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

Worked Examples to Eurocode 2: Volume 1

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

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Foreword

The introduction of European standards to UK construction is a significant event as, for the first time, all design and construction codes within the EU will be harmonised. The ten design standards, known as the Eurocodes, will affect all design and construction activities as all current British Standards for structural design are due to be withdrawn in 2010.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 coordinated 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, Worked Examples to Eurocode 2: Volume 1 is to distil from Eurocode 2, other Eurocodes and other sources the material that is commonly used in the design of concrete framed buildings.

These worked examples are published in two parts. Volume 2 will include chapters on Foundations, Serviceability, Fire and Retaining walls.

Acknowledgements

The original ideas for this publication emanates from the research project `Eurocode 2: Transition from UK to European concrete design standards', which was led by the BCA and part funded by the DTI under their PII scheme and was overseen by a Steering Group and the CIEG. The work has been brought to fruition by The Concrete Centre from early initial drafts by various authors listed on the inside back cover. The concrete industry acknowledges and appreciates the support given by many individuals, companies and organisations in the preparation of this document. These are listed on the inside back cover.

We gratefully acknowledge the authors of the initial drafts and the help and advice given by Robin Whittle in checking the text. Thanks are also due to Gillian Bond, Kevin Smith, Sally Huish and the design team at Michael Burbridge Ltd for their work on the production.

Published by The Concrete Centre, part of the Mineral Products Association
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Cement and Concrete Industry Publications (CCIP) are produced through an industry initiative to publish technical guidance in support of concrete design and construction. CCIP publications are available from the Concrete Bookshop at **www.concretebookshop.com** **Tel:** +44 (0)7004-607777

CCIP-041
Published December 2009
ISBN 978-1-9046818-84-7
Price Group P

Printed by Michael Burbridge Ltd, Maidenhead, UK.

Worked Examples to Eurocode 2: Volume 1

Contents

Symbols and abbreviations used in this publication

Symbol	Definition
A	Cross-sectional area; Accidental action
A	Site altitude, m (snow)
A	Altitude of the site in metres above mean sea level (wind)
A, B, C	Variables used in the determination of λ_{lim}
A_c	Cross-sectional area of concrete
A_d	Design value of an accidental action
A_{Ed}	Design value of a seismic action
A_{ref}	Reference area of the structure or structural element (wind)
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_{sw}	Cross-sectional area of shear reinforcement; Area of punching shear reinforcement in one perimeter around the column
$A_{sw,min}$	Minimum cross-sectional area of shear reinforcement; Minimum area of punching shear reinforcement in one perimeter around the column
A_t	Area of tensile reinforcement in flat slab column strips
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_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_1, a_2,	Distance from edge of support to centre of support
a_1, b_1	Dimensions of the control perimeter around an elongated support (punching shear)
a.m.s.l.	Altitude above mean sea level
b	Overall width of a cross-section, or flange width in a T- or L-beam
b	Breadth of building (wind)
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_1	Half of distance between adjacent webs of downstand beams
C_e	Exposure coefficient (snow)
C_t	Thermal coefficient (snow)
C_w	Shear centre

Symbol	Definition
c_1, c_2	Dimensions of a rectangular column. For edge columns, c_1 is measured perpendicular to the free edge (punching shear)
c_{alt}	Altitude factor (wind)
c_d	Dynamic factor (wind)
c_{dir}	Directional factor (wind)
$c_{e,flat}$	Exposure factor (wind)
c_f	Force coefficient (wind)
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_{pe}	(External) pressure coefficient (wind)
$c_{pe,10}$	(External) pressure coefficient for areas > 1 m^2 (wind)
c_{pi}	Internal pressure coefficient (wind)
c_{prob}	Probability factor (wind)
c_{season}	Season factor (wind)
c_s	Size factor (wind)
c_y, c_x	Column dimensions in plan
Δc_{dev}	Allowance made in design for deviation
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
E	Effect of action; Integrity (in fire); Elastic modulus
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
EI	Bending stiffness
E_s	Design value of modulus of elasticity of reinforcing steel
Exp.	Expression
EQU	Static equilibrium
e	Eccentricity
e_0	Minimum eccentricity in columns
e_2	Deflection (used in assessing M_2 in slender columns)
e_i	Eccentricity due to imperfections
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
FEM	Fixed end moment
F_c (F_s)	Force in concrete (steel)
F_d	Design value of an action
F_E	Tensile force in reinforcement to be anchored
F_k	Characteristic value of an action
F_{rep}	Representative action (= yF_k where y = factor to convert characteristic to representative action)
F_s	Tensile force in reinforcement
F_{td}	Design value of the tensile force in longitudinal reinforcement
ΔF_{td}	Additional tensile force in longitudinal reinforcement due to the truss shear model

Symbol	Definition
$F_{V,Ed}$	Total vertical load (on braced and bracing members)
F_w	Resultant characteristic force due to wind. (See section 2.6)
f_{bd}	Ultimate bond stress
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
$f_{ct,d}$	Design tensile strength of concrete ($\alpha_{ct} f_{ct,k} / \gamma_C$)
$f_{ct,k}$	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_{sc}	Compressive stress in compression reinforcement at ULS
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
$G_{k,sup}$	Upper characteristic value of a permanent action
$G_{k,inf}$	Lower 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	Height of building (wind)
h	Overall depth of a cross-section; Height
h_{ave}	Obstruction height (wind)
h_{dis}	Displacement height (wind)
h_f	Depth of footing; Thickness of flange
h_s	Depth of slab
I	Second moment of area of concrete section; Inertia
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)
K'	Value of K above which compression reinforcement is required
K_v	A correction factor for axial load
K_ϕ	A correction factor for creep
k	Coefficient or factor
k	Relative flexibility or relative stiffness
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

Symbol	Definition
$l_{b,rqd}$	Basic anchorage length
l_{eff}	Effective span
l_n	Clear span
l_y, l_z	Spans of a two-way slab in the *y* and *z* 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_t	Design transfer moment to column from a flat slab
m	Number of vertical members contributing to an effect
m	Mass
N	Axial force
N	Basic span-to-effective-depth ratio, l/d, for $K = 1.0$
$N_{0Ed,fi}$	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	Relative axial force $N_{Ed}/(A_c f_{cd})$
n_{bal}	The value of n at maximum moment resistance
n_0, n_s	Number of storeys
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
q_b	Basic wind pressure
q_p	Peak wind pressure
$q_p(z_e)$	Peak velocity pressure at reference height z_e, (wind)
R	Resistance; Mechanical resistance (in fire)
R_A	Reaction at support A
R_B	Reaction at support B
R_d	Design value of the resistance to an action
r	Radius
r_m	Ratio of first order end moments in columns at ULS
SLS	Serviceability limit state(s) – corresponding to conditions beyond which specified service requirements are no longer met
s	Spacing
s	Snow load on a roof

Symbol	Definition
s_k	Characteristic ground snow load
s_r	Radial spacing of perimeters of shear reinforcement
s_t	Tangential spacing shear reinforcement along perimeters of shear reinforcement
t	Thickness; Time being considered; Breadth of support
t_0	The age of concrete at the time of loading
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_{out}	Perimeter at which shear reinforcement is no longer required
V	Shear force
V_{Ed}	Design value of the applied shear force
$V_{Rd,c}$	Shear resistance of a member without shear reinforcement
$V_{Rd,max}$	Shear resistance of a member limited by the crushing of compression struts
$V_{Rd,cmin}$	Minimum shear resistance of member considering concrete alone
$V_{Rd,s}$	Shear resistance of a member governed by the yielding of shear reinforcement
v_b	Basic wind velocity
$v_{b,0}$	The fundamental basic wind velocity being the characteristic 10 minute wind velocity at 10 m above ground level in open country
$v_{b,map}$	Fundamental basic wind velocity from Figure NA.1 m/s
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
$v_{Rd,max}$	Capacity of concrete struts expressed as a stress
W_1	Factor corresponding to a distribution of shear (punching shear)
W_e	Peak external wind load
W_k	Characteristic value of wind action (NB not in the Eurocodes and should be regarded as a form of Q_k, characteristic value of a variable action)
w_k	Characteristic unit wind load.
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 between buildings (wind)
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	Zone number obtained from map (snow)
z	Lever arm of internal forces

Symbol	Definition
z	Reference height (wind)
z_e	Reference height for windward walls of rectangular buildings (wind)
α	Angle; Angle of shear links to the longitudinal axis; Ratio
α_A	A coefficient for use with a representative variable action taking into account area supported
α_1, α_2, α_3 α_4, α_5, α_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
α_e	Modular ratio = E_s/E_{cd}
α_n	A coefficient for use with a representative variable action taking into account number of storeys supported
β	Angle; Ratio; Coefficient
β	Factor dealing with eccentricity (punching shear)
γ	Partial factor
γ_C	Partial factor for concrete
γ_F	Partial factor for actions, F
γ_G	Partial factor for permanent actions, G
$\gamma_{Gk,sup}$	Partial factor to be applied to $G_{k,inf}$
$\gamma_{Gk,inf}$	Partial factor to be applied to $G_{k,sup}$
γ_Q	Partial factor for variable actions, Q
γ_M	Partial factor for material (usually γ_C or γ_S)
γ_S	Partial factor for reinforcing steel
δ	Redistribution ratio equals ratio of the redistributed moment to the elastic bending moment (1 – % redistribution)
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
ε_{sc}	Compressive strain in reinforcement
ε_{st}	Tensile strain in reinforcement
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_i, μ_1, μ_2	Snow load shape factors
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)
ρ	Required tension reinforcement ratio
ρ	Density of air (wind)
ρ'	Reinforcement ratio for required compression reinforcement, A_{s2}/bd

Symbol	Definition
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_{ly}, ρ_{lz}	Reinforcement ratio of bonded steel in the *y* and *z* direction
ρ_0	Reference reinforcement ratio $f_{ck}{}^{0.5'}\ 10^{-3}$
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
$\sigma_{sc}(\sigma_{st})$	Stress in compression (and tension) reinforcement
σ_{sd}	Design stress in the bar at the ultimate limit state
σ_{su}	Unmodified service stress in reinforcement determined from ULS loads (See Figure C3)
$\varphi(\infty,t_0)$	Final value of creep coefficient
φ_{ef}	Effective creep factor
ϕ	Bar 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 = $A_s f_{yd}/A_c f_{cd} \leq 1$

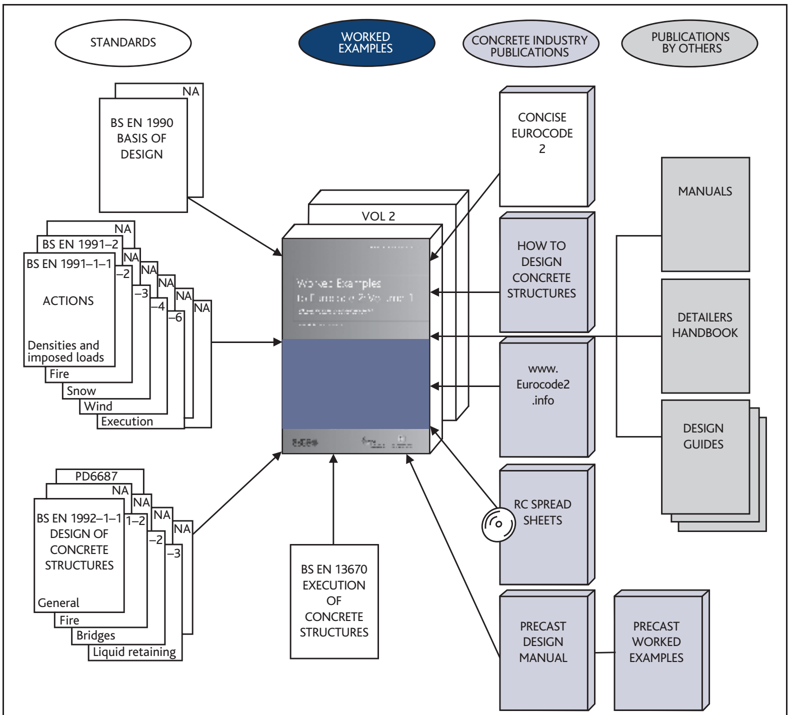
1 Introduction

1.1 Aim

The aim of this publication is to illustrate through worked examples how BS EN 1992–1–1[1] (Eurocode 2) may be used in practice to design in-situ concrete building structures. It is intended that these worked examples will explain how calculations to Eurocode 2 may be performed. Eurocode 2 strictly consists of four parts (Parts 1–1, 1–2, 2 and 3)[1-4] but for the purposes of this publication, Eurocode 2 refers to part 1–1 only, unless qualified. The worked examples will be carried out within the environment of other relevant publications listed below, and illustrated in Figure 1.1:

- The other three parts of Eurocode 2.
- Other Eurocodes.
- Material and execution standards.
- Publications by the concrete industry and others.

There are, therefore, many references to other documents and while it is intended that this publication, referred to as *Worked examples*, can stand alone, it is anticipated that users may require several of the other references to hand, in particular, *Concise Eurocode 2*[5], which summarises the rules and principles that will be commonly used in the design of reinforced concrete framed buildings to Eurocode 2.



Note
The term Standards encompasses British Standards, Eurocodes, National Annexes (NA) and Published Documents

Figure 1.1
Worked examples in context

The designs are in accordance with BS EN 1992–1–1[1], as modified by the UK National Annex[1a] and explained in PD 6687[6].

Generally, the calculations are cross-referenced to the relevant clauses in all four parts of Eurocode 2[1–4] and, where appropriate, to other documents. See Figure 1.2 for a guide to presentation. References to BS 8110[7] refer to Part 1 unless otherwise stated.

Generally, the 'simple' examples depend on equations and design aids derived from Eurocode 2. The derived equations are given in Appendix A and the design aids from Section 15 of *Concise Eurocode 2*[5] are repeated in Appendix B.

The examples are intended to be appropriate for their purpose, which is to illustrate the use of Eurocode 2 for in-situ concrete building structures. There are simple examples to illustrate how typical hand calculations might be done using available charts and tables derived from the Code. These are followed by more detailed examples illustrating the detailed workings of the Codes. In order to explain the use of Eurocode 2, several of the calculations are presented in detail far in excess of that necessary in design calculations once users are familiar with the Code. To an extent, the designs are contrived to show valid methods of designing elements, to give insight and to help in validating computer methods. They are not necessarily the most appropriate, the most economic or the only methods of designing the members illustrated.

Sections 1 and 2		Worked examples
Cl. 6.4.4	Relevant clauses or figure numbers from BS EN 1992–1–1 (if the reference is to other parts, other Eurocodes or other documents this will be indicated)	Cl. 6.4.4
NA	From the relevant UK National Annex (generally to BS EN 1992–1–1)	NA
Cl. 6.4.4 & NA	From both BS EN 1992–1–1 and UK National Annex	Cl. 6.4.4 & NA
Fig. 2.1 Section 5.2	Relevant parts of this publication	Fig. 2.1 Section 5.2
EC1-1-1: 6.4.3	From other Eurocodes: BS EN 1990, BS EN 1991, BS EN 1992–1–2, etc	EC1-1-1: 6.4.3
PD 6687[6]	*Background paper to UK National Annexes BS EN 1992–1*	PD 6687[6]
Concise	*Concise Eurocode 2*[5]	Concise
How to: Floors [8]	*How to design concrete structures using Eurocode 2*[8]*: Floors*	How to: Floors[8]
Grey shaded tables	In Appendices, derived content in tables not from Eurocode 2	

Figure 1.2
Guide to presentation

As some of the detailing rules in Eurocode 2 are generally more involved than those to BS 8110, some of the designs presented in this publication have been extended into areas that have traditionally been the responsibility of detailers. These extended calculations are not necessarily part of 'normal' design but are included at the end of some calculations. It is assumed that the designer will discuss and agree with the detailer areas of responsibility and the degree of rationalisation, the extent of designing details, assessment of curtailment and other aspects that the detailer should undertake. It is recognised that in the vast majority of cases, the rules given in detailing manuals[8,9] will be used. However, the examples are intended to help when curtailment, anchorage and lap lengths need to be determined.

1.2 Eurocode: Basis of structural design

In the Eurocode system BS EN 1990, Eurocode: *Basis of structural design*[10] overarches all the other Eurocodes, BS EN 1991 to BS EN 1999. BS EN 1990 defines the effects of actions, including geotechnical and seismic actions, and applies to all structures irrespective of the material of construction. The material Eurocodes define how the effects of actions are resisted by giving rules for design and detailing of concrete, steel, composite, timber, masonry and aluminium. (see Figure 1.3).

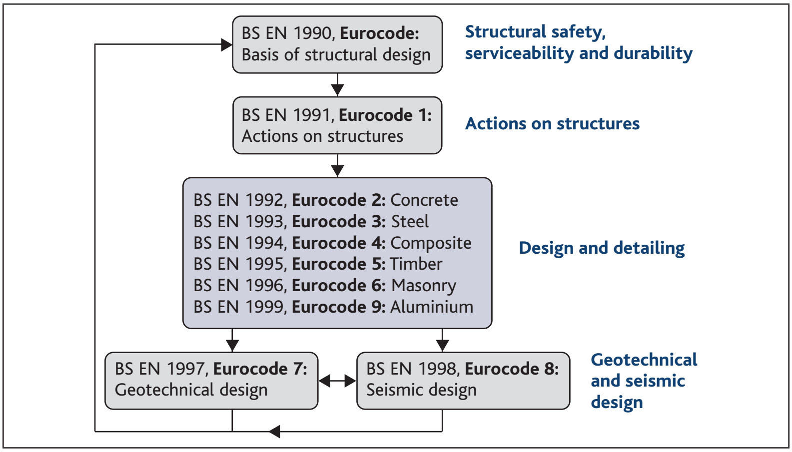


Figure 1.3
The Eurocode hierarchy

BS EN 1990 provides the necessary information for the analysis of structures including partial and other factors to be applied to the actions from Eurocode 1. It establishes the principles and requirements for the safety, serviceability and durability of structures. It describes the basis for design as follows:

EC0: 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.

In other words, it shall be designed using limit states principles to have adequate:

- Stability.
- Structural resistance (including structural resistance in fire).
- Serviceability.
- Durability.

For building structures, a design working life of 50 years is implied.

BS EN 1990 states that limit states should be verified in all relevant design situations: persistent, transient or accidental. No relevant limit state shall be exceeded when design values for actions and resistances are used in design. The limit states are:

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

All actions are assumed to vary in time and space. Statistical principles are applied to arrive at the magnitude of the partial load factors to be used in design to achieve the required reliability index (level of safety). There is an underlying assumption that the actions themselves are described in statistical terms.

1.3 Eurocode 1: Actions on structures

Actions are defined in the 10 parts of BS EN 1991 Eurocode 1: *Actions on structures*[11]:

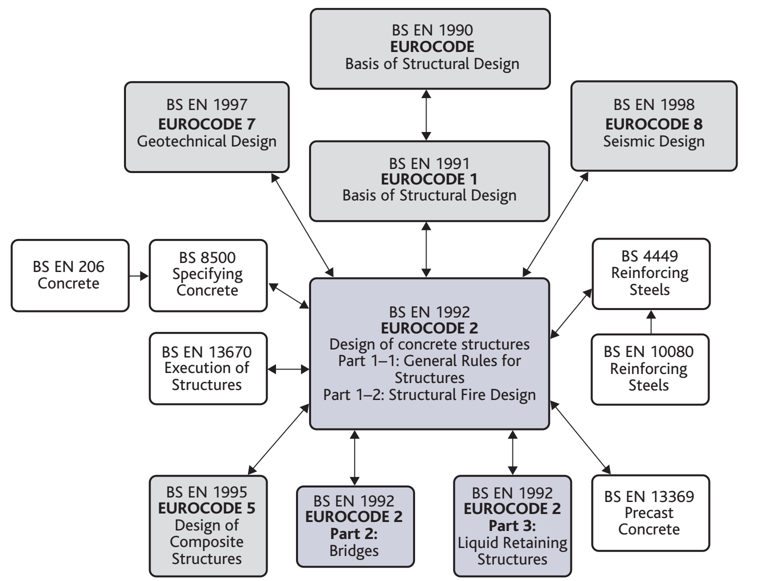
BS EN 1991–1–1: 2002: *Densities, self-weight, imposed loads for buildings*
BS EN 1991–1–2: 2002: *Actions on structures exposed to fire*
BS EN 1991–1–3: 2003: *Snow loads*
BS EN 1991–1–4: 2005: *Wind actions*
BS EN 1991–1–5: 2003: *Thermal actions*
BS EN 1991–1–6: 2005: *Actions during execution*
BS EN 1991–1–7: 2006: *Accidental actions*
BS EN 1991–2: 2003: *Actions on structures. Traffic loads on bridges*
BS EN 1991–3: 2006: *Cranes and machinery*
BS EN 1991–4: 2006: *Silos and tanks*

This publication is mainly concerned with designing for the actions defined by Part–1–1: *Densities, self-weight, imposed loads for buildings*.

Design values of actions and load arrangements are covered in Section 2.

1.4 Eurocode 2: Design of concrete structures

Eurocode 2: *Design of concrete structures*[1–4] operates within an environment of other European and British standards (see Figure 1.3). It is governed by BS EN 1990[10] and subject to the actions defined in Eurocodes 1[11], 7[12] and 8[13]. It depends on various materials and execution standards and is used as the basis of other standards. Part 2, *Bridges*[3], and Part 3, *Liquid retaining and containment structures*[4], work by exception to Part 1–1 and 1–2, that is, clauses in Parts 2 and 3 confirm, modify or replace clauses in Part 1–1.



Note
For clarity National Annexes and explanatory documents (e.g. PD 6687, and Non-Contradictory Complementary Information – NCCI) are not shown.

Figure 1.4
Eurocode 2 in context

1.5 National Annexes

It is the prerogative of each CEN Member State to control levels of safety in that country. As a result, some safety factors and other parameters in the Eurocodes, such as climatic conditions, durability classes and design methods, are subject to confirmation or selection at a national level. The decisions made by the national bodies become Nationally Determined Parameters (NDPs) which are published in a National Annex (NA) for each part of each Eurocode. The National Annex may also include reference to non-contradictory complementary information (NCCI), such as national standards or guidance documents.

This publication includes references to the relevant National Annexes as appropriate.

1.6 Basis of the worked examples in this publication

The design calculations in this publication are in accordance with:

- BS EN 1990, *Eurocode: Basis of structural design*[10] and its UK National Annex[10a].
- BS EN 1991, Eurocode 1: *Actions on structures* in 10 parts[11] and their UK National Annexes[11a].
- BS EN 1992–1–1, Eurocode 2 – Part 1–1: *Design of concrete structures – General rules and rules for buildings*[1] and its UK National Annex[1a].
- BS EN 1992–1–2, Eurocode 2 – Part 1–2: *Design of concrete structures – General rules – Structural fire design*[2] and its UK National Annex[2a].
- PD 6687, Background paper to the *UK National Annexes*[6].
- BS EN 1997, Eurocode 7: *Geotechnical design – Part 1. General rules*[12] and its UK National Annex[12a].

They use materials conforming to:

- BS 8500–1: *Concrete – Complementary British Standard to BS EN 206–1: Method of specifying and guidance to the specifier*[14].
- BS 4449: *Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification*[15].

They make reference to several publications, most notably:

- *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*[5].
- *How to design concrete structures using Eurocode 2*[8].

The execution of the works is assumed to conform to:

- PD 6687 *Background paper to the UK National Annexes BS EN 1992–1*.[6]
- NSCS, *National structural concrete specification for building construction*, 3rd edition[16] May 2004.

Or, when available

- BS EN 13670: *Execution of concrete structures.* Due 2010[17]. As implemented by specifications such as:
- NSCS, *National structural concrete specification for building construction*, 4th edition[18] CCIP-050, due 2010.

1.7 Assumptions

1.7.1 Eurocode 2

Eurocode 2 assumes that:

Cl. 1.3

- 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.

PD 6687[6]

- The requirements for execution and workmanship given in EN 13670 are complied with.

1.7.2 The worked examples

Unless noted otherwise, the calculations in this publication assume:

ECO: Table 2.1

- A design life of 50 years.

Table 3.1

- The use of C30/37 concrete.

BS 4449

- The use of Grade A, B or C reinforcement, designated 'H' in accordance with BS 8666[19].

Table 4.1, BS 8500: Table A.1

- Exposure class XC1.

Building Regs[20,21]

- 1 hour fire resistance.

Generally each calculation is rounded and it is the rounded value that is used in any further calculation.

1.8 Material properties

Material properties are specified in terms of their characteristic values. This usually corresponds to the lower 5% fractile of an assumed statistical distribution of the property considered.

The values of γ_C and γ_S, partial factors for materials, are indicated in Table 1.1.

Table 2.1 & NA

Table 1.1
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

1.9 Execution

In the UK, DD ENV 13670[22] is currently available but without its National Application Document. For building structures in the UK, the background document PD 6687[6] considers the provisions of the National Structural Concrete Specification (NSCS)[16] to be equivalent to those in EN 13670 for tolerance class 1. When published, BS EN 13670[17] and, if appropriate, the corresponding National Application Document will take precedence.

2 Analysis, actions and load arrangements

2.1 Methods of analysis

2.1.1 ULS

At the ultimate limit state (ULS) the type of analysis should be appropriate to the problem being considered. The following are commonly used:

- Linear elastic analysis.
- Linear elastic analysis with limited redistribution.
- Plastic analysis.

Cl. 5.1.1(7)

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 with $f_{ck} \leq 50$ MPa the minimum allowable ratio of the redistributed moment to the moment in the linear analysis, δ, is 0.70 where Class B or Class C reinforcement is used or 0.80 where Class A reinforcement is used.

Cl. 5.5.4 & NA

Within the limits set, coefficients for moment and shear derived from elastic analysis may be used to determine forces in regular structures (see Appendix B). The design of columns should be based on elastic moments without redistribution.

Cl. 5.1.1

Plastic analysis may be used for design at ULS provided that the required ductility can be assured, for example: by limiting x_u/d (to ≤ 0.25 for concrete strength classes ≤ C50/60); using Class B or C reinforcement; or ensuring the ratio of moments at intermediate supports to moments in spans is between 0.5 and 2.0.

Cl. 5.6.2

2.1.2 SLS

At the serviceability limit state (SLS) linear elastic analysis may be used. Linear elastic analysis may be carried out assuming:

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

Cl. 5.4(1)

2.2 Actions

Actions refer to loads applied to the structure as defined below:

- Permanent actions are actions for which the variation in magnitude with time is negligible. (EC1-1-1: 2.1)
- Variable actions are actions for which the variation in magnitude with time is not negligible. (EC1-1-1: 2.2, 3.3.1(2))
- 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. (EC1-1-7)

Imposed deformations are not considered in this publication.

2.3 Characteristic values of actions

The values of actions given in the various parts of Eurocode 1: *Actions on structures*[11] are taken as characteristic values. The characteristic value of an action is defined by one of the following three alternatives:

EC0: 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 actions with known statistical distributions, such as wind or snow.
- A nominal value – used for some variable and accidental actions.

2.4 Variable actions: imposed loads

2.4.1 General

Imposed loads on buildings are divided into categories. Those most frequently used in concrete design are shown in Table 2.1.

EC1-1-1: Tables 6.1, 6.7, 6.9 & NA

Category	Description
A	Areas for domestic and residential activities
B	Office areas
C	Areas of congregation
D	Shopping areas
E	Storage areas and industrial use (including access areas)
F	Traffic and parking areas (vehicles < 30 kN)
G	Traffic and parking areas (vehicles > 30 kN)
H	Roofs (inaccessible except for maintenance and repair)
I	Roofs (accessible with occupancy categories A – D)
K	Roofs (accessible for special services, e.g. for helicopter landing areas)

Table 2.1
Categories of imposed loads

Notes
1 Category J is not used.
2 For forklift loading refer to BS EN 1991–1–1 Cl. 6.2.3.

2.4.2 Characteristic values of imposed loads

Characteristic values for commonly used imposed loads are given in Tables 2.2 to 2.8.

Table 2.2
A: domestic and residential

EC1-1-1: Tables 6.1, 6.2 & NA.3

Sub-category	Example	Imposed loads	
		q_k (kN/m^2)	Q_k (kN)
A1	All usages within self-contained dwelling units. Communal areas (including kitchens) in small[a] blocks of flats	1.5	2.0
A2	Bedrooms and dormitories, except those in self-contained single family dwelling units and in hotels and motels	1.5	2.0
A3	Bedrooms in hotels and motels; hospital wards; toilet areas	2.0	2.0
A4	Billiard/snooker rooms	2.0	2.7
A5	Balconies in single-family dwelling units and communal areas in small[a] blocks of flats	2.5	2.0
A6	Balconies in hostels, guest houses, residential clubs. Communal areas in larger[a] blocks of flats	Min. 3.0 [b]	Min. 2.0 [c]
A7	Balconies in hotels and motels	Min. 4.0 [b]	Min. 2.0 [c]

Notes
a Small blocks of flats are those with ≤ 3 storeys and ≤ 4 flats per floor/staircase. Otherwise they are considered to be larger blocks of flats
b Same as the rooms to which they give access, but with a minimum of 3.0 kN/m^2 or 4.0 kN/m^2
c Concentrated at the outer edge

Table 2.3
B: offices

EC1-1-1: Tables 6.1, 6.2 & NA.3

Sub-category	Example	Imposed loads	
		q_k (kN/m^2)	Q_k (kN)
B1	General use other than in B2	2.5	2.7
B2	At or below ground floor level	3.0	2.7

Table 2.4
C: areas of congregation

EC1-1-1: Tables 6.1, 6.2 & NA.3

Sub-category	Example	Imposed loads	
		q_k	Q_k
C1	**Areas with tables**		
C11	Public, institutional and communal dining rooms and lounges, cafes and restaurants (Note: use C4 or C5 if appropriate)	2.0	3.0
C12	Reading rooms with no book storage	2.5	4.0
C13	Classrooms	3.0	3.0
C2	**Areas with fixed seats**		
C21	Assembly areas with fixed seating[a]	4.0	3.6
C22	Places of worship	3.0	2.7
C3	**Areas without obstacles for moving people**		
C31	Corridors, hallways, aisles in institutional type buildings, hostels, guest houses, residential clubs and communal areas in larger[b] blocks of flats	3.0	4.5
C32	Stairs, landings in institutional type buildings, hostels, guest houses, residential clubs and communal areas in larger[b] blocks of flats	3.0	4.0
C33	Corridors, hallways, aisles in other[c] buildings	4.0	4.5
C34	Corridors, hallways, aisles in other[c] buildings subjected to wheeled vehicles, including trolleys	5.0	4.5
C35	Stairs, landings in other[c] buildings subjected to crowds	4.0	4.0
C36	Walkways – Light duty (access suitable for one person, walkway width approx 600 mm)	3.0	2.0
C37	Walkways – General duty (regular two-way pedestrian traffic)	5.0	3.6
C38	Walkways – Heavy duty (high-density pedestrian traffic including escape routes)	7.5	4.5
C39	Museum floors and art galleries for exhibition purposes	4.0	4.5
C4	**Areas with possible physical activities**		
C41	Dance halls and studios, gymnasia, stages[d]	5.0	3.6
C42	Drill halls and drill rooms[d]	5.0	7.0
C5	**Areas subjected to large crowds**		
C51	Assembly areas without fixed seating, concert halls, bars and places of worship[d,e]	5.0	3.6
C52	Stages in public assembly areas[d]	7.5	4.5

Key

a Fixed seating is seating where its removal and the use of the space for other purposes is improbable

b Small blocks of flats are those with ≤ 3 storeys and ≤ 4 flats per floor/staircase. Otherwise they are considered to be 'larger' blocks of flats

c Other buildings include those not covered by C31 and C32, and include hotels and motels and institutional buildings subjected to crowds

d For structures that might be susceptible to resonance effects, reference should be made to NA.2.1

e For grandstands and stadia, reference should be made to the requirements of the appropriate certifying authority

EC1-1-1:
Tables 6.1, 6.2
& NA.3

Table 2.5
D: shopping areas

Sub-category	Example	Imposed loads	
		q_k (kN/m^2)	Q_k (kN)
D	**Shopping areas**		
D1	Areas in general retail shops	4.0	3.6
D2	Areas in department stores	4.0	3.6

EC1-1-1:
Tables 6.3, 6.4
& NA.4, NA.5

Table 2.6
E: storage areas and industrial use (including access areas)

Sub-category	Example	Imposed loads	
		q_k (kN/m^2)	Q_k (kN)
E1	**Areas susceptible to accumulation of goods including access areas**		
E11	General areas for static equipment not specified elsewhere (institutional and public buildings)	2.0	1.8
E12	Reading rooms with book storage, e.g. libraries	4.0	4.5
E13	General storage other than those specified[a]	2.4/m	7.0
E14	File rooms, filing and storage space (offices)	5.0	4.5
E15	Stack rooms (books)	2.4/m height (min. 6.5)	7.0
E16	Paper storage and stationery stores	4.0/m height	9.0
E17	Dense mobile stacking (books) on mobile trolleys in public and institutional buildings	4.8/m height	7.0
E18	Dense mobile stacking (books) on mobile trucks in warehouses	4.8/m height (min. 15.0)	7.0
E19	Cold storage	5.0/m height (min. 15.0)	9.0
E2	**Industrial use**	See BS EN 1991–1–1: Tables 6.5 & 6.6	
	Forklifts Classes FL1 to FL6		

Key
a Lower bound value given. More specific load values should be agreed with client

Table 2.7
F and G: traffic and parking areas

Sub-category	Example	Imposed loads	
		q_k (kN/m^2)	Q_k (kN)
F	**Traffic and parking areas (vehicles < 30 kN)**		
	Traffic and parking areas (vehicles < 30 kN)	2.5	5.0
G	**Traffic and parking areas (vehicles > 30 kN)**		
	Traffic and parking areas (vehicles > 30 kN)	5.0	To be determined for specific use

Table 2.8
H, I and K: roofs

EC1-1-1: 6.3.4.1(2), Tables 6.9, 6.10, 6.11 & NA.7

Sub-category	Example		Imposed loads q_k (kN/m^2)	Imposed loads Q_k (kN)
H	**Roofs (inaccessible except for maintenance and repair)**			
	Roof slope, α°	< 30°	0.6	0.9
		30° < α < 60°	0.6(60 – α)/30	
		< 60°	0	
I	**Roofs (accessible with occupancy categories A – D)**			
	Categories A – D		As Tables 2.2 to 2.5 according to specific use	
K	**Roofs (accessible for special services, e.g. for helicopter landing areas)**			
	Helicopter class HC1 (< 20 kN) (subject to dynamic factor ϕ = 1.4)		––	20
	Helicopter class HC2 (< 60 kN)		––	60

Notes

1 Roofs are categorized according to their accessibility. Imposed loads for roofs that are normally accessible are generally the same as for the specific use and category of the adjacent area. Imposed loads for roofs without access are given above.

2 There is no category J.

EC1-1-1: 6.3.4 & NA

Movable partitions

EC1-1-1: 6.3.1.2 (8) & NA

The self-weight of movable partitions may be taken into account by a uniformly distributed load, q_k, which should be added to the imposed loads of floors as follows:

- For movable partitions with a self-weight of 1.0 kN/m wall length: q_k = 0.5 kN/m^2.
- For movable partitions with a self-weight of 2.0 kN/m wall length: q_k = 0.8 kN/m^2.
- For movable partitions with a self-weight of 3.0 kN/m wall length: q_k = 1.2 kN/m^2.

Heavier partitions should be considered separately.

2.4.3 Reduction factors

General

EC1-1-1: 6.3.1.2 (10) 6.3.1.2(11) & NA

Roofs do not qualify for load reductions. The method given below complies with the UK National Annex but differs from that given in the Eurocode.

Area

EC1-1-1: 6.3.1.2 (10) & NA Exp. (NA.1)

A reduction factor for imposed loads for area, α_A, may be used and should be determined using:

$\alpha_A = 1.0 - A/1000 \geq 0.75$

where

A is the area (m^2) supported with loads qualifying for reduction (i.e. categories A to E as listed in Table 2.1).

EC1-1-1: 6.3.1.2 (11) & NA Exp. (NA.2)

Number of storeys

A reduction factor for number of storeys, α_n, may be used and should be determined using:

$\alpha_n = 1.1 - n/10$	for $1 \le n \le 5$
$\alpha_n = 0.6$	for $5 < n \le 10$
$\alpha_n = 0.5$	for $n > 10$

where

n = number of storeys with loads qualifying for reduction (i.e. categories A to D as listed in Table 2.1).

EC1-1-1: 6.3.1.2 (11) & NA

Use

According to the UK NA, α_A and α_n may not be used together.

2.5 Variable actions: snow loads

EC1-1-3: 5.2(3)

In persistent or transient situations, snow load on a roof, s, is defined as being:

$s = \mu_i C_e C_t s_k$

where

EC1-1-3: 5.3.1, 5.3.2 & NA

μ_i = snow load shape factor, , either μ_1 or μ_2
μ_1 = undrifted snow shape factor
μ_2 = drifted snow shape factor
For flat roofs, 0° = α (with no higher structures close or abutting),
$\mu_1 = \mu_2 = 0.8$
For shallow monopitch roofs, 0°< α < 30° (with no higher structures close or abutting), $\mu_1 = 0.8$, $\mu_2 = 0.8\,(1 + \alpha/30)$ For other forms of roof and local effects refer to BS EN 1991–1–3 Sections 5.3 and 6

EC1-1-3: 5.2(7) &Table 5.1

C_e = exposure coefficient
For windswept topography $C_e = 0.8$
For normal topography $C_e = 1.0$
For sheltered topography $C_e = 1.2$

EC1-1-3: 5.2(8)

C_t = thermal coefficient, $C_t = 1.0$ other than for some glass-covered roofs, or similar

EC1-1-3: & NA 2.8

s_k = characteristic ground snow load kN/m^2
= $0.15(0.1Z + 0.05) + (A + 100)/525$

where

Z = zone number obtained from the map in BS EN 1991–1–3 NA Figure NA.1
A = site altitude, m

Figure NA.1 of the NA to BS EN 1991–1–3 also gives figures for s_k at 100 m a.m.s.l. associated with the zones.

For the majority of the South East, the Midlands, Northern Ireland and the north of England apart from high ground, $s_k = 0.50$ kN/m^2.

For the West Country, West Wales and Ireland the figure is less. For most of Scotland and parts of the east coast of England, the figure is more. See Figure 2.1.

Snow load is classified as a variable fixed action. Exceptional circumstances may be treated as accidental actions in which case reference should be made to BS EN 1991–1–3.

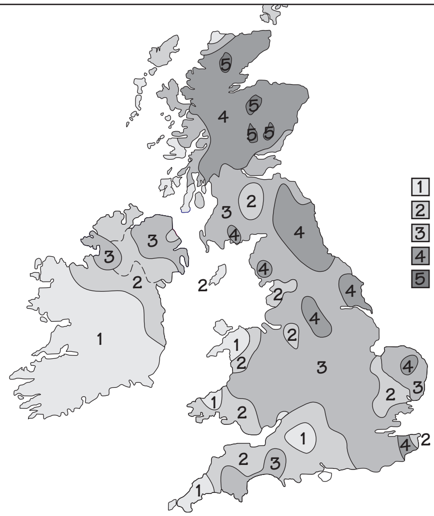


Figure 2.1
Characteristic ground snow load map (ground snow load at 100 m a.m.s.l. (kN/m^2))

EC1-1-3: NA Fig. NA.1

2.6 Variable actions: wind loads

This Section presents a very simple interpretation of Eurocode 1[11, 11a] and is intended to provide a basic understanding with respect to rectangular-plan buildings with flat roofs. In general, maximum values are given: with more information a lower value might be used. The user should be careful to ensure that any information used is within the scope of the application envisaged. The user is referred to more specialist guidance[23, 24] or BS EN 1991–1–4[25] and its UK National Annex[25a]. The National Annex includes clear and concise flow charts for the determination of peak velocity pressure, q_p.

In essence characteristic wind load can be expressed as:

$w_k = c_f\, q_{p(z)}$

where

c_f = force coefficient, which varies, but is a max. of 1.3 for overall load

$q_{p(z)} = c_{e(z)}\, c_{eT}\, q_b$

where

$c_{e(z)}$ = exposure factor from Figure 2.3

c_{eT} = town terrain factor from Figure 2.4

q_b = $0.006 v_b^2$ kN/m^2

where

$v_b = v_{b,map} c_{alt}$

where

$v_{b,map}$ = fundamental basic wind velocity from Figure 2.2

c_{alt} = altitude factor, conservatively, $c_{alt} = 1 + 0.001A$

where

A = altitude a.m.s.l

EC1-1-4: Figs NA.7, NA.8

EC1-1-4: Fig. NA.1

Symbols abbreviations and some of the caveats are explained in the sections below, which together provide a procedure for determining wind load to BS EN 1991–1–4.

2.6.1 Determine basic wind velocity, v_b

$v_b = c_{dir} c_{season} c_{prob} v_{b,0}$

where

EC1-1-4: 4.2(1) Note 2 & NA 2.4, 2.5

c_{dir} = directional factor
Conservatively, $c_{dir} = 1.0$
(c_{dir} is a minimum of 0.73 or 0.74 for wind in an easterly direction, 30° to 120°)

EC1-1-4: 4.2(2) Note 3 & NA 2.7: Fig. NA.2

c_{season} = season factor
For a 6 month return period, including winter, or greater,
$c_{season} = 1.00$

EC1-1-4: 4.2(1) Notes 4 & 5 & NA 2.8

c_{prob} = probability factor
= 1.00 for return period of 50 years

$v_{b,0}$ = $v_{b,map}\, c_{alt}$

where

EC1-1-4: 4.2(1) Note 2 & NA 2.4: Fig. NA.1

$v_{b,map}$ = fundamental basic wind velocity from Figure 2.2

EC1-1-4: 4.2(2) Note 1 & NA 2.5

c_{alt} = altitude factor
Conservatively, $c_{alt} = 1 + 0.001A$

where

A = altitude of the site in metres a.m.s.l.
Where orography is significant (i.e. the site is close to a slope steeper than 0.05), refer to NA 2.5

2.6.2 Calculate basic wind pressure, q_b

$q_b = 0.5 \rho v_b^2$

where

EC1-1-4: 4.5(1) Note 2 & NA 2.18

v_b = as above
ρ = density of air
= 1.226 kg/m³ (= 12.0 N/m³) for UK

2.6.3 Calculate peak wind pressure, $q_{p(z)}$

EC1-1-4: 4.5(1) Note 1 & NA 2.17

$q_{p(z)} = c_{e(z)}\, q_b$ for country locations
$= c_{e(z)}\, c_{eT}\, q_b$ for town locations

where

EC1-1-4: 4.5(1) Note 1, NA 2.17 & Fig. A.NA.1

q_b = as above

EC1-1-4: 4.5(1) Note 1 & NA 2.17: Fig. NA.7

$c_{e(z)}$ = exposure factor derived from Figure 2.3 at height z (see below)

EC1-1-4: 4.5(1) Note 1 & NA 2.17: Fig. NA.8

$c_{e,T}$ = exposure correction factor for town terrain derived from Figure 2.4

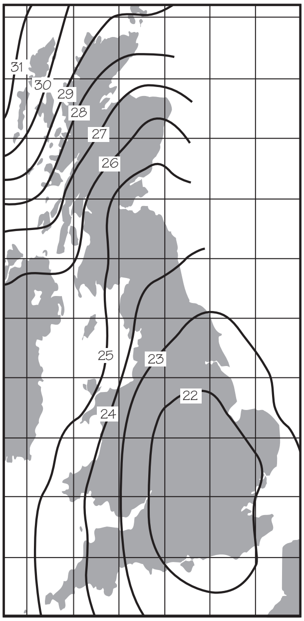
EC1-1-4: 7.2.2(1), Note & NA 2.26

z = the height at which q_p is sought
For a windward wall and when $h \leq b$, q_p is calculated at the reference height $z_e = h$. For other aspect ratios h:b of the windward wall, q_p ,is calculated at different reference heights for each part (see BS EN 1991–1–4).

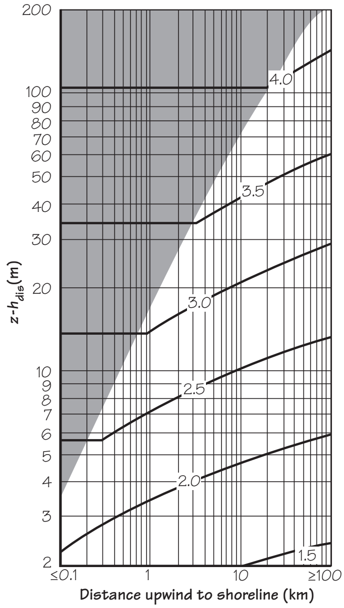
where

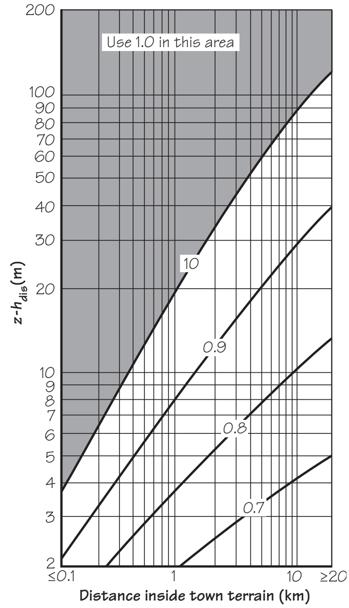
h = height of building
b = breadth of building

For leeward and side walls,
z = height of building



Note
Subject to altitude correction.





Note
Generally h_{dis} = 0. For terrain category IV (towns etc.) see BS EN 1991–1–4: A.5.

Figure 2.2
Map of fundamental basic wind velocity, $v_{b,map}$, (m/s)

EC1-1-4: 4.2(1)
Note 2 & NA 2.4: Fig. NA.1

Figure 2.3
Exposure factor $c_{e(z)}$ for sites in country or town terrain

EC1-1-4: 4.5(1)
Note 1 & NA 2.17: Fig. NA.7

Figure 2.4
Multiplier for exposure correction for sites in town terrain

EC1-1-4: 4.5(1)
Note 1 & NA 2.17: Fig. NA.8

2.6.4 Calculate characteristic wind load, w_k

$w_k = q_{p(z)} c_f$

where

$q_{p(z)}$ = as above
c_f = force coefficient for the structure or structural element
Generally
= $c_{pe} + c_{pi}$

EC1-1-4: 7,8 & NA

where
c_{pe} = (external) pressure coefficient dependent on size of area considered and zone. For areas above 1 m^2, $c_{pe,10}$ should be used.

EC1-1-4: 7.2.1(1) Note 2 & NA. 2.25

Overall loads
For the walls of rectangular-plan buildings, $c_{pe,10}$ may be determined from Table 7.1 of BS EN 1991–1–4.

EC1-1-4: 7.2.2(2) Note 1 & NA.2.27

However, for the determination of overall loads on buildings, the net pressure coefficients given in Table 2.9 may be used. In this case it will be unnecessary to determine internal wind pressure coefficients.

EC1-1-4: 7.2.2(2) Note 1 & NA.2.27, Table NA.4

Cladding loads
For areas above 1 m^2, $c_{pe,10}$ should be used. $c_{pe,10}$ may be determined from Table 7.1 of BS EN 1991–1–4. See Table 2.10.

EC1-1-4: 7.2.2(2) Note 1 & NA.2.27

EC1-1-4:
7.2.3, NA.2.28 & NA advisory note

Flat roofs

For flat roofs, according to the Advisory Note in the NA some of the values of $c_{pe,10}$ in Table 7.2 of BS EN 1991–1–4 (see Table 2.11) are significantly different from current practice in the UK. It recommends that designers should consider using the values in BS 6399:2 to maintain the current levels of safety and economy. See Table 2.12.

BS 6399:
Table 8 & Fig. 18

For other forms of roof refer to BS EN 1991–1–4 and the UK NA.
It will also be necessary to determine internal wind pressure coefficients for the design of cladding.

EC1-1-4:
NA.2.28 & NA advisory note

EC1-1-4:
7.2.9(6) Note 2

c_{pi} = internal pressure coefficient.
For no dominant openings c_{pi} may be taken as the more onerous of +0.2 and –0.3

EC1-1-4:
NA 2.27, Table NA.4

Table 2.9
Net pressure coefficient, $c_{pe,10}$, for walls of rectangular plan buildings*

h/d	Net pressure coefficient, $c_{pe,10}$
5	1.3
1	1.1
≥ 0.25	0.8

Notes
1 * in effect these values are force coefficients for determining overall loads on buildings.
2 *h* = height of building.
3 *b* = breadth of building (perpendicular to wind).
4 *d* = depth of building (parallel to wind).
5 Values may be interpolated.
6 Excludes funnelling.

EC1-1-4:
7.2.2(2) Table 7.1, Note 1 & NA 2.27: Tables NA.4a , NA.4b

Table 2.10
External pressure coefficient, $c_{pe,10}$, for walls of rectangular-plan buildings

Zone	Description	$c_{pe,10}$	
		Max.	Min.
Zone A	For walls parallel to the wind direction, areas within 0.2min[*b*; 2*h*] of windward edge		–1.2
Zone B	For walls parallel to the wind direction, areas within 0.2min[*b*; 2*h*] of windward edge		–0.8
Zone C	For walls parallel to the wind direction, areas from 0.2min[*b*; 2*h*] to min[*b*; 2*h*] of windward edge		–0.5
Zone D	Windward wall	+0.8	
Zone E	Leeward wall		–0.7
Zones D and E	Net	+1.3	

Notes
1 *h* = height of building.
2 *b* = breadth of building (perpendicular to wind).

EC1-1-4:
7.2, Table 7.2 & NA

Table 2.11
External pressure coefficient, $c_{pe,10}$ for flat roofs*

Zone	Description	$c_{pe,10}$	
		Sharp edge at eaves	With parapet
Zone F	Within 0.1min[b; 2h] of windward edge and within 0.2min[b; 2h] of return edge (parallel to wind direction)	–1.8	–1.6
Zone G	Within 0.1min[b; 2h] of windward edge and outwith 0.2min[b; 2h] of return edge (parallel to wind direction)	–1.2	–1.1
Zone H	Roof between 0.1min[b; 2h] and 0.5min[b; 2h] from windward edge	–0.7	–0.7
Zone I	Remainder between 0.5min[b; 2h] and leeward edge	±0.2	±0.2

Notes
1 * According to NA to BS EN 1991-1-4, this table is not recommended for use in the UK.
2 *h* = height of building.
3 *b* = breadth of building (perpendicular to wind).

Table 2.12
External pressure coefficient, c_{pe}, for flat roofs

Zone	Description	c_{pe}	
		Sharp edge at eaves	With parapet
Zone A	Within 0.1min[b; 2h] of windward edge and within 0.25min[b; 2h] of return edge (parallel to wind direction)	–2.0	–1.9
Zone B	Within 0.1min[b; 2h] of windward edge and outwith 0.25min[b; 2h] of return edge (parallel to wind direction)	–1.4	–1.3
Zone C	Roof between 0.1min[b; 2h] and 0.5min[b; 2h] from windward edge	–0.7	–0.7
Zone D	Remainder between 0.5min[b; 2h] and leeward edge	±0.2	±0.2

Notes
1 h = height of building.
2 b = breadth of building (perpendicular to wind).

EC1-1-4: 7.2.3, NA.2.28 & NA advisory note.

BS 6399: Table 8 & Fig. 18

2.6.5 Calculate the overall wind force, F_w

$F_w = c_s c_d \Sigma w_k A_{ref}$
where

EC1-1-4: 5.3.2, Exp. (5.4) & NA

w_k = as above

EC1-1-4: 6.2(1) a), 6.2(1) c)

$c_s c_d$ = structural factor, conservatively
= 1.0
or may be derived

EC1-1-4: 6.2(1) e) & NA.2.20

where
c_s = size factor
c_s may be derived from Exp. (6.2) or table NA.3. Depending on values of $(b + h)$ and $(z – h_{dis})$ and dividing into Zone A, B or C, a value of c_s (a factor < 1.00) may be found.

EC1-1-4: 6.3(1), Exp. (6.2) & NA.2.20, Table NA3

c_d = dynamic factor
c_d may be derived from Exp. (6.3) or figure NA.9. Depending on values of δ_s (logarithmic decrement of structural damping) and h/b, a value of c_d (a factor > 1.00) may be found.

EC1-1-4: 6.3(1), Exp. (6.3) & NA.2.20: Fig. NA9

c_d may be taken as 1.0 for framed buildings with structural walls and masonry internal walls, and for cladding panels and elements

EC1-1-4: 5.3.2, Exp. (5.4) & NA

A_{ref} = reference area of the structure or structural element

2.7 Variable actions: others

EC1-1-6, EC1-2, EC1-1-2, EC1-1-5, EC1-3 & EC1-4

Actions due to construction, traffic, fire, thermal actions, use as silos or from cranes are outside the scope of this publication and reference should be made to specialist literature.

2.8 Permanent actions

The densities and area loads of commonly used materials, sheet materials and forms of construction are given in Tables 2.13 to 2.15.

Actions arising from settlement, deformation and creep are outside the scope of this document but generally are to be considered as permanent actions. Where critical, refer to specialist literature.

Table 2.13
Bulk densities for soils and materials[11, 26]

Bulk densities	kN/m³	Bulk densities	kN/m³
Soils		**Materials**	
Clay – stiff	19–22	Concrete – reinforced	25.0
Clay – soft	16–19	Concrete – wet reinforced	26.0
Granular – loose	16–18	Glass	25.6
Granular – dense	19–21	Granite	27.3
Silty clay, sandy clay	16–20	Hardcore	19.0
Materials		Limestone (Portland stone – med. weight)	22.0
Asphalt	22.5	Limestone (marble – heavyweight)	26.7
Blocks – aerated concrete (min.)	5.0	Macadam paving	21.0
Blocks – aerated concrete (max.)	9.0	MDF	8.0
Blocks – dense aggregate	20.0	Plaster	14.1
Blocks – lightweight	14.0	Plywood	6.3
Books – bulk storage	8–11	Sandstone	23.5
Brickwork – blue	24.0	Screed – sand/cement	22.0
Brickwork – engineering	22.0	Steel/iron	77.0
Brickwork – fletton	18.0	Terracotta	20.7
Brickwork – London stock	19.0	Timber – Douglas fir	5.2
Brickwork – sand lime	21.0	Timber – European beech/oak	7.1
Chipboard	6.9	Timber – Grade C16	3.6
Concrete – aerated	10.0	Timber – Grade C24	4.1
Concrete – lightweight	18.0	Timber – Iroko/teak	6.4
Concrete – plain	24.0		

Table 2.14
Typical area loads for concrete slabs and sheet materials [11, 26]

Typical area loads	kN/m²	Typical area loads	kN/m²
Concrete slabs		**Sheet materials**	
Precast concrete solid units (100 mm)	2.50	Plaster skim coat	0.05
Precast concrete hollowcore units[a] (150 mm)	2.40	Plasterboard (12.5 mm)	0.09
Precast concrete hollowcore units[a] (200 mm)	2.87	Plasterboard (19 mm)	0.15
Precast concrete.hollowcore units[a] (300 mm)	4.07	Plywood (12.5 mm)	0.08
Precast concrete hollowcore units[a] (400 mm)	4.84	Plywood (19 mm)	0.12
Ribbed slab[b] (250 mm)	4.00	Quarry tiles including mortar bedding	0.32
Ribbed slab[b] (300 mm)	4.30	Raised floor – heavy duty	0.50
Ribbed slab[b] (350 mm)	4.70	Raised floor – medium weight	0.40
Waffle slab[c] – standard moulds (325 mm)	6.00	Raised floor – lightweight	0.30
Waffle slab[c] – standard moulds (425 mm)	7.30	Render (13 mm)	0.30
Waffle slab[c] – standard moulds (525 mm)	8.60	Screed – 50 mm	1.15
Sheet materials		Screed – lightweight (25 mm)	0.45
Asphalt (20 mm)	0.46	Stainless steel roofing (0.4 mm)	0.05
Carpet and underlay	0.05	Suspended ceiling – steel	0.10
Chipboard (18 mm)	0.12	Suspended fibreboard tiles	0.05
Dry lining on stud (20 mm)	0.15	T&G boards (15.5 mm)	0.09
False ceiling – steel framing	0.10	T&G boards (22 mm)	0.12
Felt (3 layer) and chippings	0.35	Tiles – ceramic floor on bedding	1.00
Glass – double glazing	0.52	Battens for slating and tiling	0.03
Glass – single glazing	0.30	Tiles – clay roof (max)	0.67
Insulation – glass fibre (150 mm)	0.03	Tiles – natural slate (thick)	0.65
Paving stones (50 mm)	1.20	Tiles – interlocking concrete	0.55
Plaster – two coat gypsum (12 mm)	0.21	Tiles – plain concrete	0.75

Key
a Hollowcore figures assume no topping (50 mm structural topping ≡ 1.25 kN/m²)
b Ribbed slabs: 150 web @ 750 centres with 100 mm thick flange/slab. Web slope 1:10
c Waffle slabs: 150 ribs @ 900 centres with 100 mm thick flange/slab. Web slope 1:10

Table 2.15
Loads for typical forms of construction[26]

Cavity wall	(kN/m²)
Brickwork 102.5 mm	2.40
Insulation 50 mm	0.02
Blockwork 100 mm	1.40
Plaster	0.21
Total	**4.0**
Lightweight cladding	**(kN/m²)**
Insulated panel	0.20
Purlins	0.05
Dry lining on stud	0.15
Total	**0.40**
Curtain walling	**(kN/m²)**
Allow	1.00
Precast concrete cladding	**(kN/m²)**
Facing	1.00
Precast panel (100 mm)	2.40
Insulation	0.05
Dry lining on stud	0.15
Total	**3.60**
Dry lining	**(kN/m²)**
Metal studs	0.05
Plasterboard and skim × 2	0.40
Total	**0.45**
Timber stud wall	**(kN/m²)**
Timber studs	0.10
Plasterboard and skim × 2	0.40
Total	**0.50**
Office floor	**(kN/m²)**
Carpet	0.03
Raised floor	0.30
Self-weight of 250 mm solid slab	6.25
Suspended ceiling	0.15
Services	0.30
Total	**7.03**
Office core area	**(kN/m²)**
Tiles and bedding, allow	1.00
Screed	2.20
Self-weight of 250 mm solid slab	6.25
Suspended ceiling	0.15
Services	0.30
Total	**9.90**
Stairs	**(kN/m²)**
150 mm waist (≡175 @ 25 kN/m³)	4.40
Treads 0.15 × 0.25 × 4/2 @ 25 kN/m³	1.88
Screed 0.05 @ 22 kN/m³	1.10
Plaster	0.21
Finish: tiles & bedding	1.00
Total	**8.60**

Residential floor	(kN/m²)
Carpet	0.05
Floating floor	0.15
Self-weight of 250 mm solid slab	6.25
Suspended ceiling	0.20
Services	0.10
Total	**6.75**
School floor	**(kN/m²)**
Carpet/flooring	0.05
Self-weight of 250 mm solid slab	6.25
Suspended ceiling	0.15
Services	0.20
Total	**6.60**
Hospital floor	**(kN/m²)**
Flooring	0.05
Self-weight of 250 mm solid slab	6.25
Screed	2.20
Suspended ceiling	0.15
Services (but can be greater)	0.05
Total	**8.70**
Flat roof/external terrace	**(kN/m²)**
Paving or gravel, allow	2.20
Waterproofing	0.50
Insulation	0.10
Self-weight of 250 mm solid slab ceiling	6.25
Suspended ceiling	0.15
Services	0.30
Total	**9.50**
Timber pitched roof	**(kN/m²)**
Tiles (range 0.50–0.75)	0.75
Battens	0.05
Felt	0.05
Rafters	0.15
Insulation	0.05
Plasterboard & skim	0.15
Services	0.10
Ceiling joists	0.15
Total perpendicular to roof	**1.45**
Total on plan assuming 30° pitch	**1.60**
Metal decking roof	**(kN/m²)**
Insulated panel	0.20
Purlins	0.10
Steelwork	0.30
Services	0.10
Total	**0.70**

2.9 Design values of actions

2.9.1 General case

The design value of an action, F_d, that occurs in a load case is

$$F_d = \gamma_F \psi F_k$$

where

γ_F = partial factor for the action according to the limit state under consideration. Table 2.16 indicates the partial factors to be used in the UK for the combinations of representative actions in building structures.

ψF_k may be considered as the representative action, F_{rep}, appropriate to the limit state being considered

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 nature of the limit state under consideration and the joint probability of the actions occurring simultaneously. It can assume the value of 1.0 for a permanent action or ψ_0 or ψ_1 or ψ_2 for a variable action. Table 2.17 shows how characteristic values of variable actions are converted into representative values. This table is derived from BS EN 1990[10] and its National Annex[10a].

F_k = characteristic value of an action as defined in Sections 2.2 and 2.3.

ECO: Tables A1.2(A), A1.2(B), A1.2(C), A1.4 & NA

Table 2.16
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 ($Q_{k,i}$)[d]
a) Equilibrium (EQU)			
	1.10 (0.9)[a]	1.50 (0.0)[a]	$\psi_{0,i}$ 1.50 (0.0)[a]
b) Strength at ULS (STR/GEO) not involving geotechnical actions			
Either			
Exp. (6.10)	1.35 (1.0)[a]	1.5	$\psi_0$1.5
or the worst case of			
Exp. (6.10a)	1.35 (1.0)[a]	$\psi_0$1.5	$\psi_0$1.5
and			
Exp. (6.10b)	1.25 (1.0)[a]	1.5	$\psi_0$1.5
c) Strength at ULS (STR/GEO) with geotechnical actions			
Worst case of			
Set B	1.35 (1.0)[a]	1.5 (0.0)[a]	
and			
Set C	1.0	1.3	
d) Serviceability			
Characteristic	1.00	1.00	$\psi_{0,i}$ 1.00
Frequent	1.00	$\psi_{1,1}$ 1.00	$\psi_{2,i}$ 1.00
Quasi-permanent	1.00	$\psi_{2,1}$ 1.00	$\psi_{2,i}$ 1.00
e) Accidental design situations			
Exp. (6.11a)	1.0	A_d[b]	$\psi_{1,i}$ (main) $\psi_{2,i}$ (others)
f) Seismic			
Exp. (6.12a/b)	1.0	A_{Ed}[c]	$\psi_{2,i}$

Key
a Value if favourable (shown in brackets)
b Leading accidental action, A_d, is unfactored
c Seismic action, A_{Ed}
d Refer to BS EN 1990: A1.2.2 & NA

Notes
1 The values of ψ are given in Table 2.17.
2 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 its National Annex.

2.9.2 Design values at ULS

ECO: 6.4.3.2(3)

For the ULS of strength (STR), the designer may choose between using Expression (6.10) or the worst case of Expression (6.10a) or Expression (6.10b).

Single variable action

At ULS, the design value of actions is

either

Exp. (6.10) $1.35\ G_k + 1.5\ Q_{k,1}$

or the worst case of:

Exp. (6.10a) $1.35\ G_k + \psi_{0,1}\ 1.5\ Q_{k,1}$

and

Exp. (6.10b) $1.25\ G_k + 1.5\ Q_{k,1}$

where

G_k = permanent action

$Q_{k,1}$ = single variable action

$\psi_{0,1}$ = combination factor for a single variable load (see Table 2.17)

ECO: A1.2.2 & NA

EC1-1-1: 3.3.2

Table 2.17
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 a.m.s.l.[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

Key
a On roofs, imposed loads, snow loads and wind loads should not be applied together.

Notes
1. The numerical values given above are in accordance with BS EN 1990 and its UK National Annex.
2. Categories K and L are assumed to be as for Category H

Expression (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).

Expression (6.10) is always equal to or more conservative than the less favourable of Expressions (6.10a) and (6.10b). Expression (6.10b) will normally apply when the permanent actions are not greater than 4.5 times the variable actions (except for storage loads, category E in Table 2.17, where Expression (6.10a) always applies).

Therefore, except in the case of concrete structures supporting storage loads where $\psi_0 = 1.0$, or for mixed use, Expression (6.10b) will usually apply. Thus, for members supporting vertical actions at ULS, $1.25G_k + 1.5Q_k$ will be appropriate for most situations and applicable to most concrete structures (see Figure 2.5).

Compared with the use of Expression (6.10), the use of either Expression (6.10a) or (6.10b) leads to a more consistent reliability index across lightweight and heavyweight materials.

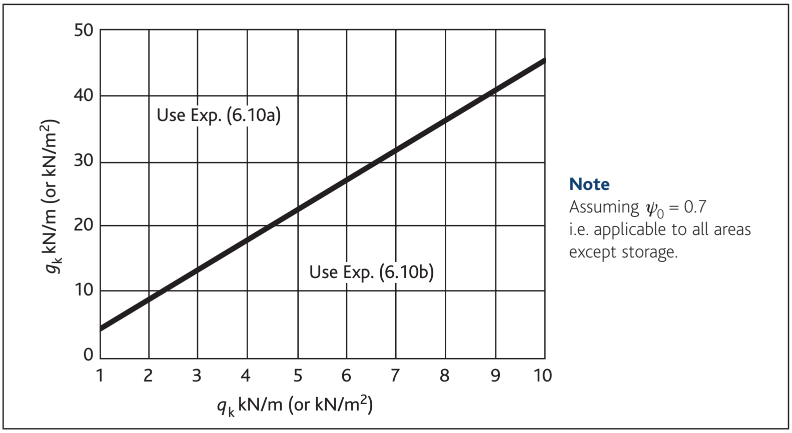


Figure 2.5
When to use Exp. (6.10a) or Exp. (6.10b)

Accompanying variable actions

Again the designer may choose between using Expression (6.10) or the less favourable of Expressions (6.10a) or (6.10b).

ECO: 6.4.3.2(3)

Either

Exp. (6.10) $1.35\ G_k + 1.5\ Q_{k,1} + \Sigma(\psi_{0,i}\ 1.5\ Q_{k,i})$
or the worst case of:
Exp. (6.10a) $1.35\ G_k + \psi_{0,1}\ 1.5\ Q_{k,1} + \Sigma(\psi_{0,i}\ 1.5\ Q_{k,i})$
and
Exp. (6.10b) $1.25\ G_k + 1.5\ Q_{k,1} + \Sigma(\psi_{0,i}\ 1.5\ Q_{k,i})$

where

G_k = permanent action
$Q_{k,1}$ = 1st variable action
$Q_{k,i}$ = ith variable action
$\psi_{0,1}$ = characteristic combination factor for 1st variable load (see Table 2.17)
$\psi_{0,i}$ = characteristic combination factor for ith variable load (see Table 2.17)

In the above, $Q_{k,1}$ (and $\psi_{0,i}$) refers to the leading variable action and $Q_{k,i}$ (and $\psi_{0,i}$) refers to accompanying independent variable actions. In general, the distinction between the two types of actions will be obvious (see Figure 2.6); where it is not, each load should in turn be treated as the leading action. Also, the numerical values for partial factors given in the UK National Annex[10a] are used in the equations above. The value of ψ_0 depends on the use of the building and should be obtained from the UK National Annex for BS EN 1990 (see Table 2.17).

ECO: A1.2.2, A1.3.1 & NA

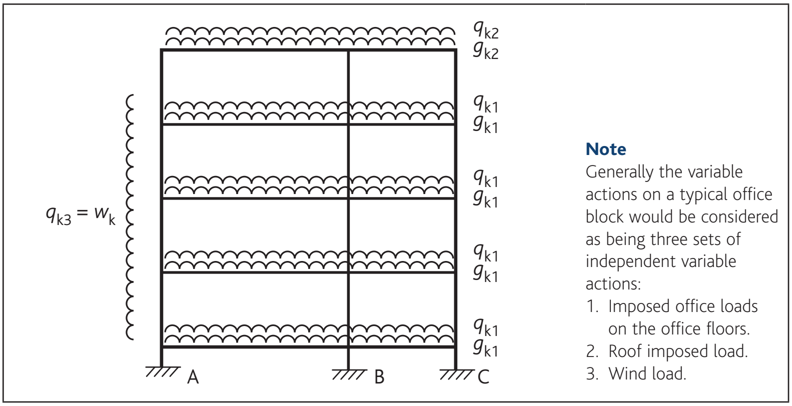


Figure 2.6
Independent variable actions

The expressions take into account the probability of joint occurrence of loads by applying the $\psi_{0,i}$ factor to the accompanying variable action. The probability that these combined actions will be exceeded is deemed to be similar to the probability of a single action being exceeded.

If the two independent variable actions $Q_{k,1}$ and $Q_{k,2}$ are associated with different spans and the use of Expression (6.10b) is appropriate, then in one set of analyses apply

$1.25G_k + 1.5Q_{k,1}$ to the $Q_{k,1}$ spans

and $1.25G_k + \psi_{0.i}\ 1.5Q_{k,1}$ to the $Q_{k,2}$ spans.

In associated analyses apply

$1.25G_k + \psi_{0,i}\ 1.5Q_{k,1}$ to the $Q_{k,1}$ spans

and $1.25G_k + 1.25Q_{k,2}$ to the $Q_{k,2}$ spans.

See Example 2.11.2 (two variable actions).

2.9.3 Design values at SLS

ECO: 6.5 & Table A1.4

There are three combinations of actions at SLS (or load combination at SLS). These are given in Table 2.18. The combination and value to be used depends on the nature of the limit state being checked. Quasi-permanent combinations are associated with deformation, crack widths and crack control. Frequent combinations may be used to determine whether a section is cracked or not. The numeric values of ψ_0, ψ_1 and ψ_2 are given in Table 2.17.

Colloquially

ψ_0 has become known as the 'characteristic' value

ψ_1 has become known as the 'frequent' value

ψ_2 has become known as the 'quasi-permanent' value

ECO: Table A1.4

Table 2.18
Partial factors to be applied in the verification of the SLS

Combination	Permanent actions G_k		Variable actions Q_k	
	Unfavourable[a]	Favourable[a]	Leading[b]	Others[b]
Characteristic	$G_{k,sup}$	$G_{k,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{k,sup}$	$G_{k,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,sup}$	$G_{k,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

Key
a Generally $G_{k,sup}$ and $G_{k,inf}$ may be taken as G_k. See Section 2.9.5
b ψ factors are given in Table 2.17

2.9.4 Design values for other limit states

Load combinations are given in Table 2.16 for

a) Equilibrium (EQU),

b) Strength at ULS not involving geotechnical actions,

c) Strength at ULS with geotechnical actions,

d) Serviceability,

e) Accidental and

f) Seismic design situations.

2.9.5 Variations in permanent actions

When the variation of a permanent action is not small then the upper ($G_{kj,sup}$) and the lower ($G_{kj,inf}$) characteristic values (the 95% and 5% fractile values respectively) should be established. This procedure is necessary only when the coefficient of variation (= 100 × standard deviation/ mean) is greater than 10. In terms of permanent actions, variations in the self-weight of concrete in concrete frames are considered small.

ECO: 4.1.2, 4.1.2 (3)

PD 6687[6]: 2.8.4

At ULS where the variation is not small, $\gamma_{Gk,sup}$ should be used with $G_{kj,sup}$ and $\gamma_{Gk,inf}$ with $G_{kj,inf}$. Similarly, where the variation is not small, at SLS $G_{kj,sup}$ should be used where actions are unfavourable and $G_{kj,inf}$ used where favourable.

Where checks, notably checks on static equilibrium (EQU), are very sensitive to variation of the magnitude of a permanent action from one place to another, the favourable and unfavourable parts of this action should be considered as individual actions. In such 'very sensitive' verifications $\gamma_{G,sup}$ and $\gamma_{G,inf}$ should be used.

ECO: 6.4.3 (4)

2.10 Load arrangements of actions: introduction

The process of designing concrete structures involves identifying relevant design situations and limit states. These include persistent, transient or accidental situations. In each design situation the structure should be verified at the relevant limit states.

ECO: 3.2

In the analysis of the structure at the limit state being considered, the maximum effect of actions should be obtained using a realistic arrangement of loads. Generally variable actions should be arranged to produce the most unfavourable effect, for example to produce maximum overturning moments in spans or maximum bending moments in supports.

For building structures, design concentrates mainly on the ULS, the ultimate limit state of strength (STR), and SLS, the serviceability limit state. However, it is essential that all limit states are considered. The limit states of equilibrium (EQU), strength at ULS with geotechnical actions (STR/GEO) and accidental situations must be taken into account as appropriate.

ECO: 3.3, 3.4, 6.4, 6.5

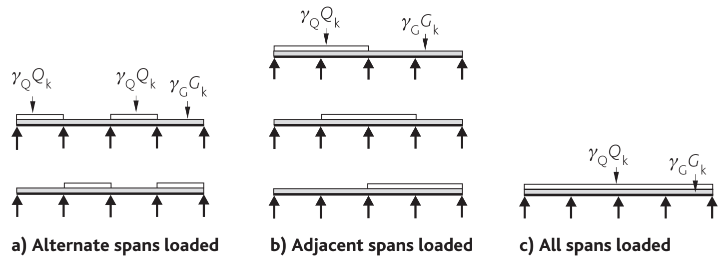
2.11 Load arrangements according to the UK National Annex to Eurocode

In building structures, any of the following sets of simplified load arrangements may be used at ULS and SLS (See Figure 2.7).

Cl. 5.1.3 & NA

- 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 $\gamma_G G_k + \gamma_G G_k$, 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).
  - The ratio of the variable action, Q_k, to the permanent action, G_k, does not exceed 1.25.
  - The 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.



Note

Whilst the use of Exp. (6.10) is indicated, these arrangements may equally be used with Exp. (6.10a) or (6.10b).

Figure 2.7
Load arrangements for beams and slabs according to UK NA to Eurocode

2.12 Examples of loading

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: **Continuous beam in a domestic structure**	Calculated by: chg	Job no.: CCIP – 041
		Checked by: web	Sheet no.: 1
		Client: TCC	Date: Oct 09

2.12.1 Continuous beam in a domestic structure

Determine the appropriate load combination and ultimate load for a continuous beam of four 6 m spans in a domestic structure supporting a 175 mm slab at 6 m centres.

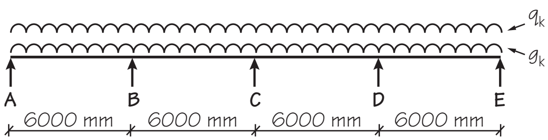


Figure 2.8 Continuous beam in a domestic structure

a) **Actions** — **kN/m**

Permanent action, g_k

Self-weight, 175 mm thick slabs : 0.17 x 25 x 6.0		= 26.3
E/o self-weight downstand 800 × 225 : 0.80 x 0.225 x 25		= 4.5
50 mm screed @ 22 kN/m^3 : 0.05 x 22 x 6.0		= 6.6
Finishes and services : 0.50 x 6.0		= 3.0
Dividing wall 2.40 × 4.42 (200 mm dense blockwork with plaster both sides)		= 10.6
Total	g_k	= 51.0

Variable action, q_k

Imposed, dwelling @ 1.5 kN/m^2 : 1.5 x 6.0		= 9.0
Total	q_k	= 9.0

Ultimate load, n

Assuming use of Exp. (6.10), $n = 1.35 \times 51 + 1.5 \times 9.0 =$ = 82.4

Assuming use of worst case of Exp. (6.10a) or Exp. (6.10b)

Exp. (6.10a): $n = 1.35 \times 51 + 0.7 \times 1.5 \times 9.0 =$ = 78.3

Exp. (6.10b): $n = 1.25 \times 51 + 1.5 \times 9.0 =$ = 77.3

In this case Exp. (6.10a) would be critical‡

∴ ultimate load = 78.3

‡ This could also be determined from Figure 2.5 or by determining that $g_k > 4.5q_k$

2.12.2 Continuous beam in mixed use structure

EC1-1-1: 6.3.1.1 &NA, ECO: A.1.2.2. &NA

Determine the worst case arrangements of actions for ULS design of a continuous beam supporting a 175 mm slab @ 6 m centres. Note that the variable actions are from two sources as defined in Figure 2.9.:

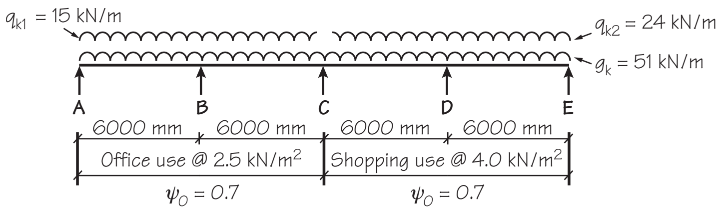


Figure 2.9 Continuous beam in mixed-use structure

a) Load combination

Load combination Exp. (6.10a) or Exp. (6.10b) will be used, as either will produce a smaller total load than Exp. (6.10). It is necessary to decide which expression governs.

i) Actions — **kN/m**

Permanent action

As before, Example 2.12.1 — $g_k = 51.0$

Variable action

Office @ 2.5 kN/m^2 — $q_{k1} = 15.0$

Shopping @ 4.0 kN/m^2 — $q_{k2} = 24.0$

Ultimate load, *n*

For office use:

Exp. (6.10a): $n = 1.35 \times 51 + 0.7 \times 1.5 \times 15.0$ $= 84.6$

Exp. (6.10b): $n = 1.25 \times 51 + 1.5 \times 15.0$ $= 86.3$

For shopping use:

Exp. (6.10a): $n = 1.35 \times 51 + 1.5 \times 0.7 \times 24.0$ $= 94.1$

Exp. (6.10b): $n = 1.25 \times 51 + 1.5 \times 24.0$ $= 99.8$

By inspection Exp. (6.10b) governs in both cases‡

b) Arrangement of ultimate loads

As the variable actions arise from different sources, one is a leading variable action and the other is an accompanying variable action. The unit loads to be used in the various arrangements are:

‡ This could also be determined from Figure 2.5 or by determining that $g_k > 4.5q_k$

i) Actions kN/m

Permanent

1.25 x 51.0 = 63.8

Variable

Office use

as leading action, $\gamma_Q Q_k$ = 1.5 x 15 = 22.5

as accompanying action, $\psi_0 \gamma_Q Q_k$ = 0.7 x 1.5 x 15 = 15.75

Shopping use

as leading action, $\gamma_Q Q_k$ = 1.5 x 24 = 36.0

as accompanying action, $\psi_0 \gamma_Q Q_k$ = 0.7 x 1.5 x 24 = 25.2

ii) For maximum bending moment in span AB

The arrangement and magnitude of actions of loads are shown in Figure 2.10. The variable load in span AB assumes the value as leading action and that in span CD takes the value as an accompanying action.

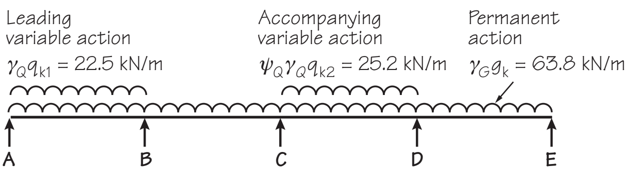


Figure 2.10 For maximum bending moment in span AB

iii) For maximum bending moment in span CD

The load arrangement is similar to that in Figure 2.10, but now the variable load in span AB takes its value as an accompanying action
(i.e. 15.75 kN/m) and that in span CD assumes the value as leading action (36 kN/m).

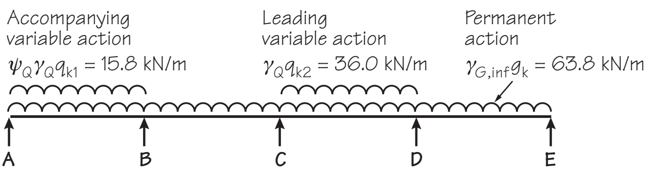


Figure 2.11 For maximum bending moment in span CD

iv) For maximum bending moment at support B

The arrangement of loads is shown in Figure 2.12. As both spans AB and BC receive load from the same source, no reduction is possible (other than that for large area‡).

EC1-1-1: 6.3.1.1 (10) & NA

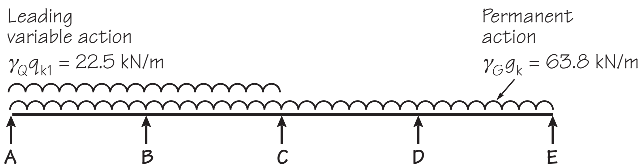


Figure 2.12 For maximum bending moment at support B

v) For maximum bending moment at support D

The relevant arrangement of loads is shown in Figure 2.13. Comments made in d) also apply here.

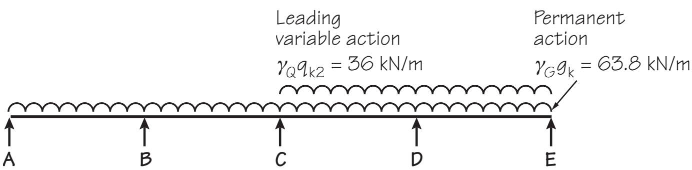


Figure 2.13 For maximum bending moment at support D

vi) For critical curtailment and hogging in span CD

The relevant arrangement of loads is shown in Figure 2.14.

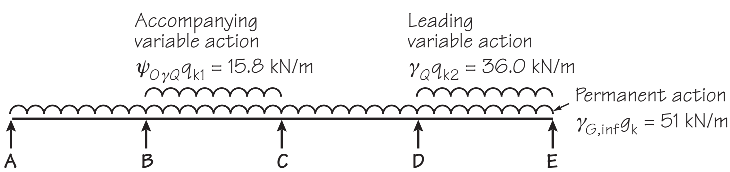


Figure 2.14 For curtailment and hogging in span CD

Eurocode 2 requires that all spans should be loaded with either $\gamma_{G,sup}$ or $\gamma_{G,inf}$ (as per Table 2.16). As illustrated in Figure 2.14, using $\gamma_{G,inf}$ = 1.0 might be critical for curtailment and hogging in spans.

Cl. 2.4.3(2)

‡ Variable actions may be subjected to reduction factors: α_A, according to the area supported (m^2), $\alpha_A = 1.0 - A/1000 \geq 0.75$.

EC1-1-1: 6.3.1.2 (10) & NA

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Propped cantilever	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

2.12.3 Propped cantilever

Determine the Equilibrium, ULS and SLS (deformation) load combinations for the propped cantilever shown in Figure 2.15. The action P at the end of the cantilever arises from the permanent action of a wall. ECO: Table 1.2(B), Note 3

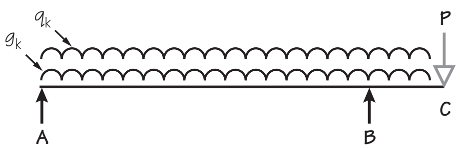


Figure 2.15 Propped cantilever beam and loading

For the purposes of this example, the permanent action P is considered to be from a separate source than the self-weight of the structure so both $\gamma_{G,sup}$ and $\gamma_{G,inf}$ need to be considered.

ECO: Table A1.2 (A) & NA

a) Equilibrium limit state (EQU) for maximum uplift at A

ECO: 6.4.3.1 (4), Table A1.2 (A) & NA

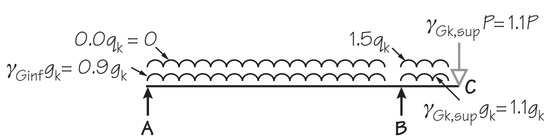


Figure 2.16 EQU: maximum uplift at A

b) Ultimate limit state (ULS)

i) For maximum moment at B and anchorage of top reinforcement BA

ECO: Tables A1.1, A1.2 (B) & NA

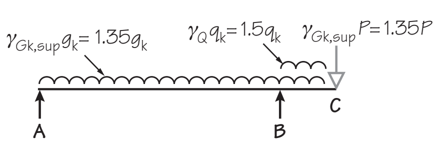


Figure 2.17 ULS: maximum moment at B

Notes

$\gamma_{Gk,inf}\,g_k = 1.0\,g_k$ may be critical in terms of curtailment of top bars BA.

ii) For maximum sagging moment AB

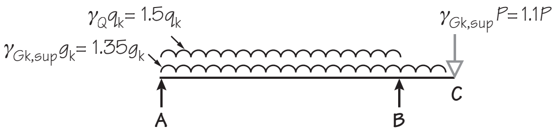


Figure 2.18 ULS: maximum span moment AB

ECO: Table A1.1, A1.2 (B) & NA

Notes

1 Depending on the magnitude of g_k, q_k length AB and BC, $\gamma_{Gk,inf}\, g_k$ (= 1.0 g_k) may be more critical for span moment.

2 The magnitude of the load combination indicated are those for Exp. (6.10) of BS EN 1990. The worst case of Exp. (6.10a) and Exp. (6.10b) may also have been used.

3 Presuming supports A and B were columns then the critical load combination for Column A would be as Figure 2.18. For column B the critical load combination might be either as Figure 2.17 or 2.18.

c) Serviceability limit state (SLS) of deformation: (quasi-permanent loads)

i) For maximum deformation at C

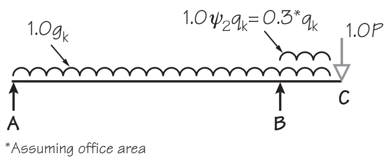


Figure 2.19 SLS: maximum deformation at C

ECO: Tables A1.1, A1.2.2, A1.4 & NA

ii) For maximum deformation AB

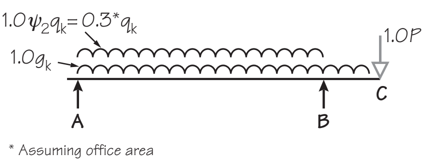


Figure 2.20 SLS: maximum deformation AB

Notes

Quasi-permanent load combinations may also be used for calculations of crack widths or controlling cracking, i.e. the same load combinations as shown in Figures 2.19 and 2.20 may be used to determine SLS moment to determine stress in reinforcement. The characteristic and/or frequent combinations may be appropriate for other SLS limit states: for example, it is recommended that the frequent combination is used to determine whether a member has cracked or not.

	Project details	Calculated by	chg	Job no.	CCIP-041
The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Overall stability	Checked by	web	Sheet no.	1
		Client	TCC	Date	Oct 09

2.12.4 Overall stability (EQU)

For the frame shown in Figure 2.21, identify the various load arrangements to check overall stability (EQU) against overturning. Assume that the structure is an office block and that the loads q_{k2} and q_{k3} may be treated as arising from one source.

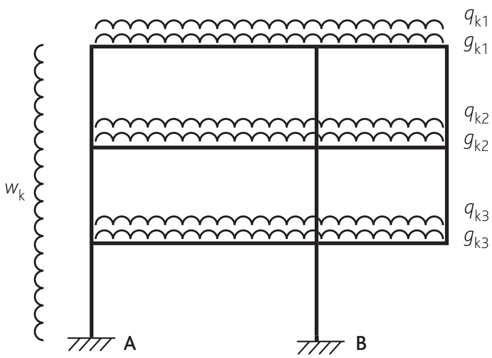


Figure 2.21 Frame configuration

a) EQU – Treating the floor imposed load as the leading variable action

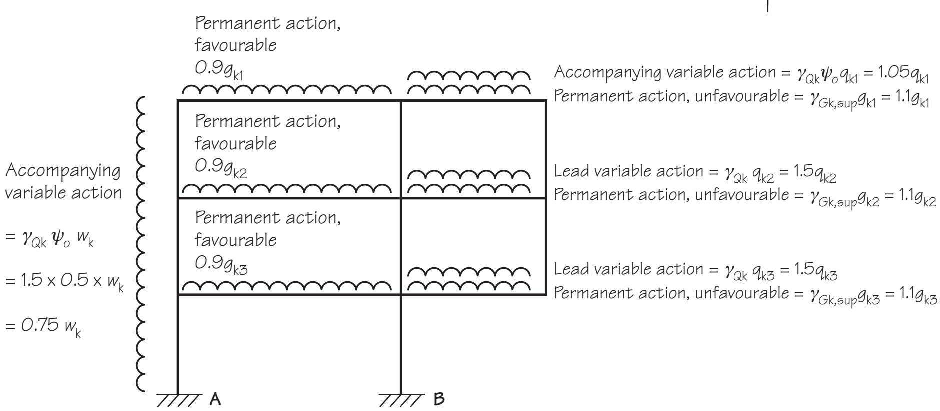


Figure 2.22 Frame with floor variable action as leading variable action

Tables 2.16 & 2.17

See Table 2.17 for values of ψ_0

b) **EQU – Treating the roof imposed load as the leading variable action**

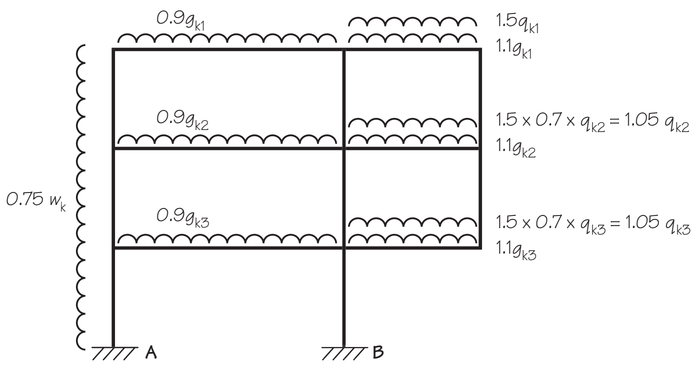


Figure 2.23 Frame with roof variable action as leading variable action

Tables 2.16 & 2.17

c) **EQU – Treating wind as the leading variable action**

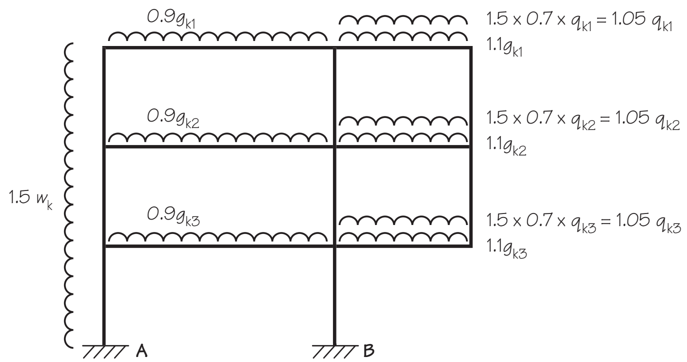


Figure 2.24 Frame with wind as lead variable action

Tables 2.16 & 2.17

3 Slabs

3.0 General

The calculations in this section are presented in the following sub-sections:

3.1 A simply supported one-way slab
3.2 A continuous one-way slab
3.3 A continuous ribbed slab
3.4 A bay of a flat slab
3.5 A stair flight.

These calculations are intended to show what might be deemed typical hand calculations. They are illustrative of the Code and are not necessarily best practice. The first three sub-sections include detailing checks e.g. curtailment lengths determined strictly in accordance with the provisions of BS EN 1992–1–1. The flat slab calculation is supplemented by a commentary.

A general method of designing slabs is shown below.

■ Determine design life.	**EC0 & NA Table NA.2.1**
■ Assess actions on the slab.	**EC1 & NA**
■ Assess durability requirements and determine concrete strength.	**Table 4.1** **BS 8500-1: Tables A4 & A5**
■ Check cover requirements for appropriate fire resistance period.	**EC2-1-2: Tables 5.8, 5.9, 5.10 & 5.11**
■ Calculate minimum cover for durability, fire and bond requirements.	**Cl. 4.4.1**
■ Determine which combinations of actions apply.	**EC0 & NA Tables NA.A1.1 & NA.A1.2 (B)**
■ Determine loading arrangements.	**Cl. 5.1.3(1) & NA**
■ Analyse structure to obtain critical moments and shear forces.	**Cl. 5.4, 5.5, 5.6**
■ Design flexural reinforcement.	**Cl. 6.1**
■ Check deflection.	**Cl. 7.4**
■ Check shear capacity.	**Cl. 6.2**
■ Other design checks:	
Check minimum reinforcement	**Cl. 9.3.1.1(1), 9.2.1.1(1)**
Check cracking (size or spacing of bars)	**Cl. 7.3, Tables 7.2N & 7.3N**
Check effects of partial fixity	**Cl. 9.3.1.2(2)**
Check secondary reinforcement.	**Cl. 9.3.1.1(2), 9.3.1.4(1)**
■ Check curtailment.	**Cl. 9.3.1.1(4), 9.2.1.3, Fig. 9.2**
■ Check anchorage.	**Cl. 9.3.1.2, 8.4.4, 9.3.1.1(4)** **Cl. 9.2.1.5(1), 9.2.1.5(2)**
■ Check laps.	**Cl. 8.7.3**

3.1 Simply supported one-way slab

This calculation is intended to show a typical basic hand calculation.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details	Calculated by	chg	Job no.	CCIP – 041
	Simply supported one-way slab	Checked by	web	Sheet no.	1
		Client	TCC	Date	Oct 09

A 175 mm thick slab is required to support screed, finishes, an office variable action of 2.5 kN/m^2 and demountable partitions (@ 2 kN/m). The slab is supported on load-bearing block walls. f_{ck} = 30 MPa, f_{yk} = 500 MPa. Assume a 50-year design life and a requirement for 1 hour resistance to fire.

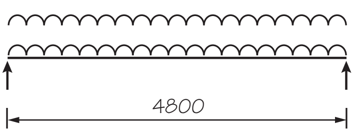


Figure 3.1 Simply supported one-way slab

3.1.1 Actions

		kN/m^2	
Permanent:			
Self-weight 0.175 × 25		= 4.4	EC1-1-1: Table A1
50 mm screed		= 1.0	
Finishes, services		= 0.5	
	Total	g_k = 5.9	
Variable:			
Offices, general use B1		= 2.5	EC1-1-1: Tables 6.1, 6.2 & NA
Movable partitions @ 2.0 kN/m		= 0.8	
	Total	q_k = 3.3	EC1-1-1: 6.3.12(8)

3.1.2 Cover

Nominal cover, c_{nom}: — **Exp. (4.1)**

$c_{nom} = c_{min} + \Delta c_{dev}$

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

where — **Cl. 4.4.1.2(3)**

$c_{min,b}$ = minimum cover due to bond = diameter of bar

Assume 12 mm main bars.

$c_{min,dur}$ = minimum cover due to environmental conditions — **Table 4.1.**

Assuming XCI and using C30/37 concrete, — BS 8500-1:

$c_{min,dur}$ = 15 mm — Table A4.

Δc_{dev} = allowance in design for deviation. — **Cl. 4.4.1.2(3)**

Assuming no measurement of cover,

Δc_{dev} = 10 mm

$\therefore c_{nom}$ = 15 + 10 = 25 mm

Fire:
Check adequacy of section for 1 hour fire resistance (i.e. REI 60).
Thickness, $h_{s,min}$ = 80 mm cf. 175 mm proposed ∴ OK
Axis distance, a_{min} = 20 mm cf. 25 + ϕ/2 = 31 i.e. not critical ∴ OK
∴ choose c_{nom} = 25 mm

EC2-1-2: 4.1(1), 5.1(1) & Table 5.8

3.1.3 Load combination (and arrangement)

Ultimate load, *n*:
By inspection, BS EN 1990 Exp. (6.10b) governs
∴ $n = 1.25 \times 5.9 + 1.5 \times 3.3 = 12.3$ kN/m²

Fig. 2.5
EC0: Exp. (6.10b)

3.1.4 Analysis

Design moment:
$M_{Ed} = 12.3 \times 4.8^2/8 = 35.4$ kNm

Shear force:
$V = 12.3 \times 4.8/2 = 29.5$ kN/m

3.1.5 Flexural design

Effective depth:
$d = 175 - 25 - 12/2 = 144$ mm

Flexure in span:
$K = M_{Ed}/bd^2f_{ck} = 35.4 \times 10^6/(1000 \times 144^2 \times 30) = 0.057$
$z/d = 0.95$
$z = 0.95 \times 144 = 137$ mm
$A_s = M_{Ed}/f_{yd}z = 35.4 \times 10^6/(137 \times 500/1.15) = 594$ mm²/m
($\rho = 0.41\%$)

Try H12 @ 175 B1 (645 mm²/m)

Fig. 3.5
Appendix A1
Table C5

3.1.6 Deflection

Check span-to-effective-depth ratio.
Basic span-to-effective-depth ratio for $\rho = 0.41\% = 20$
$A_{s,prov}/A_{s,req} = 645/599 = 1.08$
Max. span = $20 \times 1.08 \times 144 = 3110$ mm i.e. < 4800 mm ∴ no good

Appendix B
Table 7.4N & NA
Exp. (7.17)

Consider in more detail:
Allowable $l/d = N \times K \times F1 \times F2 \times F3$
where

$N = 25.6$ ($\rho = 0.41\%$, $f_{ck} = 30$ MPa)
$K = 1.0$ (simply supported)
$F1 = 1.0$ ($b_{eff}/b_w = 1.0$)
$F2 = 1.0$ (span < 7.0 m)
$F3 = 310/\sigma_s \leq 1.5$

Cl. 7.4.2,
Appendix C7,
Tables C10-C13

where‡

$\sigma_s = \sigma_{su} (A_{s,req}/A_{s,prov})\ 1/\delta$ — **Cl. 7.4.2, Exp. (7.17) Table 7.4N, & NA Table NA.5: Note 5**

where

$\sigma_{su} \approx 242$ MPa (From Figure C3 and — Figure C3

$g_k/q_k = 1.79$, $\psi_2 = 0.3$, $\gamma_G = 1.25$)

δ = redistribution ratio = 1.0

$\therefore\ \sigma_s \approx 242 \times 594/645 = 222$

$\therefore$ F3 = 310/222 = 1.40 ≤ 1.5 — Figure C3

$\therefore$ Allowable $l/d = 25.6 \times 1.40 = 35.8$

Actual $l/d = 4800/144 = 33.3$ $\therefore$ OK

Use H12 @ 175 B1 (645 mm^2/m)

3.1.7 Shear

By inspection, OK

However, if considered critical:

V = 29.5 kN/m as before

V_{Ed} = 29.5 – 0.14 × 12.3 = 27.8 kN/m — **Cl. 6.2.1(8)**

v_{Ed} = 27.8 × 10^3/144 × 10^3 = 0.19 MPa — **Cl. 6.2.2(1);**

$v_{Rd,c}$ = 0.53 MPa — Table C6

$\therefore$ No shear reinforcement required

3.1.8 Summary of design

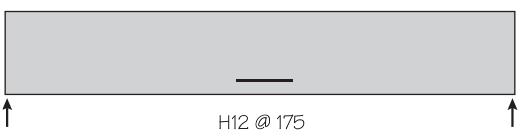


Figure 3.2 Simply supported slab: summary

3.1.9 Detailing checks

It is presumed that the detailer would take the design summarised above and detail the slab to normal best practice, e.g. to SMDSC[9] or to *How to design concrete structures using Eurocode 2*,[8] Chapter 10, *Detailing*. This would usually include dimensioning and detailing curtailment, laps, U-bars and also undertaking the other checks detailed below. See also 3.2.10 detailing checks for a continuous one-way slab.

a) Minimum areas

Minimum area of reinforcement:

$A_{s,min} = 0.26\ (f_{ctm}/f_{yk})\ b_t d \geq 0.0013\ b_t d$ — **Cl. 9.3.1.1, 9.2.1.1**

where

b_t = width of tension zone — **Table 3.1**

$f_{ctm} = 0.30 \times f_{ck}^{0.666}$

‡ See Appendix B1.5

$A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 1000 \times 144/500 = 216\ mm^2/m$

($\rho = 0.15\%$)

∴ H12 @ 175 B1 OK

Crack control: — Table 7.2N & NA

OK by inspection.

Maximum spacing of bars: — Cl. 9.3.1.1.(3)

< $3h$ < 400 mm — OK

Secondary reinforcement:

20% $A_{s,req} = 0.2 \times 645 = 129\ mm^2/m$

Use H10 @ 350 (224) B2 — Cl. 9.3.1.1.(2)

Edges: effects of assuming partial fixity along edge

Top steel required = 0.25 × 594 = 149 mm^2/m — Cl. 9.3.1.2.(2)

Use H10 @ 350 (224) T2 B2 as U-bars extending 960 mm into slab§

b) **Curtailment**

Curtailment main bars:

Curtail main bars 50 mm from or at face of support. — SMDSC[9]: Fig. 6.4; How to[8]: Detailing

At supports:

50% of A_s to be anchored from face of support. — Cl. 9.3.1.2.(1)

Use H12 @ 350 B1 T1 U-bars

In accordance with SMDSC[9] detail MS3 lap U-bars 500 mm with main steel, curtail T1 leg of U-bar 0.1l (= say 500 mm) from face of support.

§ A free unsupported edge is required to use 'longitudinal and transverse reinforcement' generally using U-bars with legs at least 2h long. For slabs 150 mm deep or greater, SMDSC[9] standard detail recommends U-bars lapping 500 mm with bottom steel and extending 0.1l top into span. — Cl. 9.3.1.4.(1)

3.2 Continuous one-way solid slab

This calculation is intended to show in detail the provisions of designing a slab to Eurocode 2 using essentially the same slab as used in Example 3.1.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Continuous one-way solid slab	Calculated by: chg	Job no.: CCIP – 041
		Checked by: web	Sheet no.: 1
		Client: TCC	Date: Oct 09

A 175 mm thick continuous slab is required to support screed, finishes, an office variable action of 2.5 kN/m^2 and demountable partitions (@ 2 kN/m). The slab is supported on 200 mm wide load-bearing block walls at 6000 mm centres. f_{ck} = 30, f_{yk} = 500 and the design life is 50 years. A fire resistance of 1 hour is required.

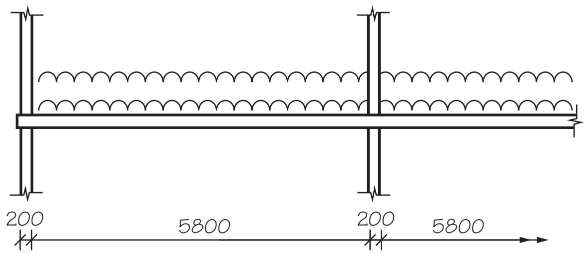


Figure 3.3 Continuous solid slab

3.2.1 Actions

kN/m^2

Permanent:

As Section 3.1.1 g_k = 5.9

Variable:

As Section 3.1.1 q_k = 3.3 — EC1-1-1: 6.3.1.2(8)

3.2.2 Cover

Nominal cover, c_{nom}:

As Section 3.1.2 c_{nom} = 25 mm

3.2.3 Load combination (and arrangement)

Ultimate action (load):

As Section 3.1.3, BS EN 1990 Exp. (6.10b) governs — Fig. 2.5 EC0: Exp. (6.10b)

∴ $n = 1.25 \times 5.9 + 1.5 \times 3.3 = 12.3$ kN/m^2

3.2.4 Analysis

Clear span, l_n = 5800 mm

a_1 = min[h/2; t/2] = min[175/2; 200/2] = 87.5 mm — Cl. 5.3.2.2(1)

a_2 = min[h/2; t/2] = min[175/2; 200/2] = 87.5 mm

l_{eff} = 5975 mm

Bending moment:

End span	$M_{Ed} = 0.086 \times 12.3 \times 5.975^2$	$= 37.8$ kNm/m	**Cl. 5.1.1(7)** Table C2
1st internal support	$M_{Ed} = 0.086 \times 12.3 \times 5.975^2$	$= 37.8$ kNm/m	
Internal spans and supports	$M_{Ed} = 0.063 \times 12.3 \times 5.975^2$	$= 27.7$ kNm/m	

Shear:

End support	$V_{Ed} = 0.40 \times 12.3 \times 5.975$	$= 29.4$ kN/m
1st interior support	$V_{Ed} = 0.60 \times 12.3 \times 5.975$	$= 44.1$ kN/m

3.2.5 Flexural design: span

a) End span (and 1st internal support)

Effective depth, d:

$d = h - c_{nom} - \phi/2$

$= 175 - 25 - 12/2 = 144$ mm

Relative flexural stress, K:

$K = M_{Ed}/bd^2f_{ck} = 37.8 \times 10^6/1000 \times 144^2 \times 30 = 0.061$

$K' = 0.207$ — Appendix A1

or restricting x/d to 0.45

$K' = 0.168$

∴ by inspection, section is under-reinforced

(i.e. no compression reinforcement required).

Lever arm, z: — **Fig. 3.5** Appendix A1

$z = (d/2)\,[1 + (1 - 3.53K)^{0.5}] \leq 0.95d$‡

$= (144/2)\,[1 + (1 - 3.53 \times 0.061)^{0.5}] = 0.945d = 136$ mm

Area of steel, A_s:

$A_s = M_{Ed}/f_{yd}z$

$= 37.8 \times 10^6/(500/1.15 \times 136) = 639$ mm²/m

($\rho = 0.44\%$)

Try H12 @ 175 B1 (645 mm²/m)

b) Internal spans and supports

Lever arm, z: — **Fig. 3.5** Appendix A1

By inspection, $z = 0.95d = 0.95 \times 144 = 137$ mm

Area of steel, A_s:

$A_s = M_{Ed}/f_{yd}z$

$= 27.7 \times 10^6/(500/1.15 \times 137) = 465$ mm²/m

($\rho = 0.32\%$)

Try H12 @ 225 B1 (502 mm²/m)

‡ Designers may choose to use another form of this equation:
$z/d = 0.5 + (0.25 - 0.882K)^{0.5} \leq 0.95$

3.2.6 Deflection: end span

Check end span-to-effective-depth ratio.

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ — Appendix B

where

N = basic effective depth to span ratio:

$\rho = 0.44\%$

$\rho_0 = f_{ck}^{0.5} \times 10^{-3} = 0.55\%$ ∴ use Exp. (7.16a) — **Cl. 7.4.2(2)**

$N = 11 + 1.5 f_{ck}^{0.5}\, \rho_0/\rho + 3.2 f_{ck}^{0.5}\, (\rho_0/\rho - 1)^{1.5}$ — **Exp. (7.16a)**

$= 11 + 1.5 \times 30^{0.5} \times 0.55/0.44 + 3.2 \times 30^{0.5}\, (0.55/0.44 - 1)^{1.5}$

$= 11.0 + 10.3 + 2.2 = 23.5$

K = structural system factor — **Cl. 7.4.2**

= 1.3 (end span of continuous slab)

F1 = flanged section factor — **Cl. 7.4.2**

$= 1.0\ (b_{eff}/b_w = 1.0)$

F2 = factor for long spans associated with brittle partitions — **Cl. 7.4.2**

= 1.0 (span < 7.0 m)

$F3 = 310/\ \sigma_s \le 1.5$ — **Cl. 7.4.2, Exp. (7.17) Table 7.4N & NA, Table NA.5: Note 5**

where[‡]

$\sigma_s = (f_{yk}/\ \gamma_S)\ (A_{s,req}/A_{s,prov})\ (\text{SLS loads/ULS loads}\ (1/\delta)$ — **Exp. (7.17)**

$= f_{yd} \times (A_{s,req}/A_{s,prov}) \times (g_k + \psi_2\, q_k)/(\gamma_G g_k + \gamma_Q q_k)\ (1/\delta)$ — ECO: A1.2.2

$= (500/1.15) \times (639/645) \times [(5.9 + 0.3 \times 3.3)/12.3] \times 1.08$[§] — Table C14

$= 434.8 \times 0.99 \times 0.56 \times 1.08 = 260$ MPa

$F3 = 310/260 = 1.19$

Note: $A_{s,prov}/A_{s,req} \le 1.50$ — **Table 7.4N & NA, Table NA.5: Note 5**

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

$= 23.5 \times 1.3 \times 1.0 \times 1.19$

$= 36.4$

Max. span = 36.4 × 144 = 5675 mm, i.e. < 5795 mm ∴ No good

Try increasing reinforcement to H12 @ 150 B1 (754 mm²/m)

$\sigma_s = 434.8 \times 639/754 \times 0.56 \times 1.08 = 223$

$F3 = 310/223 = 1.39$

Allowable $l/d = 23.5 \times 1.3 \times 1.0 \times 1.39$

$= 42.5$

‡ See Appendix B1.5

§ The use of Table C3 implies certain amounts of redistribution, which are defined in Table C14.

Max. span = 42.5 × 144 = 6120 mm, i.e. > 5795 mm OK

∴ H12 @ 150 B1 (754 mm^2/m) OK

3.2.7 Deflection: internal span

Check internal span-to-effective-depth ratio.

Allowable l/d = N × K × F1 × F2 × F3

where

N = basic effective depth to span ratio:

ρ = 0.32%

$\rho_0 = f_{ck}^{0.5} \times 10^{-3} = 0.55\%$ ∴ use Exp. (7.16a) — Cl. 7.4.2(2)

$N = 11 + 1.5 f_{ck}^{0.5}\, \rho_0/\rho + 3.2 f_{ck}^{0.5}\, (\rho_0/\rho - 1)^{1.5}$ — Exp. (7.16a)

$= 11 + 1.5 \times 30^{0.5} \times 0.55/0.32 + 3.2 \times 30^{0.5}\, (0.55/0.32 - 1)^{1.5}$

$= 11.0 + 14.1 + 10.7 = 35.8$

K = structural system factor

= 1.5 (interior span of continuous slab) — Cl. 7.4.2

F1 = flanged section factor

= 1.0 (b_{eff}/b_w = 1.0) — Cl. 7.4.2

F2 = factor for long spans associated with brittle partitions

= 1.0 (span < 7.0 m) — Cl. 7.4.2

F3 = 310/ σ_s ≤ 1.5 — Cl. 7.4.2, Exp. (7.17), Table 7.4N & NA, Table NA.5 Note 5.

where

$\sigma_s = f_{yd} \times (A_{s,req}/A_{s,prov}) \times (g_k + \psi_2 q_k)/(\gamma_G g_k + \gamma_Q q_k)\ (1/\delta)$ — Exp. (7.17), ECO: A1.2.2

$= (500/1.15) \times (465/502) \times [(5.9 + 0.3 \times 3.3)/12.3] \times 1.03$ — Table C14

$= 434.8 \times 0.93 \times 0.56 \times 1.03 = 233$ MPa

F3 = 310/233 = 1.33

Allowable l/d = N × K × F1 × F2 × F3

= 35.8 × 1.5 × 1.0 × 1.33

= 71.4

Max. span = 71.4 × 144 = 10280 mm i.e. > 5795 mm OK

Use H12 @ 225 B1 (502 mm^2/m) in internal spans

3.2.8 Shear

Design shear force, V_{Ed}:

At d from face of end support,

V_{Ed} = 29.4 – (0.144 + 0.0875) × 12.3 = 26.6 kN/m — Cl. 6.2.1(8)

At d from face of 1st interior support,

V_{Ed} = 44.1 – (0.144 + 0.0875) × 12.3 = 41.3 kN/m

Shear resistance, $V_{Rd,c}$:

$V_{Rd,c} = (0.18/\gamma_C)k(100\,\rho_l\, f_{ck})^{0.333}\, b_w d \geq 0.0035 k^{1.5} f_{ck}^{0.5} b_w d$ — Cl. 6.2.2(1)

where

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

$k = 2.0$

$\rho_l = A_{sl}/bd$

Assuming 50% curtailment (at end support)

$= 50\% \times 754/(144 \times 1000) = 0.26\%$

$V_{Rd,c} = (0.18/1.5) \times 2.0 \times (100 \times 0.26/100 \times 30)^{0.33} \times 1000 \times 144$

$= 0.12 \times 2 \times 1.97 \times 1000 \times 144$

$= 0.47 \times 1000 \times 144 = 68.1$ kN/m

But $V_{Rd,cmin} = 0.035k^{1.5} f_{ck}^{0.5} b_w d$

where

$k = 1 + (200/d)^{0.5} \leq 2.0$; as before $k = 2.0$

$V_{Rd,cmin} = 0.035 \times 2^{1.5} \times 30^{0.5} \times 1000 \times 144$

$= 0.54 \times 1000 \times 144 = 77.6$ kN/m

$\therefore V_{Rd,c} = 77.6$ kN/m

∴ OK, no shear reinforcement required at end or 1st internal supports

∴ H12 @ 150 B1 & H12 @ 175 T1 OK

By inspection, shear at other internal supports OK.

3.2.9 Summary of design

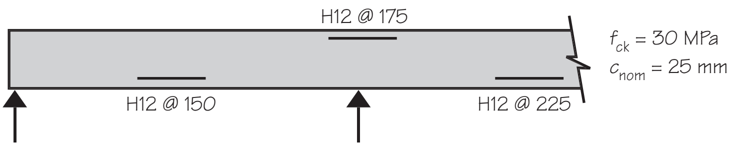


Figure 3.4 Continuous solid slab: design summary

Commentary

It is usually presumed that the detailer would take the design summarised above together with the general arrangement illustrated in Figure 3.3 and detail the slab to normal best practice. The detailer's responsibilities, standards and timescales should be clearly defined but it would be usual for the detailer to draw and schedule not only the designed reinforcement but all the reinforcement required to provide a compliant and buildable solution. The work would usually include checking the following aspects and providing appropriate detailing :

- Minimum areas
- Curtailment lengths
- Anchorages

- Laps
- U-bars
- Rationalisation
- Critical dimensions
- Details and sections

The determination of minimum reinforcement areas, curtailment lengths, anchorages and laps using the principles in Eurocode 2 is shown in detail in the following calculations. In practice these would be determined from published tables of data or by using reference texts[8, 9]. Nonetheless the designer should check the drawing for design intent and compliance with standards. It is therefore necessary for the designer to understand and agree the principles of the detailing used.

3.2.10 Detailing checks

a) Minimum areas

Minimum area of longitudinal tension (flexural) reinforcement — Cl. 9.3.1.1, 9.2.1.1

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

where

b_t = width of tension zone

$f_{ctm} = 0.30 \times f_{ck}{}^{0.667}$ — Table 3.1

$A_{s,min} = 0.26 \times 0.30 \times 30^{0.667} \times 1000 \times 144/500 = 216$ mm^2/m

($\rho = 0.15\%$)

∴ H12 @ 225 B1 OK

Secondary (transverse reinforcement)

Minimum 20% $A_{s,req}$ — Cl. 9.3.1.1(2)

20% $A_{s,req} = 0.2 \times 502 = 100$ mm^2/m

Consider $A_{s,min}$ to apply as before. — SMDSC[9]

$A_{s,min} = 216$ mm^2/m

Try H10 @ 350 B2 (224 mm^2/m)

Check edge.

Assuming partial fixity exists at edges, 25% of A_s is required to extend 0.2 × the length of the adjacent span. — Cl. 9.3.1.2(2)

$A_{s,req} = 25\% \times 639 = 160$ mm^2/m

$A_{s,min}$ as before = 216 mm^2/m — Cl. 9.3.1.1, 9.2.1.1

∴ Use H10 @ 350 (224 mm^2/m) U-bars at edges

Curtail 0.2 × 5975 = 1195 mm, say 1200 mm measured from face of support‡.	**Cl. 9.3.1.2(2)**
Maximum spacing of bars Maximum spacing of bars < 3h < 400 mm OK	**Cl. 9.3.1.1.(3)**
Crack control As slab < 200 mm, measures to control cracking are unnecessary.	**Cl. 7.3.3(1)**
However, as a check on end span: Loading is the main cause of cracking, ∴ use Table 7.2N or Table 7.3N for w_{max} = 0.4 mm and σ_s = 241 MPa (see deflection check). Max. bar size = 20 mm or max. spacing = 250 mm ∴ H12 @ 150 B1 OK.	**Cl. 7.3.3(2), 7.3.1.5** **Table 7.2N & interpolation, Table 7.3N & interpolation**
End supports: effects of partial fixity Assuming partial fixity exists at end supports, 15% of A_s is required to extend 0.2 × the length of the adjacent span. $A_{s,req}$ = 15% × 639 = 96 mm^2/m But, $A_{s,min}$ as before = 216 mm^2/m (ρ = 0.15%) One option would be to use bob bars, but choose to use U-bars Try H12 @ 450 (251 mm^2/m) U-bars at supports	**Cl. 9.3.1.2(2)** **Cl. 9.3.1.1, 9.2.1.1**
Curtail 0.2 × 5975 = say, 1200 mm measured from face of support.‡	**Cl. 9.3.1.2(2)**
b) **Curtailment** **i) End span, bottom reinforcement** Assuming end support to be simply supported, 50% of A_s should extend into the support. 50% × 639 = 320 mm^2/m Try H12 @ 300 (376 mm^2/m) at supports	**Cl. 9.3.1.2(1)**
In theory, 50% curtailment of reinforcement may take place a_l from where the moment of resistance of the section with the remaining 50% would be adequate to resist the applied bending moment. In practice, it is usual to determine the curtailment distance as being a_l from where $M_{Ed} = M_{Ed,max}/2$.	**Cl. 9.3.1.2(1) Note, 9.2.1.3 (2)**

‡Detail MS2 of SMDSC[9], suggests 50% of T1 legs of U-bars should extend 0.3l (= say 1800 mm) from face of support by placing U-bars alternately reversed.

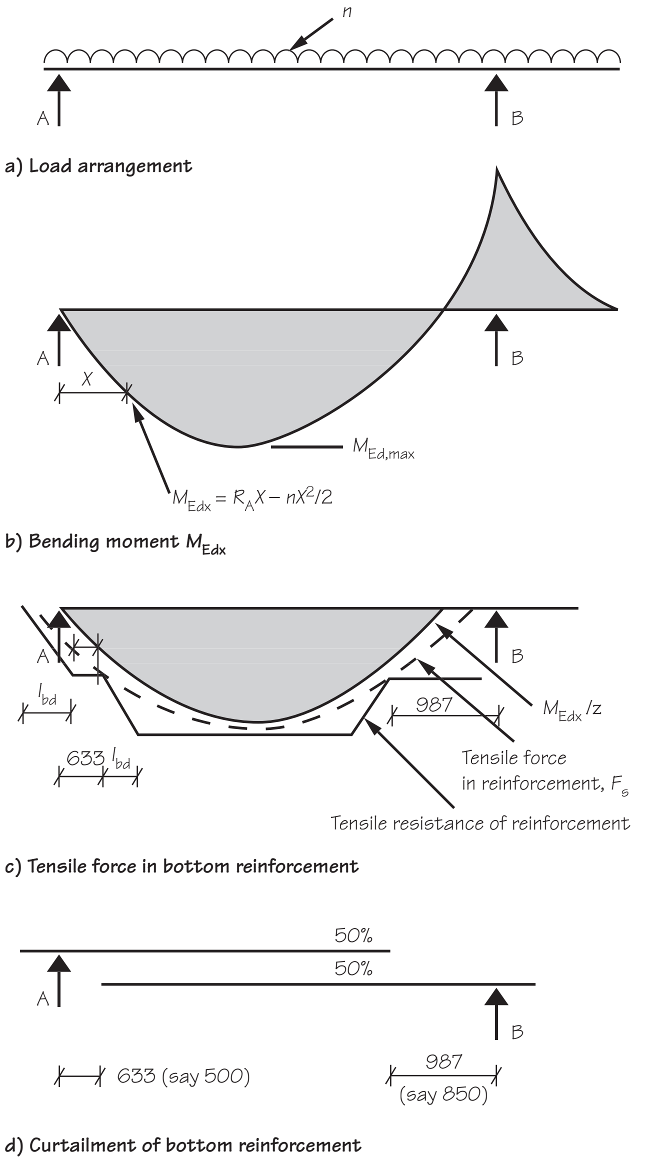


Figure 3.5 Curtailment of bottom reinforcement: actions, bending moments, forces in reinforcement and curtailment

Thus, for a single simply supported span supporting a UDL of n,

$M_{Ed,max} = 0.086nl^2$; $R_A = 0.4nl$

At distance, X, from end support, moment,

$M_{Ed}@X = R_A X - nX^2/2$

$\therefore$ when $M@X = M_{Ed,max}/2$:

$0.086nl^2/2 = 0.4nlX - nX^2/2$

Assuming $X = xl$ $0.043nl^2 = 0.4nlxl - nx^2l^2/2$ $0.043 = 0.4x - x^2/2$ $0 = 0.043 - 0.4x + x^2/2$ $x = 0.128$ or 0.672, say 0.13 and 0.66 ∴ at end support 50% moment occurs at 0.13 x span 0.13 × 5975 = 777 mm	
Shift rule: for slabs, a_l may be taken as d (= 144 mm), ∴ curtail to 50% of required reinforcement at 777 – 144 = 633 mm from centreline of support. Say 500 mm from face of support A	**Cl. 9.2.1.3(2), 6.2.2(5)**
∴ in end span at 1st internal support 50% moment occurs at 0.66 × span 0.66 × 5975 = 3944 mm	
Shift rule: for slabs a_l may be taken as d (= 144 mm), ∴ curtail to 50% of required reinforcement at 3944 + 144 = 4088 mm from support A or 5975 – 4088 = 987 mm from centreline of support B. Say 850 mm from face of support B	**Cl. 9.2.1.3(2), 6.2.2(5)**
ii) 1st interior support, top reinforcement Presuming 50% curtailment of reinforcement is required this may take place a_l from where the moment of resistance of the section with the remaining 50% would be adequate. However, it is usual to determine the curtailment distance as being a_l from where $M_{Ed} = M_{Ed,max}/2$. Thus, for the 1st interior support supporting a UDL of n, $M_{Ed,maxT} = 0.086nl^2$; $R_B = 0.6nl$ At distance Y from end support, moment, $M_{Ed}@Y = M_{Ed,maxT} - R_AY + nY^2/2$ ∴ when $M@Y = M_{Ed,maxT}/2$ $0.086nl^2/2 = 0.086nl^2 - 0.6nlY + nY^2/2$ Assuming $Y = yl$ $0.043nl^2 = 0.086nl^2 - 0.6nlyl + ny^2l^2/2$ $0 = 0.043 - 0.6y + y^2/2$ $y = 0.077$ (or 1.122), say 0.08 ∴ at end support 50% moment occurs at 0.08 × span 0.08 × 5975 = 478 mm	**Cl. 9.3.1.2(1) Note, 9.2.1.3(2)**
Shift rule: for slabs, a_l may be taken as d 144 mm ∴ curtail to 50% of required reinforcement at 478 + 144 = 622 mm from centreline of support. 50% of reinforcement may be curtailed at, say, 600 mm from either face of support B	**Cl. 9.2.1.3(2), 6.2.2(5)**

100% curtailment may take place a_l from where there is no hogging moment. Thus,
when $M@Y = M_{Ed,maxT}/2$
$0 = 0.086nl^2 - 0.6nlY + nY^2/2$

Assuming $Y = yl$
$0 = 0.086 - 0.6y + y^2/2$
$y = 0.166$ (or 1.034), say 0.17
∴ at end support 50% moment occurs at 0.17 × span
0.17 × 5975 = 1016 mm
Shift rule: for slabs, a_l may be taken as d
∴ curtail to 100% of required reinforcement at 1016 + 144
= 1160 mm from centreline of support.

100% of reinforcement may be curtailed at, say, 1100 mm from either face of support B.

iii) Support B bottom steel at support

At the support 25% of span steel required — Cl. 9.3.1.1(4), 9.2.1.5(1), 9.2.1.4(1)

0.25 × 639 = 160 mm²
$A_{s,min}$ as before = 216 mm²/m — Cl. 9.3.1.1, 9.2.1.1

For convenience use H12 @ 300 B1 (376 mm²/m)

c) Anchorage at end support

As simply supported, 50% of A_s should extend into the support. — Cl. 9.2.1.2(1) & Note, 9.2.1.4(2)
This 50% of A_s should be anchored to resist a force of
$F_E = V_{Ed} \times a_l/z$ — Exp. (9.3)
where
V_{Ed} = the absolute value of the shear force
a_l = d, where the slab is not reinforced for shear — Cl. 9.2.1.3(2)
z = lever arm of internal forces
$F_E = 29.4 \times d/0.95$‡ d = 30.9 kN/m
Anchorage length, l_{bd}: — Cl. 8.4.4
$l_{bd} = \alpha l_{b,rqd} \geq l_{b,min}$ — Exp. (8.4)
where
α = conservatively 1.0
$l_{b,rqd}$ = basic anchorage length required
= $(\phi/4)(\sigma_{sd}/f_{bd})$ — Exp. (8.3)
where
ϕ = diameter of the bar = 12 mm
σ_{sd} = design stress in the bar at the ultimate limit state
= $F_E/A_{s,prov}$
= 30.9 × 1000/376 = 81.5 MPa

‡ Maximum z = 0.947 at mid-span and greater towards support.

f_{bd} = ultimate bond stress = $2.25\ \eta_1\ \eta_2\ f_{ct,d}$	Cl. 8.4.2(2)
where η_1 = 1.0 for 'good' bond conditions and 0.7 for all other conditions = 1.0 η_2 = 1.0 for bar diameter ≤ 32 mm $f_{ct,d}$ = design tensile strength = $\alpha_{ct}\ f_{ct,k}/\ \gamma_C$. For f_{ck} = 30 MPa = 1.0 × 2.0/1.5 = 1.33 MPa	Cl. 3.1.6(2) & NA, Tables 3.1 & 2.1N
$\therefore f_{bd}$ = 2.25 × 1.33 = 3.0 MPa $l_{b,rqd}$ = (12/4) (81.5/1.33) = 183 mm $l_{b,min}$ = max(10*d*, 100 mm) = 120 mm l_{bd} = 183 mm measured from face of support By inspection, using U-bars, OK	Exp. (8.6) Fig. 9.3
d) Laps Lap H12 @ 300 U-bars with H12 @ 150 straights. Tension lap, $l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd}\ \alpha\ l_{0min}$	Exp. (8.10)
where α_1 = 1.0 (straight bars) $\alpha_2 = 1 - 0.15\ (c_d - \phi)/\phi$	Table 8.2
where c_d = min(pitch, side cover or cover) = 25 mm ϕ = bar diameter = 12 mm	Fig. 8.4
α_2 = 0.84 α_3 = 1.0 (no confinement by reinforcement) α_5 = 1.0 (no confinement by pressure)	Table 8.2
α_6 = 1.5 $l_{b,rqd} = (\phi/4)\ \sigma_{sd}/f_{bd}$	Table 8.3 Exp. (8.3)
where σ_{sd} = the design stress at ULS at the position from where the anchorage is measured. Assuming lap starts 500 mm from face of support (587.5 mm from centreline of support): M_{Ed} = 29.5 × 0.59 – 12.3 × 0.59^2/2 = 15.2 kNm σ_{sd} = $M_{Ed}/(A_s z)$ = 15.2 × 10^6/(376 × 144/0.95) = 267 MPa	
f_{bd} = ultimate bond stress = $2.25\ \eta_1\ \eta_2\ f_{ct,d}$	Cl. 8.4.2(2)

where

η_1 = 1.0 for 'good' conditions

η_2 = 1.0 for ϕ < 32 mm

$f_{ct,d} = \alpha_{ct} f_{ct,k}/\gamma_C$

where

α_{ct} = 1.0 — Cl. 3.1.6 (2) & NA

$f_{ct,k}$ = 2.0 — Table 3.1

γ_C = 1.5 — Table 2.1N & NA

$\therefore f_{bd}$ = 2.25 × 2.0/1.5 = 3.0 MPa

$l_{b,rqd} = (\phi/4)\ \sigma_{sd}/f_{bd}$

= (12/4) × (267/3) = 267 mm

$l_{0min}\ b = \max[0.3\ \alpha_6\ l_{b,rqd};\ 15\phi/\ 200\ \text{mm}]$ — Exp. 8.6

= max[0.3 × 1.5 × 229; 15 × 12; 200]

= max[124; 180; 200] = 200 mm

$\therefore l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0min}$

= 1.0 × 0.84 × 1.0 × 1.0 × 1.5 × 329 ≥ 200 = 414 mm

But good practice suggests minimum lap of max[tension lap; 500]

∴ lap with bottom reinforcement = 500 mm starting 500 from face of support. — SMDSC[9]: MS2

3.2.11 Summary of reinforcement details

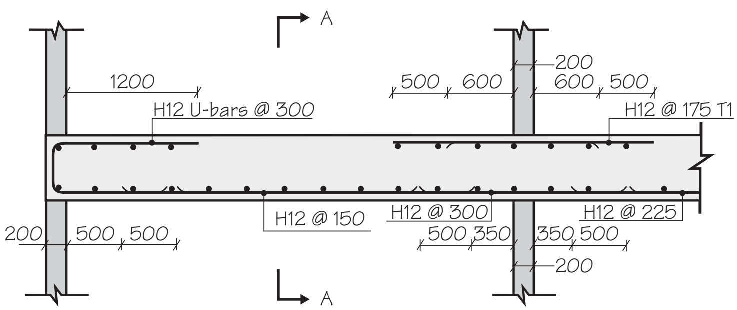


Figure 3.6 Continuous solid slab: reinforcement details

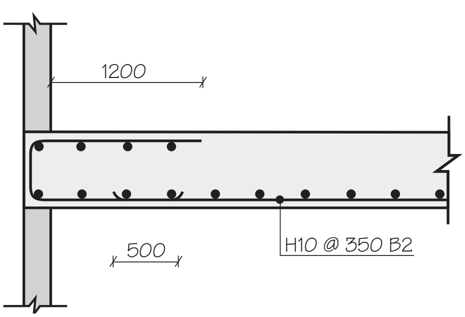


Figure 3.7 Section A–A showing reinforcement details at edge

3.3 Continuous ribbed slab

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details **Continuous ribbed slab**	Calculated by chg	Job no. CCIP – 041
		Checked by web	Sheet no. 1
		Client TCC	Date Oct 09

This continuous 300 mm deep ribbed slab has spans of 7.5 m, 9.0 m and 7.5 m and is required for an office to support a variable action of 5 kN/m². It is supported on wide beams that are the same depth as the slab designed in Section 4.3. One hour fire resistance is required: internal environment. The ribs are 150 mm wide @ 900 mm centres. Links are required in span to facilitate prefabrication of reinforcement. Assume that partitions are liable to be damaged by excessive deflections. In order to reduce deformations yet maintain a shallow profile use f_{ck} = 35 MPa and f_{yk} = 500 MPa.

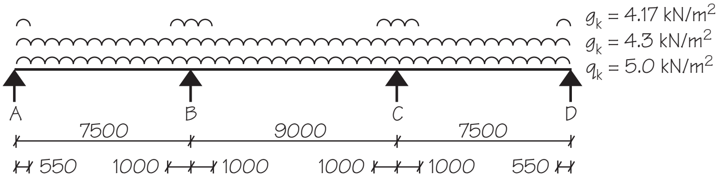


Figure 3.8 Continuous ribbed slab example

Notes on ribbed slab design

There are various established methods for analysing ribbed slabs and dealing with the solid areas:

- Using UDLs simplifies the analysis and remains popular. One method is to ignore the weight of the solid part of the slab in the analysis of the ribbed slab. (The weight of the solid area is then added to the loads on the supporting beam). This ignores the minor effect the solid areas have on bending in the ribbed slab.
- Alternatively the weight of the solid part of the slab is spread as a UDL over the whole span. This is conservative both in terms of moment and shears at solid/shear interfaces but underestimates hogging in internal spans.
- The advent of computer analysis has made analysis using patch loads more viable and the resulting analysis more accurate.
- The ribbed part of the slab may be designed to span between solid areas. (The ribs span *d*/2 into the solid areas, which are assumed to act as beams in the orthogonal direction.) However, having to accommodate torsions induced in supporting beams and columns usually makes it simpler to design from centreline of support to centreline of support.
- Analysis programs can cope with the change of section and therefore change of stiffness along the length of the slab. Moments would be attracted to the stiffer, solid parts at supports. However, the difference in stiffness between the ribbed and the solid parts is generally ignored.

In line with good practice analysis, this example is carried out using centreline of support to centreline of support and patch loads‡. Constant stiffness along the length of the slab has been assumed.

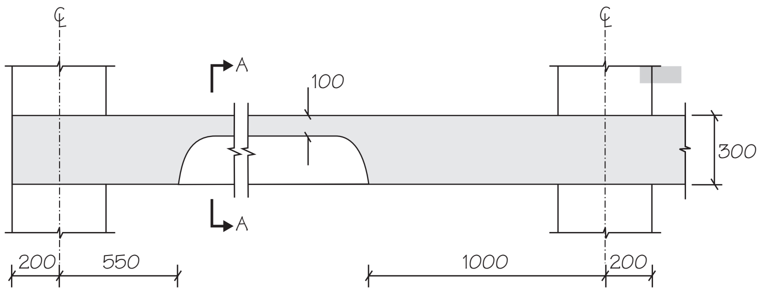


Figure 3.9 Long section through slab

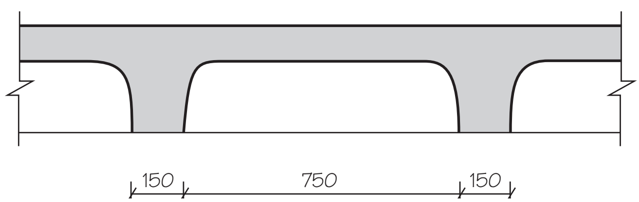


Figure 3.10 Section A–A: section through ribbed slab

3.3.1 Actions

Permanent: UDL kN/m²

			kN/m²		
Self-weight:					
Rib	0.15 × 0.2 × 25/0.9	=	0.833		
Slope	2 × (1/2) × 0.2/10 × 0.2 × 25/0.9	=	0.112		
Slab	0.1 × 2.5	=	2.500		
Cross rib	0.19 × 0.71 × 0.2 × 25/(0.9 × 7.5)	=	0.100		
Total self-weight		=	3.545	≈	3.55
Ceiling				=	0.15
Services				=	0.30
Raised floor				=	0.30
Total permanent actions				g_k =	4.30

‡ In this case, assuming the patch load analysis is accurate, taking the weight of solid area to be spread over the whole span would overestimate span and support moments by 6–8% and shears at the solid/rib interface by 8–9%. Ignoring the weight of the solid area in the analysis of this ribbed slab would lead to underestimates of span moments by 1%, support moments by 3% and no difference in the estimation of shear at the solid shear interface. The latter may be the preferred option.

Permanent: patch load

Extra over solid in beam area as patch load

$(0.2 \times 25 - 0.833) = 4.167$ $\quad g_k \approx 4.17$

Variable

Imposed $= 4.00$*

Allowance for partitions $= 1.00$*

Total variable action $g_k = 5.00$

3.3.2 Cover

Nominal cover, c_{nom}:

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

$c_{min} = \max(c_{min,b}; c_{min,dur})$

where

$c_{min,b}$ = minimum cover due to bond — Cl. 4.4.1.2(3)

= diameter of bar.

Assume 20 mm main bars and 8 mm links

$c_{min,dur}$ = minimum cover due to environmental conditions.

Assuming XC1 and C30/37 concrete, $c_{min,dur} = 15$ mm — Table 4.1. BS 8500-1: Table A4

Δc_{dev} = allowance in design for deviation. Assuming no measurement of cover $\Delta c_{dev} = 10$ mm — Cl. 4.4.1.2(3)

$\therefore c_{nom}$ = 20 + 10 to main bars or

= 15 + 10 to links ∴ critical

Fire: — EC2-1-2: 5.7.5(1)

Check adequacy of section for REI 60.

Minimum slab thickness, $h_s = 80$ mm OK — EC2-1-2: Table 5.8

Axis distance required

Minimum rib width $b_{min} = 120$ mm with $a = 25$ mm — EC2-1-2: Table 5.6

or $b_{min} = 200$ mm with $a = 12$ mm

∴ at 150 mm wide (min.) $a = 20$ mm

By inspection, not critical.

Use 25 mm nominal cover to links

3.3.3 Load combination and arrangement

Ultimate load, n:

By inspection, Exp. (6.10b) is critical — Fig. 2.5 ECO: Exp. (6.10b)

$n_{slab} = 1.25 \times 4.30 + 1.5 \times 5.0 = 13.38$ kN/m^2

$n_{solid\ areas} = 1.25 \times (4.30 + 4.17) + 1.5 \times 5.0 = 18.59$ kN/m^2

*Client requirements. See also BS EN 1991–1–1, Tables 6.1, 6.2, Cl. 6.3.2.1(8) & NA.

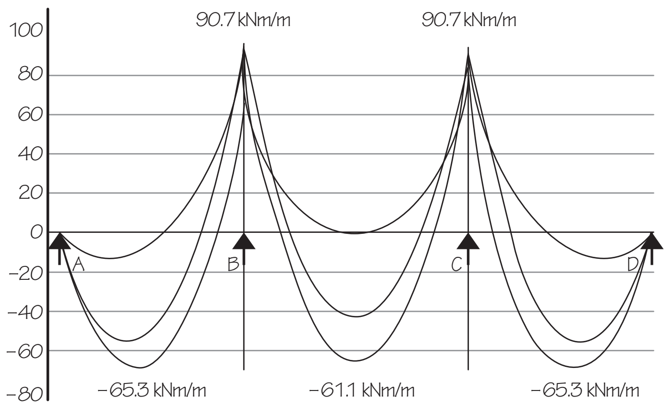
Arrangement:
Choose to use all-and-alternate-spans-loaded.

Cl. 5.1.3(1) & NA option b

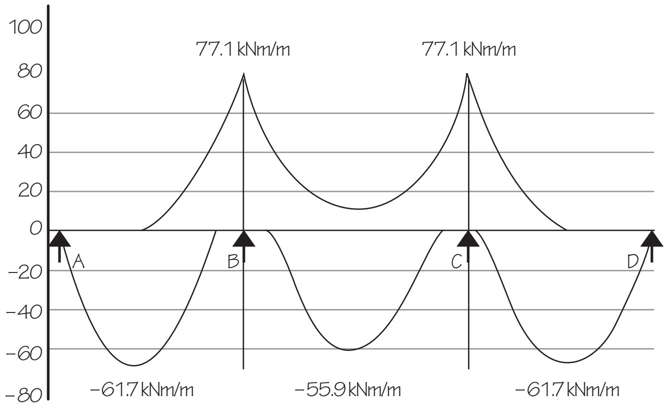
3.3.4 Analysis

Analysis by computer, includes 15% redistribution at support and none in the span.§

ECO: A1.2.2 & NA, 5.3.1 (6)



a) Elastic moments



b) Redistributed envelope

Figure 3.11 Bending moment diagrams

§ Note 1: A ribbed slab need not be treated as discrete elements provided rib spacing ≤ 1500 mm, depth of the rib ≤ 4 × its width, the flange is > 0.1 × distance between ribs and transverse ribs are provided at a clear spacing not exceeding 10 × overall depth of the slab.

Note 2: As 7.5 m < 85% of 9.0 m, coefficients presented in *Concise Eurocode 2*[5] are not applicable.

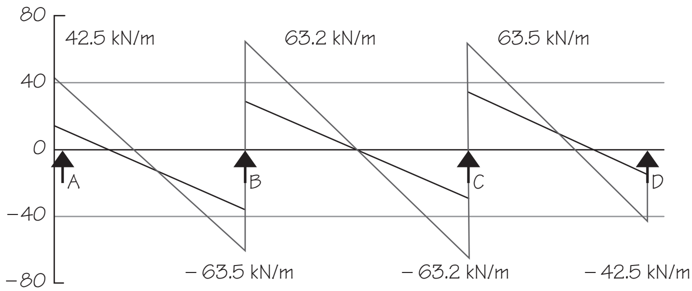


Figure 3.12 Redistributed shears, kN/m

At solid/rib interface:

AB @ 550 mm from A

M_{Ed} (sagging) = 20.4 kNm/m ≡ 18.3 kNm/rib

V_{Ed} = 32.5 kN/m ≡ 29.3 kN/rib

BA @1000 mm from B

M_{Ed} (hogging) = 47.1 kNm/m ≡ 42.4 kNm/rib

V_{Ed} = 45.4 kN/m ≡ 40.9 kN/rib

BC @ 1000 mm from B

M_{Ed} (hogging) = 43.0 kNm/m ≡ 38.7 kNm/rib

V_{Ed} = 45.1 kN/m ≡ 40.6 kN/rib

Symmetrical about centreline of BC.

3.3.5 Flexural design, span A–B

a) Span A–B: Flexure

M_{Ed} = 61.7 kNm/m

= 55.5 kNm/rib

$K = M_{Ed}/bd^2f_{ck}$

where

b = 900 mm

d = 300 – 25 – 8 – 20/2 = 257
assuming 8 mm link at H20 in span

f_{ck} = 35 MPa

$\therefore K = 55.5 \times 10^6/(900 \times 257^2 \times 35) = 0.027$

$K' = 0.207$ — Appendix A1

or restricting x/d to 0.45

$K' = 0.168$

$K \le K'$ ∴ section under-reinforced and no compression reinforcement required.

$z = (d/2)\,[1 + (1 - 3.53K)^{0.5}] \le 0.95d$	Appendix A1
$= (257/2)\,(1 + 0.951) \le 0.95 \times 257$	
$= 251 \le 244 \;\therefore\; z = 244$ mm	
But $z = d - 0.4x$	Appendix A1
$\therefore\; x = 2.5(d - z) = 2.5(257 - 244) = 33$ mm	
$\therefore$ By inspection, neutral axis is in flange	
$A_s = M_{Ed}/f_{yd}z$	
where	
$f_{yd} = 500/1.15 = 434.8$ MPa	
$= 55.5 \times 10^6/(434.8 \times 244) = 523$ mm^2/rib	
Try 2 no. H20/rib (628 mm^2/rib)	
b) Span A–B: Deflection	
Allowable $l/d = N \times K \times F1 \times F2 \times F3$	Appendix C7
where	
N = Basic l/d: check whether $\rho > \rho_0$ and whether to use Exp. (7.16a) or Exp. (7.16b)	**Cl. 7.4.2(2)**
$\rho_0 = f_{ck}^{\;0.5}/1000 = 35^{0.5}/1000 = 0.59\%$	
$\rho = A_s/A_c^{\ddagger} = A_{s,req}/[b_w d + (b_{eff} - b_w)h_f]$	PD 6687[6]
where	
b_w = min. width between tension and compression chords. At bottom assuming 1/10 slope to rib:	
$= 150 + 2 \times (25 + 8 + 20/2)/10$	
$= 159$ mm	
$\rho = 523/(159\;(\;257 + (900 - 159) \times 100)$	
$= 523/114963$	
$= 0.45\%$	
$\rho < \rho_0 \;\therefore$ use Exp. (7.16a)	
$N = 11 + 1.5f_{ck}^{\;0.5}\rho/\rho_0 + 3.2f_{ck}^{\;0.5}(\rho/\rho_0 - 1)^{1.5}]$	**Exp. (7.16a)**
$= 11 + 1.5 \times 35^{0.5} \times 0.055/0.045 + 3.2 \times 35^{0.5}\,(0.055/0.045 - 1)^{1.5}$	
$= [11 + 10.8 + 2.0] = 22.8$	
K = (end span) 1.3	**Table 7.4N & NA, Table NA.5: Note 5**
$F1 = (b_{eff}/b_w = 5.66)\;0.8$	
$F2 = 7.0/l_{eff} = 7.0/7.5 =$ (span > 7.0 m) 0.93	**Cl. 7.4.2(2)**
$F3 = 310/\sigma_s \le 1.5$	**Cl. 7.4.2, Exp. (7.17) & NA; Table NA.5**

‡ Section 2.18 of PD 6687 [6] suggests that ρ in T-beams should be based on the area of concrete above the centroid of the tension steel.

where[‡]

$\sigma_s = (f_{yk}/\gamma_S)\ (A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$

$= 434.8(523/628)\ [\ (4.30 + 0.3 \times 5.0)/13.38]$ $(65.3/61.7^{§})$

$= 434.8 \times 0.83 \times 0.43 \times 1.06$

$= 164$ MPa

F3 $= 310/\sigma_s$

$= 310/164 = 1.89^{\#}$ but ≤ 1.5, therefore say 1.50

∴ Permissible $l/d = 22.8 \times 1.3 \times 0.8 \times 0.93 \times 1.50 = 33.0$

Actual $l/d = 7500/257 = 29.2$ ∴ OK

Use 2 no. H20/rib (628 mm^2/rib)

c) Support A (and D): flexure (sagging) at solid/rib interface

Reinforcement at solid/rib interface needs to be designed for both moment and for additional tensile force due to shear (shift rule) — **Cl. 9.2.1.3.(2)**

$M_{Ed,max} = 18.3$ kNm/rib

$V_{Ed,max} = 29.3$ kNm/rib

At solid/rib interface

$A_s = M_{Ed}/f_{yd}z + \Delta F_{td}/f_{yd}$ — **Cl. 9.2.1.3.(2), Fig. 9.2**

where

$z = (d/2)\ [1 + (1 - 3.53K)^{0.5}] \leq 0.95d$

where

$K = M_{Ed}/bd^2f_{ck}$

where

$b = 900$ mm

$d = 300 - 25 - 8 - 25 - 20/2 = 232$

assuming 8 mm links and H25B in edge beam

$f_{ck} = 30$

$= 18.3 \times 10^6/(900 \times 232^2 \times 35) = 0.011$

---

‡ See Appendix B1.5

§ In analysis, 15% redistribution of support moments led to redistribution of span moments:
$\delta = 61.7/65.3 = 0.94$.

\# Both $A_{s,prov}/A_{s,req}$ and any adjustment to N obtained from Exp. (7.16a) or Exp. (7.16b) is restricted to 1.5 by Note 5 to Table NA.5 in the UK NA. Therefore, $310/\sigma_s$ is restricted to 1.5.

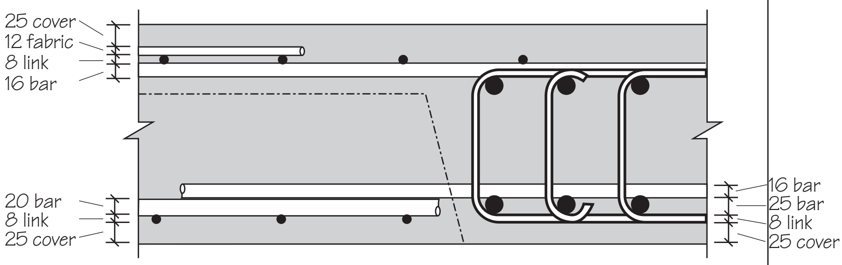


Figure 3.13 Section at solid/rib intersection

$\therefore z = (232/2)(1 + 0.980) \leq 0.95 \times 232$ — Appendix A1

$= 230 \leq 220 \therefore z = 220$ mm

$f_{yd} = 434.8$ MPa

$\Delta F_{td} = 0.5V_{Ed}(\cot\theta - \cot\alpha)$ — **Cl. 6.2.3(7), Exp. (6.18)**

where

θ = angle between the concrete compression strut and the beam axis. Assume $\cot\theta = 2.5$ (as a maximum) — **Cl. 6.2.3(1)** Appendix A2

α = angle between shear reinforcement and the beam axis. For vertical links, $\cot\alpha = 0$ — Appendix C, Table C6

$\Delta F_{td} = 1.25V_{Ed} = 1.25 \times 29.3 = 36.6$ kN — **Cl. 6.2.3(1)**

$A_s = 18.3 \times 10^6/(434.8 \times 220) + 36.6 \times 10^3/434.8$

$= 191 + 84\ \text{mm}^2 = 275\ \text{mm}^2$

$\therefore$ Try 1 no. H20 B in end supports*

d) Support B (and C) (at centreline of support)

$M_{Ed} = 77.1$ kNm/m

$= 69.4$ kNm/rib

$K = M_{Ed}/bd^2f_{ck}$

where

$d = 300 - 25\text{ cover} - 12\text{ fabric} - 8\text{ link} - 20/2$

$= 245$

$K = 69.4 \times 10^6/(900 \times 245^2 \times 35) = 0.037$

By inspection, $K \leq K'$

$z = (245/2)[1 + (1 - 3.53K)^{0.5}] \leq 0.95d$

$= (245/2)(1 + 0.932) < 0.95d$

$= 237$ mm

$A_s = M_{Ed}/f_{yd}z$

$= 69.4 \times 10^6/434.8 \times 237 = 673\ \text{mm}^2/\text{rib}$

* An alternative method would have been to calculate the reinforcement required to resist M_{Ed} at the shift distance, a_l, from the interface.

e) Support B (and C): flexure (hogging) at solid/rib interface

Reinforcement at solid/rib interface needs to be designed for both moment and for additional tensile force due to shear (shift rule). — Cl. 9.2.1.3.(2)

$M_{Ed,max}$ = 42.4 kNm/rib max.

$V_{Ed,max}$ = 40.9 kNm/rib max.

$A_s = M_{Ed}/f_{yd}z + \Delta F_{td}/f_{yd}$ — Cl. 9.2.1.3.(2)

where

$z = (245/2)\ [1 + (1 - 3.53\ K)^{0.5}] \leq 0{\cdot}95d$

where

$K = M_{Ed}/bd^2f_{ck}$

$= 42.4 \times 106/(150 \times 245^2 \times 35)$

$= 0.135$

Check $K \leq K'$

$K' = 0.168$ for $\delta = 0.85$ (i.e. 15% redistribution) — Appendix C, Table C4

∴ Section under-reinforced: no compression reinforcement required — Appendix A

∴ $z = (245/2)\ (1 + 0.723) \leq 232 = 211$ mm

f_{yd} = 434.8 MPa

$\Delta F_{td} = 0.5V_{Ed}\ (\cot\theta - \cot\alpha)$ — Cl. 6.2.3(7), Exp. (6.18)

where

θ = angle between the concrete compression strut and the beam axis. Assume $\cot\theta = 2.5$ (as a maximum) — Cl. 6.2.3(1) Appendix A2; Table C6

α = angle between shear reinforcement and the beam axis. For vertical links, $\cot\alpha = 0$ — Cl. 6.2.3(1)

$\Delta F_{td} = 1.25V_{Ed} = 1.25 \times 40.9 = 51.1$ kN

$A_s = 42.4 \times 10^6/(434.8 \times 211) + 51.1 \times 10^3/434.8$

$= 462 + 117\ \text{mm}^2 = 579\ \text{mm}^2\text{/rib}$

To be spread over b_{eff} where by inspection, $b_{eff} = 900$. — Cl. 9.2.1.2(2)

∴ Centre of support more critical (679 mm²/rib required). — Cl. 5.3.2.1(3)

Top steel may be spread across b_{eff} where — Cl. 9.2.1.2(2), 5.3.2

$b_{eff} = b_w + b_{eff1} + b_{eff2} \leq b$

$= b_w + 2 \times 0.1 \times 0.15 \times (l_1 + l_2)$

$= 150 + 0.03 \times (7500 + 9000) \leq 900$

$= 645$ mm

∴ Use 2 no. H16 above rib and 3 no. H12 between (741 mm²/rib) where 2 no. H16 and 2 no. H12 are within b_{eff}

3.3.6 Flexural design, span BC

a) Span B–C: Flexure

M_{Ed} = 55.9 kNm/m

= 50.3 kNm/rib

$K = M_{Ed}/bd^2 f_{ck}$
$= 50.3 \times 10^6/900 \times 257^2 \times 35$
$= 0.02$ i.e. $\leq K'$ (as before $K' = 0.168$)

By inspection,
$z = 0.95d = 0.95 \times 257 = 244$ mm

By inspection, neutral axis is in flange.

$A_s = M_{Ed}/f_{yd}z$
$= 50.3 \times 10^6/434.8 \times 244 = 474$ mm^2

Try 2 no. H20/rib (628 mm^2/rib)

b) Span B–C: Deflection

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ — Section C7

where

N = Basic l/d — **Cl. 7.4.2(2)**

$\rho = 474/(159$ ($\times 257 + (900 - 159) \times 100)$
$= 474/114963$
$= 0.41\%$

$\rho_0 = 0.59\%$ (for $f_{ck} = 30$)

$\therefore \rho < \rho_0$ use Exp. (7.16a)

$N = 11 + 1.5 f_{ck}^{0.5} \rho_0/\rho + 3.2 f_{ck}^{0.5} (\rho_0/\rho - 1)^{1.5}$ — **Exp. (7.16a)**
$= 11 + 1.5 \times 35^{0.5} \times 0.055/0.041 + 3.2 \times 35^{0.5} (0.055/0.041 - 1)^{1.5}$
$= 11 + 11.9 + 3.8 = 26.7$

K = (internal span) 1.5 — **Table 7.4N, & NA, Table NA.5: Note 5**

$F1 = (b_{eff}/b_w = 6.0)$ 0.8

$F2 = 7.0/l_{eff} = 7.0/9.0 =$ (span > 7.0 m) 0.77 — **Cl. 7.4.2(2)**

$F3 = 310/\sigma_s \leq 1.5$ — **Cl. 7.4.2, Exp. (7.17) & NA: Table NA.5**

where

$\sigma_s = (f_{yk}/\gamma_S)(A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$
$= 434.8 \times (474/628)$ [(4.30 + 0.3 × 5.0)/13.38](61.1/55.9)
$= 434.8 \times 0.75 \times 0.43 \times 1.09$
$= 153$ MPa

$F3 = 310/\sigma_s$
$= 310/153 = 2.03$ therefore, say $= 1.50^‡$ — **NA, Table NA.5: Note 5**

$\therefore$ Permissible $l/d = 26.8 \times 1.5 \times 0.8 \times 0.77 \times 1.50 = 37.1$

Actual $l/d = 9000/257 = 35$ $\therefore$ OK

$\therefore$ Use 2 H20/rib (628 mm^2/rib)

‡ Both $A_{s,prov}/A_{s,req}$ and any adjustment to N obtained from Exp. (7.16a) or Exp. (7.16b) is restricted to 1.5 by Note 5 to Table NA.5 in the UK NA.

3.3.7 Design for shear

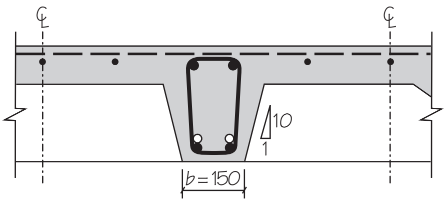


Figure 3.14 Section through rib

a) Support A (and D) at solid/rib interface

Shear at solid/rib interface = 29.3 kN/rib

Taking solid area as the support, at *d* from face of support — Cl. 6.2.1(8)

V_{Ed} = 29.3 – 0.232 × 0.90 × 13.38 = 26.5 kN/rib — Cl. 6.2.2(1) & NA

Resistance

$V_{Rd,c} = (0.18/\gamma_C)k\,(100\,\rho_l\,f_{ck})^{0.333}\,b_w d$

where

γ_C = 1.5

k = $1 + (200/d)^{0.5} \leq 2$

= $1 + (200/257)^{0.5}$

= 1.88

$\rho_l = A_{sl}/b_w d$

where

A_{sl} = assume only 1 H20 anchored = 314 mm^2

b_w = min. width between tension and compression chords.

At bottom assuming 1/10 slope to rib:

= 150 + 2 × (25 + 8 + 20/2)/10

= 159 mm

d = 257 mm as before

ρ_l = 314/(159 × 257) = 0.0077

f_{ck} = 35

$\therefore V_{Rd,c}$ = $(0.18/1.5)\ 1.88\ (100 \times 0.0077 \times 35)^{0.333} \times 159 \times 257$

= 0.68 × 159 × 257 = 27.8 kN/rib

∴ No shear links required. — Cl. 6.2.1(5)

But use nominal links to allow prefabrication.

b) Support B (and C) at solid/rib interface

Shear at solid/rib interface = 40.9 kN/rib [max(B_A; B_C)]

At *d* from face of support — Cl. 6.2.1(8)

V_{Ed} = 40.9 – 0.245 × 13.38 × 0.9 = 37.9 kN/rib

Resistance: $V_{Rd,c} = (0.18/\gamma_C)k(100\,\rho_l f_{ck})^{0.333} b_w d$ where $\gamma_C = 1.5$ $k = 1 + (200/d)^{0.5} \leq 2$ $= 1 + (200/245)0.5$ $= 1.90$ $\rho_l = A_{sl}/b_w d$ where $A_{sl} =$ 2 H16 = 402 mm^2 $b_w =$ 159 mm as before $d =$ 245 mm as before $\rho_l = 0.0103$ $f_{ck} =$ 35 MPa $\therefore V_{Rd,c} = (0.18/1.5)\,1.9\,(100 \times 0.0103 \times 35)^{0.333} \times 159 \times 245$ $= 0.75 \times 159 \times 245 = 29.2$ kN/rib ∴ Shear links required.	Cl. 6.2.2(1) & NA
Shear links required for a distance: $(37.9 - 29.2)/(13.38 \times 0.9) + 245 = 722 + 245 = 967$ mm from interface.	
Check shear capacity: $V_{Rd,max} = \alpha_{cw}\, b_w\, z\nu f_{cd}/(\cot\theta + \tan\theta)$ where $\alpha_{cw} = 1.0$ $b_w =$ 159 mm as before $z = 0.9d$ $\nu = 0.6\,(1 - f_{ck}/250) = 0.528$ $f_{cd} = 35/1.5 = 23.3$ MPa $\theta =$ angle of inclination of strut. Rearranging formula above: $(\cot\theta + \tan\theta) = \alpha_{cw} b_w z \nu f_{cd}/V_{Ed}$ $= \dfrac{(1.0 \times 159 \times 0.9 \times 245 \times 0.528 \times 23.3)}{41.6 \times 103}$ $= 10.4$	Exp. (6.9) & NA
By inspection, $\cot^{-1}\theta << 21.8$. But $\cot\theta$ restricted to 2.5 and $\therefore \tan\theta = 0.4$. $V_{Rd,max} = 1.0 \times 159 \times 0.9 \times 245 \times 0.528 \times 20/(2.5 + 0.4) = 127.6$ kN ∴ OK	Cl. 6.2.3(2) & NA

Shear links: shear resistance with links

$V_{Rd,s} = (A_{sw}/s)\ z\ f_{ywd} \cot\theta \le V_{Rd,max}$ — Exp. (6.8)

where

A_{sw}/s = area of legs of links/link spacing

z = $0.9d$ as before

f_{ywd} = 500/1.15 = 434.8

$\cot\theta$ = 2.5 as before

∴ for $V_{Ed} \le V_{Rd,s}$

$A_{sw}/s \ge V_{Ed}/z\ f_{ywd} \cot\theta$

$\ge 37.9 \times 10^3/(0.9 \times 245 \times 434.8 \times 2.5) \ge 0.158$

Maximum spacing of links = $0.75d$ = 183 mm — Cl. 9.2.2(6)

∴ Use H8 @ 175 cc in 2 legs ($A_{sw}/s = 0.57$) for min. 967 mm into rib

3.3.8 Indirect supports

Cl. 9.2.5, Fig. 9.7

As the ribs of the slab are not supported at the top of the supporting beam sections (A, B, C, D), additional vertical reinforcement should be provided in these supporting beams and designed to resist the reactions. This additional reinforcement should consist of links within the supporting beams (see Beams design, Section 4.3.9).

Support A (and D) at solid/rib interface:

V_{Ed} = 26.5 kN/rib — Fig. 9.7

$A_{s,req} = 26.3 \times 1000/(500/1.15) = 60\ mm^2$

This area is required in links within $h/6$ = 300/6 = 50 mm of the ribbed/solid interface and within $h/2$ = 300/2 = 150 mm of the centreline of the rib.

Support B (and C) at solid/rib interface:

V_{Ed} = 37.9 kN/rib

$A_{s,req} = 37.9 \times 1000/(500/1.15) = 87\ mm^2$ placed similarly

3.3.9 Other checks

Check shear between web and flange — Cl. 6.4.2 (6) & NA

By inspection, $V_{Ed} \le 0.4\ f_{ct,d}$ ∴ OK

3.3.10 Summary of design

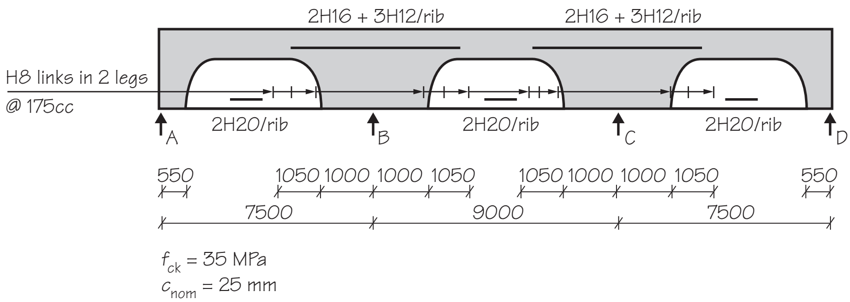


Figure 3.15 Summary of design

Commentary

It is usually presumed that the detailer would take the above design and detail the slab to normal best practice. As stated in Section 3.2.9, the detailer's responsibilities, standards and timescales should be clearly defined but it would be usual for the detailer to draw and schedule not only the designed reinforcement but all the reinforcement required to provide a buildable solution.

The work would usually include checking the following aspects and providing appropriate detailing:

- Minimum areas
- Curtailment lengths
- Anchorages
- Laps
- U-bars
- Rationalisation
- Details and sections

The determination of minimum reinforcement areas, curtailment lengths and laps using the principles in Eurocode 2 is shown in detail in the following calculations. In practice these would be determined from published tables of data or by using reference texts [12, 21]. Nonetheless the designer should check the drawing for design intent and compliance with standards. It is therefore necessary for the designer to understand and agree the principles of the details used.

3.3.11 Detailing checks

a) Minimum areas

i) Minimum area of reinforcement in flange

$A_{s,min} = 0.26\ (f_{ctm}/f_{yk})\ b_t d \geq 0.0013\ b_t d$ **Cl. 9.3.1.1**

where b_t = width of tension zone $f_{ctm} = 0.30 \times f_{ck}^{0.666}$	Cl. 9.2.1.1, Exp. (9.1N) Table 3.1
$A_{s,min} = 0.26 \times 0.30 \times 35^{0.666} \times 1000 \times 100/500 = 166$ mm^2/m ($\rho = 0.17\%$) ∴ Use A142 in flange (say OK)	BS 8666[19]
ii) Secondary reinforcement Not applicable.	
iii) Maximum spacing of bars Maximum spacing of bars < 3 h < 400 mm By inspection. OK	Cl. 9.3.1.1.(3)
iv) Crack control Loading is the main cause of cracking ∴ use Table 7.2N or Table 7.3N for w_{max} = 0.3 mm and max. σ_s = 200 MPa (see deflection check) Max. bar size = 25 mm or max. spacing = 250 mm OK by inspection	Cl. 7.3.3(2) Cl. 7.3.1.5 Table 7.2N Table 7.3N
v) Effects of partial fixity Assuming partial fixity exists at end supports, 15% of A_s is required to extend 0.2 × the length of the adjacent span. $A_{s,req}$ = 15% × 525 = 79 mm^2/rib For the rib in tension: $A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 159 \times 257/500 = 55$ mm^2/rib	Cl. 9.3.1.2(2)
b) Curtailment Wherever possible simplified methods of curtailing reinforcement would be used. The following is intended to show how a rigorous assessment of curtailment of reinforcement might be undertaken.	
i) End support A: bottom steel at support Check anchorage. As simply supported, 25% of A_s should be anchored in support. 25% × 595 = 148 mm^2 Use 1 no. H20/rib (314 mm^2/rib)	Cl. 9.3.1.1(4), 9.3.1.2(1) & Note, Cl. 9.2.1.4(1) & NA
ii) Check anchorage length Envelope of tensile force: To resist envelope of tensile force, provide reinforcement to a_l or l_{bd} beyond centreline of support. For members without shear reinforcement, $a_l = d = 232$ By inspection, $\sigma_{sd} = 0$, $l_{bd} = l_{bd,min}$ = max(10ϕ, 100 mm)	Cl. 9.3.1.1(4), 9.2.1.3(1), Cl. 9.2.1.3(2), 9.2.1.3(3), Fig. 9.2 Cl. 9.2.1.3
iii) Indirect support As anchorage may be measured from face of indirect support, check force to be resisted at solid/rib interface: $F_s = M_{Ed}/z + F_E$	Cl. 9.3.1.1(4), 9.2.1.4(2), 9.2.1.4(3), Fig. 9.3b

where M_{Ed} = 18.3 kNm/rib z = 220 as before F_E = $V_{Ed} \times a_l/z$ where V_{Ed} = 29.3 kN/rib a_l = $z \cot \theta/2$ $\therefore F_E$ = $V_{Ed} \cot \theta/2$ = 29.3 × 1.25 = 36.6 kN/rib $F_s = 18.6 \times 10^6/(220 \times 10^3) + 36.6 = 121.1$ kN	Exp. (9.3) Cl. 9.2.1.3, Exp. (9.2)
iv) Anchorage length: $l_{bd} = \alpha l_{b,rqd} \geq l_{b,min}$ where α = conservatively 1.0 $l_{b,rqd} = (\phi/4)\ (\sigma_{sd}/f_{bd})$ where ϕ = 20 σ_{sd} = design stress in the bar at the ULS = 121.1 × 1000/314 = 385 MPa f_{bd} = ultimate bond stress = $2.25\ \eta_1\ \eta_1\ f_{ct,d}$ where η_1 = 1.0 for good bond conditions η_2 = 1.0 for bar diameter ≤ 32 mm $f_{ct,d} = \alpha_{ct}\ f_{ct,k}/\gamma_C$ = 1.0 × 2.2/1.5 = 1.47 MPa f_{bd} = 2.25 × 1.47 = 3.31 MPa $\therefore l_{b,rqd}$ = (20/4) (385/3.31) = 581 mm $l_{b,min}$ = max[10ϕ; 100 mm] = 200 mm $\therefore l_{bd}$ = 581 mm measured from solid/rib intersection. i.e. 31 mm beyond centreline of support‡.	Cl. 8.4.4, Exp. (8.4) Exp. (8.3) Cl. 8.4.2(2) Cl. 3.1.6(2), Tables 3.1, 2.1 & NA Fig. 9.3
v) End support A: top steel Assuming partial fixity exists at end supports, 15% of A_s is required to extend at least 0.2 × the length of the adjacent span§. $A_{s,req}$ = 15% × 525 = 79 mm^2/rib $A_{s,min} = 0.26 \times 0.30 \times 35^{0.666} \times 159 \times 257/500 = 68$ mm^2/rib <u>Use 2 no. H12 T1/rib in rib and 2 no. H10 T1/rib between ribs</u> <u>(383 mm^2/rib)</u>	Cl. 9.3.1.2(2) Cl. 9.3.1.1 Cl. 9.2.1.1(1), Exp. (9.1N)
‡ Whilst this would comply with the requirements of Eurocode 2, it is common practice to take bottom bars 0.5 × a tension lap beyond the centreline of support (= 250 mm beyond the centreline of support; see model detail MS1 in SMDSC[9]). § It is usual to curtail 50% of the required reinforcement at 0.2*l* and to curtail the remaining 50% at 0.3*l* or line of zero moment (see model detail MS2 in SMDSC[9]).	

vi) Support B (and C): top steel

At the centreline of support (2 no. H16 T + 3 no. H12 T)/rib are required. The intention is to curtail in two stages, firstly to 2 no. H16 T/rib then to 2 no. H12 T/rib.

Curtailment of 2 no. H16 T/rib at support (capacity of 2 no. H12 T/rib + shift rule):
Assume use of 2 no. H12 T throughout in midspan:
Assuming z = 211 mm as before,
M_{R2H12T} = 2 × 113 × 434.8 × 211
= 20.7 kNm/rib (23.0 kNm/m)
(**Note:** section remains under-reinforced)

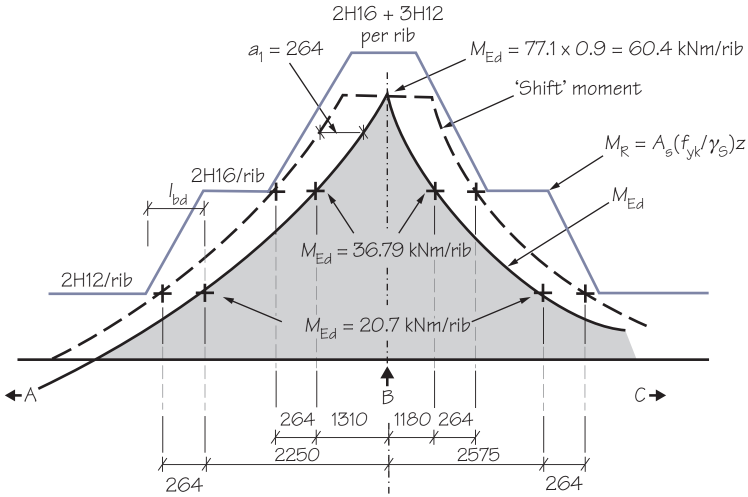
From analysis M_{Ed} = 23.0 kNm/m occurs at 2250 mm (towards A) and 2575 mm (towards B).
Shift rule: $\alpha_l = z \cot \theta/2$
Assuming z = 211 mm as before
α_l = 1.25 × 211 = 264 mm
∴ 2 no. H12 T are adequate from 2250 + 264 = 2513 mm from B towards A and 2575 + 263 = 2838 mm from B towards C.
∴ Curtail 2 no. H16 T @ say 2600 from B_A and 2850 from B_C

Curtailment of 3 no. H12 T/rib at support (capacity of 2 no. H16 T/rib + shift rule):
M_{R2H16T} = 2 × 201 × 434.8 × 211
= 36.9 kNm/rib (41.0 kNm/m)
(**Note:** section remains under-reinforced)
From analysis M_{Ed} = 41.0 kNm/m occurs at 1310 mm (towards A) and 1180 mm
(towards C).
Shift rule: α_l = 263 mm as before
∴ 2 no. H16 T are adequate from 1310 + 263 = 1573 mm from B towards A and 1180 + 263 = 1443 mm from B towards C.
∴ Curtail 3 no. H12 at say 1600 from B (or C).
(See Figure 3.16)

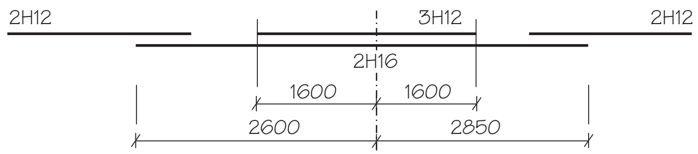
vii) Support B (and C): bottom steel at support — **Cl. 9.3.1.1(4), 9.2.1.5(1), 9.2.1.4(1)**

At the support 25% of span steel required
0.25 × 628 = 157 mm^2
Try 1 no. H16 B/rib (201)

This reinforcement may be anchored into indirect support or carried through. — **Fig. 9.4**



a) Design moments and moment resistance



b) Curtailment of reinforcement

Figure 3.16 Curtailment of top reinforcement at B per rib

viii) Support B (and C): bottom steel curtailment BA and BC

To suit prefabrication 2 no. H20/rib will be curtailed at solid/rib interface, 1000 mm from B_A (B towards A) and B_C.

From analysis, at solid/rib interface sagging moment = 0.

From analysis, at a_1 from solid/rib interface, i.e. at 1000 + 1.25 × 244 = 1303 mm

at 1305 mm from B_A sagging moment = say 5 kNm/rib

at 1305 mm from B_C sagging moment = 0

Use 1 no. H16 B/rib (201)

c) Laps

At A_B, check lap 1 no. H20 B to 2 no. H20 B in rib full tension lap:

$l_0 = \alpha_1 \alpha_6 l_{b,rqd} > l_{0,min}$ — Exp. (8.10)

where

α_1 = 1.0 (c_d = 45 mm, i.e. < 3ϕ) — Table 8.2

α_6 = 1.5 (as > 50% being lapped)

$l_{b,rqd} = (\phi/4)(\sigma_{sd}/f_{bd})$

where

ϕ = 20

σ_{sd} = 434.8

f_{bd} = 3.0 MPa as before

$l_{0,min}$ = max. 10ϕ or 100 = 200 — Exp. (8.6)

$l_0 = 1.0 \times 1.5 \times (20/4) \times 434.8/3.0$

= 1087 mm, say = 1200 mm — SMDSC[9]

At B_A and B_C, check lap 2 no. H12 T to 2 no. H16 T in rib – full tension lap:

$l_0 = \alpha_1\ \alpha_6\ l_{b,rqd} > l_{0,min}$ — Exp. (8.10)

where

α_1 = 0.7 (c_d = 45 mm, i.e. > 3ϕ) — Table 8.2

α_6 = 1.5 (as > 50% being lapped)

$l_{b,rqd} = (\phi/4)\ (\sigma_{sd}/f_{bd})$

where

ϕ = 20

σ_{sd} = 434.8

f_{bd} = 2.1 (3.0 MPa as before but η_1 = 0.7 for "not good bond conditions") — Cl. 8.4.2

$l_{0,min}$ = max. 10ϕ or 100 = 120

$l_0 = 0.7 \times 1.5 \times (12/4) \times 434.8/2.1$

= 651 mm, say = 700 mm — SMDSC[9]

But to aid prefabrication take to solid/rib intersection 1000 mm from centre of support.

At B_A and B_C, check lap 1 no. H16 B to 2 no. H20 B in rib:

By inspection, nominal say, 500 mm — SMDSC[9]

d) RC detail of ribbed slab

Links not shown for clarity. Cover 25 mm to links.

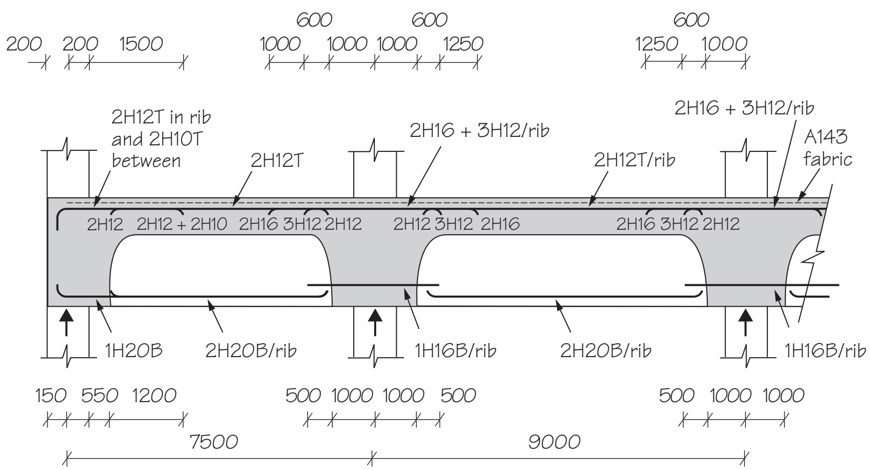


Figure 3.17 Curtailment of flexural reinforcement in ribbed slab

3.4 Flat slab

This example is for the design of a reinforced concrete flat slab without column heads. The slab is part of a larger floor plate and is taken from *Guide to the design and construction of reinforced concrete flat slabs*[27], where finite element analysis and design to Eurocode 2 is illustrated. As with the *Guide*, grid line C will be designed but, for the sake of illustration, coefficients will be used to establish design moments and shears in this critical area of the slab.

The Concrete Centre — Part of the Mineral Products Association	Project details: **Flat slab**	Calculated by: chg	Job no.: CCIP – 041
		Checked by: web	Sheet no.: 1
		Client: TCC	Date: Oct 09

The slab is for an office where the specified load is 1.0 kN/m² for finishes and 4.0 kN/m² imposed (no partitions). Perimeter load is assumed to be 10 kN/m. Concrete is C30/37. The slab is 300 mm thick and columns are 400 mm square. The floor slabs are at 4.50 m vertical centres. A 2 hour fire rating is required.

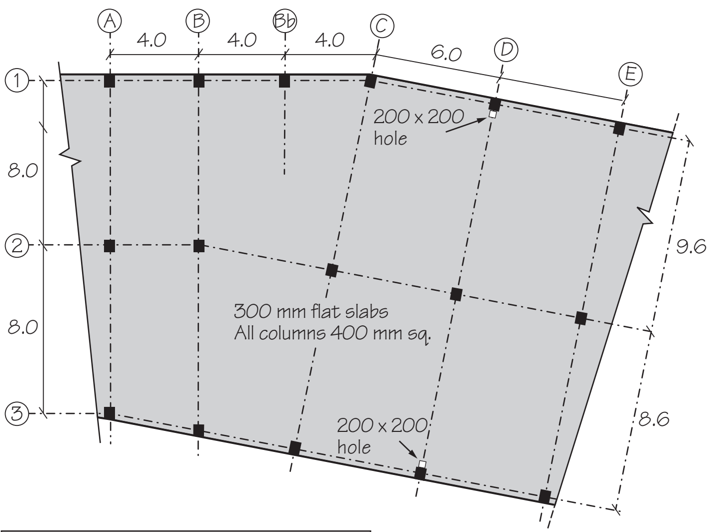


Figure 3.18 Part plan of flat slab

3.4.1 Actions

	kN/m²	
Permanent:		
Self-weight 0.30 × 25	= 7.5	EC1-1-1: Table A1
Finishes	= 1.0	
Total	g_k = 8.5	
Variable:		
Offices	q_k = 4.0‡	

‡ Client requirement. See also BS EN 1991–1–1, Tables 6.1, 6.2, Cl. 6.3.2.1(8) & NA.

3.4.2 Cover

c_{nom}:

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

$c_{min} = \max[c_{min,b}; c_{min,dur}; 10 \text{ mm}]$ — Cl. 4.4.1.2(3)

where

$c_{min,b}$ = 20 mm, assuming 20 mm diameter reinforcement

$c_{min,dur}$ = 15 mm for XC1 and using C30/37 — Table 4.1. BS 8500-1: Table A4.

Δc_{dev} = 10 mm

Fire:

For 2 hours resistance, a_{min} = 35 mm ∴ not critical — EC2-1-2: Table 5.9

∴ c_{nom} = 20 + 10 = 30 mm

3.4.3 Load combination and arrangement

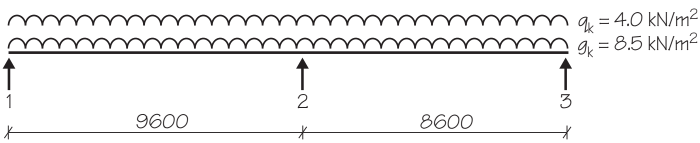


Figure 3.19 Panel centred on grid C

Ultimate load, n: — Fig. 2.5

By inspection, Exp. (6.10b) is critical. — EC0: Exp. (6.10b)

$n = 1.25 \times 8.50 + 1.5 \times 4.0 = 16.6$ kN/m^2

Arrangement:

Choose to use all-and-alternate-spans-loaded load cases and coefficients‡. — Cl. 5.1.3(1) & NA: Table NA.1 (option b)

3.4.4 Analysis grid line C

Consider grid line C as a bay 6.0 m wide. (This may be conservative for grid line C but is correct for grid line D etc.)

M_{Ed}

Effective spans:

9600 – 2 × 400/2 + 2 × 300/2 = 9500 mm — Cl. 5.3.2.2(1)

8600 – 2 × 400/2 + 2 × 300/2 = 8500 mm

Check applicability of moment coefficients:

8500/9500 = 0.89 ∴ as spans differ by less than 15% of larger span, coefficients are applicable. — Tables C2 & C3

‡The all-spans-loaded case with 20% redistribution of support moments would also have been acceptable but would have involved some analysis. The use of Table 5.9 in BS EN 1992–1–2 (Fire resistance of solid flat slabs) is restricted to where redistribution does not exceed 15%; the coefficients presume 15% redistribution at supports. — Cl. 5.3.1 & NA; Table C3

As two span, use table applicable to beams and slabs noting increased coefficients for central support moment and shear. — Table C3

Design moments in bay.

Spans:

$M_{Ed} = (1.25 \times 8.5 \times 0.090 + 1.5 \times 4.0 \times 0.100) \times 6.0 \times 9.5^2 = 842.7$ kNm

Support:

$M_{Ed} = 16.6 \times 0.106 \times 6.0 \times 9.5^2 = 952.8$ kNm

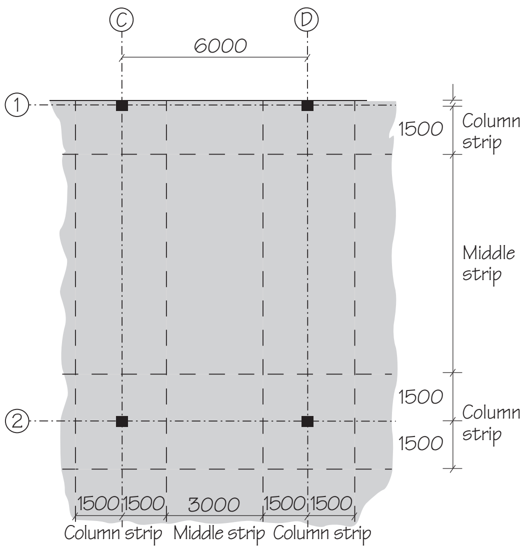


Figure 3.20 Column and middle strips

Apportionment of moments between column strips and middle strips: — Table I.1; CS Flat slab guide[27]

	Apportionment (as %)	
	Column strip	**Middle strip**
–ve (hogging)	Long span = 70%§ Short span = 75%	Long span = 30% Short span = 25%
+ve (sagging)	50%	50%

Parallel to grid C, column strip is $l_y/2 = 3$ m wide. The middle strip is also 3 m wide. — Table I.1 NA.3[1a]; Fig. I.1

Long span moments:

	M_{Ed}	
	Column strip, 3 m wide	**Middle strip, 3 m wide**
–ve (hogging)	$0.70 \times 952.8/3.0 = 222.3$ kNm/m	$0.30 \times 952.8/3.0 = 95.3$ kNm/m
+ve (sagging)	$0.50 \times 842.7/3.0 = 140.5$ kNm/m	$0.50 \times 842.7/3.0 = 140.5$ kNm/m

§ The Concrete Society's TR 64[27] recommends a percentage, k_1, based on l_y/l_z. Assuming $l_y/l_z = 1.5$ the distribution of moments in the long span between column strips and middle strips is given as 70% and 30%.

Punching shear force, V_{Ed}:
At C2,
V_{Ed} = 16.6 × 6.0 × 9.6[‡] × 0.63 × 2 = 1204.8 kN — Table C3

At C1 (and C3)
V_{Ed} = 16.6 × 6.0 × 9.6 × 0.45 + (10 + 0.2 × 0.3 × 25)[§] × 1.25 × 6.0 — Table C3
= 516.5 kN

3.4.5 Design grid line C

Effective depth, d:

d = 300 – 30 – 20/2 = 260 mm

a) Flexure: column strip and middle strip, sagging

M_{Ed} = 140.5 kNm/m

$K = M_{Ed}/bd^2f_{ck} = 140.5 \times 10^6/(1000 \times 260^2 \times 30) = 0.069$

z/d = 0.94 — Table C5

z = 0.94 × 260 = 244 mm

$A_s = M_{Ed}/f_{yd}z = 140.5 \times 10^6/(244 \times 500/1.15)$ = 1324 mm²/m

(ρ = 0.51%)

Try H20 @ 200 B1 (1570 mm²/m)

b) Deflection: column strip and middle strip

Check span-to-effective-depth ratio. — Appendix B

Allowable l/d = N × K × F1 × F2 × F3 — **Cl. 7.4.2(2)**

where — Appendix C

N = 20.3 (ρ = 0.51%, f_{ck} = 30) — Tables C10–C13

K = 1.2 (flat slab)

F1 = 1.0 (b_{eff}/b_w = 1.0)

F2 = 1.0 (no brittle partitions)[#]

F3 = $310/\sigma_s \le 1.5$ — **Cl. 7.4.2, Exp. (7.17) Table 7.4N, & NA, Table NA.5 Note 5**

where*

$\sigma_s = \sigma_{su} (A_{s,req}/A_{s,prov})\ 1/\delta$

where

σ_{su} = (500/1.15) × (8.5 + 0.3 × 4.0)/16.6 = 254 MPa

(or ≈ 253 MPa; from Figure C3

$G_k/Q_k = 2.1$, $\psi_2 = 0.3$ and $\gamma_G = 1.25$) — Fig. C3

δ = redistribution ratio = 1.03

∴ σ_s ≈ 253 × (1324/1570)/1.03 = 207 — Fig. C14

∴ F3 = 310/207 = 1.50[†]

∴ Allowable l/d = 20.3 × 1.2 × 1.50 = 36.5

---

‡ As punching shear force (rather than a beam shear force) 'effective' span is not appropriate.

§ Cladding and strip of slab beyond centre of support.

Otherwise for flat slabs 8.5/9.5 = 0.89 as span > 8.5 m.

* See Appendix B1.5

† In line with Note 5 to Table NA.5, 1.50 is considered to be a maximum for $310/\sigma_s$. — **Cl. 7.4.2(2)**

Actual $l/d = 9500/260 = 36.5$ ∴ OK‡	
Use H20 @ 200 B1 (1570)§	
c) Flexure: column strip, hogging	
$M_{Ed} = 222.3$ kNm/m	
$K = M_{Ed}/bd^2f_{ck} = 222.3 \times 10^6/(1000 \times 260^2 \times 30) = 0.109$	
$z/d = 0.89$	Table C5
$z = 0.89 \times 260 = 231$ mm	
$A_s = M_{Ed}/f_{yd}z = 222.3 \times 10^6/(231 \times 500/1.15) = 2213$ mm²/m	
($\rho = 0.85\%$)	
Try H20 @ 125 T1 (2512 mm²/m)#	
d) Flexure: middle strip, hogging	
$M_{Ed} = 95.3$ kNm/m	
$K = M_{Ed}/bd^2f_{ck} = 95.3 \times 10^6/(1000 \times 260^2 \times 30) = 0.069$	
$z/d = 0.95$	Table C5
$z = 0.95 \times 260 = 247$ mm	
$A_s = M_{Ed}/f_{yd}z = 95.3 \times 10^6/(247 \times 500/1.15) = 887$ mm²/m	
($\rho = 0.34\%$)	
Try H16 @ 200 T1 (1005 mm²/m)	
e) Requirements	
i) In column strip, inside middle 1500 mm	**Cl. 9.4.1(2)**
There is a requirement to place 50% of A_t within a width equal to 0.125 of the panel width on either side of the column.	
Area required = $(3 \times 2213 + 3 \times 887)/2$ mm²	
= 4650 mm²	
Over width = $2 \times 0.125 \times 6.0$ m = 1500 mm	
i.e. require 4650/1.5 = 3100 mm²/m for 750 mm either side of the column centreline.	
Use H20 @ 100 T1 (3140 mm²/m)	
750 mm either side of centre of support (16 no. bars)	
($\rho = 0.60\%$)	
ii) In column strip, outside middle 1500 mm	
Area required = $3.0 \times 2213 - 16 \times 314$ mm²	
= 1615 mm²	
Over width = $3000 - 2 \times 750$ mm = 1500 mm	
i.e. 1077 mm²/m	
Use H20 @ 250 T1 (1256 mm²/m)	
in remainder of column strip	
‡ Note: Continuity into columns will reduce sagging moments and criticality of deflection check (see Figures 3.26 and 3.27).	
§ Note requirement for at least 2 bars in bottom layer to carry through column.	
# The hogging moment could have been considered at face of support to reduce the amount of reinforcement required.	**Cl. 9.4.1(3)**

iii) In middle strip Use H16 @ 200 T1 (1005 mm^2/m)

iv) Perpendicular to edge of slab at edge column

Design transfer moment to column $M_t = 0.17\ b_e d^2 f_{ck}$ — Cl. 9.4.2(1), I.1.2(5)

where

$b_e = c_z + y = 400 + 400 = 800$ mm — Fig. 9.9

$M_t = 0.17 \times 800 \times 260^2 \times 30 \times 10^{-6} = 275.8$ kNm

$K = M_{Ed}/bd^2 f_{ck} = 275.8 \times 10^6/(800 \times 260^2 \times 30) = 0.170$

$z/d = 0.82$

$z = 0.82 \times 260 = 213$ mm

$A_s = M_{Ed}/f_{yd}z = 275.8 \times 10^6/(213 \times 500/1.15) = 2978$ mm^2/m

This reinforcement to be placed within $c_x + 2c_y = 1100$ mm — SMDSC[9]

Try 10 no. H20 T1 U-bars in pairs @ 200 (3140 mm^2) local to column (max. 200 mm from column)

Note:

Where a 200 × 200 hole occurs on face of column, b_e becomes 600 mm and pro rata, $A_{s,req}$ becomes 2233 mm^2 i.e. use 4 no. H20 each side of hole (2512 mm^2).

v) Perpendicular to edge of slab generally

Assuming that there is partial fixity along the edge of the slab, top reinforcement capable of resisting 25% of the moment in the adjacent span should be provided — Cl. 9.3.1.2(2), 9.2.1.4(1) & NA

$0.25 \times 2213 = 553$ mm^2/m OK

vi) Check minimum area of reinforcement

$A_{s,min} = 0.26\ (f_{ctm}/f_{yk})\ b_t d \geq 0.0013\ b_t d$ — Cl. 9.3.1.1, 9.2.1.1

where

b_t = width of tension zone — Table 3.1

$f_{ctm} = 0.30 \times f_{ck}^{0.666}$

$A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 1000 \times 260/500 = 390$ mm^2/m

($\rho = 0.15\%$)

Use H12 @ 200 (565 mm^2/m)

The reinforcement should extend $0.2h$ from edge = 600 mm — Cl. 9.3.1.4(2)

3.4.6 Analysis grid line 1 (grid 3 similar)

Consider grid line 1 as being 9.6/2 + 0.4/2 = 5.0 m wide with continuous spans of 6.0 m. Column strip is 6.0/4 + 0.4/2 = 1.7 m wide. Consider perimeter load is carried by column strip only. — Cl. 5.1.1(4)

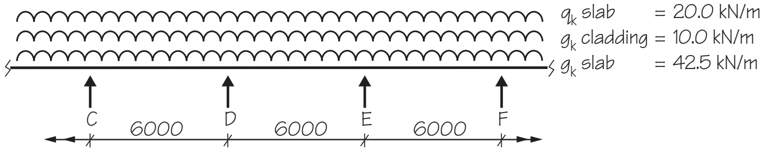


Figure 3.21 Edge panel on grid 1 (grid 3 similar)

Actions:

Permanent from slab $g_k = 5 \times 8.5$ kN/m^2 = 42.5 kN/m

Variable from slab $q_k = 5 \times 4.0$ kN/m^2 = 20.0 kN/m

Permanent perimeter load g_k = 10.0 kN/m

Load combination and arrangement:

Cl. 5.1.3(1) & NA: Table NA.1 (option c)

As before, choose to use all-spans-loaded case and coefficients

Ultimate load, *n*:

Fig. 2.5
ECO: Exp. (6.10b)

By inspection, Exp. (6.10b) is critical.

$n = 1.25 \times (42.5 + 10) + 1.5 \times 20 = 95.6$ kN/m

Perimeter load, $10 \times 1.25 = 12.5$ kN/m

Effective span, l_{eff}

Cl. 5.3.2.2(1)

Effective span $= 6000 - 2 \times 400/2 + 2 \times 300/2 = 5900$

Design moments in bay, M_{Ed}:

In spans (worst case, end span assuming pinned support)

Table C2

$M_{Ed} = 0.086 \times 83.0 \times 5.9^2 = 248.5$ kNm

At supports (worst case 1st support)

Table C2

$M_{Ed} = 0.086 \times 83.0 \times 5.9^2 = 248.5$ kNm

Additional moment in column strip only due to perimeter load, spans (and supports, worst case)

$M_{Ed} = 0.086 \times 12.5 \times 5.9^2 = 37.4$ kNm

Apportionment to column strips and middle strips:

NA.3[1a]: Fig. I.1
Table I.1
CS Flat slab guide[27]

	Apportionment (as %)	
	Column strip, 1.7 m wide	**Middle strip**
−ve (hogging)	Short span = 75%	Short span = 25%
+ve (sagging)	50%	50%

Short span moments:

	M_{Ed}	
	Column strip, 1.7 m wide	**Middle strip, 3.3 m wide**
−ve (hogging)	$(0.75 \times 248.5 + 37.4)/1.70$ = 131.6 kNm/m	$0.25 \times 248.5/3.3$ = 18.8 kNm/m
+ve (sagging)	$(0.50 \times 248.5 + 37.4)/1.70$ = 95.1 kNm/m	$0.50 \times 248.5/3.3$ = 37.6 kNm/m

Punching shear force, V_{Ed}	
For internal supports, as before = 516.5 kN	
For penultimate support, 516.5 × 1.18 = 609.5 kN	Table C3

3.4.7 Design grid line 1 (grid 3 similar)

Cover:

c_{nom} = 30 mm as before

d = 300 – 30 – 20 – 20/2 = 240 mm

a) Flexure: column strip, sagging

M_{Ed} = 95.1 kNm/m

$K = M_{Ed}/bd^2f_{ck}$ = 95.1 × 10⁶/(1000 × 240² × 30) = 0.055 — Table C5

z/d = 0.95

z = 0.95 × 240 = 228 mm

$A_s = M_{Ed}/f_{yd}z$ = 95.1 × 10^6/(228 × 500/1.15) = 959 mm²/m

(ρ = 0.40%)

Try H16 @ 200 B2 (1005 mm²/m)

b) Deflection: column strip

Check span-to-effective-depth ratio. — Appendix B

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ — Appendix C7, Tables C10–C13

where

N = 26.2 (ρ = 0.40%, f_{ck} = 30) — **Cl. 7.4.2, Exp. (7.17), Table 7.4N & NA, Table NA.5: Note 5**

K = 1.2 (flat slab)

F1 = 1.0 (b_{eff}/b_w = 1.0)

F2 = 1.0 (no brittle partitions)

F3 = 310/ $\sigma_s \leq 1.5$

where

$\sigma_s = \sigma_{su} (A_{s,req}/A_{s,prov})\ 1/\delta$

where

$\sigma_{su} \approx$ 283 MPa (from Figure C3 and G_k/Q_k = 3.6, ψ_2 = 0.3, γ_G = 1.25) — Fig. C3

δ = redistribution ratio = 1.08 — Table C14

$\therefore\ \sigma_s \approx$ 283 × (959/1005)/1.08 = 250 — Fig. C3

$\therefore$ F3 = 310/250 = 1.24

$\therefore$ Allowable l/d = 26.2 × 1.2 × 1.24 = 39.0

Actual l/d = 5900/240 = 24.5 $\therefore$ OK

Use H16 @ 200 B2 (1005 mm²/m)

c) Flexure: middle strip, sagging

M_{Ed} = 37.6 kNm/m

By inspection, z = 228 mm

$A_s = M_{Ed}/f_{yd}z$ = 37.6 × 10^6/(228 × 500/1.15) = 379 mm²/m

(ρ = 0.56%)

By inspection, deflection OK.

Check minimum area of reinforcement. — Cl. 9.3.1.1, 9.2.1.1

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

where

b_t = width of tension zone

$f_{ctm} = 0.30 \times f_{ck}^{0.666}$ — Table 3.1

$A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 1000 \times 240/500 = 361\ mm^2/m$

($\rho = 0.15\%$)

Use H12 @ 300 T2 (376 mm^2/m)

d) Flexure: column strip, hogging

$M_{Ed} = 131.6$ kNm/m

$K = M_{Ed}/bd^2 f_{ck} = 131.6 \times 10^6/(1000 \times 240^2 \times 30) = 0.076$ — Table C5

$z/d = 0.928$

$z = 0.928 \times 240 = 223$ mm

$A_s = M_{Ed}/f_{yd}z = 131.6 \times 10^6/(223 \times 500/1.15) = 1357\ mm^2/m$

($\rho = 0.56\%$)

Try H20 @ 200 T2 (1570 mm^2/m)‡

e) Flexure: middle strip, hogging

$M_{Ed} = 18.8$ kNm/m

By inspection, $z = 228$ mm — Table C5

$A_s = M_{Ed}/f_{yd}z = 18.8 \times 10^6/(228 \times 500/1.15) = 190\ mm^2/m$

($\rho = 0.08\%$)

$A_{s,min}$ as before $= 361\ mm^2/m$ — Cl. 9.3.1.1, 9.2.1.1

($\rho = 0.15\%$)

Try H12 @ 300 T2 (376 mm^2/m)

f) Requirements

There is a requirement to place 50% of A_t within a width equal to 0.125 of the panel width on either side of the column. As this column strip is adjacent to the edge of the slab, consider one side only: — Cl. 9.4.1(2)

Area required $= (1.5 \times 1357 + 3.3 \times 190)/2\ mm^2$

$= 1334\ mm^2$

Within $= 0.125 \times 6.0$ m $= 750$ mm of the column centreline, i.e. require $1334/0.75 = 1779\ mm^2/m$ for 750 mm from the column centreline.

‡ The hogging moment could have been considered at face of support to reduce the amount of reinforcement required. This should be balanced against the effect of the presence of a 200 × 200 hole at some supports which would have the effect of increasing K but not unduly increasing the total amount of reinforcement required in the column strip (a 1.5% increase in total area would been required).

Allowing for similar from centreline of column to edge of slab:

Use 6 no. H20 @ 175 T2(1794 mm^2/m)

(ρ = 0.68%)

between edge and to 750 mm from centre of support

In column strip, outside middle 1500 mm, requirement is for
1.7 × 1357 – 6 × 314 = 422 mm^2 in 750 mm, i.e. 563 mm^2/m

Use H12 @ 175 T2 (646 mm^2/m) in remainder of column strip

In middle strip Use H12 @ 300 T2 (376 mm^2/m)

3.4.8 Analysis grid line 2

Consider panel on grid line 2 as being 9.6/2 + 8.6/2 = 9.1 m wide and continuous spans of 6.0 m. Column strip is 6.0/2 = 3.0 m wide. (See Figure 3.20).

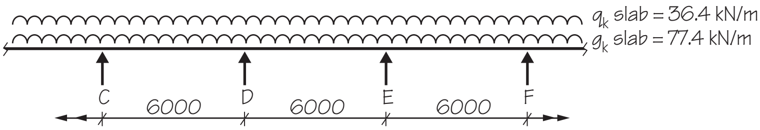


Figure 3.22 Internal panel on grid 2

Slab g_k = 9.1 × 8.5 kN/m^2 = 77.4 kN/m
Slab q_k = 9.1 × 4.0 kN/m^2 = 36.4 kN/m

Actions, load combination and arrangement:
Choose to use all-spans-loaded case. **Cl. 5.1.3(1) & NA: Table NA.1 (option c)**

Ultimate load, n:
By inspection, Exp. (6.10b) is critical. Fig. 2.5 ECO: Exp. (6.10b)
n = 1.25 × 77.4 + 1.5 × 36.4 = 151.4 kN/m

Effective span, l_{eff}:
Effective span = 5900 mm as before. **Cl. 5.3.2.2(1)**

Design moments in bay, M_{Ed}:
Spans (worst case, end span assuming pinned support)
M_{Ed} = 0.086 × 151.4 × 5.9^2 = 453.2 kNm Table C2

Support (worst case 1st support)
M_{Ed} = 0.086 × 151.4 × 5.9^2 = 453.2 kNm Table C2
Additional moment in column strip only due to perimeter load.

Apportionment to column strips and middle strips:

	M_{Ed}	
	Column strip, 3.0 m wide	**Middle strip, 6.1 m wide**
−ve (hogging)	0.75 × 453.2/3.0 = 113.3 kNm/m	0.25 × 453.2/6.1 = 18.5 kNm/m
+ve (sagging)	0.50 × 453.2/3.0 = 75.5 kNm/m	0.50 × 453.2/6.1 = 37.1 kNm/m

Punching shear force, V_{Ed}, as before.

3.4.9 Design grid line 2

Effective depth, d

d = 300 – 30 – 20 – 20/2 = 240 mm

a) Flexure: column strip, sagging

M_{Ed} = 75.5 kNm/m

By inspection, z = 228 mm — Table C5

$A_s = M_{Ed}/f_{yd}z$ = 75.5 × 10^6/(228 × 500/1.15) = 761 mm²/m

(ρ = 0.32%)

Try H16 @ 250 B2 (804 mm²/m)

Deflection: column strip

By inspection, OK.

b) Flexure: column strip, sagging

M_{Ed} = 37.1 kNm/m

By inspection, z = 228 mm

$A_s = M_{Ed}/f_{yd}z$ = 37.1 × 10^6/(228 × 500/1.15) = 374 mm²/m

(ρ = 0.55%)

By inspection, deflection OK. Try H10 @ 200 B2 (393 mm²/m)

c) Flexure: column strip, hogging

M_{Ed} = 113.3 kNm/m

$K = M_{Ed}/bd^2f_{ck}$ = 113.3 × 10^6/(1000 × 240² × 30) = 0.065 — Table C5

z/d = 0.94

z = 0.928 × 240 = 225 mm

$A_s = M_{Ed}/f_{yd}z$ = 113.3 × 10^6/(225 × 500/1.15) = 1158 mm²/m

(ρ = 0.48%)

Try H20 @ 250 T2 (1256 mm²/m)‡

d) Flexure: middle strip, hogging

M_{Ed} = 18.5 kNm/m

By inspection, z = 228 mm

‡ The hogging moment could have been considered at face of support to reduce the amount of reinforcement required.

$A_s = M_{Ed}/f_{yd}z = 18.5 \times 10^6/(228 \times 500/1.15) = 187\ mm^2/m$	
($\rho = 0.08\%$)	Table C5
As before minimum area of reinforcement governs	
$A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 1000 \times 240/500 = 361\ mm^2/m$	Cl. 9.3.1.1, 9.2.1.1
($\rho = 0.15\%$)	
Try H12 @ 300 B2 (376 mm^2/m)	

e) Requirements

Regarding the requirement to place 50% of A_t within a width equal to 0.125 of the panel width on either side of the column:

Area required = $(3.0 \times 1158 + 6.1 \times 187)/2\ mm^2$

= $2307\ mm^2$

Within = $2 \times 0.125 \times 6.0$ m = 1500 mm centred on the column centreline,

i.e. require 2307/1.5 = 1538 mm^2/m for 750 mm either side of the column centreline.

Use H20 @ 200T2 (1570 mm^2/m)

750 mm either side of centre of support

($\rho = 0.60\%$)

In column strip, outside middle 1500 mm, requirement is for $3.0 \times 1158 - 1.5 \times 1570 = 1119\ mm^2$ in 1500 mm, i.e. 764 mm^2/m

Use H16 @ 250 T2 (804 mm^2/m) in remainder of column strip

In middle strip: Use H12 @ 300 T2 (376 mm^2/m)

3.4.10 Punching shear, central column, C2

At C2, applied shear force, V_{Ed} = 1204.8 kN‡

a) Check at perimeter of column

$v_{Ed} = \beta V_{Ed}/u_i d < v_{Rd,max}$	Cl. 6.4.3(2), 6.4.5(3)
where	
β = factor dealing with eccentricity; recommended value 1.15	Fig. 6.21N & NA
V_{Ed} = applied shear force	
u_i = control perimeter under consideration.	
For punching shear adjacent to interior columns	Cl. 6.4.5(3)
$u_0 = 2(c_x + c_y) = 1600$ mm	
d = mean effective depth = (260 + 240)/2 = 250 mm	Exp. (6.32)
$v_{Ed} = 1.15 \times 1204.8 \times 10^3/1600 \times 250 = 3.46$ MPa	
$v_{Rd,max} = 0.5 \nu f_{cd}$	Cl. 6.4.5(3) Note

‡ Column C2 is taken to be an internal column. In the case of a penultimate column, an additional elastic reaction factor should have been considered.

where

$\nu = 0.6(1 - f_{ck}/250) = 0.528$

$f_{cd} = \alpha_{cc}\lambda f_{ck}/\gamma_C = 1.0 \times 1.0 \times 30/1.5 = 20$

$= 0.5 \times 0.528 \times 20 = 5.28$ MPa ∴ OK — Table C7 §

b) Check shear stress at control perimeter u_1 (2d from face of column) — **Cl. 6.4.2**

$v_{Ed} = \beta V_{Ed}/u_1 d < v_{Rd,c}$

where

β, V_{Ed} and d as before

u_1 = control perimeter under consideration. — **Fig. 6.13**

For punching shear at 2d from interior columns

$u_1 = 2(c_x + c_y) + 2\pi \times 2d = 4741$ mm

$v_{Ed} = 1.15 \times 1204.8 \times 10^3/4741 \times 250 = 1.17$ MPa

$v_{Rd,c} = 0.18/\ \gamma_C\ k\ (100\ \rho_l f_{ck})^{0.333}$ — **Exp. (6.47) & NA**

where

$\gamma_C = 1.5$

$k = 1 + (200/d)^{0.5} \le 2\ \ k = 1 + (200/250)^{0.5} = 1.89$

$\rho_l = (\rho_{ly}\ \rho_{lz})0.5 = (0.0085 \times 0.0048)^{0.5} = 0.0064$ — **Cl. 6.4.4.1(1)**

where

ρ_{ly}, ρ_{lz} = Reinforcement ratio of bonded steel in the y and z direction in a width of the column plus 3d each side of column#

$f_{ck} = 30$

$v_{Rd,c} = 0.18/1.5 \times 1.89 \times (100 \times 0.0064 \times 30)^{0.333} = 0.61$ MPa

∴ Punching shear reinforcement required — Table C5*

c) Perimeter at which punching shear links are no longer required — **Exp. (6.54)**

$u_{out} = V_{Ed} \times \beta/(d\ v_{Rd,c})$

$u_{out} = 1204.8 \times 1.15 \times 10^3/(250 \times 0.61) = 9085$ mm

Length of column faces = 4 × 400 = 1600 mm

Radius to $u_{out} = (9085 - 1600)/2\pi = 1191$ mm from face of column

Perimeters of shear reinforcement may stop 1191 – 1.5 × 250 = 816 m from face of column — **Cl. 6.4.5(4) & NA**

Shear reinforcement (assuming rectangular arrangement of links):

$s_{r,max} = 250 \times 0.75 = 187$, say = 175 mm — **Cl. 9.4.3(1)**

§ At the perimeter of the column, $v_{Rd,max}$ assumes the strut angle is 45°, i.e. that cot θ = 1.0. Where cot θ = < 1.0, $v_{Rd,max}$ is available from Table C7.

The values used here for ρ_{ly}, ρ_{lz} ignore the fact that the reinforcement is concentrated over the support. Considering the concentration would have given a higher value of $V_{Rd,c}$ at the expense of further calculation to determine ρ_{ly}, ρ_{lz} at 3d from the side of the column.

* $v_{Rd,c}$ for various values of d and ρ_l is available from Table C6.

Inside 2d control perimeter, $s_{t,max}$ = 250 × 1.5 = 375, say 350 mm — Cl. 9.4.3(2)

Outside control perimeter $s_{t,max}$ = 250 × 2.0 = 500 mm

Assuming vertical reinforcement:

At the basic control perimeter, u_1, 2d from the column[‡]:

$A_{sw} \geq (v_{Ed} - 0.75v_{Rd,c})\ s_r\ u_1/1.5f_{ywd,ef})$ — Exp. (6.52)

where

$f_{ywd,ef}$ = effective design strength of reinforcement

= (250 + 0.25d) < f_{yd} = 312 MPa — Cl. 6.4.5(1)

For perimeter u_1

A_{sw} = (1.17 – 0.75 × 0.61) × 175 × 4741/(1.5 × 312)

= 1263 mm² per perimeter

$A_{sw,min} \geq 0.08f_{ck}^{0.5}(s_r\ s_t)/(1.5\ f_{yk}\ \sin\alpha + \cos\alpha)$ — Exp. (9.11)

where

$A_{sw,min}$ = minimum area of a single leg of link

α = angle between main reinforcement and shear reinforcement; for vertical reinforcement sin α = 1.0

$A_{sw,min} \geq 0.08 \times 30^{0.5}$ (175 × 350)/(1.5 × 500) = 36 mm²

∴ Try H8 legs of links (50 mm²)

$A_{sw}/u_1 \geq$ 1263/4741 = 0.266 mm²/mm

Using H8 max. spacing = min[50/0.266; 1.5d] — Cl. 9.4.3

= min[188; 375] = 188 mm cc

∴ Use min. H8 legs of links at 175 mm cc around perimeter u_1

Perimeters at 0.75d = 0.75 × 250 = 187.5 mm — Cl. 9.4.3(1)

say = 175 mm centres

d) Check area of reinforcement > 1263 mm² in perimeters inside u_1[§]

1st perimeter to be > 0.3d but < 0.5d from face of column. Say 0.4d = 100 mm from face of column. — Fig. 9.10, Cl. 9.4.3(4)

By inspection of Figure 3.23 the equivalent of 10 locations are available at 0.4d from column therefore try 2 × 10 no. H10 = 1570 mm².

By inspection of Figure 3.23 the equivalent of 18 locations are available at 1.15d from column therefore try 18 no. H10 = 1413 mm².

By inspection of Figure 3.23 the equivalent of 20 locations are available at 1.90d from column therefore try 20 no. H10 = 1570 mm².

By inspection of Figure 3.23 beyond u_1 to u_{out} grid of H10 at 175 x 175 OK.

‡ Clause 6.4.5 provides Exp. (6.52), which by substituting v_{Ed} for $v_{Rd,c}$, allows calculation of the area of required shear reinforcement, A_{sw}, for the basic control perimeter, u_1. — Cl. 6.4.5 Exp. 6.5.2

§ The same area of shear reinforcement is required for all perimeters inside or outside perimeter u_1. See Commentary on design, Section 3.4.14. Punching shear reinforcement is also subject to requirements for minimum reinforcement and spacing of shear reinforcement (see Cl. 9.4.3). — Cl. 9.4.3

e) **Summary of punching shear refreshment required at column C2**

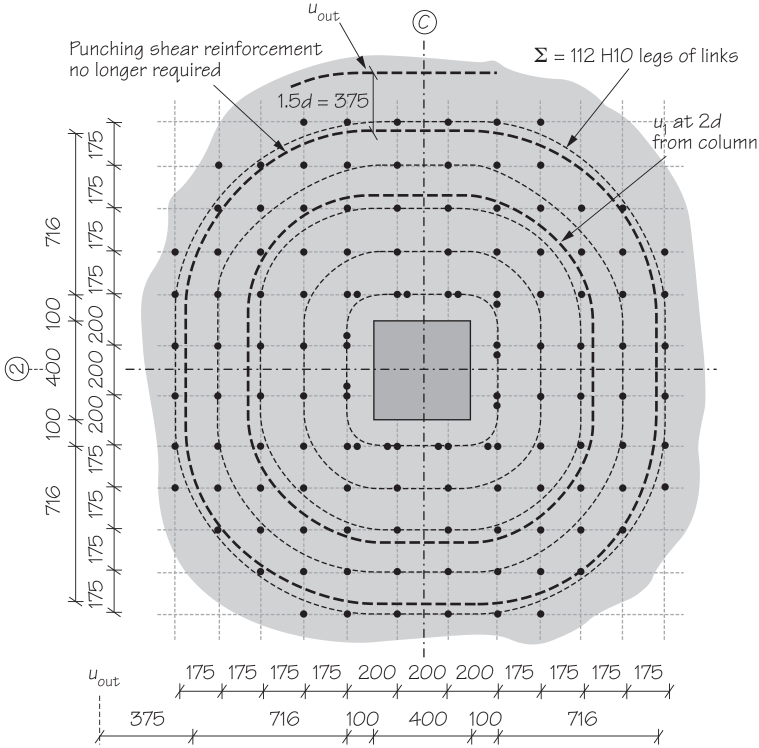


Figure 3.23 Punching shear links at column C2 (112 no. links) (column D2 similar)

3.4.11 Punching shear, edge column

Assuming penultimate support,

$V_{Ed} = 1.18 \times 516.5 = 609.5$ kN — Table C3

a) **Check at perimeter of column**

$v_{Ed} = \beta V_{Ed}/u_i d < v_{Rd,max}$ — **Cl. 6.4.3(2), 6.4.5(3)**

where

β = factor dealing with eccentricity; recommended value 1.4 — **Fig. 6.21N & NA**

V_{Ed} = applied shear force

u_i = control perimeter under consideration. — **Cl. 6.4.5(3)**

For punching shear adjacent to edge columns

$u_0 = c_2 + 3d < c_2 + 2c_1$

= 400 + 750 < 3 × 400 mm

= 1150 mm

d = as before 250 mm — **Exp. (6.32)**

$v_{Ed} = 1.4 \times 609.5 \times 10^3/1150 \times 250 = 2.97$ MPa

$v_{Rd,max}$, as before = 5.28 MPa ∴ OK — **Cl. 6.4.5(3) Note**

b) Check shear stress at basic perimeter u_1 (2.0d from face of column) — Cl. 6.4.2

$v_{Ed} = \beta V_{Ed}/u_1 d < v_{Rd,c}$

where

β, V_{Ed} and d as before

u_1 = control perimeter under consideration. — Fig. 6.15

For punching shear at 2d from edge column columns

$u_1 = c_2 + 2c_1 + \pi \times 2d = 2771$ mm

$v_{Ed} = 1.4 \times 609.5 \times 10^3/2771 \times 250 = 1.23$ MPa

$v_{Rd,c} = 0.18/\ \gamma_C \times k \times (100\ \rho_l f_{ck})^{0.333}$ — Exp. (6.47) & NA

where

γ_C = 1.5

k = as before = $1 + (200/250)^{0.5} = 1.89$

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

where — Cl. 6.4.4.1(1)

ρ_{ly}, ρ_{lz} = Reinforcement ratio of bonded steel in the y and z direction in a width of the column plus 3d each side of column.

ρ_{ly}: (perpendicular to edge) 10 no. H20 T2 + 6 no. H12 T2 in 2 × 750 + 400, i.e. 3818 mm² in 1900 mm

$\therefore \rho_{ly}$ = 3818/(250 × 1900) = 0.0080

ρ_{lz}: (parallel to edge) 6 no. H20 T1 + 1 no. T12 T1 in 400 + 750 i.e. 1997 mm² in 1150 mm.

$\therefore \rho_{lz}$ = 1997/(250 × 1150) = 0.0069

$\rho_l = (0.0080 \times 0.0069)^{0.5} = 0.0074$

f_{ck} = 30

$v_{Rd,c} = 0.18/1.5 \times 1.89 \times (100 \times 0.0074 \times 30)^{0.333} = 0.64$ MPa — Table C6[‡]

$\therefore$ Punching shear reinforcement required

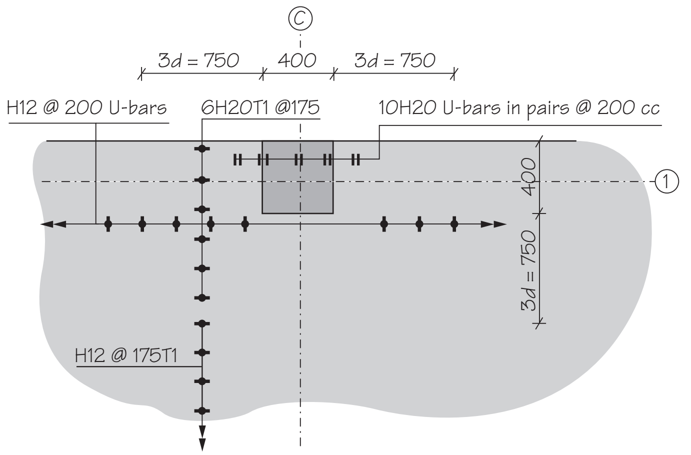


Figure 3.24 Flexural tensile reinforcement adjacent to columns C1 (and C3)

[‡] $v_{Rd,c}$ for various values of d and ρ_l is available from Table C6.

c) Perimeter at which punching shear links no longer required — Exp. (6.54)

$u_{out} = 609.5 \times 1.4 \times 10^3/(250 \times 0.64) = 5333$ mm

Length attributable to column faces = 3 × 400 = 1200 mm

∴ radius to u_{out} from face of column

= say (5333 – 1200)/π = 1315 mm from face of column

Perimeters of shear reinforcement may stop 1370 – 1.5 × 250 = 940 mm from face of column. — Cl. 6.4.5(4) & NA

d) Shear reinforcement

As before, $s_{r,max}$ = 175 mm; $s_{t,max}$ = 350 mm and $f_{ywd,ef}$ = 312 MPa — Cl. 9.4.3(1), 9.4.3(2)

For perimeter u_1

$A_{sw} \geq (v_{Ed} - 0.75v_{Rd,c})\, s_r\, u_1/1.5f_{ywd,ef}$ — Exp. (6.52)

= (1.23 – 0.75 × 0.64) × 175 × 2771/(1.5 × 312)

= 777 mm² per perimeter

$A_{sw,min} \geq 0.08 \times 30^{0.5}\,(175 \times 350)/(1.5 \times 500) = 36\ \text{mm}^2$ — Exp. (9.11)

$A_{sw}/u_1 \geq 777/2771 = 0.28\ \text{mm}^2/\text{mm}$

Using H8 max. spacing = 50/0.28 = 178 mm cc

∴Use min. H8 (50 mm²) legs of links at 175 mm cc around perimeters: perimeters at 175 mm centres

e) Check area of reinforcement > 777 mm² in perimeters inside u_1§ — Fig. 9.10, Cl. 9.4.3(4)

1st perimeter to be > 0.3*d* but < 0.5*d* from face of column. Say 0.4*d* = 100 mm from face of column

By inspection of Figure 3.27 the equivalent of 6 locations are available at 0.4*d* from column therefore try 2 × 6 no. H10 = 942 mm²

By inspection of Figure 3.27 the equivalent of 12 locations are available at 1.15*d* from column therefore try 12 no. H10 = 942 mm²

By inspection of Figure 3.27 the equivalent of 14 locations are available at 1.90*d* from column therefore try 14 no. H10 = 1099 mm²

By inspection of Figure 3.27 beyond u_1 to u_{out} grid of H10 at 175 x 175 OK.

3.4.12 Punching shear, edge column with hole

Check columns D1 and D3 for 200 × 200 mm hole adjacent to column.

As previously described use 4 no. H20 U-bars each side of column for transfer moment.

Assuming internal support, V_{Ed} = 516.5 kN

§ See *Commentary on design* Section 3.4.14. Punching shear reinforcement is also subject to requirements for minimum reinforcement and spacing of shear reinforcement (see Cl. 9.4.3). — Cl. 9.4.3

a) Check at perimeter of column	
$v_{Ed} = \beta V_{Ed}/u_i d < v_{Rd,max}$	**Cl. 6.4.3(2), 6.4.5(3)**
where	
β = factor dealing with eccentricity; recommended value 1.4	**Fig. 6.21N & NA**
V_{Ed} = applied shear force	
u_i = control perimeter under consideration. For punching shear adjacent to edge columns $u_0 = c_2 + 3d < c_2 + 2c_1$	**Cl. 6.4.5(3)**
$= 400 + 750 < 3 \times 400$ mm	
$= 1150$ mm	
Allowing for hole, $u_0 = 1150 - 200 = 950$ mm	
d = 250 mm as before	**Exp. (6.32)**
$v_{Ed} = 1.4 \times 516.5 \times 10^3/950 \times 250 = 3.06$ MPa	
$v_{Rd,max}$ as before = 5.28 MPa ∴ OK	**Cl. 6.4.5(3) Note**
b) Check shear stress at basic perimeter u_1 (2.0*d* from face of column)	**Cl. 6.4.2**
$v_{Ed} = \beta V_{Ed}/u_1 d < v_{Rd,c}$	
where	
β, V_{Ed} and d as before	
u_1 = control perimeter under consideration. For punching shear at 2*d* from edge column columns	**Fig. 6.15**
$u_1 = c_2 + 2c_1 + \pi \times 2d = 2771$ mm	
Allowing for hole	
$200/(c_1/2)$: $x/(c_1/2 + 2d)$	**Fig. 6.14**
200/200: x/(200 + 500)	
∴ $x = 700$ mm	
$u_1 = 2771 - 700 = 2071$ mm	
$v_{Ed} = 1.4 \times 516.5 \times 10^3/2071 \times 250 = 1.40$ MPa	
$v_{Rd,c} = 0.18/\gamma_C \times k \times (100\ \rho_l f_{ck})^{0.333}$	**Exp. (6.47) & NA**
where	
γ_C = 1.5	
k = as before = $1 + (200/250)^{0.5} = 1.89$	
$\rho_l = (\rho_{ly}\rho_{lz})^{0.5}$	
where	
ρ_{ly}, ρ_{lz} = Reinforcement ratio of bonded steel in the y and z direction in a width of the column plus 3*d* each side of column	**Cl. 6.4.4.1(1)**
ρ_{ly}: (perpendicular to edge) 8 no. H20 T2 + 6 no. H12 T2 in 2 × 720 + 400 – 200, i.e. 3190 mm^2 in 1640 mm.	
∴ ρ_{ly} = 3190/(240 × 1640) = 0.0081	
ρ_{lz}: (parallel to edge) 6 no. H20 T1 (5 no. are effective) + 1 no. T12 T1 in 400 + 750 – 200, i.e. 1683 mm^2 in 950 mm.	
∴ ρ_{lz} = 1683/(260 × 950) = 0.0068	

$\rho_l = (0.0081 \times 0.0068)^{0.5} = 0.0074$

$f_{ck} = 30$

$v_{Rd,c} = 0.18/1.5 \times 1.89 \times (100 \times 0.0074 \times 30)^{0.33} = 0.64$ MPa — Table C6‡

∴ punching shear reinforcement required

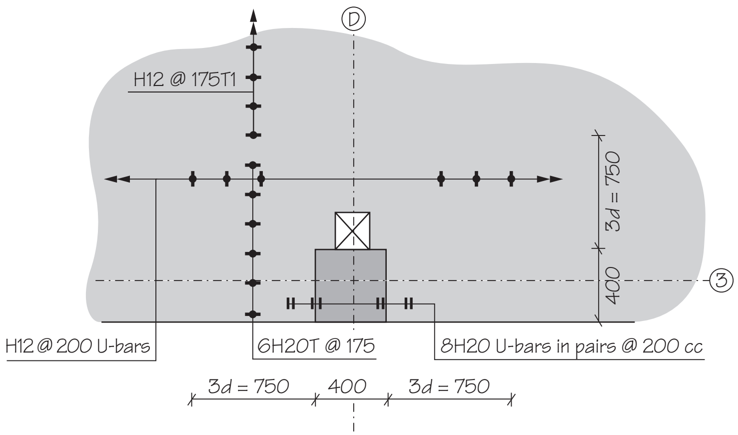


Figure 3.25 Flexural tensile reinforcement adjacent to columns D1 and D3

c) Perimeter at which punching shear links no longer required — Exp. (6.54)

$u_{out} = 516.5 \times 1.4 \times 10^3/(250 \times 0.64) = 4519$ mm

Length attributable to column faces = 3 × 400 = 1200 mm

Angle subtended by hole from centre of column D1 (See Figures 3.25 & 3.27) = 2

$\tan^{-1}(100/200) = 2 \times 26.5° = 0.927$ rads.

∴ radius to u_{out} from face of column

= say $(4519 - 1200)/(\pi - 0.927) = 1498$ mm from face of column

Perimeters of shear reinforcement may stop 1498 – 1.5 × 250 — Cl. 6.4.5(4) & NA

= 1123 mm from face of column

d) Shear reinforcement

As before, $s_{r,max} = 175$ mm; $s_{t,max} = 350$ mm and $f_{ywd,ef} = 312$ MPa — Cl. 9.4.3(1) 9.4.3(2)

For perimeter u_1

$A_{sw} \geq (v_{Ed} - 0.75v_{Rd,c})\ s_r\ u_1/1.5f_{ywd,ef})$ per perimeter — Exp. (6.52)

$= (1.40 - 0.75 \times 0.64) \times 175 \times 2071/(1.5 \times 312)$

$= 712$ mm^2 per perimeter

$A_{sw,min} \geq 0.08 \times 30^{0.5}\ (175 \times 350)/(1.5 \times 500) = 36$ mm^2

$A_{sw}/u_1 \geq 712/2071 = 0.34$ mm^2/mm

‡ $v_{Rd,c}$ for various values of d and ρ_l is available from Table C6.

Using H8 (50 mm²) max. spacing = min[50/0.3; 1.5d]

= min[147; 375] = 147 mm cc   No good

Try using H10, max. spacing = 78.5/0.34 = 231 mm cc, say 175 cc

∴ Use min. H10 (78.5 mm²) legs of links at 175 mm cc around perimeters: perimeters at 175 mm centres

Check min. 9 no. H10 legs of links (712 mm²) in perimeter u_1, 2_d from column face.

e) Check area of reinforcement > 712 mm² in perimeters inside u_1‡

1st perimeter to be 100 mm from face of column as before.

By inspection of Figure 3.27 the equivalent of 6 locations are available at 0.4d from column therefore try 2 × 6 no. H10 = 942 mm².

Fig. 9.10, Cl. 9.4.3(4)

By inspection of Figure 3.27 the equivalent of 10 locations are available at 1.15d from column therefore try 10 H10 = 785 mm².

By inspection of Figure 3.27 beyond 1.15d to u_{out} grid:

H10 at 175 x 175 OK.

3.4.13 Summary of design

Grid C flexure

End supports:	
Column strip: (max. 200 mm from column)	10 no. H20 U-bars in pairs
(where 200 × 200 hole use 8 no. H20 T1 in U-bars in pairs)	
Middle strip:	H12 @ 200 T1
Spans 1–2 and 2–3:	
Column strip and middle strip:	H20 @ 200 B
Central support:	
Column strip centre: for 750 mm either side of support:	H20 @ 100 T1
Column strip outer:	H20 @ 250 T1
Middle strip:	H16 @ 200 T1

Grid 1 (and 3) flexure

Spans:	
Column strip:	H16 @ 200 B2
Middle strip:	H12 @ 300 B2

‡ See *Commentary on design* Section 3.4.14. Punching shear reinforcement is also subject to requirements for minimum reinforcement and spacing of shear reinforcement.

Cl. 9.4.3

Interior support:
Column strip centre: 6 no. H20 @ 175 T2
Column strip outer: H12 @ 175 T2
Middle strip: H12 @ 300 T2

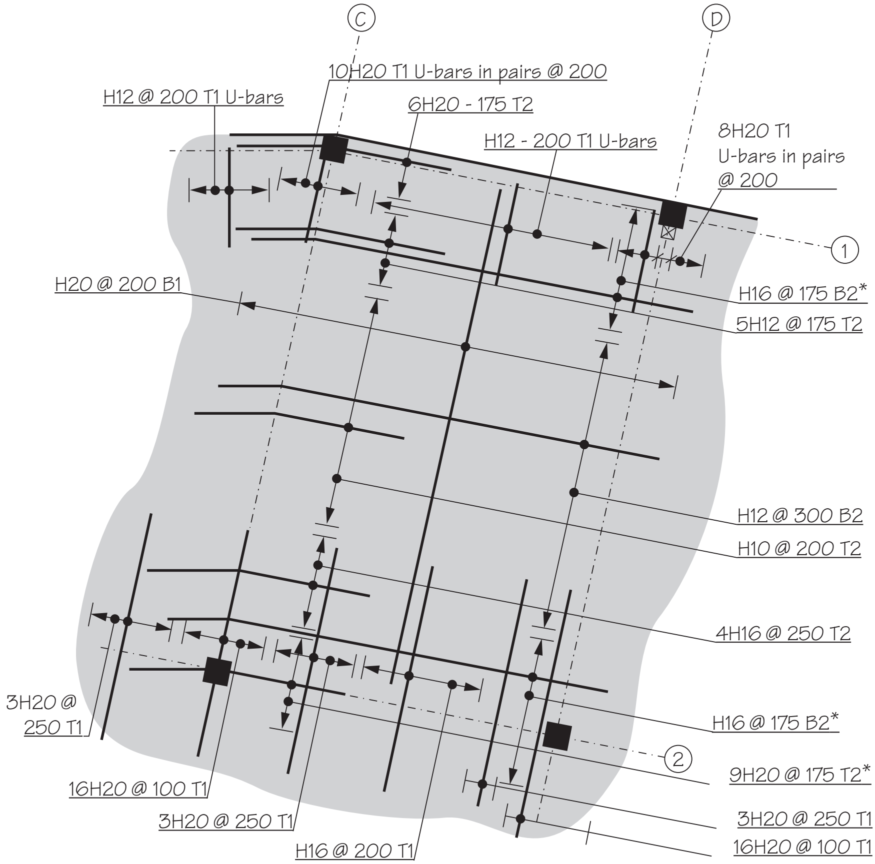
Grid 2 flexure
Spans:
Column strip: H16 @ 250 B2
Middle strip: H10 @ 200 B2

Interior support:
Column strip centre: H20 @ 200 T2
Column strip outer: H16 @ 250 T2
Middle strip: H12 @ 300 T2
See Figure 3.26

Punching shear
Internal (e.g. at C2):
Generally, use H10 legs of links in perimeters at max. 175 mm centres, but double up on 1st perimeter
Max. tangential spacing of legs of links, $s_{t,max}$ = 270 mm
Last perimeter, from column face, min. 767 mm
See Figure 3.26

Edge (e.g. at C1, C3 assuming no holes):
Generally, use H10 legs of links in perimeters at max. 175 mm centres but double up on 1st perimeter
Max. tangential spacing of legs of links, $s_{t,max}$ = 175 mm
Last perimeter, from column face, min. 940 mm

Edge (e.g. at D1, D3 assuming 200 × 200 hole on face of column):
Generally, use H10 legs of links in perimeters at max. 175 mm centres but double up on 1st perimeter
Max. tangential spacing of legs of links, $s_{t,max}$ = 175 mm
Last perimeter, from column face, min. 1123 mm
See Figure 3.27



Note:* Spacing rationalised to suit punching shear links

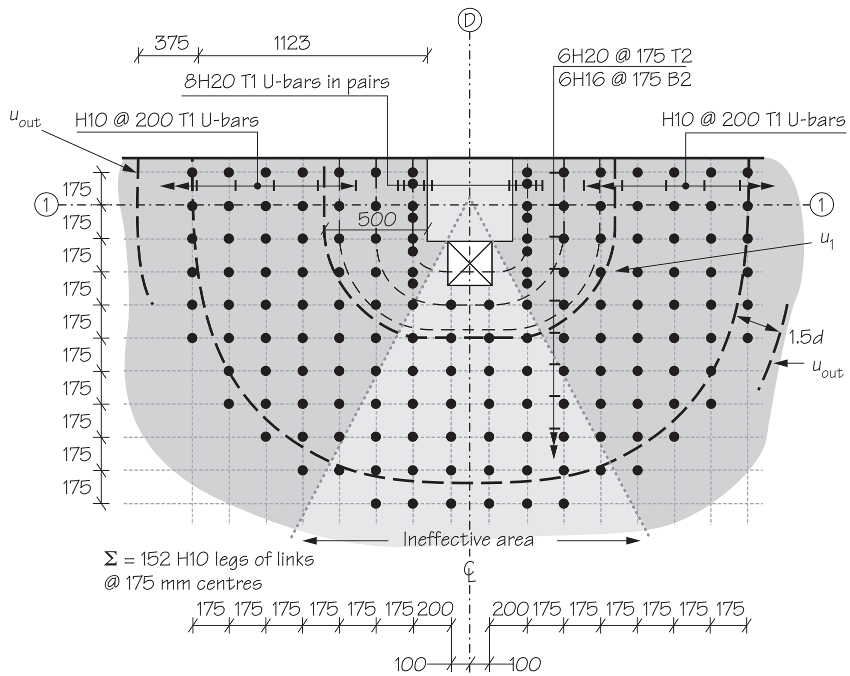
Figure 3.26 Reinforcement details bay C–D, 1–2

3.4.14 Commentary on design

a) Method of analysis

The use of coefficients in the analysis would not usually be advocated in the design of such a slab. Nonetheless, coefficients may be used and, unsurprisingly, their use leads to higher design moments and shears, as shown below.

Method	Moment in 9.6 m span per 6 m bay (kNm)	Centre support moment per 6 m bay (kNm)	Centre support reaction V_{Ed} (kN)
Coefficients	842.7	952.8	1205
Continuous beam	747.0	885.6	1103
Plane frame columns below	664.8	834.0	1060
Plane frame columns above and below	616.8	798.0	1031



Note: For internal column see Figure 3.23

Figure 3.27 Punching shear links at column D1 (and D3) (penultimate support without hole similar)

These higher moments and shears result in rather more reinforcement than when using other more refined methods. For instance, the finite element analysis used in *Guide to the design and construction of reinforced concrete flat slabs*[27] for this bay, leads to:

- H16 @ 200 B1 in spans 1–2 (cf. H20 @ 200 B1 using coefficients)
- H20 @ 125 T1 at support 2 (cf. H20 @ 100 T1 using coefficients)
- 3 perimeters of shear links at C2 for V_{Ed} = 1065 kN (cf. 5 perimeters using coefficients)
- 2 perimeters of shear links at C3 (cf. 7 perimeters using coefficients)

b) Effective spans and face of support

In the analysis using coefficients, advantage was taken of using effective spans to calculate design moments. This had the effect of reducing span moments. **Cl. 5.3.2.2(1)**

At supports, one may base the design on the moment at the face of support. This is borne out by *Guide to the design and construction of reinforced concrete flat slabs*[27] that states that hogging moments greater than those at a distance $h_c/3$ may be ignored (where h_c is the effective diameter of a column or column head). This is in line with BS 8110[7] and could have been used to reduce support moments. **Cl. 5.3.2.2(3)**

c) **Punching shear reinforcement**

Arrangement of punching shear links

According to the literal definition of A_{sw} in Exp. (6.52), the same area of shear reinforcement is required for all perimeters inside or outside perimeter u_1 (rather than $(A_{sw}/u_1)/s_r$ being considered as the required density of shear reinforcement on and within perimeter u_1). For perimeters inside u_1, it might be argued that Exp. (6.50) (enhancement close to supports) should apply. However, at the time of writing, this expression is deemed applicable only to foundation bases. Therefore, large concentrations of shear reinforcement are required close to the columns – in this example, this included doubling up shear links at the 1st perimeter.

Exp. 6.52

Exp. 6.50

Similar to BS 8110[7] figure 3.17, it is apparent that the requirement for punching shear reinforcement is for a punching shear zone 1.5d wide. However, in Eurocode 2, the requirement has been 'simplified' in Exp. (6.52) to make the requirement for a perimeter (up to 0.75d wide). It might appear reasonable to apply the same 40%:60% rule (BS 8110 Cl. 3.7.7.6) to the first two perimeters to make doubling of punching shear reinforcement at the first perimeter unnecessary: in terms of Eurocode 2 this would mean 80% A_{sw} on the first perimeter and 120% A_{sw} on the second. Using this arrangement it would be possible to replace the designed H10 links in the first two perimeters with single H12 links.

BS 8110: Fig. 3.17

BS 8110: Cl. 3.7.7.6

Outside u_1, the numbers of links could have been reduced to maintain provision of the designed amount of reinforcement A_{sw}. A rectangular arrangement of H12 links would have been possible (within perimeter u_1, 350 × 175; outside u_1, 500 × 175). However, as the grid would need to change orientation around each column (to maintain the 0.75d radial spacing) and as the reinforcement in B2 and T2 is essentially at 175 centres, it is considered better to leave the arrangement as a regular square grid.

Cl. 9.4.3(1)

Use of shear reinforcement in a radial arrangement, e.g. using stud rails, would have simplified the shear reinforcement requirements.

$V_{Ed}/V_{Rd,c}$

In late 2008, a proposal was made for the UK National Annex to include a limit of 2.0 or 2.5 on $V_{Ed}/V_{Rd,c}$ (or $v_{Ed}/v_{Rd,c}$) within punching shear requirements. It is apparent that this limitation could have major effects on flat slabs supported on relatively small columns. For instance in Section 3.4.12, edge column with hole, $V_{Ed}/V_{Rd,c}$ = 2.18.

Curtailment of reinforcement

In this design, the reinforcement would be curtailed and this would be done either in line with previous examples or, more practically, in line with other guidance[20, 21].

3.5 Stair flight

This example is for a typical stair flight.

The Concrete Centre	Project details: Stair flight	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

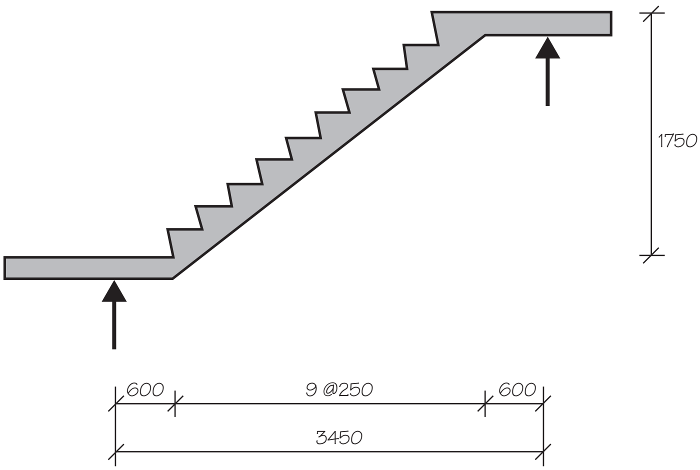


Figure 3.28 Stair flight

3.5.1 Loads

		kN/m²
Permanent (worse case flight)		
Assume 160 waist	$0.160 \times 305/250 \times 25$	= 4.88
Treads	$4 \times 0.25 \times 0.175/2 \times 25$	= 2.19
50 mm screed	0.5×22	= 1.10
Finishing		= 0.03
		g_k = 8.20
Variable action: crowd loading		q_k = 4.00

EC1-1-1: Table 6.1, 6.2 & NA.3

3.5.2 Moment

$M_{Ed} = (8.20 \times 1.25 + 4.00 \times 1.5) \times 3.45^2/8$
$= 24.2$ kNm/m

3.5.3 Design

$d = 160 - c_{nom} - \phi/2$

where

$c_{nom} = 25$ mm (for XC1)

$\phi = 12$ mm (assumed)

$\therefore d = 129$ mm

Concise: Table 4.2; BS 8500

$K = M_{Ed}/bd^2f_{ck} = 24.2 \times 10^6/(1000 \times 129^2 \times 30)$

$= 0.048$

$z/d = 0.95$ — Table C5

$z = 0.95 \times 129$

$= 122$ mm

$A_s = M_{Ed}/f_{yd}z$

$= 24.2 \times 10^6/[(500/1.15) \times 122]$

$= 456$ mm^2/m ($\rho = 0.35\%$)

Try H12 @ 250 (452 mm^2/m) ∴ OK)

3.5.4 Check deflection

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ — Appendix C7, Table C10

where

$N = 32.7$

$K = 1.0$ — Table C11

$F1 = 1.0$

$F2 = 1.0$

$F3 = 1.0$ (say)

∴ Allowable $l/d = 32.7$

Actual $l/d = 3450/129$

$= 26.7$ ∴ OK

∴ Provide H12 @ 250 B.

4 Beams

4.0 General

The calculations in this Section are presented in the following parts:

4.1 Continuous beam on pin supports – a simply supported continuous beam showing what might be deemed typical hand calculations.
4.2 A heavily loaded L-beam.
4.3 A continuous wide T-beam. This example is analysed and designed strictly in accordance with the provisions of Eurocode 2.

They are intended to be illustrative of the Code and not necessarily best practice.

A general method of designing beams is shown below. In practice, several of these steps may be combined.

■ Determine design life.	EC0 & NA Table NA.2.1
■ Assess actions on the beam.	EC1 & NAs
■ Assess durability requirements and determine concrete strength.	**Table 4.1** BS 8500–1: Tables A4, A5
■ Check cover requirements for appropriate fire resistance period.	EC2–1–2: Tables 5.8, 5.9, 5.10, 5.11
■ Calculate minimum cover for durability, fire and bond requirements.	**Cl. 4.4.1**
■ Determine which combinations of actions apply.	EC0 & NA Tables NA.A1.1, NA.A1.2 (B)
■ Determine loading arrangements.	**Cl. 5.1.3(1) & NA**
■ Analyse structure to obtain critical moments and shear forces.	**Cl. 5.4, 5.5, 5.6**
■ Design flexural reinforcement.	**Cl. 6.1**
■ Check deflection.	**Cl. 7.4**
■ Check shear capacity.	**Cl. 6.2**
■ Other design checks:	
Check minimum reinforcement.	**Cl. 9.3.1.1(1), 9.2.1.1(1)**
Check cracking (size or spacing of bars).	**Cl. 7.3, Tables 7.2N, 7.3N**
Check effects of partial fixity.	**Cl. 9.3.1.2(2)**
Check secondary reinforcement.	**Cl. 9.3.1.1(2), 9.3.1.4(1)**
■ Check curtailment.	**Cl. 9.3.1.1(4), 9.2.1.3, Fig. 9.2**
■ Check anchorage.	**Cl. 9.3.1.2, 8.4.4, 9.3.1.1(4), 9.2.1.5(1), 9.2.1.5(2)**
■ Check laps.	**Cl. 8.7.3**

4.1 Continuous beam on pin supports

This calculation is intended to show a typical hand calculation for a continuous simply supported beam using coefficients to determine moments and shears.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Continuous beam on pin supports	Calculated by: chg	Job no.: CCIP – 041
		Checked by: web	Sheet no.: 1
		Client: TCC	Date: Oct 09

A 450 mm deep x 300 mm wide rectangular beam is required to support office loads of g_k = 30.2 kN/m and q_k = 11.5 kN/m over 2 no. 6 m spans. f_{ck} = 30 MPa, f_{yk} = 500 MPa. Assume 300 mm wide supports, a 50-year design life and a requirement for a 2-hour resistance to fire in an external but sheltered environment.

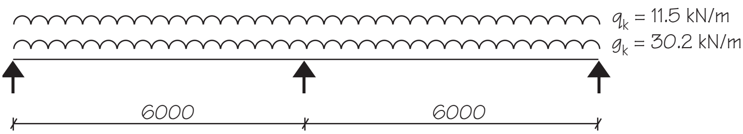


Figure 4.1 Continuous rectangular beam

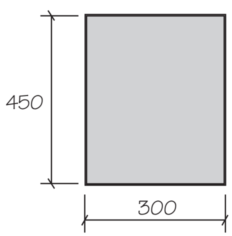


Figure 4.2 Section through beam

4.1.1 Actions

Permanent g_k = 30.2 kN/m and variable q_k = 11.5 kN/m

4.1.2 Cover

Nominal cover, c_{nom}:

$c_{nom} = c_{min} + \Delta c_{dev}$ Exp. (4.1)

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

where

$c_{min,b}$ = minimum cover due to bond Cl. 4.4.1.2(3)

= diameter of bar. Assume 25 mm main bars

$c_{min,dur}$ = minimum cover due to environmental conditions. Assuming XC3 (moderate humidity or cyclic wet and dry) and secondarily XF1 (moderate water saturation without de-icing salt) using C30/37 concrete, | Table C3 BS 8500-1[14]: Table A4; How to: Building structures[8]

$c_{min,dur}$ = 25 mm

Δc_{dev} = allowance in design for deviation. Assuming no measurement of cover, Δc_{dev} = 10 mm | Cl. 4.4.1.2(3)

∴ c_{nom} = 25 + 10 = 35 mm

Fire:

Check adequacy of section for 2 hours fire resistance (i.e. REI = 120) | EC2-1-2: 5.6.3(1), Table 5.6

For b_{min} = 300 mm, minimum axis distance, a = 35 mm ∴ OK

c_{nom} = 35 mm

4.1.3 Load combination (and arrangement)

Load combination:

By inspection, BS EN 1990 Exp. (6.10b) governs | Fig. 2.5

∴ n = 1.25 × 30.2 + 1.5 × 11.5 = 50.8 kN/m | EC0: Exp. (6.10b)

Arrangement:

Choose to use all-and-alternate-spans-loaded load cases, i.e. use coefficients. | Cl. 5.1.3(1) & NA Table NA.1 (option b) Table C3

The coefficients used assume 15% redistribution at supports. As the amount of redistribution is less than 20%, there are no restrictions on reinforcement grade. The use of Table 5.6 in BS EN 1992–1–2 is restricted to where redistribution does not exceed 15%. | Table C3 | Cl. 5.5(4) & NA EC2-1-2: 5.6.3(1), Table 5.6

4.1.4 Analysis

Design moments:

Spans

M_{Ed} = (1.25 × 30.2 × 0.090 + 1.5 × 11.5 × 0.100) × 6.0^2 | Appendix C1, Table C3

= 122.3 + 62.1 = 184.4 kNm

Support

M_{Ed} = 50.8 × 0.106 × 6.0^2 = 193.8 kNm | Table C3

Shear force:

V_{AB} = 0.45 × 6.0 × 50.8 = 137.2 kN

V_{AB} = 0.63 × 6.0 × 50.8 = 192.0 kN

4.1.5 Flexural design

Effective depth:

Assuming 10 mm links:

d = 450 – 35 – 10 – 25/2 = 392 mm

Flexure in span:

$K = M_{Ed}/bd^2f_{ck} = 184.4 \times 10^6/(300 \times 392^2 \times 30) = 0.133$ — Fig. 3.5

$z/d = 0.864$ — Appendix A1 Table C5

$z = 0.864 \times 392 = 338$ mm

$A_s = M_{Ed}/f_{yd}z = 184.4 \times 10^6/(434.8 \times 338) = 1255$ mm^2

Try 3 no. H25 B (1473 mm^2)

($\rho = 1.25\%$)

Check spacing:

Spacing of outer bars = 300 − 2 × 35 − 2 × 10 − 25 = 185 mm

Assuming 10 mm diameter link,

∴ spacing = 98 mm

Steel stress under quasi-permanent loading:

$\sigma_s = (f_{yk}/\gamma_S)(A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$

$= f_{yd} \times (A_{s,req}/A_{s,prov}) \times (g_k + \psi_2 q_k)/(\gamma_G g_k + \gamma_Q q_k)(1/\delta)$

$= (500/1.15) \times (1255/1473) \times [(30.2 + 0.3 \times 11.5)/50.8](1/1.03)$

$= 434.8 \times 0.91 \times 0.66 \times 0.97 = 237$ MPa — Cl. 7.3.3(2)

As exposure is XC3, max. crack width $w_{max} = 0.3$ mm — Cl. 7.3.1(5) & NA

∴ Maximum bar size = 16 mm or max. spacing = 200 mm ∴ OK — Table 7.2N & NA

∴ Use 3 H25 B (1473 mm^2)

Deflection:

Check span-to-effective-depth ratio. — Appendix B

Basic span: effective depth ratio for $\rho = 1.25\%$: — Table 7.4N & NA

$l/d = 18 + [(1.25 - 0.5)/(1.5/0.5)] \times (26 - 18) = 24.0$

Max. span = 24.0 × 392 = 9408 mm ∴ OK

Flexure, support:

$M_{Ed} = 193.8$ kNm

$K = M_{Ed}/bd^2f_{ck}$

where

$d = 450 - 35 - 10 - 25/2 = 392$ mm

$K = 193.8 \times 10^6/(300 \times 392^2 \times 30) = 0.142$

By inspection, $K \leq K'$ ($0.142 \times 0.168^{‡}$)

∴ no compression reinforcement required.

$z = 0.85d$ — Appendix A1 Table C5

$= 0.85 \times 392 = 333$ mm

$A_s = M_{Ed}/f_{yd}z$

$= 193.8 \times 10^6/434.8 \times 333 = 1338$ mm^2

Try 3 no. H25 T (1473 mm^2)

($\rho = 1.13\%$)

‡ K' is limited to 0.208. However, if, as is usual practice in the UK, x/d is limited to 0.45, z/d is as a consequence limited to 0.82 and K' to 0.168.

4.1.6 Shear

a) Support B (critical)

Shear at central support = 192.0 kN

At d from face of support[§]

V_{Ed} = 192.0 – (0.300/2 + 0.392) × 50.8 = 164.50 kN — Cl. 6.2.1(8)

v_{Ed} = V_{Ed}/bd

= 164.5 × 10^3/(392 × 300) = 1.40 MPa

Maximum shear capacity:

Assuming f_{ck} = 30 MPa and cot θ = 2.5[#]

$v_{Rd,max}$* = 3.64 MPa — Table C7

$v_{Rd,max} > v_{Ed}$ ∴ OK

Shear reinforcement:

Assuming $z = 0.9d$ — Cl. 6.2.3(1)

$A_{sw}/s \geq V_{Ed}/(0.9d \times f_{ywd} \times \cot \theta)$ — Cl. 6.2.3(3), Exp. (6.8)

≥ 164.5 × 10^3/(0.9 × 392 × (500/1.15) × 2.5) = 0.429

More accurately,

$A_{sw}/s \geq V_{Ed}/(z \times f_{ywd} \times \cot \theta)$ — Cl. 6.2.3(3), Exp. (6.8)

≥ 164.5 × 10^3/(333 × 1087) = 0.454

Minimum shear links,

$A_{sw,min}/s = 0.08 b_w f_{ck}^{0.5}/f_{yk}$ — Cl. 9.2.2(5)

= 0.08 × 300 × $30^{0.5}$/500 = 0.263. Not critical

Max. spacing = 0.75d = 0.75 × 392 = 294 mm — Cl. 9.2.2(6)

<u>Use H8 @ 200 (A_{sw}/s = 0.50)</u>

b) Support A (and C)

Shear at end support = 137.2 kN

At face of support,

V_{Ed} = 137.2 – (0.150 + 0.392) × 50.8 = 109.7 kN — Cl. 6.2.1(8)

By inspection, shear reinforcement required and cot θ = 2.5 — Fig. C1a)

$A_{sw}/s \geq V_{Ed}/(z \times f_{ywd} \times \cot \theta)$ — Appendix C5.3

≥ 109.7 × 10^3/[353 × (500/1.15) × 2.5] = 0.285

<u>Use H8 @ 200 (A_{sw}/s = 0.50) throughout[‡]</u>

[§] Where applied actions are predominantly uniformly distributed, shear may be checked at d from the face of support. See also Section 4.2.11. — Cl. 6.2.1(8)

[#] The absolute maximum for $v_{Rd,max}$ (and therefore the maximum value of v_{Ed}) would be 5.28 MPa when cot θ would equal 1.0 and the variable strut angle would be at a maximum of 45°.

* For determination of $V_{Rd,max}$ see Section 4.2.10.

[‡] As maximum spacing of links is 294 mm, changing spacing of links would appear to be of limited benefit.

4.1.7 Summary of design

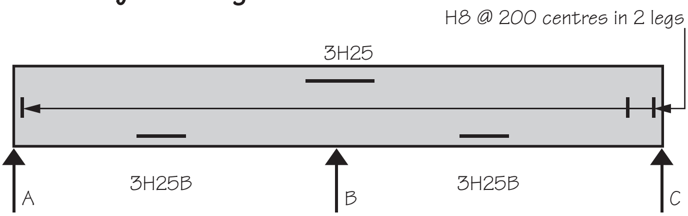


Figure 4.3 Continuous rectangular beam: Summary of design

Commentary

It is usually presumed that the detailer would take the design summarised above and detail the beam to normal best practice[8,9]. The design would go no further where standard detailing is all that is required. Where the element is non-standard (e.g. where there are point loads), it should be incumbent on the designer to give the detailer specific information about curtailment, laps, etc. as illustrated below. The detailer's responsibilities, standards and timescales should be clearly defined but it would be usual for the detailer to draw and schedule not only the designed reinforcement but all the reinforcement required to provide a compliant and buildable solution. The work would usually include the checking the following aspects and providing appropriate detailing:

- Minimum areas
- Curtailment lengths
- Anchorages
- Laps
- U-bars
- Rationalisation
- Critical dimensions
- Details and sections

The determination of minimum reinforcement areas and curtailment lengths, using the principles in Eurocode 2 is shown below. In practice these would be determined from published tables of data or by using reference texts[8,9]. Nonetheless, the designer should check the drawing for design intent and compliance with the standards. It is therefore necessary for the designer to understand and agree the principles of the detailing used.

4.1.8 Detailing checks

a) Minimum areas

$A_{s,min} = 0.26(f_{ctm}/f_{yk})b_t d \geq 0.0013 b_t d$ Cl. 9.2.1.1

where

b_t = width of tension zone | Table 3.1

$f_{ctm} = 0.30 \times f_{ck}^{0.666}$

$A_{s,min} = 0.26 \times 0.30 \times 30^{0.666} \times 300 \times 392/500 = 177\ \text{mm}^2$

b) Curtailment of main bars | How to: Detailing

Bottom: curtail

75% main bars $0.08l$ from end support = 480 mm say 450 mm from A

70% main bars $0.30l - a_l = 0.3 \times 6000 - 1.125 \times d$

$= 1800 - 1.125 \times 392$

$= 1359$ mm say 1350 from A

Top: curtail

40% main bars $0.15l + a_l = 900 + 441$

$= 1341$ mm say 1350 from B

65% main bars $0.30l + a_l = 1800 + 441$

$= 2241$ mm say 2250 from B

At supports: | Cl. 9.2.1.2.(1), 9.2.1.4(1) & NA

25% of A_s to be anchored at supports

25% of 1225 mm^2 = 314 mm^2 | Cl. 9.2.1.5(1)

Use min. 2 no. H16 (402 mm^2) at supports A, B and C

In accordance with SMDSC[9] detail MB1 lap U-bars tension lap with main steel

= 780 mm (in C30/37 concrete, H12, 'poor' bond condition) | How to: Detailing

= say 800 mm

c) Summary of reinforcement details

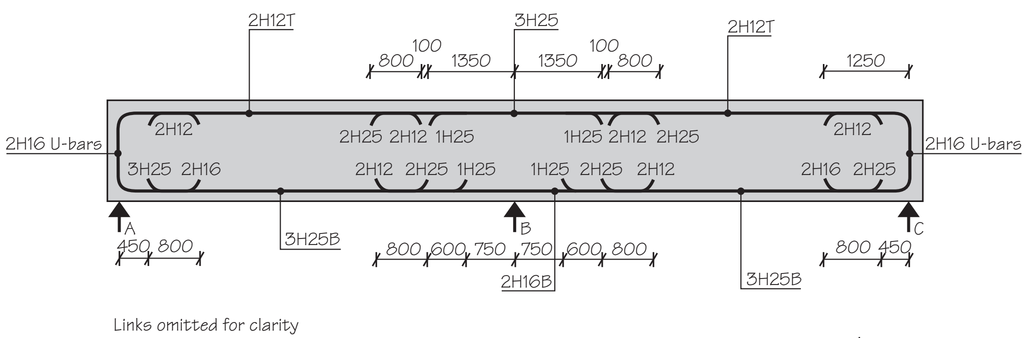


Figure 4.4 Continuous rectangular beam: reinforcement details

Note Subsequent detailing checks may find issues with spacing rules especially if the 'cage and splice bar' method of detailing were to be used. 2H32s T&B would be a suitable alternative to 3H25s T&B.

4.2 Heavily loaded L-beam

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Heavily loaded L-beam	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

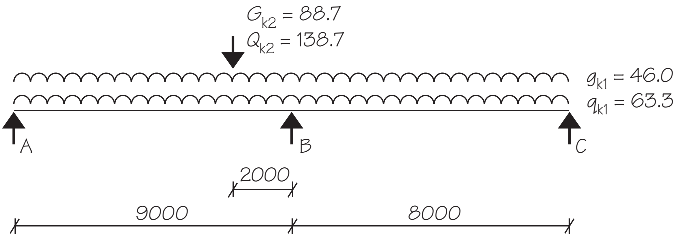


Figure 4.5 Heavily loaded L-beam

This edge beam supports heavy loads from storage loads. The variable point load is independent of the variable uniformly distributed load. The beam is supported on 350 mm square columns 4000 mm long. f_{ck} = 30 MPa; f_{yk} = 500 MPa. The underside surface is subject to an external environment and a 2-hour fire resistance requirement. The top surface is internal and subject to a 2-hour fire resistance requirement. Assume that any partitions are liable to be damaged by excessive deflections.

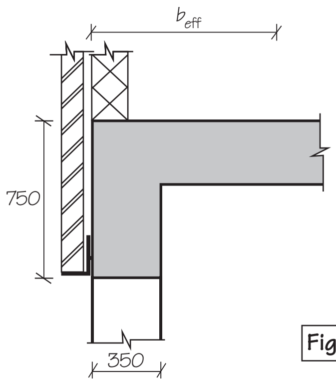


Figure 4.6 Section through L-beam

4.2.1 Actions

Permanent:

UDL from slab and cladding g_k = 46.0 kN/m

Point load from storage area above = 88.7 kN

Variable:

From slab q_k = 63.3 kN/m

Point load from storage area above = 138.7 kN

4.2.2 Cover

a) Nominal cover, c_{nom}, underside and side of beam

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

where

$c_{min,b}$ = minimum cover due to bond — Cl. 4.4.1.2(3)
= diameter of bar. Assume 32 mm main bars and 10 mm links

$c_{min,dur}$ = minimum cover due to environmental conditions. Assuming primarily XC3/XC4 exposure (moderate humidity or cyclic wet and dry); secondarily XF1 exposure (moderate water saturation without de-icing salt, vertical surfaces exposed to rain and freezing) and C30/37 concrete, — Table 4.1 BS 8500-1: Table A4

$c_{min,dur}$ = 25 mm

Δc_{dev} = allowance in design for deviation. Assuming no measurement of cover Δc_{dev} = 10 mm — Cl. 4.4.1.2(3)

$\therefore c_{nom}$ = 32 + 10 = 42 mm to main bars
or = 25 + 10 = 35 mm to links

Use c_{nom} = 35 mm to links (giving c_{nom} = 45 mm to main bars)

b) Fire

Check adequacy of section for 2 hours fire resistance REI 120. — EC2-1-2: 5.6.3
By inspection, web thickness OK. — EC2-1-2: Table 5.6

Axis distance, a, required = 35 mm OK by inspection. — EC2-1-2: Table 5.6

$\therefore$ Try 35 mm nominal cover bottom and sides to 10 mm link.

Nominal cover, c_{nom}, top:
By inspection,

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

where

$c_{min,b}$ = minimum cover due to bond — Cl. 4.4.1.2(3)
= diameter of bar. Assume 32 mm main bars and 10 mm links

$c_{min,dur}$ = minimum cover due to environmental conditions. Assuming primarily XC1 and C30/37 concrete, — Table 4.1 BS 8500-1: Table A4
$c_{min,dur}$ = 15 mm

Δc_{dev} = allowance in design for deviation. Assuming no measurement of cover Δc_{dev} = 10 mm — Cl. 4.4.1.2(3)

$\therefore\ c_{nom}$ = 32 + 10 = 42 mm to main bars

or = 15 + 10 = 25 mm to links

Use c_{nom} = 35 mm to links (giving c_{nom} = 45 mm to main bars)

4.2.3 Idealisation, load combination and arrangement

Load combination: — Table 2.5; ECO: A1.2.2, NA & Exp. (6.10a)

As loads are from storage, Exp. (6.10a) is critical.

Idealisation:

This element is treated as a continuous beam framing into columns 350 × 350‡ × 4000 mm long columns below. — Cl. 5.3.1(3)

Arrangement: — Cl. 5.1.3(1) & NA: Table NA.1 (option b)

Choose to use all-and-alternate-spans-loaded.

4.2.4 Analysis

Analysis by computer (spreadsheet TCC 41 Continuous Beam (A+D).xls in *RC spreadsheets V.3*[28] assuming frame action with 350 mm square columns 4 m long fixed at base. Beam inertia based on T-section, b_{eff} wide) with 15% redistribution at central support, limited redistribution of span moment and consistent redistribution of shear. — ECO: A1.2.2 & NA; Cl. 5.3.1 (6)

Table 4.2 Elastic and redistributed moments, kNm

Span number	1	2
Elastic *M*	1168	745
Redistributed *M*	1148	684
δ	0.98	0.92

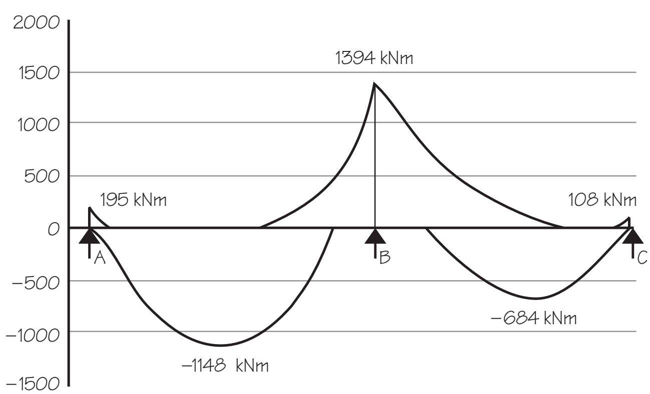


Figure 4.7 Redistributed envelope, kNm

‡ Note: 350 × 350 is a minimum for columns requiring a fire resistance of 120 minutes. — EC2-1-2: Table 5.2a

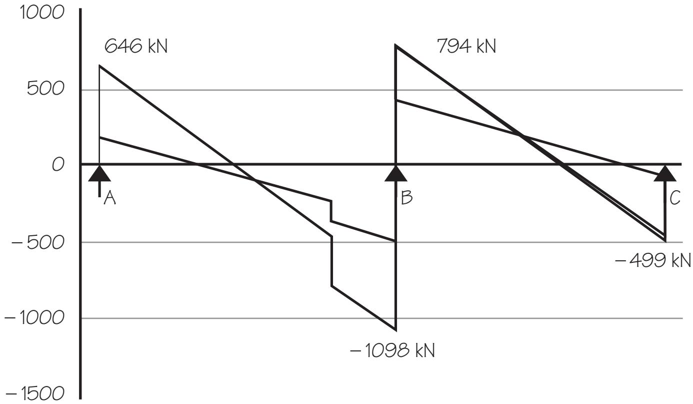


Figure 4.8 Redistributed shears, kN

4.2.5 Flexural design, support A

M_{Ed} = 195 kNm in hogging	Cl. 9.2.1.2(1), 9.2.1.4(1) & NA
$M_{Ed,min}$ = 1148 × 0.25 in hogging and in sagging	
= 287 kNm	
$K = M_{Ed}/bd^2 f_{ck}$	
where	Cl. 5.3.2.1, Fig. 5.3
$b = b_{eff} = b_{eff1} + b_w + b_{eff2}$	
where	
$b_{eff1} = (0.2b_1 + 0.1l_0) \leq 0.2\, l_0 \leq b_1$	
where	
b_1 = distance between webs/2	
l_0 = nominal: assume 0§	
$\therefore\ b_{eff1} = 0$ mm $= b_{eff2}$	Fig. 5.2
$\therefore b = b_w = 350$ mm	
$d = 750 - 35 - 10 - 32/2 = 689$ mm	
assuming 10 mm link and H32 in support.	
$f_{ck} = 30$ MPa	
$K = 287 \times 10^6/(350 \times 689^2 \times 30)$	
= 0.058	
Restricting x/d to 0.45	Appendix A1
$K' = 0.168$	
$K \leq K'$ ∴ section under-reinforced and no compression reinforcement required.	
$z = (d/2)\,[1 + (1 - 3.53K)^{0.5}] \leq 0.95d$	Appendix A1
$= (689/2)\,(1 + 0.89) \leq 0.95 \times 689$	
$= 652 \leq 654\ \therefore\ z = 652$ mm	
$A_s = M_{Ed}/f_{yd}z$	

§ The distance l_0 is described as the distance between points of zero moment, 'which may be obtained from Figure 5.2'. In this case $l_0 = 0$. (see Figure 4.11).

Cl. 5.3.2.1(2)
Fig. 5.2
Fig. 4.11

where

f_{yd} = 500/1.15 = 434.8 MPa

= 287 × 10^6/(434.8 × 652) = 1012 mm^2

Try 2 no. H32 U-bars (1608 mm^2)

Check anchorage of H32 U-bars.

Bars need to be anchored distance 'A' into column | SMDSC: 6.4.2

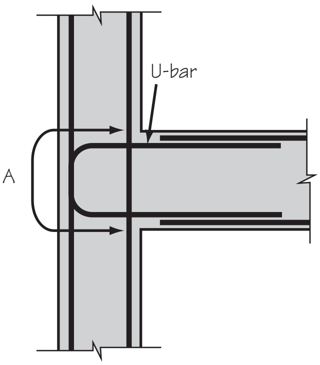


Figure 4.9 Distance A

Assuming column uses 35 mm cover, 10 mm links and 32 mm bars:

Distance A = 2 [350 – 2 (35 + 10)] – 32/2 – 32/2 + 750 – [2 (35 + 10)] – 2 × 32/2 – (4 – π) (3.5 + 0.5) × 32 | SMDSC[9], BS 8666[19]: Table 2

= 488 + 628 – 110 = 1006 mm

Anchorage length, | Cl. 8.4.4, Exp. (8.4)

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

where

α = conservatively 1.0

$l_{b,rqd} = (\phi/4)\ (\sigma_{sd}/f_{bd})$ | Exp. (8.3)

where

ϕ = 32

σ_{sd} = design stress in the bar at the ULS

= 434.8 × 1012/1608 = 274 MPa

f_{bd} = ultimate bond stress

= 2.25 $\eta_1\ \eta_2\ f_{ct,d}$ | Cl. 8.4.2 (2)

where

η_1 = 1.0 for good bond conditions

η_2 = 1.0 for bar diameter ≤ 32 mm

$f_{ct,d} = \alpha_{ct} f_{ctk}/\gamma_C$ — Cl. 3.1.6 (2), Tables 3.1 & 2.1, & NA

= 1.0 × 2.0/1.5

= 1.33 MPa

f_{bd} = 2.25 × 1.33 = 3.0 MPa

$l_{b,rqd}$ = (32/4) (274/3.0) = 731 mm‡

$l_{b,min}$ = max[10ϕ; 100 mm] = 250 mm

∴ l_{bd} = 731 mm i.e. < 1006 mm — ∴ OK

Use 2 no. H32 U-bars

4.2.6 Flexural design, span AB

a) Span AB – Flexure

M_{Ed} = 1148 kNm

$K = M_{Ed}/bd^2f_{ck}$

where — Cl. 5.3.2.1, Fig. 5.3

$b = b_{eff} = b_{eff1} + b_w + b_{eff2}$

where

$b_{eff1} = (0.2b_1 + 0.1l_O) \le 0.2\,l_O \le b_1$

where

b_1 = distance between webs/2

Assuming beams at 7000 mm cc

= (7000 – 350)/2 = 3325 mm

$l_O = 0.85 \times l_1 = 0.85 \times 9000 = 7650$ mm§ — Fig. 5.2

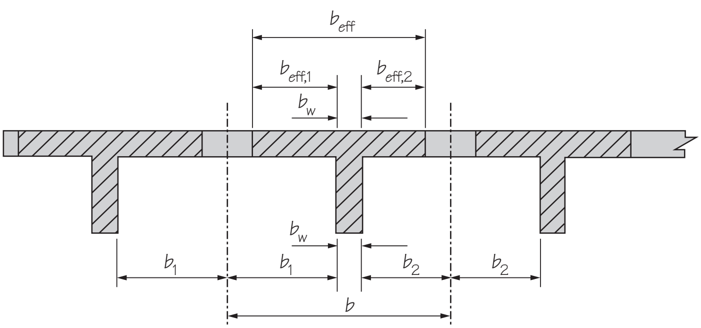


Figure 4.10 Effective flange width b_{eff} — Fig. 5.3

‡ Anchorage lengths may be obtained from published tables. In this instance, a figure of 900 mm may be obtained from Table 13 of Section 10 of *How to design concrete structures using Eurocode 2*. — How to: Detailing[8]

§ The distance l_O is described as the distance between points of zero shear, which may be obtained from Figure 5.2'. From the analysis, l_O could have been taken as 7200 mm. — Cl. 5.3.2.1(2) Figure 5.2

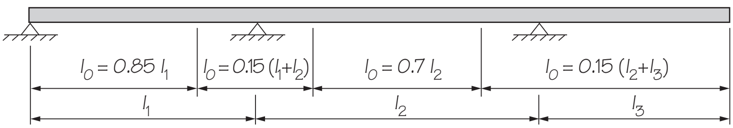


Figure 4.11 Elevation showing definition of l_0 for calculation of flange width

Fig. 5.2

b_{eff1} = $0.2 \times 3325 + 0.1 \times 7650 \le 0.2 \times 7650 \le 3325$
= $1430 \le 1530 \le 3325$
= 1430 mm

b_w = 350 mm

b_{eff2} = $(0.2b_2 + 0.1l_0) \le 0.2\ l_0 \le b_2$

where

b_2 = 0 mm

b_{eff2} = 0 mm

b = 1430 + 350 + 0 = 1780 mm

d = effective depth
= 750 – 35 – 10 – 32/2 = 689 mm
assuming 10 mm link and H32 in span

f_{ck} = 30 MPa

$K = 1148 \times 10^6/(1780 \times 689^2 \times 30)$
$= 0.045$

Restricting x/d to 0.45, — Appendix A1

$K' = 0.168$

$K \le K'$ ∴ section under-reinforced and no compression reinforcement required.

z = lever arm — Appendix A1
$= (d/2)\ [1 + (1 - 3.53K)^{0.5}] \le 0.95d$
$= (689/2)\ (1 + 0.917) \le 0.95 \times 689$
$= 661 \le 654$ ∴ $z = 654$ mm

But $z = d - 0.4x$

∴ by inspection, neutral axis is in flange and as $x < 1.25\ h_f$, design as rectangular section. — Appendix A1

$A_s = M_{Ed}/f_{yd}z$

where

f_{yd} = 500/1.15 = 434.8 MPa
= $1148 \times 10^6/(434.8 \times 654) = 4037\ mm^2$

Try 5 no. H32 B (4020 mm²) (say OK)

Check spacing of bars.

Spacing of bars = [350 – 2 × (35 + 10) – 32]/(5 – 1)
= 57

Clear spacing = 57 – 32 mm = 25 mm between bars

Minimum clear distance between bars | Cl. 8.2(2) & NA

= max[bar diameter; aggregate size + 5 mm]

= max[32; 20 + 5]

= 32 mm i.e. > 25 mm

∴ 5 no. H32 B no good

For 4 bars in one layer, distance between bars = 44 mm so

Try 4 no. H32 B1 + 2 no. H32 B3

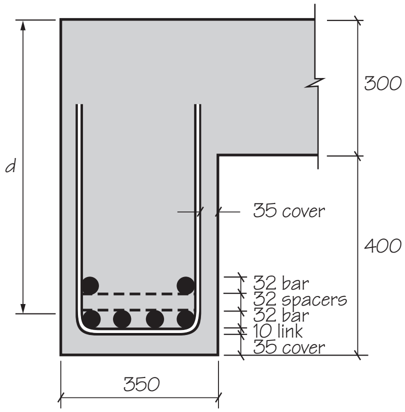


Figure 4.12 Span AB bottom reinforcement

$d = 750 - 35 - 10 - 32/2 - 0.333 \times 2 \times 32 = 668$ mm

$K = 1148 \times 10^6/(1780 \times 668^2 \times 30) = 0.048$

$K \leq K'$ ∴ section under-reinforced and no compression reinforcement required. | Appendix A1

$z = (d/2)\ [1 + (1 - 3.53K)^{0.5}] \leq 0.95d$ | Appendix A1

$= (668/2)\ (1 + 0.911) \leq 0.95 \times 668$

$= 639 \leq 635$ ∴ $z = 635$ mm

∴ by inspection, neutral axis in flange so design as rectangular section. | Appendix A1

$A_s = M_{Ed}/f_{yd}z$

$= 1148 \times 10^6/(434.8 \times 635) = 4158\ \text{mm}^2$

∴ 4 no. H32 B1 + 2 no. H32 B3 (4824 mm²) OK

b) Span AB – Deflection

Check end span-to-effective-depth ratio. | Appendix B

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ | Appendix C7

where

N = Basic l/d: check whether $\rho > \rho_0$ and whether to use Exp. (7.16a) or (7.16b) | **Cl. 7.4.2(2), Exp. (7.16a), Exp. (7.16b)**

$\rho = A_s/A_c{}^{‡} = A_{s,req}/[b_w d + (b_{eff} - b_w)h_f]$ $= 4158/[350 \times 668 + (1780 - 350) \times 300]$ $= 4158/662800$ $= 0.63\%$ $\rho_0 = f_{ck}{}^{0.5}/1000 = 30^{0.5}/1000 = 0.55\%$ $\rho > \rho_0$ ∴ use Exp. (7.16b)	PD 6687[6]
$N = 11 + 1.5 f_{ck}{}^{0.5}\ \rho_0/(\rho - \rho') + f_{ck}{}^{0.5}(\rho'/\rho_0)^{0.5}/12$ $= 11 + 1.5\ (30^{0.5} \times 0.055/(0.063 - 0) + 30^{0.5}(0/0.55)^{1.5}$ $= 11 + 7.2 + 0 = 18.2$	**Exp. (7.16b)**
K = (end span) = 1.3	**Table 7.4N & NA**
F1 = (b_{eff}/b_w = 1780/350 = 5.1) = 0.80	**Cl. 7.4.2(2),** Appendix C7
F2 = 7.0/l_{eff} (span > 7.0 m)	**Cl. 7.4.2(2)**
where l_{eff} = 9000 mm	**Cl. 5.3.2.2(1)**
F2 = 7.0/9.0 = 0.77	
F3 = 310/σ_s ≤ 1.5	**Cl. 7.4.2, Exp. (7.17), Table 7.4N & NA, Table NA.5 Note 5**
where σ_s in simple situations = $(f_{yk}/\gamma_S)\ (A_{s,req}/A_{s,prov})$ (SLS loads/ ULS loads) (1/δ). However in this case separate analysis at SLS would be required to determine σ_s. Therefore as a simplification use the conservative assumption:	Appendix B
$310/\sigma_s = (500/f_{yk})\ (A_{s,req}/A_{s,prov})$ $= (500/500) \times (4824/4158) = 1.16$ ∴ Permissible l/d = 18.2 × 1.3 × 0.80 × 0.77 × 1.16 = 16.9 Actual l/d = 9000/668 = 13.5 Permissible more than actual ∴ OK ∴ 4 no. H32 B1 + 2 no. H32 B3 (4824 mm^2) OK	**Exp. (7.17)**

4.2.7 Flexural design, support B

At centreline of support B, M = 1394 kNm	
From analysis, at face of support M_{EdBA} = 1209 kNm M_{EdBC} = 1315 kNm $K = M_{Ed}/b_w d^2 f_{ck}$	**Cl. 5.3.2.2(3)**

‡ 2.18 of PD 6687[6] suggests that ρ in T sections should be based on the area of concrete above the centroid of the tension steel.

where

$b_w = 350$ mm

$d = 750 - 35 - 12 - 32/2 = 687$ mm

assuming 10 mm link and H32 in support but allowing for H12 T in slab

$f_{ck} = 30$ MPa

$\therefore K = 1315 \times 10^6/(350 \times 687^2 \times 30) = 0.265$

for $\delta = 0.85$, $K' = 0.168$: to restrict x/d to 0.45, $K' = 0.167$ — Appendix A1 Table C4

$\therefore$ Compression steel required

$z = (d/2)\,[1 + (1 - 3.53\,K')^{0.5}]$

$= (687/2)\,[1 + (1 - 3.53 \times 0.167)^{0.5}]$

$= (687/2)\,(1 + 0.64) < 0.95d$

$= 563$ mm

$A_{s2} = (K - K')f_{ck}bd^2/f_{sc}(d - d_2)$ — **Fig. 3.5,** Appendix A1, **How to: Beams**

where

$d_2 = 35 + 10 + 32/2 = 61$ mm

$f_{sc} = 700(x - d_2)/x < f_{yd}$

where

$x = 2.5\,(d - z) = 2.5\,(687 - 563) = 310$ mm

$f_{sc} = 700 \times (310 - 61)/310 < 500/1.15$

$= 562$ MPa but limited to ≤ 434.8 MPa

$\therefore A_{s2} = (0.265 - 0.167) \times 30 \times 350 \times 687^2/[434.8(687 - 61)\,] = 1784\text{ mm}^2$

Try 4 no. H25 B (1964 mm²)

$A_s = M'/f_{yd}z + A_{s2}\,f_{sc}/f_{yd}$ — Appendix A1

$= K'f_{ck}bd^2/(f_{yd}z) + A_{s2}\,f_{sc}/f_{yd}$

$= 0.167 \times 30 \times 350 \times 687^2/(434.8 \times 563) + 1570 \times 434.8/434.8$

$= 3380 + 1784 = 5164\text{ mm}^2$

Try 4 no. H32 T + 4 no. H25 T (5180 mm²)

This reinforcement should be spread over b_{eff}. — **Cl. 9.2.1.2(2), Fig. 9.1, Cl. 5.3.2.1, Fig. 5.3**

$b_{eff} = b_{eff1} + b_w + b_{eff2}$

where

$b_{eff1} = (0.2b_1 + 0.1l_0) \leq 0.2\,l_0 \leq b_1$

where

b_1 = distance between webs/2.

Assuming beams at 7000 mm cc

$= (7000 - 350)/2 = 3325$ mm

$l_0 = 0.15 \times (l_1 + l_2)$ — **Fig. 5.2**

$= 0.15 \times (9000 + 8000) = 2550$ mm

$\therefore b_{eff1} = 0.2 \times 3325 + 0.1 \times 2550 \leq 0.2 \times 2550 \leq 3325$

$= 920 \leq 510 \leq 3325$

$= 510$ mm

$b_w = 350$ mm

$b_{eff2} = (0.2b_2 + 0.1l_0) \leq 0.2\, l_0 \leq b_2$

where

$b_2 = 0$ mm

$b_{eff2} = 0$ mm

$\therefore\ b_{eff} = 510 + 350 + 0 = 860$ mm

Use 4 no. H32 T + 4 no. H25 T (5180 mm^2) @ approx 100 mm cc

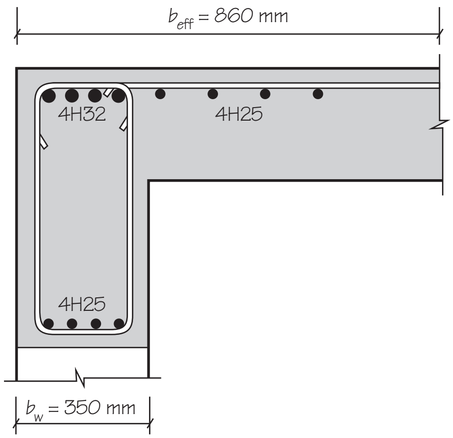


Figure 4.13 Support B reinforcement

4.2.8 Flexural design, span BC

a) Span BC – Flexure

$M_{Ed} = 684$ kNm

$K = M_{Ed}/bd^2f_{ck}$

where

$b = b_{eff} = b_{eff1} + b_w + b_{eff2}$ Cl. 5.3.2.1, Fig. 5.3

where

$b_{eff1} = (0.2b_1 + 0.1l_0) \leq 0.2\, l_0 \leq b_1$

where

b_1 = distance between webs/2.
Assuming beams at 7000 mm cc

$= (7000 - 350)/2 = 3325$ mm Fig. 5.2

$l_0 = 0.85 \times l_1 = 0.85 \times 8000 = 6800$ mm

$b_{eff1} = 0.2 \times 3325 + 0.1 \times 6800 \leq 0.2 \times 6800 \leq 3325$

$= 1345 \leq 1360 \leq 3325$

$= 1360$ mm

b_w = 350 mm
$b_{eff2} = (0.2b_2 + 0.1l_0) \le 0.2\, l_0 \le b_2$
where
b_2 = 0 mm
b_{eff2} = 0 mm
$\therefore\ b = 1360 + 350 + 0 = 1710$ mm
$d = 750 - 35 - 10 - 32/2 = 689$ mm
assuming 10 mm link and H32 in span.
$f_{ck} = 30$ MPa
$\therefore\ K = 684 \times 10^6 / (1710 \times 689^2 \times 30)$
$= 0.028$

By inspection, $K \le K'$ ∴ section under-reinforced and no compression reinforcement required. — Appendix A1

$z = (d/2)\,[1 + (1 - 3.53K)^{0.5}] \le 0.95d$ — Appendix A1
$= (689/2)\,(1 + 0.95) \le 0.95 \times 689$
$= 672 > 655\ \therefore\ z = 655$ mm

By inspection, $x < 1.25\, h_f$; design as rectangular section — Appendix A1

$A_s = M_{Ed}/f_{yd}z$
$= 684 \times 10^6/(434.8 \times 655) = 2402\ \text{mm}^2$
Try 2 no. H32 B + 2 no. H25 B (2590 mm^2)

b) Span BC – Deflection
By inspection, compared with span AB — OK

4.2.9 Flexural design, support C
By inspection, use 2 no. H25 U-bars as support A.
Use 2 no. H25 U-bars

4.2.10 Design for beam shear, support A
At d from face of support
$V_{Ed} = 646 - (350/2 + 0.689) \times (1.35 \times 46.0 + 1.5 \times 63.3)$ — Cl. 6.2.1(8); EC0: A1.2.2, NA & Exp. (6.10a)
$= 646 - 0.864 \times 157.1 = 510.3$ kN
Check maximum shear resistance.
$V_{Rd,\,max} = \alpha_{cw}\, b_w\, z \nu f_{cd}/(\cot\theta + \tan\theta)$ — Exp. (6.9) & NA
where
$\alpha_{cw} = 1.0$ — Cl. 6.2.3 & NA
b_w = 350 mm as before
$z = 0.9d$ — Cl. 6.2.3(1)

$\nu = 0.6\ (1 - f_{ck}/250) = 0.6\ (1 - 30/250) = 0.528$	Cl. 6.2.3(3) Note 1, Exp. (6.6N) & NA
$f_{cd} = 30/1.5 = 20.0$ MPa	Cl. 2.4.2.4(1) & NA
θ = angle of inclination of strut. $= 0.5 \sin^{-1} \{v_{Ed,z}/[0.20\ f_{ck}\ (1 - f_{ck}/250)\ ]\} \geq \cot^{-1} 2.5$ where $v_{Ed,z} = V_{Ed}/b_z = V_{Ed}/(b \times 0.9d)$ $= 510.3 \times 10^3/(350 \times 0.9 \times 689) = 2.35$ MPa $\theta = 0.5 \sin^{-1} \{2.35/[0.20 \times 30\ (1 - 30/250)\ ]\} \geq \cot^{-1} 2.5$ $= 0.5 \sin^{-1} (0.445) \geq \cot^{-1} 2.5$ $= 0.5 \times 26.4° \geq 21.8°$ $= 21.8°$	Exp. (6.9), Appendix A2
$\therefore V_{Rd,max} = 1.0 \times 350 \times 0.90 \times 689 \times 0.528 \times 20.0/(2.5 + 0.4) = 790$ kN $\therefore$ OK	
Shear reinforcement: Shear links: shear resistance with links	
$V_{Rd,s} = (A_{sw}/s)\ z\ f_{ywd} \cot \theta$ $\therefore A_{sw}/s \geq V_{Ed}/z\ f_{ywd} \cot \theta$	Exp. (6.8)
where A_{sw}/s = area of legs of links/link spacing $z = 0.9d$ as before $f_{ywd} = 500/1.15 = 434.8$ $\cot \theta = 2.5$ as before	Cl. 2.4.2.4(1) & NA
$A_{sw}/s \geq 510.3 \times 10^3/(0.9 \times 689 \times 434.8 \times 2.5) = 0.76$	
Minimum $A_{sw}/s = \rho_{w,min} b_w \sin \alpha$	Cl. 9.2.2(5), Exp. (9.4)
where $\rho_{w,min} = 0.08 \times f_{ck}^{0.5}/f_{yk} = 0.08 \times 30^{0.5}/500$ $= 0.00088$ $b_w = 350$ mm as before α = angle between shear reinforcement and the longitudinal axis. For vertical reinforcement $\sin \alpha = 1.0$	Exp. (9.5N) & NA
$\therefore$ Minimum $A_{sw}/s = 0.00088 \times 350 \times 1 = 0.03$ But,	
maximum spacing of links longitudinally $= 0.75d = 516$ mm $\therefore$ Try H10 @ 200 cc in 2 legs ($A_{sw}/s = 0.78$)	Cl. 9.2.2(6)
4.2.11 Design for high beam shear, support B	
As uniformly distributed load predominates consider at d from face of support.	Cl. 6.2.1(8)

V_{Ed} = 1098 − (350/2 + 0.689) × (1.35 × 46.0 + 1.5 × 63.3)
= 1098 − 0.864 × 157.1 = 962.3 kN

By inspection, shear reinforcement required and cot θ < 2.5.

Check $V_{Rd,\,max}$ (to determine θ)

Check maximum shear resistance.

As before,

$V_{Rd,\,max} = \alpha_{cw}\, b_w\, z\nu f_{cd}/(\cot\theta + \tan\theta)$. — Exp. (6.9) & NA

where

α_{cw}, b_w, z, ν and f_{cd} as before

$\theta = 0.5 \sin^{-1} \{v_{Ed,z}/[0.20\, f_{ck}\, (1 - f_{ck}/250)\,]\,\} \geq \cot^{-1}2.5$ — Exp. (6.9)

where

$v_{Ed,z} = V_{Ed}/bz = V_{Ed}/(b0.9d)$ — Cl. 6.2.3(1)

$= 962.3 \times 10^3/(350 \times 0.9 \times 687) = 4.45$ MPa

$\theta = 0.5 \sin^{-1} \{4.45/[0.20 \times 30\, (1 - 30/250)\,]\,\} \geq \cot^{-1}2.5$ — Exp. (6.9)

$= 0.5 \sin^{-1} (0.843) \geq \cot^{-1}2.5$

$= 0.5 \times 57.5° \geq 21.8°$

$= 28.7°$

cot θ = 1.824 i.e. > 1.0 ∴ OK — Cl. 6.2.3(2) & NA

tan θ = 0.548

∴ $V_{Rd,max}$ = 1.0 × 350 × 0.90 × 687 × 0.528 × 20.0/(1.824 + 0.548)
= 963.4 kN

OK
(i.e. $V_{Rd,max} \approx V_{Ed}$)

Shear reinforcement:

Shear links: shear resistance with links

$V_{Rd,s} = (A_{sw}/s) z f_{ywd} \cot\theta$ — Exp. (6.8)

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

$A_{sw}/s \geq 962.3 \times 10^3/(0.9 \times 687 \times 434.8 \times 1.824) = 1.96$

∴ Use H10 @ 150 cc in 4 legs (A_{sw}/s = 2.09)

4.2.12 Design for beam shear (using design chart), support B_C

At d from face of support, — Cl. 6.2.1(8)

V_{Ed} = 794 − 0.864 × 157.1 = 658.3 kN

$v_{Ed,z} = V_{Ed}/bz = V_{Ed}/(b0.9d)$

$= 658.3 \times 10^3/\,(350 \times 0.9 \times 687) = 3.04$ MPa

From chart A_{sw}/s_{reqd}/m width = 2.75 — Fig. C1b)

$A_{sw}/s_{reqd} = 2.75 \times 0.35 = 0.96$

∴ Use H10 in 2 legs @ 150 mm cc (A_{sw}/s = 1.05)

4.2.13 Check shear capacity for general case

In mid span use H10 in 2 legs @ 300 mm cc ($A_{sw}/s = 0.52$)

$\equiv A_{sw}/s_{reqd}$/m width = 1.48 and an allowable $v_{Ed,z}$ = 1.60 MPa

$\equiv 1.60 \times 350 \times 0.90 \times 687 = V_{Ed} = 346$ kN

Fig. C1b)

From analysis, V_{Ed} = 346.2 kN occurs at:

(646 – 346)/157.1 = 1900 mm from A,

(1098 – 346 – 1.25 × 88.7 – 1.5 × 138.7)/157.1 = 2755 mm from B_A,

(794 – 346)/157.1 = 2850 mm from B_C

and

(499 – 346)/157.1 = 970 mm from C

4.2.14 Summary of design

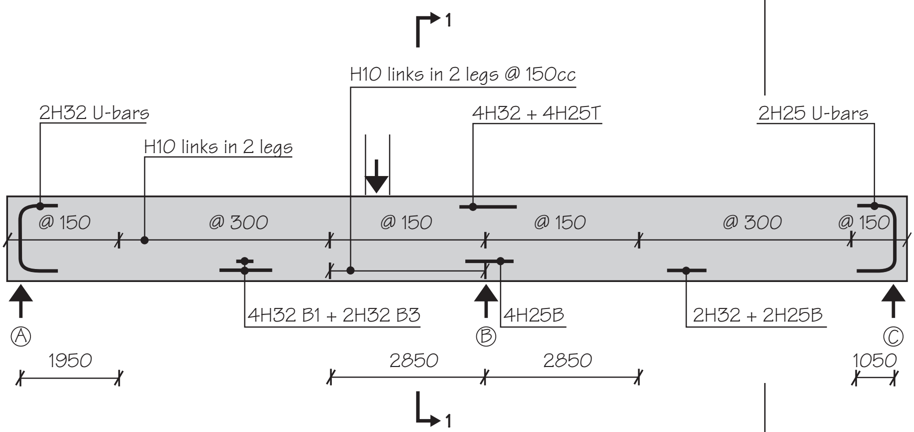


Figure 4.14 Summary of L-beam design

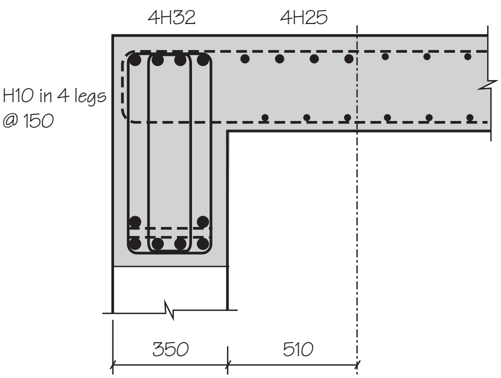


Figure 4.15 L-beam section 1–1

4.3 Continuous wide T-beam

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Continuous wide T-beam	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

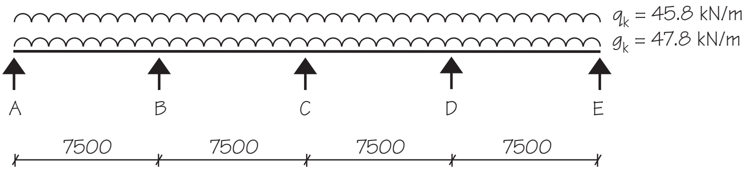


Figure 4.16 Continuous wide T-beam

This central spine beam supports the ribbed slab in Example 3.3. The 300 mm deep ribbed slab is required for an office to support a variable action of 5 kN/m^2. The beam is the same depth as the slab and is supported on 400 mm square columns, see Figure 4.17. f_{ck} = 35 MPa; f_{yk} = 500 MPa. A 1-hour fire resistance is required in an internal environment. Assume that partitions are liable to be damaged by excessive deflections.

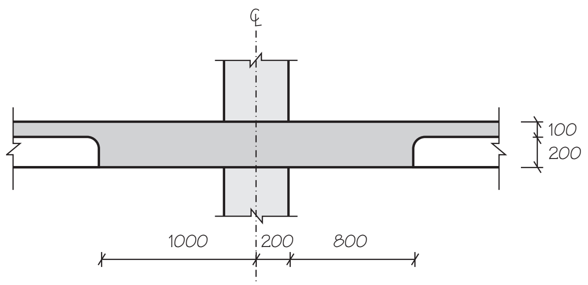


Figure 4.17 Section through T-beam

4.3.1 Actions

Permanent, UDL‡:

From analysis of slab, g_k = 47.8 kN/m

Variable:

From analysis of slab, q_k = 45.8 kN/m

‡ The actions may also have been estimated assuming an elastic reaction factor of 1.1 for the slab viz:

	kN/m
Permanent: UDL	
Loads from ribbed slab (7.50 + 9.0)/2 × 4.30 × 1.1 =	39.0
Self-weight/patch load extra over solid 2.0 × 4.17 =	8.3
	47.3
Variable:	
Imposed (7.50 + 9.0)/2 × 5.00 × 1.1 =	45.4

4.3.2 Cover

Nominal cover, c_{nom}:

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

where

$c_{min,b}$ = minimum cover due to bond — Cl. 4.4.1.2(3)

= diameter of bar. Assume 25 mm main bars and 8 mm links — Table 4.1

$c_{min,dur}$ = minimum cover due to environmental conditions. Assuming XC1 and C30/37 concrete, $c_{min,dur}$ = 15 mm — BS 8500-1; Table A4

Δc_{dev} = allowance in design for deviation. Assuming no measurement of cover Δc_{dev} = 10 mm — Cl. 4.4.1.2(3)

∴ c_{nom} = 15 + 10 = 25 mm to links

or = 25 + 10 = 35 mm to main bars

Use 10 mm diameter links to give c_{nom} = 35 mm to main bars and 25 mm to links (as per ribbed slab design).

Fire: — EC2-1-2: 5.6.3

Check adequacy of section for REI 60. — EC2-1-2: Table 5.6

Axis distance required: — EC2-1-2: Table 5.6

Minimum width b_{min} = 120 mm with a = 25 mm

or b_{min} = 200 mm with a = 12 mm

∴ at 2000 mm wide (min.) a < 12 mm

By inspection, not critical.

<u>Use 25 mm nominal cover to links</u>

4.3.3 Idealisation, load combination and arrangement

Load combination:

By inspection, Exp. (6.10b) is critical. — Fig. 2.5

47.8 × 1.25 + 45.8 × 1.5 = 128.5 kN/m‡ — EC0: Exp. (6.10b)

Idealisation:

This element is treated as a beam on pinned supports.

The beam will be provided with links to carry shear and to accommodate the requirements of Cl. 9.2.5 – indirect support of the ribbed slab described in Section 3.3.8.

Arrangement:

Choose to use all-and-alternate-spans-loaded. — Cl. 5.1.3(1) & NA: Table NA.1 (option b)

‡ cf. 126.7 kN/m from analysis of slab (63.2 kN/m + 63.5 kN/m). See Figure 3.12.

4.3.4 Analysis

Analysis by computer, assuming simple supports and including 15% redistribution at supports (with in this instance consequent redistribution in span moments).

ECO: A1.2.2 & NA;
Cl. 5.3.1 (6)
5.3.1(6)

Table 4.3 Elastic and redistributed moments, kNm

Span number	1	2	3	4
Elastic *M*	641.7	433.0	433.0	641.7
Redistributed *M*	606.4	393.2	393.2	606.4
δ	0.945	0.908	0.908	0.945

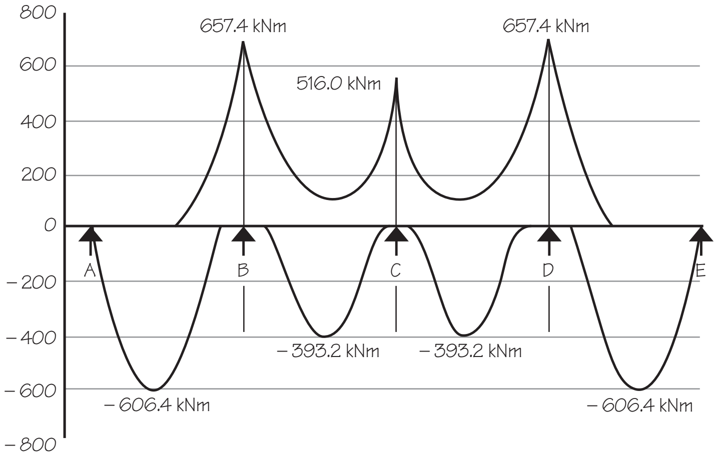


Figure 4.18 Redistributed envelope, kNm

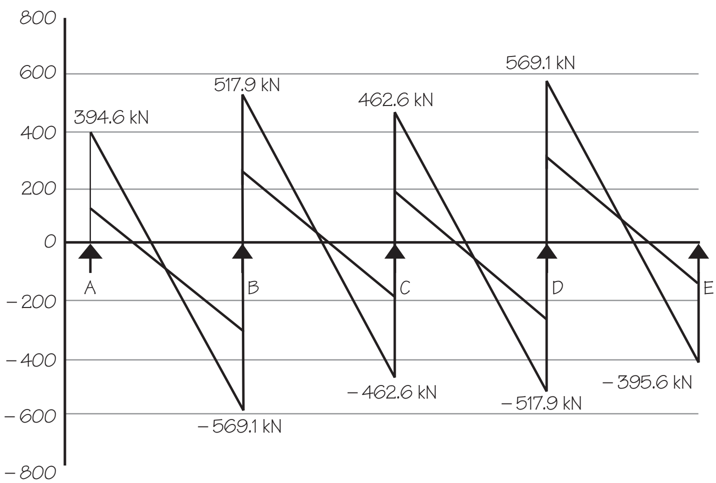


Figure 4.19 Redistributed shears, kN

4.3.5 Flexural design, span AB

a) Span AB (and DE) – Flexure

M_{Ed} = 606.4 kNm

$K = M_{Ed}/bd^2f_{ck}$

where | Cl. 5.3.2.1, Fig. 5.3

$b = b_{eff} = b_{eff1} + b_w + b_{eff2}$

where

$b_{eff1} = (0.2b_1 + 0.1l_0) \leq 0.2l_0 \leq b_1$

where

b_1 = distance between webs/2

Referring to Figures 3.8 and 3.9

= (7500 – 1000 – 550)/2 = 2975 mm

$l_0 = 0.85 \times l_1 = 0.85 \times 7500 = 6375$ mm | Fig. 5.2

$b_{eff1} = 0.2 \times 2975 + 0.1 \times 6375 \leq 0.2 \times 6375 \leq 2975$

$= 1232 \leq 1275 \leq 2975$

= 1232 mm

b_w = 2000 mm

$b_{eff2} = (0.2b_2 + 0.1l_0) \leq 0.2\ l_0 \leq b_2$

where

b_2 = distance between webs/2.

Referring to Figures 3.8 and 3.9

= (9000 – 1000 – 550)/2 = 3725 mm

l_0 = 6375 mm as before

$b_{eff2} = 0.2 \times 3725 + 0.1 \times 6375 \leq 0.2 \times 6375 \leq 3725$

$= 1382 \leq 1275 \leq 3725$

= 1275 mm

b = 1232 + 2000 + 1275 = 4507 mm

d = 300 – 25 – 10 – 25/2 = 252 mm

assuming 10 mm link and H25 in span.

f_{ck} = 35 MPa

$K = 606.4 \times 10^6/(4507 \times 252^2 \times 35)$

= 0.061

$K' = 0.207$ | Appendix A1

or restricting x/d to 0.45

$K' = 0.168$

$K \leq K'$ ∴ section under-reinforced and no compression reinforcement required.

$z = (d/2)\ [1 + (1 - 3.53K)^{0.5}] \leq 0.95d$ | Appendix A1

$= (252/2)\ (1 + 0.886) \leq 0.95 \times 252$

$= 238 \leq 239$ ∴ z = 238 mm

But $z = d - 0.4x$	Appendix A1
$\therefore x = 2.5(d - z) = 2.5(252 - 236) = 32$ mm	
$\therefore$ neutral axis in flange.	
$A_s x < 1.25h_f$ design as rectangular section.	
$A_s = M_{Ed}/f_{yd}z$	
where	
$f_{yd} = 500/1.15 = 434.8$ MPa	
$= 606.4 \times 10^6/(434.8 \times 239) = 5835$ mm^2	
Try 12 no. H25 B (5892 mm^2)	
b) Span AB – Deflection	
Check span-to-effective-depth ratio.	Appendix B
Allowable $l/d = N \times K \times F1 \times F2 \times F3$	Appendix C7
where	**Cl. 7.4.2(2), Exp. (7.16a), Exp. (7.16b)**
N = Basic l/d: check whether $\rho > \rho_0$ and whether to use Exp. (7.16a) or (7.16b)	PD 6687[6]
$\rho = A_s/A_c{}^{\ddagger} = A_{s,req}/[b_w d + (b_{eff} - b_w)h_f]$	
$= 5835/[2000 \times 252 + (4507 - 2000) \times 100]$	
$= 5835/754700$	
$= 0.77\%$	
$\rho_0 = f_{ck}{}^{0.5}/1000 = 30^{0.5}/1000 = 0.59\%$	
$\rho > \rho_0$ $\therefore$ use Exp. (7.16b)	
$N = 11 + 1.5f_{ck}{}^{0.5}\,\rho_0/(\rho - \rho') + f_{ck}{}^{0.5}(\rho'/\rho_0)^{0.5}/12$	**Exp. (7.16b)**
$= 11 + 1.5\,(35^{0.5} \times 0.059/(0.077 - 0) + 35^{0.5}(0/0.59)^{1.5}$	
$= 11 + 6.8 + 0 = 17.8$	
K = (end span) = 1.3	**Table 7.4N & NA**
F1 = $(b_{eff}/b_w = 4057/2000 = 2.03) = 0.90$	**Cl. 7.4.2(2),** Appendix C7
F2 = $7.0/l_{eff}$ (span > 7.0 m)	**Cl. 7.4.2(2), 5.3.2.2(1)**
where	
$l_{eff} = 7100 + 2 \times 300/2 = 7400$ mm	
F2 = $7.0/7.4 = 0.95$	
F3 = $310/\sigma_s \le 1.5$	**Cl. 7.4.2, Exp. (7.17), Table 7.4N & NA, Table NA.5 Note 5**
where§	
$\sigma_s = (f_{yk}/\gamma_S)\,(A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$	
$= 434.8 \times (5835/5892)\,[(47.8 + 0.3 \times 45.8)/(1.25 \times 47.8 + 1.5 \times 45.8)] \times (1/0.945)$	
$= 434.8 \times 0.99 \times 0.48 \times 1.06$	
$= 219$ MPa	

‡ 2.18 of PD 6687[6] suggests that ρ in T sections should be based on the area of concrete above the centroid of the tension steel.

§ See Appendix B1.5

$F3 = 310/\sigma_s$

$= 310/219 = 1.41$

$\therefore$ Permissible $l/d = 17.8 \times 1.3 \times 0.90 \times 0.95 \times 1.41 = 27.9$

Actual $l/d = 7500/252 = 29.8$ $\therefore$ no good

Try 13 no. H25 B (6383 mm²)

$F3 = 310/\sigma_s$

$= 310/219 \times 13/12 = 1.53^{\ddagger}$ = say 1.50

$\therefore$ Permissible $l/d = 17.8 \times 1.3 \times 0.90 \times 0.95 \times 1.50 = 29.7$

Actual $l_{eff}/d = 7400/252 = 29.4$ Say OK

Use 13 no. H25 B (6383 mm²)

4.3.6 Flexural design, support B (and D)

At centreline of support:

$M = 657.4$ kNm

At face of support: Cl. 5.3.2.2(3)

$M_{Ed} = 657.4 - 0.2 \times 517.9 + 0.202 \times 128.5/2$

$= 657.4 - 101.0$

$= 556.4$ kNm

$K = M_{Ed}/b_w d^2 f_{ck}$

where

$b_w = 2000$ mm

$d = 300 - 25$ cover $- 12$ fabric $- 8$ link $- 16$ bar $- 25/2$ bar

$= 226$ mm

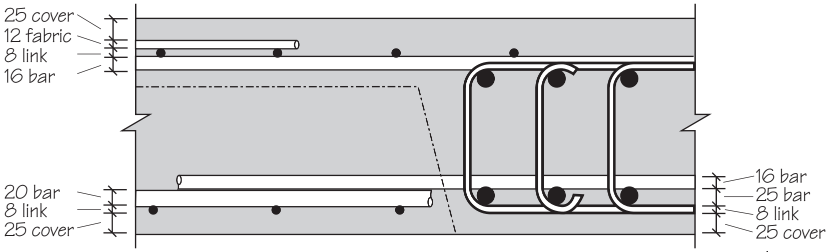


Figure 4.20 Section at rib-beam interface

$K = 556.4 \times 10^6/(2000 \times 226^2 \times 35) = 0.156$

By inspection, $K < K'$

$K' = 0.167$ maximum (or for $\delta = 0.85$, $K' = 0.168$) Appendix A1 Table C.4

$\therefore$ No compression steel required.

‡ Both $A_{s,prov}/A_{s,req}$ and any adjustment to N obtained from Exp. (7.16a) or Exp. (7.16b) is restricted to 1.5 by Note 5 to Table NA.5 in the UK NA. NA, Table NA.5

$z = (226/2)[1 + (1 - 3.53\ K')^{0.5}]$
$= (226/2)[1 + (1 - 3.53 \times 0.156)^{0.5}]$
$= (226/2)\ (1 + 0.67) < 0.95d$
$= 189$ mm

$A_s = M_{Ed}/f_{yd}z$
$= 556.4 \times 10^6/(434.8 \times 189) = 6770\ \text{mm}^2$

Try 14 no. H25 T (6874 mm^2)

To be spread over b_{eff} — Cl. 9.2.1.2(2), Fig. 9.1

$b_{eff} = b_{eff1} + b_w + b_{eff2}$ — Cl. 5.3.2.1, Fig. 5.3

where
$b_{eff1} = (0.2b_1 + 0.1l_O) \leq 0.2l_O \leq b_1$
where
b_1 referring to Figure 3.9
$= (7500 - 1000 - 550)/2 = 2975$ mm
$l_O = 0.15 \times (l_1 + l_2) = 0.15 \times (7500 + 7500) = 2250$ mm
$b_{eff1} = 0.2 \times 2975 + 0.1 \times 2250 \leq 0.2 \times 2250 \leq 2975$
$= 820 \leq 450 \leq 2975$ — Fig. 5.2
$= 450$ mm
$b_w = 2000$ mm
$b_{eff2} = 450$ mm as before
$\therefore\ b_{eff} = 450 + 2000 + 450 = 2900$ mm

Check cracking: — Cl. 7.3.3
Spacing = $2900 - 2 \times (25 - 10 - 25/2)/(14 - 1) = 216$ mm
$\sigma_s = (f_{yk}/\gamma_S)\ (A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$
$= 434.8 \times (6770/6874)\ [\ (47.8 + 0.3 \times 45.8)/$
$(1.25 \times 47.8 + 1.5 \times 45.8) \times (1/0.85)$
$= 434.8 \times 0.98 \times 0.48 \times 1.18 = 241$ MPa

As loading is the cause of cracking satisfy either Table 7.2N or Table 7.3N — Cl. 7.3.3(2) & Note

For $w_k = 0.4$ and $\sigma_s = 240$ MPa max. spacing = 250 mm $\therefore$ OK — Table 7.3N

4.3.7 Flexural design, span BC (and CD similar)

a) Flexure

$M_{Ed} = 393.2$ kNm
$K = M_{Ed}/bd^2f_{ck}$
where — Cl. 5.3.2.1, Fig. 5.3
$b = b_{eff} = b_{eff1} + b_w + b_{eff2}$
where
$b_{eff1} = (0.2b_1 + 0.1l_O) \leq 0.2l_O \leq b_1$

where

b_1 referring to Figure 3.9

$= (7500 - 1000 - 550)/2 = 2975$ mm

$l_0 = 0.70 \times l_2 = 0.7 \times 7500 = 5250$ mm

$b_{eff1} = 0.2 \times 2975 + 0.1 \times 5250 \leq 0.2 \times 5250 \leq 2975$ — **Fig. 5.2**

$= 1120 \leq 1050 \leq 2975$

$= 1050$ mm

$b_w = 2000$ mm

$b_{eff2} = (0.2b_2 + 0.1l_0) \leq 0.2l_0 \leq b_2$

where

b_2 = distance between webs/2

Referring to Figures 3.8 and 3.9

$= (9000 - 1000 - 550)/2 = 3725$ mm

$l_0 = 5250$ mm as before

$b_{eff2} = 0.2 \times 3725 + 0.1 \times 5250 \leq 0.2 \times 5250 \leq 3725$

$= 1270 \leq 1050 \leq 3725$

$= 1270$ mm

$b = 1050 + 2000 + 1270 = 4320$ mm

$d = 252$ mm as before

assuming 10 mm link and H25 in span

$f_{ck} = 30$

$K = 393.2 \times 10^6/(4320 \times 252^2 \times 35)$

$= 0.041$

By inspection, $K \leq K'$ ∴ section under-reinforced and no compression reinforcement required. — Appendix A1

$z = (d/2)\ [1 + (1 - 3.53K)0.5] \leq 0.95d$ — Appendix A1

$= (252/2)\ (1 + 0.924) \leq 0.95 \times 252$

$= 242 > 239$ ∴ $z = 239$ mm

By inspection, $x < 1.25\ h_f$ ∴ design as rectangular section — Appendix A1

$A_s = M_{Ed}/f_{yd}z$

$= 393.2 \times 10^6/(434.8 \times 239) = 3783$ mm^2

Try 8 no. H25 B (3928 mm^2)

b) Deflection

By inspection, compared to span AB OK

But for the purposes of illustration:

Check span-to-effective-depth ratio. — Appendix B

Allowable $l/d = N \times K \times F1 \times F2 \times F3$ — Appendix C7

where

N = Basic l/d: check whether to use Exp. (7.16a) or (7.16b) — **Cl. 7.4.2(2)**

$\rho_0 = 0.59\%$ (for $f_{ck} = 35$)

$\rho = A_s/A_c$ ‡ $= A_{s,req}/[b_w d + (b_{eff} - b_w)h_f]$

where

$b_w = 2000$ mm

$\rho = 3783/(2000 \times 252 + (4320 - 2000) \times 100)$

$= 3783/736000$

$= 0.51\%$

$\rho < \rho_0$ ∴ use Exp. (7.16a)

Exp. (7.16a)

$N = 11 + 1.5f_{ck}^{0.5}\ \rho_0/\rho + 3.2f_{ck}^{0.5}(\rho_0/\rho - 1)1.5$

$= 11 + 1.5 \times 35^{0.5} \times 0.059/0.051 + 3.2 \times 35^{0.5}(0.059/0.051 - 1)^{1.5}$

$= 11 + 10.2 + 23.5 = 17.8$

$= 44.7$

Table 7.4N & NA

K = (internal span) = 1.5

Cl. 7.4.2(2), Appendix C7

F1 = (b_{eff}/b_w = 4320/2000 = 2.16) = 0.88

Cl. 7.4.2(2)

F2 = $7.0/l_{eff}$ = 7.0/7.4 = (span > 7.0 m) = 0.95

Cl. 7.4.2, Exp. (7.17) Table 7.4N, & NA, Table NA.5 Note 5

F3 = $310/\sigma_s \le 1.5$

where§

$\sigma_s = (f_{yk}/\gamma_S)\ (A_{s,req}/A_{s,prov})$ (SLS loads/ULS loads) $(1/\delta)$

$= 434.8 \times (3783/3828)\ [(47.8 + 0.3 \times 45.8)/(1.25 \times 47.8 + 1.5 \times 45.8)] \times (1/0.908)$

$= 434.8 \times 0.99 \times 0.48 \times 1.10$

$= 227$ MPa

F3 = $310/\ \sigma_s$

$= 310/227 = 1.37$

∴ Permissible $l/d = 44.7 \times 1.37 \times 0.88 \times 0.95 \times 1.37 = 70.1$

Actual $l/d = 7500/252 = 29.8$

∴ OK

Use 8 no. H25 B (3928 mm^2)#

c) Hogging

How to: Detailing

Assuming curtailment of top reinforcement at $0.30l + a_l$,

From analysis M_{Ed}

at $0.3l$ from BC (& DC) = 216.9 kNm

at $0.3l$ from CB (& CD) = 185.6 kNm

$K = 216.9 \times 10^6/(2000 \times 226^2 \times 35) = 0.061$

By inspection, $K < K'$

‡ 2.18 of PD 6687[6] suggests that ρ in T sections should be based on the area of concrete above the centroid of the tension steel.

§ See Appendix B1.5

12 no. H20 B (3768 mm^2) used to suit final arrangement of links.

z = (226/2)[1 + (1 – 3.53 K')$^{0.5}$]
= (226/2)[1 + (1 – 3.53 × 0.061)$^{0.5}$]
= (226/2) (1 + 0.89) < 0.95d
= 214 mm < 215 mm

A_s = $M_{Ed}/f_{yd}z$
= 216.9 × 10^6/(434.8 × 214) = 2948 mm^2

Use 12 no. H20 T (3748 mm^2)
(to suit links and bottom steel)

Top steel at supports may be curtailed down to 12 no. H20 T at $0.3l + a_l$ = 0.3 × 7500 + 1.25 × 214 = 2518 say 2600 mm from centreline of support. Cl. 9.2.1.3(2)

4.3.8 Flexural design, support C

At centreline of support,
M = 516.0 kNm
At face of support, Cl. 5.3.2.2(3)
M_{Ed} = 516.0 – 0.2 × 462.6 + 0.20^2 × 128.5/2
= 516.0 – 90.0
= 426.0 kNm
K = $M_{Ed}/b_w d^2 f_{ck}$
where
b_w = 2000 mm
d = 226 mm as before

K = 426.0 × 10^6/(2000 × 226^2 × 35) = 0.119
By inspection, $K < K'$

z = (226/2) [1 + (1 – 3.53K)$^{0.5}$]
= (226/2) [1 + (1 – 3.53 × 0.119)$^{0.5}$]
= (226/2) (1 + 0.76) < 0.95d
= 199 mm

A_s = $M_{Ed}/f_{yd}z$
= 426.0 × 10^6/(434.8 × 199) = 4923 mm^2

Try 10 no. H25 T (4910 mm^2)‡

4.3.9 Design for beam shear

a) Support A (and E)

At d from face of support,
V_{Ed} = 394.6 – (0.400/2 + 0·252) × 128.5 = 336.5 kN Cl. 6.2.1(8)

Maximum shear resistance:
By inspection, $V_{Rd,max}$ OK and cot θ = 2.5

‡ 12 no. H25 used to suit final arrangement of links.

However, for the purpose of illustration: check shear capacity,
$V_{Rd,max} = \alpha_{cw}\, b_w\, z\nu f_{cd}/\ (\cot\theta + \tan\theta)$

where — Exp. (6.9) & NA
α_{cw} = 1.0
b_w = 2000 mm as before
z = 0.9d
ν = 0.6 [1 − f_{ck}/250] = 0.516 — Cl. 6.2.3(1)
f_{cd} = 35/1.5 = 23.3 MPa
θ = angle of inclination of strut.
By inspection, $\cot^{-1}\theta$ << 21.8. But $\cot\theta$ restricted to 2.5 and ∴ $\tan\theta$ = 0.4. — Cl. 6.2.3(2) & NA

$V_{Rd,max}$ = 1.0 × 2000 × 0.90 × 252 × 0.516 × 23.3/(2.5 + 0.4)
= 2089.5 kN

∴ OK

Shear links: shear resistance with links
$V_{Rd,s} = (A_{sw}/s)\, z\, f_{ywd} \cot\theta \geq V_{Ed}$ — Exp. (6.8)
∴ for $V_{Ed} \leq V_{Rd,s}$
$A_{sw}/s \geq V_{Ed}/z\, f_{ywd} \cot\theta$

where
A_{sw}/s = area of legs of links/link spacing
z = 0.9d as before
f_{ywd} = 500/1.15 = 434.8
$\cot\theta$ = 2.5 as before

$A_{sw}/s \geq 336.5 \times 10^3/(0.9 \times 252 \times 434.8 \times 2.5) = 1.36$ — Cl. 9.2.2(5),
Minimum $A_{sw}/s = \rho_{w,min} b_w \sin\alpha$ — Exp. (9.4)

where
$\rho_{w,min}$ = 0.08 × $f_{ck}^{0.5}/f_{yk}$ = 0.08 × $35^{0.5}$/500 = 0.00095 — Exp. (9.5N) & NA
b_w = 2000 mm as before
α = angle between shear reinforcement and the longitudinal axis.
For vertical reinforcement $\sin\alpha$ = 1.0
Minimum A_{sw}/s = 0.00095 × 2000 × 1 = 1.90
But,
maximum spacing of links longitudinally = 0.75d = 183 mm — Cl. 9.2.2(6)
Maximum spacing of links laterally = 0.75d ≤ 600 mm = 183 mm — Cl. 9.2.2(8)
H10s required to maintain 35 mm cover to H25

∴ Use H10 @ 175 cc both ways
i.e. H10 in 12[§] legs @ 175 mm cc (A_{sw}/s = 5.38)

§ (2000 mm – 2 × 25 mm cover – 10 mm diameter)/175 = 11 spaces, ∴ 12 legs.

b) Support B (and C and D)

By inspection, the requirement for minimum reinforcement and, in this instance, for H10 legs of links will outweigh design requirements. Nonetheless check capacity of $A_{sw}/s = 5.38$

$V_{Rd,s} = (A_{sw}/s)\, z\, f_{ywd} \cot\theta$ — Exp. (6.8)

$= 5.38 \times 0.9 \times 252 \times 434.8 \times 2.5 = 1326.3$ kN

Maximum shear at support = 517.9 kN

i.e. capacity of minimum links not exceeded.

By inspection, the requirement for indirect support of the ribs of the slab using 87 mm²/rib within 150 mm of centreline of ribs (at 900 mm centres) and within 50 mm of rib/solid interface is adequately catered for and will not unduly effect the shear capacity of the beam. — Cl. 9.2.5, Section 3.4.8

Use 150 mm centres to tie in with 900 mm centres of ribs

$\therefore$ Use H10 in 12 legs @ 150 mm cc ($A_{sw}/s = 6.28$) throughout beam

4.3.10 Check for punching shear, column B

As the beam is wide and shallow it should be checked for punching shear.

At B, applied shear force, $V_{Ed} = 569.1 + 517.9 = 1087.0$ kN.

Check at perimeter of 400 × 400 mm column: — Cl. 6.4.3(2), 6.4.5(3)

$v_{Ed} = \beta V_{Ed}/u_i d < v_{Rd,max}$

where

β = factor dealing with eccentricity; recommended value 1.15 — Fig. 6.21N & NA

V_{Ed} = applied shear force

u_i = control perimeter under consideration. For punching shear adjacent to interior columns $u_0 = 2(c_x + c_y) = 1600$ mm — Cl. 6.4.5(3)

d = mean $d = (245 + 226)/2 = 235$ mm

$v_{Ed} = 1.15 \times 1087.0 \times 10^3/1600 \times 235 = 3.32$ MPa — Exp. (6.32)

$v_{Rd,max} = 0.5\nu f_{cd}$ — Cl. 6.4.5(3) Note

where

$\nu = 0.6(1 - f_{ck}/250) = 0.516$ — Exp. (6.6) & NA

$f_{cd} = \alpha_{cc}\lambda f_{ck}/\gamma_C = 1.0 \times 1.0 \times 35/1.5 = 23.3$

$v_{Rd,max} = 0.5 \times 0.516 \times 23.3 = 6.02$ MPa $\therefore$ OK — Table C7‡

Check shear stress at basic perimeter u_1 ($2.0d$ from face of column): — Cl. 6.4.2

$v_{Ed} = \beta V_{Ed}/u_1 d < v_{Rd,c}$

where

β, V_{Ed} and d as before — Fig. 6.13

‡ In this case, at the perimeter of the column, it is assumed that the strut angle is 45°, i.e. that $\cot\theta = 1.0$. In other cases, where $\cot\theta < 1.0$, $v_{Rd,max}$ is available from Table C7.

u_1 = control perimeter under consideration. For punching shear at $2d$ from interior columns
= $2(c_x + c_y) + 2\pi \times 2d$
= $1600 + 2\pi \times 2 \times 235 = 4553$ mm

$v_{Ed} = 1.15 \times 1087.0 \times 10^3/4553 \times 235 = 1.17$ MPa

$v_{Rd,c} = 0.18/\gamma_C \times k \times (100\rho_l f_{ck})^{0.333}$ — **Exp. (6.47) & NA**

where
$\gamma_C = 1.5$
$k = 1 + (200/d)^{0.5} \leq 2$
$= 1 + (200/235)^{0.5} = 1.92$
$\rho_l = (\rho_{ly}, \rho_{lz})^{0.5}$ — **Cl. 6.4.4.1(1)**
where
ρ_{ly}, ρ_{lz} = Reinforcement ratio of bonded steel in the y and z direction in a width of the column plus $3d$ each side of column.
$= 6874/(2000 \times 226) = 0.0152$
$\rho_{lz} = 741/(900 \times 245) = 0.0036$
$\rho_l = (0.0152 \times 0.0036)^{0.5} = 0.0074$
$f_{ck} = 35$

$v_{Rd,c} = 0.18/1.5 \times 1.92 \times (100 \times 0.0074 \times 35)^{0.333} = 0.68$ MPa§ — Table C6#

∴ punching shear reinforcement required

Shear reinforcement (assuming rectangular arrangement of links):
At the basic control perimeter, u_1, $2d$ from the column:

$A_{sw} \geq (v_{Ed} - 0.75v_{Rd,c})\, s_r\, u_1/1.5f_{ywd,ef})$ — **Exp. (6.52)**

where
s_r = 175 mm — **Cl. 9.4.3(1)**
$f_{ywd,ef}$ = effective design strength of reinforcement
$= (250 + 0.25d) < f_{yd} = 309$ MPa — **Cl. 6.4.5(1)**

For perimeter u_1
$A_{sw} = (1.17 - 0.75 \times 0.68) \times 175 \times 4553/(1.5 \times 309) = 1135$ mm^2 per perimeter

Try 15 no. H10 (1177 mm^2)

§ See Section 3.4.14 with respect to possible limit of 2.0 or 2.5 on $V_{Ed}/V_{Rd,c}$ within punching shear requirements.

$v_{Rd,c}$ for various values of d and ρ_l is available from Table C6.

Check availability of reinforcement‡:

1st perimeter to be > $0.3d$ but < $0.5d$, i.e between 70 mm and 117 mm from face of column. Say $0.4d$ = 100 mm from face of column. | **Fig. 9.10, 9.4.3(4)**

By inspection of Figure 4.21. the equivalent of 14 locations are available between 70 mm and 117 mm from face of column therefore say OK.

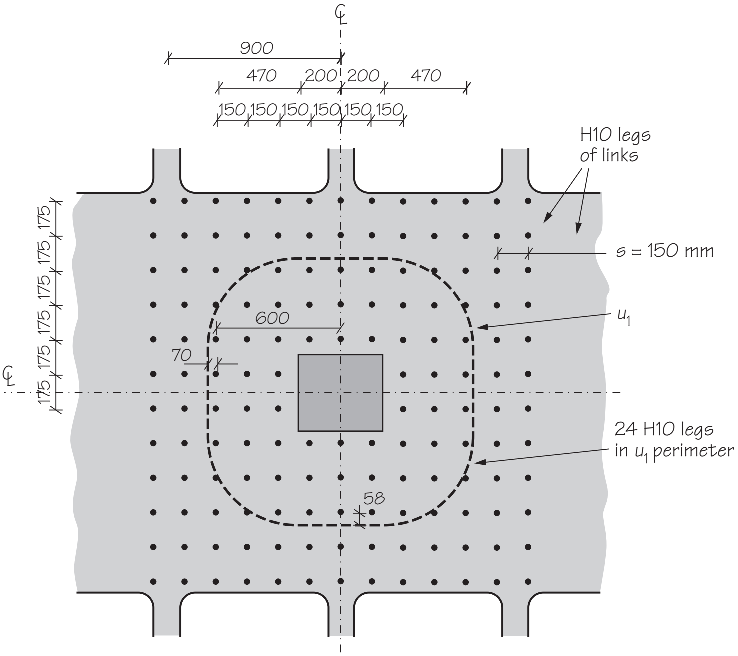


Figure 4.21 Shear links and punching shear perimeter u_1

Perimeter at which no punching shear links are required:

$u_{out} = V_{Ed} \times \beta/(d \times v_{Rd,c})$

$u_{out} = 1087 \times 1.15 \times 10^3/(235 \times 0.68) = 7826$ mm

Length of column faces = 4 × 400 = 1600 mm

Radius to $u_{out} = (7823 - 1600)/2\pi = 990$ mm

from face of column i.e. in ribs, therefore beam shear governs

‡ The same area of shear reinforcement is required for all perimeters inside or outside perimeter u_1. See Section 3.4.13.

Punching shear reinforcement is also subject to requirements for minimum reinforcement and spacing of shear reinforcement (see Cl. 9.4.3). The centre of links from the centreline of the column shown in Figure 4.21 have been adjusted to accommodate a perimeter of links at between $0.3d$ and $0.5d$ from the column face.

4.3.11 Summary of design

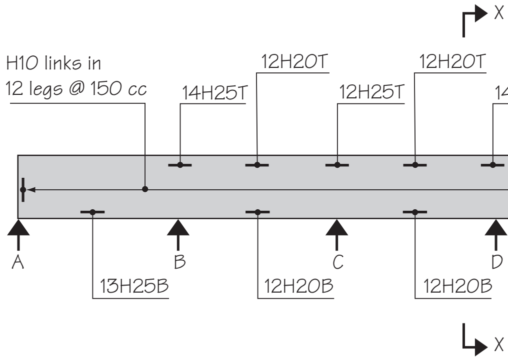


Figure 4.22 Summary of design

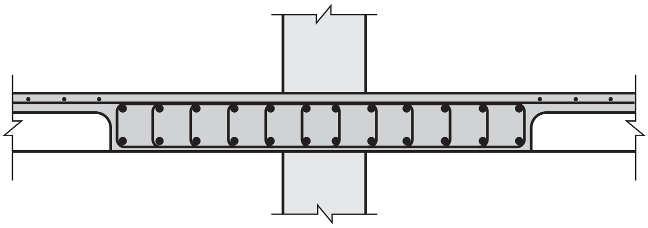


Figure 4.23 Section X–X

5 Columns

5.0 General

The calculations in this section illustrate:

5.1 Design of a non-slender edge column using hand calculation.

5.2 Design of a perimeter column using iteration of equations to determine reinforcement requirements.

5.3 Design of an internal column with high axial load.

5.4 Design of a slender column requiring a two-hour fire resistance.

In general, axial loads and first order moments are assumed to be available. The designs consider slenderness in order to determine design moments, M_{Ed}. The columns are designed and checked for biaxial bending. The effects of allowing for imperfections are illustrated.

A general method of designing columns is as follows. In practice, several of these steps may be combined.

■ Determine design life.	**EC0 & NA Table NA 2.1**
■ Assess actions on the column.	**EC1 (10 parts) & UK NAs**
■ Determine which combinations of actions apply.	**EC0 & NA Tables NA A1.1 & NA A1.2(B)**
■ Assess durability requirements and determine concrete strength.	**BS 8500–1**
■ Check cover requirements for appropriate fire resistance period.	**Approved Document B, EC2–1–2**
■ Determine cover for fire, durability and bond.	**Cl. 4.4.1**
■ Analyse structure for critical combination moments and axial forces.	**Section 5**
■ Check slenderness and determine design moments.	**Section 5.8**
■ Determine area of reinforcement required.	**Section 6.1**
■ Check spacing of bars and links.	**Sections 8 & 9**

5.1 Edge column

The intention of this calculation is to show a typical hand calculation that makes reference to design charts.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details **Edge column**	Calculated by chg	Job no. CCIP – 041
		Checked by web	Sheet no. 1
		Client TCC	Date Oct 09

A 300 mm square column on the edge of a flat slab structure supports an axial load of 1620 kN and first order moments of 38.5 kNm top and –38.5 kNm bottom in one direction only‡. The concrete is grade C30/37, f_{ck} = 30 MPa and cover, c_{nom}, = 25 mm. The 250 mm thick flat slabs are at 4000 mm vertical centres.

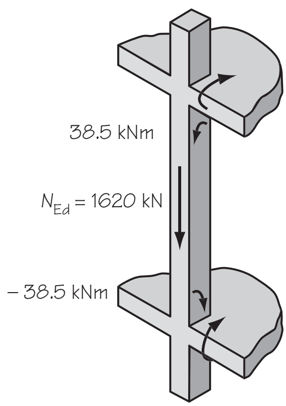


Figure 5.1 Forces in edge column

5.1.1 Check slenderness, λ

Effective length§, l_0 = factor × l — **Cl. 5.8.3.2**

where

factor = from Table C16, condition 2 each end — Table C16, PD 6687: 2.10

= 0.85

l = clear height = 3750 mm

∴ l_0 = 0.85 x 3750 = 3187 mm

Slenderness $\lambda = l_0/i$ — **Exp. (5.14)**

‡ For examples of load take-downs and 1st order moment analysis see Section 5.3.2

§ Effective lengths are covered in Eurocode 2 Cl. 5.8.3.2 and Exp. (5.15). The effective length of most columns will be $l/2 < l_0 < l$ (see Eurocode 2 Figure 5.7f). PD 6687[6] Cl. 2.10 suggests that using the procedure outlined in Eurocode 2 (5.8.3.2(3) and 5.8.3.2(5)) leads to similar effective lengths to those tabulated in BS 8110[7] and reproduced in Table 5.1 of *Concise Eurocode 2*[5] and in this publication as Table C16. For simplicity, tabular values are used in this example. However, experience suggests that these tabulated values are conservative.

Fig. 5.7
PD 6687[6]: Cl. 2.10
Cl. 5.8.3.2(3)
5.8.3.2(5)
Table C16

where

i = radius of gyration

= $h/12^{0.5}$ for rectangular sections

λ = 3187 × 3.46/300 = 36.8

5.1.2 Limiting slenderness, λ_{lim}

$\lambda_{lim} = 20\ ABC/n^{0.5}$ — Exp. (5.13N)

where

A = 0.7 (default) — Cl. 5.8.3.1(1)

B = 1.1 (default)

$C = 1.7 - r_m = 1.7 - M_{01}/M_{02}$

$= 1.7 - 38.5/(-38.5) = 2.7$

$n = N_{Ed}/A_c f_{cd} = 1620 \times 10^3/(300^2 \times 0.85 \times 30/1.5)$

$= 1.06$

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

$= 20 \times 0.7 \times 1.1 \times 2.7/1.06^{0.5}$

In this example λ_{lim} = 40.4 i.e. > 36.8 ∴ Column not slender

5.1.3 Design moments

$M_{Ed} = \max[M_{02};\ M_{0Ed} + M_2;\ M_{01} + 0.5M_2]$ — Cl. 5.8.8.2(1)

where

$M_{02} = M + e_i N_{Ed} \geq e_0 N_{Ed}$ — Cl. 5.8.8.2, 6.1.4

where

M = 38.5 kNm

$e_i = l_0/400$ — Cl. 5.2.7, 5.2.9

$e_0 = \max[h/30;\ 20] = \max[300/30;\ 20] = 20$ mm — Cl. 6.1.4

$M_{02} = 38.5 + 1620 \times 3.187/400 \geq 0.02 \times 1620$

$= 38.5 + 12.9 \geq 32.4$ kNm

$= 51.4$ kNm

$M_{0Ed} = 0.6M_{02} + 0.4M_{01} \geq 0.4M_{02}$

$= 0.6 \times 51.4 + 0.4 \times (-38.5 + 12.9) \geq 0.4 \times 51.4$

$= 20.6 \geq 20.6$

$= 20.6$

M_2 = 0 (column is not slender)

$M_{01} = M_{02}$

∴ $\max[M_{02};\ M_{0Ed} + M_2;\ M_{01} + 0.5M_2]$ = 51.4 kNm ∴ M_{Ed} = 51.4 kNm

5.1.4 Design using charts (see Appendix C)

$d_2 = c_{nom} + \text{link} + \phi/2 = 25 + 8 + 16 = 49$

$d_2/h = 49/300 = 0.163$

∴ interpolating between d_2/h = 0.15 and 0.20 — Figs. C5c), C5d)

for

$N_{Ed}/bhf_{ck} = 1620 \times 10^3/(300^2 \times 30) = 0.60$

$M_{Ed}/bh^2f_{ck} = 51.4 \times 10^6/(300^3 \times 30) = 0.063$
$A_sf_{yk}/bhf_{ck} = 0.24$
$A_s = 0.24 \times 300^2 \times 30/500 = 1296\ mm^2$

Try 4 no. H25 (1964 mm^2)

5.1.5 Check for biaxial bending

$\lambda_y/\lambda_z \approx 1.0$ — Cl. 5.8.9
i.e. $\lambda_y/\lambda_z \leq 2.0$ ∴ OK but check Exp. (5.38b) — Exp. (5.38a)

As a worst case M_{Edy} may coexist with e_0N_{Ed} about the orthogonal axis: — Cl. 6.1(4)

$$\frac{e_y/h_{eq}}{e_z/b_{eq}} = \frac{(M_{Edz}/N_{Ed})/h}{(M_{Edy}/N_{Ed})/b} = \frac{M_{Edz}}{M_{Edy}}$$ — Exp. (5.38b)

Imperfections need to be taken into account in one direction only. — Cl. 5.8.9(2)
∴ As a worst case for biaxial bending

$M_{Edz} = M + 0 = 38.5$ kNm
$M_{Edy} = e_0N_{Ed} = 32.4$ kNm

$$\frac{M_{Edz}}{M_{Edy}} = \frac{38.5}{32.4} = 1.19 \text{ i.e. } > 0.2 \text{ and } < 5.0$$ — Exp. (5.38b)

∴ Biaxial check required — Cl. 5.8.9(4)

Check whether
$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$ — Exp. (5.39)
where
M_{Edz} = 38.5 kNm
M_{Edy} = 32.4 kNm
$M_{Rdz} = M_{Rdy}$ — Figs. C5c), C5d)

To determine M_{Rdz}, find M_{Ed}/bh^2f_{ck} (and therefore moment capacity) by interpolating between $d_2/h = 0.15$ (Figure C5c) and 0.20 (Figure C5d) for the proposed arrangement and co-existent axial load.

Assuming 4 no. H25,
$A_sf_{yk}/bhf_{ck} = 1964 \times 500/(300^2 \times 30) = 0.36$

Interpolating for $N_{Ed}/bhf_{ck} = 0.6$,
$M_{Ed}/bh^2f_{ck} = 0.094$
∴ $M_{Rdz} = M_{Rdy} = 0.094 \times 300^3 \times 30 = 76.1$ kNm

a is dependent on N_{Ed}/N_{Rd} — Cl. 5.8.9(4), Notes to Exp. (5.39)
where
N_{Ed} = 1620 kN as before

$N_{Rd} = A_c f_{cd} + A_s f_{yd}$
$= 300^2 \times 0.85 \times 30/1.5 + 1964 \times 500/1.15$
$= 1530.0 + 853.9$
$= 2383.9$ kN

$N_{Ed}/N_{Rd} = 1620/2383.9 = 0.68$

∴ a = 1.48 by interpolating between values given for N_{Ed}/N_{Rd} = 0.1, (1.0) and N_{Ed}/N_{Rd} = 0.7, (1.5)

$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a = (38.5/76.1)^{1.48} + (32.4/76.1)^{1.48}$ **Exp. (5.39)**
$= 0.36 + 0.28$
$= 0.64$ ∴ OK.

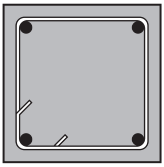
∴ 4 no. H25 OK

5.1.6 Links

Diameter min. $\phi/4 = 25/4 = 8$ mm **Cl. 9.5.3 & NA**
Max. spacing = $0.6 \times 300 = 180$ mm **Cl. 9.5.3(3), Cl. 9.5.3(4)**

Links at say 175 mm cc

5.1.7 Design summary



4 H25
H8 links @ 175 cc
25 mm cover
f_{ck} = 30 MPa

Figure 5.2 Design summary: edge column

5.2 Perimeter column (internal environment)

This example is intended to show a hand calculation for a non-slender perimeter column using iteration (of *x*) to determine the reinforcement required.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Perimeter column (internal environment)	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

This 300 × 300 mm perimeter column is in an internal environment and supports three suspended floors and the roof of an office block. It is to be designed at ground floor level where the storey height is 3.45 m and the clear height in the N–S direction (*z* direction) is 3.0 m and 3.325 m in the E–W direction (*y* direction). One-hour fire resistance is required and f_{ck} = 30 MPa.

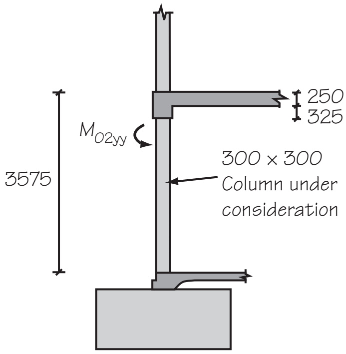


Figure 5.3 Perimeter column (internal environment)

From first order analysis, load case 1:
N_{Ed} = 1129.6 kN; M_{02yy} = 89.6 kNm; M_{02zz} = 0

Load case 2:
N_{Ed} = 1072.1 kN; M_{02yy} = 68.7 kNm; M_{02zz} = 6.0 kNm

5.2.1 Cover

$c_{nom} = c_{min} + \Delta c_{dev}$ — Exp. (4.1)

where

c_{min} = max[$c_{min,b}$; $c_{min,dur}$]

where

$c_{min,b}$ = diameter of bar. Assume 32 mm bars and 8 mm links — Cl. 4.4.1.2(3)
= 32 mm to main bars, 32 – 8 = 24 mm to links
= say 25 mm

$c_{min,dur}$ = minimum cover due to environmental conditions. Assume XC1.

$c_{min,dur}$ = 15 mm

c_{min} = 25 mm

Δc_{dev} = 10 mm — Cl. 4.4.1.3(3)

Therefore c_{nom} = 25 + 10 = 35 mm to links c_{nom} = 35 mm to links.

5.2.2 Fire resistance

Check validity of using Method A and Table 5.2a of BS EN 1992–1–2:	EC2-1-2: 5.3.2, Table 5.2a
$l_{0,fi} \approx 0.7 \times 3.325$ i.e. < 3.0 m ∴ OK.	EC2-1-2: 5.3.3(3)
$e = M_{02yy}/N_{Ed} = 89.6 \times 10^6/1129.6 \times 10^3 = 79$ mm	EC2-1-2: 5.3.2 & NA
$e_{max} = 0.15h = 0.15 \times 300 = 45$ mm ∴ no good.	
Check validity of using Method B and Table 5.2b: $e_{max} = 0.25b = 75$ mm ∴ no good.	EC2-1-2: 5.3.3
Use BS EN 1992–1–2 Annex C Tables C1–C9.	EC2-1-2: Annex C
Assume min. 4 no. H25 = 1964 mm² (≡ 2.2%)‡	EC2-1-2: 5.3.3(2)

$\omega = A_s f_{yd}/A_c f_{cd} = 0.022 \times (500/1.15)/(0.85 \times 30/1.5)$

$= 0.56$

$e \approx 0.25b$ and ≤ 100 mm

$\lambda = l_0/i$

where (EC2-1-2: 5.3.3(2), 5.3.3(3))

$l_0 = 0.7 \times 3.325 = 2327$ mm

i = radius of gyration $= (I/A)^{0.5} = h/12^{0.5}$

where

I = inertia $= bh^3/12$

A = area $= bh$

h = height of section

b = breadth of section

$= 300/12^{0.5} = 87$ mm

$\lambda = 2327/87 = 276$

$n = N_{0Ed,fi}/0.7(A_c f_{cd} + A_s f_{yd})$ (EC2-1-2: 5.3.3(2))

$= 0.7 \times 1129.6/0.7(300^2 \times 0.85 \times 30/1.5 + 1964 \times 500/1.15)$

$= 1129.6/2383.9$

$= 0.47$

∴ interpolate for $\lambda = 30$ and $n = 0.47$ between

from Table C.5 of BS EN 1992–1–2 ($\omega = 0.5$, $e = 0.25b$): (EC2-1-2: Table C.5)

minimum dimension, $b_{min} = 235$, and axis distance, $a = 35$ mm

and

from Table C.8 of BS EN 1992–1–2 ($\omega = 1.0$, $e = 0.25b$): (EC2-1-2: Table C.8)

‡ Using 4 no. H20 gives $\omega = 0.34$, $n = 0.54$ and $b_{min} = 310$ mm ∴ no good.

b_{min} = 185, and
a = 30 mm
∴ for $\omega = 0.56$,
b_{min} = 228, and
a = 35 mm

OK to use Method B but use min. 4 no. H25

5.2.3 Structural design: check slenderness

Effective length, l_0:

$l_0 = 0.5l\ [1 + k_1/(0.45 + k_1)\ ]^{0.5}\ [1 + k_2/(0.45 + k_2)\ ]^{0.5}$ — **Exp. (5.15)**

where

k_1, k_2 = relative stiffnesses top and bottom

But conservatively, choose to use tabular method[§]. For critical direction, the column is in condition 2 at top and condition 3 at bottom (pinned support). — Table C16

$l_0 = 0.95 \times 3325 = 3158$ mm

Slenderness ratio, λ: — **Cl. 5.8.3.2(1)**

$\lambda = l_0/i$

where

i = radius of gyration = $(I/A)^{0.5} = h/12^{0.5}$

$\lambda = 3158 \times 12^{0.5}/300 = 36.5$ — $\lambda = 36.5$

Limiting slenderness ratio, λ_{lim} — **Cl. 5.8.3.1(1) & NA**

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

where

$A = 1/(1 + 0.2\ \phi_{ef})$. Assume 0.7 — **Cl. 5.8.4**

$B = (1 + 2\ A_s f_{yd}/A_c f_{cd})^{0.5}$ — **Cl. 5.8.3.1(1)**

$= (1 + 2\omega)^{0.5}$

Assuming min. 4 no. H25 (for fire)

$\omega = 0.56$ as before

$B = (1 + 2 \times 0.56)^{0.5} = 1.46$

$C = 1.7 - r_m$ — **Cl. 5.8.3.1(1)**

where

$r_m = M_{01}/M_2$

Assuming conservatively that $M_{01} = 0$

$r_m = 0$

$C = 1.7$

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

$= 1129.6 \times 10^3/(300^2 \times 0.85 \times 30/1.5)$

$= 0.74$

§ See footnote to Section 5.1.1.

λ_{lim} = 20 × 0.7 × 1.46 × 1.7/0.74$^{0.5}$
= 40.4 λ_{lim} = 40.4

∴ as $\lambda < \lambda_{lim}$ column is not slender and 2nd order moments are not required.

Column is not slender

5.2.4 Design moments, M_{Ed}

$M_{Ed} = M_{0Ed} + M_2 \geq e_0 N_{Ed}$ — Cl. 5.8.8.2(1), 5.8.8.2(3)

But as column is not slender, M_2 = 0, ∴ — Cl. 6.1.4

$M_{Ed} = M_{0ed} = M + e_i N_{Ed} \geq e_0 N_{Ed}$

where

M = moment from 1st order analysis

$e_i N_{Ed}$ = effect of imperfections‡

where

$e_i = l_0/400$ — Cl. 5.2(7), 5.2.9, 5.8.8.2(1)

$e_0 = h/30 > 20$ mm — Cl. 6.1.4

Load case 1:

M_{Edy} = 89.6 + (3158/400) × 1129.6 × 10^{-3} > 0.02 × 1129.6
= 89.6 + 8.9 > 22.6 = 98.5 kNm

Load case 2:

M_{Edy} = 68.7 kNm

M_{Edz} = 6.0 + (l_0/400) × 1072.1 × 10^{-3} > 0.02 × 1072.1

where — Table C16

l_0 = 0.9 × 3000
= 13.2 > 21.4 = 21.4 kNm

5.2.5 Design using iteration of x

For axial load: — Concise: Section 6.2.2, Appendix A3

$A_{sN}/2 = (N_{Ed} - \alpha_{cc}\eta f_{ck} b d_c/\gamma_C)/(\sigma_{sc} - \sigma_{st})$

For moment: — Appendices A3, C9.2,

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

where

M_{Ed} = 98.5 × 10^6

N_{Ed} = 1129.6 × 10^3

α_{cc} = 0.85 — Cl. 3.1.6(1) & NA

η = 1.0 for $f_{ck} \leq 50$ MPa — Exp. (3.21)

‡The effects of imperfections need only be taken into account in the most unfavourable direction. — Cl. 5.8.9(2)

f_{ck} = 30
b = 300
h = 300
d_c = depth of compression zone
= λx — Exp. (3.19)
= $0.8x < h$

where
x = depth to neutral axis

d_2 = 35 + 8 + 25/2 = 55 mm assuming H25
γ_C = 1.5 — Table 2.1N
σ_{sc}, (σ_{st}) = stress in reinforcement in compression (tension)

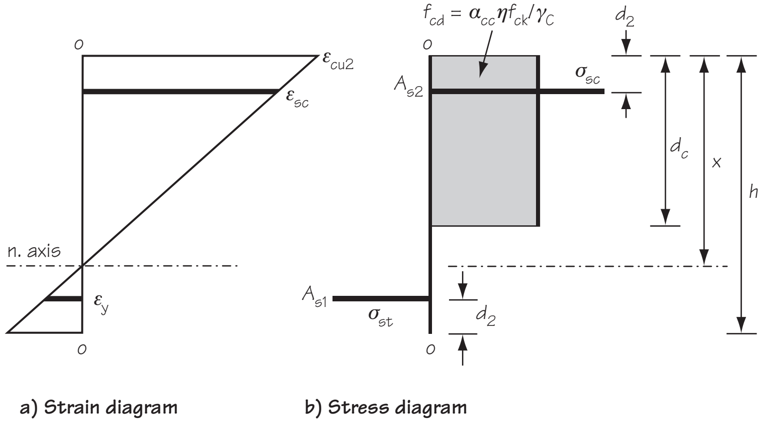


Figure 5.4 Section in axial compression and bending — Fig. 6.1

Try x = 200 mm

$\varepsilon_{cu} = \varepsilon_{cu2} = 0.0035$

$$\varepsilon_{sc} = \frac{0.0035 \times (x - d_2)}{x} = \frac{0.0035 \times (200 - 55)}{200}$$

= 0.0025

σ_{sc} = $0.0025 \times 200000 \le f_{yk}/\gamma_S$
= $500 \le 500/1.15$
= 434.8 MPa

ε_{st} = $0.0035(h - x - d_2)/x = 0.0035(300 - 200 - 55)/200$
= 0.0008

σ_{st} = $0.0008 \times 200000 \le 500/1.15$
= 160 MPa

$$A_{sN}/2 = \frac{1129.6 \times 10^3 - 0.85 \times 1.0 \times 30 \times 300 \times 200 \times 0.8/(1.5 \times 10^3)}{434.8 - 160}$$

$$= \frac{(1129.6 - 816.0) \times 10^3}{274.8} = 1141 \text{ mm}^2$$

$$A_{sM}/2 = \frac{98.5 \times 10^6 - 0.85 \times 1.0 \times 30 \times 300 \times 200 \times 0.8\,(300/2 - 200 \times 0.8/2)/(1.5 \times 10^3)}{(300/2 - 55)\,(434.8 + 160)}$$

$$= \frac{(98.5 - 57.1) \times 10^6}{95 \times 594.8} = 733 \text{ mm}^2$$

Similarly for x = 210 mm

$\varepsilon_{cu} = 0.0035$

$\varepsilon_{sc} = 0.0026 \quad \therefore \sigma_{sc} = 434.8$

$\varepsilon_{st} = 0.0006 \quad \therefore \sigma_{st} = 120$ MPa

$$A_{sN}/2 = \frac{(1129.6 - 856.8) \times 10^3}{434.8 - 120} = 866 \text{ mm}^2$$

$$A_{sM}/2 = \frac{(98.5 - 56.5) \times 10^6}{95 \times 554.8} = 796 \text{ mm}^2$$

Similarly for x = 212 mm

$\sigma_{sc} = 434.8$

$\varepsilon_{st} = 0.00054 \ \therefore \varepsilon_{st} = 109$ MPa

$$A_{sN}/2 = \frac{(1129.6 - 865.0) \times 10^3}{434.8 - 109} = 812 \text{ mm}^2$$

$$A_{sM}/2 = \frac{(98.5 - 56.3) \times 10^6}{95 \times 543.8} = 816 \text{ mm}^2$$

$\therefore$ as $A_{sN}/2 \approx A_{sM}/2$, x = 212 mm is approximately correct and

$A_{sN} \approx A_{sM}, \approx 1628$ mm^2

$\therefore$ Try 4 no. H25 (1964 mm^2)

5.2.6 Check for biaxial bending

By inspection, not critical. — **Cl. 5.8.9(3)**

[**Proof**:

Section is symmetrical and $M_{Rdz} > 98.5$ kNm.

Assuming $e_y/e_z > 0.2$ and biaxial bending is critical, and assuming exponent $a = 1$ as a worst case for load case 2: — **Exp. (5.39)**

$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a = (21.4/98.5)^1 + (68.7/98.5)^1$

= 0.91 i.e. < 1.0 $\therefore$ OK.]

5.2.7 Links

Minimum size links = 25/4 = 6.25, say 8 mm

Spacing: minimum of — **Cl. 9.5.3(3), 9.5.3(4)**

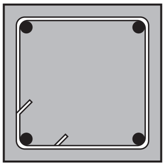
a) 0.6 × 20 × 25 = 300 mm,

b) 0.6 × 300 = 180 mm or

c) 0.6 × 400 = 240 mm

Use H8 @ 175 mm cc

5.2.8 Design summary



4 H25

H8 links @ 175 cc

c_{nom} = 35 mm to links

Figure 5.5 Design summary: perimeter column

5.3 Internal column

The flat slab shown in Example 3.4 (reproduced as Figure 5.6) is part of an 8-storey structure above ground with a basement below ground. The problem is to design column C2 between ground floor and 1st floor.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Internal column	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

The design forces need to be determined. This will include the judgement of whether to use Exp. (6.10) or the worse case of Exp. (6.10a) and (6.10b) for the design of this column.

The suspended slabs (including the ground floor slab) are 300 mm thick flat slabs at 4500 mm vertical centres. Between ground and 5th floors the columns at C2 are 500 mm square; above 5th floor they are 465 mm circular. Assume an internal environment, 1-hour fire resistance and f_{ck} = 50 MPa.

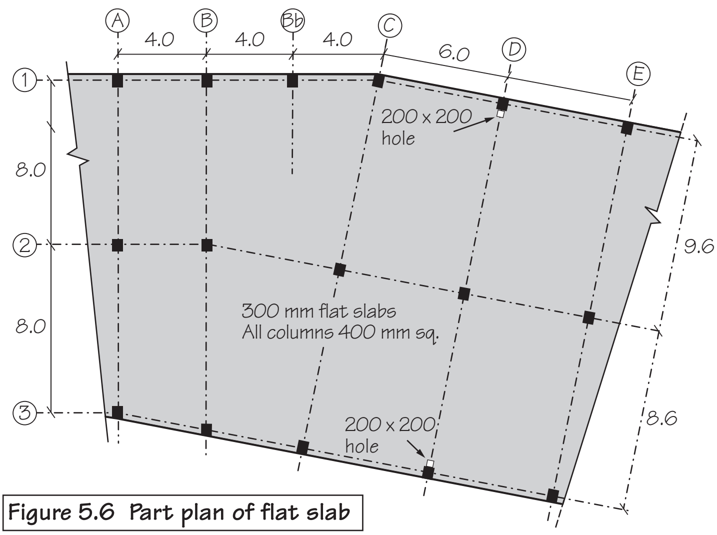


Figure 5.6 Part plan of flat slab

5.3.1 Design forces

In order to determine design forces for this column it is first necessary to determine vertical loads and 1st order moments.

5.3.2 Load take-down

Actions:

Roof:

$g_k = 8.5$, $q_k = 0.6$ — EC1-1-1: 6.3.4, NA & Table NA.7

Floors:
$g_k = 8.5$, $q_k = 4.0$ — Section 3.4

In keeping with Section 3.4 use coefficients to determine loads in take-down. — Section 3.4

Consider spans adjacent to column C2: — Table C3
Along grid C, consider spans to be 9.6 m and 8.6 m and C2 to be the internal of 2-span element.
Therefore elastic reaction factor = 0.63 + 0.63 = 1.26
Along grid 2 consider spans to be 6.0 m and 6.2 m and internal of multiple span.
Elastic reaction factor = 0.5 + 0.5 = 1.00

Load take-down for column C2.

Item	Calculation	G_k		Q_k	
		From item	Cumulative total	From item	Cumulative total
Roof	$= [erf_y \times (l_{z1} + l_{z2})/2] \times [erf_z \times (l_{y1} + l_{y2})/2] \times (g_k + q_k)$ $= [1.0 \times (6.0 + 6.2)/2] \times [1.26 \times (9.6 + 8.6)/2] \times (8.5 + 0.6)$ $= 69.9 \times (8.5 + 0.6)$ =	594.5		42.0	
Col 8 – R	$= \pi (0.465/2)^2 \times (4.5 - 0.3) \times 25$ =	17.9	**612.4**		**42.0**
8th	$= 1.0 \times (6.0 + 6.2)/2 \times 1.26 \times (9.6 + 8.6)/2 \times (8.5 + 4.0)$ =	594.5		279.7	
Col 7 – 8	as before	17.9	**1224.8**		**321.7**
7th	a.b.	594.5		279.7	
Col 6 – 7	a.b.	17.9	**1837.2**		**601.4**
6th	a.b.	594.5		279.7	
Col 5 – 6	a.b.	17.9	**2449.6**		**881.1**
5th	a.b.	594.5		279.7	
Col 4 – 5	$= 0.5 \times 0.5 \times (4.5 - 0.3) \times 25$ =	26.3	**3070.4**		**1160.8**
4th	as before	594.5		279.7	
Col 3 – 4	a.b.	26.3	**3691.2**		**1440.5**
3rd	a.b.	594.5		279.7	
Col 2 – 3	a.b.	26.3	**4312.0**		**1720.2**
2nd	a.b.	594.5		279.7	
Col 1 – 2	a.b.	26.3	**4932.8**		**1999.7**
1st	a.b.	594.5		279.8	
Col G – 1	a.b.	26.3	**5553.6**		**2279.5**
At above ground floor		—	**5553.6**	—	**2279.5**

5.3.3 Design axial load, ground–1st floor, N_{Ed}

a) Axial load to Exp. (6.10)

$N_{Ed} = \gamma_G G_k + \gamma_Q Q_{k1} + \psi_0 \gamma_Q Q_{ki}$

ECO: Exp. (6.10) & NA

where

ECO: A1.2.2 & NA

$\gamma_G = 1.35$

$\gamma_Q = 1.50$

$\psi_{0,1} = 0.7$ (offices)

Q_{k1} = leading variable action (subject to reduction factor α_A or α_n)

Q_{ki} = accompanying action (subject to α_A or α_n)

EC1-1-1: 6.3.1.2 (10), 6.3.1.2 (11), & NA

where

$\alpha_A = 1 - A/1000 \geq 0.75$

$= 1 - 9 \times 69.9/1000 = 0.37 \geq 0.75$

$= 0.75$

$\alpha_n = 1.1 - n/10$ for $1 \leq n \leq 5$

$= 0.6$ for $5 \leq n \leq 10$ and

$= 0.5$ for $n > 10$

where

n = number of storeys supported

$\alpha_n = 0.6$ for 8‡ storeys supported

$\therefore$ as $\alpha_n < \alpha_A$, use $\alpha_n = 0.6$

Assuming the variable action of the roof is an independent variable action:

$N_{Ed} = 1.35 \times 5553.6 + 1.5 \times (2279.5 - 42.0) \times 0.6 + 0.7 \times 1.5 \times 42.0$

$= 1.35 \times 5553.6 + 1.5 \times 2237.5 + 0.7 \times 1.5 \times 42.0$

$= 7497.4 + 2013.8 + 44.1$

$= 9555.3$ kN

To Exp. (6.10), $N_{Ed} = 9555.3$ kN

b) Axial load to Exp. (6.10a)

$N_{Ed} = \gamma_G G_k + \psi_{0,1} \gamma_Q Q_{k1} + \psi_{0,1} \gamma_Q Q_{ki}$

ECO: Exp. (6.10a) & NA

$= 1.35 \times 5553.6 + 0.7 \times 1.5 \times 0.6\ (279.8 + 1999.7)$

$= 7497.4 + 1436.1$

$= 8933.4$ kN

To Exp. (6.10a), $N_{Ed} = 8933.4$ kN

c) Axial load to Exp. (6.10b)

$N_{Ed} = \xi \gamma_G G_k + \gamma_Q Q_{k1} + \psi_{0,1} \gamma_Q Q_{ki}$

ECO: Exp. (6.10) & NA

‡ According to BS EN 1991–1–1 6.3.1.2[11] the imposed load on the roof is category H and therefore does not qualify for reduction factor α_n.

assuming the variable action of the roof is an independent variable action:

$= 0.925 \times 1.35 \times 5553.6 + 1.5 \times (2279.5 - 42.0) \times 0.6 + 0.7 \times 1.5 \times 42.0$
$= 1.25 \times 5553.6 + 1.5 \times 2237.5 \times 0.6 + 0.7 \times 1.5 \times 42.0$
$= 6942.1 + 2013.8 + 44.1$
$= 9000.0$ kN

To Exp. (6.10b), $N_{Ed} = 9000.0$ kN

5.3.4 First order design moments, M

a) Grid C

Consider grid C to determine M_{yy} in column (about grid 2)

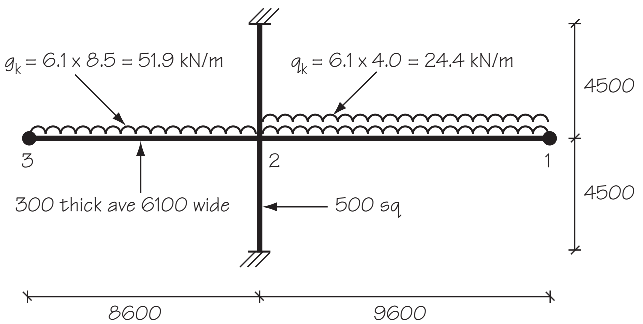


Figure 5.7 Subframe on column C2 along grid C

Actions:

$g_k = (6.0 + 6.2)/2 \times 8.5 = 51.9$ kN/m

$q_k = (6.0 + 6.2)/2 \times 4.0 = 24.4$ kN/m

Relative stiffness of lower column:

Assuming remote ends of slabs are pinned, relative stiffness

$$= \frac{b_{lc}d_{lc}^3/L_{lc}}{b_{lc}d_{lc}^3/L_{lc} + b_{uc}d_{uc}^3/L_{uc} + 0.75b_{23}d_{23}^3/L_{23} + 0.75b_{21}d_{21}^3/L_{21}}$$

where

b = breadth
d = depth
L = length
$_{lc}$ = lower column, $_{uc}$ = upper column
$_{23}$ = beam 23, similarly $_{21}$ = beam 21

$$= \frac{0.5^4/4.5}{2 \times 0.5^4/4.5 + 0.75 \times 6.1 \times 0.3^3/8.6 + 0.75 \times 6.1 \times 0.3^3/9.6}$$

$= 0.0139/(0.0278 + 0.0144 + 0.0129) = 0.252$

1st order moment using Exp. (6.10)

FEM 23[‡] $= 1.35 \times 51.9 \times 8.6^2/12 = 431.8$ kNm

FEM 21 $= (1.35 \times 51.9 + 1.5 \times 24.4) \times 9.6^2/12$

$= 106.7 \times 9.6^2/12 = 819.5$ kNm

$M_{lower,yy} = 0.252 \times [819.5 - 431.8] = 97.7$ kNm

1st order moment using Exp. (6.10a)

FEM 23 $= 1.25 \times 51.9 \times 8.6^2/12 = 399.8$ kNm

FEM 21 $= (1.25 \times 51.9 + 1.5 \times 24.4) \times 9.6^2/12$

$= 101.5 \times 9.6^2/12 = 779.5$ kNm

$M_{lower,yy} = 0.252 \times (779.5 - 399.8) = 95.7$ kNm

1st order moment using Exp. (6.10b)

FEM 23 $= 1.35 \times 51.9 \times 8.6^2/12 = 431.8$ kNm

FEM 21 $= (1.35 \times 51.9 + 0.7 \times 1.5 \times 24.4) \times 9.6^2/12$

$= 95.7 \times 9.6^2/12 = 735.0$ kNm

$M_{lower,yy} = 0.252 \times (735.0 - 431.8) = 76.4$ kNm

∴ Exp. (6.10a) critical

b) Grid 2

Consider grid 2 to determine M_{zz} in column (about grid C)

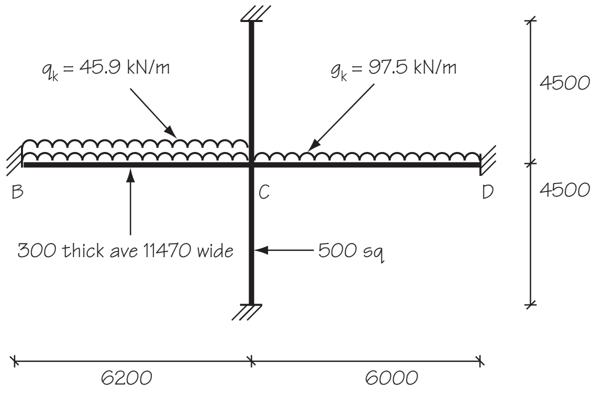


Figure 5.8 Subframe on column C2 along grid 2

Actions:

$g_k = 0.63 \times (8.6 + 9.6) \times 8.5$

$= 11.47 \times 8.5 = 97.5$ kN/m

$q_k = 11.47 \times 4.0 = 45.9$ kN/m

Relative stiffness of lower column: **Cl. 5.8.3.2(4)** PD 6687

Assuming remote ends of slabs are fixed, relative stiffness

‡ FEM 23 = Fixed end moment in span 23 at grid 2.

$$= \frac{0.5^4/4.5}{2 \times 0.5^4/4.5 + 11.47 \times 0.3^3/6.2 + 11.47 \times 0.3^3/6.0}$$

$= 0.0139/(0.0278 + 0.0500 + 0.0516) = 0.107$

1st order moment using Exp. (6.10)

FEM CB = $(1.35 \times 97.5 + 1.5 \times 45.9) \times 6.2^2/12$

$= 200.5 \times 6.2^2/12 = 642.3$ kNm

FEM CD = $1.35 \times 97.5 \times 6.0^2/12 = 394.9$ kNm

$M_{lower,zz} = 0.107 \times (642.3 - 394.9) = 26.5$ kNm

1st order moment using Exp. (6.10a)

FEM CB = $1.25 \times 97.5 \times 6.0^2/12 = 365.6$ kNm

FEM CD = $(1.25 \times 97.5 + 1.5 \times 45.9) \times 6.22/12$

$= 190.7 \times 6.22/12 = 611.0$ kNm

$M_{lower,zz} = 0.107 \times (611.0 - 365.6) = 26.3$ kNm

1st order moment using Exp. (6.10b)

FEM CB = $(1.35 \times 97.5 + 0.7 \times 1.5 \times 45.9) \times 6.2^2/12$

$= 190.7 \times 6.2^2/12 = 576.0$ kNm

FEM CD = $1.35 \times 97.5 \times 6.0^2/12 = 394.9$ kNm

$M_{lower,zz} = 0.107 \times (576.0 - 394.9) = 19.4$ kNm

∴ Exp. (6.10a) critical again

5.3.5 Summary of design forces in column C2 ground–1st

Design forces

Method	N_{Ed}	M_{yy} **about grid 2**	M_{zz} **about grid C**
Using Exp. (6.10)	9555.3 kN	97.7 kNm	26.5 kNm
Using Exp. (6.10a)	8933.4 kN	95.7 kNm	26.3 kNm
Using Exp. (6.10b)	9000.0 kN	76.4 kNm	19.4 kNm

Notes:

1) To determine maximum 1st order moments in the column, maximum out-of-balance moments have been determined using variable actions to one side of the column only. The effect on axial load has, conservatively, been ignored.

2) It may be argued that using coefficients for the design of the slab and reactions to the columns does not warrant the sophistication of using Exps (6.10a) and (6.10b). Nevertheless, there would appear to be some economy in designing the column to Exp. (6.10a) or Exp. (6.10b) rather than Exp. (6.10). The use of Exp. (6.10a) or Exp. (6.10b) is perfectly valid and will be followed here.

To avoid duplicate designs for both Exps (6.10a) and (6.10b), a worse case of their design forces will be used, thus:

$N_{Ed} = 9000$ kN, $M_{yy} = 95.7$ kNm, $M_{zz} = 26.3$ kNm

5.3.6 Design: cover

$c_{nom} = c_{min} + \Delta c_{dev}$	Exp. (4.1)
where	
$c_{min} = \max[c_{min,b}; c_{min,dur}]$	
where	
$c_{min,b}$ = diameter of bar. Assume 32 mm bars and 8 mm links.	Cl. 4.4.1.2(3)
= 32 – 8 = 24 mm to link	
$c_{min,dur}$ = minimum cover due to environmental conditions. Assume XC1.	
$c_{min,dur}$ = 15 mm	BS 8500-1: Table A4
c_{min} = 24 mm, say 25 mm to link	
Δc_{dev} = 10 mm	
∴ c_{nom} = 25 + 10 = 35 mm	Cl. 4.4.1.3 & NA

5.3.7 Design: fire resistance

Check validity of using Method A and Table 5.2a	EC2-1-2: 5.3.2, Table 5.2a
a) Check $l_{0,fi} \leq 3.0$ m	
where	
l_0 = effective length of column in fire	
= 0.5 × clear height	
= 0.5 × (4500 – 300)	
= 2100 mm OK	
b) Check $e \leq e_{max} = 0.15h = 0.15 \times 500 = 75$ mm	
$e = M_{0Ed,fi}/N_{0Ed,fi}$	
$= M_0/N_{Ed}$	EC2-1-2: 5.3.2(2)
$= 99.5 \times 10^6/8933 \times 10^3 = 11$ mm OK	
c) Check amount of reinforcement ≤ 4% OK	
Assuming μ_{fi} = 0.7	
b_{min} = 350 with	EC2-1-2: Table 5.2a
a_{min} = 40 mm OK	
For fire using Method A and Table 5.2a is valid	

5.3.8 Structural design: check slenderness

Effective length, l_0:	
$l_0 = 0.5l\,[1 + k_1/(0.45 + k_1)]^{0.5}\,[1 + k_2/(0.45 + k_2)]^{0.5}$	Exp. (5.15)
where	
k_1 and k_2 are relative flexibilities at top and bottom of the column.	
$k_i = (EI_{col}/l_{col})/\Sigma(2EI_{beam}/l_{beam}) \geq 0.1$	PD 6687[6]‡

‡ PD 6687 states that to allow for cracking, the contribution of each beam should be taken as $2EI/l_{beam}$

Critical direction is where k_1 and k_2 are greatest i.e. where slab spans are greater	How to[8]: Columns

$$k_1 = k_2 = \frac{b_{lc}d_{lc}^3/L_{lc}}{2b_{23}d_{23}^3/L_{23} + 2b_{21}d_{21}^3/L_{21}}$$

$= (0.5^4/4.5)/(2 \times 6.1 \times 0.3^3/8.6 + 2 \times 6.1 \times 0.3^3/9.6)$

$= (0.0625)/(0.0383 + 0.0343)$

$= 0.86$

$l_0 = 0.5\ (4500 - 300)\ [1 + 0.86/(0.45 + 0.86)]^{0.5}\ [1 + 0.86/(0.45 + 0.86)]^{0.5}$

$l_0 = 0.5 \times 4200 \times 1.66$

$= 0.828 \times 4200 = 3478$ mm

Slenderness ratio, λ: — Cl. 5.8.3.2(1)

$\lambda = l_0/i$

where

i = radius of gyration = $(I/A)^{0.5} = h/12^{0.5}$

$\therefore\ \lambda = 3478 \times 12^{0.5}/500 = 24.1$

Limiting slenderness ratio, λ_{lim}: — Cl. 5.8.3.1(1) & NA Exp. (5.13N)

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

where

$A = 1/(1 + 0.2\ \phi_{ef})$. Assume 0.7 as per default

$B = (1 + 2\omega)^{0.5}$. Assume 1.1 as per default

$C = 1.7 - r_m$

where

$r_m = M_{01}/M_2 = -84.9/109.3 = -0.78$

$C = 1.7 + 0.78 = 2.48$

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

$= 8933 \times 10^3/(500^2 \times 0.85 \times 50/1.5)$

$= 1.26$

$\therefore\ \lambda_{lim} = 20 \times 0.7 \times 1.1 \times 2.48/1.26^{0.5} = 34.0$

$\therefore$ as $\lambda < \lambda_{lim}$ column is not slender and 2nd order moments are not required

5.3.9 Design moments, M_{Ed}

$M_{Ed} = M + e_i N_{Ed} \geq e_0 N_{Ed}$ — Cl. 5.8.8.2(1), 6.1(4)

where

M = moment from 1st order analysis

$e_i N_{Ed}$ = effect of imperfections — Cl. 5.8.8.2(1)

where

e_i = $l_0/400$ — Cl. 5.2.7

e_0N_{Ed} = minimum eccentricity — Cl. 6.1(4)

where

e_0 = $h/30 \geq 20$ mm

M_{Edyy} = $95.7 + (3570/400) \times 8933 \times 10^{-3} \geq 0.02 \times 8933$

= $95.7 + 79.7 \geq 178.7$

= $175.4 < 178.7$ kNm

M_{Edzz} = $18.8 + 79.7 \geq 178.7$

= 178.7 kNm ∴ Both critical.

However, imperfections need only be taken in one direction – where they have the most unfavourable effect

∴ Use M_{Edzz} = 178.7 with M_{Edyy} = 95.7 kNm — Cl. 5.8.9(2)

5.3.10 Design using charts

$M_{Edyy}/bh^2f_{ck} = 178.9 \times 10^6/(500^3 \times 50) = 0.03$ — Figs C5a) to C5e)

$N_{Ed}/bhf_{ck} = 9000 \times 10^3/(500^2 \times 50) = 0.72$

Choice of chart based on d_2/h

where

d_2 = depth to centroid of reinforcement in half section assuming 12 bar arrangement with H32s

d_2 = $35 + 8 + (32/2) + (2/6)\,[500 + 2 \times (35 + 8 + 32/2\,)/3]$

= $59 + (1/3) \times 127$

= 101

∴ $d_2/h = 101/500 = 0.2$ Use Figure C5d)

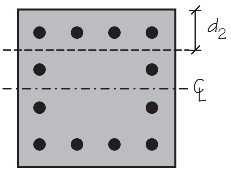


Figure 5.9 Depth, d_2, to centroid of reinforcement in half section

From Figure C5d) — Fig. C5d)

A_sf_{yk}/bhf_{ck} = 0.30

A_s = $0.29 \times 500 \times 500 \times 50/500$

= 7500 mm^2

Try 12 no. H32 (9648 mm^2)‡

‡ Using design actions to Exp. (6.10) would have resulted in a requirement for 8500 mm^2.

5.3.11 Check biaxial bending

Cl. 5.8.9

Slenderness: $\lambda_y \approx \lambda_z$ ∴ OK.

Cl. 5.8.9(3)

Eccentricities: as $h = b$ check e_y/e_z

M_{Edz} critical. (Imperfections act in z direction.)

$$e_y/e_z = \frac{95.7 \times 10^6/9000 \times 10^3}{178.7 \times 10^6/9000 \times 10^3}$$

$= 0.54$ i.e. > 0.2 and < 5

Cl. 5.9.3(3), Exp. (5.38b)

∴ Design for biaxial bending.

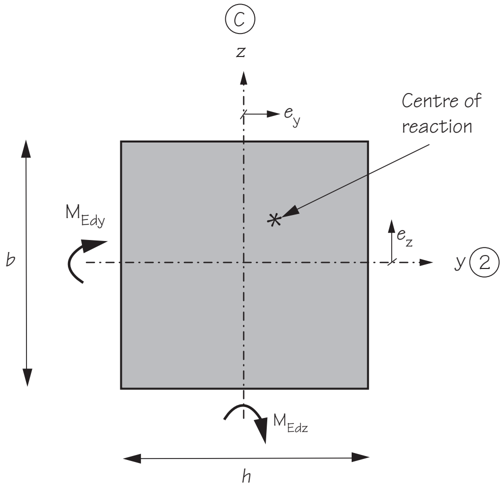


Figure 5.10 Eccentricities

5.3.12 Design for biaxial bending

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

Cl. 5.9.3(4), Exp. (5.39)

For load case 2

where

$M_{Edz} = 178.7$ kNm

$M_{Edy} = 95.7$ kNm

$M_{Rdz} = M_{Rdy}$ = moment resistance. Using charts:

From Figure C4d), for $d_2/h = 0.20$ and

$A_s f_{yk}/bhf_{ck} = 9648 \times 500/500 \times 500 \times 50$

$= 0.39$

$N_{Ed}/bhf_{ck} = 9000 \times 10^3/(500^2 \times 50)$

$= 0.72$

$M_{Rd}/bh^2 f_{ck} = 0.057$

Fig. C5d)

∴ $M_{Rd} \approx 0.057 \times 500^3 \times 50$

$= 356.3$ kNm

a = exponent dependent upon N_{Ed}/N_{Rd} — Cl. 5.8.3(4)

where

$N_{Rd} = A_c f_{cd} + A_s f_{yd}$

$= 500 \times 500 \times 0.85 \times 50/1.5 + 9648 \times 500/1.15$

$= 7083 + 3216$

$= 10299$ kN

$N_{Ed}/N_{Rd} = 9000/10299 = 0.87.$

Interpolating between values given for $N_{Ed}/N_{Rd} = 0.7$ (1.5) and for $N_{Ed}/N_{Rd} = 1.0$ (2.0) — Notes to Exp. (5.39)

$\therefore a = 1.67$

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

$(178.7/356.3)^{1.67} + (95.7/356.3)^{1.67} = 0.32 + 0.11$

$= 0.43$ i.e. < 1.0 $\therefore$ OK

Use 12 no. H32

5.3.13 Links

Minimum diameter of links: $= \phi/4 = 32/4$ — Cl. 9.5.3 & NA

$= 8$ mm

Spacing, either: — Cl. 9.5.3(3), 9.5.3(4)

a) $0.6 \times 20 \times \phi = 12 \times 32 = 384$ mm,

b) $0.6 \times h = 0.6 \times 500 = 300$ mm or

c) $0.6 \times 400 = 240$ mm.

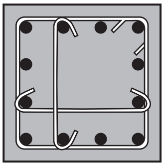
$\therefore$ Use H8 links at 225 mm cc

Number of legs:

Bars at 127 mm cc i.e. < 150 mm $\therefore$ no need to restrain bars in face but good practice suggests alternate bars should be restrained. — Cl. 9.5.3(6), SMDSC: 6.4.2

$\therefore$ Use single leg on face bars both ways @ 225 mm cc

5.3.14 Design summary



12 H32
H8 links @ 225 cc
35 mm to link
500 mm sq
$f_{ck} = 50$ MPa

Figure 5.11 Design summary: internal column

5.4 Small perimeter column subject to two-hour fire resistance

This calculation is intended to show a small slender column subject to a requirement for 2-hour fire resistance. It is based on the example shown in Section 4.2.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details: Small perimeter column subject to two-hour fire resistance	Calculated by: chg	Job no. CCIP – 041
		Checked by: web	Sheet no. 1
		Client: TCC	Date: Oct 09

The middle column, B, in Figure 4.5, supports two levels of storage loads and is subject to an ultimate axial load of 1824.1 kN‡. From analysis it has moments of 114.5 kNm in the plane of the beam and 146.1 kNm perpendicular to the beam (i.e. about the z axis).

The column is 350 mm square, 4000 mm long, measured from top of foundation to centre of slab. It is supporting storage loads, in an external environment (but not subject to de-icing salts) and is subject to a 2-hour fire resistance requirement on three exposed sides. Assume the base is pinned.

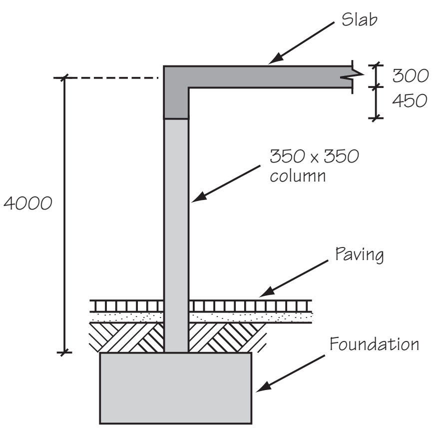


Figure 5.12 Perimeter column

5.4.1 Cover

Nominal cover, c_{nom}

$c_{nom} = c_{min} + \Delta c_{dev}$ — **Exp. (4.1)**

where

$c_{min} = \max[c_{min,b}; c_{min,dur}]$

‡ $G_k = 562.1$; $Q_k = 789.1$; as column supports loads from 2 levels $\alpha_n = 0.9$; as imposed loads are from storage $\psi_0 = 1.0$ ∴ $g_Q = 1.50$ and $g_Q = 1.35$. ∴ Ultimate axial load, $N_{Ed} = 1.35 \times 562.1 + 1.5 \times 0.9 \times 789.1 = 1824.1$ kN.

where

$c_{min,b}$ = diameter of bar. Assume 32 mm main bars and 10 mm links

$c_{min,dur}$ = minimum cover due to environmental conditions. Assuming primarily XC3/XC4, secondarily XF1, $c_{min,dur}$ = 25 mm — Cl. 4.4.1.2(3)

Δc_{dev} = allowance in design for deviation = 10 mm — BS 8500-1[14] : Table A4

∴ Try c_{nom} = 32 + 10 = 42 mm to main bars

or = 25 + 10 = 35 mm to 8 mm links

Try c_{nom} = 35 mm to 8 mm links.

5.4.2 Fire resistance

a) Check adequacy of section for R120 to Method A

Axis distance available = 43 mm + $\phi/2$

Required axis distance to main bars, a for 350 mm square column — EC1-1-2: 5.3.1(1) & NA 5.3.2, Table 5.2a

For μ_{fi} = 0.5, a = 45 mm; and

for μ_{fi} = 0.7, a = 57 mm, providing:

- 8 bars used – OK but check later
- $l_{0,fi} \le 3$ m – OK but check
- $e \le e_{max} = 0.15h = 0.15 \times 350 = 52$ mm

but $e = M_{OEd,fi}/N_{OEd,fi}$

$= 0.7 \times 146.1 \times 10^6 / 0.7 \times 1824.1 \times 10^3$

$= 80$ mm ∴ no good

Try Method B

b) Check adequacy of section for R120 to Method B — EC2-1-2: 5.3.3, Table 5.2b

Determine parameters n, ω, and e, and check λ_{fi}.

Assume 4 no. H32 + 4 no. H25 = (5180 mm^2: 4.2%)

(say 4.2% OK – integrity OK) — Cl. 9.5.2(3)

$n = N_{OEd,fi}/0.7(A_c f_{cd} + A_s f_{yd})$ — EC2-1-2: Exp. (5.8a)

$= 0.7 \times 1824.1 \times 10^3 / 0.7\,(350 \times 350 \times \alpha_{cc} \times f_{ck}/\gamma_C + 5180 \times 500/\gamma_S)$

$= 1276.9 \times 10^3 / 0.7\,(350 \times 350 \times 0.85 \times 30/1.5 + 5180 \times 500/1.15)$

$= 1276.9 \times 10^3 / 0.7\,(2082.5 + 2252.0)$

$= 0.42$ OK

ω = mechanical ratio — EC2-1-2: 5.3.3(2)

$= A_s f_{yd}/A_c f_{cd} \le 1.0$

$= 2252/2082$

$= 1.08 \ge 1$

But say within acceptable engineering tolerance ∴ use $\omega = 1.0$ OK

e = first order eccentricity — EC2-1-2: Exp. (5.8b)

$= M_{OEd,fi}/N_{OEd,fi}$

$= 0.7 \times 146.1 \times 10^6/0.7 \times 1824.1 \times 10^3$ — EC2-1-2: 2.4.2(3)

$= 80$ mm as before $\equiv 0.23h$. OK

λ_{fi} = slenderness in fire

$= l_{0,fi}/i$

where — EC2-1-2: 5.3.2(2)

$l_{0,fi}$ = effective length of column in fire

$= 0.7l = 0.7 \times 4000 = 2800$ mm — Note 2

i = radius of gyration

$= h/3.46$ for a rectangular section

$\therefore \lambda_{fi} = 2800/(350/3.46)$

$= 27.7 < 30$ ∴ OK

Table 5.2b valid for use in this case.

Interpolating from BS EN 1992–1–2 Table 5.2b for $n = 0.42$ and $\omega = 1.0$, column width = 350 mm and axis distance = say, 48 mm

∴ Axis distance = 43 mm + $\phi/2$ is OK

c) As additional check, check adequacy of section to Annex B3 and Annex C — EC2-1-2: 5.3.3(1), Annex C & NA

Using BS EN 1992–1–2 Table C.8

For $\omega = 1.0$, $e = 0.25b$, R120, $\lambda = 30$ — EC2-1-2: Annex C(2)

and interpolating between $n = 0.3$ and $n = 0.5$,

$b_{min} = 350$ mm, $a_{min} = 48$ mm.

∴ Axis distance = 43 mm + $\phi/2$ is OK

∴ 4 no. H32 + 4 no. H25 with 35 mm cover to 8 mm links

($a = 55$ mm min.) OK

5.4.3 Structural design: check slenderness about z axis

Effective length, l_0, about z axis:

$l_{0z} = 0.5l\,[1 + k_1/(0.45 + k_1)]^{0.5}\,[1 + k_2/(0.45 + k_2)]^{0.5}$ — **Exp. (5.15)**

where — PD 6687: 2.10

l = clear height between restraints

$= 4000 - 300/2 = 3850$ mm

k_1, k_2 = relative flexibilities of rotational restraints at ends 1 and 2 respectively

$k_1 = [EI_{col}/l_{col}]/[2EI_{beam1}/l_{beam1} + 2EI_{beam2}/l_{beam2}] \geq 0.1$ — **Cl. 5.8.3.2(3)**

where — PD 6687

Treating beams as rectangular and cancelling E throughout:

$I_{col}/l_{col} = 3504/(12 \times 3850) = 3.25 \times 10^5$

$I_{beam1}/l_{beam1} = 8500 \times 300^3/12 \times 6000$

$= 31.8 \times 10^5$

$I_{beam2}/l_{beam2} = 0$

$k_1 = 3.25/(2 \times 31.8) = 0.051 \geq 0.1$

$k_1 = 0.1$

k_2 = by inspection (pinned end assumed) $= \infty$

$\therefore\ l_{0z} = 0.5 \times 3850 \times [1 + 0.1/(0.45 + 0.1)\ ]^{0.5}\ [1 + \infty/(0.45 + \infty)]^{0.5}$
$= 0.5 \times 3850 \times 1.087 \times 1.41$
$= 0.77 \times 3850 = 2965$ mm

Slenderness ratio, λ_z: — Cl. 5.8.3.2(1)

$\lambda_z = l_{0z}/i$

where

i = radius of gyration = $h/3.46$

$\lambda_z = 3.46 l_{0z}/h = 3.46 \times 2965/350$ $= 29.3$

Limiting slenderness ratio, λ_{lim}: — Cl. 5.8.3.1(1) Exp. (5.13N)

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

where

$A = 0.7$
$B = 1.1$[‡]
$C = 1.7 - r_m$

where

$r_m = M_{01}/M_{02}$
say $M_{01} = 0$ (pinned end) $\therefore\ r_m = 0$

$C = 1.7 - 0 = 1.7$
n = relative normal force = $N_{Ed}/A_c f_{cd}$
$= 1824.1 \times 10^3/(350^2 \times 0.85 \times 30/1.5)$
$= 0.88$

$\therefore\ \lambda_{lim,z} = 20 \times 0.7 \times 1.1 \times 1.7/0.88^{0.5}$
$= 27.9$

$\therefore$ As $\lambda_z > \lambda_{lim,z}$ column is slender about z axis.

5.4.4 Check slenderness on y axis

Effective length, l_0, about z axis:

$l_{0y} = 0.5 l_y\ [1 + k_1/(0.45 + k_1)\ ]^{0.5}\ [1 + k_2/(0.45 + k_2)\ ]^{0.5}$ — Exp. (5.15)

where

l_y = clear height between restraints
= $4000 + 300/2 - 750 = 3400$ mm
k_1 = relative column flexibility at end 1
= $(I_{col}/l_{col})/[\Sigma 2(I_{beam}/l_{beam})]$

where

$I_{col}/l_{col} = 350^4/12 \times 3400 = 3.68 \times 10^5$

‡ On first pass the default value for B is used. It should be noted that in the final design $\omega = A_s f_{yd}/A_c f_{cd} = 6432 \times (500/1.15)\ /\ (350^2 \times 30 \times 0.85/1.5) = 2796/2082 = 1.34$. So $B = (1 + 2\ \omega)^{0.5} = (1 + 1.34)^{0.5} = 1.92$ and the column would not have been deemed 'slender'. $B = 1.1$ relates approximately to a column with $f_{ck} = 30$ MPa and $\rho = 0.4\%$. — Cl. 5.8.3.1(1), & NA, EC2-1-2: 5.3.3(2)

* PD 6687 states that to allow for cracking, the contribution of each beam should be taken as $2EI/l_{beam}$ — PD 6687*

Treating beams as rectangular	
$I_{beamAB}/l_{beamAB} = 350 \times 750^3/[12 \times (9000 - 350)]$	
$= 14.2 \times 10^5$	
$I_{beamBC}/l_{beamBC} = 350 \times 750^3/[12 \times (8000 - 350)]$	
$= 16.1 \times 10^5$	
$k_1 = 3.68/(2 \times (16.1 + 14.2) = 0.060 \geq 0.1$	
$k_1 = 0.1$	
$k_2 = \infty$ (pinned end assumed)	
$\therefore l_{0y} = 0.5 \times 3400\ [1 + 0.1/(0.45 + 0.1)\ ]^{0.5}\ [1 + \infty/(0.45 + \infty)\ ]^{0.5}$	Exp. (5.15)
$= 0.5 \times 3400 \times 1.087 \times 1.41$	
$= 0.77 \times 3400 = 2620$ mm	
Slenderness ratio, λ_y:	
$\lambda_y = 3.46 l_{0y}/h = 3.46 \times 2620/350 = 25.9$	
Limiting slenderness ratio, λ_{lim}:	
$\lambda_{lim,y} = \lambda_{lim,z} = 27.9$	
As $\lambda_y < \lambda_{lim,y}$, column ***not*** slender in y axis.	

5.4.5 Design moments: M_{Edz} about z axis

$M_{Edz} = \max[M_{02};\ M_{0Ed} + M_2;\ M_{01} + 0.5M_2]$	Cl. 5.8.8.2
where	
$M_{02} = M_z + e_i N_{Ed} \geq e_0 N_{Ed}$	Cl. 5.8.8.2(1), 6.1.4
where	
M_z = 146.1 kNm from analysis	
$e_i N_{Ed}$ = effect of imperfections	
where	
$e_i = l_0/400$	Cl. 5.2.7
e_0 = 20 mm	
$\therefore M_{02} = 146.1 + (2965/400) \times 1824.1 \geq 0.02 \times 1824.1$	
$= 146.1 + 13.4 > 36.5$	
= 159.5 kNm	
M_{0Ed} = equivalent 1st order moment at about z axis at about mid-height may be taken as M_{0ez} where	
$M_{0ez} = (0.6M_{02} + 0.4M_{01}) \geq 0.4M_{02}$	Cl. 5.8.8.2(2)
$= 0.6 \times 159.5 + 0.4 \times 0 \geq 0.4 \times 159.5$ = 95.7 kNm	
M_2 = nominal 2nd order moment = $N_{Ed} e_2$	Cl. 5.8.8.2(3)
where	
$e_2 = (1/r)\ l_0^2/10$	Cl. 5.8.8. 3
where	
$1/r$ = curvature = $K_v K_\varphi [f_{yd}/(E_s \times 0.45d)]$	Exp. (5.34)
where	
K_v = a correction factor for axial load	
$= (n_u - n)/(n_u - n_{bal})$	

where

$n_u = 1 + \omega$

where

ω = mechanical ratio

$= A_s f_{yd}/A_c f_d$

= 1.08 as before

$n_u = 2.08$

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

= 1824.1/2082 = 0.88

n_{bal} = the value of n at maximum moment resistance

= 0.40 (default)

K_v = (2.08 – 0.88)/(2.08 – 0.40)

= 1.20/1.68 = 0.71

K_φ = a correction factor for creep

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

where

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

= 0.35 + 30/200 – 29.3/150

= 0.35 + 0.15 – 0.195

= 0.305

φ_{ef} = effective creep coefficient‡ | Cl. 5.8.4(2)

$= \varphi_{(\infty,t0)} M_{0,Eqp}/M_{0Ed}$

where

$\varphi_{(\infty,t0)}$ = final creep coefficient | Cl. 3.1.4(2)

= from Figure 3.1 for inside conditions | Fig. 3.1a

h = 350 mm, C30/37, $t_0 = 15$

≈ 2.4

$M_{0,Eqp}$ = 1st order moment due to quasi-permanent loads

$$\approx \frac{G_k + \varphi_2 Q_k}{\xi\gamma_G G_k + \varphi_0\gamma_Q Q_k} \times M_z + e_i N_{Ed}$$

$$= \frac{63.3 + 0.8 \times 46.0}{1.35 \times 63.3 + 1.5 \times 46.0} \times M_z + e_i N_{Ed}$$

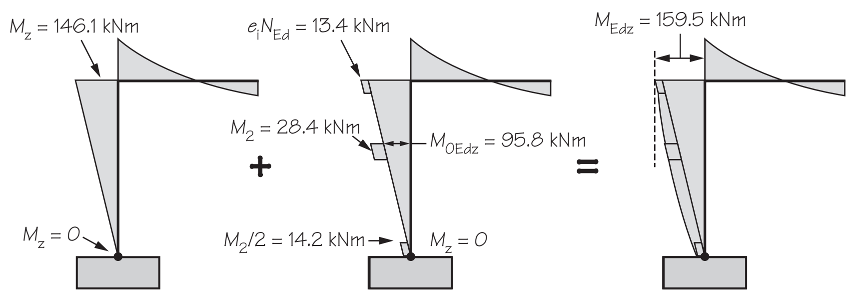
$$= \frac{100.1}{154.5} \times 146.1 + 13.4$$

= 108.1 kNm

$M_{0Ed} = M_{02}$ = 159.5 kNm

‡ With reference to Exp. (5.13N), φ_{ef} may be taken as equal to 2.0. However, for the purpose of illustration the full derivation is shown here. | Exp. (5.1.3N)

$$K_\varphi = 1 + 0.305 \times 2.4 \times 108.1/159.5$$
$$= 1.50$$
$$f_{yd} = 500/1.15 = 434.8 \text{ MPa}$$
$$E_s = 200000 \text{ MPa}$$ Cl. 3.2.7(3)
$$d = \text{effective depth}$$
$$= 350 - 35 - 8 - 16 = 291 \text{ mm}$$
$$1/r = 0.71 \times 1.50 \times 434.8/(200000 \times 0.45 \times 291)$$
$$= 0.0000177$$
$$l_0 = 2965 \text{ mm as before}$$
$$e_2 = (1/r)\, l_0^2/10$$
$$= 0.0000177 \times 2965^2/10 \qquad = \underline{15.6 \text{ mm}}$$
$$\therefore M_2 = N_{Ed} e_2 = 1824.1 \times 10^3 \times 15.6 \qquad = \underline{28.4 \text{ kNm}}$$
$$M_{01} \qquad = \underline{0}$$
$$\therefore M_{Edz} = \max[M_{02z};\ M_{0Edz} + M_2;\ M_{01} + 0.5M_2]$$
$$= \max[159.5;\ 95.7 + 28.4;\ 0 + 28.4/2] \qquad = \underline{159.5 \text{ kNm}}$$



a) 1st order moments from analysis

b) Including 2nd order moments: $M_{Edz} = \max[M_{02}, M_{0Ed} + M_2, M_{01} = 0.5M_2]$

c) Design moments: M_{Edz} about z axis

Figure 5.13 Design moments M_{Edz}

5.4.6 Design moments: M_{Edy} about y axis

$$M_{Edy} = \max[M_{02y};\ M_{0Edy} + M_2;\ M_{01} + 0.5M_2]$$

where

$$M_{02y} = M_y + e_i N_{Ed} \geq e_0 N_{Ed}$$
$$= 114.5 + 13.4^{\S} \geq 36.7 \text{ kNm}$$
$$= 127.9 \text{ kNm}$$
$$M_{0Edy} = (0.6M_{02y} + 0.4\, M_{01y}) \geq 0.4M_{02y}$$
$$= 0.6 \times 114.5 + 0.4 \times 0$$

§ Imperfections need to be taken into account in one direction only. Cl. 5.8.9(2)

$= 68.7$ kNm

$M_2 = 0$ (as column is not slender not slender about y axis).

$\therefore M_{Edy} = 127.9$ kNm

5.4.7 Design in each direction using charts

In z direction: $N_{Ed}/bhf_{ck} = 1824.1 \times 10^3/(350^2 \times 30)$
$= 0.50$

$M_{Ed}/bh^2f_{ck} = 159.5 \times 10^6/(350^3 \times 30)$
$= 0.124$

Assuming 8 bar arrangement, centroid of bars in half section:

$d_2 \geq 35 + 8 + 16 + (350/2 - 35 - 8 - 16) \times 1/4$ — Fig. C4e)
$\geq 59 + 29 = 88$ mm

$d_2/h = 0.25$

From Figure C4e) — Fig. C4e)

$A_sf_{yk}/bhf_{ck} = 0.50$

$A_s = 0.50 \times 350^2 \times 30/500 = 3675$ mm^2

$\therefore$ 4 no. H32 + 4 no. T25 (5180 mm^2) OK.

In y direction: $M_{Ed}/bh^2f_{ck} = 127.9 \times 10^6/(350^3 \times 30)$
$= 0.10$

$N_{Ed}/bhf_{ck} = 0.50$

From Figure C4e)

$A_sf_{yk}/bhf_{ck} = 0.34$

$A_s = 0.34 \times 3502 \times 30/500 = 2499$ mm^2

$\therefore$ 4 no. H32 + 4 no. T25 (5180 mm^2) OK.

5.4.8 Check biaxial bending

$\lambda_y \approx \lambda_z \therefore$ OK. — **Exp. (5.38a)**

$e_z = M_{Edy}/N_{Ed}$

$e_y = M_{Edz}/N_{Ed}$

$$\frac{e_y/h_{eq}}{e_z/b_{eq}} = \frac{M_{Edz}}{M_{Edy}} = \frac{159.5}{127.9} = 1.25$$ — **Exp. (5.38b)**

$\therefore$ need to check biaxial bending

$(M_{Edz}/M_{Rdz})^a + (M_{Edy}/M_{Rdy})^a \leq 1.0$ — **Exp. (5.39)**

where

$M_{Rdz} = M_{Rdy}$ = moment resistance $\therefore$

Using Figure C4e) — Fig. C4e)

$A_sf_{yk}/bhf_{ck} = 5180 \times 500/(350^2 \times 30)$
$= 0.70$

for $N_{Ed}/bhf_{ck} = 0.50$

$M_{Ed}/bh^2f_{ck} = 0.160$

$\therefore M_{Rd} = 0.160 \times 350^3 \times 30$

$= 205.8$ kNm

a depends on N_{Ed}/N_{Rd}

where

$N_{Rd} = A_c f_{cd} + A_s f_{yd}$

$= 350^2 \times 0.85 \times 30/1.5 + 5180 \times 500/1.15$

$= 2082.5 + 2252.2$

$= 4332.7$ kN

$N_{Ed}/N_{Rd} = 1824.1/4332.7 = 0.42$

$\therefore a = 1.27$

$(159.5/205.8)^{1.27} + (114.5/205.8)^{1.27} = 0.72 + 0.47$ — Cl. 5.8.9(4)

$= 1.19$

$\therefore$ No good

$\therefore$ Try 8 no. T32 (6432 mm^2)

For $A_s f_{yk}/bhf_{ck} = 6432 \times 500/(350^2 \times 30)$

$= 0.88$

for $N_{Ed}/bhf_{ck} = 0.50$

$M_{Ed}/bh^2 f_{ck} = 0.187$ — Fig. C4e)

$\therefore M_{Rd} = 240.5$ kNm

Check biaxial bending

$(159.5/245.7)^{1.27} + (114.5/245.7)^{1.27} = 0.59 + 0.39 = 0.98$ OK

5.4.9 Check maximum area of reinforcement

$A_s/bd = 6432/350^2 = 5.2\% > 4\%$ — Cl. 9.5.2(3) & NA

However, if laps can be avoided in this single lift column then the integrity of the concrete is unlikely to be affected and 5.2% is considered OK. OK — PD 6687: 2.19

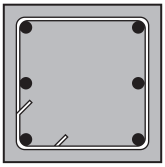
5.4.10 Design of links

Diameter min. = 32/4 = 8 mm — Cl. 9.5.3 & NA

Spacing max. = 0.6 × 350 = 210 mm — Cl. 9.5.3(3), 9.5.3(4)

$\therefore$ Use H8 @ 200 mm cc

5.4.11 Design summary



8 H32
H8 links @ 200 cc
35 mm cover to link
No laps in column section

Note
The beam should be checked for torsion.

Figure 5.14 Design summary: small perimeter column

6 Walls

6.0 General

Walls are defined as being vertical elements whose lengths are four times greater than their thicknesses. Their design does not differ significantly from the design of columns in that axial loads and moments about each axis are assessed and designed for.

The calculations in this section illustrate the design of a single shear wall.

Generally, the method of designing walls is as follows. In practice, several of these steps may be combined.

- Determine design life. — **EC0 & NA Table NA 2.1**
- Assess actions on the wall. — **EC1 (10 parts) & UK NAs**
- Determine which combinations of actions apply. — **EC0 & NA: Tables NA A1.1 & NA: A1.2(B)**
- Assess durability requirements and determine concrete strength. — **BS 8500–1**
- Check cover requirements for appropriate fire resistance period. — **Approved Document B EC2–1–2**
- Determine cover for fire, durability and bond. — **Cl. 4.4.1**
- Analyse structure for critical combination moments and axial forces. — **Section 5**
- Check slenderness and determine design moments. — **Section 5.8**
- Determine area of reinforcement required. — **Section 6.1**
- Check spacing of bars. — **Sections 8 & 9**

6.1 Shear wall

Example 6.1 shows the design of a simple linear shear wall as typically used in medium-rise buildings. Similar principles may be applied to walls that are shaped as C, L, T, Z and rectangles in-plan, but issues of limiting flange dimensions and shear at corners need to be addressed. The example shows only ULS design as, apart from minimum areas of steel to control cracking, SLS issues are generally non-critical in medium-rise structures. For shear walls in high-rise structures, reference should be made to specialist literature[29].

The example is intended to show how a shear wall providing part of the lateral stability in one direction in a medium rise structure might be designed by hand.

Axial loads and first order moments are determined. The design considers slenderness in order to determine design moments, M_{Ed}, in the plane perpendicular to the wall. The effects of allowing for imperfections are also illustrated.

The Concrete Centre™ PART OF THE MINERAL PRODUCTS ASSOCIATION	Project details			
	Shear wall	Calculated by chg	Job no. CCIP – 041
		Checked by web	Sheet no. 1
		Client TCC	Date Oct 09

Wall A is 200 mm thick and, in addition to providing vertical support to 200 mm flat slabs at roof level and floors 1 to 3, it helps to provide lateral stability to the four-storey office block. Assuming the stair itself provides no lateral stability, the wall is to be designed for the critical section at ground and first floor level using BS EN 1990 Exp. (6.10). The concrete is C30/37. The wall is supported on pad foundations and the ground floor is ground bearing.

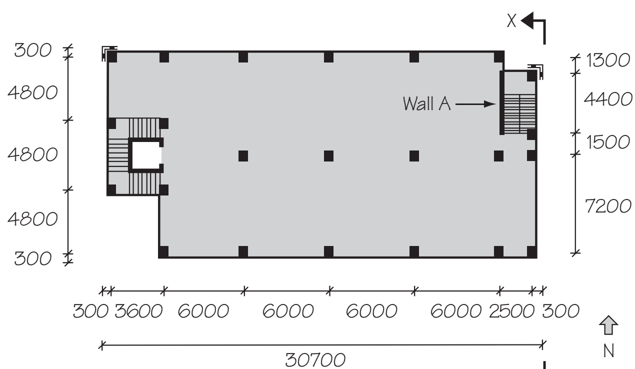


Figure 6.1 Typical floor plan

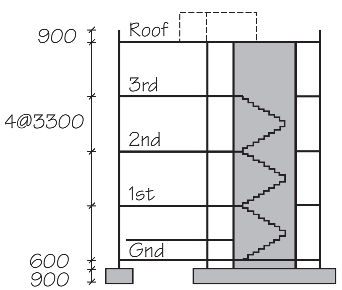


Figure 6.2 Section X–X

6.1.1 Actions

		Permanent actions g_k (kN/m²)	Variable actions q_k (kN/m²)	
				Section 2.8
Roof	Paving 40 mm	1.00		
	Waterproofing	0.50		
	Insulation	0.10		
	Suspended ceiling	0.15		
	Services	0.30		
	Self-weight 200 mm slab	5.00		Section 2.4.2
		7.05		
	Imposed load		0.60	
Floor slabs	Carpet	0.03		Section 2.8
	Raised floor	0.30		
	Suspended ceiling	0.15		
	Services	0.30		
	Self-weight 200 mm slab	5.00		
		5.78		Section 2.4.2
	Imposed load		2.50	
Ground floor slab (ground bearing)	Carpet	0.03		
	Raised floor	0.30		
	Services	0.15		
	Self-weight 200 mm slab	5.00		
		5.48		Section 2.4.2
	Imposed load		3.00	
Stairs	150 waist @ 30	4.40		Section 2.8
	Treads 0.15 × 0.25 × 25 × 4/2 =	1.88		
	Screed 0.05 × 22 =	1.10		
	Plaster	0.21		
	Tiles and bedding	1.00		
		8.59		
	Imposed load		2.50	Section 2.4.2
Cavity wall	102 mm brickwork	2.37		Section 2.8
	50 mm insulation	0.02		
	100 mm blockwork	1.40		
	Plaster	0.21		
		4.00		
RC wall	200 mm wall	5.00		
	Plaster both sides	0.42		Section 2.8
		5.42		
Wind	w_k =		1.10	EC1-1-4 & NA

6.1.2 Load take-down

Consider whole wall.

Item	Calculation	G_k From item	G_k Cum. total	Q_k From item	Q_k Cum. total
Roof	(6.0/2 + 2.5/2) × (4.4 + 1.5/2) × (7.05 + 0.6) =	154.3		13.1	
Roof	(6.0/2) × (1.3/2) × (7.05 + 0.6) =	13.7		1.2	
Wall	3.3 × 4.4 × 5.42 =	78.7			
		246.7		14.3	
	At above 3rd floor		**246.7**		**14.3**
3rd floor	(6.0/2) × (1.3/2 + 4.4 + 1.5/2) × (5.78 + 2.5) =	100.6		43.5	
Landing	(2.5/2 × 1.5/2) × (5.78 + 2.5)	11.6		5.0	
Wall	a. b.	78.7			
Stair	say 1.1 × 4.4 (8.59 + 2.5)	41.6		12.1	
		232.5		60.6	
	At above 2nd floor		**479.2**		**74.9**
2nd floor, landing, wall and stair a. b.		232.5		60.6	
	At above 1st floor		**711.7**		**135.5**
1st floor, landing, wall and stair a. b.		232.5		60.6	
	At above ground floor		**944.2**		**196.1**
Ground floor assume 1 m all round =	2 × (1.3/2 + 4.40 + 1.5/2) × (5.48 + 3.0) =	63.6		34.8	
250 mm wall to foundation	4.4 × 0.2 × 0.6 × 25 =	13.2			
		76.8			
	At above foundation		**1021.0**		**230.9**

6.1.3 Design actions due to vertical load at ground–1st

$G_k = 944.2$ $\qquad$ $G_k/\text{m} = 944.2/4.4 = 214.6$ kN/m

$Q_k = \alpha_n \times 196.1$

where

$\alpha_n = 1.1 - n/10$

where

n = no. of storeys qualifying for reduction‡

$= 3$

$= 1.1 - 3/10 = 0.8$

$\therefore\ Q_k = 0.8 \times 196.1 = 156.9$ kN $\qquad$ $Q_k/\text{m} = 156.9/4.4 = 35.7$ kN/m

EC1-1-1: 6.3.1.2(11) & NA

‡ Includes storeys supporting Categories A (residential and domestic), B (office), C (areas of congregation) and D (shopping), but excludes E (storage and industrial), F (traffic), G (traffic) and H (roofs).

6.1.4 Vertical loads from wind action: moments in plane

Consider wind loads, N–S

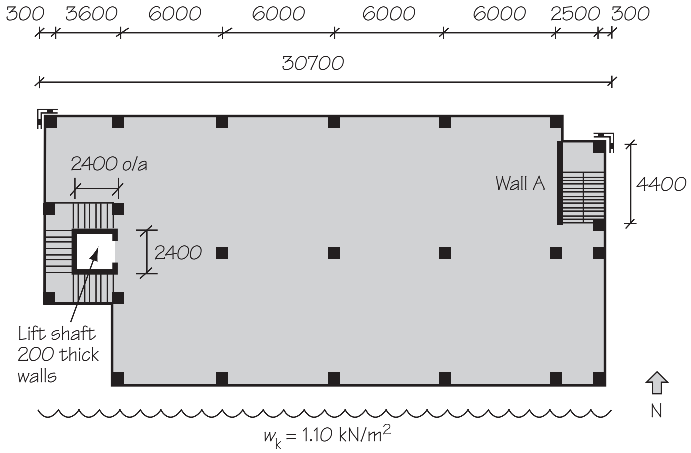


Figure 6.3 Lateral stability against wind loads N–S

Check relative stiffness of lift shaft and wall A to determine share of load on wall A.

Lift shaft: $I_{LS} = 2.4^4/12 - 2.0^4/12 - 0.2 \times 1.6^3/12$
$= 1.36\ m^4$

Wall A: $I_{WallA} = 0.2 \times 4.4^3/12$
$= 1.41\ m^4$

where I = inertia

∴ Wall A takes 1.41/(1.41 + 1.36) = 51% of wind load.

Check shear centre to resolve the effects of torsion.
Determine centre of gravity, CoG_L of the lift shaft.

	Area, A	Lever arm, x	Ax
2.4 × 2.4 =	5.76	1.2	6.912
–2.0 × 2.0 =	–4.00	1.2	–4.800
–1.6 × 0.2 =	–0.32	2.3	–0.732
	1.44		1.38

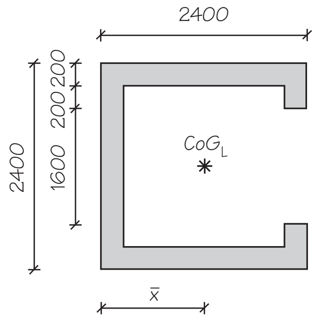


Figure 6.4 Lift shaft

$\bar{x} = Ax/A = 1.38/1.44 = 0.956$ m

i.e. from face of lift shaft to CoG of shaft

$= 2.40 - 0.956 = 1.444$ m

Shear centre, C_w of walls, from centreline of wall A

$$= \frac{I_{LS} \times (1.44 + 24.00 + 0.05^{‡})}{I_{LS} + I_{WallA}} = \frac{1.36 \times 25.49}{1.36 + 1.41} = 12.56 \text{ m from wall A}$$

or $= 12.56 + 2.80 - 0.05 = 15.31$ from east end of building.

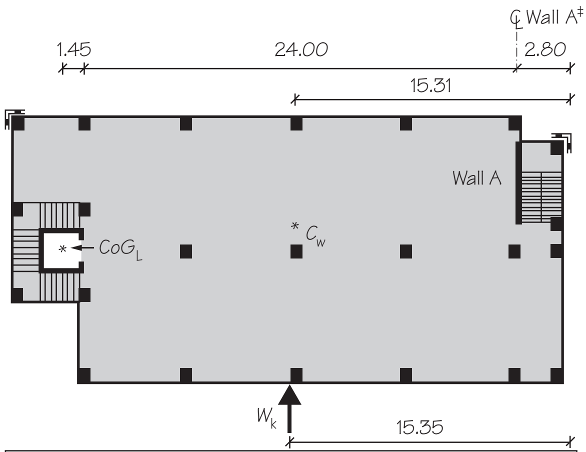


Figure 6.5 Shear centre, C_w and centre of action, W_k

Centre of action (30.7/2 = 15.35 m from end of building) and shear centre (almost) coincide. ∴ there is no torsion to resolve in the stability system for wind in a N–S direction.#

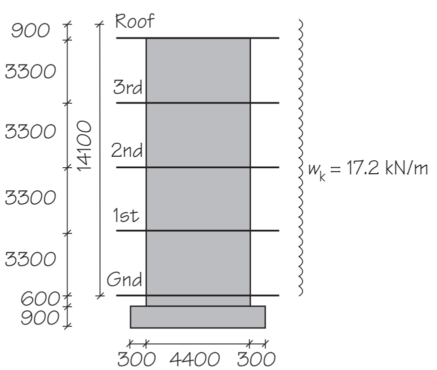


Figure 6.6 Wall A – wind loads N–S

∴ Wall A takes 51% of wind load, so characteristic wind load on wall A,

$w_{k,\text{ wall A}} = 51\% \times w_k \times L_x = 51\% \times 1.1 \times 30.7 = 17.2$ kN/m

‡ Assuming centreline of wall A is 50 mm to right hand side of grid.

Had there been significant torsion this would have been resolved into +/– forces in a couple based on the shear walls.

∴ at just above ground floor, characteristic in-plane moment in wall A, M_k, due in this case to wind

= 17.2 × 14.1^2/2 = 1709.8 kNm

Resolving into couple using 1 m either end of wall‡, characteristic wind load in each end, W_k

= 1709.8/3.4 = ±502.9 kN

6.1.5 Effects of global imperfections in plane of wall A

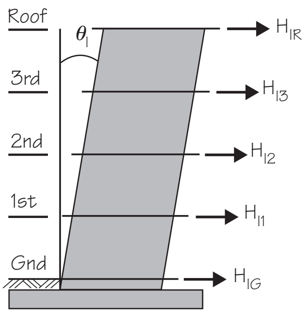


Figure 6.7 Global imperfections

Global imperfections can be represented by forces H_i at floor level where

$H_i = \theta_i(N_b - N_a)$ — Exp. (5.4)

where

$\theta_i = (1/200)\ \alpha_h \alpha_m$ — Cl. 5.2(1), 5.2(5), 5.2(8) & NA

where

$\alpha_h = 0.67 \le 2/l^{0.5} \le 1.0$

$= 0.67 \le 2/14.7^{0.5} \le 1.0$

$= 0.67 \le 0.52 \le 1.0$

$= 0.67$

$\alpha_m = [0.5(1 + 1/m)]^{0.5}$

where

m = no. of members contributing to the total effect

= 25 vertical elements on 4 floors

= 100

‡ For medium-rise shear walls there are a number of methods of design. Cl. 9.6.1 suggests strut-and-tie (see Volume 2 of these worked examples[30]). Another method[26] is to determine elastic tensile and compression stresses from N_{Ed}/bL +/– $6M_{Ed}/bL^2$ and determine reinforcement requirements based on those maxima. The method used here assumes a couple, consisting of 1.0 m of wall either end of the wall. The reinforcement in tension is assumed to act at the centre of one end and the concrete in compression (with a rectangular stress distribution) acts at the centre of the other end. The forces generated by the couple add or subtract from the axial load in the 1 m ends of the walls. The method is useful for typical straight shear walls of say 2.5 to 5.0 m in length. — Vol. 2

$\therefore\ \alpha_m = 0.71$

$\therefore\ \theta_i = 0.67 \times 0.71/200$

$= 0.0024$

N_b, N_a = axial forces in members below and above

$(N_b - N_a)$ = axial load from each level

At roof level

Area $= 30.4 \times 14.5 - 1.3 \times 2.5 - 3.6 \times 4.8 = 420.3\ m^3$

Perimeter $= 2 \times (30.4 + 14.5) = 89.8$ m

$(N_a - N_b)$ = axial load from roof level

$= 420.3 \times (7.05 + 0.6) + 89.8 \times 0.9 \times 4.0 = 3286.4 + 252.2$ kN

At 3rd floor

$(N_a - N_b) = 420.3 \times (5.78 + 2.5) + 89.8 \times 3.3 \times 4.0 = 3615.7 + 1050.8$ kN

At 2nd floor

$(N_a - N_b) = 3615.7 + 1050.8$ kN

At 1st floor

$(N_a - N_b) = 3615.7 + 1050.8$ kN

$H_{iR} = 0.0024 \times (3286.4 + 252.2) = 7.9 + 0.6 = 8.5$ kN

$H_{i3} = H_{i2} = H_{i1} = 0.0024 \times (3615.7 + 1050.8) = 8.7 + 2.5 = 11.2$ kN

Characteristic design moment at ground floor,

$M_k = 8.5 \times 13.2 + 11.2 \times (9.90 + 6.60 + 3.30)$

$= 112.2 + 221.8 = 334.0$ kNm

As before, wall A resists 51% of this moment. Resolving into couple using 1 m either end of wall,

$\therefore\ G_{kH}$[§] $= 0.51 \times 334.0/3.4 = \pm 50.1$ kN

i.e. $G_{kH} = \pm 50.1$ kN/m

6.1.6 Check for global second order effects

To check whether the building might act as a sway frame check — Cl. 5.8.3.3(1)

$$F_{V,Ed} \leq k_1 \frac{n_s}{n_s + 1.6} = \frac{\Sigma E_{cd} I_c}{L^2}$$ — Exp. (5.18)

where

$F_{V,Ed}$ = Total vertical load (on braced and bracing members)

where

Floor area $= (30.7 - 2 \times 0.15) \times 14.4 - (2 \times 0.15) - 3.6 \times 4.8 - 1.3 \times 2.5$

$= 428.6 - 20.5 = 408.1$

§ As H_i derives mainly from permanent actions its resulting effects are considered as being a permanent action too.

Loads	G_k	Q_k
from roof: 408 (7.05 + 0.6) =	2876	245
3–1 floors: 3 × 408 (5.78 + 2.5) =	7075	3060
Allow cavity wall at 1st floor and above		
(3 × 3.30 + 0.9) × 2 × (30.4 + 14.1) × 4.0 =	3845	
	13705	3305
Imposed load reduction 20% (see 6.2.3)		661
	13705	2644

$\therefore F_{V,Ed}$ ≈ 13705 × 1.35 + 1.5 × 2644
= 22468 kN

k_l = 0.31 — Cl. 5.8.2(6) & NA

n_s = number of storeys
= 4 (including roof)

E_{cd} = E_{cm}/γ_{CE} = 33/1.2 = 27.5 GPa — Table 3.1, 5.8.6(3) & NA

I_c = Inertia of bracing members
in N–S direction
I_c = 1.36 + 1.41 = 2.77 m^4 (See Section 6.1.4)
in E–W direction
I_{LS}, with reference to Figure 6.4

h × d	Area, A	x	Ax	Ax^2	I
2.4 × 2.4 =	5.76	1.2	6.912	8.294	2.765
–2.0 × 2.0 =	–4.00	1.2	–4.800	–5.760	–1.333
–1.6 × 0.2 =	–0.32	2.3	–0.732	–1.683	–0.001
	1.44		1.38	0.851	1.431

as before (6.1.4), $\bar{x}$ = 1.38/1.44
= 0.956 m

$I_{LS} = I_{NU} = Ax^2 + I - A\bar{x}^2$
= 0.851 + 1.431 – 1.44 × 0.956
= 0.965 m^4

L = total height of building above level of moment restraint
= 14.7 (see Figure 6.6)

Check

$$k_l\left(\frac{n_s}{n_s+1.6}\right)\times\left(\frac{\Sigma E_{cd}I_c}{L^2}\right)$$ on weak E–W axis:

= 0.31 × [4/(4 + 1.6)] × 27500 × 10^3 × (0.965/14.7^2)
= 27200 kN
i.e. > $F_{V,Ed}$ $\therefore$ no need to design for 2nd order effects.

6.1.7 Design moments – perpendicular to plane of wall

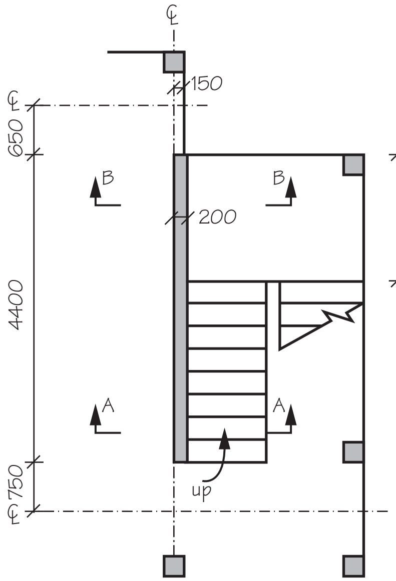


Figure 6.8 Plan of wall A and location of sections A–A and B–B

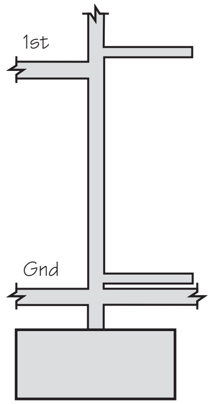


Figure 6.9 Section A–A

Section A–A @ 1st floor

The slab frames into the wall. For the purposes of assessing fixed end moments, the width of slab contributing to the moments in the wall is assumed to be the length of the wall plus distances half way to adjacent supports either end. Therefore, consider the fixed end moment for 1.50/2 + 4.40 + 1.30/2 = 5.8 m width of adjoining slab framing into the 4.4 m long shear wall (see Figure 6.8).

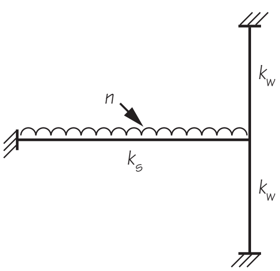


Figure 6.10 Subframe section A–A @ 1st floor

FEM‡: assuming imposed load is a leading variable action:

ECO: Exp. (6.10) & NA

$= nl^2/8$

$= 5.8\ (1.35 \times 5.78 + 1.5 \times 2.5) \times 6.0^2/8$

$= 5.8 \times 11.6 \times 6^2/8 = 302.8$ kNm

‡ FEM: fixed end moment

k_w = EI/l = $E \times 4400 \times 200^3/(12 \times 3300)$
= $E \times 8.88 \times 10^5$
k_s = $EI/2l$= $E \times 5800 \times 200^3/(2 \times 12 \times 6000)$
= $E \times 3.22 \times 10^5$
M = $302.8 \times 8.88/(2 \times 8.8 + 3.22)$
= 302.8×0.42 = 121.2 kNm
i.e. 121.2/4.40 = 27.5 kNm/m @ ULS

Similarly, assuming imposed load is an accompanying action:
FEM = $5.8\ (1.35 \times 5.78 + 0.7 \times 1.5 \times 2.5) \times 6^2/8$
= $5.8 \times 10.4 \times 6^2/8$ = 271.4 kNm
$\therefore M$ = $271.4 \times 0.42/4.40$ = 25.9 kNm/m @ ULS

Section A–A @ ground floor

By inspection not critical – nominal moment.

Section B–B @ 1st

Consider the landing influences half of wall (2.2 m long) and that this section of wall is subject to supporting half the slab considered before at 1st floor level at Section A–A.

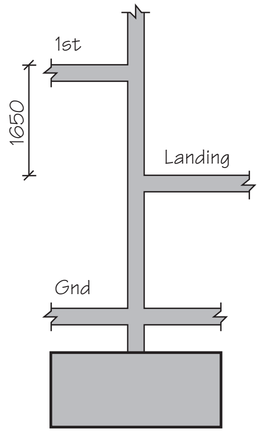


Figure 6.11 Section B–B

FEM = 302.8/2 = 151.4 kNm
k_w = I/l
= $2200 \times 200^3/(12 \times 1650) = 8.88 \times 10^5$
k_s = $3.22 \times 10^5/2 = 1.61 \times 10^5$
M = $151.4 \times 8.88/(2 \times 8.88 + 1.61)$
= 151.4×0.46
= 69.6 kNm
i.e. 63.8/2.2 = 31.6 kNm/m @ ULS

Similarly, assuming imposed load is an accompanying action:

FEM = 5.8 (1.35 × 5.78 + 0.7 × 1.5 × 2.5) × $6^2/8$

= 5.8 × 10.4 × $6^2/8$ = 271.4 kNm

∴M = 271.4 × 0.46/(2 × 2.2) = 28.4 kNm/m @ ULS

Section B–B @ landing level and ground floor

By inspection not critical

6.1.8 Consider slenderness of wall at ground floor

To derive maximum slenderness (at south end of wall), ignore effect of landing.

Effective length, l_0 = 0.75 × (3300 – 200) = 2325 — Table C16

$\lambda = 3.46 \times l_0/h$ = 3.46 × 2325/200 = 40.2 — **Cl. 5.8.3.2(1)**

Limiting slenderness, $\lambda_{lim} = 20\ ABC/n^{0.5}$ — **Cl. 5.8.3.1(1), Exp. (5.13N)**

where

A = 0.7

B = 1.1

C = 1.7 – r_m

where

$r_m = M_{01}/M_{02}$

= say = –0.25

C = 1.95

$n = N_{Ed}/A_c f_d$

where

N_{Ed} = 214.6 × 1.25 + 31.2 × 1.5 × 0.7 + 502.9 × 1.5 + 98.2 × 1.5 × 0.7‡

= 268.3 + 32.8 + 754.4 + 103.1

= 1158.6 kN

$A_c f_d$ = 200 × 1000 × 0.85 × 30/1.5 = 3400 kN

∴ n = 0.34

∴ λ_{lim} = 20 × 0.7 × 1.1 × 1.95/$0.34^{0.5}$ = 51.5

∴ As $\lambda < \lambda_{lim}$ wall is not slender and ∴ no secondary moments

6.1.9 Summary: design forces on wall, ground–1st floor

At ground to 1st consider maxima.

Vertical loads G_k = 214.6 kN/m

Q_k = 35.7 kN/m

Vertical load due to in-plane bending and wind W_k = ±502.9 kN/m

Vertical load due to in-plane bending and imperfections

G_{kH} = ±50.1 kN/m

Maximum moment out of plane, floor imposed load as leading action

M = 31.6 kN/m @ ULS

Maximum moment out of plane, floor imposed load as accompanying action

M = 28.4 kN/m @ ULS

‡ Assuming wind load is lead variable action.

6.1.10 Combinations of actions at ground–1st floor

a) At ULS, for maximum axial load, W_k is leading variable action

$N_{Ed} = 1.35G_k + 1.5Q_{k1} + 1.5\psi_0 Q_{ki}$
$= 1.35\ (214.6 + 50.1) + 1.5 \times 502.9 + 1.5 \times 0.7 \times 35.7$
$= 357.3 + 754.4 + 37.5$
$= 1149.2$ kN/m

$M_{Ed} = M + e_i N_{Ed} \geq e_0 N_{Ed}$ — Cl. 5.8.8.2(1), 6.1.4

where

M = moment from 1st order analysis
= 28.4 kNm/m

$e_i = l_0/400 = 2325/400 = 5.8$ mm — Cl. 5.2(7), 5.2(9)

$e_0 = h/30 \geq 20$ mm $= 20$ mm — Cl. 6.1.4

$M_{Ed} = 28.4 + 0.0058 \times 1149.2.1 \geq 0.020 \times 1149.2$
$= 28.4 + 6.7 \geq 23.0 = 35.1$ kNm/m

b) At ULS, for minimum axial load, W_k is leading variable action

$N_{Ed} = 1.0 \times 214.6 - 1.35 \times 50.1 - 1.5 \times 502.9 + 0 \times 35.7$
$= -607.4$ kN/m (tension)

$M_{Ed} = 28.4^{\ddagger} + 0.0058 \times 607.4 \geq 0.020 \times 602.4$
$= 28.4 + 3.5 \geq 23.0$
$= 31.9$ kNm/m

c) At ULS, for maximum out of plane bending assuming Q_k is leading variable action

$N_{Ed} = 1.35\ (214.6 + 50.1) + 1.5 \times 35.7 + 1.5 \times 0.5 \times 502.9$
$= 357.3 + 53.6 + 377.2$
$= 788.1$ kN/m

$M_{Ed} = 31.6 + 0.0058 \times 788.1 \geq 0.020 \times 788.1$
$= 31.6 + 4.6 \geq 15.8$
$= 36.2$ kNm/m

or

$N_{Ed} = 1.0 \times 214.6 - 1.35 \times 50.1 - 0 \times 31.2 - 1.5 \times 0.5 \times 502.9$
$= 214.6 - 67.6 - 0 - 377.2$
$= -230.2$ kN/m (tension)

$M_{Ed} = 31.6 + 0.0058 \times 230.2$
$= 33.0$ kNm/m

d) Design load cases

Consolidate c) into a) and b) to consider two load cases:

$N_{Ed} = 1149.4$ kN/m,
$M_{Ed} = 36.2$ kN/m (out of plane)

and

$N_{Ed} = -607.4$ kN/m,
$M_{Ed} = 36.2$ kN/m (out of plane)

‡ Strictly incompatible with $Q_k = 0$. However, allow $Q_k = 0$.

6.1.11 Design: cover above ground

$c_{nom} = c_{min} + \Delta c_{dev}$

where

$c_{min} = \max[c_{min,b};\ c_{min,dur}]$ — Exp. (4.1)

where

$c_{min,b}$ = diameter of bar = 20 mm vertical or 10 mm lacers

$c_{min,dur}$ = for XC1 = 15 mm

Δc_{dev} = 10 mm

$\therefore c_{nom}$ = 15 + 10 = <u>25 mm to lacers</u>

<u>(35 mm to vertical bars)</u>

6.1.12 Fire resistance

Assuming 1-hour fire resistance required for, as a worst case, $\mu_{fi} = 0.7$ and fire on both sides.

Min. thickness = 140 mm, min. axis distance = 10 mm <u>i.e. not critical</u> — EC2-1-2: Table 5.4

6.1.13 Design using charts

For compressive load:

$d_2/h = (25 + 10 + 16/2)/200 = 0.215$

$\therefore$ interpolate between charts C5d) and C5e) for — Figs C5d), C5e)

$N_{Ed}/bhf_{ck} = 1149.4 \times 10^3/(200 \times 1000 \times 30) = 0.192$

$M_{Ed}/bh^2f_{ck} = 36.2 \times 10^6/(200^2 \times 1000 \times 30) = 0.030$

Gives:

$A_s f_{yk}/bhf_{ck} = 0$ $\therefore$ minimum area of reinforcement required

$= 0.002\ A_c$ — Cl. 9.6.2 & NA

$= 0.002 \times 200 \times 1000$

$= 400\ \text{mm}^2/\text{m}$

$= 200\ \text{mm}^2/\text{m}$ each face

max. 400 mm cc, min. 12 mm diameter — Cl. 9.6.2(3); SMDSC

<u>Try T12 @ 400</u>

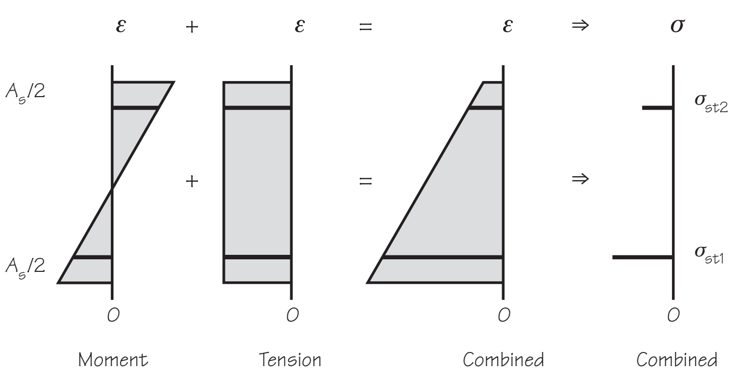


Figure 6.12 Stresses and strains in wall subject to tension and out of plane moment

For tensile load and moment:

Working from first principles, referring to Figure 6.12 and ignoring contribution from concrete in tension,

$N_{Ed} = (\sigma_{st1} + \sigma_{st2}) \times A_s/2$

and $M_{Ed} = (\sigma_{st1} - \sigma_{st2}) \times A_s/2 \times (d - d_2)$

so $\sigma_{st1} + \sigma_{st2} = 2N_{Ed}/A_s$

and $\sigma_{st1} - \sigma_{st2} = 2M_{Ed}/[(d - d_2)A_s]$

$\therefore\ 2\sigma_{st1} = 2N_{Ed}/A_s + 2M_{Ed}/[(d - d_2)A_s]$

$\therefore\ A_s = (N_{Ed}/\sigma_{st1}) + M_{Ed}/(d - d_2)\sigma_{st1}$

$\sigma_{st1} = f_{yk}/\gamma_S = 500/1.15 = 434.8$

$\therefore\ A_s = 607.4 \times 10^3/434.8 + 36.2 \times 10^6/[(157 - 43) \times 434.8]$

$= 1397 + 730$

$= 2127\ mm^2$

$\sigma_{st2} = 2N_{Ed}/A_s - \sigma_{st1} = 571.7 - 434.8 = 136\ MPa$

By inspection all concrete is in tension zone and may be ignored.

Use 6 no. H16 @ 200 cc both sides for at least 1 m each end of wall (2412 mm^2).

6.1.14 Horizontal reinforcement

$A_{s,\ hmin} = 0.001A_s$ or 25% $A_{s\ vert}$ — Cl. 9.6.3(1) & NA

$= 200\ mm^2$ or $0.25 \times 2036 = 509\ mm^2/m$

$\therefore$ requires 254 mm^2/m each side

Spacing ≤ 400 mm — Cl. 9.6.3(2)

Links not required. — Cl. 9.6.4(1)

Use H10 @ 300 (262 mm^2/m) both sides.

6.1.15 Check for tension at top of foundation

Permanent and variable: — Section 6.1.2

$G_k = 1021.0/4.4 = 232.0$ kN/m

$Q_k = 230.9/4.4 = 52.5$ kN/m

Wind: — Section 6.1.4

$M_k = 17.2 \times 14.1 \times [14.1/2 + 0.6] = 1855.3$ kN/m

Resolved into couple 1 m either end of wall

$W_{kw} = 1855.3/3.4 = \pm 545.7$ kN/m

Global imperfections: — Section 6.1.5

$M_k = 8.5 \times 13.8 + 11.2 \times (10.5 + 7.2 + 3.9 + 0.6)$

$= 365.9$ kNm

$G_{kH} = 365.9 \times 0.51/3.4 = 54.9$ kN/m

At ULS for maximum axial tension W_k is lead imposed load:

N_{Ed} = $1.0 \times 232.0 - 1.35 \times 54.9 - 1.5 \times 545.7 + 0 \times 52.5$
= -660.7 kN/m

M_{Ed} = nominal = $e2N_{Ed} = 0.02 \times 660.7$ — Cl. 6.1.4
= 13.2 kNm/m

As before

$$A_s = \frac{N_{Ed}}{f_{yk}/\gamma_M} + \frac{M_{Ed}}{(d - d_2)f_{yk}/\gamma_M}$$

= $660.7 \times 10^3/434.8 + 13.2 \times 10^6/[(157 - 43) \times 434.8]$
= 1520 + 266
= 1786 mm^2 i.e. not critical

∴ Use 6 no. H16 @ 200 cc b.s. for at least 1 m either end of wall (2412 mm^2).

6.1.16 Check for axial compression at top of foundation

At ULS for maximum axial compression W_k is lead imposed load:

N_{Ed} = $1.35 \times 232.0 + 1.35 \times 54.9 + 1.5 \times 545.7 + 0.7 \times 1.5 \times 52.5$
= 1261.0 kN/m

M_{Ed} = nominal = $e2N_{Ed} = 0.02 \times 1261.0$ — Cl. 6.1.4
= 25.2 kNm/m

By inspection not critical (minimum reinforcement required). — Section 6.1.13

∴ tension critical as above.

6.1.17 Design: cover below ground

$c_{nom} = c_{min} + \Delta c_{dev}$

where

c_{min} = max[$c_{min,b}$; $c_{min,dur}$] — Exp. (4.1)

where

$c_{min,b}$ = diameter of bar = 16 mm vertical or 10 mm lacers

$c_{min,dur}$ = for assumed Aggressive Chemical Environment for Concrete (ACEC) class AC1 ground conditions — BS 8500-1 Annex A[14], How to: Building structures[8]
= 25 mm

Δc_{dev} = 10 mm

c_{nom} = 25 + 10 = 35 mm to lacers
(45 mm to vertical bars)

In order to align vertical bars from foundation into Gnd–1st floor lift as starter bars, locally increase thickness of wall to say

250 mm thick with c_{nom} = 50 mm

6.1.18 Check stability

Assume base extends 0.3 m beyond either end of wall A, i.e. is 5.0 m long and is 1.2 m wide by 0.9 m deep.

Overturning moments

Wind (see Figure 6.6)

M_k = 17.2 x 14.1 x [14.1/2 + 1.5]
= 2073.5 kNm

Global imperfections (see Section 6.1.5)

M_k = 0.51 x [8.5 x 14.7 + 11.2 x (11.4 + 8.1 + 4.8 + 1.5)]
= 0.51 x [125.0 + 11.2 x 25.8]
= 0.51 x 414.0
= 211 kNm

Restoring moment

M_k = (1021.0 + 5.0 x 1.2 x 0.9 x 25 + 0 x 230.9) x (0.3 + 2.2)
= 2890 kNm

At ULS of EQU,

Overturning moment
= fn($\gamma_{Q,1}Q_{k1} + \gamma_{G,sup}G_k$)
= 1.5 x 2073.5 + 1.1 x 211.0 = 3342.4 kNm

Restoring moment
= fn($\gamma_{G,inf}G_k$)
= 0.9 x 2890 = 2601 kNm i.e. > 1818.4 kNm

∴ no good

Try 1.05 m outstand

Restoring moment

M_k = 2890 (1.05 + 2.2) / (0.3 + 2.2)
= 3757.0 kNm

At ULS, restoring moment = 0.9 x 357.0
= 3381.3 kNm

∴OK. Use 1.05 m outstand to wall.

ECO: Table A1.2(A) & NA

Fig. 6.6

Fig. 6.7

ECO: Table A1.2(A) & NA

ECO: Table A1.2(A) & NA

6.1.19 Design summary

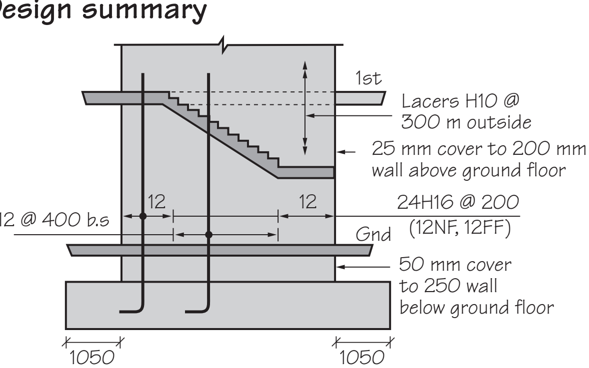


Figure 6.13 Wall design summary

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Appendix A: Derived formulae

A1 Flexure: beams and slabs

A1.1 Singly reinforced sections

The rectangular stress block shown below in Figure A1 may be used.

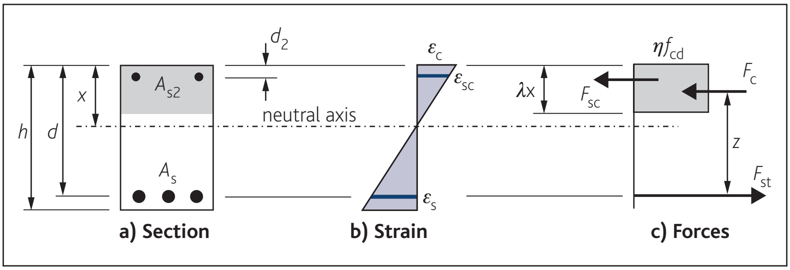

Figure A1
Strains and forces in a section

Fig. 3.5

For grades of concrete up to C50/60, $\varepsilon_{cu} = 0.0035$, $\eta = 1$ and $\lambda = 0.8$

3.1.6(1), 2.4.2.4(1) & NA

$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C = 0.85\, f_{ck}/1.5$

$f_{yd} = f_{yk}/\ \gamma_S = f_{yk}/1.15 = 0.87 f_{yk}$

2.4.2.4(1) & NA

For singly reinforced sections, the design equations can be derived as follows:

Lever arm, *z*

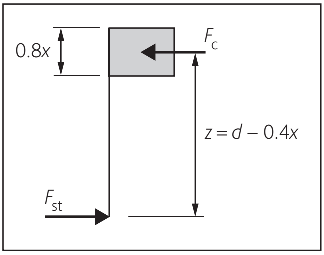

Figure A2
Beam lever arm

$F_c = (0.85\, f_{ck}/1.5)\, b\, (0.8x) = 0.453 f_{ck}\, bx$

$F_{st} = 0.87 A_s f_{yk}$

Consider moment‡, M, about the centre of the tension force:

$M = 0.453 f_{ck} bxz$

Now $z = d - 0.4x$

$\therefore \quad x = 2.5(d - z)$

$M \quad = 0.453 f_{ck}\, b 2.5(d - z)z$

$\quad = 1.1333\, (f_{ck}\, bzd - f_{ck}\, b\, z^2)$

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

$\quad = 1.1333\, (f_{ck}\, bzd - f_{ck}\, bz^2)/bd^2 f_{ck}$

$\quad = 1.1333(zd - z^2)/d^2$

$\therefore 0 \quad = 1.1333[(z/d)^2 - (z/d)] + K$

$\quad = (z/d)^2 - (z/d) + 0.88235K$

‡ In practice the design moment, M_{Ed} would be used.

Solving the quadratic equation:

$z/d = [1 + (1 - 3.529K)^{0.5}]/2$

$z = d[1 + [1 - 3.529K)^{0.5}]/2$

Table C5

It is considered good practice in the UK to limit z/d to a maximum of 0.95d. (This guards against relying on very thin sections of concrete which at the extreme top of a section may be of questionable strength.) Tables giving values of z/d and x/d for values of K may be used.

Area of reinforcement, A_s

Taking moments about the centre of the compression force:

$M = 0.87 A_s f_{yk} z$

$A_s = M/(0.87 f_{yk} z)$

Limiting value of relative flexural compressive stress, K'

Assuming no redistribution takes place, a limiting value (on the strength of concrete in compression) for K can be calculated (denoted K') as follows.

Table 3.1

ε_{cu3} = concrete strain = 0.0035
ε_s = reinforcement strain
= $500/(1.15 \times 200 \times 10^3) = 0.0022$

From strain diagram, Figure A1

$x = 0.0035d/(0.0035 + 0.0022)$
$= 0.6d$

From equations above:

$M = 0.453 f_{ck}\, bxz$
$M' = 0.453 f_{ck}\, b\, 0.6d\, (d - 0.4 \times 0.6\, d)$
$= 0.207 f_{ck}\, b\, d^2$

$\therefore K' = 0.207$

It is often considered good practice to limit the depth of the neutral axis to avoid 'over-reinforcement' (i.e. to ensure that the reinforcement is yielding at failure, thus avoiding brittle failure of the concrete). Often x/d is limited to 0.45. This is referred to as the balanced section because at the ultimate limit state the concrete and steel reach their ultimate strains at the same time[31]. This is not a Eurocode 2 requirement and is not accepted by all engineers.

Nonetheless for $x = 0.45d$

From equations above:

$M = 0.453 f_{ck}\, bxz$
$M' = 0.453 f_{ck}\, b\, 0.45d\, (d - 0.4 \times 0.45d)$
$= 0.167 f_{ck}\, bd^2$
$\therefore K' = 0.167$

Cl. 5.5(4)

x/d is also restricted by the amount of redistribution carried out. For $f_{ck} \leq 50$ MPa

$\delta \geq 0.4 + (0.6 + 0.0014 \varepsilon_{cu}) x_u / d$

where

d = redistributed moment/elastic bending moment before redistribution
x_u = depth of the neutral axis at ULS after redistribution
ε_{cu} = compressive strain in the concrete at ULS

This gives the values in Table A1.

Table A1
Limits on *K'* with respect to redistribution ratio, δ

δ	1	0.95	0.9	0.85	0.8	0.75	0.7
% redistribution	0	5	10	15	20	25	30
K'	0.208	0.195	0.182	0.168	0.153	0.137	0.120

If $K > K'$ the section should be resized or compression reinforcement is required. In line with consideration of good practice outlined above, **this publication adopts a maximum value of $K' = 0.167$**.

A1.2 Compression reinforcement, A_{s2}

The majority of beams used in practice are singly reinforced, and these beams can be designed using the formula derived above. In some cases, compression reinforcement is added in order to:

- Increase section strength where section dimensions are restricted, i.e. where $K > K'$
- To reduce long term deflection
- To decrease curvature/deformation at ultimate limit state

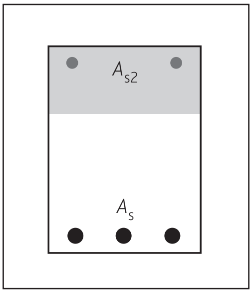

Figure A3
Beam with compression reinforcement

With reference to Figure A1, there is now an extra force
$F_{sc} = 0.87A_{s2}f_{yk}$

The area of tension reinforcement can now be considered in two parts, the first part to balance the compressive force in the concrete, the second part to balance the force in the compression steel. The area of tension reinforcement required is therefore:

$A_s = K' f_{cu}\, bd^2/(0.87f_{yk}\, z) + A_{s2}$

where

z is calculated using K' instead of K

A_{s2} can be calculated by taking moments about the centre of the tension force:

$M = M' + 0.87f_{yk}\, A_{s2}\, (d - d_2)$

$M = K' f_{cu}\, bd^2 + 0.87f_{yk}\, A_{s2}\, (d - d_2)$

Rearranging:
$A_{s2} = (K - K')f_{ck}\, bd^2/[0.87\, f_{yk}\, (d - d_2)]$

A2 Shear

A2.1 Shear resistance (without shear reinforcement), $V_{Rd,c}$

$V_{Rd,c} = [C_{Rd,c}\, k(100\rho 1\, f_{ck})^{1/3} + k_1\, \sigma_{cp}]\, b_w d \geq (v_{min} + k_1\, \sigma_{cp})\, b_w d$ Exp. (6.2)

where

$C_{Rd,c} = 0.18/\gamma_C = 0.18/1.5 = 0.12$ NA

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

$\rho_1 = A_s/(b_w d) \le 0.02$
$k_1 = 0.15$
$\sigma_{cp} = 0$ for non-prestressed concrete
$v_{min} = 0.035k^{1.5} f_{ck}^{0.5}$
$\therefore V_{Rd,c} = 0.12k(100\, \rho_1 f_{ck})^{1/3}\, b_w d \ge 0.035k^{1.5} f_{ck}^{0.5}\, b_w d$

A2.2 Shear capacity

The capacity of a concrete section with vertical shear reinforcement to act as a strut, $V_{Rd,max}$:

Exp. (6.9)

$V_{Rd,max} = \alpha_{cw} b_w Z \nu_1 f_{cd}/(\cot\theta + \tan\theta)$

where

Cl. 6.2.3(3) Note 1, Exp. (6.6N) & NA

$\alpha_{cw} = 1.0$
$\nu_1 = \nu = 0.6\,[1 - f_{ck}/250]$
$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C = 1.00 \times f_{ck}/1.5$
$\therefore V_{Rd,max} = 0.40\, b_w z\, f_{ck}\, [1 - f_{ck}/250]/(\cot\theta + \tan\theta)$

Rearranging this equation gives:

$\theta = 0.5 \sin^{-1}\,[v_{Edz}/(0.20 f_{ck}\,[1 - f_{ck}/250])] \ge \cot^{-1} 2.5$

where

$v_{Edz} = V_{Ed}/bz = V_{Ed}/(b0.9d)$

In most cases, where $\cot\theta = 2.5$, $\theta = 21.8°$

$V_{Rd,max,\cot\theta = 2.5} = 0.138 b_w z\, f_{ck}\, [1 - f_{ck}/250]$

or

$v_{Rd,max,\cot\theta = 2.5} = 0.138 f_{ck}\, [1 - f_{ck}/250]$

where

$v_{Rd,max,\cot\theta = 2.5} = V_{Rd,max,\cot\theta = 2.5}/(bz)$
$= V_{Rd,max,\cot\theta = 2.5}/(0.9bd)$

Where $\cot\theta > 2.5$, the angle of the strut and $v_{Rd,max}$ should be calculated, or $v_{Rd,max}$ may be looked up in tables or charts (e.g. Table C7 or Figure C1).

A2.3 Shear reinforcement

Exp. (6.13)

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

where

A_{sw} = cross-sectional area of the shear reinforcement
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, $\cot\alpha = 0$ and $\sin\alpha = 1.0$

Rearranging for vertical links:

$A_{sw}/s \ge V_{Ed}/z f_{ywd} \cot\theta$

or

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

Minimum area of shear reinforcement

Exp. (9.5N) & NA

$A_{sw,min}/(s b_w \sin\alpha) \ge 0.08 f_{ck}^{0.5}/f_{yk}$

where

s = longitudinal spacing of the shear reinforcement
b_w = breadth of the web

α = angle of the shear reinforcement to the longitudinal axis of the member. For vertical links sin α = 1.0.

Rearranging for vertical links:

$A_{sw,min}/s \geq 0.08 b_w \sin \alpha\, f_{ck}^{0.5}/f_{yk}$

A3 Columns

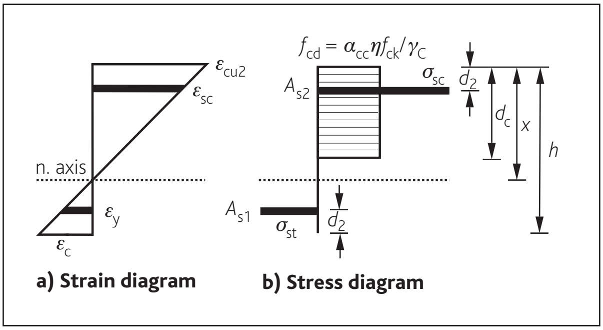

Figure A4
Section in axial compression and bending

Fig. 6.1

For axial load

$N_{Ed} = f_{cd} b d_c + A_{s2}\sigma_{sc} - A_{s1}\sigma_{st}$

But as $A_{s2} = A_{s1} = A_{sN}/2$

$N_{Ed} = f_{cd} b d_c + A_{sN}(\sigma_{sc} - \sigma_{st})/2$

$N_{Ed} - f_{cd} b d_c = A_{sN}(\sigma_{sc} - \sigma_{st})/2$

$(N_{Ed} - f_{cd} b d_c)/(\sigma_{sc} - \sigma_{st}) = A_{sN}/2$

$A_{sN}/2 = (N_{Ed} - f_{cd} b d_c)/(\sigma_{sc} - \sigma_{st})$

$\therefore A_{sN}/2 = (N_{Ed} - \alpha_{cc}\eta f_{ck} b d_c/\gamma_C)/(\sigma_{sc} - \sigma_{st})$

For moment about centre of column

$M_{Ed} = f_{cd} b d_c(h/2 - d_c/2) + A_{s2}\sigma_{sc}(h/2 - d_2) + A_{s1}\sigma_{st}(h/2 - d_2)$

But as $A_{s2} = A_{s1} = A_{sM}/2$

$M_{Ed} = f_{cd} b d_c(h/2 - d_c/2) + A_{sM}\,(\sigma_{sc} + \sigma_{st})(h/2 - d_2)/2$

$M_{Ed} - f_{cd} b d_c(h/2 - d_c/2) = A_{sM}\,(\sigma_{sc} + \sigma_{st})(h/2 - d_2)/2$

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

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

Solution

Iterate x such that $A_{sN} = A_{sM}$

Note

For sections wholly in compression, the strain is limited such that average strain $\leq \varepsilon_{cs} = 0.00175$ (assuming bilinear stress–strain relationship).

Cl. 6.1(6), Fig. 6.1, Table 3.1

Appendix B: Serviceability limit state

B1 Deflection

In many cases, particularly with slabs, deflection is critical to design.

Eurocode 2, Cl. 7.4 allows for deflection to be controlled by using span:depth ratio (*L*/*d*) checks in accordance with Cl. 7.4.2 or by calculation in accordance with Cl. 7.4.3. It is important to differentiate between the various methods used in checking deformation as they will each give different answers. Three popular methods are discussed below. Only that described in Section B1.1 below is suitable for hand calculation.

B1.1 TCC method[5,19]

The in-service stress of reinforcement, σ_s, is used to determine a factor, $310/\sigma_s$, which is used to modify the basic span:effective depth ratio as allowed in Cl. 7.4.2(2) of Eurocode 2[2] and moderated by the National Annex[2a]. This method, highlighted as factor F3 in *Concise Eurocode 2*[5], is intended to be used in hand calculations to derive (conservative) values of σ_s from available ULS moments. In accordance with Note 5 of Table NA.5 of the UK NA[2a], the ratio for $A_{s,prov}/A_{s,req}$ is restricted to 1.5: in effect this limits the factor $310/\sigma_s$ to 1.5.

where[‡]

$$\sigma_s = (f_{yk}/\gamma_S)\,(w_{qp}/w_{ult})\,(A_{s,req}/A_{s,prov})\,/\delta \le 310/1.5$$

where

f_{yk} = characteristic strength of reinforcement = 500 MPa
γ_S = partial factor for reinforcement = 1.15
w_{qp} = quasi-permanent load (UDL assumed)
w_{perm} = ultimate load (UDL assumed)
$A_{s,req}$ = area of reinforcement required
$A_{s,prov}$ = area of reinforcement provided
δ = redistribution ratio

B1.2 RC Spreadsheets method[28]

The RC spreadsheets TCCxx.xls [28] use the span:depth method of checking deformation but use an accurate method for determining σ_s (see B3.2 below), which again is used to determine the moderating factor = $310/\sigma_s$. Again, in accordance with Note 5 of Table NA.5 of the UK NA[2a], the ratio for $A_{s,prov}/A_{s,req}$ is restricted to 1.5: in effect this limits the factor $310/\sigma_s$ to 1.5.

Separate analyses using quasi-permanent loads need to be carried out. For each span, an SLS neutral axis depth is determined, then σ_c and σ_s are derived for the quasi-permanent load conditions. The factor σ_s is used in accordance with Eurocode 2[2] and the current National Annex[2a], to modify the basic span:effective depth ratio.

Whilst this method gives a more accurate and less conservative assessment of σ_s, it is only suitable for computer spreadsheet applications. See also Appendix B5.

In the analysis of slabs and beams, supports are usually assumed to be pinned. In reality supports have some continuity, especially at end supports. Usually, nominal top steel is assumed and provided in the top of spans and is used in the determination of section properties.

B1.3 Rigorous analysis

Rigorous analysis, such as that used in the series of RC Spreadsheets TCCxxR.xls may be used to assess deformation in accordance with Eurocode 2, Cl. 7.4.3.

[‡] See Appendix B1.5

In the spreadsheets, sections at 1/20th points along the length of a span are checked to determine whether the flexural tensile stress in the section is likely to exceed the tensile strength of the concrete during either construction or service life: separate analyses are undertaken using frequent loads, quasi-permanent and temporary loads. If the flexural tensile strength is exceeded under frequent loads, then the section is assumed to be cracked and remain cracked: cracked section properties are used to determine the radius of curvature for that 1/20th of span. If flexural tensile strength is not exceeded, un-cracked section properties are used.

Radii of curvature are calculated for each 1/20th span increment of the element using the relevant properties and moments derived from analysis of quasi-permanent actions. Deformation is calculated from the increments' curvatures via numerical integration over the length of each span.

The method is in accordance with The Concrete Society's publication TR58[32]. Again the method is suitable only for computer applications and not for hand calculation.

B1.4 Differing results

During 2008, it became increasingly apparent that there are inconsistencies between the results given by the rigorous calculation method and span:depth methods described in Eurocode 2. Using the rigorous method gives deflections that are greater than would be expected from the assumptions stated for L/d methods i.e. deflection limits of $L/250$ overall (see Cl. 7.4.1(4)) or $L/500$ after construction (see Cl. 7.4.1(5)). It is suspected that this disparity is the same as that experienced between span:depth and calculation methods in BS 8110: a disparity that was recognised as long ago as 1971[33]. The rigorous method described above relies on many assumptions and is largely uncalibrated against real structures. As noted in TR58, there is an urgent need for data from actual structures so that methods may be calibrated. It should be noted that the rigorous analysis method observations were made using frequent loads where, in accordance with Eurocode 2, quasi-permanent loads are called for.

End spans are usually critical. With respect to the rigorous analysis method, it has been suggested that for end-spans, the TCC and RC-spreadsheet methods result in deflections close to the limits stated in Eurocode 2, provided that a nominal end-support restraining moment is present where none is assumed in analysis. Caution is therefore necessary in true pinned end-support situations but where some continuity exists, this disparity may be addressed by ensuring that appropriate amounts of reinforcement, in accordance with the Code and National Annex, are provided at end supports.

The NDP for Cl. 9.2.1.2(1) in the UK NA[2a] to BS EN 1992–1–2 stipulates that 25% of end span moment should be used to determine end support reinforcement. This is usually accommodated by providing 25% of end span bottom steel as top steel at end supports. It is on this basis that the calculations in this publication are considered as being further substantiated.

B1.5 Note regarding factor $310/\sigma_s$ (factor F3)

At the time of publication (December 2009) the authors were aware of a probable change to UK NA[2a] Table NA.5 which, in effect, would mean that the factor $310/\sigma_s$ (F3) = $A_{s,prov}/A_{s,req}$ ≤ 1.5, thus disallowing the accurate method outlined in Sections 3.1, 3.2, 3.3, 3.4, 4.3 and Appendices B1.1, B1.2 and C7.

B2 Neutral axis at SLS

To find x, neutral axis, and services stresses σ_c and σ_s for a concrete section, at SLS, consider the cracked section in Figure B1

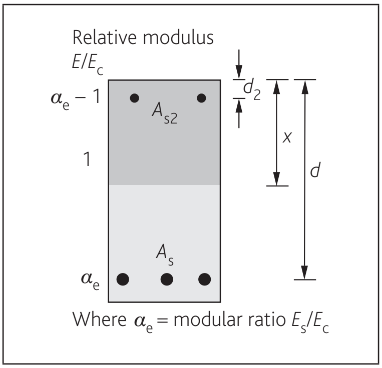

Figure B1
Cracked concrete section at SLS

From first principles, for a fully cracked transformed section,
Total area of section, $A = bx + A_s\alpha_e + A_{s2}(\alpha_e - 1)$

1st moment of area, $A_y = bx^2/2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)$

For a slab, b = 1000, therefore

$A = 1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)$

$A_y = 500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)$

Neutral axis depth, x

$x = A_y/A$

$= [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]/[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

Therefore

$x[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)] = [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]$

$0 = [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)] - x[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

$= 500x^2 - x[1000x] + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)] - x[A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

$= -500x^2 - x[A_s\alpha_e + A_{s2}(\alpha_e - 1)] + [A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]$

Solving the quadratic

$x = -b \pm b^2 - 4\alpha c)^{0.5}]/2a$

$$x = \frac{-[A_s\alpha_e + A_{s2}(\alpha_e - 1) \pm \{[A_s\alpha_e + A_{s2}(\alpha_e - 1)]^2 + 4 \times 500 \times [A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]\}^{0.5}]}{(2 \times 500)}$$

or transposing,

$$x = \frac{[-(\alpha_e - 1)A_{s2} - \alpha_e A_s + \{[(\alpha_e - 1)A_{s2} + \alpha_e A_s]^2 + 2000[(\alpha_e - 1)A_{s2}d_2 + \alpha_e A_s d]\}^{0.5}]}{1000}$$

or

$$x = \frac{[-(\alpha_e - 1)A_{s2} - \alpha_e A_s + \{[(\alpha_e - 1)A_{s2} + \alpha_e A_s]^2 + 2b[(\alpha_e - 1)A_{s2}d_2 + \alpha_e A_s d]\}^{0.5}]}{b}$$

This expression is used in the RC spreadsheets[28].

B3 SLS stresses in concrete, σ_c, and reinforcement, σ_s

B3.1 Singly reinforced section

Consider the singly reinforced section in Figure B2.

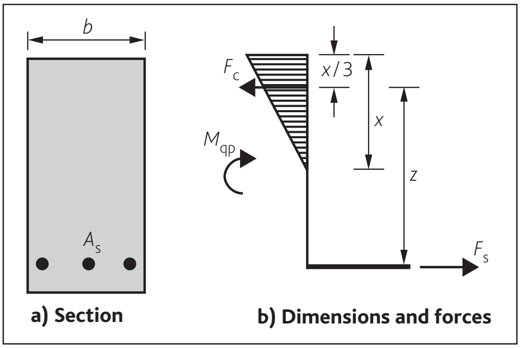

Figure B2
SLS stresses: singly reinforced section

Consider moments about F_c:

$M_{qp} = F_s z = F_s(d - x/3)$
$F_s = M_{qp}/(d - x/3)$
$\sigma_s = M_{qp}/[A_s(d - x/3)]$
$\sigma_s A_s = M_{qp}/(d - x/3) = xb\sigma_c/2$
$\sigma_c = 2\sigma_s A_s/xb$

B3.2 Doubly reinforced section

Consider the singly reinforced section in Figure B3.

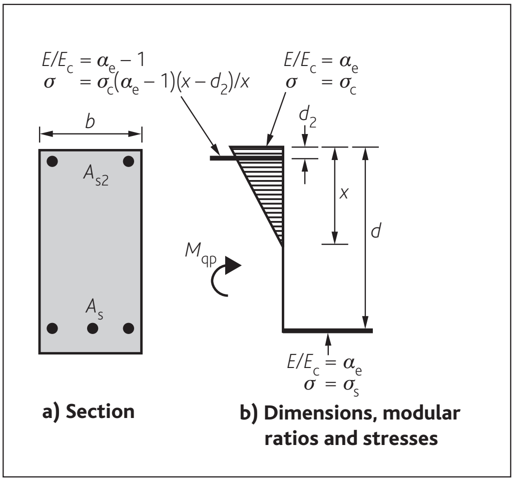

Figure B3
SLS stresses

Consider moment, M_{qp}, about bottom reinforcement, A_s[34].
$M_{qp} = A_{s2}(d - d_2)(\alpha_e - 1)\{(x - d_2)/x\}\ \sigma_c + \sigma_c\ b(x/2)(d - x/3)$

Therefore

$\sigma_c = M_{qp} / [A_{s2}(d - d_2)(\alpha_e - 1)\ )\{(x - d_2)/x\} + b(x/2)(d - x/3)]$

And from stress diagram

$\sigma_s = \sigma_c\ \alpha_e(d - x)/x$

Appendix C: Design aids

The following tables, text and figures have been derived from Eurocode 2 and are provided as design aids for designers in the UK. These design aids have been referenced in the text and generally have been taken from Section 15 of *Concise Eurocode 2*[5].

C1 Design values of actions

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 BS EN 1990[10]
  or the worse case of
- $1.35G_k + \psi_0 1.5Q_k$ Exp. (6.10a)
  and
- $1.25G_k + 1.5Q_k$ 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.

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 0.7 for storage.

C2 Values of actions

The values of actions (i.e. loads) are defined in Eurocode 1[11]. The parts of Eurocode 1 are given in Table C1. 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[6] makes clear, until the appropriate European standards become available, designers may consider using current practice or current British Standards in conjunction with Eurocode 2, provided they are compatible with Eurocode 2 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 C1
The parts of Eurocode 1[11]

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

C3 Analysis

Analysis is dealt with in Section 5 of *Concise Eurocode 2*. Where appropriate the coefficients given in Tables C2 and C3 can be used to determine design moments and shear for slabs and beams at ULS.

Table C2
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

Notes

1 Applicable to one-way spanning slabs where the area of each bay exceeds 30 m^2, $Q_k \leq 1.25G_k$ and $q_k \leq 5$ kN/m^2, substantially uniform loading (at least 3 spans, minimum span ≥ 0.85 maximum (design) span.

2 Design moment = coeff x n x span^2 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.

3 Basis: Yield line design (assumed 20% redistribution[7])

Table C3
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]

Notes

1 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).

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

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

4 Design moment at supports = coeff x n x span^2
or in spans = (coeff g_k x $\gamma_G g_k$ + coeff q_k x $\psi\gamma_Q q_k$) x span^2.

5 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, Expressions (6.10), (6.10a) or (6.10b). See Section C1.

6 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 Eurocode 2, Cl. 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 Eurocode 2, Cl. 9.3.1.2).

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

Cl. 9.2.1.2

Cl. 9.3.1.2

C4 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 in compression
  K' may be determined from Table C4 and is dependent on the redistribution ratio used.

Table C4
Values for *K'*

Redistribution ratio, δ	z/d for K'[a]	K'[a]	$1 - \delta$
1.00	0.76 (0.82)	0.208 (0.168)	0%
0.95	0.78 (0.82)	0.195 (0.168)	5%
0.90	0.80 (0.82)	0.182 (0.168)	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%

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

Key
a It is recommended that x/d is limited to 0.45[35]. As a consequence z/d is limited to a minimum of 0.820 and K' to a minimum of 0.168.

- If $K \leq K'$, section is **under-reinforced**.
  For rectangular sections:
  $A_{s1} = M_{Ed}/f_{yd}z$
  where
  A_{s1} = area of tensile reinforcement
  M_{Ed} = design moment
  $f_{yd} = f_{yk}/\gamma_S = 500/1.15 = 434.8$ MPa
  $z = d[0.5 + 0.5(1 - 3.53K)^{0.5}] \leq 0.95d$
  Values of z/d (and x/d) may be taken from Table C5

  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 C5
  h_f = thickness of flange

How to: Beams[8]

  For flanged beams where $x \geq 1.25h_f$, refer to *How to design concrete structures using Eurocode 2*[8].

- 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
  If $d_2/x > 0.375$ then the term A_{s2} should be replaced by the term
  $1.6(1 - d_2/x)\, A_{s2}$
  $M' = K'bd^2 f_{ck}$
  $f_{sc} = 700(x_u - d_2)/x_u \leq f_{yd}$
  where
  d_2 = effective depth to compression reinforcement

$x_u = (\delta - 0.4)d$

where

δ = redistribution ratio

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

Table C5
Values of *z/d* and *x/d* for singly reinforced rectangular sections

K	*z/d*	*x/d*	$(1 - \delta)_{max}$ [a]
0.04	0.950 [b]	0.125	30%
0.05	0.950 [b]	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 [c]	0.460 [c]	14%
0.18	0.802 [c]	0.495 [c]	11%
0.19	0.787 [c]	0.533 [c]	7%
0.20	0.771 [c]	0.572 [c]	3%
0.208	0.758 [c]	0.606 [c]	0%

Note
$f_{ck} \leq 50$ MPa

Key
a Maximum allowable redistribution
b Practical limit
c It is recommended that *x/d* is limited to 0.450[35]. As a consequence *z/d* is limited to a minimum of 0.820 and *K'* to 0.168.

C5 Design for beam shear

C5.1 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 C6.

Table C6
Shear resistance without shear reinforcement, $v_{Rd,c}$ (MPa)

$\rho_l = A_{sl}/b_w d$	Effective depth d (mm)										
	≤ 200	225	250	275	300	350	400	450	500	600	750
≤ 0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.69	0.68	0.65	0.64	0.62	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.74	0.73	0.71	0.69	0.67	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.79	0.78	0.75	0.73	0.71	0.70	0.67	0.65
1.75%	0.90	0.87	0.85	0.83	0.82	0.79	0.77	0.75	0.73	0.71	0.68
≥ 2.00%	0.94	0.91	0.89	0.87	0.85	0.82	0.80	0.78	0.77	0.74	0.71

Notes
1 Table derived from Eurocode 2 and UK National Annex.
2 Table created for f_{ck} = 30 MPa assuming vertical links.
3 For $\rho_l \geq 0.4\%$ and
f_{ck} = 25 MPa, apply factor of 0.94
f_{ck} = 35 MPa, apply factor of 1.05
f_{ck} = 40 MPa, apply factor of 1.10
f_{ck} = 45 MPa, apply factor of 1.14
f_{ck} = 50 MPa, apply factor of 1.19
Not applicable for f_{ck} > 50 MPa

C5.2 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 C7, 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 C7
Capacity of concrete struts expressed as a stress, $v_{Rd,max}$

f_{ck}	$v_{Rd,max}$ (MPa)							Strength reduction factor, ν
	$\cot\theta$	2.50	2.14	1.73	1.43	1.19	1.00	
	θ	2.18°	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
1 Table derived from Eurocode 2 and UK National Annex assuming vertical links, i.e. $\cot\alpha = 0$
2 $\nu = 0.6[1 - (f_{ck}/250)]$
3 $v_{Rd,max} = \nu f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$

C5.3 Shear reinforcement design

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

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 C1a) or C1b) as indicated by the blue arrows in Figure C1a). These figures may also be used to estimate the value of cot θ.

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 C8).

Table C8

Values of $A_{sw,min}/sb_w$ for beams for vertical links and f_{yk} = 500 MPa and compatible resistance, v_{Rd}

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)	0.72	0.80	0.88	0.95	1.01	1.07	1.13
v_{Rd} for $A_{sw,min}/sb_w$ (MPa)	0.78	0.87	0.95	1.03	1.10	1.17	1.23

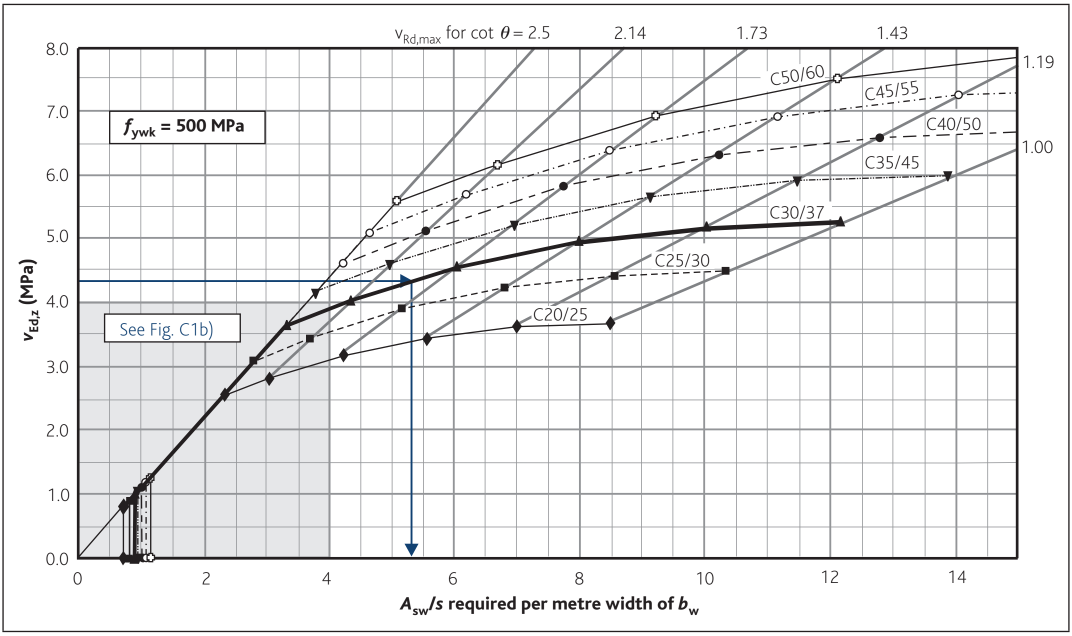

Figure C1a)

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

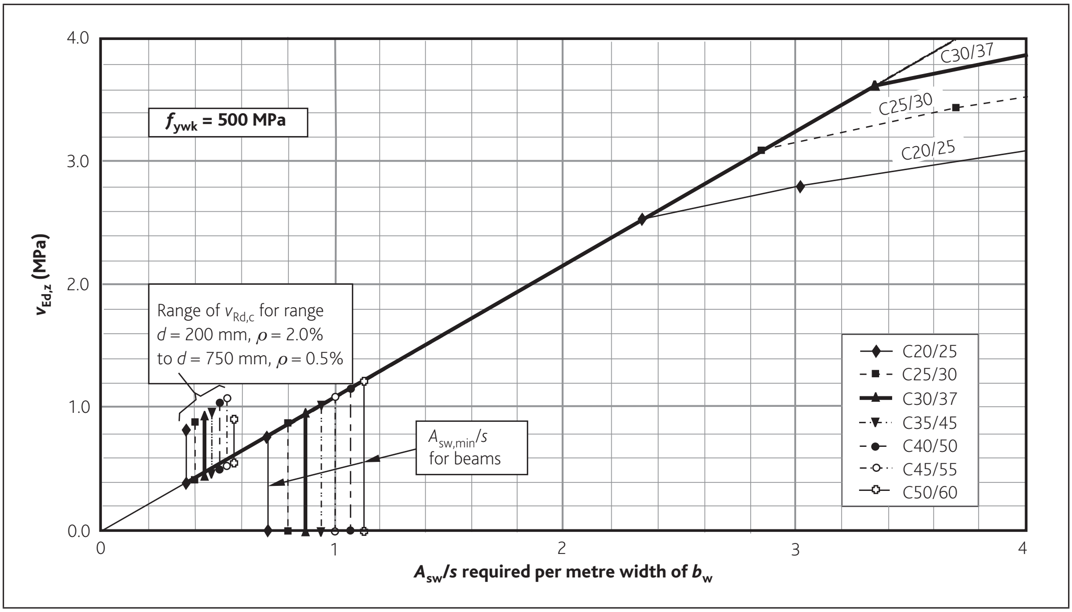

Figure C1b)
Diagram to determine A_{sw}/s required (for slabs and beams with low shear stress)

C6 Design for punching shear

Determine if punching shear reinforcement is required, initially at u_1, then if necessary at subsequent perimeters, u_i. If $v_{Ed} > v_{Rd,c}$ then punching shear reinforcement is required
where

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

where

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

$v_{Rd,c}$ = shear resistance without shear reinforcement (see Table C6)

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

A_{sw} = area of shear reinforcement in one perimeter around the column.
For $A_{sw,min}$ see *Concise Eurocode 2*, Section 10.4.2 and for layout see Section 12.4.3

s_r = radial spacing of perimeters of shear reinforcement

u_1 = basic control perimeter $2d$ from column face

$f_{ywd,ef}$ = effective design strength of reinforcement = $(250 + 0.25d) \leq f_{ywd}$. For Grade 500 shear reinforcement see Table C9

Concise: 10.4.2, 12.4.3

Table C9
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

At the column perimeter, check $v_{Ed} \leq V_{Rdmax}$ for cot θ = 1.0 given in Table C7.

C7 Check deflection

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 $\rho' = 0$ from Section 10.5.2 of *Concise Eurocode 2* or Table C10 or Figure C2

Concise: 10.5.2

K = factor to account for structural system. See Table C11

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 C12)
where
b_{eff} is defined in Section 5.2.2 of *Concise Eurocode 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.

Concise: 5.2.2

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 C13

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 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 C3 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 C14)

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

‡ See Appendix B1.5

Table 7.4N

Table C10
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%

Notes
1 Where $\rho = A_s/bd$.
2 For T-sections ρ is the area of reinforcement divided by the area of concrete above the centroid of the tension reinforcement.
3 The values for span-to-effective-depth have been based on Table 7.4N in Eurocode 2, using $K = 1$ (simply supported) and $\rho' = 0$ (no compression reinforcement required).
4 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.

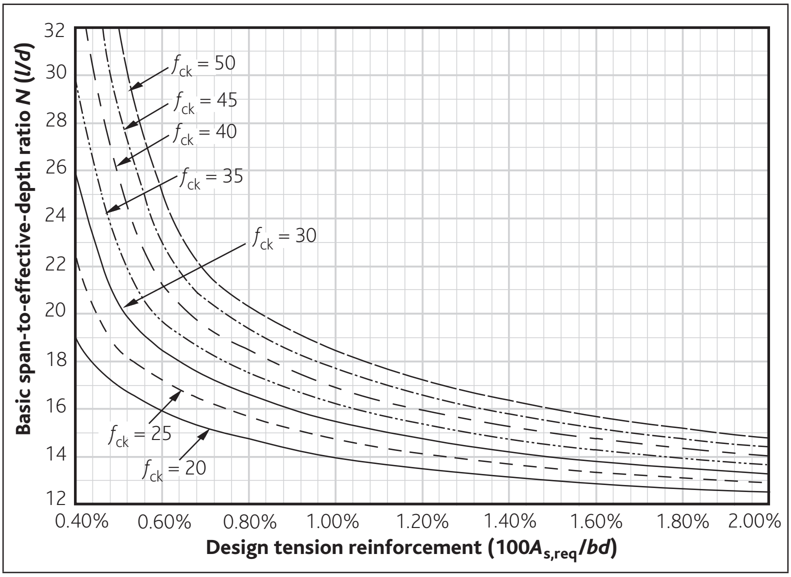

Figure C2
Basic span-to-effective depth ratios, N, for $K = 1$, $\rho' = 0$

Table C11
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 C12
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

Table C13
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

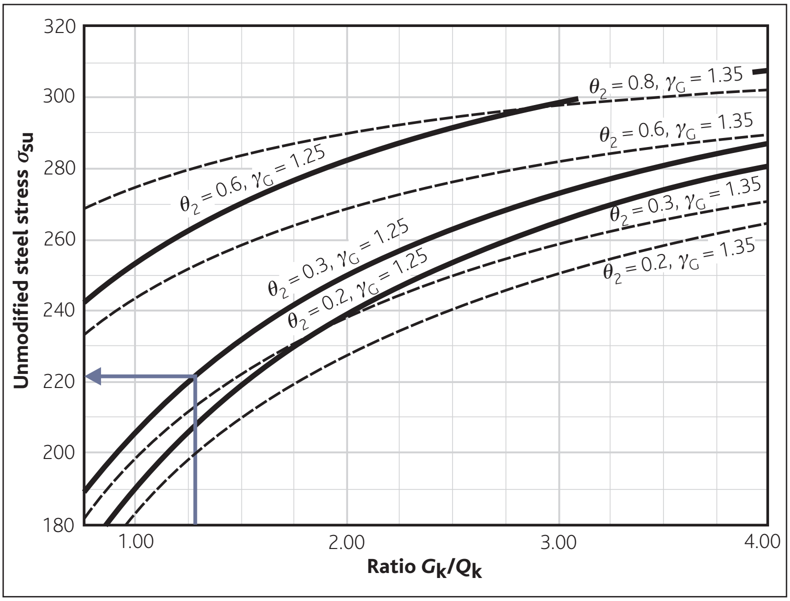

Figure C3
Determination of unmodified SLS, stress in reinforcement, σ_{su}

Table C14
(1/δ) 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/δ)	83%	87%	91%	95%	100%	105%	111%	118%	125%	133%	143%

Notes

1 Where coefficients from Table C2 have been used in design and where $Q_k \approx 1.25G_k$, the coefficients in Table C2 may be considered to represent moment distribution of:
–8% near middle of end span with pinned end support
–22% at first interior support, as a worst case
+3% near middle of internal spans, as a worst case
–28% at interior supports, as a worst case.

2 Where coefficients from Table C3 have been used in design and where $Q_k \approx G_k$, the coefficients in Table C3 may be considered to represent moment redistribution of:
+3% near middle of end span with pinned end support, as a worst case
+9% near middle of internal spans, as a worst case
–15% at all interior supports.

C8 Control of cracking

Cracking may be controlled by restricting either maximum bar diameter or maximum bar spacing to the relevant diameters and spacings given in Table C15. The appropriate SLS stress in reinforcement, σ_s, may be determined as outlined for F3 in Section C7.

Minimum areas and aspects of detailing should be checked.

Table C15
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

1 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.

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

C9 Design for axial load and bending

C9.1 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.

It is necessary to calculate effective lengths in order to determine whether a column is slender (see Eurocode 2, Cl. 5.8.3.2 and Expression (5.15)). The effective length of most columns will be $l/2 < l_0 < l$ (see Eurocode 2 Figure 5.7f). PD 6687[6] Cl. 2.10 suggests that using the procedure outlined in Eurocode 2 (5.8.3.2(3) and 5.8.3.2(5)) leads to similar effective lengths to those tabulated in BS 8110[7] as reproduced below as Table C16. Experience suggests that these tabulated values are conservative.

Table C16
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*[35]. The values are those used in BS 8110: Part 1: 1997[7] for braced columns. These values are close to those values that would be derived if the contribution from adjacent columns were ignored.

C9.2 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.

- 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)

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 = $\lambda x \leq h$

Concise: Fig. 6.3

Concise: Fig. 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

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

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 Figures C4a) to e).

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

C9.3 Rectangular column charts

Alternatively A_s may be estimated from column charts.

Figures C4a) to C4e) 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

f_{yk} ≤ 500 MPa

Simplified stress block assumed.

A_s = total area of reinforcement required

= $(A_s f_{yk}/bhf_{ck})bhf_{ck}/f_{yk}$

where

$(A_s f_{yk}/bhf_{ck})$ is derived from the appropriate design chart interpolating as necessary between charts for the value of d_2/h for the section.

b = breadth of section

h = height of section

Where reinforcement is not concentrated in the corners, a conservative approach is to calculate an effective value of d_2 as illustrated in Figures C4a) to e).

d_2 = effective depth to steel in layer 2

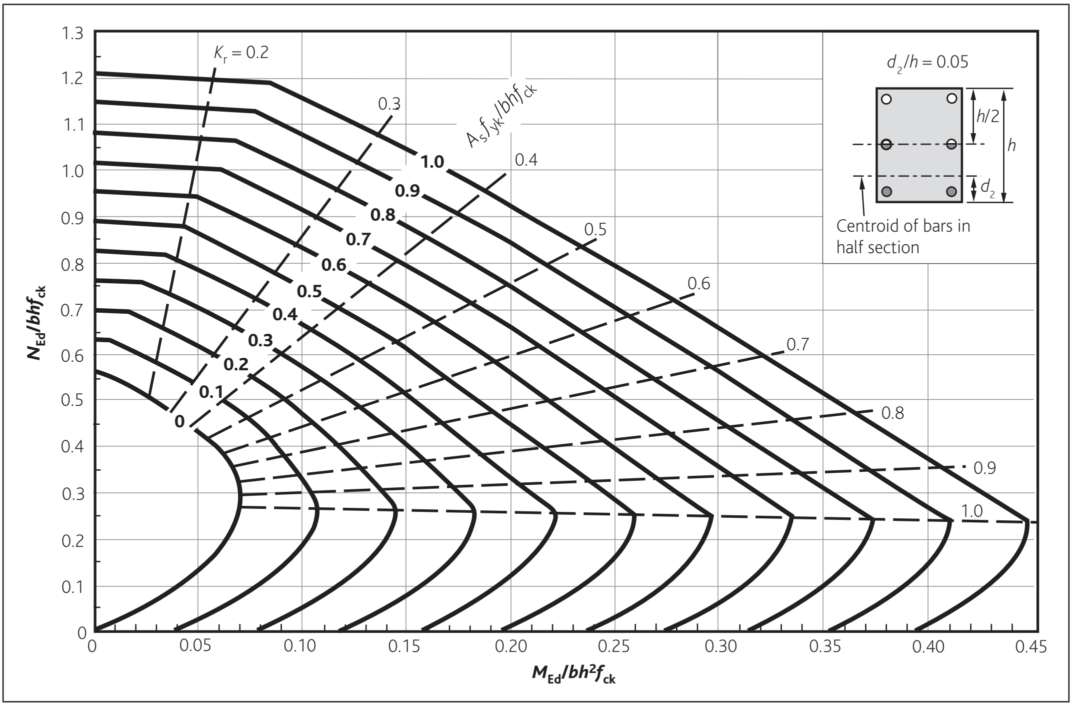

Figure C4a)
Rectangular columns d_2/h = 0.05

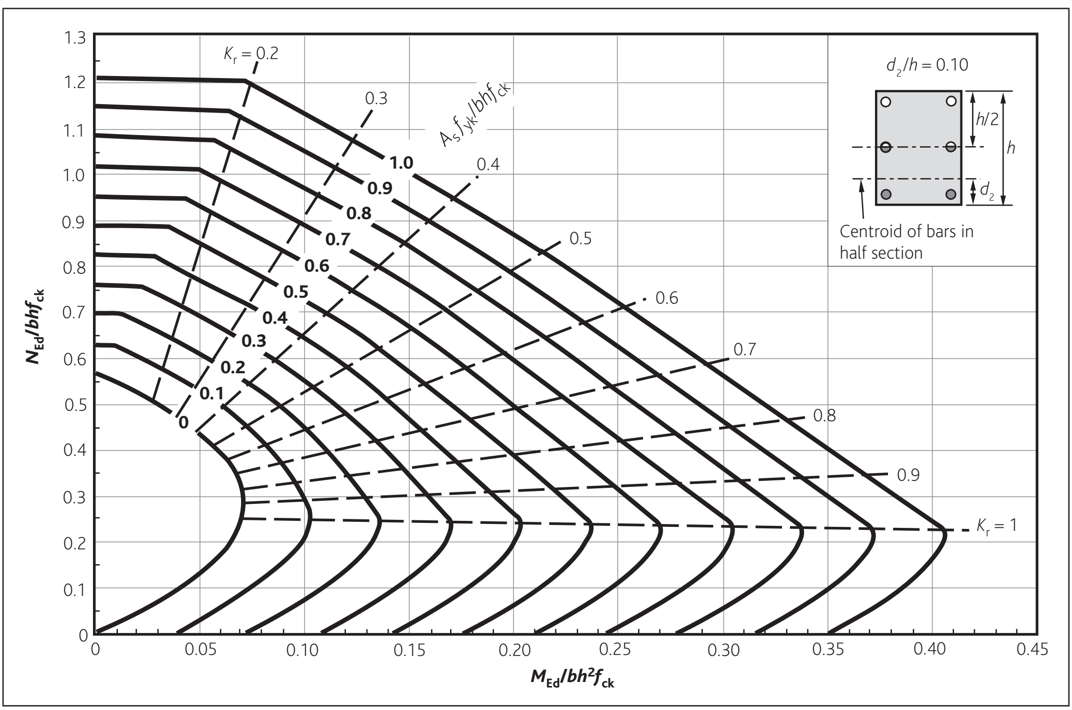

Figure C4b)
Rectangular columns d_2/h = 0.10

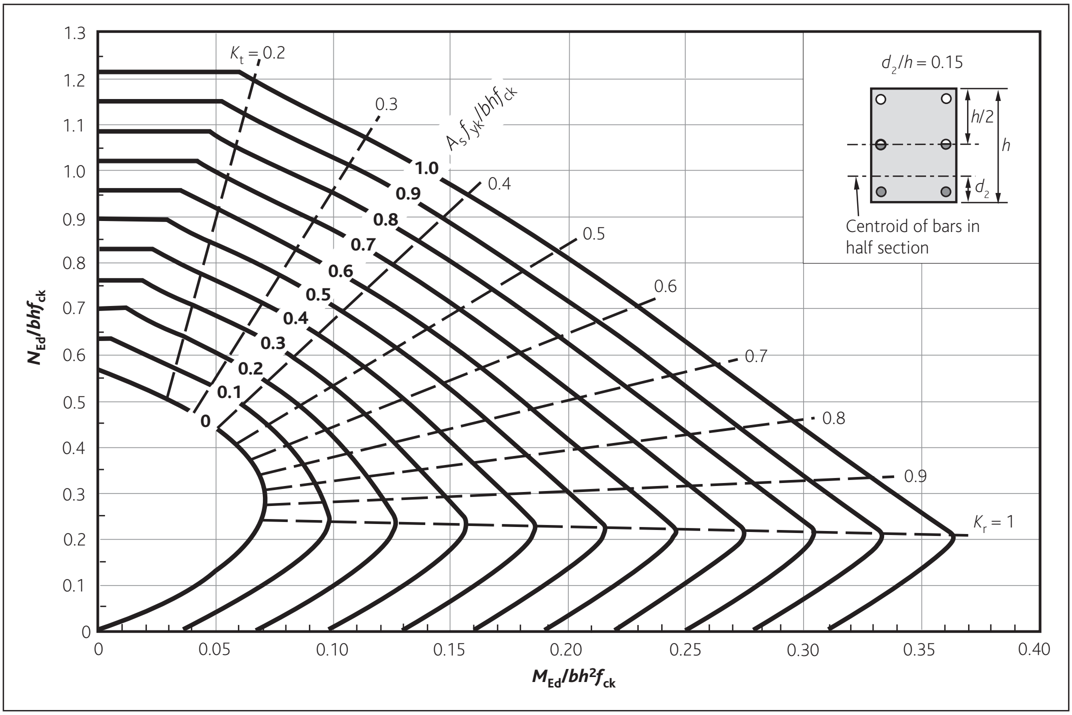

Figure C4c)
Rectangular columns $d_2/h = 0.15$

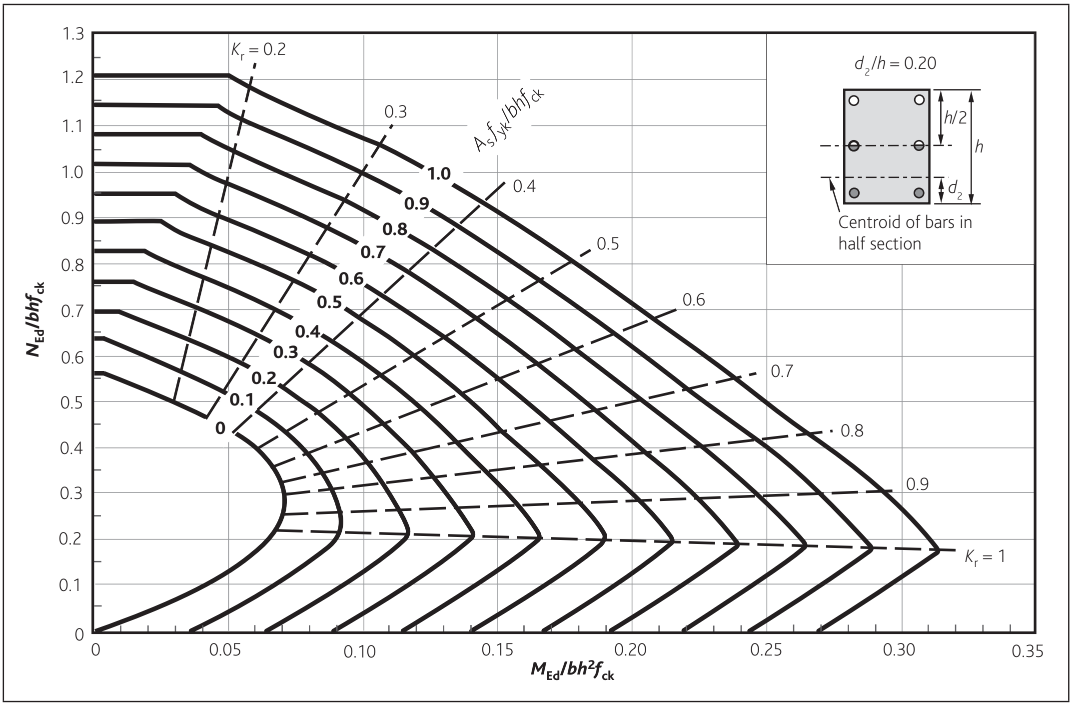

Figure C4d)
Rectangular columns $d_2/h = 0.20$

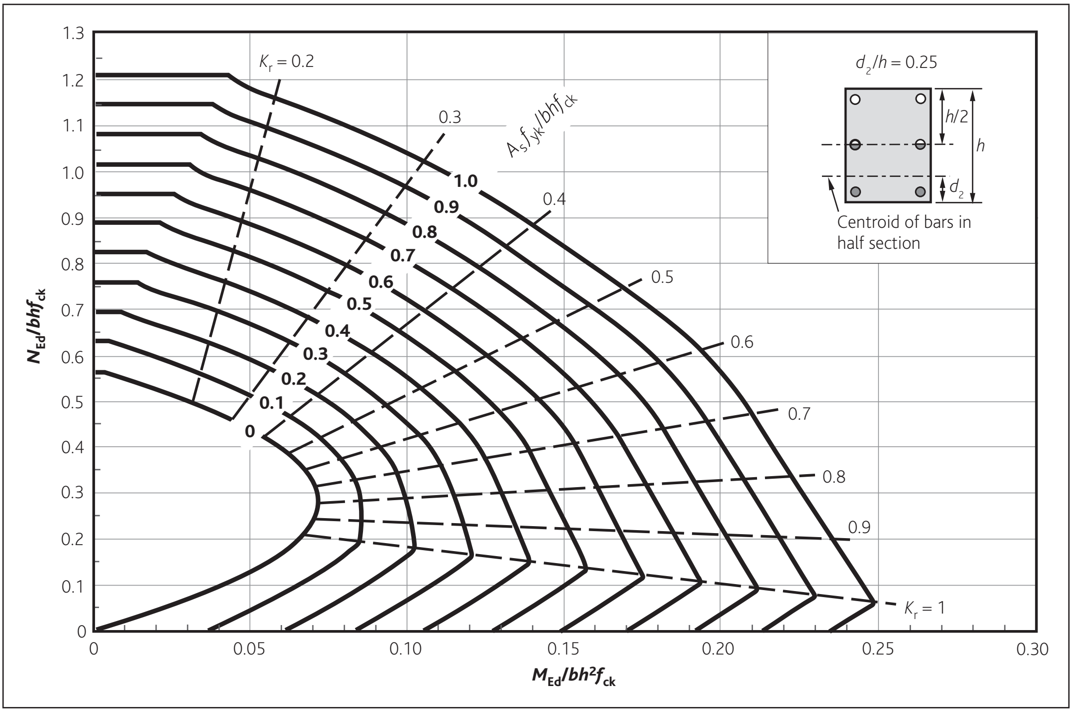

Figure C4e)
Rectangular columns $d_2/h = 0.25$

C9.4 Biaxial bending in rectangular columns

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 of *Concise Eurocode 2*.

Concise: 5.6.3

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

C9.5 Circular column charts

In a similar manner to C9.3, the area of reinforcement for circular columns A_s may be estimated from the charts in Figures C5a) to C5d).

In these charts:
α_{cc} = 0.85
f_{ck} ≤ 50 MPa
f_{yk} = 500 MPa
A_s = total area of reinforcement required
= $(A_s f_{yk}/h^2 f_{ck}) h^2 f_{ck}/f_{yk}$
where $(A_s f_{yk}/h^2 f_{ck})$ is derived from the appropriate design chart interpolating as necessary.
d/h = effective depth/overall diameter.

C9.6 Links

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.

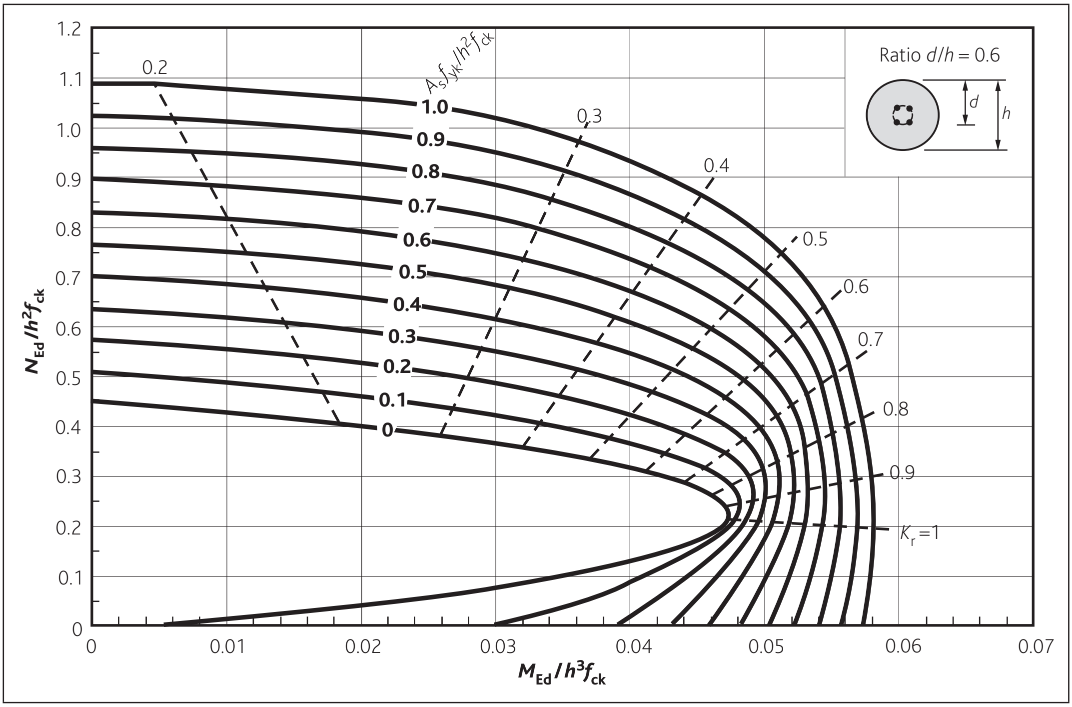

Figure C5a)
Circular columns *d*/*h* = 0.6

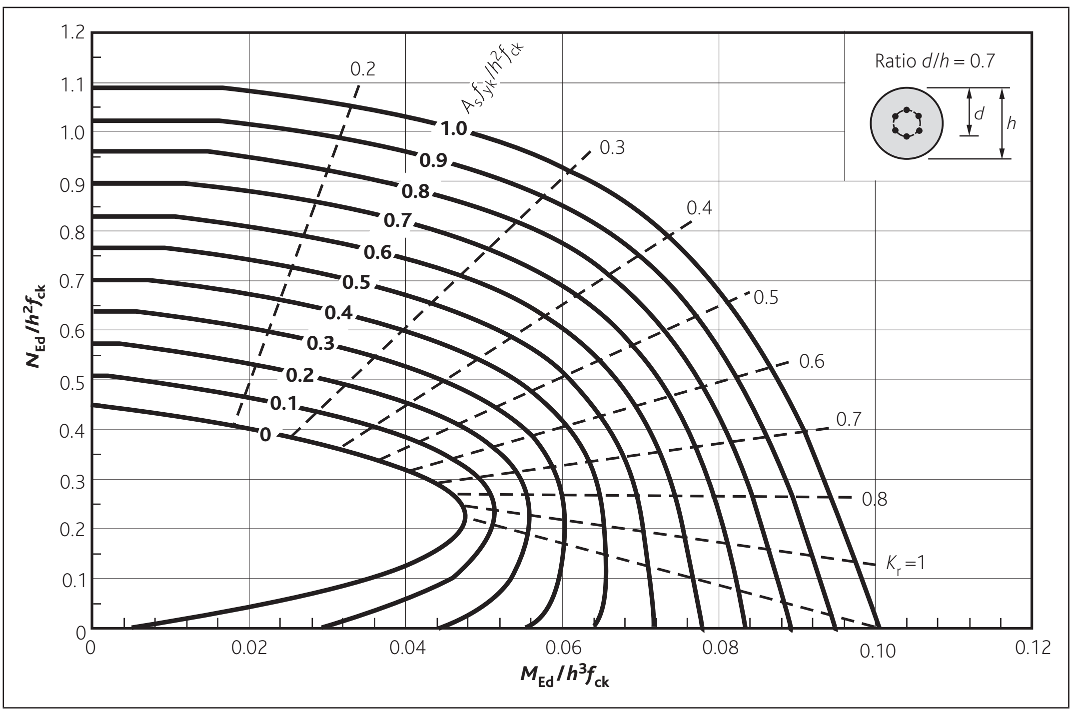

Figure C5b)
Circular columns *d*/*h* = 0.7

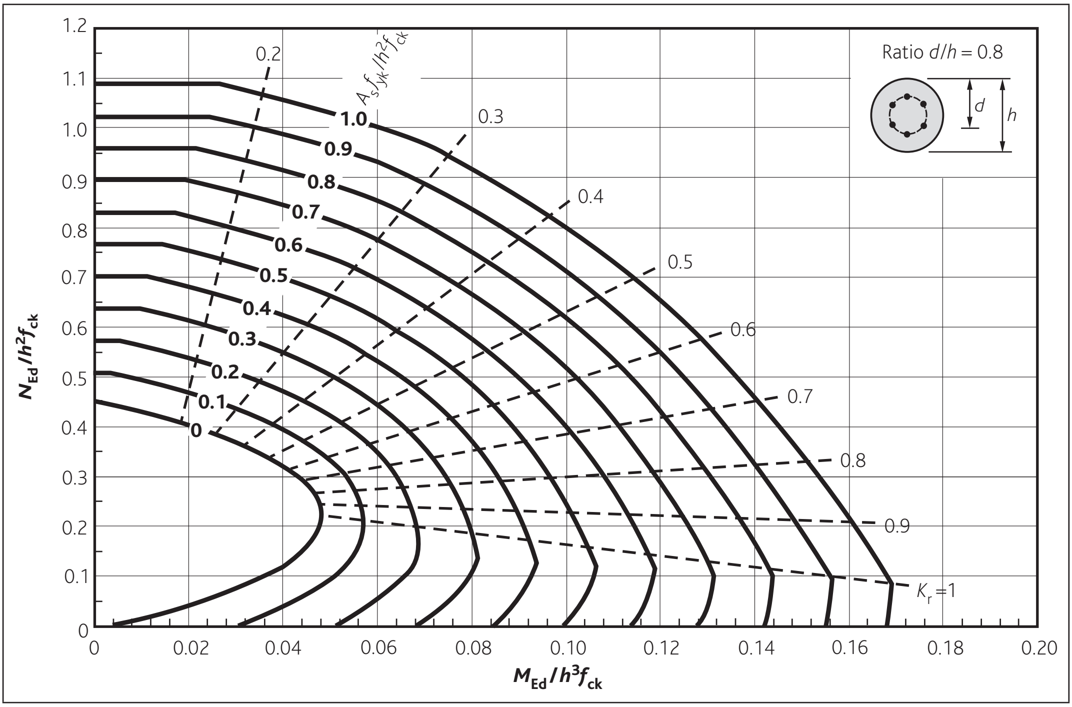

Figure C5c)
Circular columns *d/h* = 0.8

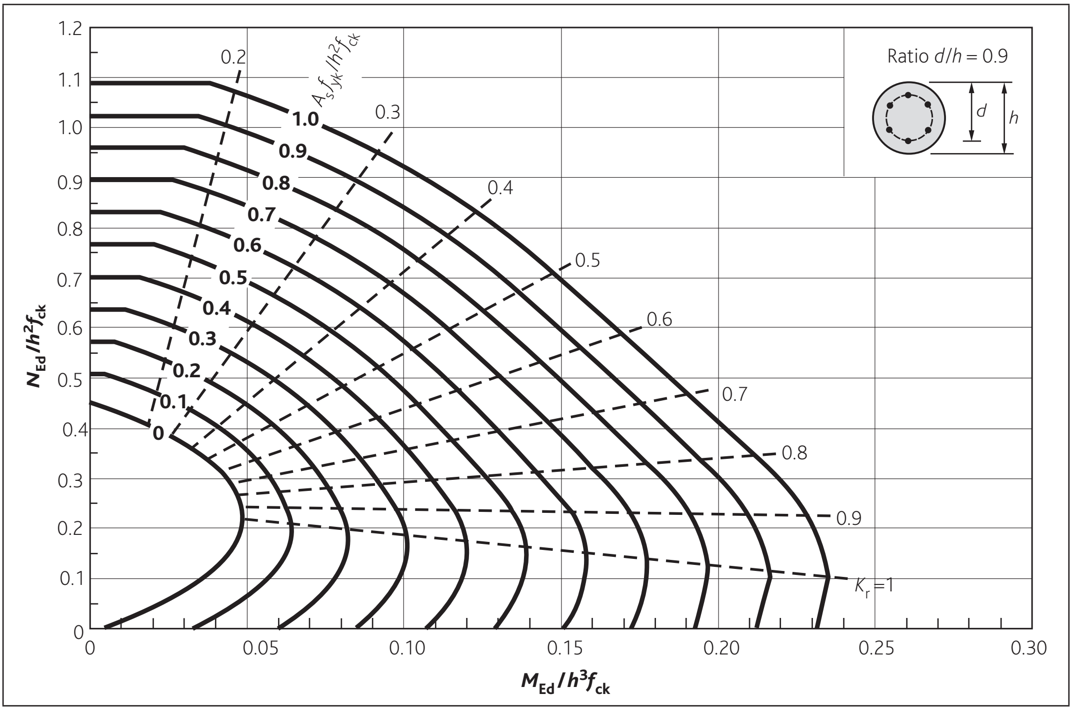

Figure C5d)
Circular columns *d/h* = 0.9

Eurocode 2 resources

Publications

Concise Eurocode 2
CCIP-005, The Concrete Centre, 2006
A handbook for the design of in-situ concrete buildings to Eurocode 2 and its UK National Annex

How to design concrete structures using Eurocode 2
CCIP-004, The Concrete Centre, 2006
Guidance for the design and detailing of a broad range of concrete elements to Eurocode 2

Economic concrete frame elements to Eurocode 2
CCIP-025, The Concrete Centre, 2009
A selection of reinforced concrete frame elements in multi-storey buildings

Precast Eurocode 2: Design manual
CCIP-014, British Precast Concrete Federation, 2008
A handbook for the design of precast concrete building structures to Eurocode 2 and its National Annex

Precast Eurocode 2: Worked examples
CCIP-034, British Precast Concrete Federation, 2008
Worked examples for the design of precast concrete buildings to Eurocode 2 and its National Annex

Concrete buildings scheme design manual
CCIP-051, The Concrete Centre 2009
A handbook for the ISructE chartered membership examination, based on EC2

Properties of concrete for use in Eurocode 2
CCIP-029, The Concrete Centre, 2008
How to optimize the engineering properties of concrete in design to Eurocode 2

Standard method of detailing structural concrete
Institution of Structural Engineers/ The Concrete Society, 2006
A manual for best practice

Manual for the design of concrete building structures to Eurocode 2
Institution of Structural Engineers, 2006
A manual for the design of concrete buildings to Eurocode 2 and its National Annex

BS EN 1992-1-1, Eurocode 2 – Part 1-1: Design of concrete structures – General rules and rules for buildings
British Standards Institution, 2004

National Annex to Eurocode 2 – Part 1-1
British Standards Institution, 2005

Software

RC spreadsheets: V3. User guide and CD
CCIP-008. The Concrete Centre, 2006
Excel spreadsheets for design to BS 8110 and Eurocode 2 and its UK National Annex

Websites

Eurocode 2 – www.eurocode2.info
Eurocodes Expert – www.eurocodes.co.uk
The Concrete Centre – www.concretecentre.com
Institution of Structural Engineers – www.istructe.org

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Initial section drafts

1 Introduction	Nary Narayanan
2 Analysis, actions and load arrangements	Nary Narayanan
3 Slabs	Charles Goodchild
4 Beams	Charles Goodchild, Rod Webster
5 Columns	Tony Jones, Jens Tandler
6 Walls	Tony Jones, Jens Tandler
Appendix A: Derivation of formulae	Charles Goodchild, Rod Webster, Owen Brooker
Appendix B: Serviceability limit state	Charles Goodchild, Nary Narayanan
Appendix C: Design aids	Charles Goodchild, Rod Webster

Worked Examples to Eurocode 2: Volume 1

This publication gives examples of the design to Eurocode 2 of common reinforced concrete elements in reinforced concrete framed buildings.

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

Volume 1 Worked Examples to 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**.

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CCIP-041
Published December 2009
ISBN 1-904818-84-7
Price Group P

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