

For the global analysis of structures imperfections may be represented by an inclination θ_i of the whole structure. 5.2(5)

$\theta_i = (1/200)\alpha_h\alpha_m$

where

$\alpha_h = 0.67 \leq 2/l^{0.5} \leq 1.0$
$\alpha_m = [0.5\ (1 + 1/m)]^{0.5}$
l = height of the structure in metres
m = number of vertical members contributing to the effect

The effect of the inclination may be represented by transverse forces at each level to be included in the analysis with other actions. The horizontal action at any level is applied in the position that gives maximum moment. 5.2(7) 5.2(8)

$H_i = \theta_i N k$

where

H_i = action applied at that level
N = axial load
k = 1.0 for unbraced members
= 2.0 for braced members
= $(N_b - N_a)/N$ for bracing systems (see Figure 5.5a)
= $(N_b + N_a)/2N$ for floor diaphragms (see Figure 5.5b)
= N_a/N for roof diaphragms

where

N_b and N_a are longitudinal forces contributing to H_i

5.5.3 Other allowances in analysis

Allowances for imperfections are also made in:

- Partial factors used in cross section design.
- Compression members (see Section 5.6.2).

Fig. 5.1

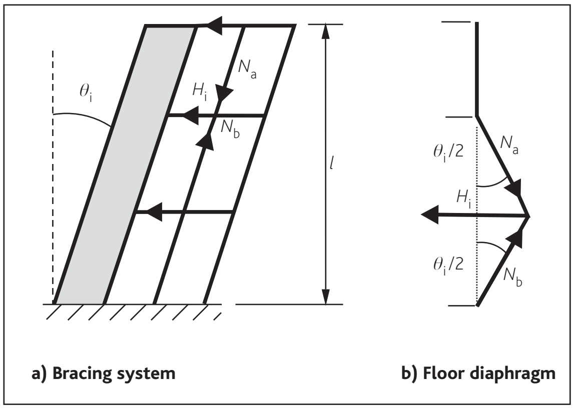


Figure 5.5
Examples of the effect of geometric imperfections

5.6 Design moments in columns

5.6.1 Definitions

5.6.1.1 Bracing members

5.8.1

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.

5.8.3.2(3)

5.6.1.2 Effective length l_0

For braced members:

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

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

PD 6687[7]

In regular structures, in which the stiffness of adjacent columns does not vary significantly (say, difference not exceeding 15% of the higher stiffness), it is recommended that the relative flexibility of each end of the column is calculated ignoring the contributions of the adjacent columns. The contribution of each attached beam should be modelled as $2EI/l_{beam}$ to allow for the effect of cracking.

5.8.3.2(2)

Examples of different buckling modes and corresponding effective length factors for isolated members are shown in Figure 5.6.

A simplified method for determining effective length factors is given in *How to design concrete structures using Eurocode 2: Columns*[20]. Conservative effective length factors for braced columns can be obtained from Table 5.1, where $l_0 = l$ x factor.

Fig. 5.7

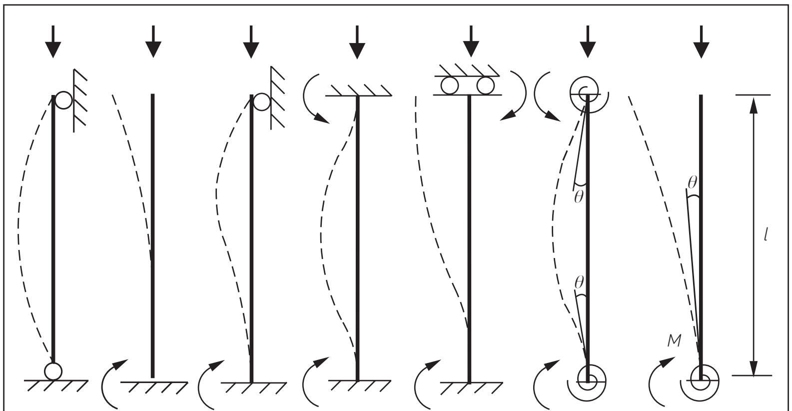


Figure 5.6
Examples of different buckling modes and corresponding effective lengths for isolated members

Table 5.1
Effective length l_0: conservative factors for braced columns

End condition at top	End condition at bottom		
	1	2	3
1	0.75	0.80	0.90
2	0.80	0.85	0.95
3	0.90	0.95	1.00

Key

Condition 1 Column connected monolithically to beams on each side that are at least as deep as the overall depth of the column in the plane considered

Where the column is connected to a foundation this should be designed to carry moment in order to satisfy this condition

Condition 2 Column connected monolithically to beams on each side that are shallower than the overall depth of the column in the plane considered by generally not less than half the column depth

Condition 3 Column connected to members that do not provide more than nominal restraint to rotation

Note

Table taken from *Manual for the design of concrete building structures to Eurocode 2* [21]. The values are those used in BS 8110: Part 1: 1997 [14] for braced columns. These values are close to those values that would be derived if the contribution from adjacent columns were ignored.

5.6.1.3 Slenderness ratio, λ

5.8.3.2(1)

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

5.6.1.4 Limiting slenderness ratio λ_{lim}

5.8.3.1(1) & NA

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

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

where

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

where

5.8.4

φ_{ef} = effective creep factor = $\varphi_{(\infty,t0)} M_{0Eqp}/M_{0Ed}$

where

3.1.4(4) 3.1.4(5)

$\varphi_{(\infty,t0)}$ = final creep ratio, which in the absence of better data, may be obtained from Figure 5.7 using procedure indicated in Figure 5.8.

In Figure 5.7:

t_0 = age of the concrete at the time of loading

h_0 = notional size $2A_c/u$, where A_c is the cross-sectional area and u is the perimeter of that part which is exposed to drying

S = cement type CEM 32.5N

N = cement types CEM 32.5R, CEM 42.5N

R = cement types CEM 42.5R, CEM 52.5N and CEM 52.5R

For structural work in the UK, Class R should be assumed. With reference to Table 4.2, CEM I cements will be 'R'. CEM II and CEM III, or their equivalents, may be 'S', 'N', or 'R'.

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)

These bending moments may be calculated for the section with maximum moment.

5.8.4(4)

Note: φ_{ef} may be taken as 0 if all the following conditions are met:

a) $\varphi_{(\infty,t0)} \leq 2.0$;
b) $\lambda \leq 75$; and
c) $M_{0Ed}/N_{Ed} \geq h$, the depth of the cross section in the relevant direction.

5.8.3.1(1) & NA

$B = (1 + 2\omega)^{0.5}$ (if ω is not known B may be taken as 1.1)

where

ω = mechanical reinforcement ratio = $(A_s/A_c)(f_{yd}/f_{cd})$, where A_s is the total area of longitudinal reinforcement

$C = 1.7 - r_m$ (If r_m is not known, C may be taken as 0.7. C is the most critical of A, B and C)

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 $C < 1.7$)

r_m = 1.0 for unbraced members and braced members in which the first order moments are caused largely by imperfections or transverse loading

If r_m is not known, C may be taken as 2.7 for columns in double curvature in braced structures through to 0.7 for constant moment, see Figure 5.9. For unbraced structures $C = 0.7$.

$n = N_{Ed}/A_c f_{cd}$

where

N_{Ed} is the design axial action at ULS

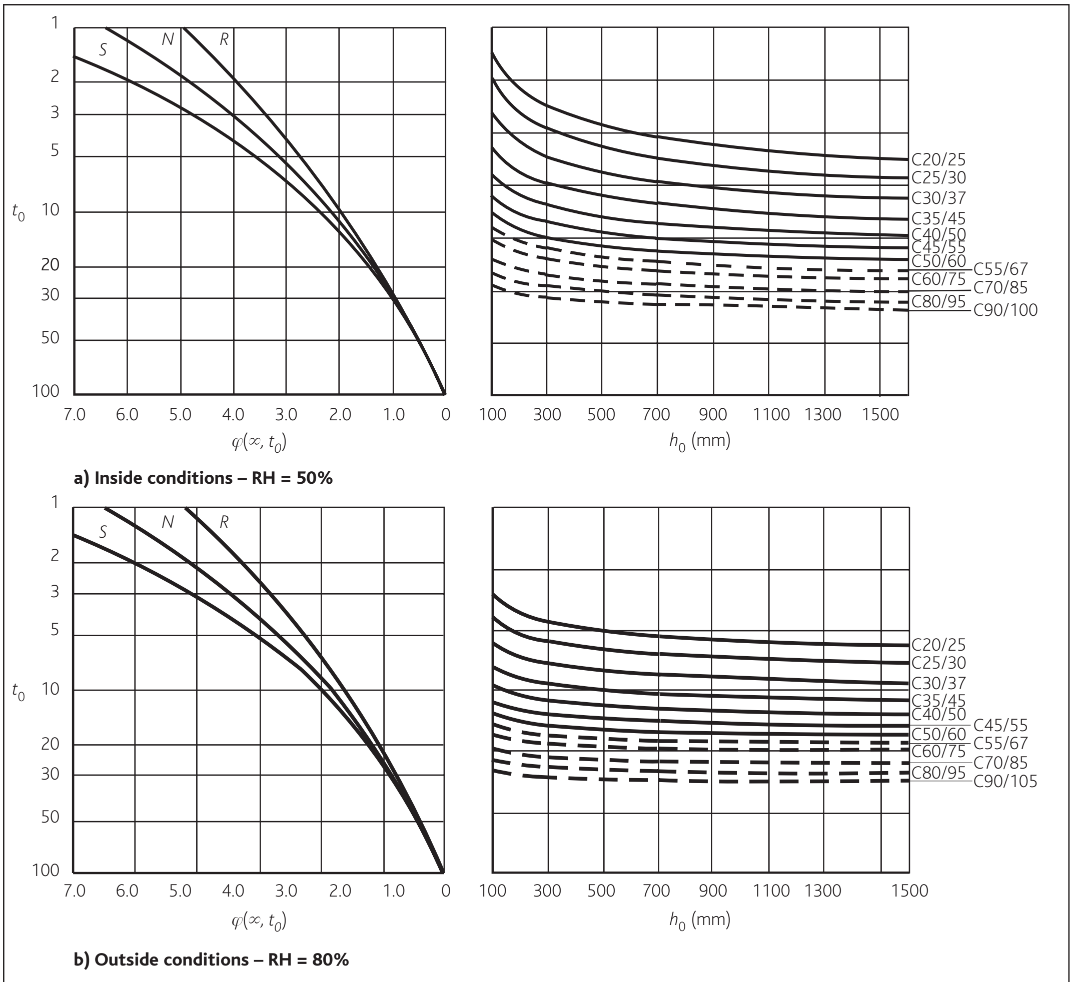
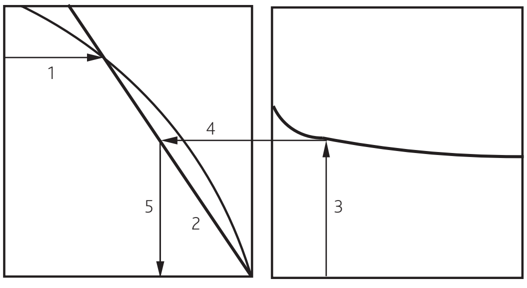


Figure 5.7
Graphs to determine values of creep coefficient $\varphi(\infty, t_0)$

Fig. 3.1



Notes

1, 2, 3, 4 & 5 are the steps indicated. Step 2 involves constructing a line starting at the origin and extending through the point where 1 intersects the relevant S, N or R curve

Intersection point between lines 4 and 5 can also be above point 1

For t_0 > 100 it is sufficiently accurate to assume t_0 = 100 and use the tangent line

Figure 5.8
How to use Figure 5.8

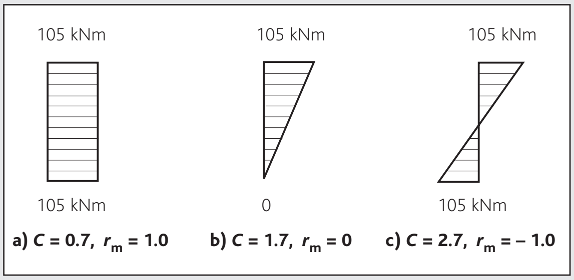


Figure 5.9
Values of C for different values of r_m

5.6.2 Design bending moments

5.6.2.1 Non-slender columns

5.8.3.1(1) When $\lambda \leq \lambda_{lim}$ i.e. when non-slender (or stocky), the design bending moment in a column is

$$M_{Ed} = M_{02}$$

where

5.8.1 M_{Ed} = design moment

5.8.8.2(2) 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.

where

$$M_{02} = M + e_i N_{Ed}$$

where

M = moment from first order analysis (elastic moments without redistribution)

5.2.7 e_i = eccentricity due to imperfections = $\theta_i \, l_0/2$

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

5.8.3.2 l_0 = effective length of column

h = depth of the section in the relevant direction

N_{Ed} = design axial action at ULS

5.6.2.2 Slender columns (nominal curvature method)

- When $\lambda > \lambda_{lim}$, i.e. when 'slender', the design bending moment in a column in a *braced* structure is

5.8.8.2 M_{Ed} = maximum of $\{M_{0Ed} + M_2; M_{02}; M_{01} + 0.5M_2\}$ (see Figure 5.10)

where

M_{0Ed} = equivalent first order moment including the effect of imperfections (at about mid height) and may be taken as = M_{0e}

where

$$M_{0e} = (0.6M_{02} + 0.4M_{01}) \geq 0.4\,M_{02}$$

where M_{02} and M_{01} are as in 5.6.2.1 above

5.8.8.2(3) M_2 = nominal second order moment in slender columns = $N_{Ed}e_2$

where

N_{Ed} = design axial action at ULS

e_2 = deflection = $(1/r)l_o^2/10$

where

$1/r$ = curvature = $K_r K_\varphi (f_{yd} / (E_s 0.45d))$ 5.8.8.3

where

$K_r = (n_u - n) / (n_u - n_{bal}) \leq 1.0$

where

$n_u = 1 + \omega$

where

ω = mechanical reinforcement ratio = $(A_s/A_c)(f_{yd}/f_{cd})$ as in 5.6.1 above

n = $N_{Ed}/A_c f_{cd}$ as defined in 5.6.1 above

n_{bal} = value of n corresponding to the maximum moment of resistance and may be taken as 0.4

Note: K_r may be derived from column charts.

$K_\varphi = 1 + \beta\varphi_{ef}$

where

$\beta = 0.35 + (f_{ck}/200) - (\lambda/150)$

where

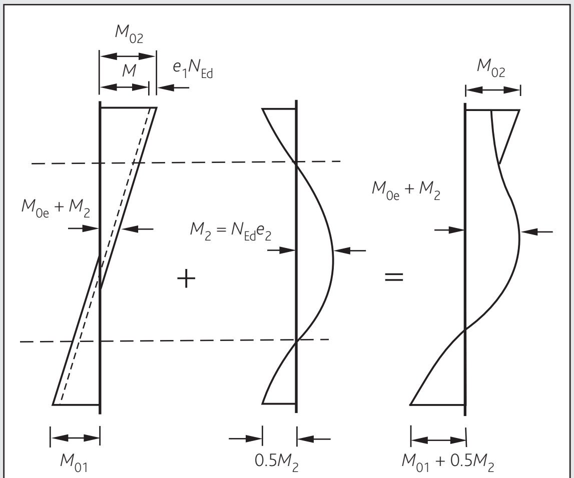
λ = slenderness ratio l_0/i 5.8.3.2

where

i = radius of gyration of the uncracked concrete section = $h/3.46$ for rectangular sections, where h is the depth in the direction under consideration and $i = d/4$ for circular sections where d is the diameter

φ_{ef} = effective creep coefficient as defined in 5.6.1 5.8.4(2)

l_0 = effective length of column



a) First order moments for 'stocky' columns **b) Additional second order moments for 'slender' columns** **c) Total moment diagram for 'slender' columns**

Figure 5.10
Moments in slender columns

- In columns in an *unbraced* structure $M_{Ed} = M_{02} + M_2$

5.6.3 Biaxial bending

5.8.9(2)

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

5.8.9(3)

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

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} = $3.46i_z$ (= h for rectangular sections)
e_z = M_{Edz}/N_{Ed}
b_{eq} = $3.46i_y$ (= b for rectangular sections)

where

N_{Ed} = design axial action at ULS
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.)

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

5.8.9(4)

Otherwise biaxially bent columns may be designed to satisfy the following:

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

where

M_{Rdy}, M_{Rdz} = moment resistance in the respective direction, corresponding to an axial load of N_{Ed}
a = an exponent:
for circular or elliptical sections, $a = 2.0$,
for rectangular sections, interpolate between
$a = 1.0$ for $N_{Ed}/N_{Rd} = 0.1$
$a = 1.5$ for $N_{Ed}/N_{Rd} = 0.7$
$a = 2.0$ for $N_{Ed}/N_{Rd} = 1.0$

Note: it is assumed that $N_{Ed}(e_i + e_2)$ act in one (critical) direction only at any one time.

Annex I & NA

5.7 Flat slabs

5.7.1 Definition

For the purposes of Section 5.7 flat slabs are slabs of uniform thickness supported on columns without beams. They may incorporate thickenings (drops) over columns.

5.7.2 Analysis

Annex I 1.1(2)

Any proven method of analysis may be used. In this publication details of ‘the equivalent frame method’ are given.

5.7.2.1 Equivalent frame method

Annex I 1.2

The structure should be divided longitudinally and transversely into frames consisting of columns and sections of slabs contained between the centre lines of adjacent panels (area bounded by four adjacent supports). The stiffness of members may be calculated using their gross cross section. For vertical loading the stiffness may be based on the full width of the panels. For horizontal loading 40% of this value should be used. Analysis should be carried out in each direction using the total action on the panel.

5.3.1(5)

The total bending moments obtained from analysis should be distributed across the width of the slab. The panels should be assumed to be divided into column and middle strips (see Figure 5.11) and the bending moments should be apportioned as given in Table 5.2. When the aspect ratio of the panel is greater than 2 the slab will tend to act as a one-way slab. Where the width of the column strip is different from 0.5 l_x, as shown in Figure 5.11, and made equal to the width of a drop, the width of the middle strip should be adjusted accordingly.

Fig. I.1

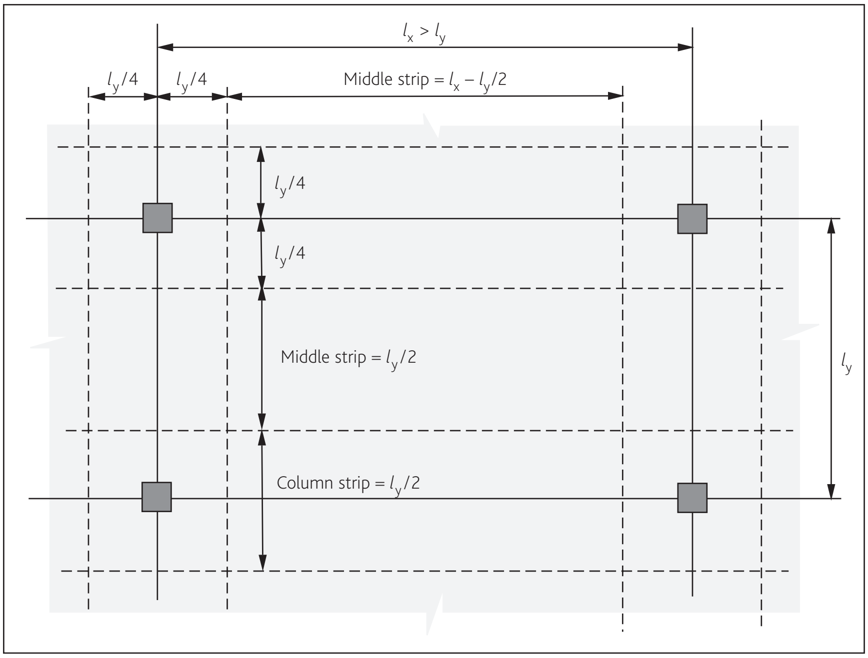


Figure 5.11
Division of panels in flat slabs

Table 5.2
Apportionment of bending moments in flat slabs – equivalent frame method

Location	Negative moments	Positive moments
Column strip	60% – 80%	50% – 70%
Middle strip	40% – 20%	50% – 30%

Notes
The total negative and positive moments to be resisted by the column and middle strips together should always add up to 100%

The distribution of design moments given in BS 8110 (column strip: hogging 75%, sagging 55%; middle strip: hogging 25%, sagging 45%) may be used

Annex I 1.2
9.4.2

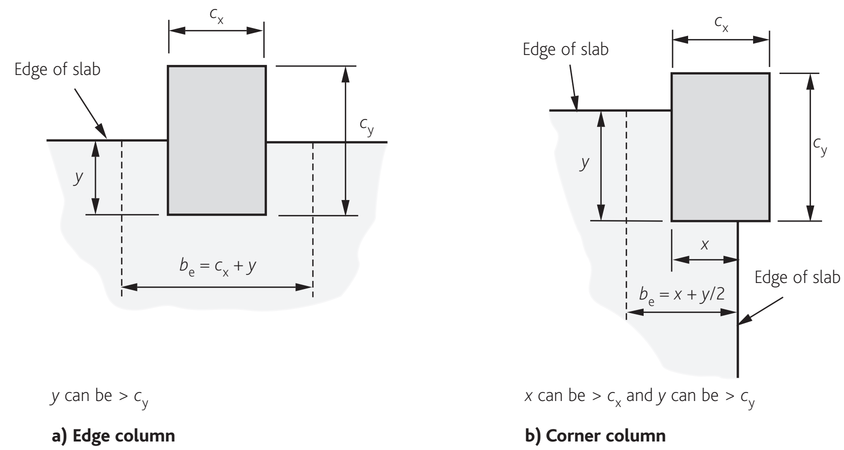
Unless there are perimeter beams that are adequately designed for torsion, the moments transferred to edge or corner columns should be limited to 0.17 $b_e d^2 f_{ck}$, where b_e is as shown in Figure 5.12.

6.4.3

6.4.3(6)

Design for punching shear should allow for the effects of moment transfer at the column/slab junction. For structures, the lateral stability of which do not rely on the frame action between the slab and columns and in which adjacent spans do not differ in length by more than 25%, the design punching shear may be obtained by enhancing the column actions. The enhancement may be taken as 1.15 for internal columns, 1.4 for edge columns and 1.5 for corner columns (see Section 8.2).

Fig. 9.9



Note
y is the distance from the edge of the slab to the innermost face of the column

Figure 5.12
Effective width, b_e, of a flat slab

5.7.2.2 Other methods of analysis

Where other methods of analysis are used, Eurocode 2 should be consulted.

5.8 Corbels

5.8.1 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.

5.8.2 Analysis

These members may be modelled either as:
a) short beams designed for bending and shear; or
b) comprising a strut-and-tie as shown in Figure 5.13.

For strut-and-tie, the internal forces should be assessed using statics. Whilst not a requirement for strut-and-tie in general, the angle θ in the model should satisfy $1.0 \le \cot\theta \le 2.5$. The width of the strut should be such that the stress in the strut does not exceed $0.6\ \nu' f_{cd}$,

Annex J.3 & NA

6.5.2.2

where

ν' = $1 - (f_{ck}/250)$
f_{cd} = $\alpha_{cc} f_{ck}/\gamma_c$
α_{cc} = 0.85

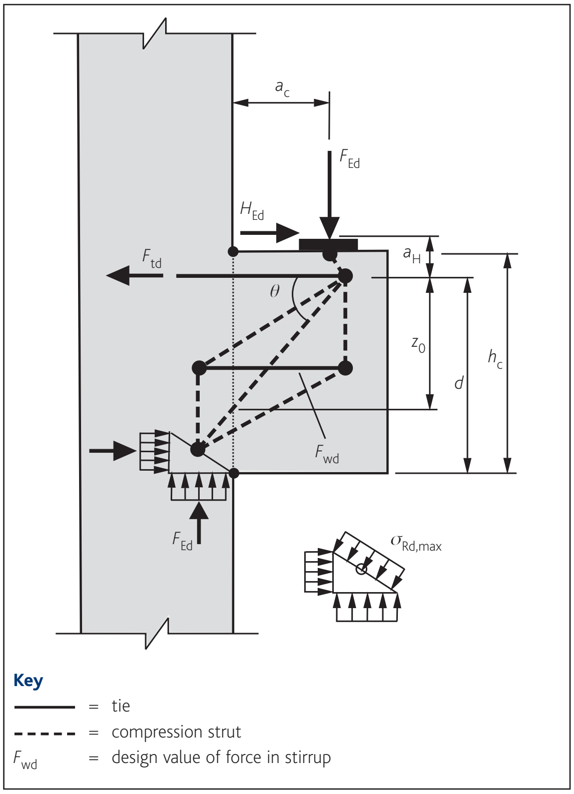


Figure 5.13
Corbel strut-and-tie model

Fig. J.5

6 Bending and axial force

6.1 Assumptions

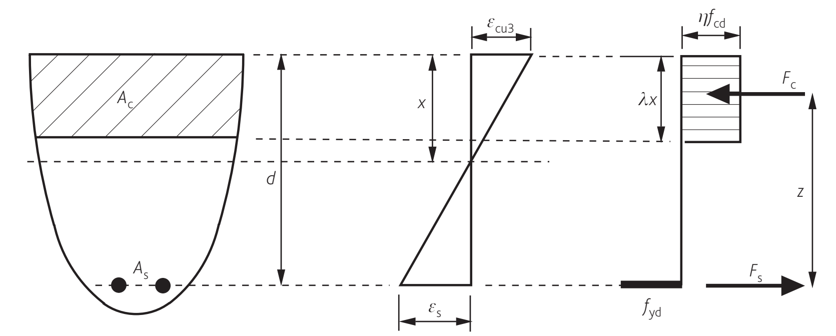
In determining the resistance of sections, the following assumptions are made.

- 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.
- Stress distribution in the section is as shown in Figure 6.1.
- 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 0.0035 (see Figure 6.3).
- For sections in pure axial compression, the compressive strain in concrete should be limited to 0.00175 (see Figure 6.3).
- For situations intermediate between these two conditions, the strain profile is defined by assuming that the strain is 0.00175 at half the depth of the section (see Figure 6.3).

3.1.7(3)
Fig. 3.5

Fig. 6.1

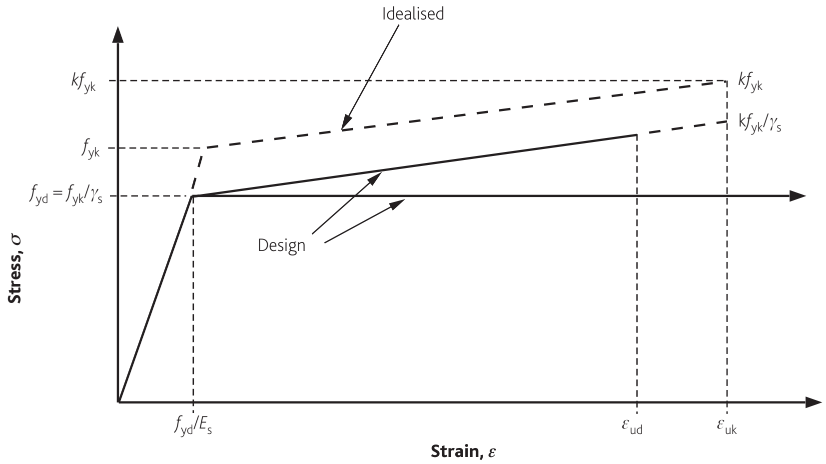
Fig. 3.5



For f_{ck} ≤ 50 MPa, $\eta = 1$ and $\lambda = 0.8$

x	=	neutral axis depth	$F_c(F_s)$	=	force in concrete (steel)
ε_{cu3}	=	ultimate compressive strain in concrete	d	=	effective depth
ε_s	=	tensile strain in reinforcement	z	=	lever arm

Figure 6.1
Rectangular stress distribution



Where

k = $(f_t / f_y)_k$

f_{yk} = yield strength of reinforcement

f_{yd} = design yield strength of reinforcement

f_t = tensile strength of reinforcement

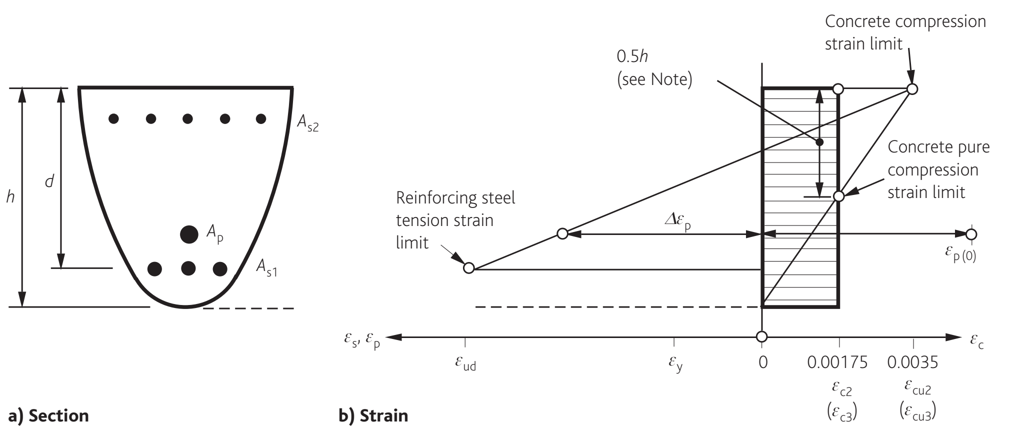
γ_s = partial factor for reinforcing steel = 1.15

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

ε_{uk} = characteristic strain of reinforcement at maximum load

Fig. 3.8

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



Where

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

A_p = area of prestressing steel

d = effective depth

h = 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 (= 0.9 ε_{uk})

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). For $f_{ck} \le 50$ MPa only, $\varepsilon_{c3} = 0.00175$

ε_{cu2} = (= ε_{cu3}) compressive strain limit in concrete not fully in pure axial compression for $f_{ck} \le 50$ MPa only, $\varepsilon_{cu3} = 0.0035$

ε_y = reinforcement yield strain

Note
Depth to pure compression strain limit has the value $(1 - \varepsilon_{c2}/\varepsilon_{cu2})\, h$ for an assumed parabolic stress-strain relationship or $(1 - \varepsilon_{c3}/\varepsilon_{cu3})\, h$ for an assumed bi-linear stress-strain relationship. For bi-linear stress-strain relationship and $f_{ck} \le 50$, $(1 - \varepsilon_{c3}/\varepsilon_{cu3}) = 0.5$

Figure 6.3
Possible strain distributions in the ultimate limit state

Fig. 6.1

6.2 Derived formulae

The following formulae may be derived by using Figures 6.1, 6.2 and 6.3.

6.2.1 Bending

Assuming K and K' have been determined:

where

$K = M/bd^2 f_{ck}$

$K' = 0.598\delta - 0.18\delta^2 - 0.21$ (see Table 6.1)

where

$\delta \leq 1.0$ = redistribution ratio (see Table 6.1)

- If $K \leq K'$

then

$A_{s1} = M/f_{yd} z$

where

A_{s1} = area of tensile reinforcement (in layer 1)

$f_{yd} = f_{yk}/\gamma_s = 500/1.15 = 434.8$ MPa

$z = d[0.5 + 0.5(1 - 3.53K)^{0.5}] \leq 0.95d$

- If $K > K'$

then

$A_{s2} = (M - M')/f_{sc}(d - d_2)$

where

A_{s2} = area of compression steel (in layer 2)

$M' = K'bd^2 f_{ck}$

$f_{sc} = 700(x_u - d_2)/x_u \leq f_{yd}$

where

d_2 = effective depth to compression steel

$x_u = (\delta - 0.4)d$

and

$A_{s1} = M'/f_{yd} z + A_{s2} f_{sc}/f_{yd}$

For $A_{s,min}$ see Section 12, Table 12.1.

Table 6.1
Values for *K'*

Percent redistribution	Redistribution ratio, δ	K'
0%	1.00	0.208
5%	0.95	0.195
10%	0.90	0.182
15%	0.85	0.168
20%	0.80	0.153
25%	0.75	0.137
30%	0.70	0.120

6.2.2 Axial load and bending

Assuming a rectangular section, symmetrical arrangement of reinforcement and ignoring side bars:

- 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
= $A_{s1} + A_{s2}$ 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 applied axial force

α_{cc} = 0.85

η = 1 for ≤ C50/60

b = breadth of section

d_c = effective depth of concrete in compression = $\lambda x \leq h$ (see Figure 6.4)

where

λ = 0.8 for ≤ C50/60

x = depth to neutral axis

h = height of section

σ_{sc} (σ_{st}) = stress in compression (and tension) reinforcement $\leq f_{yk}/\gamma_s$

3.1.6(1) & NA

- For moment

$A_{sM}/2 = [M_{Ed} - \alpha_{cc}\eta f_{ck} b d_c(h/2 - d_c/2)/\gamma_c]/[(h/2 - d_2)(\sigma_{sc} + \sigma_{st})]$

Fig. 6.1

where

A_{sM} = total area of reinforcement required to resist moment using this method.
= $A_{s1} + A_{s2}$ and $A_{s1} = A_{s2}$

- Solution

Solve by iterating x such that $A_{sN} = A_{sM}$, or refer to charts or spreadsheets etc.

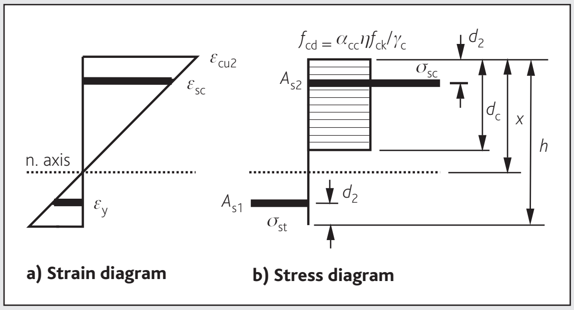


Figure 6.4
Section in axial compression and bending

7 Shear

7.1 General

7.1.1 Definitions

6.2.1

For the purposes of this section three shear resistances are used:

$V_{Rd,c}$ = resistance of a member without shear reinforcement
$V_{Rd,s}$ = resistance of a member governed by the yielding of shear reinforcement
$V_{Rd,max}$ = resistance of a member limited by the crushing of compression struts.

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

7.1.2 Requirements for shear reinforcement

If $V_{Ed} \leq V_{Rd,c}$, no calculated shear reinforcement is necessary. However, minimum shear reinforcement should still be provided (see Section 12) except in:

- 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).

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

7.1.3 Uniformly distributed loading

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

6.2.1(8)

- 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.

7.1.4 Longitudinal tension reinforcement

6.2.1(7)
9.2.1.3(2)
6.2.3(7)

The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear (see Section 12.2.2).

7.2 Resistance of members without shear reinforcement

Shear resistance of members without shear reinforcement may be calculated from:

6.2.2(1)
& NA

$$V_{Rd,c} = (0.18/\gamma_c)k(100\,\rho_l\,f_{ck})^{0.333}\,b_w\,d$$
$$\geq 0.035k^{1.5}\,f_{ck}^{\,0.5}\,b_w\,d$$

where

k = $1 + (200/d)^{0.5} \leq 2.0$ (d in mm; see Table 7.1)
γ_c = 1.5
ρ_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)

where

l_{bd} = design anchorage length
b_w = smallest width of the cross section in the tensile area

Alternatively
$V_{Rd,c} = b_w \, d \, v_{Rd,c}$ 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.

For members with actions applied on the upper side at a distance a_v, where $0.5d \le a_v \le 2d$ (see Figure 7.2), the contribution of the point load to V_{Ed} may be reduced by applying a factor $\beta = a_v/2d$. 6.2.2(6)

The longitudinal reinforcement should be completely anchored at the support.

Table 7.1
Shear resistance without shear reinforcement, $v_{Rd,c}$ (MPa)

ρ_l	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
Table derived from BS EN 1992-1-1 and the UK National Annex.
Table created for f_{ck} = 30 MPa assuming vertical links.
For $\rho_l \ge 0.4\%$ 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
Not applicable for f_{ck} > 50 MPa

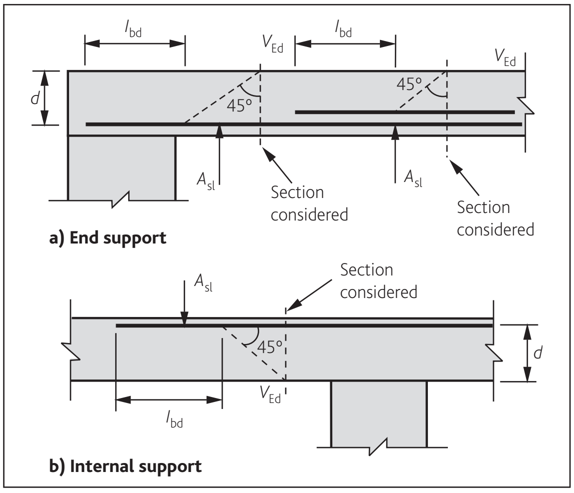


Fig. 6.3

Figure 7.1
Definition of A_{sl}

Fig. 6.4

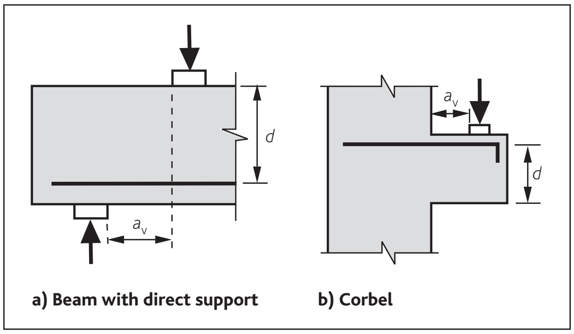


Figure 7.2
Loads near supports

6.2.3

7.3 Resistance of members requiring shear reinforcement

7.3.1 Basis

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

Fig. 6.5

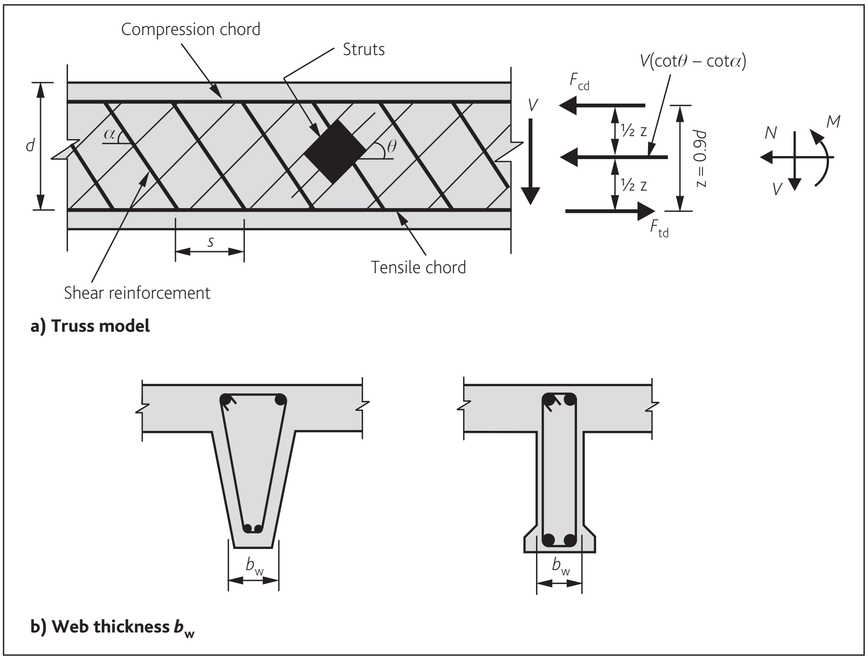


Figure 7.3
Truss model and notation for shear reinforced members

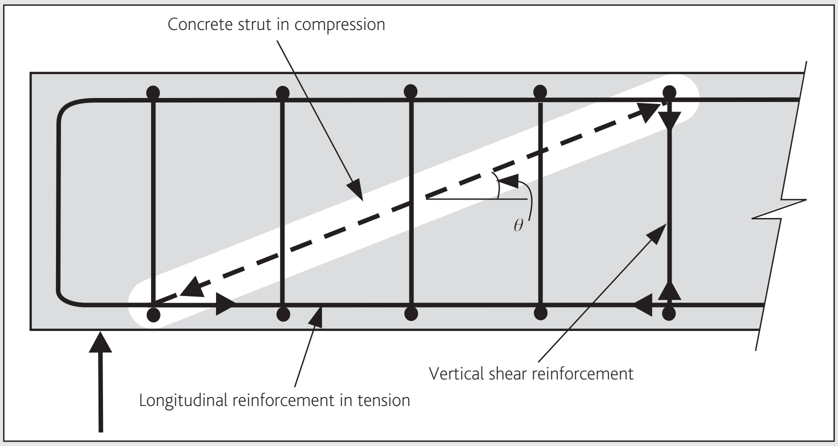


Figure 7.4
Variable strut angle, θ

7.3.2 Shear capacity check

The capacity 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:

Exp. (6.9)
Exp. (6.14)
& NA

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

where

z = lever arm: an approximate value of 0.9d may normally be used
v = $0.6\,[1 - (f_{ck}/250)]$ = strength reduction factor for concrete cracked in shear
f_{cd} = $\alpha_{cw}\, f_{ck}/\gamma_c$ with $\alpha_{cw} = 1.0$
θ = angle of inclination of the strut, such that cot θ lies between 1.0 and 2.5. The value of cot θ should be obtained by substituting V_{Ed} for $V_{Rd,max}$
α = angle of inclination of the links to the longitudinal axis. For vertical links cot α = 0.

6.2.3(3)
& NA

In most practical cases, where vertical links are used, it will be sufficient to check stresses (rather than capacities) using Table 7.2 such that:

$v_{Ed,z} \leq v_{Rd,\,max}$

where

$v_{Ed,z} = V_{Ed}/b_w z = V_{Ed}/b_w 0.9d$ = shear stress in sections *with* shear reinforcement
$v_{Rd,max} = V_{Rd,max}/b_w z = V_{Rd,max}/b_w 0.9d$

If $v_{Ed,z} \leq$ the value of $v_{Rd,max}$ for cot θ = 2.5, then θ = 21.8° and cot θ = 2.5
If $v_{Ed,z} >$ the value of $v_{Rd,max}$ for cot θ = 1.0, then the section should be resized
If $v_{Ed,z}$ is between the values for cot θ = 2.5 and cot θ = 1.0, then θ and cot θ should be calculated from the equation for $V_{Rd,max}$, but substituting V_{Ed} for $V_{Rd,max}$

Values of $v_{Rd,max}$ may be interpolated from Table 7.2.

Table 7.2
Capacity of concrete struts expressed as a stress, $v_{Rd,max}$

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°	
20		**2.54**	2.82	3.19	3.46	3.62	3.68	**0.552**
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
Table derived from BS EN 1992-1-1 and UK National Annex, assuming vertical links, i.e. cot α = 0

$v_{Rd,max} = V_{Rd,max}/b_w z = V_{Rd,max}/b_w 0.9d$

$= \nu f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$

7.3.3 Shear reinforcement required, A_{sw}/s

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

Exp. (6.13)

$V_{Rd,s} = (A_{sw}/s)zf_{ywd}(\cot\theta + \cot\alpha)\sin\alpha \geq V_{Ed}$

where

A_{sw} = cross sectional area of the shear reinforcement. (For $A_{sw,min}$ see Section 10.4.1)
s = spacing
z = lever arm (approximate value of 0.9d may normally be used)
f_{ywd} = f_{ywk}/γ_s = design yield strength of the shear reinforcement
α = angle of the links to the longitudinal axis

For vertical links, cot α = 0 and sin α = 1.0

Exp. (6.8)

$A_{sw}/s \geq V_{Ed}/zf_{ywd}\cot\theta$ or

$A_{sw}/s \geq v_{Ed,z}\, b_w/f_{ywd}\cot\theta$

7.3.4 Additional tensile forces

The additional tensile force caused by the shear model in the longitudinal reinforcement

Exp. (6.18)

$\Delta F_{td} = 0.5\, V_{Ed}(\cot\theta - \cot\alpha) \leq (M_{Ed,max}/z)$

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 12.2).

7.3.5 Members with actions applied at upper side

For members with actions applied at the upper side within a distance $0.5d < a_v < 2.0d$ for the purposes of designing the shear reinforcement, the shear force V_{Ed} may be reduced by $a_v/2d$ provided that the longitudinal reinforcement is fully anchored. In this case: 6.2.3(8)

$A_{sw} f_{ywd} \geq V_{Ed}/\sin \alpha$

where

A_{sw} = area of shear reinforcement in the central 75% of a_v (see Figure 7.5).

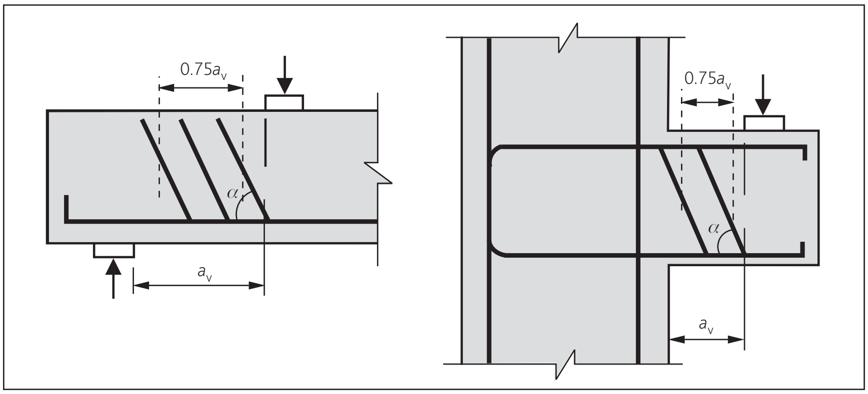


Fig. 6.6

Figure 7.5
Shear reinforcement in short shear spans with direct strut action

7.3.6 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. 6.2.1(9)

8 Punching shear

8.1 General

8.1.1 Basis of design

6.4.1

Punching shear arises when a concentrated load is applied to a small area of a slab or, most commonly, the reaction of a column against a slab. 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

At the column perimeter:

- Ensure that maximum punching shear stress is not exceeded, i.e. $v_{Ed} < v_{Rd,max}$ (otherwise resize; see Section 8.6).

At successive column perimeters:

6.4.3(2)

- Determine whether punching shear reinforcement is required, i.e. whether $v_{Ed} > v_{Rd,c}$.
- When required provide reinforcement such that $v_{Ed} \leq v_{Rd,cs}$ (see Section 8.5).

where

6.4.3(3) v_{Ed} = applied shear stress. The shear force used in the verification should be the effective force taking into account any bending moment transferred into the slab (see Sections 8.2 and 8.3)

6.4.3(1) $v_{Rd,max}$ = design value of the maximum punching shear resistance, expressed as a stress (see Sections 8.6 and Table 7.2)

6.4.4(1) $v_{Rd,c}$ = design value of punching shear resistance of a slab *without* punching shear reinforcement, expressed as a stress (see Section 8.4)

6.4.5(1) $v_{Rd,cs}$ = design value of punching shear resistance of a slab *with* punching shear reinforcement, expressed as a stress (see Section 8.5)

8.2 Applied shear stress

8.2.1 General

6.4.3(3)

The applied shear stress $v_{Ed} = \beta V_{Ed}/(u_i d)$

where

d = mean effective depth
u_i = length of the control perimeter under consideration (see Section 8.3)
V_{Ed} = applied shear force
β = factor dealing with eccentricity

8.2.2 Values of β (conservative values from diagram)

6.4.3(6)

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.

8.2.3 Values of β (using calculation method)

6.4.3(3)

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

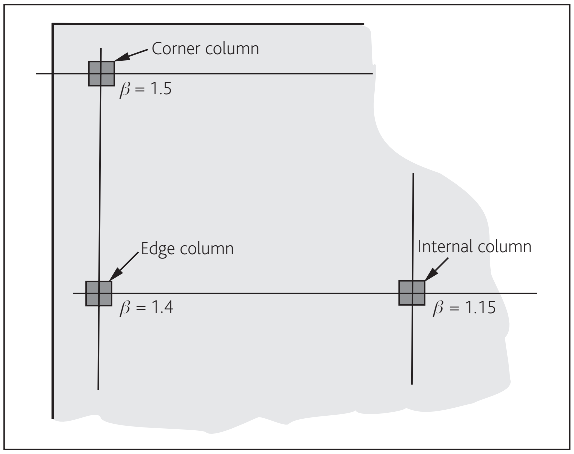


Figure 8.1
Recommended values for β

Fig. 6.21N & NA

8.2.3.1 Internal columns

a) For internal rectangular columns with loading eccentric to *one* axis:

$\beta = 1 + (kM_{Ed}/V_{Ed})(u_1/W_1)$ — 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} = total design moment
- u_1 = 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

$$= \int_0^u \left| e \right| dl$$ — Exp. (6.40)

where

- $\left| \; \right|$ = the absolute value
- e = the distance of dl from the axis about which M_{Ed} acts
- dl = a short length of the perimeter and

For a rectangular column $W_1 = c_1^2/2 + c_1c_2 + 4c_2d + 16d^2 + 2\pi dc_1$

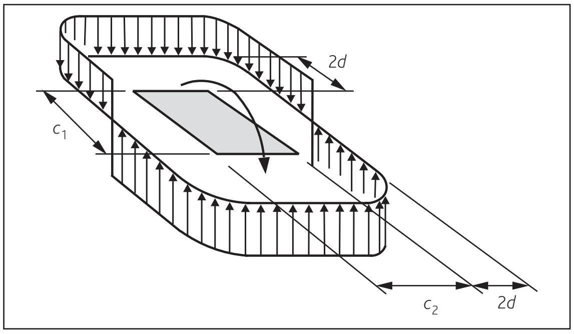


Figure 8.2
Shear distribution due to an unbalanced moment at a slab/internal column connection

Fig. 6.19

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

Table 6.1

Exp. (6.43)

b) 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)

Exp. (6.42)

c) For internal circular columns:

$\beta = 1 + 0.6\pi e/(D + 4d)$

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

Fig. 6.13

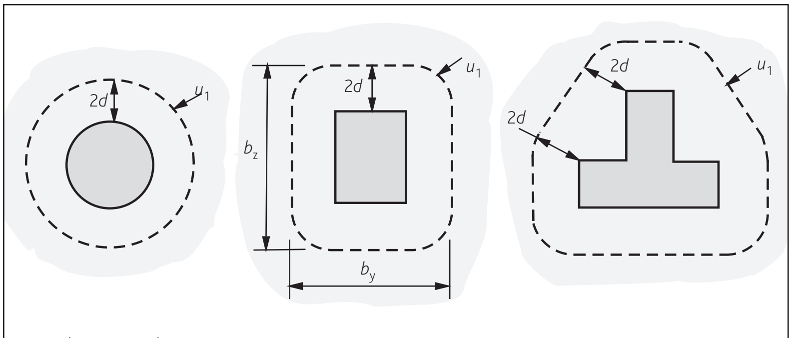


Figure 8.3
Typical basic control perimeters around loaded areas

6.4.3(4)

8.2.3.2 Edge columns

a) 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)

Exp. (6.44)

b) 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 Figure 8.5

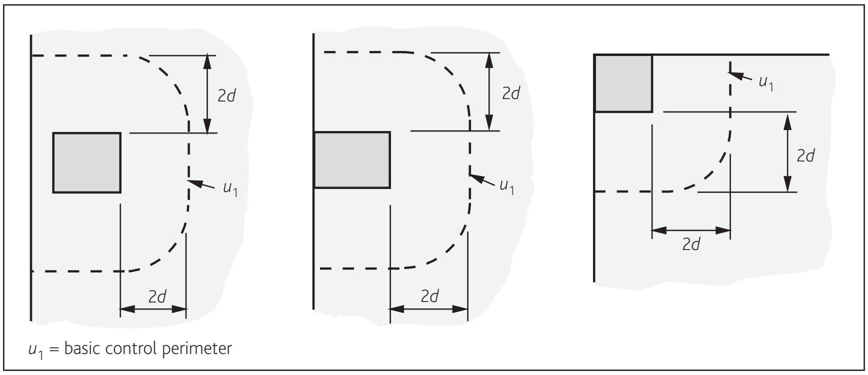


Fig. 6.15

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

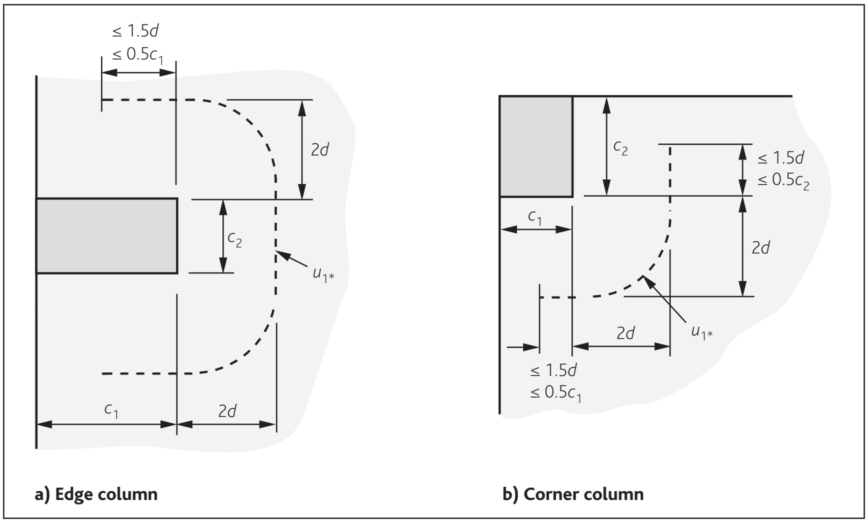


Fig. 6.20

Figure 8.5
Equivalent control perimeter u_{1*}

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

6.4.3(5)

8.2.3.3 Corner columns

For corner columns with eccentricity towards interior of the slab

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)

6.4.3(4)
6.4.3(5)

8.2.3.4 Perimeter columns where eccentricity is exterior to slab

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

$$\beta = 1 + kM_{Ed}/V_{Ed}\,u_1/W_1$$

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)

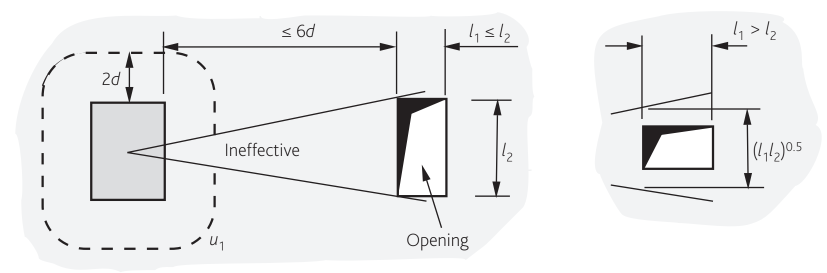
6.4.2

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. Some examples are shown in Figure 8.3.

8.3.2 Openings

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.

Fig. 6.14



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

Figure 8.6
Control perimeter near an opening

8.3.3 Perimeter columns

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 Sections 8.2.3.2 and 8.2.3.3.

8.3.4 Elongated supports

For elongated supports and walls the perimeter shown in Figure 8.5a) may be used for each end, or Figure 8.7 may be used.

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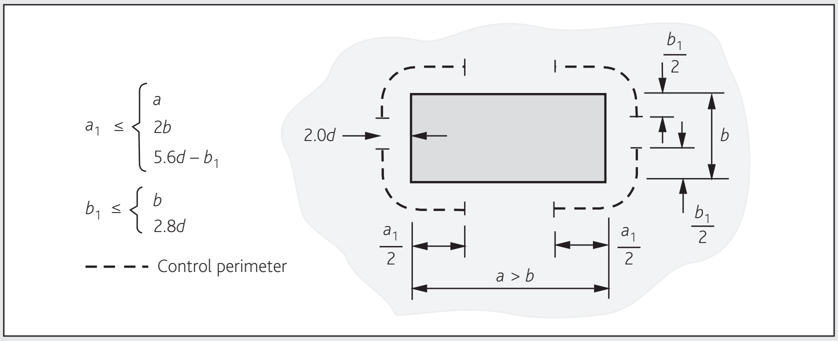


Figure 8.7
Control perimeter for elongated supports

8.3.5 Column heads

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 8.8). Where $l_H > 2h_H$ the critical sections both within the head and slab should be checked (see Figure 8.9).

Fig. 6.17

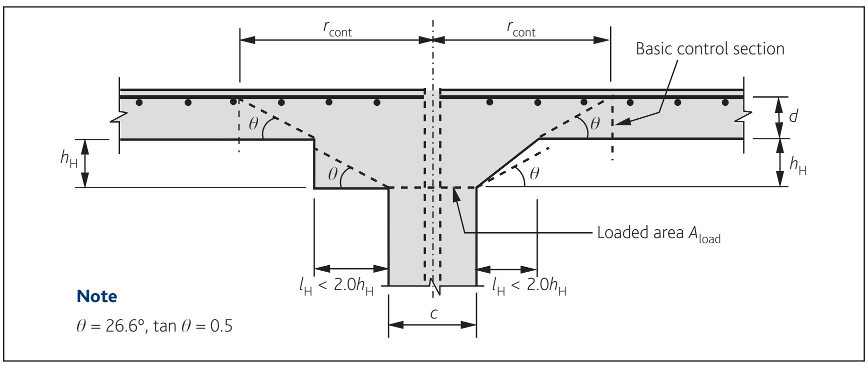


Figure 8.8
Slab with enlarged column head where $l_H < 2.0\ h_H$

Fig. 6.18

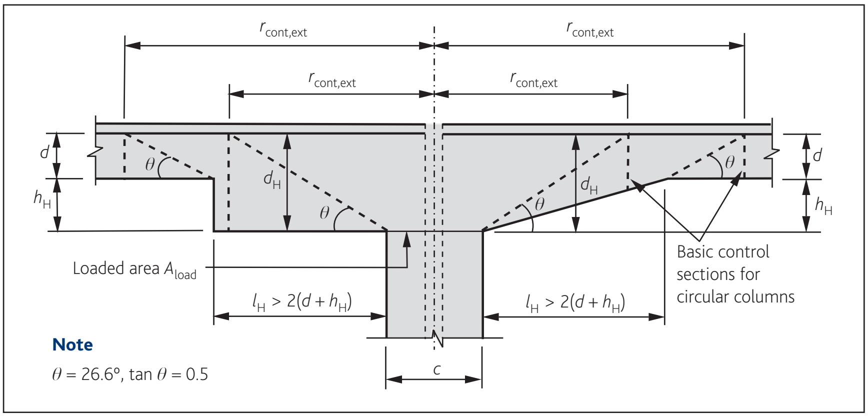


Figure 8.9
Slab with enlarged column head where $l_H > 2(d + h_H)$

8.4 Punching shear resistance *without* shear reinforcement

6.4.4

The basic control section u_1 should be checked to determine whether punching shear reinforcement is required, i.e. whether the applied shear stress, v_{Ed}, exceeds the design punching shear resistance $v_{Rd,c}$ (see Section 7, Table 7.1).

Exp. (6.47) & NA

$v_{Rd,c}$ = $(0.18/\gamma_c)k(100\ \rho_l\ f_{ck})^{0.333} \geq v_{min}$

where

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

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

where

ρ_{ly} and ρ_{lz} = the mean ratios of reinforcement in each direction over a width equal to the column dimension plus $3d$ on each side.

v_{min} = $0.035k^{1.5}\ f_{ck}^{0.5}$

8.5 Punching shear resistance *with* shear reinforcement

6.4.5

At any perimeter, where the applied shear stress v_{Ed} exceeds $v_{Rd,c}$ shear reinforcement should be provided to achieve the necessary resistance using the following relationship.

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 shear reinforcement in one perimeter around the column (for $A_{sw,min}$ see Section 10.4.2)

s_r = radial spacing of perimeters of shear reinforcement

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

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

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

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

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

At the column perimeter, u_0, the punching shear stress should be checked to ensure that

6.4.5(3)

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

where

β = factor dealing with eccentricity (see Section 8.2)
V_{Ed} = applied shear force
d = mean effective depth
u_0 = $2(c_1 + c_2)$ for interior columns
= $c_2 + 3d \le c_2 + 2c_1$ for edge columns
= $3d \le c_2 + 2c_1$ 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

$v_{Rd,max}$ = $0.5 v f_{cd}$

where

v = $0.6\,[1 - (f_{ck}/250)]$

At the column perimeter, $v_{Rd,max} = v_{Rd,max}$ for cot θ = 1.0 given in Table 7.2.

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

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 8.10, 12.5 and 12.6.

6.4.5(4) & NA

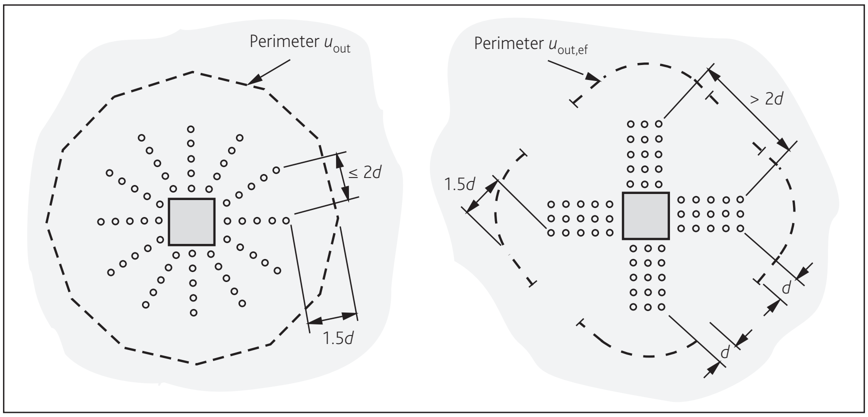


Fig. 6.22

Figure 8.10
Control perimeters at internal columns

8.8 Punching shear resistance of foundation bases

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.

6.4.4(2)

The calculations may be based on a reduced force

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

where

V_{Ed} = column load

ΔV_{Ed} = net upward force within the perimeter considered, i.e. the force due to soil pressure less self-weight of foundation 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:

Exp. (6.51)

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

where

u = the perimeter being considered

k = a coefficient depending on the ratio of the column dimensions shown in Figure 8.2 and the values for inboard columns given in Table 8.1

W ≡ W_1 described in Section 8.2.3 above but for perimeter u

The punching shear resistance $v_{Rd,c}$ and the minimum value of resistance v_{min} 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.

Fig. 6.16

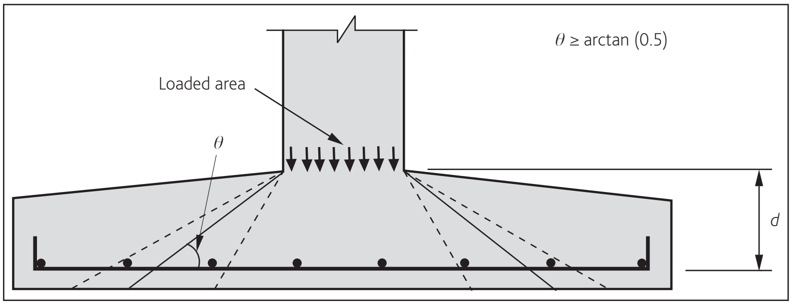


Figure 8.11
Depth of control section in a footing with variable depth

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. 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. 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

The maximum torsional capacity of a non-prestressed section is

$$T_{Rd,max} = 2\nu f_{cd} A_k t_{ef,i} \sin\theta \cos\theta$$ Exp. (6.30)

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

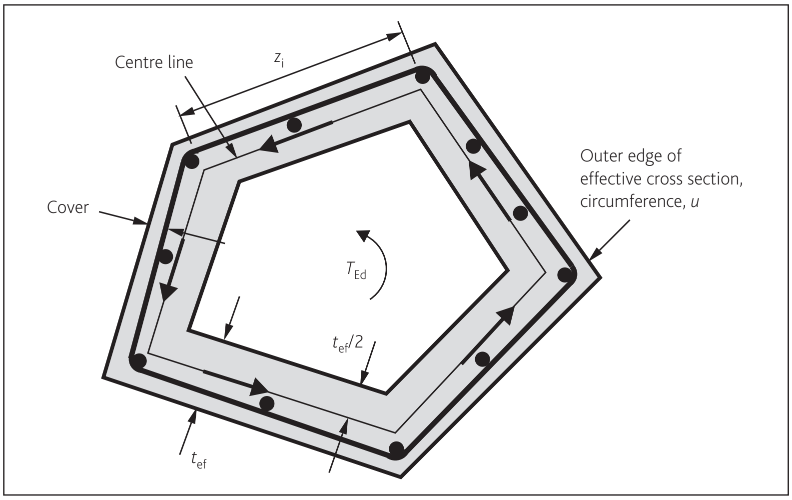


Figure 9.1
Notations used in Section 9

Fig. 6.11

The torsional capacity of a solid rectangular section with shear reinforcement on the outer periphery $T_{Rd,max}$ may be deduced from the general expression:

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

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:

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

where

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

Additional longitudinal reinforcement distributed around the periphery of the section should be provided and the area of this reinforcement should be obtained from the following expression:

Exp. (6.28)

$\sum A_{sl} = T_{Ed} u_k \cot\theta/(f_{yd} 2A_k) = (A_{sw}/s)\, u_k \cot^2\theta$

where

T_{Ed} = applied design torsion
u_k = perimeter of the area A_k
Assuming $f_{yd} = f_{ywd}$

9.3 Combined torsion and shear

In solid sections the following relationship should be satisfied:

Exp. (6.29)

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

where

$T_{Rd,max} = 2\nu f_{cd} A_k t_{ef,i} \sin\theta \cos\theta$ as in Section 9.2.
$V_{Rd,max} = v_w z \nu f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$ as in Section 7.3.2.

10 Serviceability

10.1 Introduction

The common serviceability limit states considered are:

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

For the UK, explicit checks on concrete stresses at serviceability are not normally required, unless lower values for partial factor γ_c than those shown in Section 2, Table 2.3 are used. Similarly, the steel stress need not be checked unless the values for γ_s are smaller than those indicated. 7.2, NA & PD 6687 [7]

In compression members, the provision of links in accordance with detailing rules ensures that there is no significant longitudinal cracking.

Cracking and deflection may be verified by either following calculation procedures or by observing the rules for bar diameters and bar spacing and span-to-effective-depth ratios. This publication does not consider calculation methods.

10.2 Control of cracking

Cracks may be limited to acceptable widths by the following measures:

- Provide a minimum amount of reinforcement, so that the reinforcement does not yield immediately upon formation of the first crack (see Section 10.3).
- Where *restraint* is the main cause of cracking, limit the bar diameter to that shown in Table 10.1. In this case any level of steel stress may be chosen but the chosen value must then be used in the calculation of $A_{s,min}$ and the size of the bar should be limited as shown. 7.3.3(2)
- Where *loading* is the main cause of cracking, limit the bar diameter to that shown in Table 10.1 or limit the bar spacing to that shown in Table 10.2.

  When using either table the steel stress should be calculated on the basis of a cracked section under the relevant combination of actions.

In the absence of specific requirements (e.g. water-tightness), the limiting calculated crack width w_{max} may be restricted to 0.3 mm in all exposure classes under quasi-permanent load combinations. In the absence of specific requirements for appearance, this limit may be relaxed to, say, 0.4 mm for exposure classes X0 and XC1. 7.3.1(5) & NA

In building structures subjected to bending without significant axial tension, specific measures to control cracking are not necessary where the overall depth of the member does not exceed 200 mm.

Table 7.2N

Table 10.1
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	—

Note
Table assumptions include c_{nom} = 25 mm and $f_{ct,eff}$ (= f_{ctm}) = 2.9 MPa

Table 7.3N

Table 10.2
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	—

Note
Table assumptions include c_{nom} = 25 mm and $f_{ct,eff}$ (= f_{ctm}) = 2.9 MPa

7.3.2

10.3 Minimum reinforcement areas of main bars

If crack control is required, the minimum area of reinforcement in tensile zones should be calculated for each part (flanges, web, etc.) as follows:

Exp. (7.1)

$$A_{s,min} = k_c \, k \, f_{ct,eff} \, A_{ct} / \sigma_s$$

where

- k_c = a coefficient to allow 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
  = 1.0 for pure tension and 0.4 for pure bending
- k = a coefficient to allow for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
  = 1.0 for web heights or flange widths ≤ 300 mm and k = 0.65 when these dimensions exceed 800 mm. For intermediate conditions interpolation may be used
- $f_{ct,eff}$ = mean value of the tensile strength of concrete effective at the time cracks may be first expected to occur at the appropriate age. $f_{ct,eff} = f_{ct,m}$ (see Table 3.1)
- A_{ct} = area of concrete in that part of the section which is calculated to be in the tension zone i.e. in tension just before the formation of the first crack
- σ_s = absolute value of the maximum stress permitted in the reinforcement immediately after the formation of the crack. The value should be chosen bearing in mind the limits on bar size and spacing indicated in Tables 10.1 and 10.2

See also Section 12.2.1.

10.4 Minimum area of shear reinforcement

10.4.1 Beams

The minimum area of shear reinforcement in beams $A_{sw,min}$ should be calculated from the following expression:

9.2.2(5) & NA

$A_{sw,min}/(sb_w \sin \alpha) \geq 0.08 f_{ck}^{0.5}/f_{yk}$

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 $\sin \alpha = 1.0$.

10.4.2 Flat slabs

In slabs where punching shear reinforcement is required, the minimum area of a link leg, $A_{sw,min}$ should be calculated from the following expression:

9.4.3(2)

$A_{sw,min}\ (1.5 \sin \alpha + \cos \alpha)/(s_r\ s_t) \geq 0.08 f_{ck}^{0.5}/f_{yk}$

where

s_r and s_t = spacing of shear reinforcement in radial and tangential directions respectively (see Figure 12.5)

10.5 Control of deflection

10.5.1 General

7.4.1, 7.4.2 & NA

The deflection of reinforced concrete building structures will normally be satisfactory if the beams and slabs are sized using the span-to-effective-depth ratios. More sophisticated methods (as discussed in TR58[23]) are perfectly acceptable but are beyond the scope of this publication.

7.4.3

10.5.2 Basic span-to-effective-depth ratios

Basic span-to-effective-depth ratios are given in Table 10.3.

This table has been drawn up on the assumption that the structure will be subject to its design loads only when the concrete has attained the strength assumed in design, f_{ck}. If the structure is to be loaded before the concrete attains f_{ck}, then a more detailed appraisal should be undertaken to take into account the loading and the strength of the concrete at the time of loading.

PD 6687[7]

Table 10.3 is subject to the following conditions of use:

7.4.2(2)

- Values for span-to-effective-depth ratios have been calculated using the criterion that the deflection after construction is span/500 for quasi-permanent loads.
- Table values apply to reinforcement with f_{yk} = 500 MPa, σ_s = 310 MPa, and concrete class C30/37 (f_{ck} = 30).
- Values of basic span-to-effective-depth ratios may be calculated from the formulae:

$l/d = K[11 + 1.5 f_{ck}^{0.5} \rho_0/\rho + 3.2 f_{ck}^{0.5}(\rho_0/\rho - 1)^{1.5}]$ if $\rho_0 \leq \rho$

Exp. (7.16a)

or

$l/d = K[11 + 1.5 f_{ck}^{0.5} \rho_0/(\rho - \rho') + f_{ck}^{0.5}(\rho'/\rho_0)^{0.5}/12]$ if $\rho_0 > \rho$

Exp. (7.16b)

where

l/d	=	limit span-to-effective-depth
K	=	factor to take into account different structural systems
ρ_0	=	reference reinforcement ratio = $f_{ck}^{0.5}/1000$
ρ	=	required tension reinforcement ratio = $A_{s,req}/b_d$ For flanged sections, ρ should be based on an area of concrete above centroid of tension steel
ρ'	=	required compression reinforcement ratio = A_{s2}/b_d

- When the area of steel provided $A_{s,\,prov}$ is in excess of the area required by calculation $A_{s,req}$, the table values should be multiplied by $310/\sigma_s = (500/f_{yk})\ (A_{s,prov}\ /A_{s,req}) \leq 1.5$ where σ_s = tensile stress in reinforcement in midspan (or at support of cantilever) under SLS design loads in MPa.
- In flanged beams where $b_{eff}\ /b_w$ is greater than 3, the table values should be multiplied by 0.80. For values of b_{eff}/b_w between 1.0 and 3.0 linear interpolation should be used.
- The span-to-effective depth ratio should be based on the shorter span in two-way spanning slabs and the longer span in flat slabs.
- When brittle partitions liable to be damaged by excessive deflections are supported on a slab, the table values should be modified as follows:
  a) in flat slabs in which the longer span is greater than 8.5 m, the table values should be multiplied by $8.5/l_{eff}$; and
  b) in beams and other slabs with spans in excess of 7 m, the table values should be multiplied by $7/l_{eff}$.

Table 10.3
Basic ratios of span-to-effective-depth, *l/d*, for members without axial compression

Table 7.4N & NA

Structural system		K	Highly stressed concrete ρ = 1.5%	Lightly stressed concrete ρ = 0.5%
Beams	**Slabs**			
Simply supported	One- or two-way spanning slabs simply supported	1.0	14	20
End span of continuous beams	End span of one-way spanning continuous slabs or two-way spanning slabs continuous over one long edge	1.3	18	26
Interior spans of continuous beams	Interior spans of continuous slabs	1.5	20	30
N/A	Flat slab (based on longer span)	1.2	17	24
Cantilever	Cantilever	0.4	6	8

11 Detailing – general requirements

11.1 General

8.1

These requirements for detailing apply to ribbed reinforcement and welded mesh used in structures subject predominantly to static loading.

8.9.1

The rules apply to single bars and bundled 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.

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.

11.2 Spacing of bars

8.2

Bar spacing should be such that concrete can be placed and compacted satisfactorily for the development of bond.

The clear distance between individual bars and between horizontal layers of bars should not be less than the bar diameter, the aggregate size + 5 mm, or 20 mm, whichever is the greatest.

Where bars are in a number of layers, bars in each layer should be located above each other. The spacing between the resulting columns of bars should be sufficient to allow access for vibrators to give good compaction.

11.3 Mandrel sizes for bent bars

8.3

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:

Table 8.1N & NA

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[24].

The mandrel diameter ϕ_m to avoid crushing of concrete inside the bend need not be checked if:

- Diameters noted above are used; and
- 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.

Otherwise the following minimum mandrel diameter ϕ_m should be used:

$$\phi_m \geq F_{bt}\,((1/a_b) + (1/(2\phi))/\,f_{cd}$$

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

3.1.6(1) & NA

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

where

α_{cc} = 1.0 (treated as a local bearing stress)

f_{ck} = characteristic cylinder strength. Note f_{ck} is limited to 50 MPa

8.4

11.4 Anchorage of bars

11.4.1 General

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 11.1 and 11.2.

Fig. 8.1

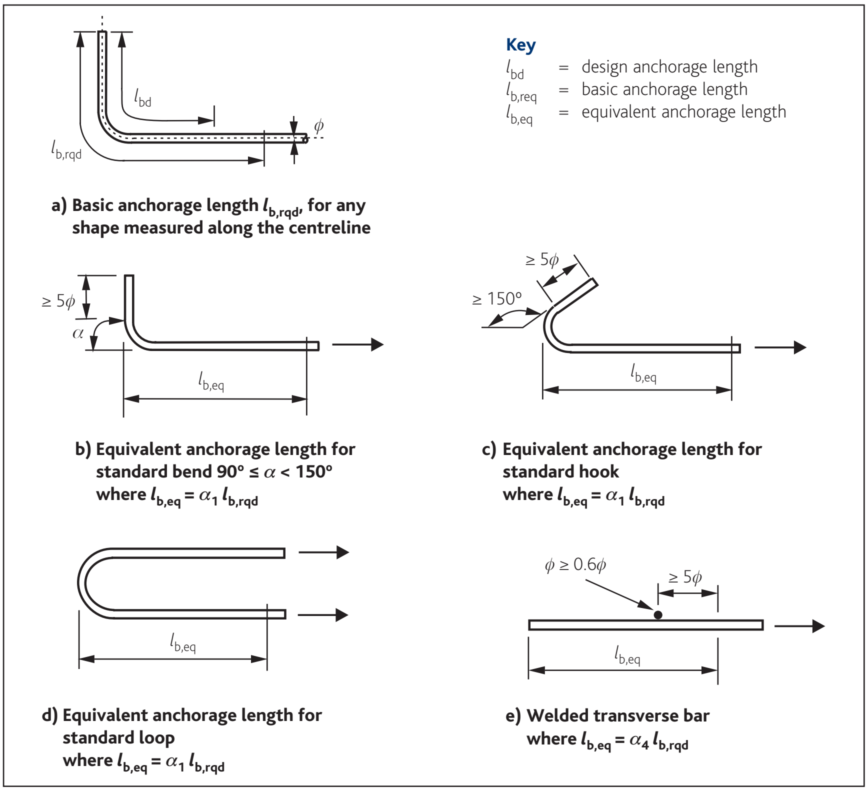


Figure 11.1
Methods of anchorage other than by a straight bar

Fig. 8.5

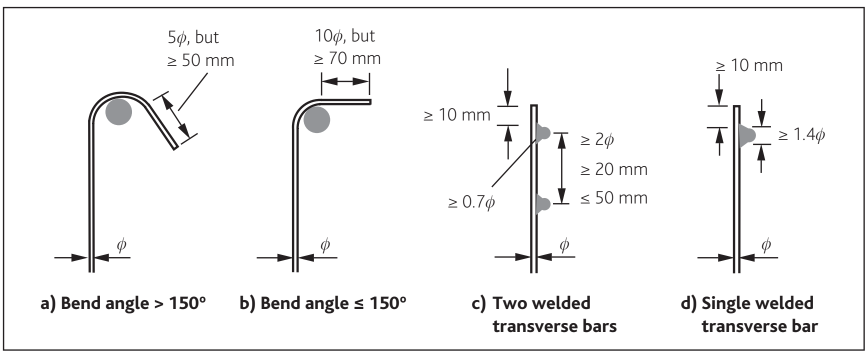


Figure 11.2
Anchorage of links

11.4.2 Design anchorage length l_{bd}

The design anchorage length l_{bd} for the shape shown in Figure 11.1a, may be taken as

8.4.4(1) Table 8.2

$$l_{bd} = \alpha l_{b,rqd} \geq l_{b,min}$$

where

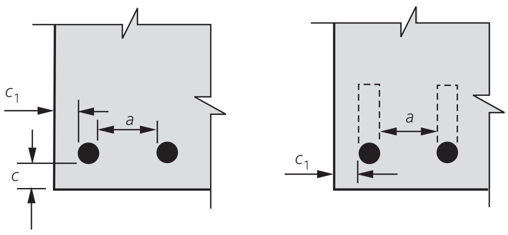
α = 1.0 generally. Otherwise, and less conservatively $\alpha = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5$

where

- α_1 = factor dealing with the shape of bar
  - = 0.7 for bent bars in tension where $c_d > 3\phi$, where c_d is defined in Figure 11.3
  - = 1.0 otherwise for bars in tension
  - = 1.0 for bars in compression
- α_2 = factor dealing with concrete 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 confinement
  - = 1.0 generally
- α_4 = factor dealing with influence of welded transverse bars
  - = 0.7 for a welded transverse bar conforming with Figure 11.1e
  - = 1.0 otherwise
- α_5 = factor dealing with pressure transverse to the plane of splitting
  - = 1.0 generally

$l_{b,rqd}$ = basic anchorage length (see Section 11.4.3)

$l_{b,min}$ = the minimum anchorage length
= maximum of $\{0.3 l_{b,rqd}; 10\phi; 100 \text{ mm}\}$ in tension bars; and
= maximum of $\{0.6 l_{b,rqd}; 10\phi; 100 \text{ mm}\}$ in compression bars.



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

Fig. 8.3

Figure 11.3
Values of c_d for beams and slabs

11.4.3 Basic anchorage length $l_{b,rqd}$

8.4.3

$$l_{b,rqd} = \text{basic anchorage length required} = (\phi/4)\,(\sigma_{sd}/f_{bd})$$

where

ϕ = diameter of the bar
σ_{sd} = design stress in the bar at the ultimate limit state
f_{bd} = ultimate bond stress (see Section 11.5)

The anchorage length should be measured along the centre line of the bar in bent bars.

11.4.4 Equivalent anchorage length $l_{b,eq}$

8.4.4(2)

As a simplification

- For the shapes shown in Figure 11.1 b) 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 11.1e $l_{b,eq} = \alpha_4 l_{b,req}$.

11.5 Ultimate bond stress

The ultimate bond stress

8.4.2(2)

$$f_{bd} = 2.25\, \eta_1\, \eta_2\, f_{ct,d}$$

where

η_1 = 1.0 for 'good' bond conditions (see Figure 11.4 for definition) and 0.7 for all other conditions (which includes elements built using slipforms)

η_2 = 1.0 for bar diameter ≤ 32 mm and (132-ϕ)/100 for bar diameter >32 mm.

$f_{ct,d}$ = ($\alpha_{ct} f_{ct,k}/\gamma_c$) is the design value of tensile strength using the value of $f_{ct,k}$ obtained from Table 3.1 and α_{ct} = 1.0

Fig. 8.2

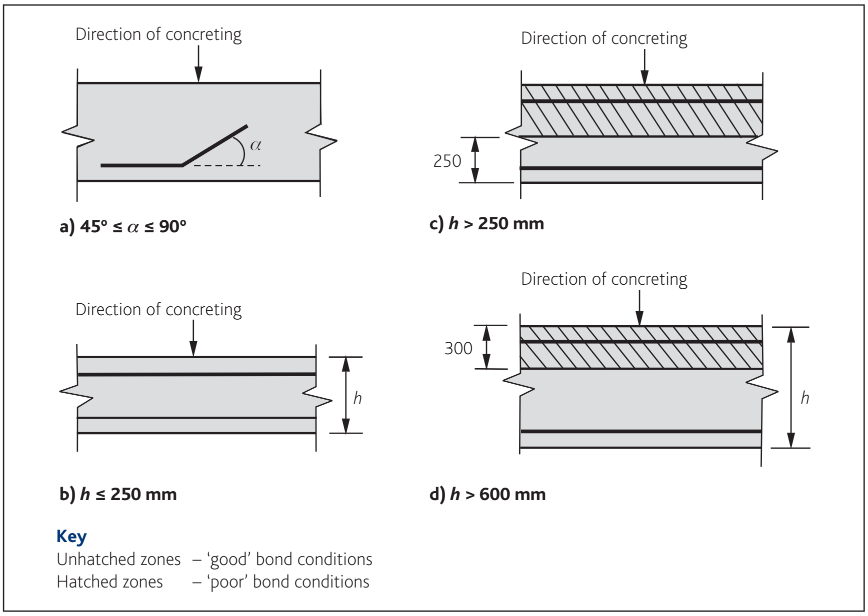


Figure 11.4
Description of bond conditions

11.6 Laps

8.7

11.6.1 General

Forces are transmitted from one bar to another by lapping, welding or using mechanical devices.

Laps of bars in a member should be staggered and not located in areas of high stress. Bars in compression and secondary reinforcement may be lapped at one place.

11.6.2 Lapping bars

Laps of bars should be arranged as shown in Figure 11.5.

The design lap length l_0 may be conservatively taken as:

$$l_0 = \alpha_1 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

Exp. (8.10)

where

α_1 = factor dealing with the shape of bar (see Section 11.4.2)
α_6 = factor dealing with % of reinforcement lapped
= $(\rho_1/25)^{0.5} \leq 0.5$ (see Table 11.1)

where

ρ_1 = percentage of reinforcement lapped within $0.65 l_0$ from the centre line of the lap being considered

Table 11.1
Values of coefficient α_6

ρ_1, % lapped bars relative to total cross-section area	< 25%	33%	50%	> 50%
α_6	1.0	1.15	1.4	1.5

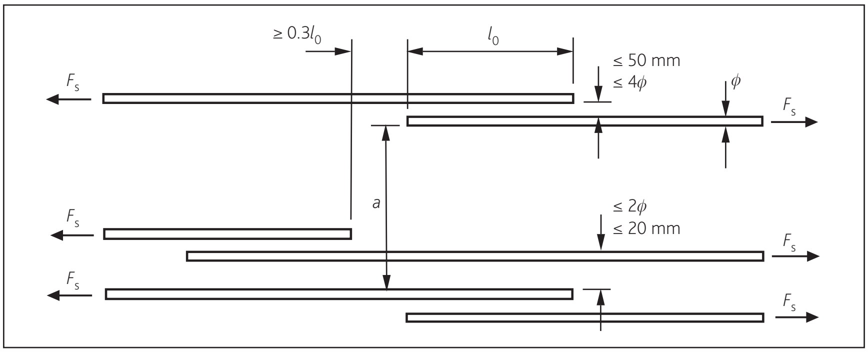


Fig. 8.7

Figure 11.5
Arranging adjacent lapping bars

8.7.5

11.6.3 Lapping fabric

Laps of fabric should be arranged as shown in Figure 11.6.

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 $(A_s/s) \leq 1200$ mm²/m (where s is the spacing of bars) and 60% if $A_s/s > 1200$ mm²/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 $< \phi < 8.5$ mm
  - ≥ 350 mm for 8.5 mm $< \phi < 12$ mm

8.7.5.1(7)
8.7.5.2(1)

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.

Fig. 8.10

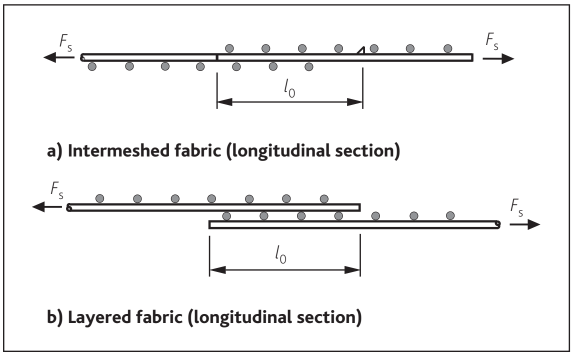


Figure 11.6
Lapping of welded fabric

8.7.4

11.6.4 Transverse reinforcement

Transverse reinforcement to resist the tension forces generated in the lap zone should be provided.

Where the diameter of the lapped bar is less than 20 mm or if the percentage of reinforcement lapped at a section is less than 25%, links and transverse reinforcement provided for other purposes may be deemed adequate.

When the above conditions do not apply, transverse reinforcement should be provided as shown in Figure 11.7. Where more than 50% of bars are lapped at one section and the spacing between adjacent laps (dimension a in Figure 11.5) $< 10\phi$, the transverse reinforcement should be in the form of links or U bars anchored into the body of the section.

In Figure 11.7, the total area of transverse reinforcement at laps $\sum A_{st} > A_s$ of one lapped bar.

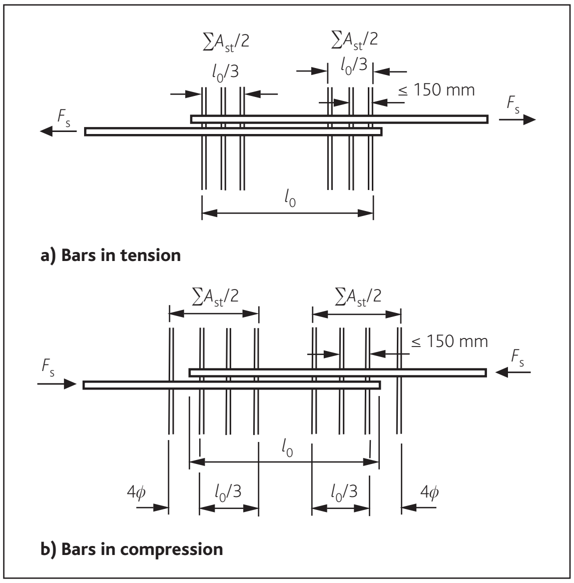


Fig. 8.9

Figure 11.7
Transverse reinforcement for lapped splices

11.6.5 Lapping large bars

8.8

For bars larger than 40 mm in diameter the following additional requirements apply:

- Bars should generally be anchored using mechanical devices. Where anchored as straight bars, links should be provided as confining reinforcement.
- Bars should not be lapped except in section 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 11.8.

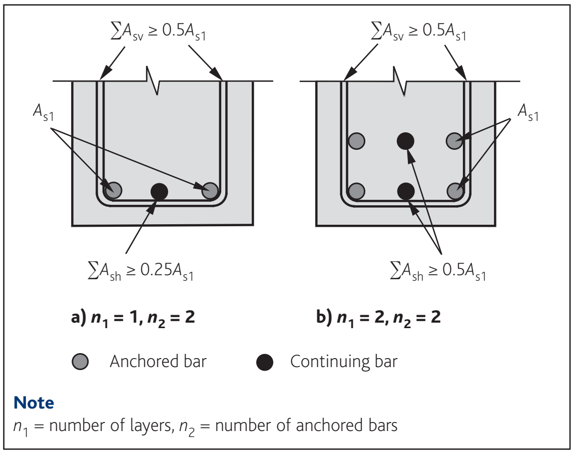


Fig. 8.11

Figure 11.8
Additional reinforcement in an anchorage for large diameter bars where there is no transverse compression

12 Detailing – particular requirements

12.1 General

9.1

This section gives particular requirements for detailing of structural elements. These are in addition to those outlined in Sections 10 and 11. The member types covered here are beams, slabs, columns and walls.

12.2 Beams

9.2.1.1

12.2.1 Longitudinal bars

The minimum area of longitudinal reinforcement $A_{s,min}$ is given in Table 12.1 and by:

$$A_{s,min} = 0.26(f_{ctm}/f_{yk})\ b_t d \geq 0.0013 b_t d$$

where

f_{ctm} = mean axial tensile strength (see Table 3.1)
f_{yk} = characteristic yield strength of reinforcement
b_t = mean breadth of the tension zone
d = effective depth

Outside lap locations the maximum area of tension or compression reinforcement is $0.04A_c$.

All longitudinal compression reinforcement should be held by transverse reinforcement with spacing not greater than 15 times the diameter of the longitudinal bar.

Table 12.1
Minimum area of longitudinal reinforcement as a proportion of $b_t d$

Strength class	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C28/35	C32/40
$A_{s,min}$ as a % of $b_t d$	0.130	0.130	0.130	0.135	0.151	0.166	0.182	0.198	0.213	0.146	0.156

9.2.1.3

12.2.2 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 included assuming linear variation of force, or ignored.

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 12.1, 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 $a_l = 1.25z$.

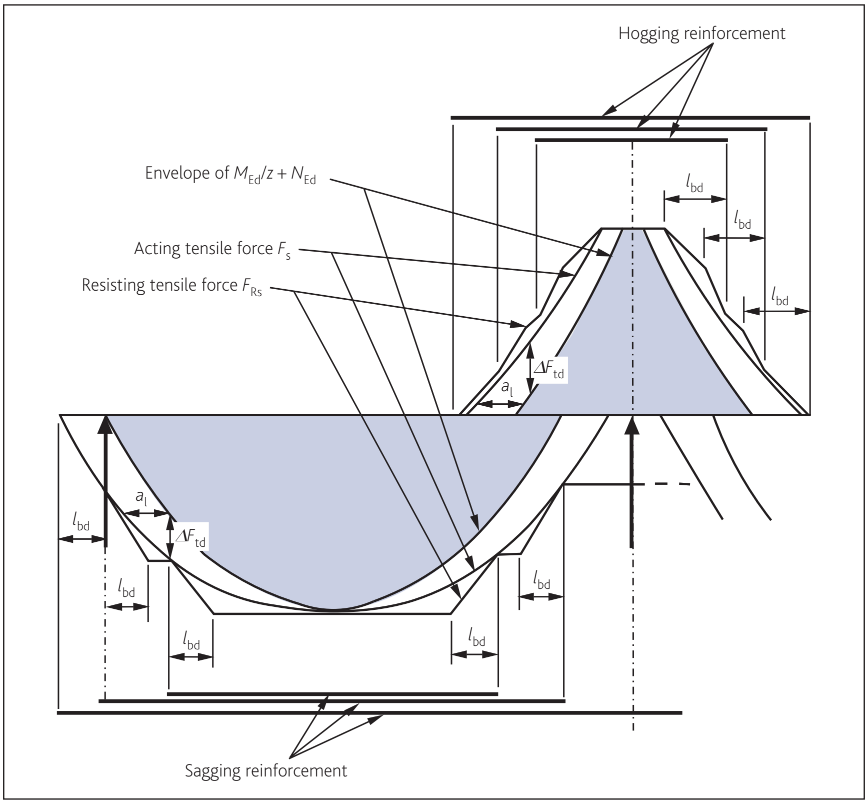


Fig. 9.2

Figure 12.1
Illustration of the curtailment of longitudinal reinforcement, taking into account the effect of inclined cracks and the resistance of reinforcement within anchorage lengths

12.2.3 Top reinforcement in end supports

In monolithic construction, supports should be designed for bending moment arising from partial fixity, even if simple supports have been assumed in design. The bending moment should be taken as 25% of the maximum bending design moment in span.

9.2.1.2(1) & NA

12.2.4 Bottom reinforcement in end supports

Where there is little or no fixity at an end support, bottom reinforcement with an area of at least 25% of the area of the steel in the span should be provided. The bars should be anchored to resist a force, F_E, of

$$F_E = (|V_{Ed}|a_l/z) + N_{Ed}$$

where

$|V_{Ed}|$ = absolute value of shear force
N_{Ed} = axial force if present and a_l is as defined in Section 12.2.2

9.2.1.4 & NA

The anchorage should be measured from the line of contact between the beam and the support.

12.2.5 Intermediate supports

9.2.1.2(2)

At intermediate supports, the tension reinforcement may be spread over b_{eff} (as defined in Figure 5.2).

12.2.6 Shear reinforcement

9.2.2(4)
9.2.2(6)

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 $0.75d(1 + \cot \alpha)$, 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.

12.2.7 Torsion reinforcement

9.2.3

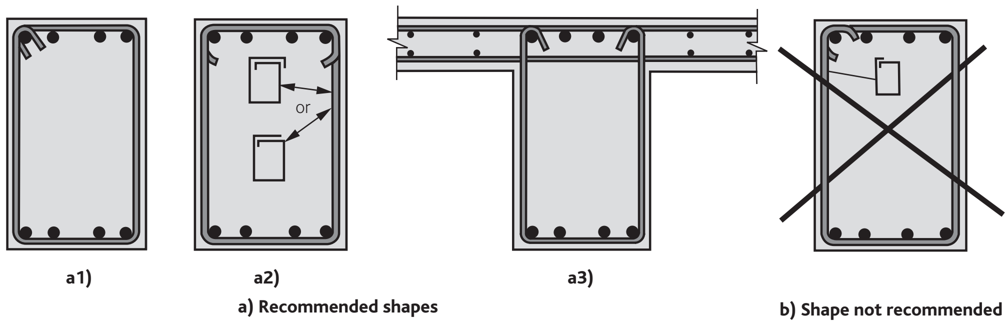
Where links are required for torsion, they should comply with the anchorage shown in Figure 12.2. The maximum longitudinal spacing of the torsion links $s_{l,max}$ should be:

$s_{l,max} \leq$ minimum of $\{u/8;\ 0.75d\ (1 + \cot \alpha);\ h;\ b\}$

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

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.

Fig. 9.6



Note
The second alternative for a2) should have a full lap length along the top

Figure 12.2
Examples of shapes for torsion links

12.2.8 Indirect supports

9.2.5

Where a beam is supported by another beam, adequate reinforcement should be provided to transfer the reaction. Where the loads are hung, this reinforcement, which is additional to other reinforcement, should be in the form of links surrounding the principal reinforcement of the supporting member. Some of these links may be placed outside the volume of concrete common to the two beams. See Figure 12.3.

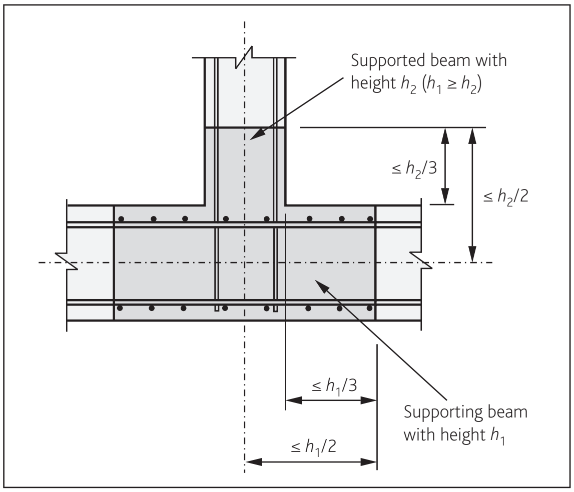


Fig. 9.7

Figure 12.3
Plan section showing supporting reinforcement in the intersection zone of two beams

9.3

12.3 One-way and two-way spanning slabs

12.3.1 Main reinforcement

9.3.1.1(1)
9.2.1(1)

The minimum area of main reinforcement is given in Table 12.1 and, as before, is derived from the expression:

$A_{s,min} = 0.26\ (f_{ctm}/f_{yk})b_t d \geq 0.0013 b_t d$

9.2.1.2(3)

Outside lap locations the maximum area of tension or compression reinforcement is $0.04A_c$.

All compression reinforcement should be held by transverse reinforcement with spacing not greater than 15 times the diameter of the main bar.

9.3.1.1(3)
& NA

The spacing of main reinforcement should generally not exceed 3*h* (but not greater than 400 mm), where *h* is the overall depth of the slab. The spacing should be reduced to 2*h* (but not greater than 250 mm), in areas of maximum moment or local to concentrated loads.

12.3.2 Secondary (distribution) reinforcement

The area of secondary reinforcement should not be less than 20% of the main reinforcement.

The spacing of secondary reinforcement should generally not exceed 3.5*h* (but not greater than 450 mm). In areas of maximum bending moment or local to concentrated load the spacing should be reduced to 3*h* (but not greater than 400 mm).

9.3.1.2

12.3.3 Reinforcement near supports

In simply supported slabs 50% of the reinforcement in the span should continue up to the support. The remaining bars should be anchored to resist a force of

$(|V_{Ed}| a_l / z) + N_{Ed}$

where

$|V_{Ed}|$ = the absolute value of the shear force
a_l = *d*, where the slab is not reinforced for shear. (If reinforced for shear use a_l for beams)
z = lever arm of internal forces
N_{Ed} = the axial force if present

The anchorage should be measured from the line of contact between the beam and the support.

When longitudinal bars are curtailed, longitudinal tensile forces arising both from bending moment and from the truss model for shear should be considered. This can be achieved by displacing the location where a bar is no longer required for bending moment by a distance a_l as defined above.

Where any partial fixity exists along the edge of a slab but is not taken into account in design, top reinforcement capable of resisting at least 25% of the maximum moment in the adjacent span should be provided and it should extend at least 0.2 times the length of the adjacent span. At end supports this moment may be reduced to 15%.

9.3.2

12.3.4 Shear reinforcement

Shear reinforcement should not be relied upon in slabs with a depth of less than 200 mm. Where shear reinforcement is provided the rules for beams should be followed.

12.4 Flat slabs

9.4.1

12.4.1 Details at internal columns

Irrespective of the division of flat slabs into column strips and middle strips (see Section 5.7) 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 is the area of reinforcement required to resist the sum of the negative moments in the two half panels on either side of the column. At least two bottom bars passing through the column should be provided in each orthogonal direction.

9.4.2

12.4.2 Details at edge and corner columns

Reinforcement perpendicular to the free edge required for the transfer of bending moments between the slab and the column should be placed within the effective width b_e shown in Figure 12.4.

As far as possible, at least two bottom bars passing through the column should be provided in each orthogonal direction. See also Section 13.4.

9.4.3

12.4.3 Punching shear reinforcement

Where punching shear reinforcement is required, it should generally be placed between the loaded area and 1.5d inside the outer control perimeter at which reinforcement is no longer required, u_{out}.

The tangential spacing of link legs should not exceed 1.5d along the first control perimeter, u_1, at 2d from the loaded area (see Figure 8.3). Beyond the first control perimeter the spacing should not exceed 2d (see Figure 12.5). For non-rectangular layouts see Figure 8.10.

The intention is to provide an even distribution/density of punching shear reinforcement within the zone where it is required. One simplification to enable rectangular perimeters of shear reinforcement is to use an intensity of A_{sw}/u_1 around rectangular perimeters.

6.4.5(4) & NA

If the perimeter at which reinforcement is no longer required is less than 3d from the face of the loaded area, the shear reinforcement should be placed in the zone 0.3d and 1.5d from the face of the loaded area. It should be provided in at least two perimeters of links, with the radial spacing of link legs not exceeding 0.75d. See Figure 12.6.

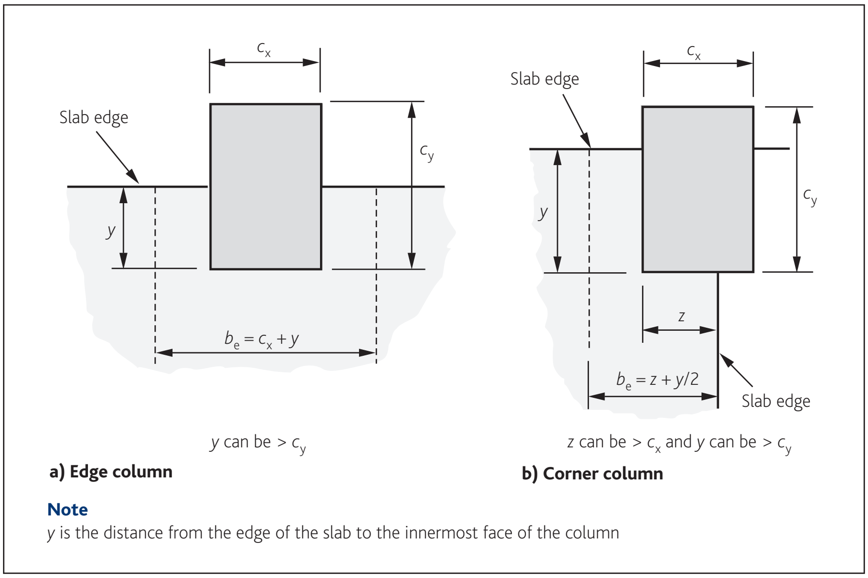


Figure 12.4
Effective width, b_e, of a flat slab

Fig. 9.9

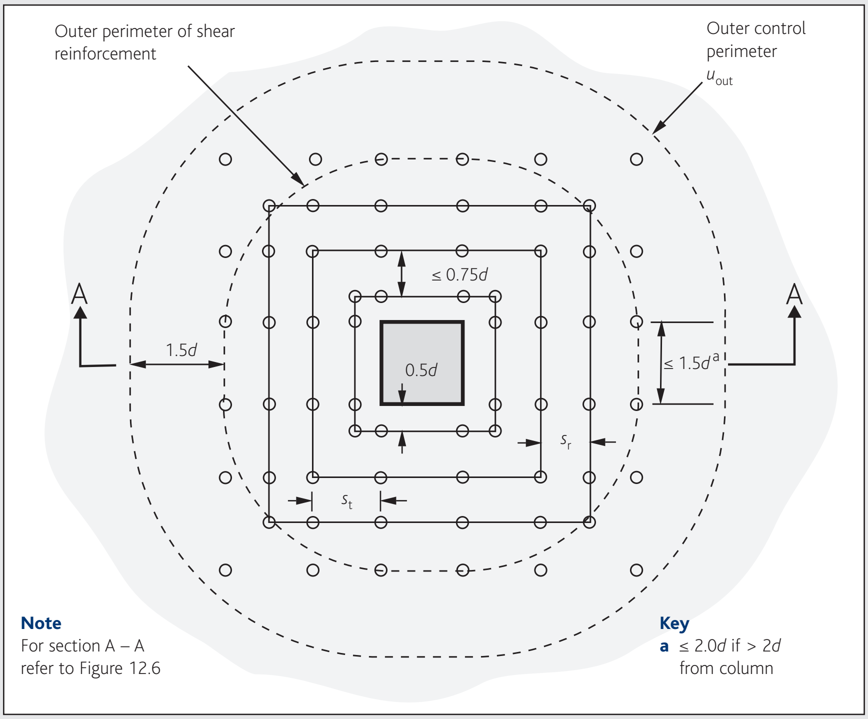


Figure 12.5
Layout of flat slab shear reinforcement

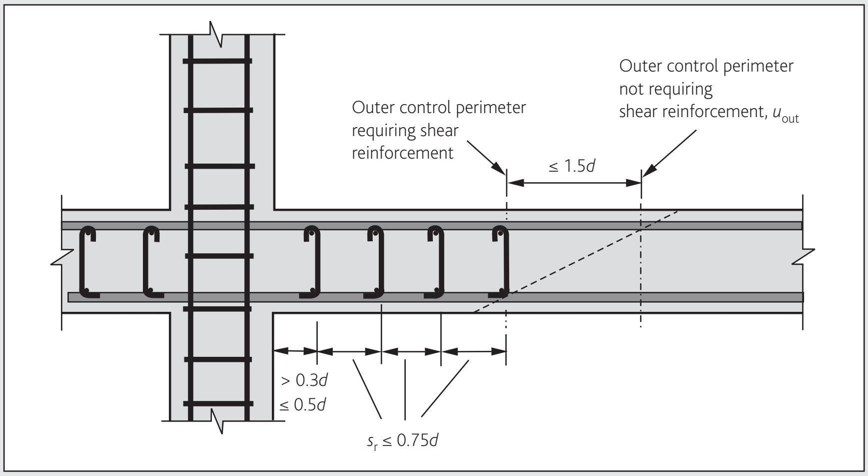


Figure 12.6
Section A – A from Figure 12.5: spacing of punching shear reinforcing links

12.5 Columns

9.5.2 & NA

12.5.1 Longitudinal reinforcement

The diameter of bars should not be less than 12 mm.

The minimum area of longitudinal reinforcement, $A_{s,min}$, is given by:

$A_{s,min} \geq$ maximum $(0.1N_{Ed}/f_{yd}; 0.002A_c)$

where

N_{Ed} = axial force
f_{yd} = design yield strength of reinforcement
A_c = cross sectional area of concrete

The area of reinforcement should not generally exceed $0.04A_c$ outside laps and $0.08A_c$ at laps. The practical upper limit should be set by taking into account the ability to place and compact the concrete.

9.5.3 & NA

12.5.2 Transverse reinforcement (links)

All longitudinal bars should be held by adequately anchored transverse reinforcement. The number of transverse links in a cross section should be such that there is no longitudinal bar further than 150 mm from a restrained bar.

The diameter of bars should not be less than 6 mm or one quarter of the diameter of the longitudinal bars, whichever is greater.

The spacing of transverse reinforcement should be the least of
a) 20 times the diameter of the longitudinal bar or
b) the lesser dimension of the column or
c) 400 mm.

The spacing should be reduced to 60% of the above value
a) for a distance equal to the larger column dimension above and below a beam or slab; and
b) over the lap length of bars larger than 14 mm. (Note: a minimum of three transverse bars should be used.)

Where the longitudinal bars are cranked at an inclination greater than 1 in 12, the spacing of transverse reinforcement should be calculated taking into account the transverse forces induced. 9.5.3(5)

12.6 Walls

12.6.1 Vertical reinforcement

9.6.2 & NA

Vertical reinforcement should be provided with a minimum area of $0.002A_c$ and a maximum of $0.04A_c$ outside laps and $0.08A_c$ at laps.

The spacing between the bars should not exceed 3 times the wall thickness or 400 mm, whichever is less.

12.6.2 Horizontal reinforcement

9.6.3 & NA

Horizontal reinforcement parallel to the faces of the wall should be provided with a minimum area equal to either 25% of the vertical reinforcement or $0.001A_c$, whichever is greater. Early age thermal and shrinkage effects should be considered when crack control is important.

The spacing of the bars should not exceed 400 mm.

12.6.3 Transverse reinforcement

9.6.4

In any part of a wall where the area of the vertical reinforcement exceeds $0.02A_c$, transverse reinforcement in the form of links should be provided in accordance with the rules for columns.

Transverse reinforcement is also required when the vertical bars form the outer layer of reinforcement. At least four links per m^2 of wall should be provided. This does not apply to welded mesh or for main bars ≤16 mm with a cover of twice the bar diameter.

12.7 Pile caps

9.8.1

The pile caps should be sized taking into account the expected deviation of the pile on site and should be such that the tie forces can be properly anchored.

The pile caps should be designed by modelling them as flexural members or as comprising struts and ties. The main tensile reinforcement should be concentrated in stress zones between the tops of the piles, within 1.5D of the centreline of the piles (where D is pile diameter). The minimum diameter of bars should be 8 mm.

Normally a cage of evenly distributed reinforcement should be provided on all faces of the pile cap.

9.8.5
& NA

12.8 Bored piles

Bored piles not exceeding 600 mm in diameter should have the minimum reinforcement shown in Table 12.2. 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[25].

Table 12.2
Longitudinal reinforcement in cast-in-place bored piles

Area of cross section of the pile (A_c)	$A_c \leq 0.5$ m^2	0.5 m$^2 \leq$ 1.0 m^2	$A_c > 1.0$ m^2
Minimum area of longitudinal reinforcement ($A_{s,bpmin}$)	$\geq 0.005A_c$	$\geq$ 25 cm^2	$\geq 0.0025A_c$

13 Tying systems

13.1 General

This Section should be considered in conjunction with the UK Building Regulations[26] and the relevant Approved Documents, which classify the buildings based on the use of the building and also specify the types of ties that are required in each class. The forces should be considered as accidental loads.

All structures should have a suitable tying system to prevent disproportionate collapse caused by human error or the accidental removal of a member or limited part of the structure or the occurrence of localised damage. This requirement will be satisfied if the following rules are observed. 9.10.1

The structure should have:

- Peripheral ties.
- Internal ties.
- Horizontal ties at columns/walls.
- Vertical ties.

In the design of ties, reinforcement should be provided to carry the tie forces noted in this Section, assuming that the reinforcement acts at its characteristic strength. Reinforcement provided for other purposes may be regarded as providing part or the whole of the required reinforcement.

All ties should be effectively continuous and be anchored at their ends.

13.2 Peripheral ties

At each floor and roof level, an effectively continuous tie should be provided within 1.2 m from the edge. Structures with internal edges (e.g. atria and courtyards) should also have similar peripheral ties. 9.10.2.2 & NA

The peripheral tie should be able to resist a tensile force of:

$F_{tie,per}$ = $(20 + 4n_0)$ kN $\leq$ 60 kN where n_0 = number of storeys

13.3 Internal ties

At each floor and roof level, internal ties should be provided in two directions approximately at right angles. 9.10.2.3 & NA

The internal ties, in whole or in part, may be spread evenly in slabs or may be grouped at or in beams, walls or other positions. If located in walls, the reinforcement should be within 0.5 m of the top or bottom of the floor slabs.

In each direction the tie needs to be able to resist a force, which should be taken as:

$F_{tie,int} = (1/7.5)(g_k + q_k)(l_r/5)F_t \geq F_t$

where

$(g_k + q_k)$ = average permanent and variable floor actions (kN/m²)
l_r = greater of the distances (in m) between centres of the columns, frames or walls supporting any two adjacent floor spans in the direction of the tie under consideration
F_t = $(20 + 4n_0) \leq 60$ kN (n_0 is the number of storeys)

The maximum spacing of internal ties should be limited to $1.5l_r$.

13.4 Ties to columns and walls

9.10.2.4 & NA

Columns and walls at the edge and corner of the structure should be tied to each floor and roof. In corner columns and walls ties should be provided in two directions.

The tie should be able to resist a force of:

$F_{\text{tie, fac}} = F_{\text{tie, col}} = \text{maximum } (2F_t;\ l_s F_t/2.5;\ 0.03\ N_{\text{Ed}})$

where

$F_{\text{tie,fac}}$ = in kN/m run of wall
$F_{\text{tie,col}}$ = in kN/column.
F_t = defined in Section 13.3 above
l_s = floor to ceiling height (in metres)
N_{Ed} = total design ultimate vertical load in wall or column at the level considered

Tying of external walls is required only if the peripheral tie is not located within the wall.

13.5 Vertical ties

9.10.2.5

BS EN1992-1-1 requires vertical ties in panel buildings of five storeys or more.

AD A[27] PD 6687[7]

However, relevant current UK Building Regulations require such ties in all buildings that fall into Class 2B and 3 as defined in Section 5 of Approved Document A[27] (see Table 13.1). In all such buildings vertical ties should be provided in columns and/or walls.

9.10.2.5(2)

Each column and wall carrying vertical load should be tied continuously from the lowest to the highest level. The tie should be capable of resisting the load received by the column or wall from any one storey under accidental design situation i.e. using Exp. (6.11b) in BS EN 1990. See Table 2.2e.

Where such ties are *not* provided either:

- The vertical member should be demonstrated for 'non-removability'. Non-removability may be assumed if the element and its connections are capable of withstanding a design action at limit state of 34 kN/m^2 in any direction over the projected area of the member together with the reactions from attached components, which themselves are subject to a loading of 34 kN/m^2. These reactions may be limited to maximum reaction that can be transmitted; or
- Each element should be considered to be removed one at a time and an alternative path demonstrated.

Where a column or wall is supported at its lowest level by an element other than a foundation, alternative load paths should be provided in the event of the accidental loss of this element.

Table 13.1
Building classes from *Approved Document A – structure* (2004 edition)[27]

Class	Building type and occupancy
1	Houses not exceeding 4 storeys Agricultural buildings Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2A	5 storey single occupancy houses Hotels not exceeding 4 storeys Flats, apartments and other residential buildings not exceeding 4 storeys Offices not exceeding 4 storeys Industrial buildings not exceeding 3 storeys Retailing premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey Single storey educational buildings All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey
2B	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys Educational buildings greater than 1 storey but not exceeding 15 storeys Retailing premises greater than 3 storeys but not exceeding 15 storeys Hospitals not exceeding 3 storeys Offices greater than 4 storeys but not exceeding 15 storeys All buildings to which members of the public are admitted which contain floor areas exceeding 2000 m^2 but less than 5000 m^2 at each storey Car parking not exceeding 6 storeys
3	All buildings defined above as Class 2A and 2B that exceed the limits on area and/or number of storeys Grandstands accommodating more than 5000 spectators Buildings containing hazardous substances and/or processes

Notes

1 For buildings intended for more than one type of use, the class should be that pertaining to the most onerous type.

2 In determining the number of storeys in a building, basement storeys may be excluded provided such basement storeys fulfil the robustness requirements of Class 2B buildings.

14 Plain concrete

14.1 General

1.5.2.2

A plain concrete member is one containing no reinforcement. Members in which the reinforcement provided is less than the minimum amounts given in Section 12 should also be treated as plain concrete for the purposes of structural design.

12.3.1 & NA

The design compressive strength $f_{cd,pl} = 0.6\,f_{ck}/\gamma_c$ (as shown in Table 14.1).

Generally the design tensile strength $f_{ctd,pl} = 0.6\,f_{ctk,0.05}/\gamma_c$ (as shown in Table 14.1).

Table 14.1
Properties of plain concrete (MPa)

Strength class (MPa)	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck}	12	16	20	25	30	35	40	45	50
$f_{cd,pl}$	4.8	6.4	8.0	10.0	12.0	14.0	16.0	18.0	20.0
$f_{ctk,0.05}$	1.10	1.33	1.55	1.80	2.03	2.25	2.46	2.66	2.85
$f_{ctd,pl}$	0.44	0.53	0.62	0.72	0.81	0.90	0.98	1.06	1.14
$\sigma_{c,lim}$	1.76	2.55	3.38	4.45	5.55	6.68	7.83	9.00	10.18

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

The design tensile strength in flexural members may be taken as:

3.1.8

$$f_{ctd,pl,fl} = (1.6 - h/1000)f_{ctd,pl} \nless f_{ctd,pl}$$

where
- h = depth of the member in mm
- f_{ctm} = value in Table 3.1
- $f_{ctd,pl}$ = value in Table 14.1

14.2 Bending and axial force

12.6.1(3)

The axial resistance N_{Rd} of a rectangular cross section with an eccentricity of load e as shown in Figure 14.1, may be taken as:

$$N_{Rd} = f_{cd}bh_w(1 - 2e/h_w)$$

where
- b = overall width
- h_w = overall depth
- e = eccentricity of N_{Ed} in the direction h_w

The value of N_{Rd} given above assumes there is no buckling involved.

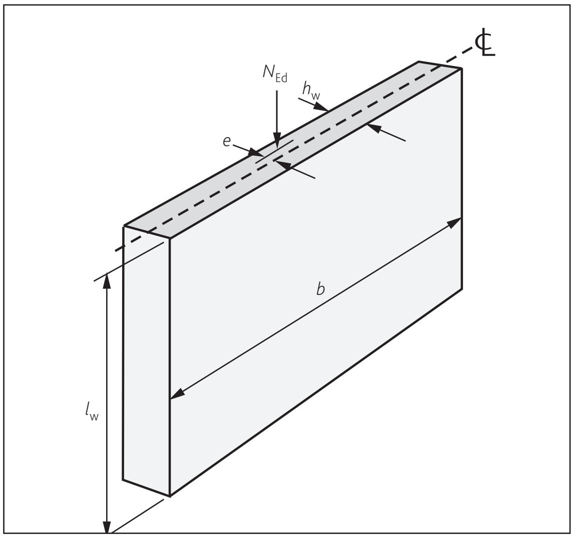


Fig. 12.1

Figure 14.1
Notation for plain walls

14.3 Shear resistance

12.6.3(2) & NA

It should be verified that

τ_{cp} = shear stress = $1.5\ V_{Ed}/A_{cc} \leq f_{cvd}$

where

A_{cc} = cross sectional area

V_{Ed} = shear force

f_{cvd} = concrete design strength in shear and compression and is dependent on the level of axial stress (see Table 14.2)

= $(f_{ctd,pl}{}^{2} + \sigma_{cp} f_{ctd,pl})^{0.5}$ when $\sigma_{cp} \leq \sigma_{c,lim}$

= $[f_{ctd,pl}{}^{2} + \sigma_{cp} f_{ctd,pl} - 0.25(\sigma_{cp} - \sigma_{c,lim})^{2}]^{0.5}$ when $\sigma_{cp} > \sigma_{c,lim}$

where

σ_{cp} = N_{Ed}/A_{cc} when N_{Ed} = normal force

$\sigma_{c,lim}$ = $f_{cd,pl} - 2[f_{ctd,pl}(f_{ctd,pl} + f_{cd,pl})]^{0.5}$ (see Table 14.1)

Table 14.2
Shear resistance f_{cvd} of plain concrete (MPa)

σ_{cp} (MPa)	f_{ck}								
	12	16	20	25	30	35	40	45	50
0.0	0.44	0.53	0.62	0.72	0.81	0.90	0.98	1.06	1.14
1.0	0.80	0.90	1.00	1.11	1.21	1.31	1.40	1.48	1.56
2.0	1.03	1.16	1.27	1.40	1.51	1.61	1.71	1.80	1.89
3.0	1.06	1.35	1.50	1.63	1.76	1.87	1.98	2.08	2.17
4.0	0.84	1.38	1.66	1.84	1.98	2.10	2.21	2.32	2.42
5.0		1.21	1.68	2.01	2.17	2.30	2.42	2.54	2.65
6.0		0.72	1.54	2.06	2.34	2.49	2.62	2.74	2.85
7.0			1.20	1.98	2.41	2.66	2.80	2.93	3.05
8.0				1.76	2.38	2.75	2.97	3.10	3.23
9.0				1.34	2.23	2.75	3.08	3.27	3.40
10.0				0.00	1.96	2.65	3.10	3.39	3.56

Note
Table derived from BS EN 1992-1-1 and National Annex

14.4 Buckling resistance of columns and walls

12.6.5.2(1)

The axial load that can be resisted by a wall with cross section bh_w may be taken as:

$$N_{Rd} = bh_w f_{cd} \phi$$

where

b = overall width of cross section
h_w = overall depth of cross section
f_{cd} = design value of concrete compressive strength
= $\alpha_{cc,pl} f_{ck}/\gamma_c$

where

$\alpha_{cc,pl}$ = 0.6
f_{ck} = characteristic cylinder strength
γ_c = partial factor for concrete

ϕ = factor accounting for eccentricity including second order effects and creep
= $1.14(1 - 2e_{tot}/h_w) - 0.02(l_0/h_w)$

where

$e_{tot} = e_0 + e_i$

where

e_0 = first order eccentricity caused by floor loads and any horizontal actions
e_i = additional eccentricity due to geometrical imperfections as defined in Section 5.6.2.1.

12.3.1(1) & NA

l_0 = effective length of column/wall
= βl_w

where

β = coefficient obtained from Table 14.3
l_w = clear height of the members

Table 14.3
The value of β for walls with different boundary conditions

Table 12.1

Boundary condition	Wall length/wall height	Coefficient β
Restrained at top and bottom	All values	1.00
Restrained at top and bottom and along one vertical edge	0.2	0.26
	0.4	0.59
	0.6	0.76
	0.8	0.85
	1.0	0.90
	1.5	0.95
	2.0	0.97
	5.0	1.00
Restrained on all four edges	0.2	0.10
	0.4	0.20
	0.6	0.30
	0.8	0.40
	1.0	0.50
	1.5	0.69
	2.0	0.80
	5.0	0.96
Cantilevers	All values	2.00

Table 14.3 is valid for walls where:

- There are no openings.
- The height of openings does not exceed 33% of the height of the wall.
- The area of openings does exceed 10% of the area of the wall.

If the conditions for the openings are not satisfied, the wall should be considered as restrained at top and bottom only.

The slenderness ratio $\lambda = l_0/i$ should not exceed 86, where i = radius of gyration. For rectangular sections $i = h_w/3.46$ (i.e. l_0/h_w should not exceed 25). 12.6.5.1

Any wall that provides restraint to another wall should satisfy the following requirements:

- Thickness of the bracing wall should be at least 50% of the thickness of the wall being braced.
- Heights of the bracing and braced walls are the same.
- Length of the bracing wall should be at least 20% of the clear height of the braced wall and there are no openings within this length.

Where there is structural continuity between the floors and the wall, the values of β given in Table 14.3 may be multiplied by 0.85.

14.5 Serviceability limit states

12.7
12.9.1

Normally special checks are not necessary where joints are provided to limit the tensile stresses caused by restraint, and walls are at least 120 mm thick. Where significant chases or recesses are incorporated it may be necessary to carry out checks.

14.6 Strip and pad foundations

12.9.3

The ratio of the depth of a strip foundation to its projection from the column/wall face may be designed to satisfy the following relationship:

$$(h_f/a) \geq 1.18(9\sigma_{gd}/f_{ctd})^{0.5}$$

where

h_f = depth of the footing
a = projection of the footing from the face of the column or wall
σ_{gd} = design value of the ground pressure
f_{ctd} = design value of concrete tensile strength in the same units as σ_{gd}

15 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.

15.1 Design values of actions

Section 2.3.4

For the ULS of strength (STR) where there is a single variable action use either:

- $1.35G_k + 1.5Q_k$ Exp. (6.10) from Eurocode[5]

or the worse case of

- $1.35G_k + \psi_0 1.5Q_k$ Exp. (6.10a)
- $1.25G_k + 1.5Q_k$ Exp. (6.10b)

  where ψ_0 = 1.0 for storage, 0.5 for snow but otherwise 0.7, see Table 2.2.

In most cases Exp. (6.10b) will be appropriate, except for storage where the use of Exp. (6.10a) is likely to be more onerous.

Section 2.3.4

For the SLS of deformation, quasi-permanent loads should be applied. These are $1.0G_k + \psi_2 Q_k$ where ψ_2 is dependent on use, e.g. 0.3 for offices and residential and 0.8 for storage. Again, see Table 2.2.

15.2 Values of actions

Section 2.3.4

The values of actions (i.e. loads) are defined in Eurocode 1, BS EN 1991[6]. The parts of Eurocode 1 are given in Table 15.1. These values are taken as characteristic values. At the time of publication, the UK National Annexes to these parts are in various states of readiness.

As PD 6687[7] makes clear, until the appropriate European standards become available, designers may consider using current practice or current British Standards in conjunction with BS EN 1992, provided they are compatible with BS EN 1992 and that the resulting reliability is acceptable.

BS EN 1991-1-1 states that the density of concrete is 24 kN/m^3, reinforced concrete, 25 kN/m^3 and wet reinforced concrete, 26 kN/m^3.

Table 15.1
The parts of Eurocode 1[6]

Reference	Title
BS EN 1991-1-1	Densities, self-weight and imposed loads
BS EN 1991-1-2	Actions on structures exposed to fire
BS EN 1991-1-3	Snow loads
BS EN 1991-1-4	Wind actions
BS EN 1991-1-5	Thermal actions
BS EN 1991-1-6	Actions during execution
BS EN 1991-1-7	Accidental actions due to impact and explosions
BS EN 1991-2	Traffic loads on bridges
BS EN 1991-3	Actions induced by cranes and machinery
BS EN 1991-4	Actions in silos and tanks

15.3 Analysis

Analysis is dealt with in Section 5. Where appropriate the coefficients given in Tables 15.2 and 15.3 can be used to determine design moments and shear for slabs and beams at ULS.

Sections 5.3 & 5.4

Table 15.2
Coefficients for use with one-way spanning slabs to Eurocode 2

Coefficient	Location						
	End support/slab connection				Internal supports and spans		
	Pinned end support		Continuous				
	Outer support	Near middle of end span	Outer support	Near middle of end span	At 1st interior support	At middle of interior spans	At interior supports
Moment	0.0	0.086	– 0.04	0.075	– 0.086	0.063	– 0.063
Shear	0.40	—	0.46	—	0.60:0.60	—	0.50:0.50

Conditions
Applicable to one-way spanning slabs where the area of each bay exceeds 30 m², $Q_k \leq 1.25G_k$ and $q_k \leq$ 5 kN/m², substantially uniform loading (at least 3 spans, minimum span ≥ 0.85 maximum (design) span.

Design moment = coeff x n x span² and design shear = coeff x n x span where n is a UDL with a single variable action = $\gamma_G g_k + \psi \gamma_Q q_k$ where g_k and q_k are characteristic permanent and variable actions in kN/m.

Basis: Yield Line design (assumed 20% redistribution[14], see Section 4.6.9.)

Table 15.3
Coefficients for use with beams (and one-way spanning slabs) to Eurocode 2

Coefficient	Location				
	Outer support	Near middle of end span	At 1st interior support	At middle of interior spans	At interior supports
Moment g_k and q_k	25% span[a]	—	0.094	—	0.075
Moment g_k	—	0.090	—	0.066	—
Moment q_k	—	0.100	—	0.086	—
Shear	0.45	—	0.63:0.55	—	0.50:0.50[b]

Conditions
For beams and slabs, 3 or more spans. (They may also be used for 2 span beams but support moment coefficient = 0.106 and internal shear coefficient = 0.63 both sides).

Generally $Q_k \leq G_k$, and the loading should be substantially uniformly distributed. Otherwise special curtailment of reinforcement is required.

Minimum span ≥ 0.85 x maximum (and design) span.

Design moment at supports = coeff x n x span² or
in spans = (coeff g_k x $\gamma_G g_k$ + coeff q_k x $\psi \gamma_Q q_k$) x span².

Design shear at centreline of supports = coeff x n x span where n is a UDL with a single variable action = $\gamma_G g_k + \psi \gamma_Q q_k$ where g_k and q_k are characteristic permanent and variable actions in kN/m.

γ_G and $\psi \gamma_Q$ are dependent on use of BS EN 1990 Exp. (6.10), Exp. (6.10a) or Exp. (6.10b). See Section 15.1.

Basis: All- and alternate-spans-loaded cases as UK National Annex and 15% redistribution at supports.

Key
a At outer support '25% span' relates to the UK Nationally Determined Parameter for BS EN 1992-1-1 9.2.1.2(1) for minimum percentage of span bending moment to be assumed at supports in beams in monolithic construction. 15% may be appropriate for slabs (see BS EN 1992-1-1 Cl 9.3.1.2).

b For beams of five spans, 0.55 applies to centre span.

Section 6.2.1

15.4 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 – $\phi/2$
  b = width of section
  K' may be determined from Table 15.4 and is dependent on the redistribution ratio used.

Section 4

- 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}/γ_s = 500/1.15 = 434.8 MPa
  z = $d[0.5 + 0.5(1 - 3.53K)^{0.5}] \leq 0.95d$
  Values of z/d (and x/d) may be taken from Table 15.5

  For flanged beams where $x < 1.25h_f$,
  $A_{s1} = M_{Ed}/f_{yd}z$

  where x = depth to neutral axis. Values of x/d may be taken from Table 15.5
  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* [28]

- If $K > K'$, section is *over-reinforced* and requires compression reinforcement.

  $A_{s2} = (M_{Ed} - M')/f_{sc}(d - d_2)$

  where
  A_{s2} = compression reinforcement
  M' = $K'bd^2 f_{ck}$
  f_{sc} = $700(x_u - d_2)/x_u \leq f_{yd}$

  where
  d_2 = effective depth to compression reinforcement
  x_u = $(\delta - 0.4)d$

  where
  δ = redistribution ratio

  Total area of steel $A_{s1} = M'/(f_{yd}z) + A_{s2} f_{sc}/f_{yd}$

Section 7.2

15.5 Design for beam shear

15.5.1 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 15.6

Table 15.4
Values for K'

Redistribution ratio, δ	z/d for K'	K'	$1 - \delta$
1.00	0.76[a]	0.208	0%
0.95	0.78[a]	0.195	5%
0.90	0.80[a]	0.182	10%
0.85	0.82	0.168	15%
0.80	0.84	0.153	20%
0.75	0.86	0.137	25%
0.70	0.88	0.120	30%

Condition
Class A reinforcement is restricted to a redistribution ratio, $\delta \leq 0.8$

Key
a See b in Table 15.5

Table 15.5
Values of z/d and x/d for singly reinforced rectangular sections

K	z/d	x/d	$(1 - \delta)_{max}$*
0.04	0.950[a]	0.125	30%
0.05	0.950[a]	0.125	30%
0.06	0.944	0.140	30%
0.07	0.934	0.165	30%
0.08	0.924	0.191	30%
0.09	0.913	0.217	30%
0.10	0.902	0.245	30%
0.11	0.891	0.272	30%
0.12	0.880	0.301	30%
0.13	0.868	0.331	27%
0.14	0.856	0.361	24%
0.15	0.843	0.393	21%
0.16	0.830	0.425	18%
0.17	0.816[b]	0.460[b]	14%
0.18	0.802[b]	0.495[b]	11%
0.19	0.787[b]	0.533[b]	7%
0.20	0.771[b]	0.572[b]	3%
0.208	0.758[b]	0.606[b]	0%

Conditions
$f_{ck} \leq 50$ MPa
* maximum allowable redistribution

Key
a Practical limit
b It is recommended that x/d is limited to 0.450 [21]. As a consequence z/d is limited to a minimum of 0.820

Table 15.6
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
Table derived from BS EN 1992-1-1 and UK National Annex.
Table created for f_{ck} = 30 MPa assuming vertical links.

For $\rho_1 \geq 0.4\%$ and f_{ck} = 25 MPa, apply factor of 0.94; f_{ck} = 40 MPa, apply factor of 1.10; f_{ck} = 50 MPa, apply factor of 1.19
f_{ck} = 35 MPa, apply factor of 1.05; f_{ck} = 45 MPa, apply factor of 1.14; Not applicable for f_{ck} > 50 MPa

Section 7.3.2

15.5.2 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 15.7, initially checking at $\cot\theta = 2.5$. Should it be required, a greater resistance may be assumed by using a larger strut angle, θ.

Table 15.7
Capacity of concrete struts expressed as a stress, $v_{Rd,max}$

f_{ck}		$v_{Rd,max}$ (MPa)						ν
	$\cot\theta$	2.50	2.14	1.73	1.43	1.19	1.00	
	θ	21.8°	25°	30°	35°	40°	45°	
20		**2.54**	2.82	3.19	3.46	3.62	3.68	**0.552**
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
Table derived from BS EN 1992-1-1 and UK National Annex assuming vertical links, i.e. $\cot\alpha = 0$
$\nu = 0.6[1 - (f_{ck}/250)]$
$v_{Rd,max} = \nu f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$

15.5.3 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

Generally $A_{sw}/s \geq v_{Ed,z} b_w / 1087$
Where f_{ywk} = 500 MPa, γ_s = 1.15 and cot θ = 2.5

Alternatively, A_{sw}/s per metre width of b_w may be determined from Figure 15.1(a) or 15.1(b) as indicated by the blue arrows in Figure 15.1(a). These figures may also be used to estimate the value of cot θ.

Section 10.4.1

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 15.8).

Table 15.8
Values of $A_{sw,min}/sb_w$ 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
$A_{sw,min}/sb_w$ for beams (x 10^3)	0.78	0.87	0.95	1.03	1.10	1.17	1.23

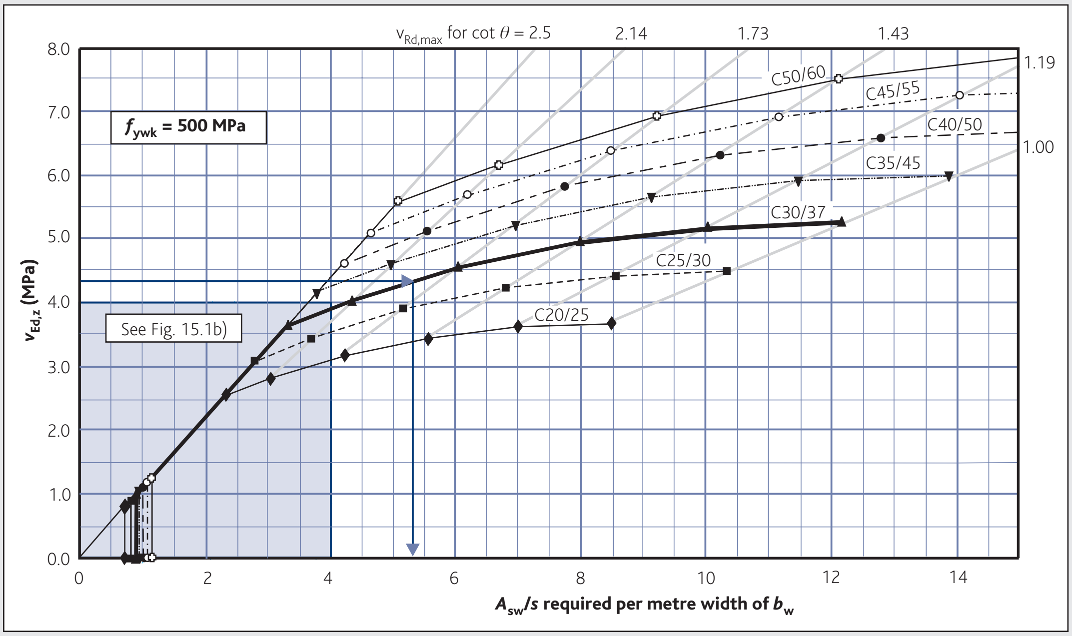


Figure 15.1a)
Diagram to determine A_{sw}/s required (for beams with high shear stress)

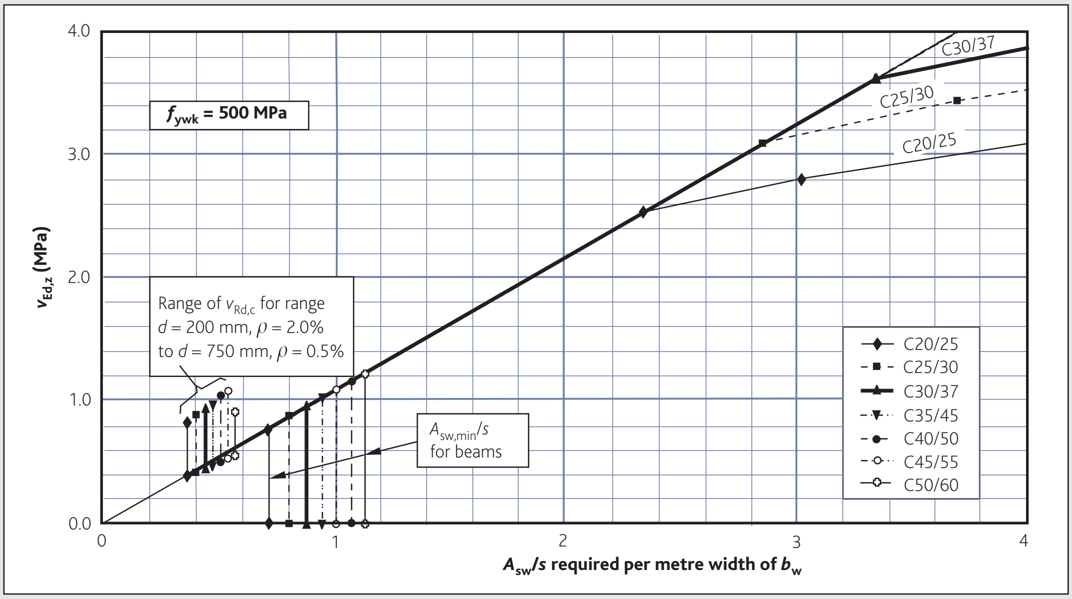


Figure 15.1b)
Diagram to determine A_{sw}/s required (for slabs and beams with low shear stress)

15.6 Design for punching shear

Determine if punching shear reinforcement is required, initially at u_1, then if necessary at subsequent perimeters, u_i.

Section 8.4

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} = applied shear force
u_i = length of the perimeter under consideration (see Sections 8.3, 8.7 and 12.4.3)
d = mean effective depth

$v_{Rd,c}$ = shear resistance without shear reinforcement (see Table 15.6)

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

Sections 10.4.2 & 12.4

A_{sw} = area of shear reinforcement in one perimeter around the column. For $A_{sw,min}$ see Section 10.4.2. For layout see Section 12.4.3
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 = $(250 + 0.25d) \le f_{ywd}$. For Grade 500 shear reinforcement see Table 15.9

Table 15.9
Values of $f_{ywd,ef}$ for grade 500 reinforcement

d	150	200	250	300	350	400	450
$f_{ywd,ef}$	287.5	300	312.5	325	337.5	350	362.5

15.7 Check deflection

Section 10.5

In general, the SLS state of deflection may be checked by using the span-to-effective-depth approach. More critical appraisal of deformation is outside the scope of this publication. To use the span-to-effective-depth approach, verify that:

Allowable $l/d = N \times K \times F1 \times F2 \times F3 \geq$ actual l/d

where

N = basic span-to-effective-depth ratio derived for K = 1.0 and ρ' = 0 from Section 10.5.2 or Table 15.10 or Figure 15.2

K = factor to account for structural system. See Table 15.11

F1 = factor to account for flanged sections. When b_{eff}/b_w = 1.0, factor F1 = 1.0. When b_{eff}/b_w is greater than 3.0, factor F1 = 0.80. For values of b_{eff}/b_w between 1.0 and 3.0, interpolation may be used (see Table 15.12)

where

b_{eff} is defined in Section 5.2.2

b_w = width of web

In I beams b_w = minimum width of web in tensile area.

In tapered webs b_w = width of web at centroid of reinforcement in web.

Table 15.10
Basic ratios of span-to-effective-depth, *N*, for members without axial compression

Required reinforcement, ρ	f_{ck} 20	25	30	35	40	45	50
0.30%	25.9	32.2	39.2	46.6	54.6	63.0	71.8
0.40%	19.1	22.4	26.2	30.4	35.0	39.8	45.0
0.50%	17.0	18.5	20.5	23.0	25.8	28.8	32.0
0.60%	16.0	17.3	18.5	19.8	21.3	23.1	25.2
0.70%	15.3	16.4	17.4	18.5	19.6	20.6	21.7
0.80%	14.8	15.7	16.6	17.6	18.5	19.4	20.4
0.90%	14.3	15.2	16.0	16.8	17.7	18.5	19.3
1.00%	14.0	14.8	15.5	16.3	17.0	17.8	18.5
1.20%	13.5	14.1	14.8	15.4	16.0	16.6	17.3
1.40%	13.1	13.7	14.2	14.8	15.3	15.8	16.4
1.60%	12.9	13.3	13.8	14.3	14.8	15.2	15.7
1.80%	12.7	13.1	13.5	13.9	14.3	14.8	15.2
2.00%	12.5	12.9	13.3	13.6	14.0	14.4	14.8
2.50%	12.2	12.5	12.8	13.1	13.4	13.7	14.0
3.00%	12.0	12.3	12.5	12.8	13.0	13.3	13.5
3.50%	11.9	12.1	12.3	12.5	12.7	12.9	13.1
4.00%	11.8	11.9	12.1	12.3	12.5	12.7	12.9
4.50%	11.7	11.8	12.0	12.2	12.3	12.5	12.7
5.00%	11.6	11.8	11.9	12.1	12.2	12.4	12.5
Reference reinforcement ratio, ρ_0	0.45%	0.50%	0.55%	0.59%	0.63%	0.67%	0.71%

Conditions

The values for span-to-effective-depth have been based on Table 10.3, using K = 1 (simply supported) and ρ' = 0 (no compression reinforcement required).

The span-to-effective-depth ratio should be based on the shorter span in two-way spanning slabs and the longer span in flat slabs.

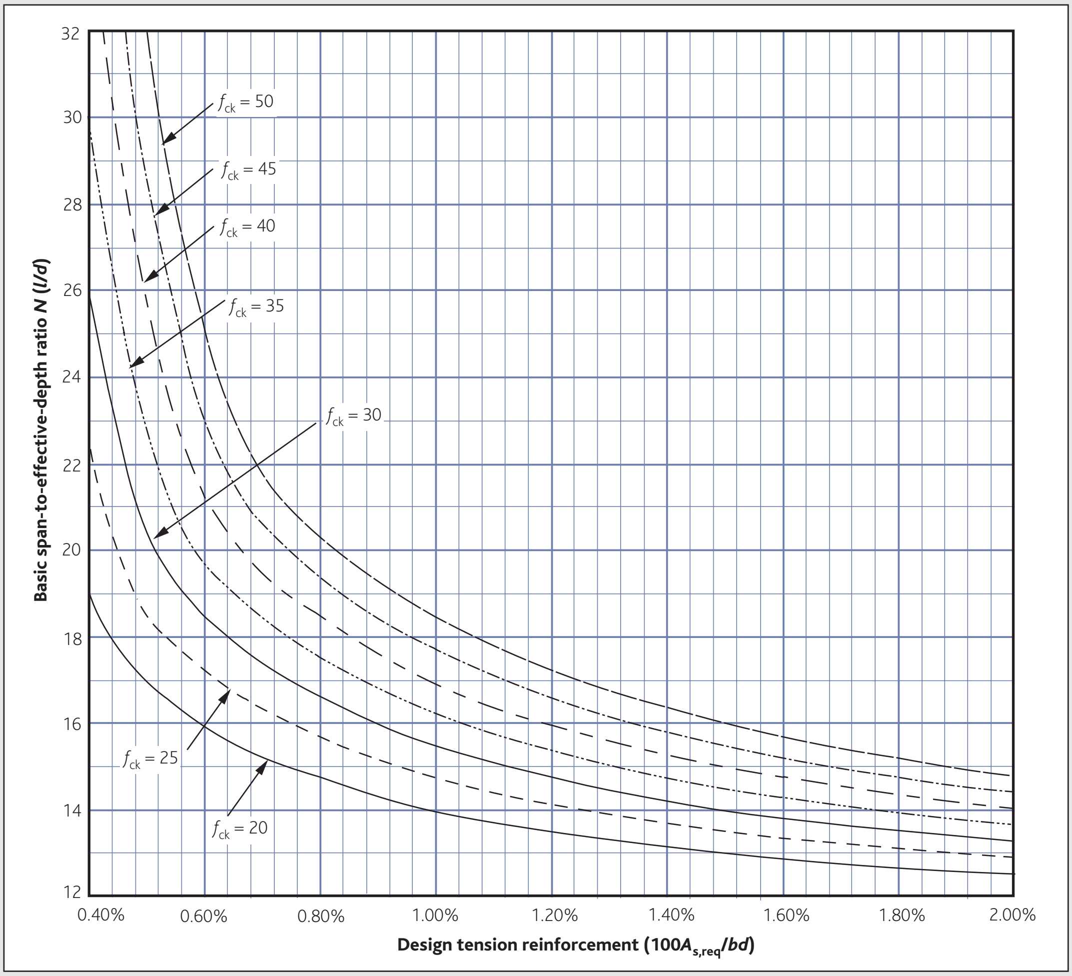


Figure 15.2
Basic span-to-effective-depth ratios, *N*, for *K* = 1, ρ' = 0

Table 15.11
***K* factors to be applied to basic ratios of span-to-effective-depth**

Structural system		*K*
Beams	**Slabs**	
Simply supported beams	One- or two-way spanning simply supported slabs	1.0
End span of continuous beams	End span of one-way spanning continuous slabs, or two-way spanning slabs continuous over one long edge	1.3
Interior spans of continuous beams	Interior spans of continuous slabs	1.5
—	Flat slabs (based on longer span)	1.2
Cantilevers	Cantilever	0.4

Table 15.12
Factor F1, modifier for flanged beams

b_{eff}/b_w	1.0	1.5	2.0	2.5	≥ 3.0
Factor	1.00	0.95	0.90	0.85	0.80

F2 = factor to account for brittle partitions in association with long spans. Generally F2 = 1.0 but if brittle partitions are liable to be damaged by excessive deflection, F2 should be determined as follows:
a) in flat slabs in which the longer span is greater than 8.5 m, F2 = $8.5/l_{eff}$
b) in beams and other slabs with spans in excess of 7.0 m, F2 = $7.0/l_{eff}$
Values of F2 may be taken from Table 15.13

F3 = factor to account for service stress in tensile reinforcement = $310/\sigma_s \leq 1.5$
Conservatively, if a service stress, σ_s, of 310 MPa is assumed for the designed area of reinforcement, $A_{s,req}$ then F3 = $A_{s,prov}/A_{s,req} \leq 1.5$.

More accurately, the serviceability stress, σ_s, may be calculated from SLS moments or may be estimated as follows:
$\sigma_s = f_{yk}/\gamma_s[(G_k + \psi_2 Q_k)/(1.25G_k + 1.5Q_k)]\ [A_{s,req}/A_{s,prov}]\ (1/\delta)$
or
$\sigma_s = \sigma_{su}\ [A_{s,req}/A_{s,prov}]\ (1/\delta)$

where

σ_{su} = the unmodified SLS steel stress, taking account of γ_M for reinforcement and of going from ultimate actions to serviceability actions
= $500/\gamma_s(G_k + \psi_2 Q_k)/(1.25G_k + 1.5Q_k)$
σ_{su} may be estimated from Figure 15.3 as indicated by the blue arrow

$A_{s,req}/A_{s,prov}$ = area of steel required divided by area of steel provided.

$(1/\delta)$ = factor to 'un-redistribute' ULS moments so they may be used in this SLS verification (see Table 15.14)

Actual l/d = actual span divided by effective depth, d.

Table 15.13
Factor F2, modifier for long spans supporting brittle partitions

Span, m	l_{eff}	≤ 7.0	7.5	8.0	8.5	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
Flat slabs	$8.5/l_{eff}$	1.00	1.00	1.00	1.00	0.94	0.85	0.77	0.71	0.65	0.61	0.57	0.53
Beams and other slabs	$7.0/l_{eff}$	1.00	0.93	0.88	0.82	0.78	0.70	0.64	0.58	0.54	0.50	0.47	0.44

Table 15.14
$(1/\delta)$ factor to be applied to unmodified σ_{su} to allow for redistribution used

Average redistribution used	20%	15%	10%	5%	0%	–5%	–10%	–15%	–20%	–25%	–30%
Redistribution ratio used, δ	1.20	1.15	1.10	1.05	1.00	0.95	0.90	0.85	0.80	0.75	0.70
$(1/\delta)$	83%	87%	91%	95%	100%	105%	111%	118%	125%	133%	143%

Notes
Where coefficients from Table 15.2 have been used in design and where $Q_k \approx 1.25G_k$, the coefficients in Table 15.2 may be considered to represent moment distribution of:
a) – 8% near middle of end span with pinned end support
b) – 22% at first interior support, as a worst case
c) + 3% near middle of internal spans, as a worst case
d) – 28% at interior supports, as a worst case.

Where coefficients from Table 15.3 have been used in design and where $Q_k \approx G_k$, the coefficients in Table 15.3 may be considered to represent moment redistribution of:
a) + 3% near middle of end span with pinned end support, as a worst case
b) + 9% near middle of internal spans, as a worst case
c) – 15% at all interior supports.

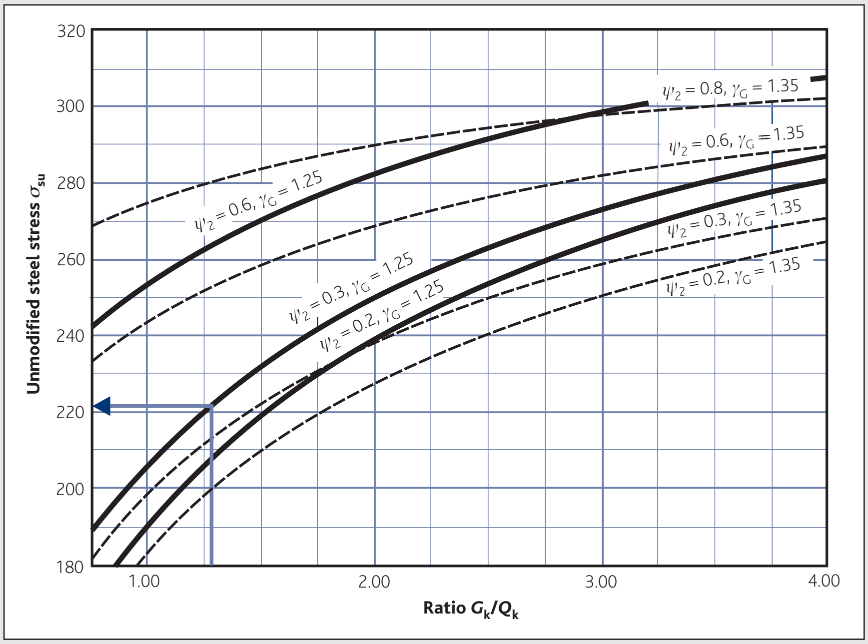


Figure 15.3
Determination of unmodified SLS, σ_{su} stress in reinforcement

15.8 Control of cracking

Section 10.2

Sections 10.3, 10.4

Cracking may be controlled by restricting either maximum bar diameter or maximum bar spacing to the relevant diameters and spacings given in Table 15.15. The appropriate SLS stress in reinforcement, σ_s, may be determined as outlined for F3 in Section 15.7.

Minimum areas and aspects of detailing should be checked.

Table 15.15
Maximum bar diameters ϕ or maximum bar spacing for crack control

Steel stress (MPa) σ_s	Maximum bar size (mm)			Maximum bar spacing (mm)	
	w_k = 0.3 mm	w_k = 0.4 mm		w_k = 0.3 mm	w_k = 0.4 mm
160	32	40	OR	300	300
200	25	32		250	300
240	16	20		200	250
280	12	16		150	200
320	10	12		100	150
360	8	10		50	100

Notes

The 'normal' limit of 0.3 mm may be relaxed to 0.4 mm for XO and XC1 exposure classes if there is no specific requirement for appearance

Table assumptions include c_{nom} = 25 mm and $f_{ct,eff}$ (= f_{ctm}) = 2.9 MPa

15.9 Design for axial load and bending

15.9.1 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).

Section 5.6

15.9.2 Design by calculation

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 applied axial force
α_{cc} = 0.85
η = 1 for ≤ C50/60
b = breadth of section
d_c = effective depth of concrete in compression = $\lambda x \le h$ (see Figure 6.4)

where

λ = 0.8 for ≤ C50/60
x = depth to neutral axis
h = height of section
σ_{sc}, (σ_{st}) = stress in compression (and tension) reinforcement

- For moment

$$A_{sM}/2 = [M_{Ed} - \alpha_{cc}\eta f_{ck} b d_c(h/2 - d_c/2)/\gamma_c]/[(h/2 - d_2)(\sigma_{sc} + \sigma_{st})$$

where

A_{sM} = total area of reinforcement required to resist moment using this method
$A_{sM} = A_{s1} + A_{s2}$ and $A_{s1} = A_{s2}$

Where reinforcement is not concentrated in the corners, a conservative approach is to calculate an effective value of d_2 as illustrated in Figure 15.4.

- Solution: iterate x such that $A_{sN} = A_{sM}$

15.9.3 Column charts

Alternatively A_s may be estimated from column charts.

Figures 15.5a) to 15.5e) 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 Figure 15.4.
d_2 = effective depth to steel in layer 2

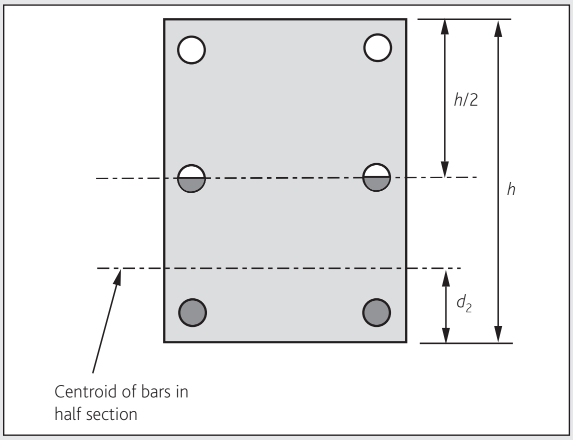


Figure 15.4
Method of assessing d_2 including side bars

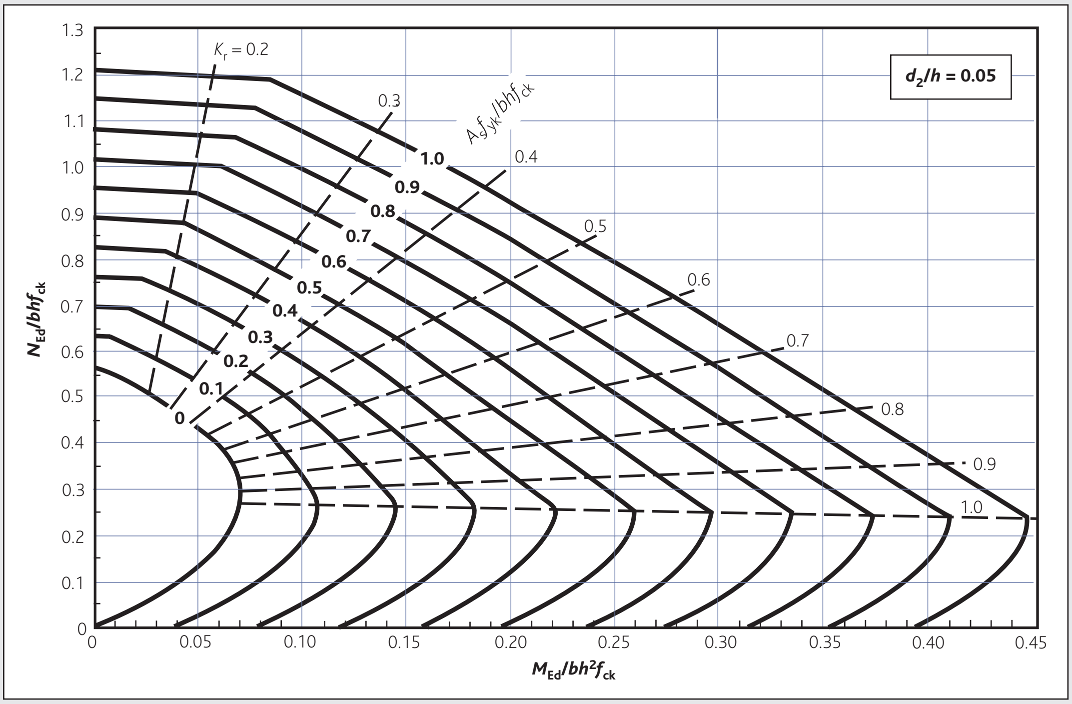


Figure 15.5a)
Rectangular columns d_2/h = 0.05

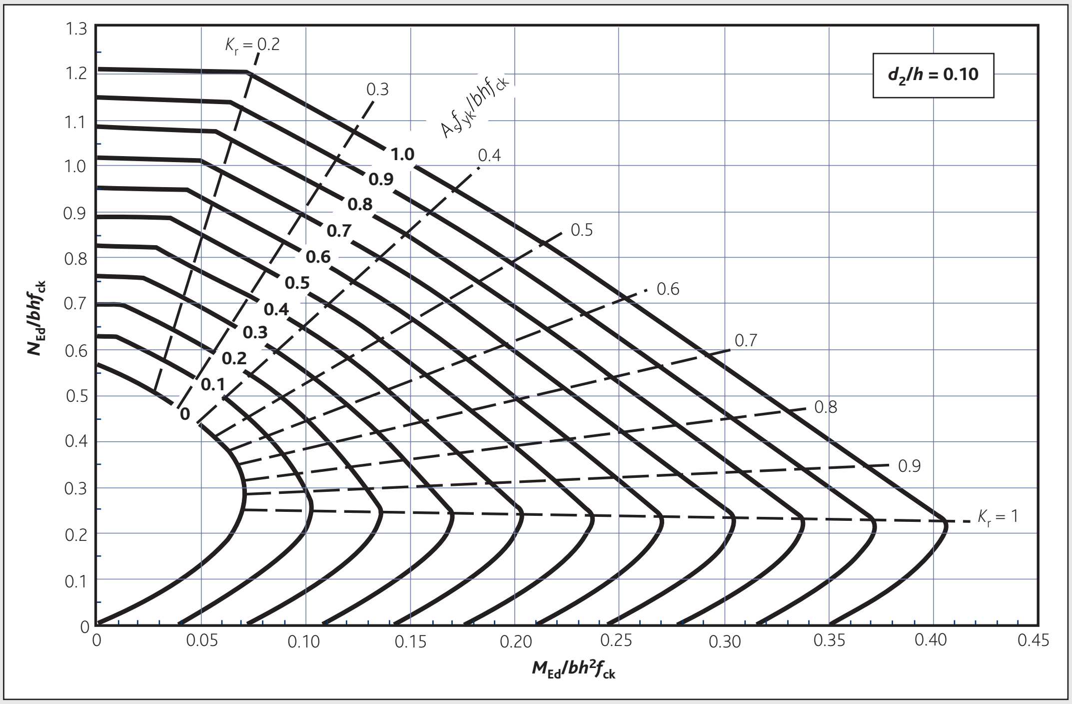


Figure 15.5b)
Rectangular columns d_2/h = 0.10

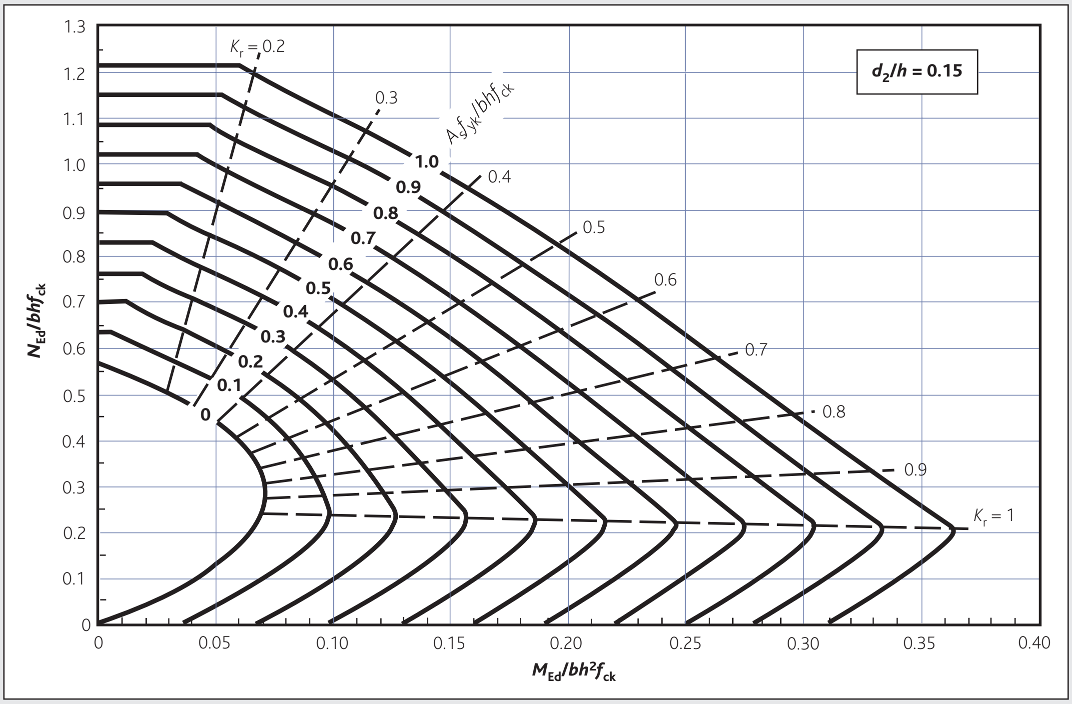


Figure 15.5c)
Rectangular columns d_2/h = 0.15

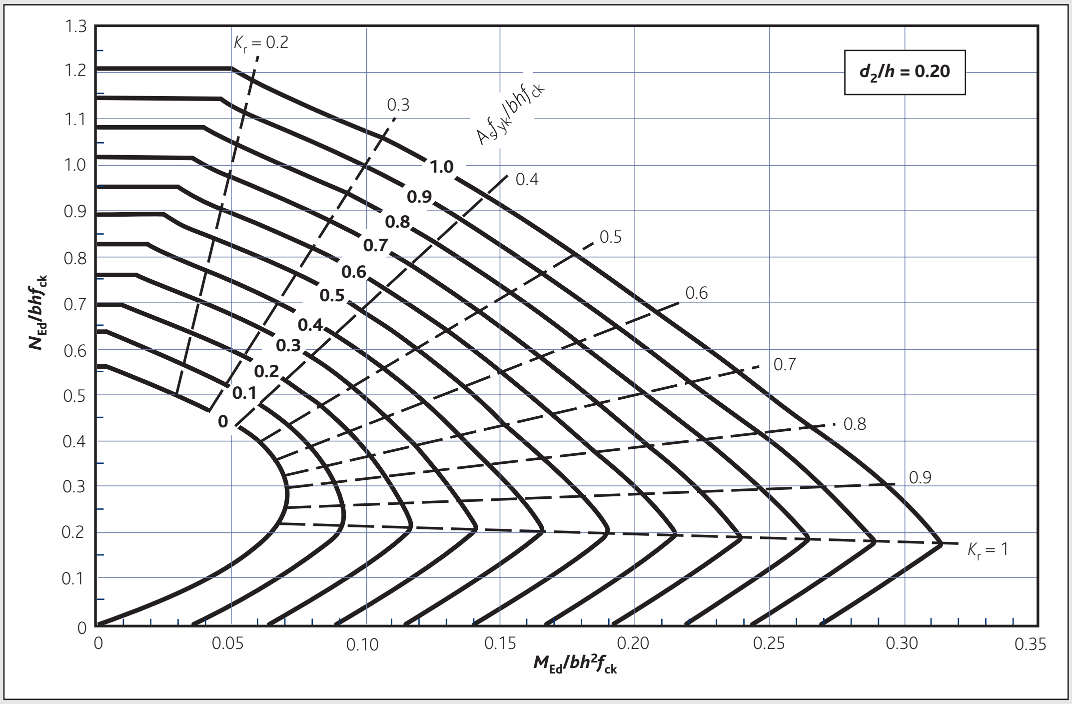


Figure 15.5d)
Rectangular columns d_2/h = 0.20

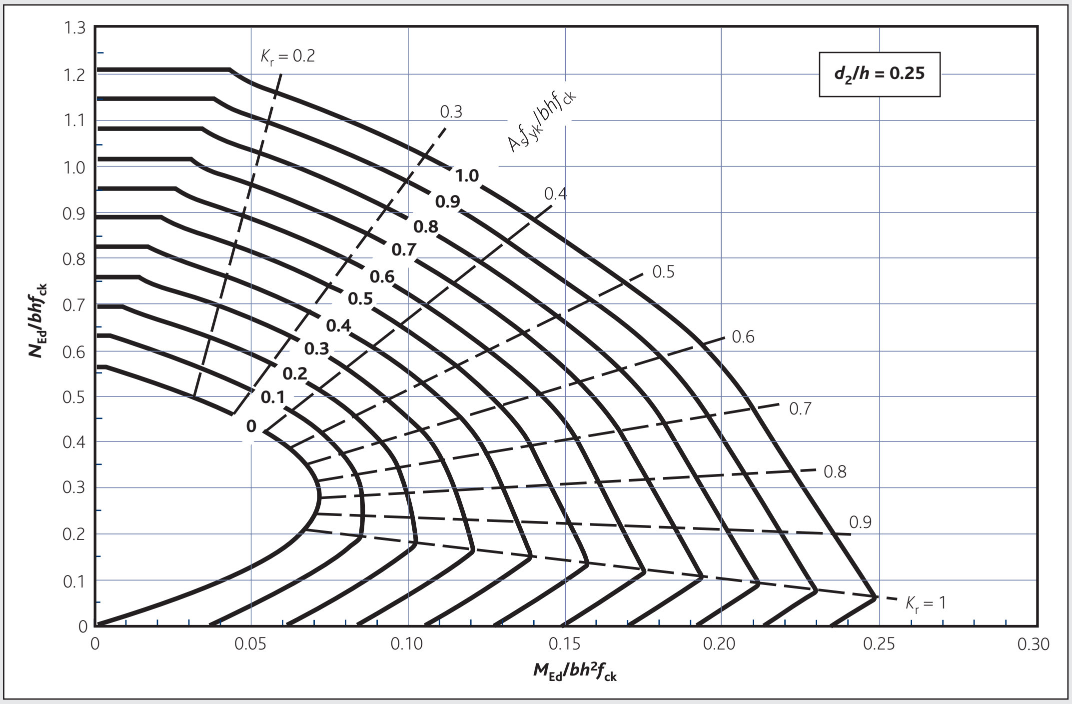


Figure 15.5e)
Rectangular columns d_2/h = 0.25

15.9.4 Biaxial bending

Section 5.6.3

As a first step, separate design in each principal direction, disregarding biaxial bending, may be undertaken. No further check is necessary if $0.5 \le \lambda_y/\lambda_z \le 2.0$ and, for rectangular sections, $0.2 \ge (e_y/h_{eq})/(e_z/b_{eq})$ or $(e_y/h_{eq})/(e_z/b_{eq}) \ge 5.0$. Otherwise see Section 5.6.3.

For square columns $(e_y/h_{eq})/(e_z/b_{eq}) = M_{Edy}/M_{Edz}$.

15.9.5 Links

Section 12.5.2

Links in columns should be at least 8 mm or maximum diameter of longitudinal bars/4 in diameter and adjacent to beams and slabs spaced at the least of:

- 12 times the minimum diameter of the longitudinal bar,
- 60% of the lesser dimension of the column, or
- 240 mm.

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Appendix: simple foundations

A1 General

This appendix is intended to give guidance on the application of Eurocode 7 Part 1[11] to the design of simple concrete foundations. It will be subject to the provisions of the as yet unpublished UK National Annex to Eurocode 7. It is recommended that further guidance is sought from other publications. These include *Eurocode 7, a Commentary*[29] and *Designers' guide to EN 1997-1*[30].

BS EN 1997: 1.1.2

Eurocode 7 is wide-ranging and provides in outline all the requirements for the design of geotechnical structures, including:

- Approaches to geotechnical design.
- Ground investigation.
- Design aspects of construction.
- Design of specific elements.

BS EN 1997: 2.1

It classifies structures and risks into three categories.

- Geotechnical Category 1 is for small, relatively simple structures with negligible risk.
- Geotechnical Category 2 is for conventional structures with no exceptional risk, e.g. spread, raft and pile foundations, retaining structures, bridge piers and abutments, embankments, and earthworks and tunnels.
- Geotechnical Category 3 is for very large or unusual structures or exceptionally difficult ground conditions and is outside the scope of Eurocode 7.

Eurocode 7 and this appendix concentrate on Geotechnical Category 2.

A2 Actions

BS EN 1997: 2.4.6.1

In Eurocode 7, design values of actions, F_d, are based on representative actions, F_{rep}.

$$F_d = \gamma_F F_{rep}$$

BS EN 1997: Exp. (2.1b)

where

γ_F = partial factor for action. See Table A1

F_{rep} = ψF_k

where

ψ = factor to convert characteristic actions to representative actions as per BS EN 1990[5] (see Tables 2.1 and 2.2)

F_k = characteristic value of an action

It is anticipated that the structural designer will specify representative actions to the geotechnical designer.

(The traditional practice of specifying and designing foundations using characteristic actions may be used by agreement. See A4.2, Prescriptive measures, below.)

A3 Methods of geotechnical design

Eurocode 7 states that no limit state e.g. stability (EQU, UPL or HYD), strength (STR or GEO) or serviceability, as defined by BS EN 1990[5], shall be exceeded. The requirements for ultimate and serviceability limit state (ULS and SLS) design may be accomplished by using, in an appropriate manner, the following alone or in combination.

BS EN 1997: 2.1(4)
BS EN 1990: 3.5

- Calculations.
- Prescriptive measures.
- Testing.
- An observational method.

A3.1 Calculations

BS EN 1997: 2.4

A3.1.1 Ultimate limit state (ULS)

BS EN 1997: 2.4.7.3.1

It is necessary to verify that $E_d \leq R_d$

where

E_d = design value of the effect of actions
R_d = design value of the resistance to an action

There are three design approaches in Eurocode 7; the UK is due to adopt design approach 1 (DA1). DA1 requires the consideration of two combinations of partial factors for actions and for soil parameters to compare ultimate loads with ultimate soil resistance. These combinations are illustrated in Figure A1. The appropriate partial factors are given in Table A1.

BS EN 1997: 2.4.7.3.4
BS EN 1990: A1.3.1(5) & NA

Table A1
Partial factors for design approach 1 (for STR/GEO excluding piles and anchorages)

BS EN 1997: Table A.3
BS EN 1997: Table A.4

Combination	Partial factor on actions, γ_F		Partial factors for soil parameters, γ_M		
	γ_G	γ_G	$\gamma_{\varphi'}{}^{c}$	$\gamma_{c'}$	γ_{cu}
Combination 1[a]	1.35 (1.0*)	1.5 (0.0*)	1.0	1.0	1.0
Combination 2[b]	1.0 (1.0*)	1.3 (0.0*)	1.25	1.25	1.4

Key
* = value if favourable
G = permanent action
Q = variable action
φ' = angle of shearing resistance (in terms of effective stress)
c' = cohesion intercept (in terms of effective stress)
c_u = undrained shear strength
a = Combination 1 partial factors on actions equate to Set B loads (BS EN 1990[5] Table NA.A1.2(B)) NB: the use of Expressions (6.10a) and (6.10b) was not anticipated within Eurocode 7.
b = Combination 2 partial factors on actions equate to Set C loads (BS EN 1990 Table NA.A1.2(C))
c = the factor $\gamma_{\varphi'}$ is applied to tan φ'

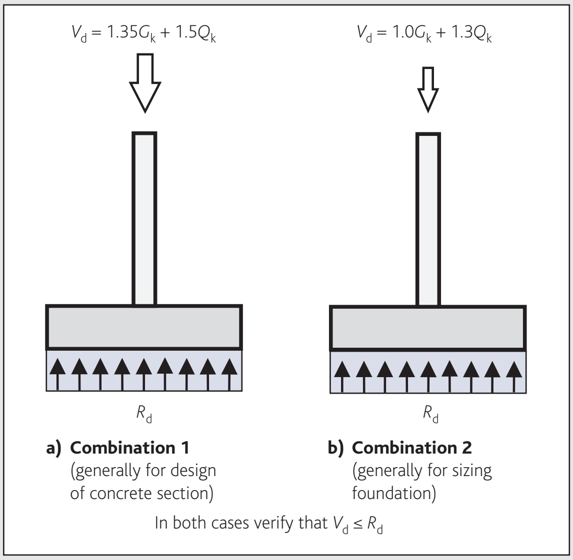

Figure A1
Combinations 1 and 2 for a pad foundation for the ULS of STR and GEO

In Figure A1:

V_d = design vertical load
= vertical component of E_d
The design vertical load, V_d should include the weight of the foundation and backfill.
R_d = design value of resistance

For a pad foundation, other limit states, notably overall stability, structural stability, sliding, heave, settlement and vibration will also need to be verified.

In principle, both combinations of partial factors should be used to check the design of the concrete section. In practice, it will often be found that Combination 1 will govern the design of the concrete section and Combination 2 will determine the size of the foundation.

A3.1.2 Serviceability limit state (SLS)

BS EN 1997: 2.4.8

Settlement should be checked either by:

- Direct calculation of the ground deformation.
- Verifying that a sufficiently low fraction of the ground strength is mobilised to keep deformations within the required serviceability limits. This is provided that a value of the deformation is not required in order to check the SLS and comparable experience exists with similar ground, structures and application methods.

BS EN 1997: 6.6.2(16)
BS EN 1997: 6.6.1(3)

If, for conventional spread foundations on firm to stiff clays, the ratio of the foundation's bearing capacity to the applied serviceability loads is ≥ 3, then calculations of settlement are unnecessary. For soft clays, calculations should always be carried out.

A3.2 Prescriptive measures

BS EN 1997: 2.5

Prescriptive measures involve using conventional and generally conservative methods (i.e. comparable experience) to design and execute foundations.

A3.3 Testing

The results from testing (e.g. from load tests on piles or ground anchors) or modelling (provided the effects of ground variations, time and scale are considered) may be used in place of, or in combination with, calculations.

BS EN 1997: 2.6

A3.4 Observational method

The observational method relates mainly to temporary works where performance is observed and responses can be made to the results of monitoring.

BS EN 1997: 2.7

A4 Geotechnical design of spread foundations

The size of spread foundations may be determined by one of the following methods:

BS EN 1997: 6.4(5)

- Calculations.
- By using a prescribed method such as using local practice.
- Using calculated allowable bearing pressures.

A4.1 Calculations

A4.1.1 Ultimate limit state (ULS)

Spread foundations shall be checked for ultimate limit states of overall stability, bearing resistance and sliding. The bearing resistance of the ground shall be determined for both short-term (i.e. undrained) and long-term (i.e. drained) conditions where applicable.

BS EN 1997: 6.5.1
BS EN 1997: 2.2(1)

A4.1.1.1 Undrained conditions

The design bearing resistance of cohesive soils in undrained conditions depends on:

- Soil's undrained strength (c_u).
- Vertical total stress at foundation level (q).
- Shape (B/L) and inclination (α) of the foundation.
- Any inclination and/or eccentricity (e) of the load.

BS EN 1997: 6.5.2.2.(1)
BS EN 1997: D3

The vertical bearing resistance per unit area

R/A' = function of $[c_u, q, B/L, \alpha, e, H]$

where

R = vertical bearing resistance
A' = effective area of the base
H = horizontal load

The design undrained bearing resistance must be calculated for both combinations of partial factors given in Table A1.

A4.1.1.2 Drained conditions

The design bearing resistance of both granular and cohesive soils in drained conditions depends on:

- Soil's weight density (i.e. unit weight, γ).
- Effective cohesion (c').
- Angle of shearing resistance (φ').
- Weight density of water (γ_w).
- Vertical effective stress at foundation level (q').

BS EN 1997: 6.5.2.2.(1)
BS EN 1997: D4

- Shape (B/L).
- Inclination (α) of the foundation.
- Any inclination and/or eccentricity (e) of the load.

(It should be noted that the bearing capacity factors that are used to determine drained bearing resistance are particularly sensitive to the angle of shearing resistance of the soil.)

The vertical bearing resistance per unit area
$R/A' \;=\;$ function of [γ, c', φ', γ_w, q', B/L, α, e, H] where the symbols are as defined above for drained bearing resistance.

The design undrained bearing resistance must be calculated for both combinations of partial factors given in Table A1.

BS EN 1997: D4
BS EN 1990: A1.3(5)

A4.1.1.3 Bearing resistances

It is anticipated that values of ULS design vertical bearing resistance, R/A', will be made available to structural designers in the 'Geotechnical design report' for a project, which itself will be based on the 'Ground investigation report'. Usually, it will be found that, with respect to sizing the foundation, Combination 2 (see Table A1) will be critical.

For eccentric loading at ULS, it is assumed that the resistance of the ground is uniformly distributed and is centred on the centre of gravity of the applied load. For a base $L \times B$ with load eccentricities e_x and e_y,

$A' \;=\; (B - 2e_x)(L - 2e_y)$ (see Figure A2)

Sliding resistance should be checked in a similar manner.

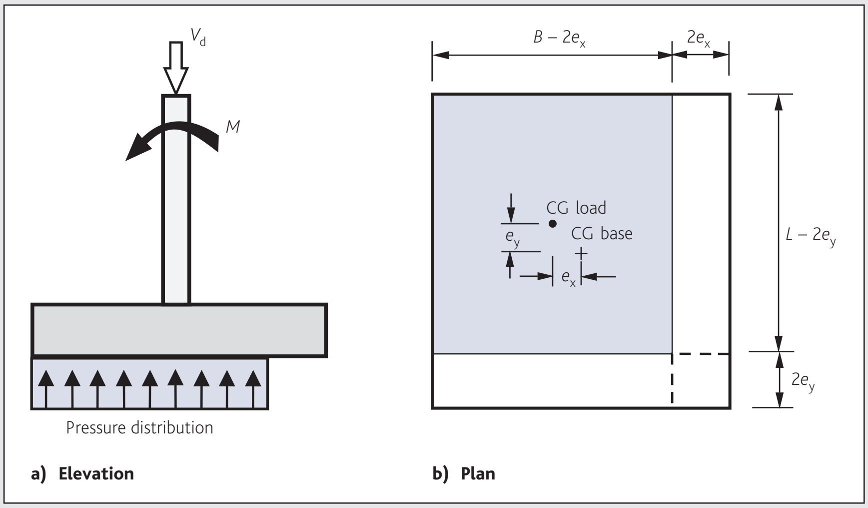

Figure A2
Effective area for eccentric loading at ULS

BS EN 1997: 6.6

A4.1.2 Serviceability limit state (SLS)

A4.1.2.1 Settlement

For the serviceability limit state, settlement should be checked by calculation or may, in the case of spread foundations on clays, be assumed to be satisfactory if the ratio of the foundation's ULS bearing capacity to the applied serviceability loads (with $\gamma_G = 1.0$, $\gamma_Q = 1.0$ and ψ'_2 as appropriate) is ≥ 3. This approach is not valid for very soft soils so settlement calculations shall always be carried out for soft clays.

A4.1.2.2 Serviceability

Serviceability checks should be made against the Serviceability Limit State design loads. For most building structures, this would be the quasi-permanent load case (see Table 2.1d).

A4.1.3 Communications

As described in A2 above, it is anticipated that the 'currency of exchange' will be representative actions. However, in communications, it will be vital to define:

- Loads as being Combination 1, Combination 2, characteristic or more usefully, representative values of the permanent and variable actions; and
- Applicability of values of *R/A*' (whether to Combination 1 or Combination 2 at ULS, SLS, and with respect to shape of foundation, inclination of loads, etc).

A4.2 Prescriptive measures

The practice of checking characteristic actions (γ_G = 1.0, γ_Q = 1.0) against allowable bearing pressures may be adopted within the requirements of Eurocode 7, in one of two ways, viz:

BS EN 1997: 2.5

- Local practice.
- Calculated allowable bearing pressures.

A4.2.1 Local practice

The allowable bearing pressures may be regarded as 'Prescriptive measures', which "may be used where comparable experience … makes [geotechnical] design calculations unnecessary". Such allowable bearing pressures could be specified by local Building Control authorities or on the basis of well established local practice such as BS 8004 Foundations[31].

BS EN 1997: 2.5(2)

A4.2.2 Calculated allowable bearing pressures

Alternatively, the writer of the site/ground investigation report could be required to calculate and/or specify allowable bearing pressures which provide designs consistent with both the ULS and SLS requirements of Eurocode 7. However, this may require knowledge of the size of the loads and, in particular, any eccentricity or inclination of loads.

A5 Piled foundations

The allowable methods for the geotechnical design of piles include:

BS EN 1997: 7.4.1

- Static load tests (shown to be consistent with calculations or other relevant experience).
- Calculations (validated by static load tests).

Structural design of piles and pile caps should be carried out to Eurocode 2.

A6 Retaining walls and other forms of foundation

The sizing and design of retaining walls and other forms of foundation are outside the scope of this publication. For these and/or where the foundations are unconventional or the risks abnormal, specialist literature should be consulted and guidance sought.