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

How to Design Concrete Structures using Eurocode 2

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Foreword

The introduction of European standards to UK construction is a significant event. The ten design standards, known as the Eurocodes, will affect all design and construction activities as current British Standards for design are due to be withdrawn in 2010 at the latest. BS 8110, however, has an earlier withdrawal date of March 2008. The aim of this publication is to make the transition to *Eurocode 2: Design of concrete structures* as easy as possible by drawing together in one place key information and commentary required for the design and detailing of typical concrete elements.

The cement and concrete industry recognised that a substantial effort was required to ensure that the UK design profession would be able to use Eurocode 2 quickly, effectively, efficiently and with confidence. With support from government, consultants and relevant industry bodies, the Concrete Industry Eurocode 2 Group (CIEG) was formed in 1999 and this Group has provided the guidance for a co-ordinated and collaborative approach to the introduction of Eurocode 2. Part of the output of the CIEG project was the technical content for 7 of the 11 chapters in this publication. The remaining chapters have been developed by The Concrete Centre.

Acknowledgements

The content of Chapters 1 and 3 to 8 were produced as part of the project Eurocode 2: transition from UK to European concrete design standards. This project was part funded by the DTI under the Partners in Innovation scheme. The lead partner was British Cement Association. The work was carried out under the guidance of the Concrete Industry Eurocode 2 Group and overseen by a Steering Group of the CIEG (members are listed on inside back cover).

Particular thanks are due to Robin Whittle, technical editor to the CEN/TC 250/SC2 committee (the committee responsible for structural Eurocodes), who has reviewed and commented on the contents. Thanks are also due to John Kelly and Chris Clear who have contributed to individual chapters.

Gillian Bond, Issy Harvey, Kevin Smith and the designers at Media and Design Associates and Michael Burbridge Ltd have also made essential contributions to the production of this publication.

Published by The Concrete Centre
Riverside House, 4 Meadows Business Park, Station Approach, Blackwater, Camberley, Surrey GU17 9AB
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CCIP–006
Published December 2006
ISBN 1-904818-4-1
Price Group P

CCIP publications are produced on behalf of the Cement and Concrete Industry Publications Forum – an industry initiative to publish technical guidance in support of concrete design and construction.

CCIP publications are available from the Concrete Bookshop at **www.concrete bookshop.com**
Tel: +44(0)7004-607777

Printed by Michael Burbridge Ltd, Maidenhead.

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How to design concrete structures using Eurocode 2

1. Introduction to Eurocodes

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The Eurocode family

This chapter shows how to use Eurocode 2[1] with the other Eurocodes. In particular it introduces Eurocode: *Basis of structural design*[2] and Eurocode 1: *Actions on structures*[3] and guides the designer through the process of determining the design values for actions on a structure. It also gives a brief overview of the significant differences between the Eurocodes and BS 8110[4], (which will be superseded) and includes a glossary of Eurocode terminology.

The development of the Eurocodes started in 1975; since then they have evolved significantly and are now claimed to be the most technically advanced structural codes in the world. The many benefits of using Eurocode 2 are summarised below. There are ten Eurocodes covering all the main structural materials (see Figure 1). They are produced by the European Committee for Standardization (CEN), and will replace existing national standards in 28 countries.

Each country is required to publish a Eurocode with a national title page and forward but the original text of the Eurocode must appear as produced by CEN as the main body of the document. A National Annex (NA) can be included at the back of the document (see Figure 2). Throughout this publication it is assumed that the UK National Annexes will be used.

Table 1 details which existing standards relating to concrete design will be replaced by the new Eurocodes. During the implementation period it is recommended that existing standards are considered for use where the European standards have not yet been issued.

Benefits of using Eurocode 2

Learning to use the new Eurocodes will require time and effort on behalf of the designer, so what benefits will there be?

1. The new Eurocodes are claimed to be the most technically advanced codes in the world.
2. Eurocode 2 should result in more economic structures than BS 8110.
3. The Eurocodes are logical and organised to avoid repetition.
4. Eurocode 2 is less restrictive than existing codes.
5. Eurocode 2 is more extensive than existing codes.
6. Use of the Eurocodes will provide more opportunity for designers to work throughout Europe.
7. In Europe all public works must allow the Eurocodes to be used.

Figure 1
The Eurocodes

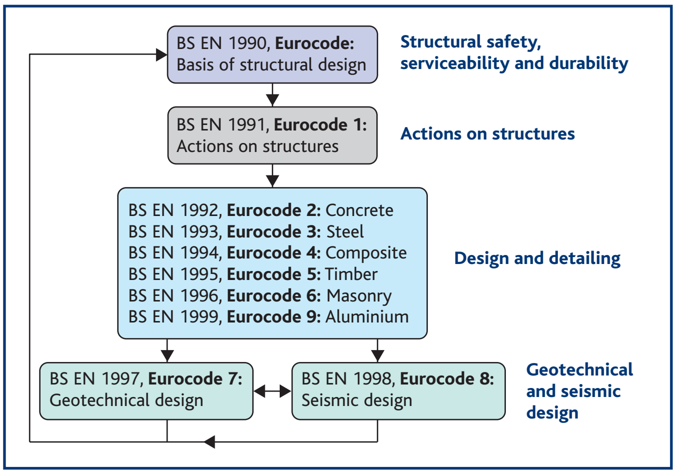


Figure 2
Typical Eurocode layout

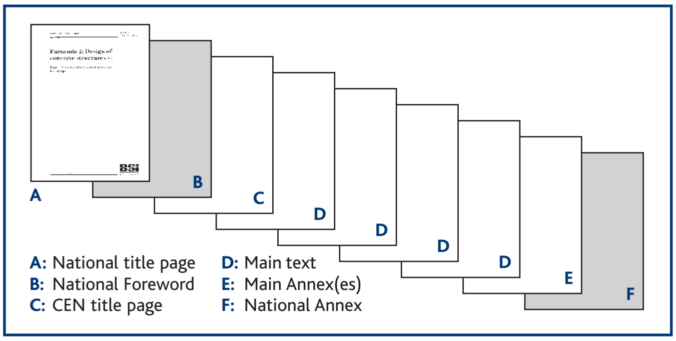


Table 1
Concrete related Eurocodes and their equivalent current standards

Eurocode	Title	Superseded standards
BS EN 1990	Basis of structural design	BS 8110: Part 1 – section 2
BS EN 1991–1–1	Densities, self-weight and imposed loads	BS 6399: Part 1 and BS 648
BS EN 1991–1–2	Actions on structures exposed to fire	–
BS EN 1991–1–3	Snow loads	BS 6399: Part 2
BS EN 1991–1–4	Wind actions	BS 6399: Part 3
BS EN 1991–1–5	Thermal actions	–
BS EN 1991–1–6	Actions during execution	–
BS EN 1991–1–7	Accidental actions	–
BS EN 1991–2	Traffic loads on bridges	BD 37/88
BS EN 1991–3	Actions induced by cranes and machinery	–
BS EN 1991–4	Silos and tanks	–
BS EN 1992–1–1	General rules for buildings	BS 8110: Parts 1, 2 and 3
BS EN 1992–1–2	Fire resistance of concrete structures	BS 8110: Part 1, Table 3.2 and BS 8110: Part 2, section 4
BS EN 1992–2	Bridges	BS 5400: Part 4
BS EN 1992–3	Liquid-retaining and containment structures	BS 8007
BS EN 1997–1	Geotechnical design – General rules	BS 6031, BS 8002, BS 8004, BS 8006, BS 8008 & BS 8081
BS EN 1997–2	Geotechnical design – Ground investigation and testing	BS 5930
BS EN 1998	Design of structures for earthquake resistance (6 parts)	–

Eurocode: Basis of structural design

This Eurocode underpins all structural design irrespective of the material of construction. It establishes principles and requirements for safety, serviceability and durability of structures. (Note, the correct title is Eurocode not Eurocode 0.) The Eurocode uses a statistical approach to determine realistic values for actions that occur in combination with each other.

There is no equivalent British Standard for Eurocode: *Basis of structural design* and the corresponding information has traditionally been replicated in each of the material Eurocodes. It also introduces new definitions (see Glossary) and symbols (see Tables 2a and 2b), which will be used throughout this publication to assist familiarity. Partial factors for actions are given in this Eurocode, whilst partial factors for materials are prescribed in their relevant Eurocode.

Representative values

For each variable action there are four representative values. The principal representative value is the characteristic value and this can be determined statistically or, where there is insufficient data, a nominal value may be used. The other representative values are combination, frequent and quasi-permanent; these are obtained by applying to the characteristic value the factors ψ_0, ψ_1 and ψ_2 respectively (see Figure 3). A semi-probabilistic method is used to derive the ψ factors, which vary depending on the type of imposed load (see Table 3). Further information on derivation of the ψ factors can be found in Appendix C of the Eurocode.

The combination value ($\psi_0 Q_k$) of an action is intended to take account of the reduced probability of the simultaneous occurrence of two or more variable actions. The frequent value ($\psi_1 Q_k$) is such that it should be exceeded only for a short period of time and is used primarily for the serviceability limit states (SLS) and also the accidental ultimate limit state (ULS). The quasi-permanent value ($\psi_2 Q_k$) may be exceeded for a considerable period of time; alternatively it may be considered as an average loading over time. It is used for the long-term affects at the SLS and also accidental and seismic ULS.

Combinations of actions

In the Eurocodes the term 'combination of actions' is specifically used for the definition of the magnitude of actions to be used when a limit state is under the influence of different actions. It should not be confused with 'load cases', which are concerned with the arrangement of the variable actions to give the most unfavourable conditions and are given in the material Eurocodes. The following process can be used to determine the value of actions used for analysis:

1. Identify the design situation (e.g. persistent, transient, accidental).
2. Identify all realistic actions.
3. Determine the partial factors (see below) for each applicable combination of actions.
4. Arrange the actions to produce the most critical conditions.

Where there is only one variable action (e.g. imposed load) in a combination, the magnitude of the actions can be obtained by multiplying them by the appropriate partial factors.

Where there is more than one variable action in a combination, it is necessary to identify the leading action ($Q_{k,1}$) and other accompanying actions ($Q_{k,i}$). The accompanying action is always taken as the combination value.

Ultimate limit state

The ultimate limit states are divided into the following categories:

EQU Loss of equilibrium of the structure.
STR Internal failure or excessive deformation of the structure or structural member.
GEO Failure due to excessive deformation of the ground.
FAT Fatigue failure of the structure or structural members.

The Eurocode gives different combinations for each of these ultimate limit states. For the purpose of this publication only the STR ultimate limit state will be considered.

For persistent and transient design situations under the STR limit state, the Eurocode defines three possible combinations, which are given in Expressions (6.10), (6.10a) and (6.10b) of the Eurocode (see Tables 4 and 5). The designer (for UK buildings) may use either (6.10) or the less favourable of (6.10a) and (6.10b).

At first sight it appears that there is considerably more calculation required to determine the appropriate load combination; however, with experience the designer will be able to determine this by inspection. 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, Table 3) where Expression (6.10a) always applies).

Therefore, for a typical concrete frame building, Expression (6.10b) will give the most structurally economical combination of actions.

> For members supporting one variable action the combination
> $1.25\ G_k + 1.5\ Q_k$ (derived from (Exp 6.10b))
> can be used provided the permanent actions are not greater than 4.5 times the variable actions (except for storage loads).

Serviceability limit state

There are three combinations of actions that can be used to check the serviceability limit states (see Tables 6 and 7). Eurocode 2 indicates which combination should be used for which phenomenon (e.g. deflection is checked using the quasi-permanent combination). Care should be taken not to confuse the SLS combinations of characteristic, frequent and quasi-permanent, with the representative values that have the same titles.

Table 2a
Selected symbols for Eurocode

Symbol	Definition
G_k	Characteristic value of permanent action
Q_k	Characteristic value of single variable action
γ_G	Partial factor for permanent action
γ_Q	Partial factor for variable action
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action
ξ	Combination factor for permanent actions

Table 2b
Selected subscripts

Subscript	Definition
A	Accidental situation
c	Concrete
d	Design
E	Effect of action
fi	Fire
k	Characteristic
R	Resistance
w	Shear reinforcement
y	Yield strength

Figure 3
Representative values of variable actions[5]

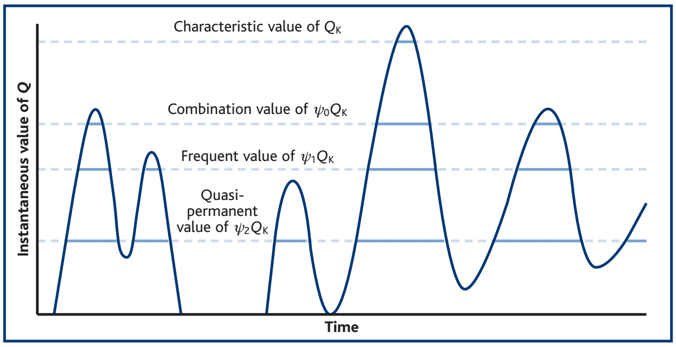


Table 3
Recommended values of ψ factors for buildings (from UK National Annex)

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see BS EN 1991–1–1)			
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*	0.7	0	0
Snow loads on buildings (see BS EN 1991–3)			
For sites located at altitude H > 1000 m above sea level	0.7	0.5	0.2
For sites located at altitude H < 1000 m above sea level	0.5	0.2	0
Wind loads on buildings (see BS EN 1991–1–4)	0.5	0.2	0
Temperature (non-fire) in buildings (see BS EN 1991–1–5)	0.6	0.5	0

Key
*See also 1991–1–1: Clause 3.3.2

Table 4

Design values of actions, ultimate limit state – persistent and transient design situations (table A1.2 (B) Eurocode)

Combination Expression reference	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Exp. (6.10)	$\gamma_{G,j,sup}\, G_{k,j,sup}$	$\gamma_{G,j,inf}\, G_{k,j,inf}$	$\gamma_{Q,1}\, Q_{k,1}$		$\gamma_{Q,1}\, \psi_{0,1}\, Q_{k,i}$
Exp. (6.10a)	$\gamma_{G,j,sup}\, G_{k,j,sup}$	$\gamma_{G,j,inf}\, G_{k,j,inf}$		$\gamma_{Q,1}\, \psi_{0,1}\, Q_{k,1}$	$\gamma_{Q,1}\, \psi_{0,1}\, Q_{k,i}$
Exp. (6.10b)	$\xi\gamma_{G,j,sup}\, G_{k,j,sup}$	$\gamma_{G,j,inf}\, G_{k,j,inf}$	$\gamma_{Q,1}\, Q_{k,1}$		$\gamma_{Q,1}\, \psi_{0,1}\, Q_{k,i}$

Note

1 Design for either Expression (6.10) or the less favourable of Expressions (6.10a) and (6.10b).

Table 5

Design values of actions, derived for UK design, ultimate limit state – persistent and transient design situations

Combination Expression reference	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Combination of permanent and variable actions					
Exp. (6.10)	1.35 G_k[a]	1.0 G_k[a]	1.5[c] Q_k		
Exp. (6.10a)	1.35 G_k[a]	1.0 G_k[a]		1.5 $\psi_{0,1}$[b] Q_k	
Exp. (6.10b)	0.925[d] x 1.35 G_k[a]	1.0 G_k[a]	1.5[c] Q_k		
Combination of permanent, variable and accompanying variable actions					
Exp. (6.10)	1.35 G_k[a]	1.0 G_k[a]	1.5[c] $Q_{k,1}$		1.5[c] $\psi_{0,i}$[b] $Q_{k,i}$
Exp. (6.10a)	1.35 G_k[a]	1.0 G_k[a]		1.5 $\psi_{0,1}$[b] Q_k	1.5[c] $\psi_{0,i}$[b] $Q_{k,i}$
Exp. (6.10b)	0.925[d] x 1.35 G_k[a]	1.0 G_k[a]	1.5[c] $Q_{k,1}$		1.5[c] $\psi_{0,i}$[b] $Q_{k,i}$

Key

a Where the variation in permanent action is not considered significant, $G_{k,j,sup}$ and $G_{k,j,inf}$ may be taken as G_k

b The value of ψ_0 can be obtained from Table NA A1.1 of the UK National Annex (reproduced here as Table 3)

c Where the accompanying load is favourable, $\gamma_{Q,i} = 0$

d The value of ξ in the UK National Annex is 0.925

Table 6

Design values of actions, serviceability limit states

Combination	Permanent actions		Variable actions		Example of use in Eurocode 2
	Unfavourable	Favourable	Leading	Others	
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	$Q_{k,1}$	$\psi_{0,i}\, Q_{k,i}$	
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{1,1}\, Q_{k,1}$	$\psi_{2,i}\, Q_{k,i}$	Cracking – prestressed concrete
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{2,1}\, Q_{k,1}$	$\psi_{2,i}\, Q_{k,i}$	Deflection

Notes

1 Where the variation in permanent action is not considered significant. $G_{k,j,sup}$ and $G_{k,j,inf}$ may be taken as G_k

2 For values of ψ_0, ψ_1 and ψ_2 refer to Table 3

Table 7

Example design combinations for deflection (quasi-permanent) derived for typical UK reinforced concrete design

Combination	Permanent actions	Variable action
	Unfavourable	Leading
Office	G_k[a]	0.3[b] $Q_{k,1}$
Shopping area	G_k[a]	0.6[b] $Q_{k,1}$
Storage	G_k[a]	0.8[b] $Q_{k,1}$

Key

a Where the variation in permanent action is not considered significant $G_{k,j,sup}$ and $G_{k,j,inf}$ may be taken as G_k

b Values of ψ_2 are taken from UK NA (see Table 3)

Eurocode 1

Eurocode 1 supersedes BS 6399: *Loading for buildings*[6] and BS 648: *Schedule of weights of building materials*[7]. It contains within its ten parts (see Table 8) all the information required by the designer to assess the individual actions on a structure. It is generally self-explanatory and it is anticipated the actions to be used in the UK (as advised in the UK National Annex) will typically be the same as those in the current British Standards. The most notable exception is the bulk density of reinforced concrete, which has been increased to 25 kN/m^3. Currently not all the parts of Eurocode 1 and their National Annexes are available, in which case it is advised that the loads recommended in the current British Standards are used.

Eurocode 2

There are four parts to Eurocode 2; Figure 4 indicates how they fit into the Eurocode system, which includes other European standards.

Part 1–1

Eurocode 2, Part 1–1: *General rules and rules for buildings*[9] is the principal part which is referenced by the three other parts. For the UK designer there are a number of differences between Eurocode 2 and BS 8110, which will initially make the new Eurocode seem unfamiliar. The key differences are listed below to assist in the familiarisation process.

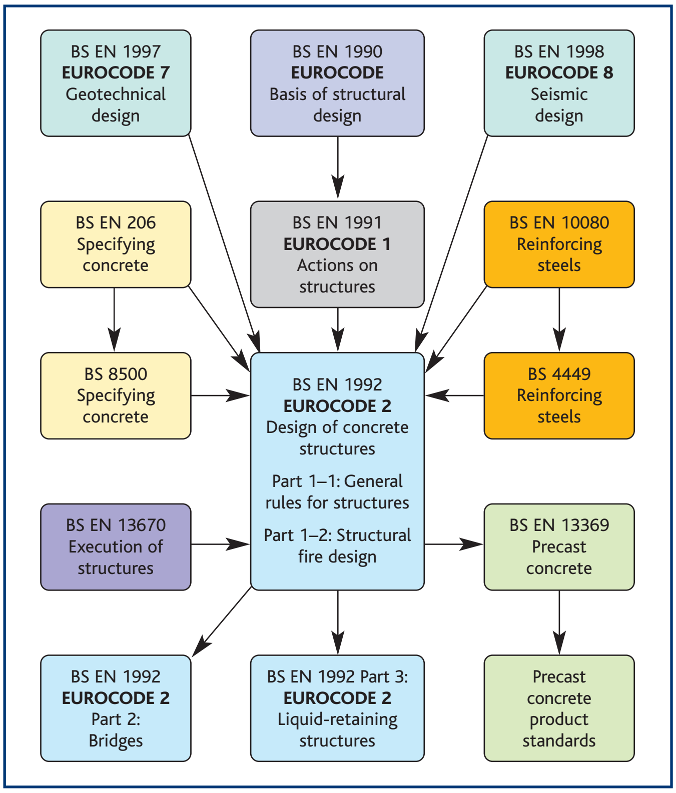
1. Eurocode 2 is generally laid out to give advice on the basis of phenomena (e.g. bending, shear etc) rather than by member types as in BS 8110 (e.g. beams, slabs, columns etc).
2. Design is based on characteristic cylinder strengths not cube strengths.
3. The Eurocode does not provide derived formulae (e.g. for bending, only the details of the stress block are expressed). This is the traditional European approach, where the application of a Eurocode is expected to be provided in a textbook or similar publication. The Eurocodes allow for this type of detail to be provided in 'Non-contradictory complementary information' (NCCI) (See Glossary).
4. Units for stress are mega pascals, MPa (1 MPa = 1 N/mm^2).
5. Eurocode 2 uses a comma for a decimal point. It is expected that UK designers will continue to use a decimal point. Therefore to avoid confusion, the comma should not be used for separating multiples of a thousand.
6. One thousandth is represented by ‰.
7. The partial factor for steel reinforcement is 1.15. However, the characteristic yield strength of steel that meets the requirements of BS 4449 will be 500 MPa; so overall the effect is negligible.
8. Eurocode 2 is applicable for ribbed reinforcement with characteristic yield strengths of 400 to 600 MPa. There is no guidance on plain bar or mild steel reinforcement in the Eurocode, but guidance is given in the background paper to the UK National Annex[10].
9. The effects of geometric imperfection ('notional horizontal loads') are considered in **addition** to lateral loads.

Table 8
Eurocode 1, its parts and dates of publication

Reference	Title	Publication date	
		Eurocode	National Annex
BS EN 1991–1–1	Densities, self-weight and imposed loads	July 2002	December 2005
BS EN 1991–1–2	Actions on structures exposed to fire	November 2002	Due October 2006[a]
BS EN 1991–1–3	Snow loads	July 2003	December 2005
BS EN 1991–1–4	Wind actions	April 2005	Due January 2007[a]
BS EN 1991–1–5	Thermal actions	March 2004	Due December 2006[a]
BS EN 1991–1–6	Actions during execution	December 2005	Due June 2007[a]
BS EN 1991–1–7	Accidental actions due to impact and explosions	September 2006	Due October 2007[a]
BS EN 1991–2	Traffic loads on bridges	October 2003	Due December 2006[a]
BS EN 1991–3	Actions induced by cranes and machinery	September 2006	Due January 2007[a]
BS EN 1991–4	Actions in silos and tanks	June 2006	Due June 2007[a]

Key
a Planned publication date (correct at time of publication) Source: BSI[8]

Figure 4
Relationship between Eurocode 2 and other Eurocodes



10. Minimum concrete cover is related to bond strength, durability and fire resistance. In addition to the minimum cover an allowance for deviations due to variations in execution (construction) should be included. Eurocode 2 recommends that, for concrete cast against formwork, this is taken as 10 mm, unless the construction is subject to a quality assurance system in which case it could be reduced to 5 mm or even 0 mm where non-conforming members are rejected (e.g. in a precast yard). It is recommended that the nominal cover is stated on the drawings and construction tolerances are given in the specification.
11. Higher strengths of concrete are covered by Eurocode 2, up to class C90/105. However, because the characteristics of higher strength concrete are different, some Expressions in the Eurocode are adjusted for classes above C50/60.
12. The 'variable strut inclination' method is used in Eurocode 2 for the assessment of the shear capacity of a section. In practice, design values for actual structures can be compared with tabulated values. Further advice can be found in Chapter 4, originally published as *Beams*[11].
13. The punching shear checks are carried out at 2*d* from the face of the column and for a rectangular column, the perimeter is rounded at the corners.
14. Serviceability checks can still be carried out using 'deemed to satisfy' span to effective depth rules similar to BS 8110. However, if a more detailed check is required, Eurocode 2 guidance varies from the rules in BS 8110 Part 2.
15. The rules for determining the anchorage and lap lengths are more complex than the simple tables in BS 8110. Eurocode 2 considers the effects of, amongst other things, the position of bars during concreting, the shape of the bar and cover.

Part 1–2

Eurocode 2, Part 1–2: *Structural fire design*[12], gives guidance on design for fire resistance of concrete structures. Although much of the Eurocode is devoted to fire engineering methods, the design for fire resistance may still be carried out by referring to tables for minimum cover and dimensions for various elements. These are given in section 5 of Part 1–2. Further advice on using the tabular method is given in Chapter 2, originally published as *Getting started*[13].

Part 2

Eurocode 2, Part 2: *Bridges*[14] applies the general rules given in Part 1–1 to the design of concrete bridges. As a consequence both Part 1–1 and Part 2 will be required to carry out a design of a reinforced concrete bridge.

Part 3

Eurocode 2, Part 3: *Liquid-retaining and containment structures*[15] applies the general rules given in Part 1–1 to the liquid-retaining structures and supersedes BS 8007[16].

Eurocode 7

Eurocode 7: *Geotechnical design*[17] is in two parts and gives guidance on geotechnical design, ground investigation and testing. It has a broad scope and includes the geotechnical design of spread foundations, piled foundations, retaining walls, deep basements and embankments. Like all the Eurocodes it is based on limit state design principles, which is a significant variation for most geotechnical design. Further guidance related to simple foundations is given in Chapter 6, originally ppublished as *Foundations*[18].

Eurocode 8

Eurocode 8: *Design of structures for earthquake resistance*[19] is divided into six parts and gives guidance on all aspects of design for earthquake resistance and covers guidance for the various structural materials for all types of structures. It also includes guidance for strengthening and repair of buildings. In areas of low seismicity it is anticipated that detailing structures to Eurocode 2 will ensure compliance with Eurocode 8.

Related Standards

BS 8500/BS EN 206

BS 8500: *Concrete – Complementary British Standard to BS EN 206–1*[20] replaced BS 5328 in December 2003 and designers should currently be using this to specify concrete. Further guidance can found in Chapter 11, originally published as *How to use BS 8500 with BS 8110*[21].

BS 4449/BS EN 10080

BS 4449: *Specification for carbon steel bars for the reinforcement of concrete*[22] has been revised ready for implementation in January 2006. It is a complementary standard to BS EN 10080 *Steel for the reinforcement of concrete*[23] and Normative Annex C of Eurocode 2. The most significant changes are that steel characteristic yield will change to 500 MPa. There are three classes of reinforcement, A, B and C, which indicate increasing ductility. Class A is not suitable for use where redistribution of 20% and above has been assumed in the design.

BS EN 13670

BS 8110 Part 1 sections 6 and 7 specify the workmanship for concrete construction. There is no equivalent guidance in Eurocode 2, and it is intended that execution (construction) will be covered in a new standard BS EN 13670 *Execution of concrete structures*[24]. This is still in preparation and is not expected to be ready for publication until 2008 at the earliest. In the intervening period the draft background paper to the UK National Annex of Eurocode 2, Part 1–1[10] recommends that designers use the *National structural concrete specification for building construction*[25], which refers to BS 8110 for workmanship.

Glossary of Eurocode terminology

Term	Definition
Principles	Clauses that are general statements, definitions, requirements and analytical models for which no alternative is permitted. They are identified by (P) after the clause number.
Application Rules	These are generally recognised rules, which comply with the principles and satisfy their requirements.
Nationally Determined Parameter (NDP)	Eurocodes may be used to satisfy national Building Regulations, which themselves will not be harmonized. NDPs are therefore used to allow a country to set its own levels of safety. NDPs also allow certain other parameters (generally influenced by climate, geography and geology) to be left open for selection nationally: NDPs are advised in the National Annex.
National Annex (NA)	A National Annex accompanies each Eurocode and it contains a) the values of NDPs b) the national decision regarding the use of Informative Annexes and c) references to NCCIs
Normative	The term used for the text of Standards that forms the core requirements. Compliance with Eurocodes will generally be judged against the normative requirements.
Informative	A term used only in relation to annexes, which seek to inform rather than require.
NCCI	Non-contradictory complementary information. References in a National Annex which contains further information or guidance which does not contradict the Eurocode.
Characteristic value	A value that may be derived statistically with a probability of not being exceeded during a reference period. The value corresponds to a specified fractile for a particular property of material or product. The characteristic values are denoted by subscript 'k' (e.g. Q_k etc). It is the principal representative value from which other representative values may be derived.
Representative value	Value used for verification of a limit state. It may be the characteristic value or an accompanying value, e.g. combination, frequent or quasi-permanent.
Design values	These refer to representative values modified by partial factors. They are denoted by subscript 'd' (e.g. $f_{cd} = f_{ck}/\gamma_c$; $Q_d = \gamma_Q Q_k$).
Action (*F*)	Set of forces, deformations or accelerations acting on the structure.
Combination of actions	Set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different and statistically independent actions.
Fixed action	Action that has a fixed distribution and position over the structure or structural member.
Free action	Action that may have various spatial distributions over the structure.
Permanent actions (*G*)	Actions that are likely to act throughout the life of the structure and whose variation in magnitude with time is negligible (e.g. permanent loads).
Variable actions (*Q*)	Actions whose magnitude will vary with time (e.g. wind loads).
Effect of action (*E*)	Deformation or internal force caused by an action.
Accidental action (*A*)	Action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life.
Accompanying action	An action in a combination that is not the leading variable action.
Transient design situation	Design situation that is relevant during a period much shorter than the design working life of the structure.
Persistent design situation	Design situation that is relevant during a period of the same order as the design working life of the structure.
Accidental design situation	Design situation involving exceptional conditions of the structure.
Irreversible serviceability limit state	Serviceability limit state where some consequences of actions will remain when the actions are removed.
Reversible serviceability limit state	Serviceability limit state where no consequences of actions will remain when the actions are removed.
Execution	Construction of the works.

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2. Getting started

O Brooker BEng, CEng, MICE, MIStructE

The design process

This chapter is intended to assist the designer determine all the design information required prior to embarking on detailed element design. It covers design life, actions on structures, load arrangements, combinations of actions, method of analysis, material properties, stability and imperfections, minimum concrete cover and maximum crack widths.

The process of designing elements will not be revolutionised as a result of using Eurocode 2[1], although much of the detail may change – as described in subsequent chapters.

Similarly, the process of detailing will not vary significantly from current practice. Guidance can be found in Chapter 10 or in *Standard method of detailing*[2]. With regard to specification, advice can be found in Chapter 1, originally published as *Introduction to Eurocodes*[3]. Concept designs prepared assuming that detailed design would be to BS 8110 may be continued through to detailed design using Eurocode 2.

In the long-term it is anticipated that Eurocode 2 will lead to more economic structures.

Design life

The design life for a structure is given in Eurocode: *Basis of structural design*[4]. The UK National Annex (NA) to Eurocode presents UK values for design life; these are given in Table 1 (overleaf). These should be used to determine the durability requirements for the design of reinforced concrete structures.

Actions on structures

Eurocode 1: *Actions on structures*[5] consists of 10 parts giving details of a wide variety of actions. Further information on the individual codes can be found in Chapter 1. Eurocode 1, Part 1–1: *General actions – Densities, self-weight, imposed loads for buildings*[6] gives the densities and self-weights of building materials (see Table 2 overleaf).

The key change to current practice is that the bulk density of reinforced concrete has been increased to 25 kN/m^3. The draft National Annex to this Eurocode gives the imposed loads for UK buildings and a selection is

Table 1
Indicative design working life (from UK National Annex to Eurocode)

Design life (years)	Examples
10	Temporary structures
10–30	Replaceable structural parts
15–25	Agricultural and similar structures
50	Buildings and other common structures
120	Monumental buildings, bridges and other civil engineering structures

Table 2
Selected bulk density of materials (from Eurocode 1, Part 1–1)

Material	Bulk density (kN/m^3)
Normal weight concrete	24.0
Reinforced normal weight concrete	25.0
Wet normal weight reinforced concrete	26.0

Figure 1
Alternate spans loaded

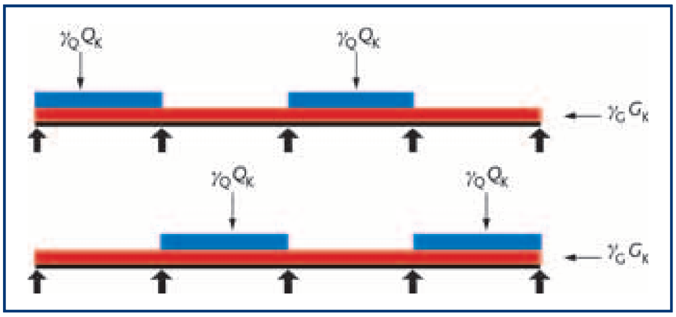


Figure 2
Adjacent spans loaded

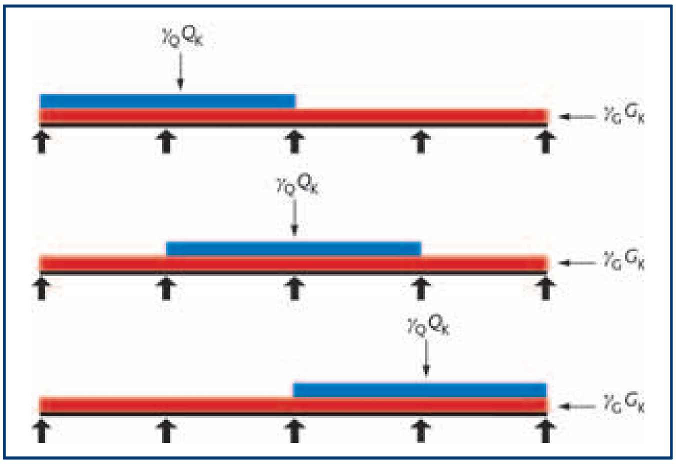
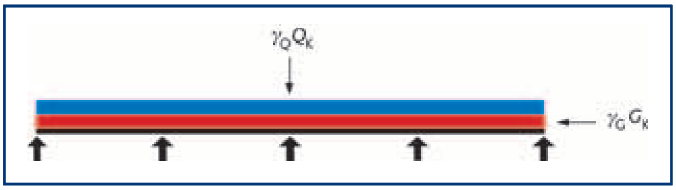


Figure 3
All spans loaded



reproduced in Table 3. It should be noted that there is no advice given for plant rooms.

At the time of writing not all the parts of Eurocode 1 and their National Annexes are available; it is advised that existing standards are considered for use where European standards have not yet been issued.

Load arrangements

The term load arrangements refers to the arranging of variable actions (e.g. imposed and wind loads) to give the most onerous forces in a member or structure and are given in Eurocode 2 and its UK NA.

For building structures, the UK NA to Eurocode 2, Part 1–1 allows any of the following sets of load arrangements to be used for both the ultimate limit state and serviceability limit state:

Load set 1. Alternate or adjacent spans loaded

The design values should be obtained from the more critical of:

- Alternate spans carrying the design variable and permanent loads with other spans loaded with only the design permanent load (see Figure 1). The value of γ_G should be the same throughout.
- Any two adjacent spans carrying the design variable and permanent loads with other spans loaded with only the design permanent load (see Figure 2). The value of γ_G should be the same throughout.

Load set 2. All or alternate spans loaded

The design values should be obtained from the more critical of:

- All spans carrying the design variable and permanent loads (see Figure 3).
- Alternate spans carrying the design variable and permanent loads with other spans loaded with only the design permanent load (see Figure 1). The value of γ_G should be the same throughout.

Generally, load set 2 will be used for beams and slabs in the UK as it requires three load arrangements to be considered, while load set 1 will often require more than three arrangements to be assessed. Alternatively, the UK NA makes the following provision for slabs.

Load set 3. Simplified arrangements for slabs

The load arrangements can be simplified for slabs where it is only necessary to consider the all spans loaded arrangement (see Figure 3), provided the following conditions are met:

- In a one-way spanning slab the area of each bay exceeds 30 m^2 (a bay means a strip across the full width of a structure bounded on the other sides by lines of support).
- The ratio of the variable actions (Q_k) to the permanent actions (G_k) does not exceed 1.25.
- The magnitude of the variable actions excluding partitions does not exceed 5 kN/m^2.

Combination of actions

The term combination of actions refers to the value of actions to be used when a limit state is under the influence of different actions.

The numerical values of the partial factors for the ULS combination can be obtained by referring to Eurocode: *Basis of structural design* or to Chapter 1.

> For members supporting one variable action the ULS combination $1.25\ G_k + 1.5\ Q_k$ (derived from Exp. (6.10b), Eurocode) can be used provided the permanent actions are not greater than 4.5 times the variable actions (except for storage loads).

There are three SLS combinations of actions – characteristic, frequent and quasi-permanent. The numerical values are given in Eurocode: *Basis of structural design*.

Material properties

Concrete

In Eurocode 2 the design of reinforced concrete is based on the characteristic cylinder strength rather than cube strength and should be specified according to BS 8500: *Concrete – complementary British Standard to BS EN 206–1*[7] (e.g. for class C28/35 concrete the cylinder strength is 28 MPa, whereas the cube strength is 35 MPa). Typical concrete properties are given in Table 4.

Concrete up to class C90/105 can be designed using Eurocode 2. For classes above C50/60, however, there are additional rules and variations. For this reason, the design of these higher classes is not considered in this publication.

It should be noted that designated concretes (e.g. RC30) still refer to the cube strength.

Reinforcing steel

Eurocode 2 can be used with reinforcement of characteristic strengths ranging from 400 to 600 MPa. The properties of steel reinforcement in the UK for use with Eurocode 2 are given in BS 4449 (2005): *Specification for carbon steel bars for the reinforcement of concrete*[8] and are summarised in Table 5 (on page 4). A characteristic yield strength of 500 MPa has been adopted by the UK reinforcement industry. There are three classes of reinforcement, A, B and C, which provide increasing ductility. Class A is not suitable where redistribution of 20% and above has been assumed in the design. There is no provision for the use of plain bar or mild steel reinforcement, but guidance is given in the background paper to the National Annex[9].

Table 3
Selected imposed loads for buildings (from draft UK National Annex to Eurocode 1, Part 1–1)

Category	Example use	q_k (kN/m²)	Q_k (kN)
A1	All uses within self-contained dwelling units	1.5	2.0
A2	Bedrooms and dormitories	1.5	2.0
A3	Bedrooms in hotels and motels, hospital wards and toilets	2.0	2.0
A5	Balconies in single family dwelling units	2.5	2.0
A7	Balconies in hotels and motels	4.0 min.	2.0 at outer edge
B1	Offices for general use	2.5	2.7
C5	Assembly area without fixed seating, concert halls, bars, places of worship	5.0	3.6
D1/2	Shopping areas	4.0	3.6
E12	General storage	2.4 per m height	7.0
E17	Dense mobile stacking in warehouses	4.8 per m height (min. 15.0)	7.0
F	Gross vehicle weight ≤ 30kN	2.5	10.0

Table 4
Selected concrete properties based on Table 3.1 of Eurocode 2, Part 1–1

Symbol	Description	Properties										
f_{ck} (MPa)	Characteristic cylinder strength	12	16	20	25	30	35	40	45	50	28[a]	32[a]
$f_{ck,cube}$ (MPa)	Characteristic cube strength	15	20	25	30	37	45	50	55	60	35	40
f_{ctm} (MPa)	Mean tensile strength	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	2.8	3.0
E_{cm}[b] (GPa)	Secant modulus of elasticity	27	29	30	31	33	34	35	36	37	32	34

Key
a Concrete class not cited in Table 3.1, Eurocode 2, Part 1–1
b Mean secant modulus of elasticity at 28 days for concrete with quartzite aggregates. For concretes with other aggregates refer to Cl 3.1.3 (2)

Table 5
Characteristic tensile properties of reinforcement

Class (BS 4449) and designation (BS 8666)	A	B	C
Characteristic yield strength f_{yk} or $f_{0.2k}$ (MPa)	500	500	500
Minimum value of $k = (f_t/f_y)_k$	≥ 1.05	≥ 1.08	≥ 1.15 < 1.35
Characteristic strain at maximum force ε_{uk} (%)	≥ 2.5	≥ 5.0	≥ 7.5

Notes
1 Table derived from BS EN 1992–1–1 Annex C, BS 4449: 2005 and BS EN 10080[10].
2 The nomenclature used in BS 4449: 2005 differs from that used in BS EN 1992–1–1 Annex C and used here.
3 In accordance with BS 8666, class H may be specified, in which case class A, B or C may be supplied.

Table 6
Bending moment and shear co-efficients for beams

	Moment	Shear
Outer support	25% of span moment	0.45 $(G + Q)$
Near middle of end span	0.090 Gl + 0.100 Ql	
At first interior support	– 0.094 $(G + Q)l$	0.63 $(G + Q)^a$
At middle of interior spans	0.066 Gl + 0.086 Ql	
At interior supports	– 0.075 $(G + Q)l$	0.50 $(G + Q)$

Key
a 0.55 $(G + Q)$ may be used adjacent to the interior span.

Notes
1 Redistribution of support moments by 15% has been included.
2 Applicable to 3 or more spans only and where $Q_k \leq G_k$.
3 Minimum span ≥ 0.85 longest span.
4 l is the effective length, G is the total of the ULS permanent actions, Q is the total of the ULS variable actions.

Table 7
Exposure classes

Class	Description
No risk of corrosion or attack	
X0	For concrete without reinforcement or embedded metal where there is no significant freeze/thaw, abrasion or chemical attack.
Corrosion induced by carbonation	
XC1	Dry or permanently wet
XC2	Wet, rarely dry
XC3/4	Moderate humidity or cyclic wet and dry
Corrosion induced by chlorides other than from seawater	
XD1	Moderate humidity
XD2	Wet, rarely dry
XD3	Cyclic wet and dry
Corrosion induced by chlorides from seawater	
XS1	Exposed to airborne salt but not in direct contact with sea water
XS2	Permanently submerged
XS3	Tidal, splash and spray zones
Freeze/thaw with or without de-icing agents	
XF1	Moderate water saturation without de-icing agent
XF2	Moderate water saturation with de-icing agent
XF3	High water saturation without de-icing agent
XF4	High water saturation with de-icing agent or sea water
Chemical attack (ACEC classes)	
Refer to BS 8500–1 and Special Digest 1[11]	

Structural analysis

The primary purpose of structural analysis in building structures is to establish the distribution of internal forces and moments over the whole or part of a structure and to identify the critical design conditions at all sections. The geometry is commonly idealised by considering the structure to be made up of linear elements and plane two-dimensional elements.

The type of analysis should be appropriate to the problem being considered. The following may be used: linear elastic analysis, linear elastic analysis with limited redistribution, and plastic analysis. Linear elastic analysis may be carried out assuming cross sections are uncracked (i.e. concrete section properties); using linear stress-strain relationships, and assuming mean values of elastic modulus.

For the ultimate limit state only, the moments derived from elastic analysis may be redistributed (up to a maximum of 30%) provided that the resulting distribution of moments remains in equilibrium with the applied loads and subject to certain limits and design criteria (e.g. limitations of depth to neutral axis).

Regardless of the method of analysis used, the following principles apply:

- Where a beam or slab is monolithic with its supports, the critical design hogging moment may be taken as that at the face of the support, but should not be taken as less than 0.65 times the full fixed end moment.
- Where a beam or slab is continuous over a support that may be considered not to provide rotational restraint, the moment calculated at the centre line of the support may be reduced by $(F_{Ed,sup}\, t/8)$, where $F_{Ed,sup}$ is the support reaction and t is the breadth of the support.
- For the design of columns the elastic moments from the frame action should be used without any redistribution.

Bending moment and shear force co-efficients for beams are given in Table 6; these are suitable where spans are of similar length and the other notes to the table are observed.

Minimum concrete cover

The nominal cover can be assessed as follows:

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

Where c_{min} should be set to satisfy the requirements below:

- safe transmission of bond forces
- durability
- fire resistance

and Δc_{dev} is an allowance which should be made in the design for deviations from the minimum cover. It should be taken as 10 mm, unless fabrication (i.e. construction) is subjected to a quality assurance system, in which case it is permitted to reduce Δc_{dev} to 5 mm.

Figure 4
Sections through structural members, showing nominal axis distance, *a*

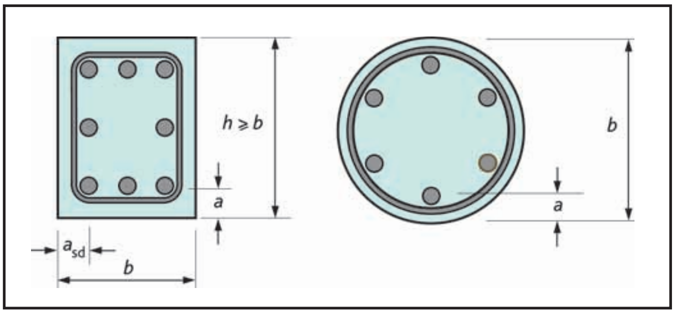


Table 9
Minimum column dimensions and axis distances for columns with rectangular or circular section – method A

Standard fire resistance	Minimum dimensions (mm) Column width (b_{min})/axis distance *(a)* of the main bars	
	Column exposed on more than one side (μ_{fi} = 0.7)	Exposed on one side (μ_{fi} = 0.7)
R 60	250/46 350/40	155/25
R 120	350/57* 450/51*	175/35
R 240	†	295/70

Notes
1 Refer to BS EN 1992–1–2 for design limitations.
2 μ_{fi} is the ratio of the design axial load under fire conditions to the design resistance of the column at normal temperature conditions. Conservatively μ_{fi} may be taken as 0.7
***** Minimum 8 bars
† Method B indicates 600/70 for R 240 and μ_{fi} = 0.7 and may be used. See EN 1992–1–2 Table 5.2b

Minimum cover for bond

The minimum cover to ensure adequate bond should not be less than the bar diameter, or equivalent bar diameter for bundled bars, unless the aggregate size is over 32 mm.

Minimum cover for durability

The recommendations for durability in Eurocode 2 are based on BS EN 206–1[12]. In the UK the requirements of BS EN 206 –1 are applied through the complementary standard BS 8500. The UK National Annex (Table 4.3 (N) (BS)) gives durability requirements that comply with BS 8500, but which significantly modify the approach taken in Eurocode 2. To determine the minimum cover for durability (and also the strength class and minimum water cement ratio) either the UK National Annex or BS 8500 can be used.

The various exposure classes from BS 8500 are given in Table 7. Selected recommendations are given in Table 8 (on page 6) for the concrete strength, minimum cement ratio, minimum concrete cover and maximum cement content for various elements in a structure based on the exposure of that element. This is taken from Chapter 11, originally published as *How to use BS 8500 with BS 8110*[13].

Design for fire resistance

Eurocode 2 Part 1–2: *Structural fire design*[14], gives several methods for determining the fire resistance of concrete elements; further guidance can be obtained from specialist literature. Design for fire resistance may still be carried out by referring to tables to determine the minimum cover and dimensions for various elements, as set out below.

Rather than giving the minimum cover, the tabular method is based on nominal axis distance, *a* (see Figure 4). This is the distance from the centre of the main reinforcing bar to the surface of the member. It is a nominal (not minimum) dimension. The designer should ensure that $a \geq c_{nom} + \phi_{link} + \phi_{bar}/2$.

There are three standard fire exposure conditions that may be satisfied:
R Mechanical resistance for load bearing
E Integrity of separation
I Insulation

Tables 9 and 10 give the minimum dimensions for columns and slabs to meet the above conditions. The tables offer more flexibility than BS 8110 in that there are options available to the designer e.g. section sizes can be reduced by increasing the axis distance. Further information is given in Eurocode 2 and subsequent chapters, including design limitations and data for walls and beams.

Table 10
Minimum dimensions and axis distances for reinforced concrete slabs

Standard fire resistance		Minimum dimensions (mm)							
		One-way spanning slab	Two-way spanning slab		Flat slab	Ribs in a two-way spanning ribbed slab (b_{min} is the width of the rib)			
			$l_y/l_x \leq 1.5$	$1.5 < l_y/l_x \leq 2$					
REI 60	h_s = a =	80 20	80 10	80 15	180 15	b_{min} = a =	100 25	120 15	≥200 10
REI 120	h_s = a =	120 40	120 20	120 25	200 35	b_{min} = a =	160 45	190 40	≥300 30
REI 240	h_s = a =	175 65	175 40	175 50	200 50	b_{min} = a =	450 70	700 60	___

Notes
1 Refer to BS EN 1992–1–2 for design limitations.
2 *a* is the axis distance (see Figure 4).
3 h_s is the slab thickness, including any non-combustible flooring.

Table 8

Selected[a] recommendations for normal-weight reinforced concrete quality for combined exposure classes and cover to reinforcement for at least a 50-year intended working life and 20 mm maximum aggregate size

Exposure conditions			Cement/ combination designations[b]	Strength class[c], maximum w/c ratio, minimum cement or combination content (kg/m³), and equivalent designated concrete (where applicable)							
Typical example	Primary	Secondary		Nominal cover to reinforcement[d]							
				$15 + \Delta c_{dev}$	$20 + \Delta c_{dev}$	$25 + \Delta c_{dev}$	$30 + \Delta c_{dev}$	$35 + \Delta c_{dev}$	$40 + \Delta c_{dev}$	$45 + \Delta c_{dev}$	$50 + \Delta c_{dev}$
Internal mass concrete	**X0**	—	All	Recommended that this exposure is not applied to reinforced concrete							
Internal elements (except humid locations)	**XC1**	—	All	C20/25, 0.70, 240 or RC20/25	<<<	<<<	<<<	<<<	<<<	<<<	<<<
Buried concrete in AC-1 ground conditions[e]	**XC2**	**AC-1**	All	—	—	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<	<<<	<<<
Vertical surface protected from direct rainfall	**XC3 & XC4**	—	All except IVB	—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<
Exposed vertical surfaces		**XF1**	All except IVB	—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	<<<	<<<	<<<	<<<
Exposed horizontal surfaces		**XF3**	All except IVB	—	C40/50,0.45, 340[g] or RC40/50XF[g]	<<<	<<<	<<<	<<<	<<<	<<<
		XF3 (air entrained)	All except IVB	—	—	C32/40, 0.55, 300 plus air[g,h]	C28/35, 0.60, 280 plus air[g,h] or PAV2	C25/30, 0.60, 280 plus air[g,h,j] or PAV1	<<<	<<<	<<<
Elements subject to airborne chlorides	**XD1**[f]	—	All	—	—	C40/50, 0.45, 360	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<	<<<	<<<
Car park decks and areas subject to de-icing spray	**XD3**[f]	—	IIB-V, IIIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340
			CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
			IIIB, IVB-V	—	—	—	—	—	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340
Vertical elements subject to de-icing spray and freezing		**XF2**	IIB-V, IIIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C32/40, 0.50, 340
			CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
			IIIB, IVB-V	—	—	—	—	—	C32/40, 0.40, 380	C32/40 0.45, 360	C32/40, 0.50, 340
Car park decks, ramps and external areas subject to freezing and de-icing salts		**XF4**	CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380[g]	<<<
		XF4 (air entrained)	IIB-V, IIIA, IIIB	—	—	—	—	—	C28/35, 0.40, 380[g,h]	C28/35 0.45, 360[g,h]	C28/35, 0.50, 340[g,h]
Exposed vertical surfaces near coast	**XS1**[f]	**XF1**	CEM I, IIA, IIB-S, SRPC	—	—	—	See BS 8500	C35/45, 0.45, 360	C32/40, 0.50, 340	<<<	<<<
			IIB-V, IIIA	—	—	—	See BS 8500	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<
			IIIB	—	—	—	C32/40, 0.40, 380	C25/30, 0.50, 340	C25/30, 0.50, 340	C25/30, 0.55, 320	<<<
Exposed horizontal surfaces near coast		**XF4**	CEM I, IIA, IIB-S, SRPC	—	—	—	See BS 8500	C40/50, 0.45, 360[g]	<<<	<<<	<<<

Key

a This table comprises a selection of common exposure class combinations. Requirements for other sets of exposure classes, e.g. XD2, XS2 and XS3 should be derived from BS 8500-1: 2002, Annex A.

b See BS 8500-2, Table 1. (CEM I is Portland cement, IIA to IVB are cement combinations.)

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

d Δc_{dev} is an allowance for deviations.

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

f Also adequate for exposure class XC3/4.

g Freeze/thaw resisting aggregates should be specified.

h Air entrained concrete is required.

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

— Not recommended

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

Stability and imperfections

The effects of geometric imperfections should be considered in combination with the effects of wind loads (i.e. not as an alternative load combination). For global analysis, the imperfections may be represented by an inclination θ_i.

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

where

$\alpha_h = (2/\sqrt{l})$, to be taken as not less than 2/3 nor greater than 1.0

$\alpha_m = [0.5\ (1 + 1/m)]^{0.5}$

l is the height of the building in metres

m is the number of vertical members contributing to the horizontal force in the bracing system.

The effect of the inclination may be represented by transverse forces at each level and included in the analysis along with other actions (see Figure 5):

Effect on bracing system: $H_i = \theta_i\,(N_b - N_a)$

Effect on floor diaphragm: $H_i = \theta_i\,(N_b + N_a)/2$

Effect on roof diaphragm: $H_i = \theta_i\,N_a$

where N_a and N_b are longitudinal forces contributing to H_i.

In most cases, an allowance for imperfections is made in the partial factors used in the design of elements. However for columns, the effect of imperfections, which is similar in principle to the above, must be considered (see Chapter 5, originally published as *Columns*[15]).

Table 11

Maximum bar size or spacing to limit crack width

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

Note

The steel stress may be estimated from the expression below (or see Figure 6):

$$\sigma_s = \frac{f_{yk}\, m\, A_{s,req}}{\gamma_{ms}\, n\, A_{s,prov}\, \delta}$$

where

f_{yk} = characteristic reinforcement yield stress

γ_{ms} = partial factor for reinforcing steel

m = total load from quasi-permanent combination

n = total load from ULS combination

$A_{s,req}$ = area of reinforcement at the ULS

$A_{s,prov}$ = area of reinforcement provided

δ = ratio of redistributed moment to elastic moment

Crack control

Crack widths should be limited to ensure appearance and durability are satisfactory. In the absence of specific durability requirements (e.g. water tightness) the crack widths may be limited to 0.3 mm in all exposure classes under the quasi-permanent combination. In the absence of requirements for appearance, this limit may be relaxed (to say 0.4 mm) for exposure classes X0 and XC1 (refer to Table 7). The theoretical size of the crack can be calculated using the expressions given in Cl 7.3.4 from Eurocode 2–1–1 or from the 'deemed to satisfy' requirements that can be obtained from Table 11, which is based on tables 7.2N and 7.3N of the Eurocode. The limits apply to either the bar size or the bar spacing, not both.

Figure 5

Examples of the effect of geometric imperfections

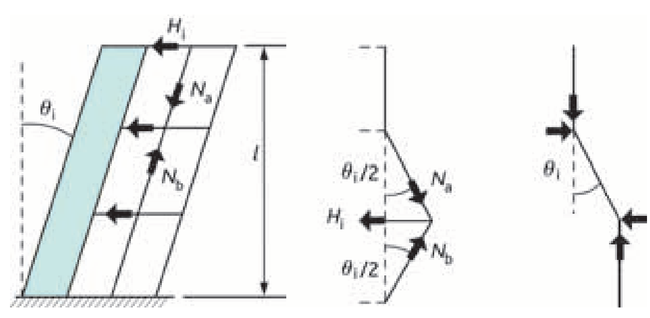
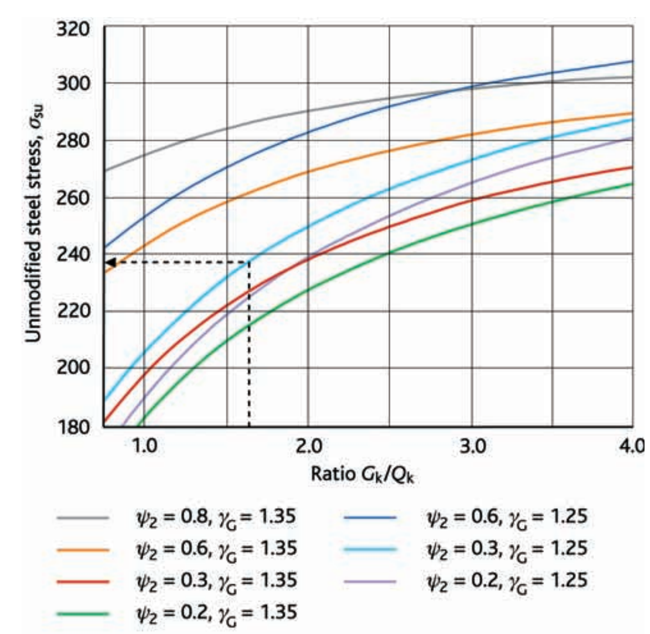


Figure 6

Determination of steel stress for crack width control



To determine stress in the reinforcement (σ_s), calculate the ratio G_k/Q_k, read up the graph to the appropriate curve and read across to determine σ_{su}.

σ_s can be calculated from the expression: $\sigma_s = \sigma_{su}\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$

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3. Slabs

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Designing to Eurocode 2

This chapter covers the analysis and design of slabs to Eurocode 2[1] which is essentially the same as with BS 8110[2]. However, the layout and content of Eurocode 2 may appear unusual to designers familiar with BS 8110. Eurocode 2 does not contain the derived formulae or specific guidance on determining moments and shear forces. This has arisen because it has been European practice to give principles in the codes and for the detailed application to be presented in other sources such as textbooks.

Chapter 1, originally published as *Introduction to Eurocodes*[3], highlighted the key differences between Eurocode 2 and BS 8110, including terminology. Chapter 7, originally published as *Flat slabs*[4] covers the design of flat slabs.

It should be noted that values from the UK National Annex (NA) have been used throughout, including values that are embedded in derived formulae. (Derivations can be found at www.eurocode2.info.) A list of symbols related to slab design is given at the end of this chapter.

Design procedure

A procedure for carrying out the detailed design of slabs is shown in Table 1. This assumes that the slab thickness has previously been determined during conceptual design. More detailed advice on determining design life, actions, material properties, methods of analysis, minimum concrete cover for durability and control of crack widths can be found in Chapter 2, originally published as *Getting started*[5].

Fire resistance

Eurocode 2, Part 1–2: *Structural fire design*[6], gives a choice of advanced, simplified or tabular methods for determining the fire resistance. Using tables is the fastest method for determining the minimum dimensions and cover for slabs. There are, however, some restrictions which should be adhered to. Further guidance on the advanced and simplified methods can be obtained from specialist literature.

Rather than giving a minimum cover, the tabular method is based on nominal axis distance, *a*. This is the distance from the centre of the main reinforcing bar to the surface of the member. It is a nominal (not minimum)

Continues page 19

Table 1
Slab design procedure

Step	Task	Further guidance	
		Chapter in this publication	Standard
1	Determine design life	2: *Getting started*	NA to BS EN 1990 Table NA.2.1
2	Assess actions on the slab	2: *Getting started*	BS EN 1991 (10 parts) and National Annexes
3	Determine which combinations of actions apply	1: *Introduction to Eurocodes*	NA to BS EN 1990 Tables NA.A1.1 and NA.A1.2 (B)
4	Determine loading arrangements	2: *Getting started*	NA to BS EN 1992–1–1
5	Assess durability requirements and determine concrete strength	2: *Getting started*	BS 8500: 2002
6	Check cover requirements for appropriate fire resistance period	2: *Getting started* and Table 2	Approved Document B. BS EN 1992–1–2: Section 5
7	Calculate min. cover for durability, fire and bond requirements	2: *Getting started*	BS EN 1992–1–1 Cl 4.4.1
8	Analyse structure to obtain critical moments and shear forces	2: *Getting started* and Table 3	BS EN 1992–1–1 section 5
9	Design flexural reinforcement	See Figure 1	BS EN 1992–1–1 section 6.1
10	Check deflection	See Figure 3	BS EN 1992–1–1 section 7.4
11	Check shear capacity	See Table 7	BS EN 1992–1–1 section 6.2
12	Check spacing of bars	2: *Getting started*	BS EN 1992–1–1 section 7.3

Note
NA = National Annex.

Table 2
Minimum dimensions and axis distances for reinforced concrete slabs (excluding flat slabs)

Standard fire resistance		Minimum dimensions (mm)						
		One-way[a,b] spanning slab	Two-way spanning slab[a,b,c,d]		Ribs in a two-way spanning ribbed slab[e]			
			$l_y / l_x \leq 1.5$[f]	$1.5 < l_y / l_x \leq 2$[f]				
REI 60	h_s =	80	80	80	b_{min} =	100	120	≥200
	a =	20	10[g]	15[g]	a =	25	15[g]	10[g]
REI 90	h_s =	100	100	100	b_{min} =	120	160	≥250
	a =	30	15[g]	20	a =	35	25	15[g]
REI 120	h_s =	120	120	120	b_{min} =	160	190	≥300
	a =	40	20	25	a =	45	40	30
REI 240	h_s =	175	175	175	b_{min} =	450	700	___
	a =	65	40	50	a =	70	60	

Notes

1. This table is taken from BS EN 1992–1–2 Tables 5.8 to 5.11. For flat slabs refer to Chapter 7.
2. The table is valid only if the detailing requirements (see note 3) are observed and in normal temperature design redistribution of bending moments does not exceed 15%.
3. For fire resistance of R90 and above, for a distance of $0.3 l_{eff}$ from the centre line of each intermediate support, the area of top reinforcement should not be less than the following:

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

   where:

   x is the distance of the section being considered from the centre line of the support.

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

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

   l_{eff} is the greater of the effective lengths of the two adjacent spans.
4. There are three standard fire exposure conditions that need to be satisfied:
   R Mechanical resistance for load bearing
   E Integrity of separation
   I Insulation
5. The ribs in a one-way spanning ribbed slab can be treated as beams and reference can be made to Chapter 4, *Beams*. The topping can be treated as a two-way slab where $1.5 < l_y / l_x \leq 2$.

Key

a The slab thickness h_s is the sum of the slab thickness and the thickness of any non-combustible flooring.

b For continuous solid slabs a minimum negative reinforcement $A_s \geq 0.005\, A_c$ should be provided over intermediate supports if

1) cold worked reinforcement is used; or

2) there is no fixity over the end supports in a two span slab; or

3) where transverse redistribution of load effects cannot be achieved.

c In two way slabs the axis refers to the lower layer of reinforcement.

d The term two way slabs relates to slabs supported at all four edges. If this is not the case, they should be treated as one-way spanning slabs.

e For two-way ribbed slabs the following notes apply:

The axis distance measured to the lateral surface of the rib should be at least (a + 10).

The values apply where there is predominantly uniformly distributed loading.

There should be at least one restrained edge.

The top reinforcement should be placed in the upper half of the flange.

f l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer span.

g Normally the requirements of BS EN 1992–1–1 will determine the cover.

Figure 1
Procedure for determining flexural reinforcement

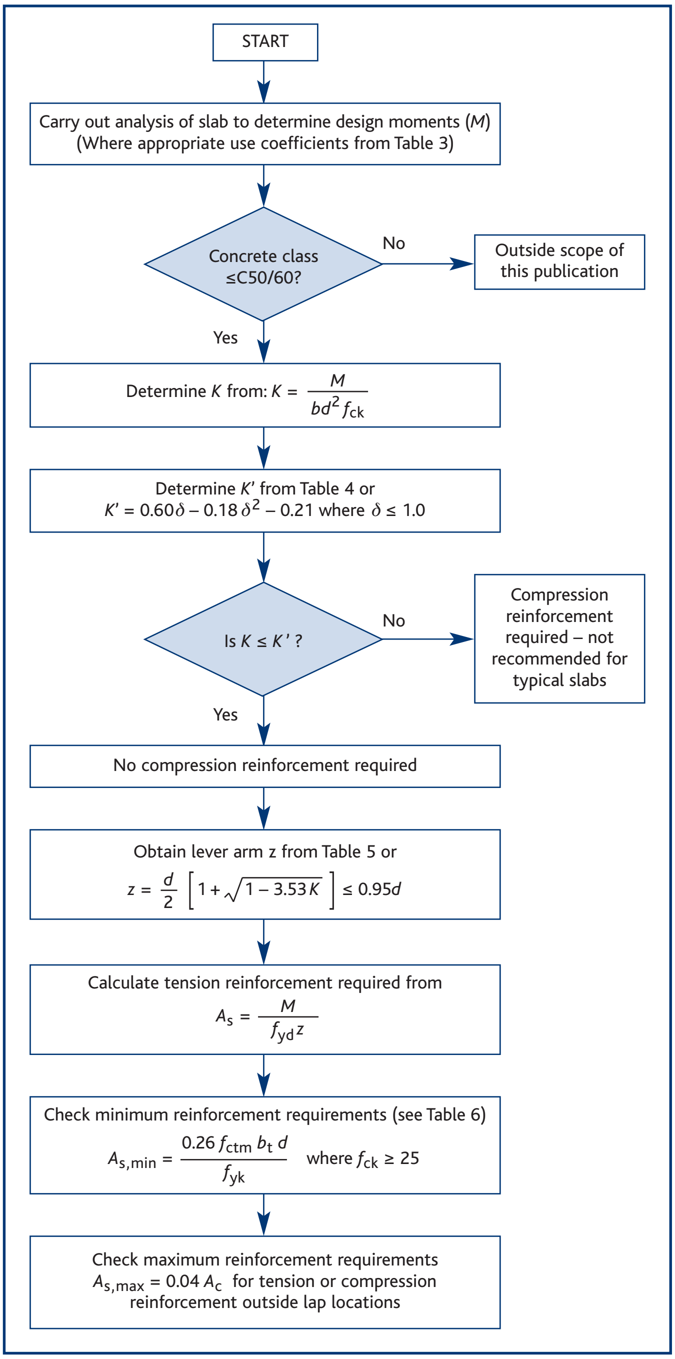


Table 3
Bending moment and shear coefficients for slabs

	End support / slab connection				First interior support	Interior spans	Interior supports
	Pinned		Continuous				
	End support	End span	End support	End span			
Moment	0	$0.086Fl$	$-0.04Fl$	$0.075Fl$	$-0.086Fl$	$0.063Fl$	$-0.063Fl$
Shear	$0.40F$		$0.46F$		$0.6F$		$0.5F$

Notes
1 Applicable to one-way spanning slabs where the area of each bay exceeds 30 m^2, $Q_k \leq 1.25\,G_k$ and $q_k \leq 5$ kN/m^2
2 F is the total design ultimate load, l is the span
3 Minimum span > 0.85 longest span, minimum 3 spans
4 Based on 20% redistribution at supports and no decrease in span moments

dimension, so the designer should ensure that

$a \geq c_{nom} + \phi_{link} + \phi_{bar}/2$.

The requirements for various types of slab are given in Table 2.

Flexure

The design procedure for flexural design is given in Figure 1; this includes derived formulae based on the simplified rectangular stress block from Eurocode 2. Where appropriate, Table 3 may be used to determine bending moments and shear forces for slabs. Further information for the design of two-way, ribbed or waffle slabs is given in the appropriate sections on pages 5 and 6.

Table 4
Values for K'

% redistribution	δ (redistribution ratio)	K'
0	1.00	0.208[a]
10	0.90	0.182[a]
15	0.85	0.168
20	0.80	0.153
25	0.75	0.137
30	0.70	0.120

Key
a It is often recomended in the UK that K' should be limited to 0.168 to ensure ductile failure.

Table 5
z/d for singly reinforced rectangular sections

K	z/d	K	z/d
≤0.05	0.950[a]	0.13	0.868
0.06	0.944	0.14	0.856
0.07	0.934	0.15	0.843
0.08	0.924	0.16	0.830
0.09	0.913	0.17	0.816
0.10	0.902	0.18	0.802
0.11	0.891	0.19	0.787
0.12	0.880	0.20	0.771

Key
a Limiting z to $0.95d$ is not a requirement of Eurocode 2, but is considered to be good practice.

Table 6
Minimum percentage of reinforcement required

f_{ck}	f_{ctm}	Minimum % ($0.26 f_{ctm}/f_{yk}$[a])
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%

Key
a Where f_{yk} = 500 MPa.

Eurocode 2 offers various methods for determining the stress-strain relationship of concrete. For simplicity and familiarity the method presented here is the simplified rectangular stress block, which is similar to that found in BS 8110 (see Figure 2).

The Eurocode gives recommendations for the design of concrete up to class C90/105. However, for concrete greater than class C50/60, the stress block is modified. It is important to note that concrete strength is based on the cylinder strength and not the cube strength (i.e. for class C28/35 the cylinder strength is 28 MPa, whereas the cube strength is 35 MPa).

Deflection

Eurocode 2 has two alternative methods of designing for deflection, either by limiting span-to-depth ratio or by assessing the theoretical deflection using the Expressions given in the Eurocode. The latter is dealt with in detail in Chapter 8, originally published as *Deflection calculations*[7].

The span-to-depth ratios should ensure that deflection is limited to span /250 and this is the procedure presented in Figure 3.

Figure 2
Simplified rectangular stress block for concrete up to class C50/60 from Eurocode 2

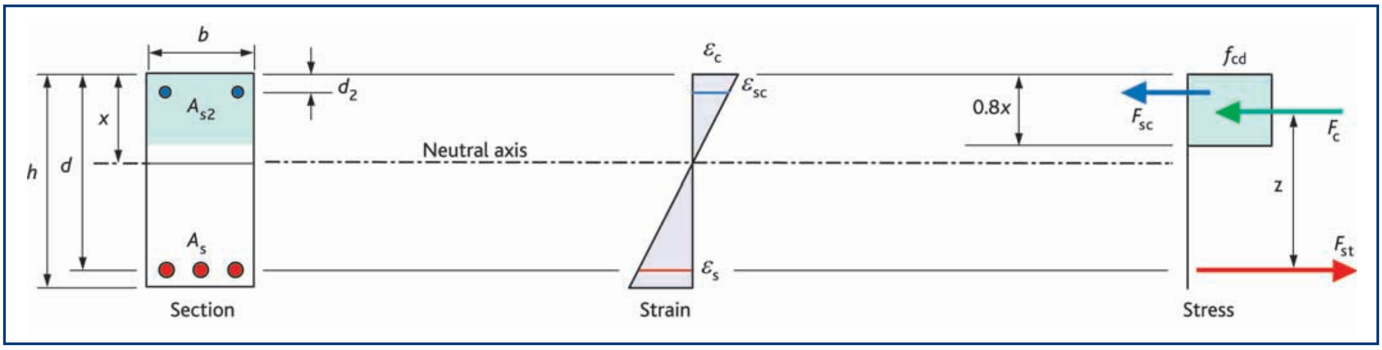
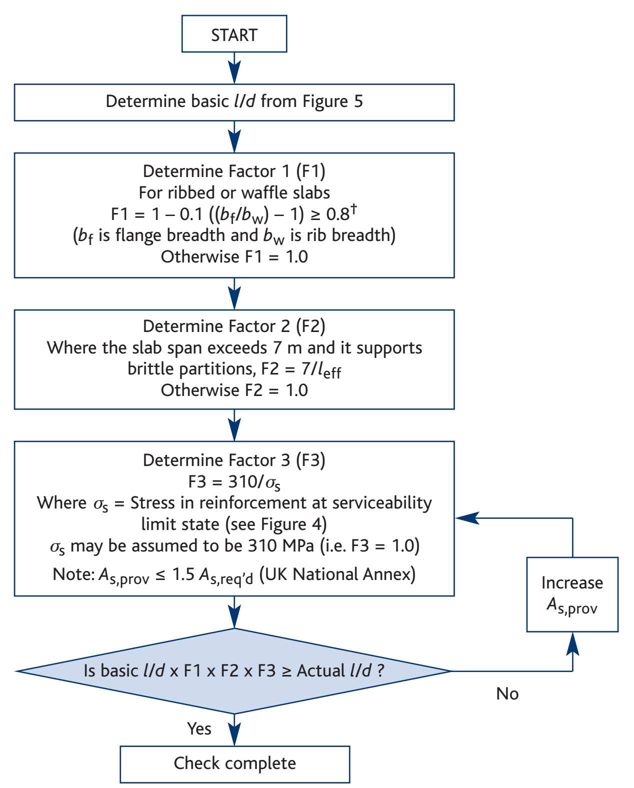
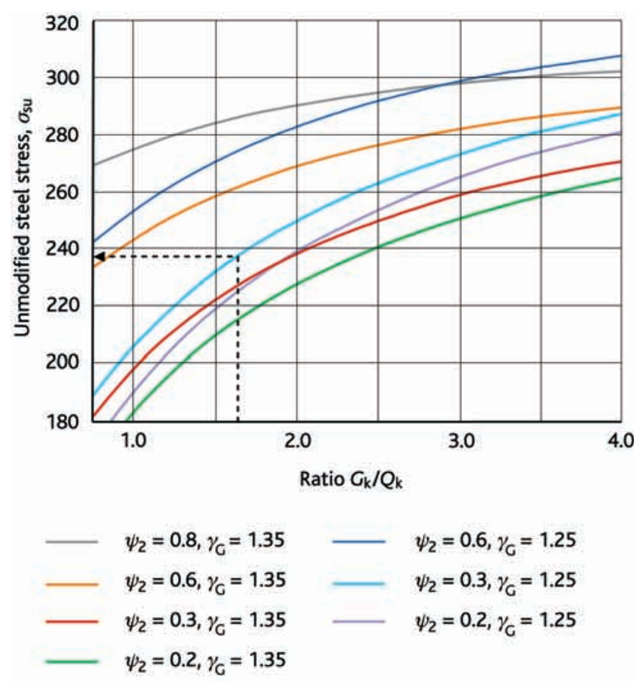


Figure 3
Procedure for assessing deflection



† The Eurocode is ambiguous regarding linear interpolation. It is understood that this was the intention of the drafting committee and is in line with current UK practice.

Figure 4
Determination of steel stress



To determine stress in the reinforcement (σ_s), calculate the ratio G_k/Q_k, read up the graph to the appropriate curve and read across to determine σ_{su}.

σ_s can be calculated from the expression: $\sigma_s = \sigma_{su}\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$

Design for shear

It is not usual for a slab to contain shear reinforcement, therefore it is only necessary to ensure that the concrete shear stress capacity without shear reinforcement ($v_{Rd,c}$ – see Table 7) is more than applied shear stress ($v_{Ed} = V_{Ed}/(bd)$). Where shear reinforcement is required, e.g. for ribs in a ribbed slab, refer to Chapter 4, originally published as *Beams*[8].

Two-way slabs

Unlike BS 8110 there is no specific guidance given in Eurocode 2 on how to determine the bending moments for a two-way slab. The assessment of the bending moment can be carried out using any suitable method from Section 5 of the Code. However, co-efficients may be obtained from Table 8 (taken from the *Manual for the design of building structures to Eurocode 2*[9]) to determine bending moments per unit width (M_{sx} and M_{sy}) where:

$$M_{sx} = \beta_{sx}\, w\, l_x^2$$

$$M_{sy} = \beta_{sy}\, w\, l_x^2$$

Where β_{sx} and β_{sy} are coefficients, l_x is the shorter span and w (load per unit area) is the STR ultimate limit state combination. For more information on combinations refer toChapter 1, originally published as *Introduction to Eurocodes*[3].

Table 7
$v_{Rd,c}$ resistance of members without shear reinforcement, MPa

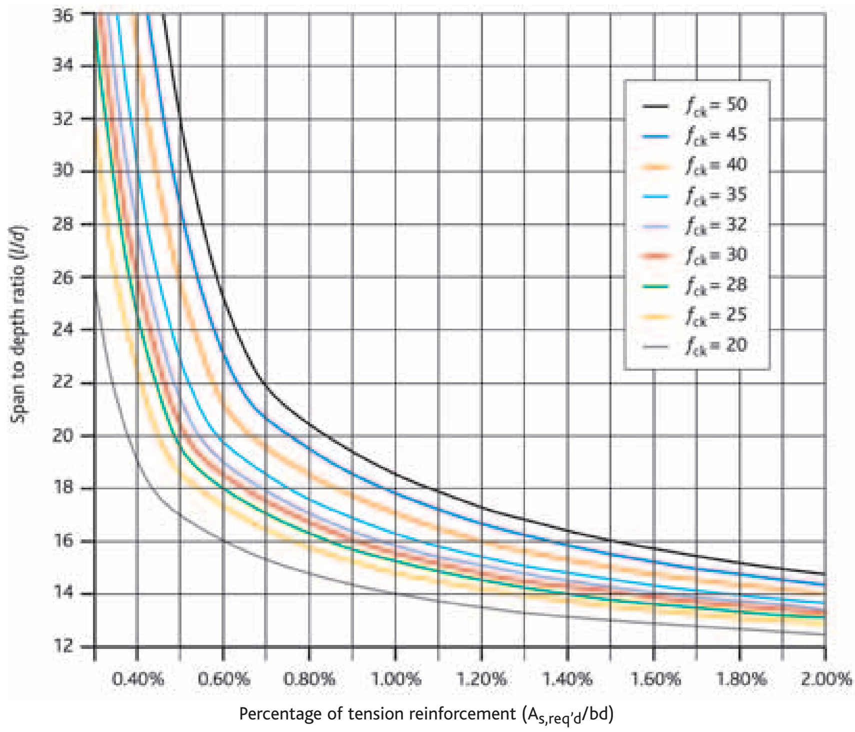
$\rho_l = A_s/(bd)$	Effective depth, d (mm)										
	≤200	225	250	275	300	350	400	450	500	600	750
0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71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

Table derived from: $v_{Rd,c} = 0.12\, k\, (100\rho_l\, f_{ck})^{1/3} \geq 0.035\, k^{1.5}\, f_{ck}^{0.5}$
where $k = 1 + \sqrt{(200/d)} \leq 2$ and $\rho_l = A_s/(bd) \leq 0.02$

Note
1. This table has been prepared for $f_{ck} = 30$.
2. Where ρ_l exceeds 0.40% the following factors may be used:

f_{ck}	25	28	32	35	40	45	50
Factor	0.94	0.98	1.02	1.05	1.10	1.14	1.19

Figure 5
Basic span-to-effective-depth ratios



Notes
1. For two-way spanning slabs, the check should be carried out on the basis of the shorter span.
2. This graph assumes simply supported span condition (K = 1.0).
   K = 1.5 for interior span condition
   K = 1.3 for end span condition
   K = 0.4 for cantilevers
3. Compression reinforcement, ρ', has been taken as 0.
4. Curves based on the following expressions:

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{1.5}\right]$$

where $\rho \leq \rho_0$

and

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{(\rho - \rho')} + \frac{\sqrt{f_{ck}}}{12}\sqrt{\frac{\rho'}{\rho_0}}\right]$$

where $\rho > \rho_0$.

Ribbed or waffle slabs

Current practices for determining forces in ribbed and waffle slabs may also be used for designs to Eurocode 2. Where a waffle slab is treated as a two-way slab refer to previous section, but note that their torsional stiffness is significantly less than for a two-way slab and the bending moment co-efficients may not be applicable. Where it is treated as a flat slab reference may be made to Chapter 7, originally published as *Flat slabs*[4]

The position of the neutral axis in the rib should be determined, and then the area of reinforcement can be calculated depending on whether it lies in the flange or web (see flow chart in Figure 6). The main differences compared with BS 8110 are that the assessment of the flange width is more sophisticated (see Figures 7 and 8).

Where a slab is formed with permanent blocks or a with a topping thickness less than 50 mm and one-tenth of the clear distance between ribs it is recommended that a longitudinal shear check is carried out to determine whether additional transverse reinforcement is required (see BS EN 1992–1–1, Cl 6.2.4).

Table 8

Bending moment coefficients for two-way spanning rectangular slabs supported by beams

Type or panel and moments considered	Short span coefficients for values of l_y/l_x					Long-span coefficients for all values of l_y/l_x
	1.0	1.25	1.5	1.75	2.0	
Interior panels						
Negative moment at continuous edge	0.031	0.044	0.053	0.059	0.063	0.032
Positive moment at midspan	0.024	0.034	0.040	0.044	0.048	0.024
One short edge discontinuous						
Negative moment at continuous edge	0.039	0.050	0.058	0.063	0.067	0.037
Positive moment at midspan	0.029	0.038	0.043	0.047	0.050	0.028
One long edge discontinuous						
Negative moment at continuous edge	0.039	0.059	0.073	0.083	0.089	0.037
Positive moment at midspan	0.030	0.045	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous						
Negative moment at continuous edge	0.047	0.066	0.078	0.087	0.093	0.045
Positive moment at midspan	0.036	0.049	0.059	0.065	0.070	0.034

Figure 6

Procedure for determining flexural capacity of flanged ribs

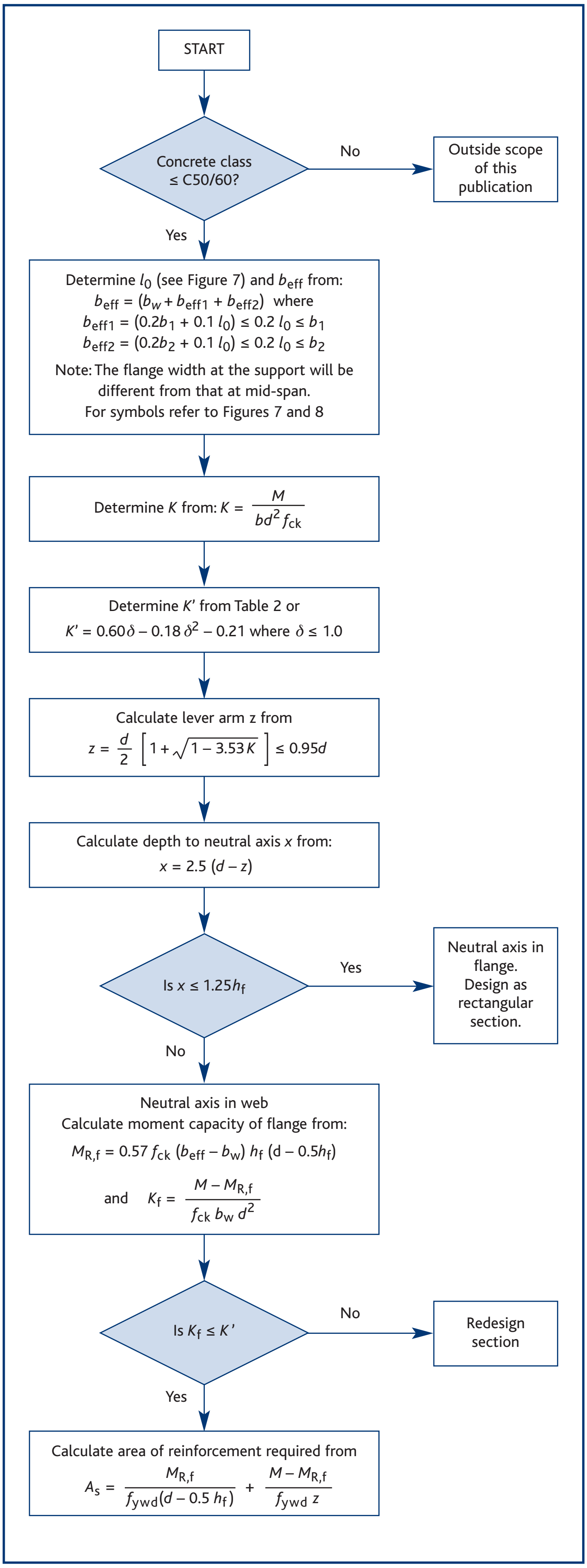


Figure 7
Definition of l_0, for calculation of effective flange width

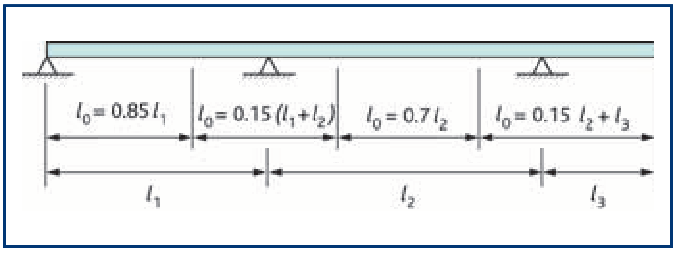
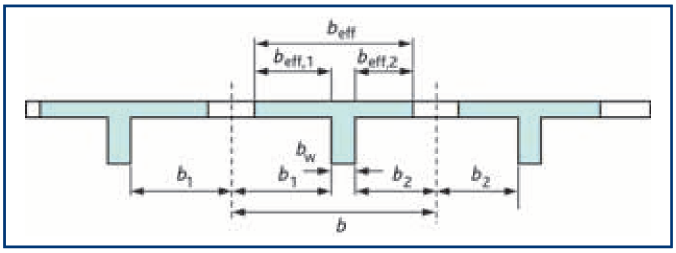


Figure 8
Effective flange width parameters



Rules for spacing and quantity of reinforcement

Minimum area of principal reinforcement

The minimum area of principal reinforcement in the main direction is $A_{s,min} = 0.26\, f_{ctm}\, b_t\, d/f_{yk}$ but not less than $0.0013 b_t d$, where b_t is the mean width of the tension zone (see Table 6). For a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value of b_t.

Minimum area of secondary reinforcement

The minimum area of secondary transverse reinforcement is 20% $A_{s,min}$. In areas near supports, transverse reinforcement is not necessary where there is no transverse bending moment.

Maximum area of reinforcement

Outside lap locations, the maximum area of tension or compression reinforcement should not exceed $A_{s,max} = 0.04\, A_c$

Minimum spacing of reinforcement

The minimum clear distance between bars should be the greater of:

- Bar diameter
- Aggregate size plus 5 mm
- 20 mm

Maximum spacing of reinforcement

For slabs less than 200 mm thick the following maximum spacing rules apply:

- For the principal reinforcement: 3*h* but not more than 400 mm
- For the secondary reinforcement: 3.5*h* but not more than 450 mm

The exception is in areas with concentrated loads or areas of maximum moment where the following applies:

- For the principal reinforcement: 2*h* but not more than 250 mm
- For the secondary reinforcement: 3*h* but not more than 400 mm

Where *h* is the depth of the slab.

For slabs 200 mm thick or greater the bar size and spacing should be limited to control the crack width and reference should be made to section 7.3.3 of the Code or Chapter 2, originally published as *Getting started*[5].

Selected symbols

Symbol	Definition	Value
A_c	Cross sectional area of concrete	bh
A_s	Area of tension steel	
A_{s2}	Area of compression steel	
$A_{s,\,prov}$	Area of tension steel provided	
$A_{s,\,req'd}$	Area of tension steel required	
b_{eff}	Effective flange width	
b_t	Mean width of the tension zone	
b_{min}	Width of beam or rib	
b_w	Width of rib web	
d	Effective depth	
d_2	Effective depth to compression reinforcement	
f_{cd}	Design value of concrete compressive strength	$\alpha_{cc}\, f_{ck}/\gamma_c$
f_{ck}	Characteristic cylinder strength of concrete	
f_{ctm}	Mean value of axial tensile strength	$0.30\, f_{ck}^{2/3}$ for $f_{ck} \leq$ C50/60 (from Table 3.1, Eurocode 2)
h_f	Flange thickness	
h_s	Slab thickness	
K	Factor to take account of the different structural systems	See Table NA.4 in UK National Annex
l_{eff}	Effective span of member	See Section 5.3.2.2 (1)
l_0	Distance between points of zero moment	
l/d	Limiting span-to-depth ratio	
l_x, l_y	Spans of a two-way slab	
M	Design moment at the ULS	
x	Depth to neutral axis	$(d - z)/0.4$
x_{max}	Limiting value for depth to neutral axis	$(\delta - 0.4)d$ where $\delta \leq 1.0$
z	Lever arm	
α_{cc}	Coefficient taking account of long term effects on compressive strength and of unfavourable effects resulting from the way load is applied	0.85 for flexure and axial loads. 1.0 for other phenomena (From UK National Annex)
δ	Ratio of the redistributed moment to the elastic bending moment	
γ_m	Partial factor for material properties	1.15 for reinforcement (γ_s) 1.5 for concrete (γ_c)
ρ_0	Reference reinforcement ratio	$\sqrt{f_{ck}}/1000$
ρ	Required tension reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_s/bd
ρ'	Required compression reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_{s2}/bd

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4. Beams

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Designing to Eurocode 2

This chapter covers the analysis and design of concrete beams to Eurocode 2[1] which is essentially the same as with BS 8110[2]. However, the layout and content of Eurocode 2 may appear unusual to designers familiar with BS 8110. Eurocode 2 does not contain the derived formulae or specific guidance on determining moments and shear forces. This has arisen because it has been European practice to give principles in the codes and for the detailed application to be presented in other sources such as textbooks.

Chapter 1, originally published as *Introduction to Eurocodes*[3], highlighted the key differences between Eurocode 2 and BS 8110, including terminology.

It should be noted that values from the UK National Annex (NA) have been used throughout, including values that are embedded in derived formulae (derivations can be found at www.eurocode2.info). A list of symbols related to beam design is given at the end of this chapter.

Design procedure

A procedure for carrying out the detailed design of beams is shown in Table 1. This assumes that the beam dimensions have previously been determined during conceptual design. Concept designs prepared assuming detailed design would be to BS 8110 may be continued through to detailed design using Eurocode 2. More detailed advice on determining design life, actions, material properties, methods of analysis, minimum concrete cover for durability and control of crack widths can be found in Chapter 2, originally published as *Getting started*[4], and in Chapter 1.

Fire resistance

Eurocode 2, Part 1–2: *Structural fire design*[5], gives a choice of advanced, simplified or tabular methods for determining the fire resistance. Using tables is the fastest method for determining the minimum dimensions and cover for beams. There are, however, some restrictions and if these apply further guidance on the advanced and simplified methods can be obtained from specialist literature[6]. Rather than giving a minimum cover, the tabular method is based on nominal axis distance, *a* (see Figure 1). This is the distance from the centre of the main reinforcing bar to the top or bottom surface of the

Continues page 27

Table 1
Beam design procedure

Step	Task	Further guidance	
		Chapter in this publication	Standard
1	Determine design life	2: *Getting started*	NA to BS EN 1990 Table NA.2.1
2	Assess actions on the beam	2: *Getting started*	BS EN 1991 (10 parts) and National Annexes
3	Determine which combinations of actions apply	1: *Introduction to Eurocodes*	NA to BS EN 1990 Tables NA.A1.1 and NA.A1.2 (B)
4	Determine loading arrangements	2: *Getting started*	NA to BS EN 1992–1–1
5	Assess durability requirements and determine concrete strength	2: *Getting started*	BS 8500: 2002
6	Check cover requirements for appropriate fire resistance period	2: *Getting started* and 'Fire resistance' section	Approved Document B. BS EN 1992–1–1: Section 5
7	Calculate min. cover for durability, fire and bond requirements	2: *Getting started*	BS EN 1992–1–1 Cl 4.4.1
8	Analyse structure to obtain critical moments and shear forces	2: *Getting started* and Table 3	BS EN 1992–1–1 section 5
9	Design flexural reinforcement	See 'Flexure' section	BS EN 1992–1–1 section 6.1
10	Check shear capacity	See 'Vertical shear' section	BS EN 1992–1–1 section 6.2
11	Check deflection	See 'Deflection' section	BS EN 1992–1–1 section 7.4
12	Check spacing of bars	2: *Getting started*	BS EN 1992–1–1 section 7.3

Note
NA = National Annex

Table 2
Minimum dimensions and axis distances for beams made with reinforced concrete for fire resistance

Standard fire resistance		Minimum dimensions (mm)							
		Possible combinations of *a* and b_{min} where *a* is the average axis distance and b_{min} is the width of the beam							
		Simply supported beams				Continuous beams			
		A	B	C	D	E	F	G	H
R60	b_{min} = a =	120 40	160 35	200 30	300 25	120 25	200 12[a]		
R90	b_{min} = a =	150 55	200 45	300 40	400 35	150 35	250 25		
R120	b_{min} = a =	200 65	240 60	300 55	500 50	200 45	300 35	450 35	500 30
R240	b_{min} = a =	280 90	350 80	500 75	700 70	280 75	500 60	650 60	700 50

Notes
1 This table is taken from BS EN 1992–1–2 Tables 5.5 and 5.6.
2 The axis distance, a_{sd}, from the side of the beam to the corner bar should be a +10 mm except where b_{min} is greater than the values in columns C and F.
3 The table is valid only if the detailing requirements (see note 4) are observed and, in normal temperature design, redistribution of bending moments does not exceed 15%.
4 For fire resistance of R90 and above, for a distance of $0.3l_{eff}$ from the centre line of each intermediate support, the area of top reinforcement should not be less than the following:

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

where:

- x is the distance of the section being considered from the centre line of the support.
- $A_{s,req}(0)$ is the area of reinforcement required for normal temperature design.
- $A_{s,req}(x)$ is the minimum area of reinforcement required at the section being considered but not less than that required for normal temperature design.
- l_{eff} is the greater of the effective lengths of the two adjacent spans.

5 For fire resistances R120 – R240, the width of the beam at the first intermediate support should be at least that in column F, if both the following conditions exist:
a there is no fixity at the end support; and
b the acting shear at normal temperature $V_{sd} > 0.67\ V_{Rd,max}$.

Key
a Normally the requirements of BS EN 1992–1–1 will determine the cover.

Figure 1
Section through structural member, showing nominal axis distances *a* and a_{sd}

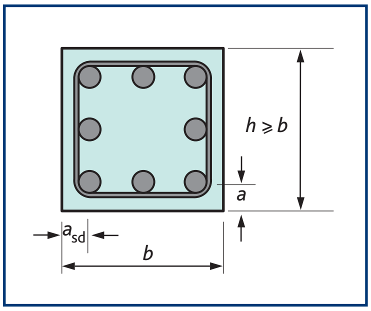
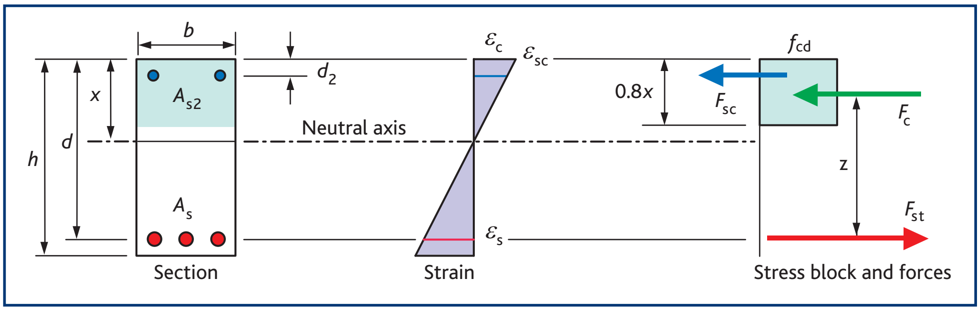


Figure 3
Simplified rectangular stress block for concrete up to class C50/60 from Eurocode 2



member. It is a nominal (not minimum) dimension, so the designer should ensure that:

$a \geq c_{nom} + \phi_{link} + \phi_{bar}/2$ and $a_{sd} = a + 10$ mm

Table 2 gives the minimum dimensions for beams to meet the standard fire periods.

Flexure

The design procedure for flexural design is given in Figure 2; this includes derived formulae based on the simplified rectangular stress block from Eurocode 2. Table 3 may be used to determine bending moments and shear forces for beams, provided the notes to the table are observed.

Table 3
Bending moment and shear coefficients for beams

	Moment	Shear
Outer support	25% of span moment	0.45 $(G + Q)$
Near middle of end span	0.090 Gl + 0.100 Ql	
At first interior support	– 0.094 $(G + Q)l$	0.63 $(G + Q)$[a]
At middle of interior spans	0.066 Gl + 0.086 Ql	
At interior supports	– 0.075 $(G + Q)l$	0.50 $(G + Q)$

Key
a 0.55 $(G + Q)$ may be used adjacent to the interior span.

Notes
1 Redistribution of support moments by 15% has been included.
2 Applicable to 3 or more spans only and where $Q_k \leq G_k$.
3 Minimum span ≥ 0.85 longest span.
4 l is the span, G is the total of the ULS permanent actions, Q is the total of the ULS variable actions.

Table 4
Values for K'

% redistribution	δ (redistribution ratio)	K'
0	1.00	0.208[a]
10	0.90	0.182[a]
15	0.85	0.168
20	0.80	0.153
25	0.75	0.137
30	0.70	0.120

Key
a It is often recomended in the UK that K' should be limited to 0.168 to ensure ductile failure.

Table 5
z/d for singly reinforced rectangular sections

K	z/d	K	z/d
≤0.05	0 950[a]	0.13	0.868
0.06	0.944	0.14	0.856
0.07	0.934	0.15	0.843
0.08	0.924	0.16	0.830
0.09	0.913	0.17	0.816
0.10	0.902	0.18	0.802
0.11	0.891	0.19	0.787
0.12	0.880	0.20	0.771

Key
a Limiting z to 0.95d is not a requirement of Eurocode 2, but is considered to be good practice.

Figure 2
Procedure for determining flexural reinforcement

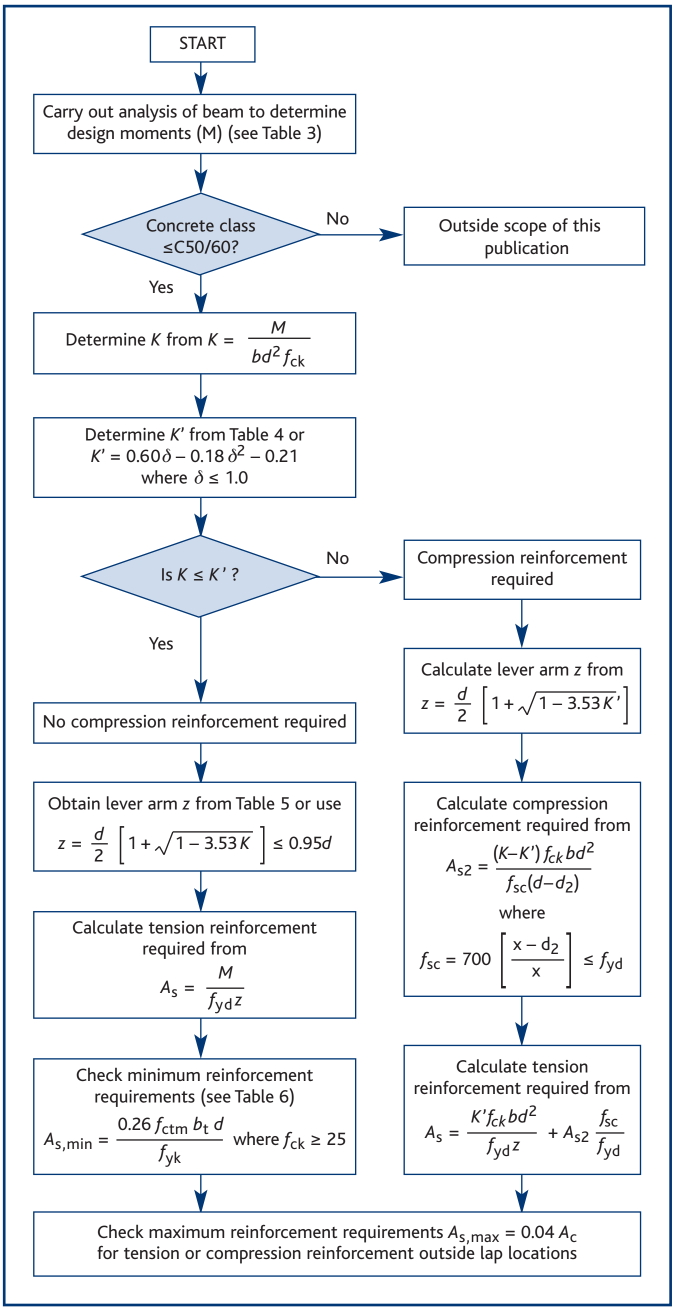


Table 6
Minimum percentage of required reinforcement

f_{ck}	f_{ctm}	Minimum percentage ($0.26 f_{ctm} / f_{yk}$[a])
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%

Key
a Assuming f_{yk} = 500 MPa

Figure 4
Strut inclination method

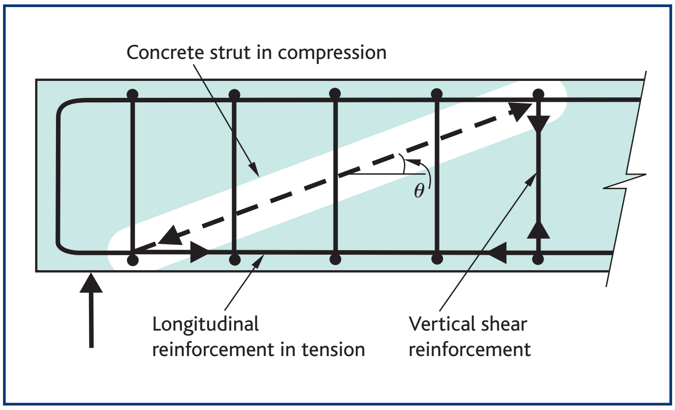


Figure 5
Procedure for determining vertical shear reinforcement

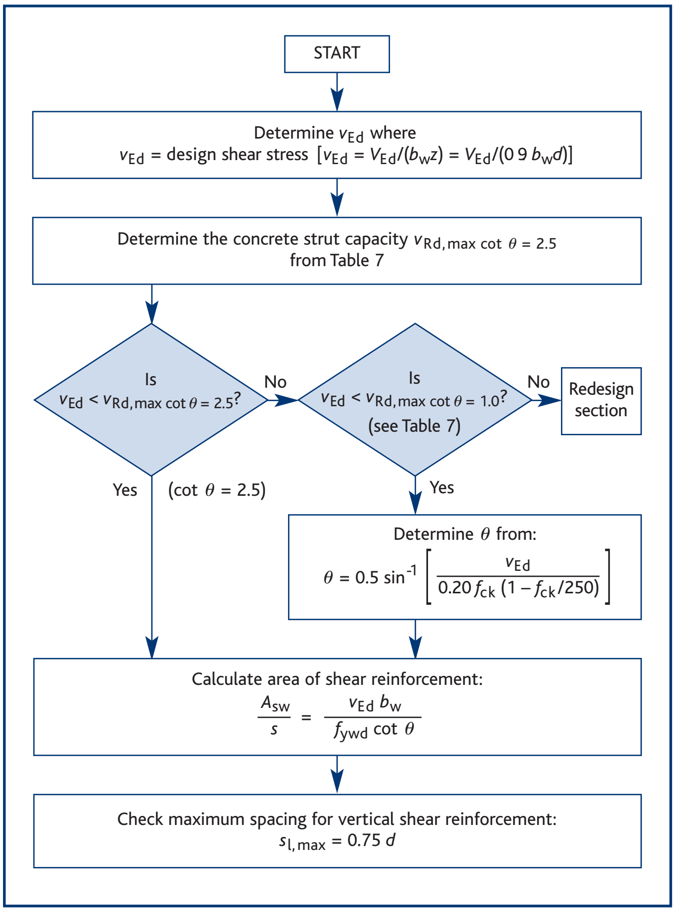


Table 7
Minimum and maximum concrete strut capacity in terms of stress

f_{ck}	$v_{Rd,max\ cot\ \theta = 2.5}$	$v_{Rd,max\ cot\ \theta = 1.0}$
20	2.54	3.68
25	3.10	4.50
28	3.43	4.97
30	3.64	5.28
32	3.84	5.58
35	4.15	6.02
40	4.63	6.72
45	5.08	7.38
50	5.51	8.00

Eurocode 2 offers various methods for determining the stress-strain relationship of concrete. For simplicity and familiarity the method presented here is the simplified rectangular stress block, which is similar to that found in BS 8110 (see Figure 3).

Eurocode 2 gives recommendations for the design of concrete up to class C90/105. However, for concrete greater than class C50/60, the stress block is modified. It is important to note that concrete strength is based on the cylinder strength and not the cube strength (i.e. for class C30/37 the cylinder strength (f_{ck}) is 30 MPa, whereas the cube strength is 37 MPa).

Vertical shear

Eurocode 2 introduces the strut inclination method for shear capacity checks. In this method the shear is resisted by concrete struts acting in compression and shear reinforcement acting in tension.

The angle of the concrete strut varies, depending on the shear force applied (see Figure 4). The procedure for determining the shear capacity of a section is shown in Figure 5 (which includes UK NA values) and is in terms of shear stress in the vertical plane rather than a vertical force as given in Eurocode 2. Where shear reinforcement is required, then the angle of the concrete strut should be calculated. For many typical beams the minimum angle of strut will apply (when cot $\theta = 2.5$ or $\theta = 21.8°$) i.e. for class C30/37 concrete the strut angle exceeds 21.8° only when the shear stress is greater than 3.27 N/mm² (refer to Table 7). As with BS 8110, there is a maximum permitted shear capacity, $v_{Rd,max}$, (when cot $\theta = 1.0$ or $\theta = 45°$), but this is not restricted to 5 MPa as in BS 8110.

Deflection

Eurocode 2 has two alternative methods for checking deflection, either a limiting span-to-depth ratio may be used or the theoretical deflection can be assessed using the expressions given in the Code. The latter is dealt with in detail in Chapter 8, originally published as *Deflection calculations*[7].

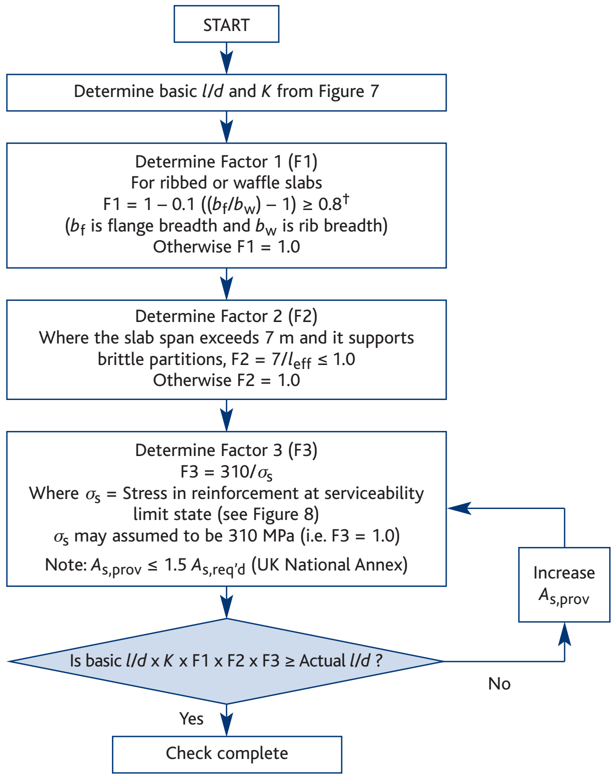
The span-to-depth ratios should ensure that deflection is limited to span/250 and this is the procedure presented in Figure 6.

Flanged beams

Flanged beams can be treated in much the same way as in BS 8110. The main differences compared with BS 8110 are that the assessment of the flange width is more sophisticated (see Figures 9 and 10) and that Eurocode 2 contains a check to confirm that the shear stress at

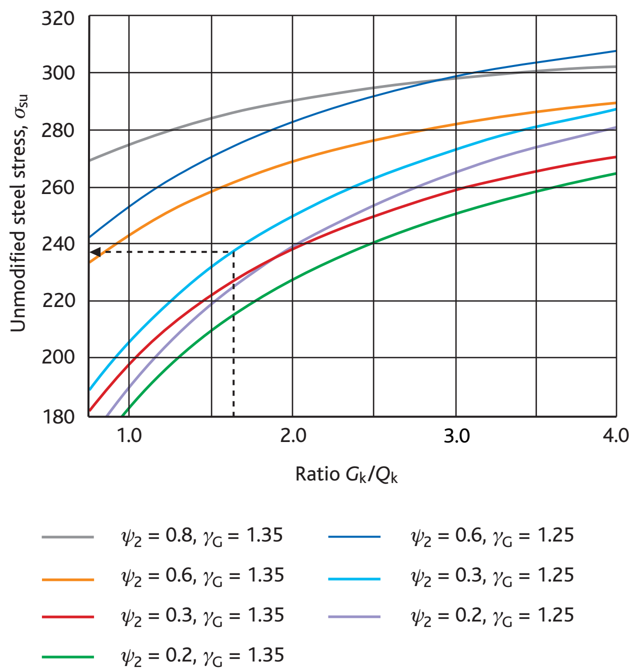
Continues page 31

Figure 6
Procedure for assessing deflection



† The Eurocode is ambiguous regarding linear interpolation. It is understood that it was the intention of the drafting committee that linear interpolation be used and this is in line with current UK practice.

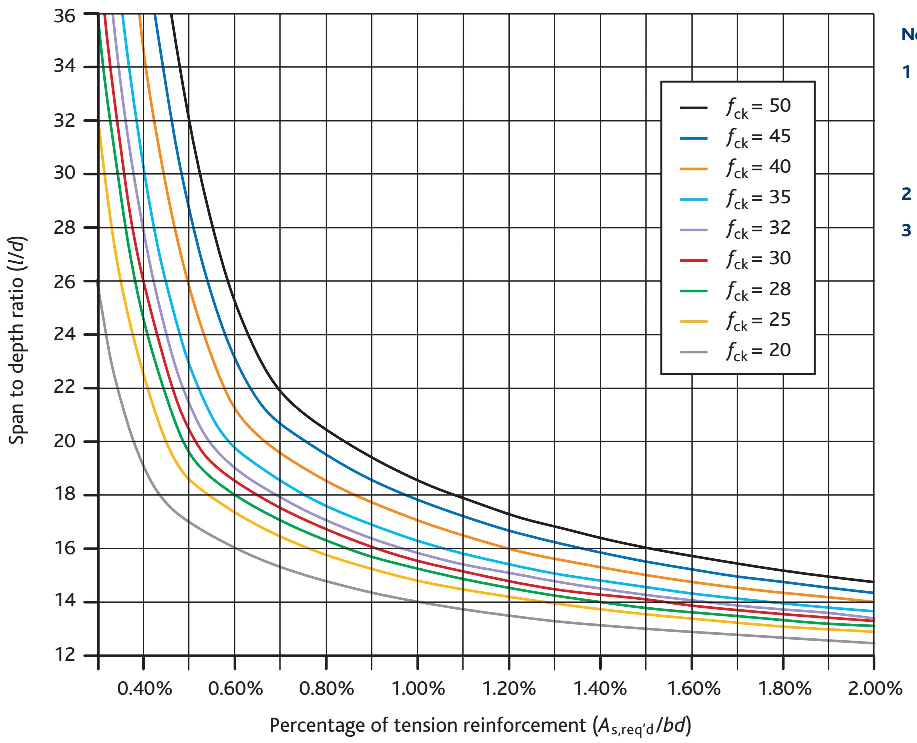
Figure 8
Determination of steel stress



To determine stress in the reinforcement (σ_s), calculate the ratio G_k/Q_k, read up the graph to the appropriate curve and read across to determine σ_{su}.

σ_s can be calculated from the expression: $\sigma_s = \sigma_{su}\left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$

Figure 7
Basic span-to-effective-depth ratios



Notes

1 This graph assumes simply supported span condition (K = 1.0).
K = 1.5 for interior span condition
K = 1.3 for end span condition
K = 0.4 for cantilevers.

2 Compression reinforcement, ρ', has been taken as 0.

3 Curves based on the following expressions:

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{1.5}\right]$$

where $\rho \leq \rho_0$

and

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{(\rho - \rho')} + \frac{\sqrt{f_{ck}}}{12}\sqrt{\frac{\rho'}{\rho_0}}\right]$$

where $\rho > \rho_0$.

Figure 11
Procedure for determining flexural capacity of flanged beams

START

Carry out analysis of beam to determine design moments, M (see Table 3)

Determine l_0 (see Figure 9) and b_{eff} from:
$b_{eff} = (b_w + b_{eff1} + b_{eff2})$ where
$b_{eff1} = (0.2b_1 + 0.1\ l_0) \leq 0.2\ l_0 \leq b_1$
$b_{eff2} = (0.2b_2 + 0.1\ l_0) \leq 0.2\ l_0 \leq b_2$
Note: The flange width at the support will be different from that at mid-span.
For symbols refer to Figures 9 and 10

Determine K from $K = \dfrac{M}{bd^2 f_{ck}}$
where $b = b_w$ at support
$b = b_{eff}$ in span

Determine K' from Table 4 or
$K' = 0.60\delta - 0.18\delta^2 - 0.21$ where $\delta \leq 1.0$

Calculate lever arm z from
$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K}\right] \leq 0.95d$$

Calculate depth to neutral axis x from:
$x = 2.5\ (d - z)$

Is $x \leq 1.25h_f$?

- Yes: Neutral axis in flange. Design as rectangular section (Figure 2) and then check longitudinal shear (Figure 14)
- No: Neutral axis in web
Calculate moment capacity of flange from:
$$M_{R,f} = 0.57\ f_{ck}\ (b_{eff} - b_w)\ h_f\ (d - 0.5h_f)$$
and $$K_f = \frac{M - M_{R,f}}{f_{ck}\ b_w\ d^2}$$

Is $K_f \leq K'$?

- No: Redesign section
- Yes: Calculate area of reinforcement required from
$$A_s = \frac{M_{R,f}}{f_{ywd}(d - 0.5\ h_f)} + \frac{M - M_{R,f}}{f_{ywd}}$$
→ Check longitudinal shear (see Figure 14)

Figure 9
Definition of l_0, for calculation of effective flange width

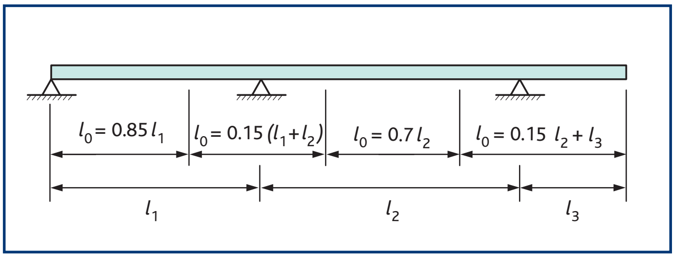


Figure 10
Effective flange width parameters

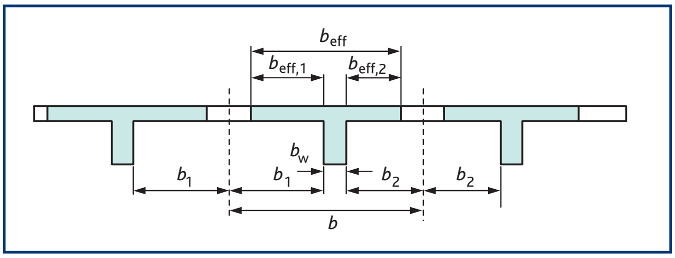


Figure 12
Placing of tension reinforcement in flanged cross section

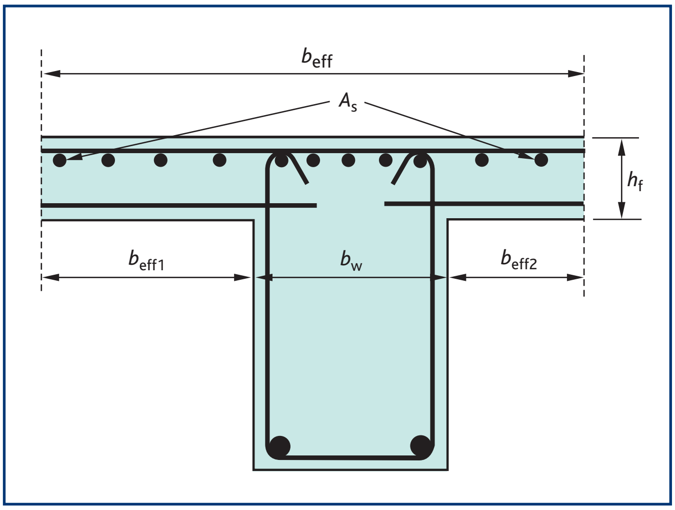
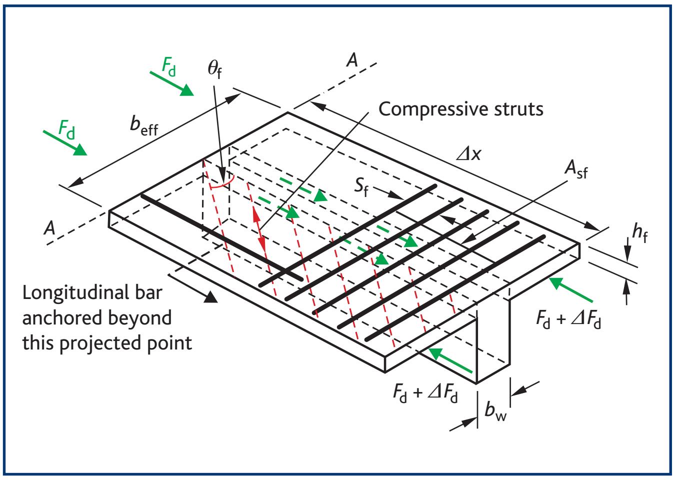


Figure 13
Notations for the connection between flange and web



the interface of the flange and web can be resisted by the transverse reinforcement in the flange. The position of the neutral axis should be determined, and then the area of reinforcement can be calculated depending whether it lies in the flange or web (see Figure 11).

At supports the tension reinforcement to resist hogging moments should be distributed across the full width of the effective flange as shown in Figure 12. The span-to-depth deflection checks using ratio of tension reinforcement should be based on area of concrete above centre of tension steel.

Longitudinal shear

The shear stress in the vertical plane between the flange and web should be assessed according to section 6.2.4 and Figure 6.7 of the Eurocode (reproduced here as Figure 13). The change in force in the flange can be assessed from the moment and lever arm at a particular location. The Eurocode states that the maximum length that can be considered for the change in force is half the distance between the maximum moment and the point where the moment is zero. Clearly, the maximum longitudinal force will occur where the change in moment, and therefore force, is the greatest; for a uniformly distributed load on a continuous beam this will be the length of beam closest to the support.

Figure 14 shows a flow chart for assessing the longitudinal shear capacity; in many cases the transverse reinforcement in the slab will be sufficient to resist the shear force. This check is included to ensure that where particularly thin flanges are used there is adequate reinforcement. The longitudinal shear capacity is based on the variable strut inclination method, which was described in the section on vertical shear.

Rules for spacing and quantity of reinforcement

Minimum area of longitudinal reinforcement

The minimum area of reinforcement is $A_{s,min} = 0.26\, f_{ctm}\, b_t\, d/f_{yk}$ but not less than $0.0013 b_t d$, where b_t is the mean width of the tension zone (see Table 6). For a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value of b

Maximum area of longitudinal reinforcement

The maximum area of tension or compression reinforcement, outside lap locations should not exceed $A_{s,max} = 0.04\, A_c$

Minimum spacing of reinforcement

The minimum clear distance between bars should be the greater of:

- Bar diameter
- Aggregate size plus 5 mm
- 20 mm

Minimum area of shear reinforcement

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

$$\frac{A_{sw}}{s b_w} \geqslant \rho_{w,min}$$

where $\rho_{w,min}$ can be obtained from Table 9.

Figure 14
Procedure for determining longitudinal shear capacity of flanged beams

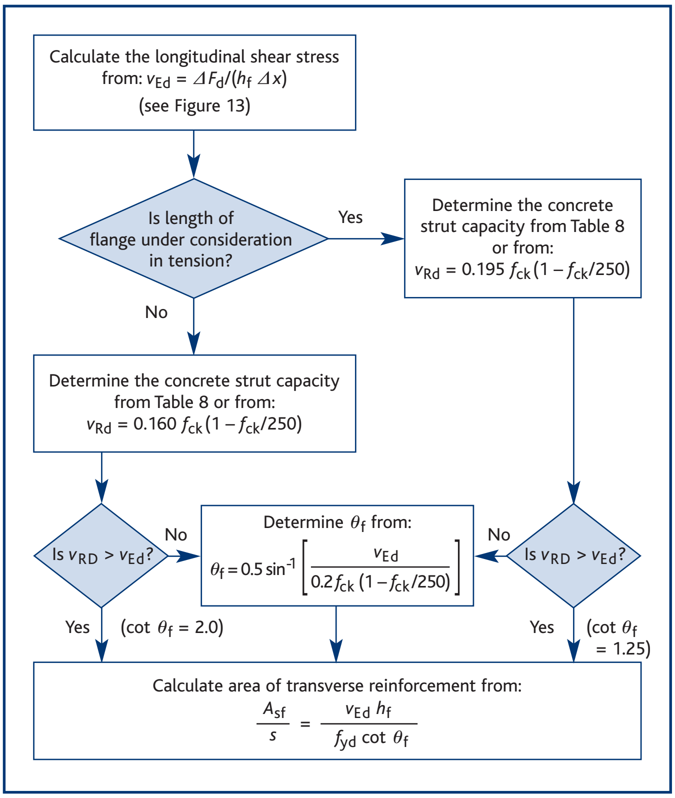


Table 8
Concrete strut capacity for longitudinal shear in flanged beams

f_{ck}	$v_{Rd,max}$	
	Flange in compression	Flange in tension
20	2.94	3.59
25	3.60	4.39
28	3.98	4.85
30	4.22	5.15
32	4.46	5.44
35	4.82	5.87
40	5.38	6.55
45	5.90	7.20
50	6.40	7.80

Table 9
Values for $\rho_{w,min}$

f_{ck}	20	25	28	30	32	35	40	45	50
$\rho_{w,min} \times 10^{-3}$	0.72	0.80	0.85	0.88	0.91	0.95	1.01	1.07	1.13

Selected symbols

Symbol	Definition	Value
A_c	Cross sectional area of concrete	
A_s	Area of tension steel	
A_{s2}	Area of compression steel	
$A_{s, prov}$	Area of tension steel provided	
$A_{s, req'd}$	Area of tension steel required	
b_{eff}	Effective flange width	
b_t	Mean width of the tension zone	
b_{min}	Width of beam or rib	
b_w	Width of section, or width of web on flanged beams	
d	Effective depth	
d_2	Effective depth to compression reinforcement	
f_{cd}	Design value of concrete compressive strength	$\alpha_{cc} f_{ck}/\gamma_c$ for $f_{ck} \leq$ C50/60
f_{ck}	Characteristic cylinder strength of concrete	
f_{ctm}	Mean value of axial tensile strength	$0.30 f_{ck}^{(2/3)}$ for $f_{ck} \leq$ C50/60 (from Table 3.1, Eurocode 2)
h_f	Flange thickness	
K	Factor to take account of the different structural systems	See table NA.4 in UK National Annex
l_{eff}	Effective span of member	See Section 5.3.2.2 (1)

Symbol	Definition	Value
l_o	Distance between points of zero moment	
l/d	Span-to-depth ratio	
M	Design moment at the ULS	
x	Depth to neutral axis	$(d-z)/0.4$
x_{max}	Limiting value for depth to neutral axis	$(\delta - 0.4)d$ where $\delta \leq 1.0$
z	Lever arm	
α_{cc}	Coefficient taking account of long term effects on compressive strength and of unfavourable effects resulting from the way load is applied	0.85 for flexure and axial loads 1.0 for other phenomena (From UK National Annex)
δ	Ratio of the redistributed moment to the elastic bending moment	
γ_m	Partial factor for material properties	1.15 for reinforcement (γ_s) 1.5 for concrete (γ_c)
ρ_0	Reference reinforcement ratio	$\sqrt{f_{ck}}/1000$
ρ	Required tension reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_s/bd (for rectangular beams)
ρ'	Required compression reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_{s2}/bd

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5. Columns

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Designing to Eurocode 2

This chapter is intended to assist engineers with the design of columns and walls to Eurocode 2[1]. It sets out a design procedure to follow and gives useful commentary on the provisions within the Eurocode. The layout and content of Eurocode 2 may appear unusual to designers familiar with BS 8110[2]. Eurocode 2 does not contain the derived formulae; this is because it has been European practice to give principles and general application rules in the codes and for detailed application rules to be presented in other sources such as textbooks or guidance documents.

Chapter 1, originally published as *Introduction to Eurocodes* [3], highlighted the key differences between Eurocode 2 and BS 8110, including terminology.

It should also be noted that values from the UK National Annex (NA) have been used throughout this publication, including values that are embedded in derived formulae. (Derivations can be found at www.eurocode2.info.) A full list of symbols related to column design is given at the end of this chapter.

Design procedure

A procedure for carrying out the detailed design of braced columns (i.e. columns that do not contribute to resistance of horizontal actions) is shown in Table 1. This assumes that the column dimensions have previously been determined during conceptual design or by using quick design methods, for example those presented in *Economic concrete frame elements*[4]. Column sizes should not be significantly different from those obtained using BS 8110. Steps 1 to 4 of Table 1 are covered by earlier chapters and the next step is therefore to consider fire resistance.

Fire resistance

Eurocode 2, Part 1–2: *Structural fire design*[5], gives a choice of advanced, simplified or tabular methods for determining fire resistance of columns. Using tables is the fastest method for determining the minimum dimensions and cover for columns. There are, however, some restrictions and if these apply further guidance can be obtained from specialist literature.[6] The simplified method may give more economic columns, especially for small columns and/or high fire resistance periods.

Rather than giving a minimum cover, the tabular method is based on nominal axis distance, *a* (see Figure 1). This is the distance from the centre of the main

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Table 1
Column design procedure

Step	Task	Further guidance	
		Chapter in the publication	Standard
1	Determine design life	2: *Getting started*	UK NA to BS EN 1990 Table NA.2.1
2	Assess actions on the column	2: *Getting started*	BS EN 1991 (10 parts) and UK National Annexes
3	Determine which combinations of actions apply	1: *Introduction to Eurocodes*	UK NA to BS EN 1990 Tables NA.A1.1 and NA.A1.2 (B)
4	Assess durability requirements and determine concrete strength	2: *Getting started*	BS 8500: 2002
5	Check cover requirements for appropriate fire resistance period	2: *Getting started* and Table 2	Approved Document B. BS EN 1992–1–2
6	Calculate min. cover for durability, fire and bond requirements	2: *Getting started*	BS EN 1992–1–1 Cl. 4.4.1
7	Analyse structure to obtain critical moments and axial forces	2: *Getting started* and 'Structural analysis' section	BS EN 1992–1–1 section 5
8	Check slenderness	See Figures 2 and 3	BS EN 1992–1–1 section 5.8
9	Determine area of reinforcement required	See Figures 2 and 3	BS EN 1992–1–1 section 6.1
10	Check spacing of bars	'Rules for spacing' section	BS EN 1992–1–1 sections 8 and 9

Note
NA = National Annex.

Table 2
Minimum column dimensions and axis distances for fire resistance

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

Note
The table is taken from BS EN 1992–1–2 Table 5.2a (method A) and is valid under the following conditions:

1. The effective length of a braced column under fire conditions $l_{o,fi} \leq$ 3m. The value of $l_{o,fi}$ may be taken as 50% of the actual length for intermediate floors and between 50% and 70% of the actual length for the upper floor column.
2. The first order eccentricity under fire conditions should be ≤ 0.15b (or h). Alternatively use method B (see Eurocode 2, Part 1–2, Table 5.2b). The eccentricity under fire conditions may be taken as that used in normal temperature design.
3. The reinforcement area outside lap locations does not exceed 4% of the concrete cross section.
4. μ_{fi} is the ratio of the design axial load under fire conditions to the design resistance of the column at normal temperature conditions. μ_{fi} may conservatively be taken as 0.7.

Key
a Minimum 8 bars
b Method B may be used which indicates 600/70 for R 240 and μ_{fi} = 0.7. See BS EN 1992–1–2 Table 5.2b

Figure 1
Section through structural member, showing nominal axis distance *a*

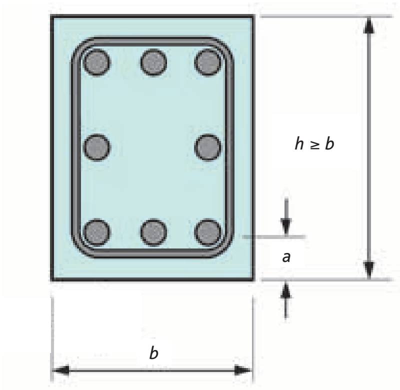


Table 3
Minimum reinforced concrete wall dimensions and axis distances for load-bearing for fire resistance

Standard fire resistance	Minimum dimensions (mm) Wall thickness/axis distance, a, of the main bars	
	Wall exposed on one side (μ_{fi} = 0.7)	Wall exposed on two sides (μ_{fi} = 0.7)
REI 60	130/10[a]	140/10[a]
REI 90	140/25	170/25
REI 120	160/35	220/35
REI 240	270/60	350/60

Notes
1. The table is taken from BS EN 1992–1–2 Table 5.4.
2. See note 4 of Table 2.

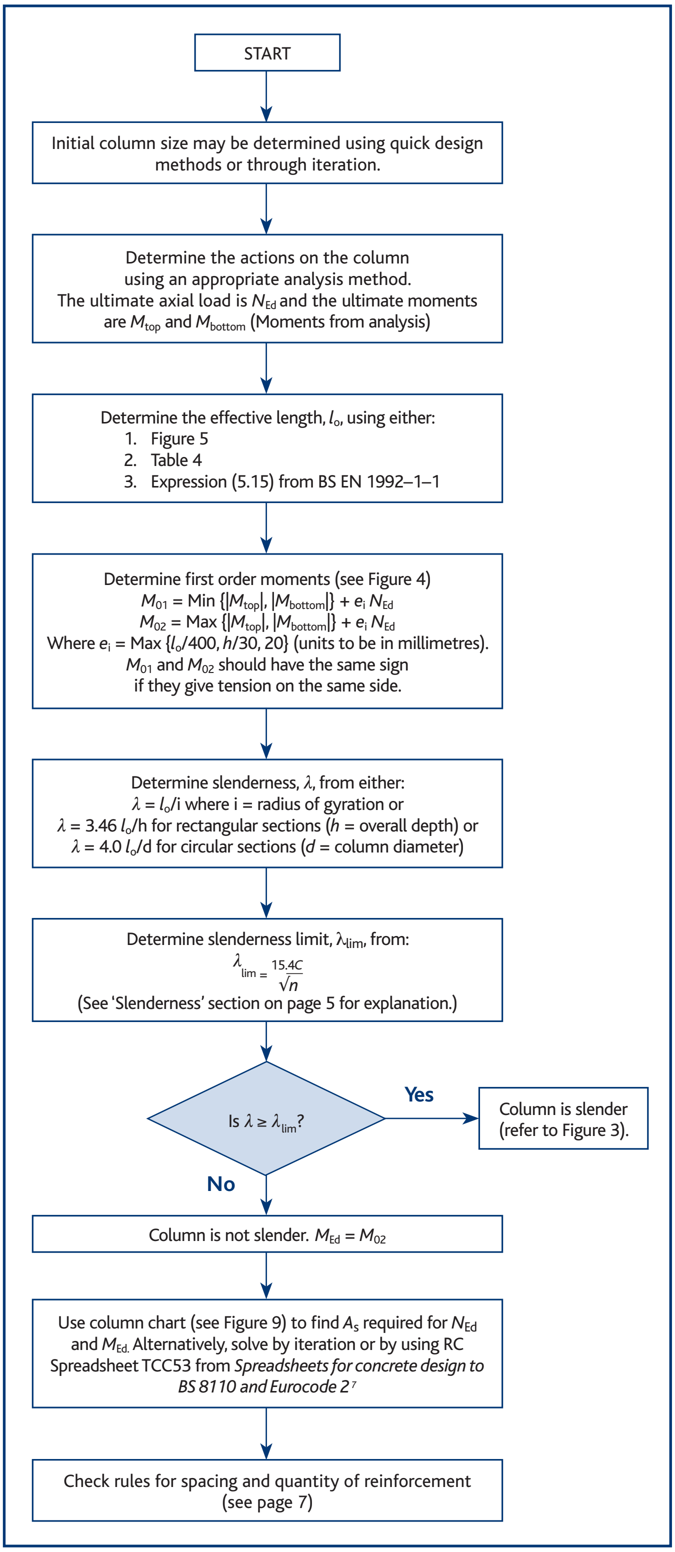
Key
a Normally the requirements of BS EN 1992–1–1 will determine the cover.

reinforcing bar to the surface of the member. It is a nominal (not minimum) dimension, and the designer should ensure that: $a \geq c_{nom} + \phi_{link} + \phi_{bar}/2$.

For columns there are two tables given in Eurocode 2 Part 1–2 that present methods A and B. Both are equally applicable, although method A has smaller limits on eccentricity than method B. Method A is slightly simpler and is presented in Table 2; limits of applicability are given in the notes. Similar data for load-bearing walls is given in Table 3.

For columns supporting the uppermost storey, the eccentricity will often exceed the limits for both methods A and B. In this situation Annex C of Eurocode 2, Part 1–2 may be used. Alternatively, consideration can be given to treating the column as a beam for determining the design fire resistance.

Figure 2
Flow chart for braced column design



Column design

A flow chart for the design of braced columns is shown in Figure 2. For slender columns, Figure 3 will also be required.

Structural analysis

The type of analysis should be appropriate to the problem being considered. The following may be used: linear elastic analysis, linear elastic analysis with limited redistribution, plastic analysis and non-linear analysis. Linear elastic analysis may be carried out assuming cross sections are uncracked (i.e. concrete section properties), using linear stress-strain relationships and assuming mean values of long-term elastic modulus.

For the design of columns the elastic moments from the frame action should be used without any redistribution. For slender columns a non-linear analysis may be carried out to determine the second order moments; alternatively use the moment magnification method (Cl 5.8.7.3) or nominal curvature method (Cl 5.8.8) as illustrated in Figure 3. The latter is expected to be adopted in the UK.

Design moments

The design bending moment is illustrated in Figure 4 and defined as:

$$M_{Ed} = \text{Max}\{M_{02}, M_{0e} + M_2, M_{01} + 0.5\, M_2\}$$

where

$M_{01} = \text{Min}\{|M_{top}|, |M_{bottom}|\} + e_i N_{Ed}$
$M_{02} = \text{Max}\{|M_{top}|, |M_{bottom}|\} + e_i N_{Ed}$
$e_i = \text{Max}\{l_o/400, h/30, 20\}$ (units to be in millimetres).
M_{top}, M_{bottom} = Moments at the top and bottom of the column
$M_{0e} = 0.6\, M_{02} + 0.4\, M_{01} \geq 0.4\, M_{02}$
$M_2 = N_{Ed}\, e_2$ where N_{Ed} is the design axial load and e_2 is deflection due to second order effects

M_{01} and M_{02} should be positive if they give tension on the same side.

A non-slender column can be designed ignoring second order effects and therefore the ultimate design moment, $M_{Ed} = M_{02}$.

The calculation of the eccentricity, e_2, is not simple and is likely to require some iteration to determine the deflection at approximately mid-height, e_2. Guidance is given in Figure 3.

Figure 3
Flow chart for slender columns (nominal curvature method)

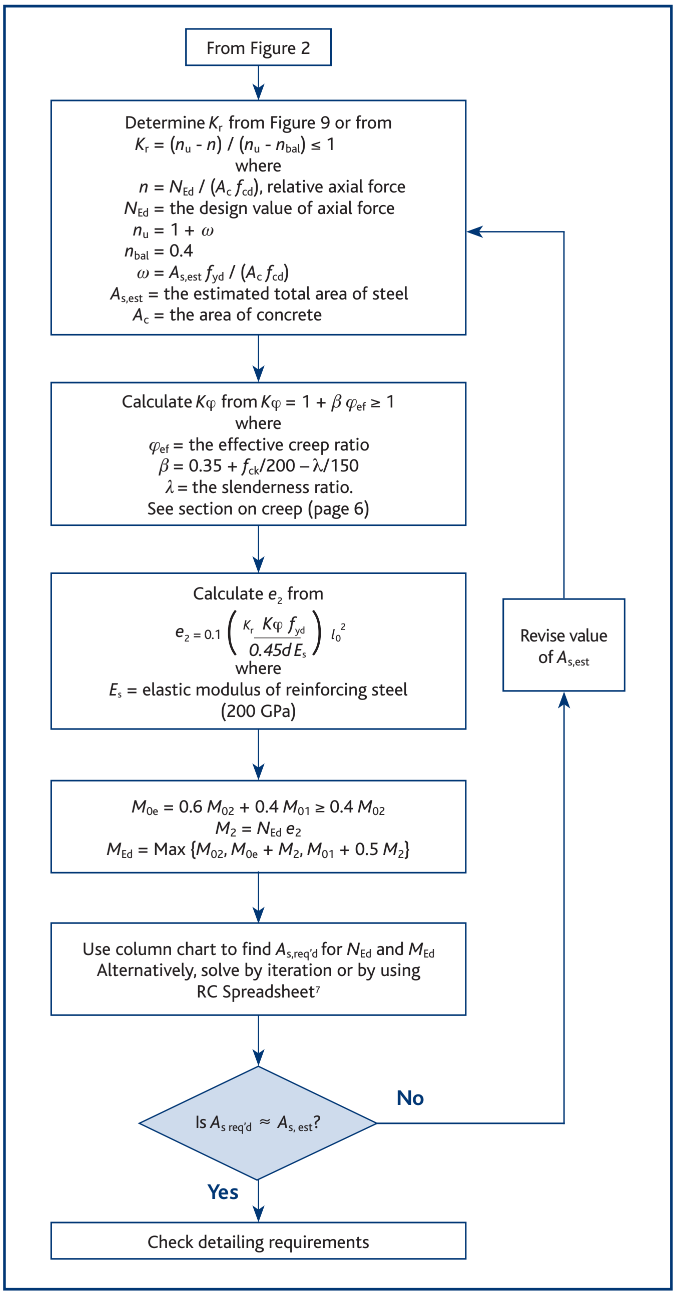
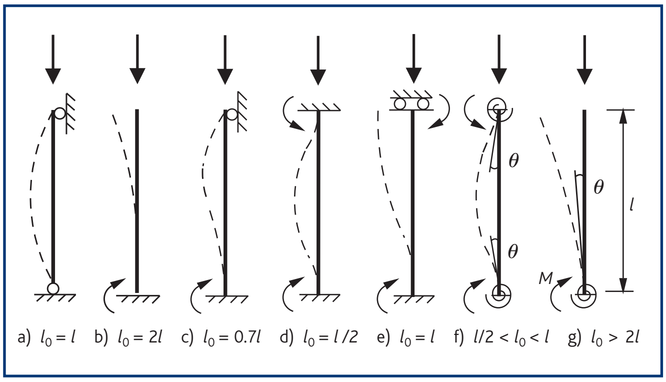


Figure 5
Effective lengths for isolated members



Effective length

Figure 5 gives guidance on the effective length of the column. However, for most real structures Figures 5f) and 5g) only are applicable, and Eurocode 2 provides two expressions to calculate the effective length for these situations. Expression (5.15) is for braced members and Expression (5.16) is for unbraced members.

In both expressions, the relative flexibilities at either end, k_1 and k_2, should be calculated. The expression for k given in the Eurocode involves calculating the rotation of the restraining members, which in practice requires the use of framework analysis software. Alternatively, PD 6687: *Background paper to the UK National annex*[8] provides a simplification, based on the stiffness of the beams attached to either side of the column. This relative stiffness, k, can therefore be calculated as follows (provided the stiffness of adjacent columns does not vary by more than 15% of the higher stiffness):

$$k = \frac{EI_c}{l_c} \bigg/ \Sigma \frac{2EI_b}{l_b} \geq 0.1$$

where

I_c, I_b are the column and beam uncracked second moments of area

l_c, l_b are the column and beam lengths

Once k_1 and k_2 have been calculated, the effective length factor, F, can be established from Table 4 for braced columns. The effective length is then $l_o = Fl$.

For a 400 mm square internal column supporting a 250 mm thick flat slab on a 7.5 m grid, the value of k could be 0.11, and therefore $l_o = 0.59l$. In the edge condition k is effectively doubled and $l_o = 0.67l$. If the internal column had a notionally 'pinned' support at its base then $l_o = 0.77l$.

It is also generally accepted that Table 3.19 of BS 8110 may conservatively be used to determine the effective length factor. In the long term, Expressions (5.15) and (5.16) will be beneficial as they are particularly suitable for incorporation into design software.

Figure 4
Design bending moments

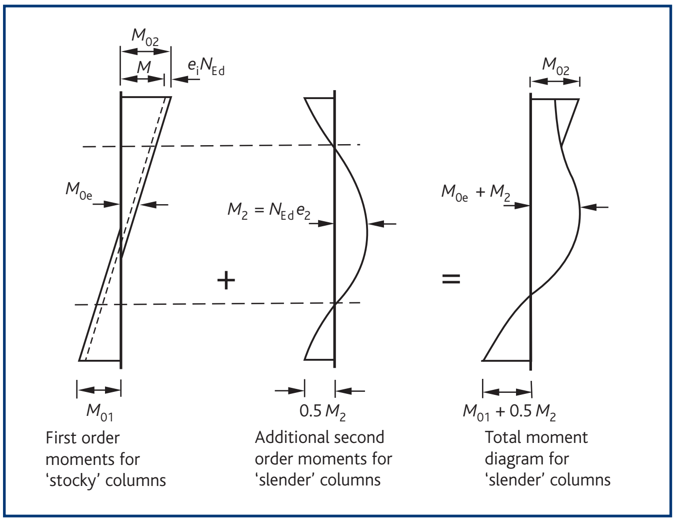


Table 4
Effective length factor, F, for braced columns

*k*2	*k*1										
	0.10	0.20	0.30	0.40	0.50	0.70	1.00	2.00	5.00	9.00	Pinned
0.10	0.59	0.62	0.64	0.66	0.67	0.69	0.71	0.73	0.75	0.76	0.77
0.20	0.62	0.65	0.68	0.69	0.71	0.73	0.74	0.77	0.79	0.80	0.81
0.30	0.64	0.68	0.70	0.72	0.73	0.75	0.77	0.80	0.82	0.83	0.84
0.40	0.66	0.69	0.72	0.74	0.75	0.77	0.79	0.82	0.84	0.85	0.86
0.50	0.67	0.71	0.73	0.75	0.76	0.78	0.80	0.83	0.86	0.86	0.87
0.70	0.69	0.73	0.75	0.77	0.78	0.80	0.82	0.85	0.88	0.89	0.90
1.00	0.71	0.74	0.77	0.79	0.80	0.82	0.84	0.88	0.90	0.91	0.92
2.00	0.73	0.77	0.80	0.82	0.83	0.85	0.88	0.91	0.93	0.94	0.95
5.00	0.75	0.79	0.82	0.84	0.86	0.88	0.90	0.93	0.96	0.97	0.98
9.00	0.76	0.80	0.83	0.85	0.86	0.89	0.91	0.94	0.97	0.98	0.99
Pinned	0.77	0.81	0.84	0.86	0.87	0.90	0.92	0.95	0.98	0.99	1.00

Figure 6
Calculating factor C

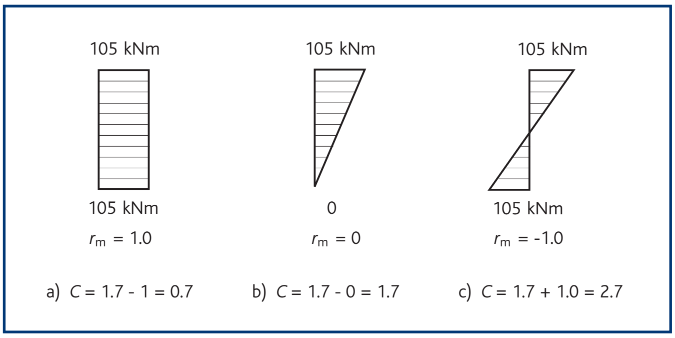


Figure 7
Stress block diagram for columns

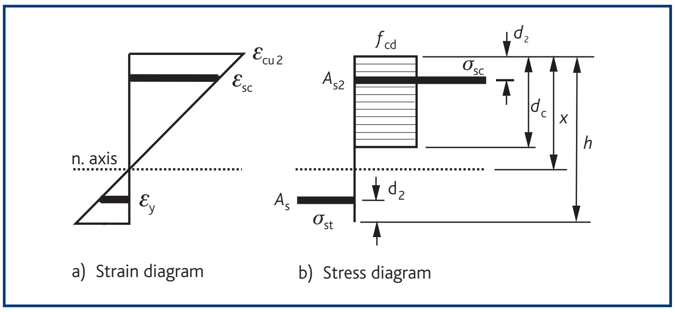
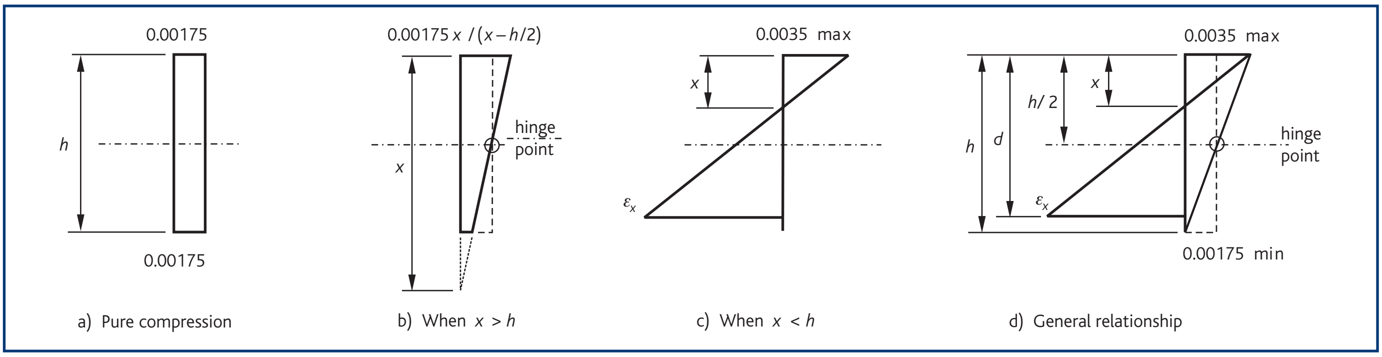


Figure 8
Strain diagrams for columns



Slenderness

Eurocode 2 states that second order effects may be ignored if they are less than 10% of the first order effects. As an alternative, if the slenderness (λ) is less than the slenderness limit (λ_{lim}), then second order effects may be ignored.

Slenderness, $\lambda = l_o/i$ where i = radius of gyration and slenderness limit.

$$\lambda_{lim} = \frac{20ABC}{\sqrt{n}} \leq \frac{15.4C}{\sqrt{n}}$$

where

$A = 1/(1+0.2\,\varphi_{ef})$ (if φ_{ef} is not known, $A = 0.7$ may be used)
$B = \sqrt{1+2\omega}$, (if ω, reinforcement ratio, is not known, $B = 1.1$ may be used)
$C = 1.7 - r_m$ (if r_m is not known, $C = 0.7$ may be used – see below)
$n = N_{Ed} / (A_c\, f_{cd})$
$r_m = M_{01}/M_{02}$
M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

If the end moments M_{01} and M_{02} give tension on the same side, r_m should be taken positive.

Of the three factors *A, B* and *C, C* will have the largest impact on λ_{lim} and is the simplest to calculate. An initial assessment of λ_{lim} can therefore be made using the default values for *A* and *B*, but including a calculation for *C* (see Figure 6). Care should be taken in determining *C* because the sign of the moments makes a significant difference. For unbraced members *C* should always be taken as 0.7.

Column design resistance

For practical purposes the rectangular stress block used for the design of beams (see Chapter 4, originally published as *Beams*[9]) may also be used for the design of columns (see Figure 7). However, the maximum compressive strain for concrete classes up to and including C50/60, when the whole section is in pure compression, is 0.00175 (see Figure 8a). When the neutral axis falls **outside** the section (Figure 8b), the maximum allowable strain is assumed to lie between 0.00175 and 0.0035, and may be obtained by drawing a line from the point of zero strain through the 'hinge point' of 0.00175 strain at mid-depth of the section. When the neutral axis lies **within** the section depth then the maximum compressive strain is 0.0035 (see Figure 8c).

The general relationship is shown in Figure 8d). For concrete classes above C50/60 the principles are the same but the maximum strain values vary.

Two expressions can be derived for the area of steel required, (based on a rectangular stress block, see Figure 7) one for the axial loads and the other for the moments:

$A_{sN}/2 = (N_{Ed} - f_{cd}\, b\, d_c) / (\sigma_{sc} - \sigma_{st})$

where

A_{sN} = Area of reinforcement required to resist axial load
N_{Ed} = Axial load
f_{cd} = Design value of concrete compressive strength
σ_{sc} (σ_{st}) = Stress in compression (and tension) reinforcement
b = Breadth of section
d_c = Effective depth of concrete in compression = $\lambda x \le h$
λ = 0.8 for ≤ C50/60
x = Depth to neutral axis
h = Height of section

$A_{sM}/2 = [M - f_{cd}\, b\, d_c(h/2 - d_c/2)] / [(h/2 - d_2)(\sigma_{sc} + \sigma_{st})]$

where

A_{sM} = Total area of reinforcement required to resist moment

Realistically, these can only be solved iteratively and therefore either computer software (e.g. RC Spreadsheet TCC53 from *Spreadsheets for concrete design to BS 8110 and EC2*[7]) or column design charts (see Figures 9a to 9e) may be used.

Creep

Depending on the assumptions used in the design, it may be necessary to determine the effective creep ratio φ_{ef} (ref. Cl. 3.1.4 & 5.8.4). A nomogram is provided in the Eurocode (Figure 3.1) for which the cement strength class is required; however, at the design stage it often not certain which class applies. Generally, Class R should be assumed. Where the ground granulated blastfurnace slag (ggbs) exceeds 35% of the cement combination or where pulverized fuel ash (pfa) exceeds 20% of the cement combination, Class N may be assumed. Where ggbs exceeds 65% or where pfa exceeds 35%, Class S may be assumed.

Biaxial bending

The effects of biaxial bending may be checked using Expression (5.39), which was first developed by Breslaer.

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \le 1.0$$

where

$M_{Edz,y}$ = Design moment in the respective direction including second order effects in a slender column
$M_{Rdz,y}$ = Moment of resistance in the respective direction
a = 2 for circular and elliptical sections; refer to Table 5 for rectangular sections
$N_{Rd} = A_c\, f_{cd} + A_s\, f_{yd}$

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Figure 9a
Column design chart for rectangular columns d_2/h = 0.05

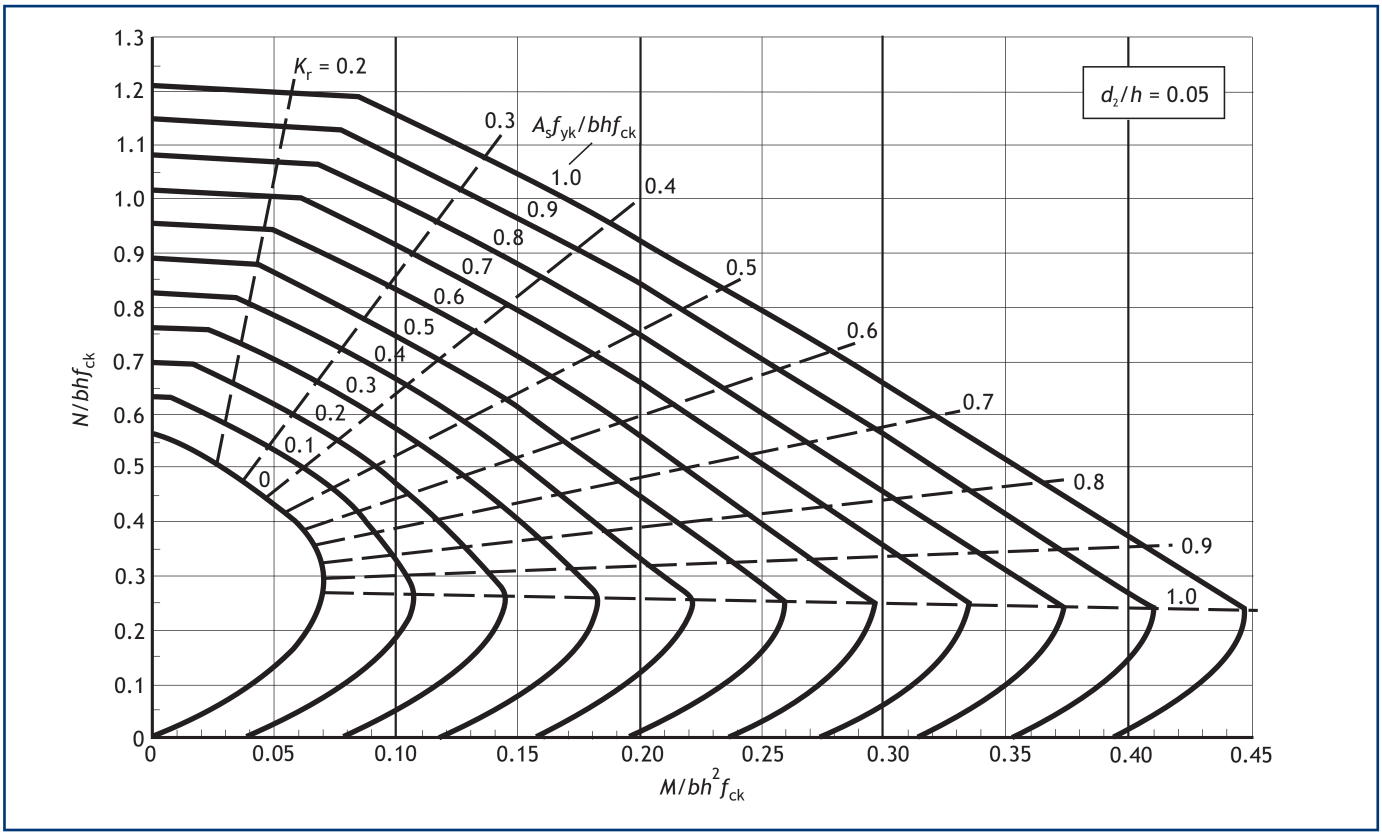

Figure 9b
Column design chart for rectangular columns d_2/h = 0.10

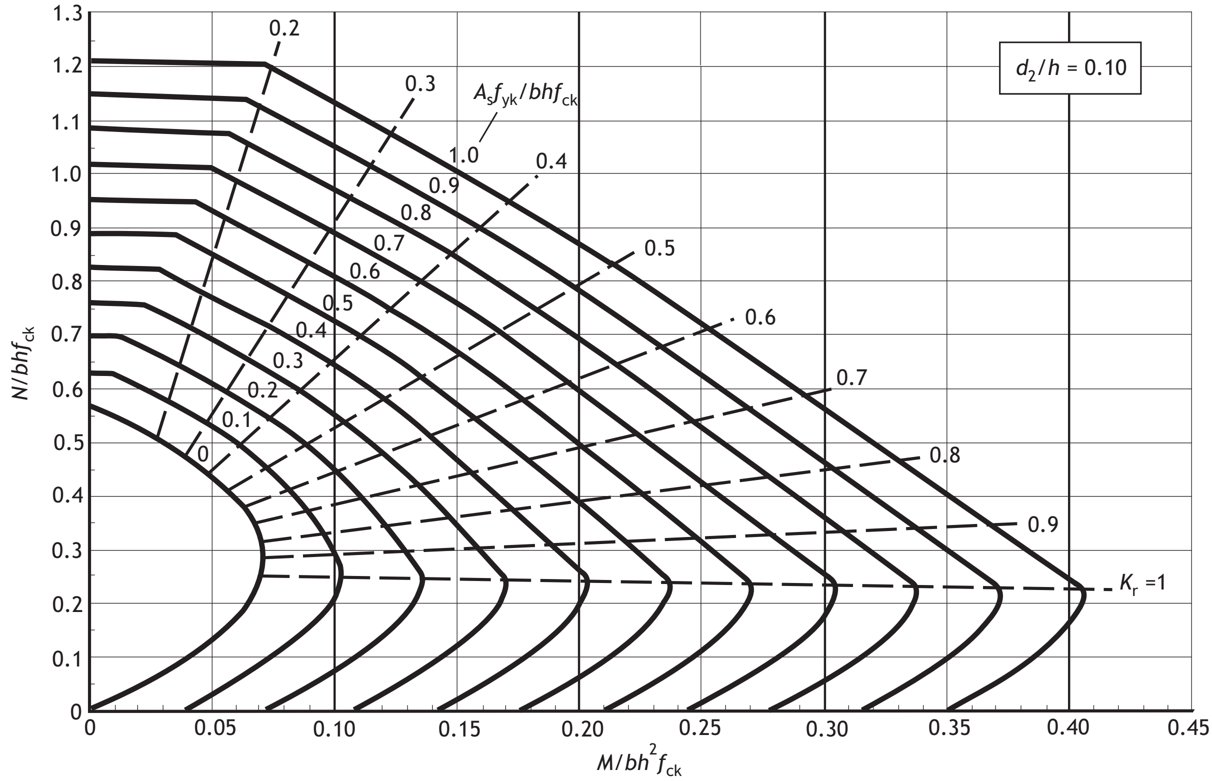

Figure 9c
Column design chart for rectangular columns d_2/h = 0.15

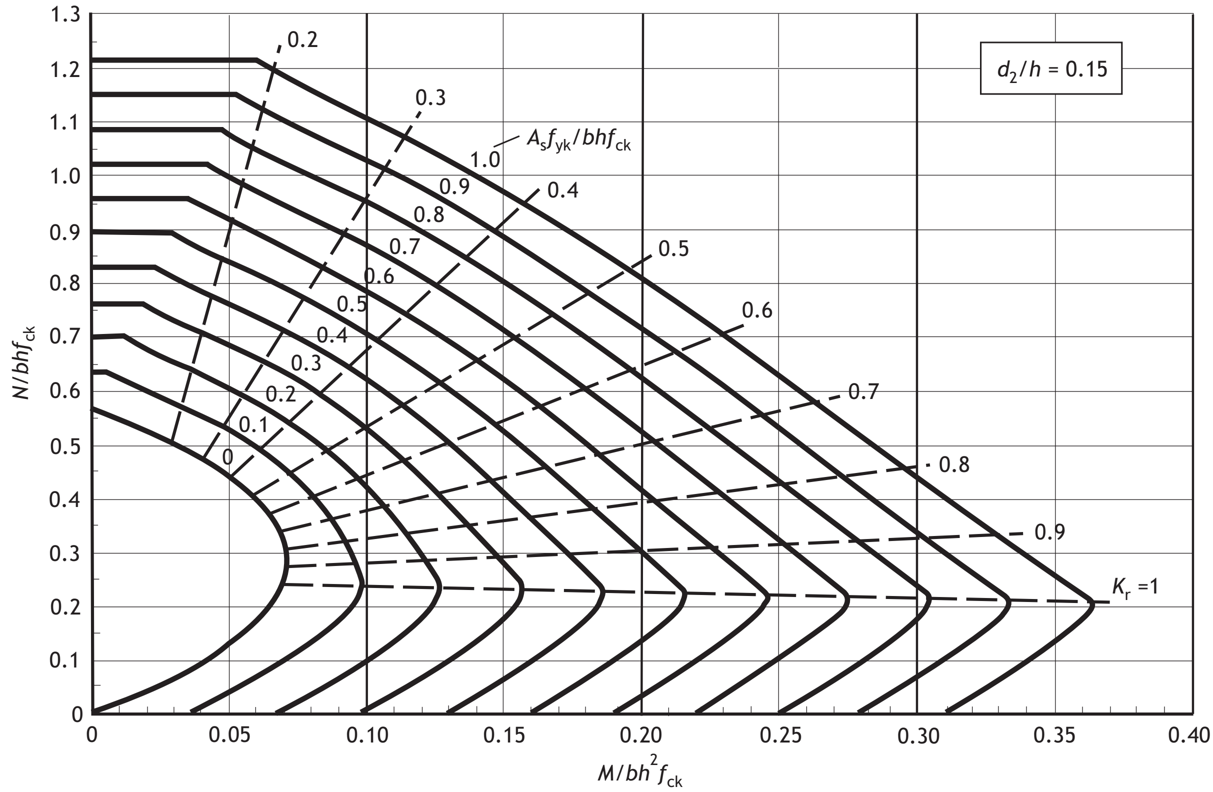

Figure 9d
Column design chart for rectangular columns d_2/h = 0.20

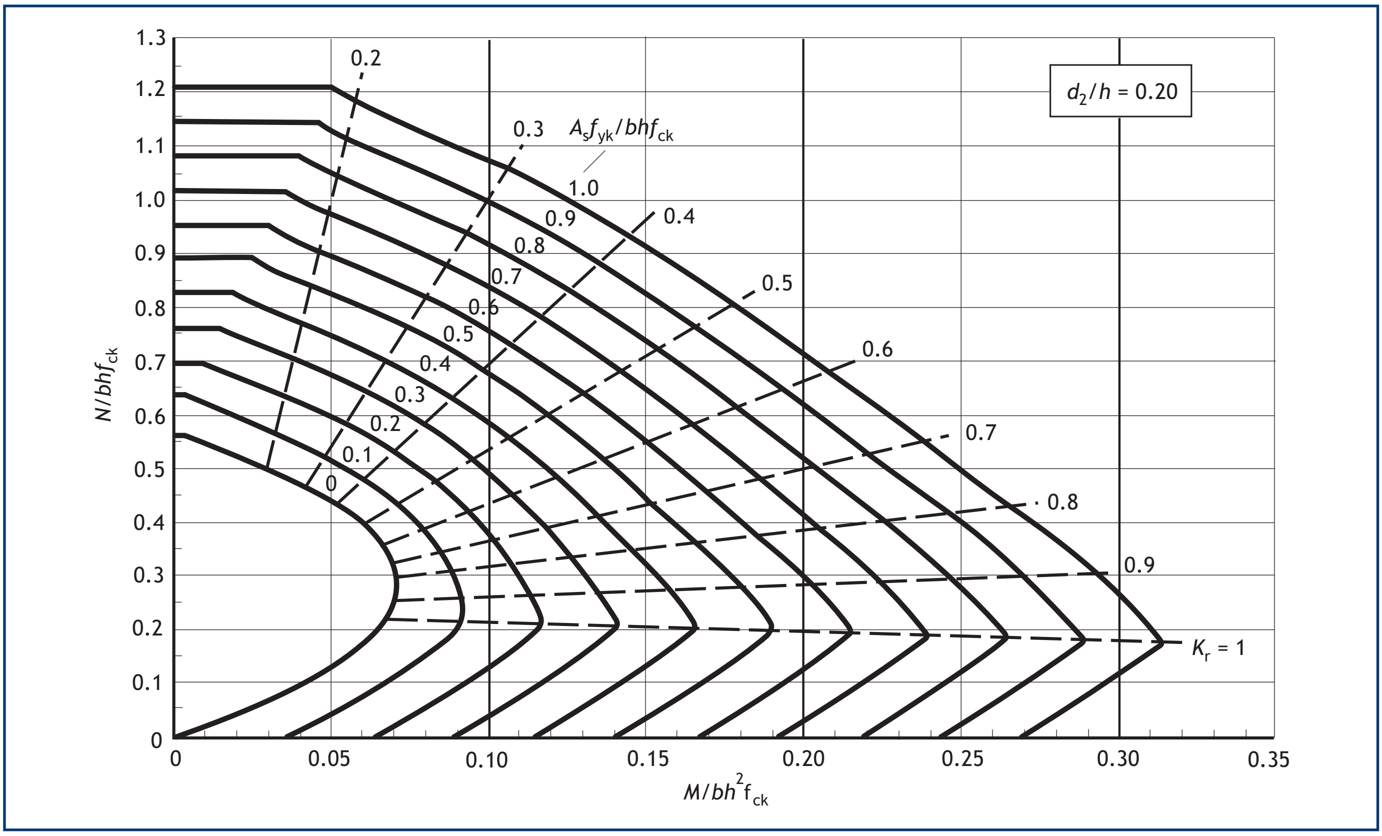

Figure 9e
Column design chart for rectangular columns d_2/h = 0.25

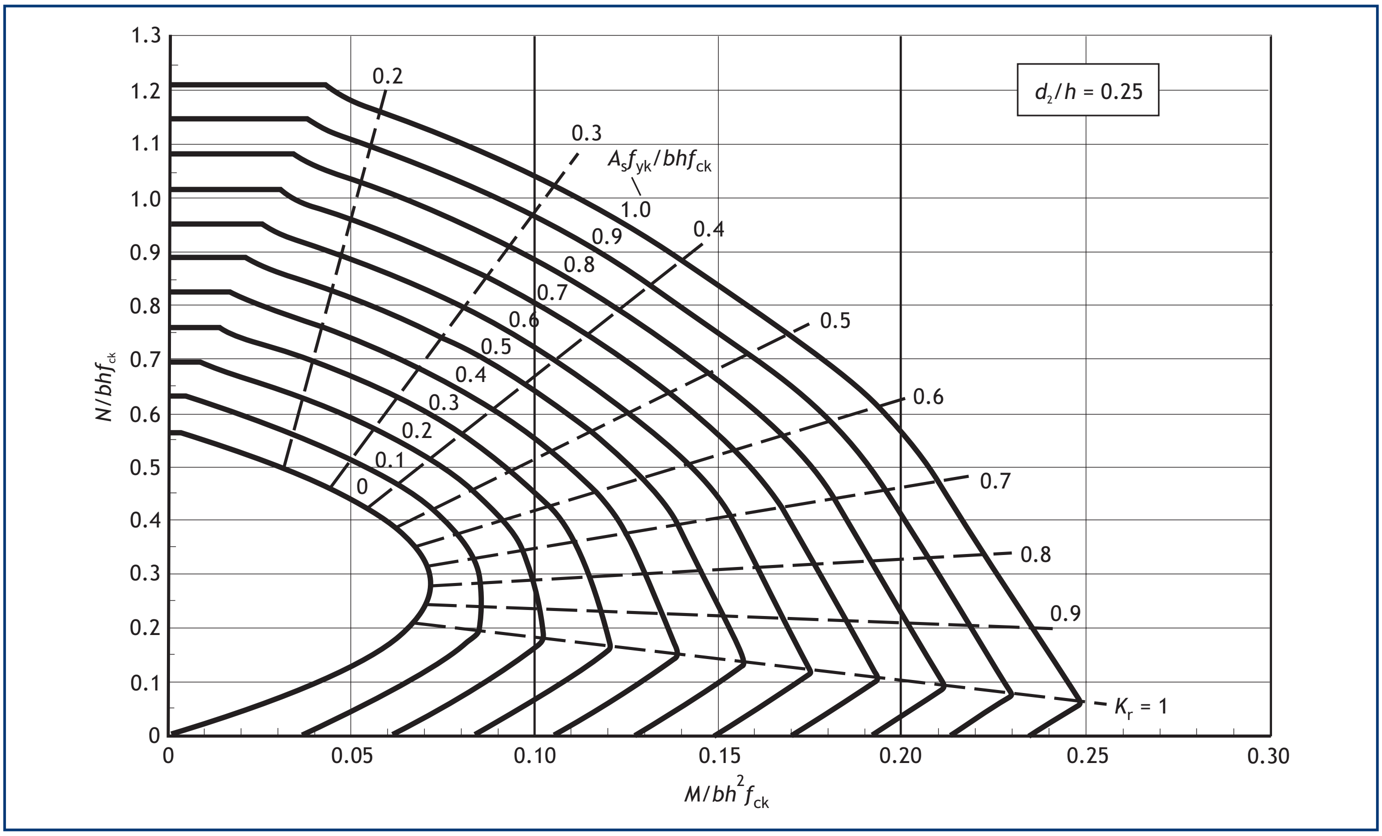

Table 5
Value of *a* for rectangular sections

N_{Ed}/N_{Rd}	0.1	0.7	1.0
a	1.0	1.5	2.0

Note
Linear interpolation may be used.

Unbraced columns

There is no comment made on the design of sway frames in Eurocode 2. However, it gives guidance on the effective length of an unbraced member in Expression (5.16). The value for C of 0.7 should always be used in Expression (5.13N). The design moments should be assessed including second order effects. The tabular method for fire resistance design (Part 1–2) does not explicitly cover unbraced columns; however reference can be made to the *Handbook to EN 1992–1–2*[6].

Walls

When the section length of a vertical element is four times greater than its thickness it is defined as a wall. The design of walls does not differ significantly from that for columns except for the following:

- The requirements for fire resistance (see Table 3).
- Bending will be critical about the weak axis.
- There are different rules for spacing and quantity of reinforcement (see below).

There is no specific guidance given for bending about the strong axis for stability. However, the principles of CIRIA Report 108[10] may be followed. Alternatively the strut and tie method may be used (section 6.5 of the Eurocode).

Rules for spacing and quantity of reinforcement

Maximum areas of reinforcement

In Eurocode 2 the maximum nominal reinforcement area for columns and walls outside laps is 4% compared with 6% in BS 8110. However, this area can be increased provided that the concrete can be placed and compacted sufficiently. If required self-compacting concrete may be used for particularly congested situations, where the reinforcing bars should be spaced to ensure that the concrete can flow around them. Further guidance can be found in *Self-compacting concrete.*[11]

Minimum reinforcement requirements

The recommended minimum diameter of longitudinal reinforcement in columns is 12 mm. The minimum area of longitudinal reinforcement in columns is given by: $A_{s,min} = 0.10\, N_{Ed}/f_{yd} \geq 0.002A_c$ Exp. (9.12N)
The diameter of the transverse reinforcement should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars.

Selected symbols

Symbol	Definition	Value
$1/r_0$	Reference curvature	$\varepsilon_{yd}/(0.45\,d)$
$1/r$	Curvature	$K_r\, K\varphi\; 1/r_0$
a	Axis distance for fire resistance	
A	Factor for determining slenderness limit	$1 / (1+0.2\,\varphi_{ef})$
A_c	Cross sectional area of concrete	bh
A_s	Area of total column reinforcement	
B	Factor for determining slenderness limit	
c	Factor depending on curvature distribution	10 (for constant cross-section)
C	Factor for determining slenderness limit	$1.7 - r_m$
d	Effective depth	
e_2	Second order eccentricity	$(1/r)l_o/c$
e_i	Eccentricity due to geometric imperfections	
E_s	Elastic modulus of reinforcing steel	200 GPa
f_{cd}	Design value of concrete compressive strength	$\alpha_{cc}\, f_{ck}/\gamma c$
f_{ck}	Characteristic cylinder strength of concrete	
l	Clear height of compression member between end restraints	
l_o	Effective length	
K_r	Correction factor depending on axial load	
$K\varphi$	Factor taking into account creep	
M_{01},M_{02}	First order moments including the effect of geometric imperfections $\lvert M_{02}\rvert \geq \lvert M_{01}\rvert$	
M_2	Nominal second order moment	$N_{Ed}\, e_2$
M_{0e}	Equivalent first order moment	$0.6\, M_{02} + 0.4\, M_{01} \geq 0.4\, M_{02}$
M_{Ed}	Ultimate design moment	
M_{Eqp}	First order bending moment under quasi-permanent loading	
n	Relative axial force	$N_{Ed}/(A_c f_{cd})$
n_{bal}	Value of n at maximum moment of resistance	0.4
n_u	Factor to allow for reinforcement in the column	$1 + \omega$
N_{Ed}	Ultimate axial load	
r_m	Moment ratio	M_{01}/M_{02}
x	Depth to neutral axis	$(d - z)/0.4$
z	Lever arm	
α_{cc}	Coefficient taking account of long term effects on compressive strength and of unfavourable effects resulting from the way load is applied	0.85 for flexure and axial loads. 1.0 for other phenomena (From UK NA)
β	Factor	$0.35 + f_{ck}/200 - \lambda/150$
ε_{yd}	Design value of strain in reinforcement	f_{yd}/E_s
γ_m	Partial factor for material properties	1.15 for reinforcement (γ_s) 1.5 for concrete (γ_c)
λ	Slenderness	l_o/i
λ_{lim}	Slenderness limit	
μ_{fi}	Degree of utilisation in a fire	$N_{Ed,fi}/N_{Rd}$
φ_{ef}	Effective creep ratio	$\varphi(\infty,t_o)\, M_{Eqp}/M_{Ed}$
$\varphi\,(\infty,t_o)$	Final creep co-efficient to Cl 3.1.4	
ω	Mechanical reinforcement ratio	$A_s\, f_{yd}/(A_c\, f_{cd})$
$\lvert x\rvert$	Absolute value of x	
Max. $\{x,y+z\}$	The maximum of values x or y + z	

Spacing requirements for columns

The maximum spacing of transverse reinforcement (i.e. links) in columns (Clause 9.5.3(1)) should not exceed:

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

At a distance greater than the larger dimension of the column above or below a beam or slab these spacings can be increased by a factor of 1.67. The minimum clear distance between the bars should be the greater of the bar diameter, aggregate size plus 5 mm or 20 mm. No longitudinal bar should be further than 150 mm from transverse reinforcement (links) in the compression zone.

Particular requirements for walls

The minimum area of vertical reinforcement in walls is given by:

$A_{s,min} = 0.002A_c$

Half of this area should be located at each face. The distance between two adjacent vertical bars should not exceed the lesser of either three times the wall thickness or 400 mm.

The minimum area of horizontal reinforcement in walls is the greater of either 25% of vertical reinforcement or 0.001 A_c. However, where crack control is important, early age thermal and shrinkage effects should be considered explicitly.

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How to design concrete structures using Eurocode 2

6. Foundations

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Eurocode 7: Geotechnical design

Scope

All foundations should be designed so that the soil safely resists the actions applied to the structure. The design of any foundation consists of two components; the geotechnical design and the structural design of the foundation itself. However, for some foundations (e.g. flexible rafts) the effect of the interaction between the soil and structure may be critical and must also be considered. Geotechnical design is covered by Eurocode 7[1], which supersedes several current British Standards including BS 5930[2], BS 8002[3] and BS 8004[4]. The new Eurocode marks a significant change in geotechnical design in that limit state principles are used throughout and this should ensure consistency between the Eurocodes. There are two parts to Eurocode 7, Part 1: *General rules* and Part 2: *Ground investigation and testing*. Guidance on the design of retaining walls can be found in Chapter 9.

The essential features of Eurocode 7, Part 1 relating to foundation design are discussed in this chapter. It should be emphasised that this publication covers only the design of simple foundations, which are a small part of the scope of Eurocode 7. Therefore it should not be relied on for general guidance on this Eurocode. At the time of writing it is anticipated that the National Annex (NA) for Part 1 will be published in July 2007.

Limit states

The following ultimate limit states (ULS) should be satisfied for geotechnical design; they each have their own combinations of actions. (For an explanation of Eurocode terminology please refer to Chapter 1, originally published as *Introduction to Eurocodes*[5].)

EQU Loss of equilibrium of the structure.
STR Internal failure or excessive deformation of the structure or structural member.
GEO Failure due to excessive deformation of the ground.
UPL Loss of equilibrium due to uplift by water pressure.
HYD Failure caused by hydraulic gradients.

In addition, the serviceability limit states (SLS) should be satisfied. It will usually be clear that one of the limit states will govern the design and therefore it will not be necessary to carry out checks for all of them, although it is considered good practice to record that they have all been considered.

Geotechnical Categories

Eurocode 7 recommends three Geotechnical Categories to assist in establishing the geotechnical design requirements for a structure (see Table 1).

It is anticipated that structural engineers will take responsibility for the geotechnical design of category 1 structures, and that geotechnical engineers will take responsibility for category 3 structures. The geotechnical design of category 2 structures may be undertaken by members of either profession. This decision will very much depend on individual circumstances.

Methods of design and combinations

There has not been a consensus amongst geotechnical engineers over the application of limit state principles to geotechnical design. Therefore, to allow for these differences of opinion, Eurocode 7 provides for three Design Approaches to be used for the ULS. The decision on which approach to use for a particular country is given in its National Annex. In the UK Design Approach 1 will be specified in the National Annex. For this Design Approach (excluding pile and anchorage design) there are two sets of combinations to use for the STR and GEO ultimate limit states. The values for the partial factors to be applied to the actions for these combinations of partial factors are given in Table 2 and the partial factors for the geotechnical material properties are given in Table 3. Combination 1 will generally govern the structural resistance, and Combination 2 will generally govern the sizing of the foundations.

The partial factors for soil resistance to sliding and bearing should be taken as 1.0 for both combinations.

The partial factors to be applied to the actions at the EQU limit state are given in Table 4; the geotechnical material partial factors being the same as for Combination 2 in Table 3.

For the SLS, Eurocode 7 does not give any advice on whether the characteristic, frequent or quasi-permanent combination should be used. Where the prescriptive method is used for spread foundations (see page 3) then the characteristic values should be adopted. For

Table 1
Geotechnical categories of structures

Category	Description	Risk of geotechnical failure	Examples from Eurocode 7
1	Small and relatively simple structures	Negligible	None given
2	Conventional types of structure and foundation with no difficult ground or loading conditions	No exceptional risk	Spread foundations
3	All other structures	Abnormal risks	Large or unusual structures Exceptional ground conditions

Table 2
Design values of actions derived for UK design, STR/GEO ultimate limit state – persistent and transient design situations

Combination Expression reference from BS EN 1990	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Combination 1 (Application of combination 1 (BS EN 1997) to set B (BS EN 1990))					
Exp. (6.10)	1.35 G_k[a]	1.0 G_k[a]	1.5[b] Q_k	–	1.5[b] $\psi_{o,i}$[c] $Q_{k,i}$
Exp. (6.10a)	1.35 G_k[a]	1.0 G_k[a]	–	1.5 $\psi_{o,1}$[c] Q_k	1.5[b] $\psi_{o,i}$[c] $Q_{k,i}$
Exp. (6.10b)	0.925[d] x 1.35 G_k[a]	1.0 G_k[a]	1.5[b] Q_k	–	1.5[b] $\psi_{o,i}$[c] $Q_{k,i}$
Combination 2 (Application of combination 2 (BS EN 1997) to set C (BS EN 1990))					
Exp. (6.10)	1.0 G_k[a]	1.0 G_k[a]	1.3[b] $Q_{k,1}$	–	1.3[b] $\psi_{o,i}$[b] $Q_{k,i}$

Key
- **a** Where the variation in permanent action is not considered significant $G_{k,j,sup}$ and $G_{k,j,inf}$ may be taken as G_k
- **b** Where the action is favourable, $\gamma_{Q,i} = 0$ and the variable actions should be ignored
- **c** The value of ψ_o can be obtained from Table NA.A1.1 of the UK NA to BS EN 1990 (or see Table 3 of Chapter 1)
- **d** The value of ξ in the UK NA to BS EN 1990 is 0.925

Table 3
Partial factors for geotechnical material properties

	Angle of shearing resistance (apply to tan φ)	Effective cohesion	Undrained shear strength	Unconfined strength	Bulk density
Symbol	γ_{φ}	$\gamma_{c'}$	γ_{cu}	γ_{qu}	γ_{γ}
Combination 1	1.0	1.0	1.0	1.0	1.0
Combination 2	1.25	1.25	1.4	1.4	1.0

direct methods of calculation the frequent combination can be used for sizing of foundations and the quasi-permanent combination can be used for settlement calculations.

Further information on design combinations can be found in Chapter 1, originally published as *Introduction to Eurocodes*[5].

Geotechnical design report

A geotechnical design report should be produced for each project, even if it is only a single sheet. The report should contain details of the site, interpretation of the ground investigation report, geotechnical design recommendations and advice on supervision, monitoring and maintenance of the works. It is likely that this report will require input from more than one consultant, depending on whether the project is in Geotechnical Category 1, 2 or 3.

The foundation design recommendations should include bearing resistances and characteristic values for soil parameters. It should also clearly state whether the values are applicable to SLS or ULS and whether they are for Combination 1 or Combination 2.

Table 4
Design values of actions derived for UK design, EQU ultimate limit state – persistent and transient design situations

Combination Expression reference	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
Exp. (6.10)	1.1 G_k^{a}	0.90 G_k^{a}	1.5[b] Q_k	–	1.5[c] $\psi_{o,i}$ [c] $Q_{k,i}$

Key

a Where the variation in permanent action is not considered significant $G_{k,j,sup}$ and $G_{k,j,inf}$ may be taken as G_k

b Where the action is favourable, $\gamma_{Q,i} = 0$ and the variable actions should be ignored

c The value of ψ_o can be obtained from Table NA.A1.1 of the UK NA to BS EN 1990

Spread foundations

The geotechnical design of spread foundations (e.g. strip and pad foundations) is covered by section 6 of Eurocode 7, Part 1 and this gives three methods for design:

- Direct method – calculation is carried out for each limit state.
- Indirect method – experience and testing used to determine serviceability limit state parameters that also satisfy all relevant limit states (included in Eurocode 7 mainly to suit French design methods, and is not discussed further here).
- Prescriptive method in which a presumed bearing resistance is used.

For most spread foundations in the UK, settlement will be the governing criterion; traditionally 'allowable bearing pressures' have been used to limit settlement. This concept of increasing the factor of safety on bearing resistances to control settlement may still be used with the prescriptive method. The exception is for soft clays where Eurocode 7 requires settlement calculations to be undertaken.

When using the direct method, calculations are carried out for each limit state. At the ULS, the bearing resistance of the soil should be checked using partial factors on the soil properties as well as on the actions. At the SLS the settlement of the foundations should be calculated and checked against permissible limits.

The prescriptive method may be used where calculation of the soil properties is not possible or necessary and can be used provided that conservative rules of design are used. Therefore reference can continue to be made to Table 1 of BS 8004 (see Table 5) to determine presumed (allowable) bearing pressures for category 1 structures and preliminary calculations for category 2 structures. Alternatively, the presumed bearing resistance to allow for settlement can be calculated by the geotechnical designer and included in the geotechnical design report.

Table 5
Presumed allowable bearing values under static loading (from BS 8004)

Category	Type of soil	Presumed allowable bearing value (kN/m²)	Remarks
Non-cohesive soils	Dense gravel, or dense sand and gravel	> 600	Width of foundation not less than 1 m. Groundwater level assumed to be below the base of the foundation.
	Medium dense gravel, or medium dense sand and gravel	< 200 to 600	
	Loose gravel, or loose sand and gravel	< 200	
	Compact sand	> 300	
	Medium dense sand	100 to 300	
	Loose sand	< 100	
Cohesive soils	Very stiff boulder clay and hard clay	300 to 600	Susceptible to long-term consolidation settlement
	Stiff clay	150 to 300	
	Firm clay	75 to 150	
	Soft clay and silt	<75	
	Very soft clay and silt	Not applicable	

Note
These values are for preliminary design purposes only.

A flow chart showing the design process for shallow foundations is given in Figure 1.

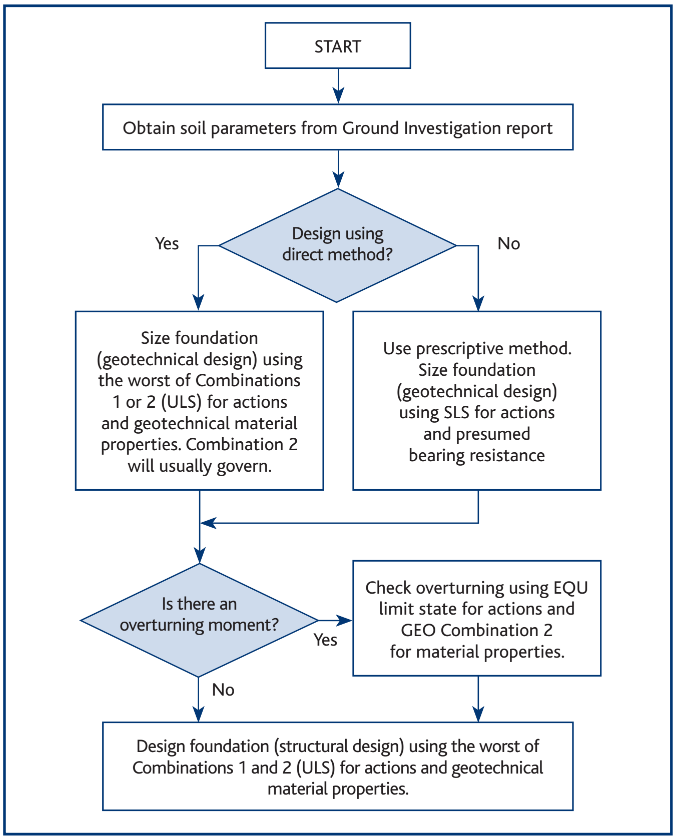
Where there is a moment applied to the foundation, the EQU limit state should also be checked. Assuming the potential overturning of the base is due to the variable action from the wind, the following combination should be used (the variable imposed action is not considered to contribute to the stability of the structure):

$0.9\ G_k + 1.5\ Q_{k,w}$ EQU combination

where:

G_k is the stabilising characteristic permanent action
(Use 1.1 G_k for a destabilising permanent action)
$Q_{k,w}$ is the destabilising characteristic variable wind action

Figure 1
Procedures for depth of spread foundations



Partial factors for the soil parameters used to determine the resistances can be obtained from Table 3 above (Combination 2).

The pressure distribution under the base should be assessed to ensure that the maximum pressure does not exceed the bearing resistances obtained from the geotechnical design report at both EQU and GEO ultimate limit states (see Figure 2). If the eccentricity is greater than L/6 at SLS, then the pressure distribution used to determine the settlement should be modified because tension cannot occur between the base and the soil. In this case the designer should satisfy himself that there will be no adverse consequences (e.g. excessive rotation of the base). It should also be noted that the ULS pressure distribution diagram will be rectangular and not trapezoidal.

Reinforced concrete pads

Where the pad foundations require reinforcement the following checks should be carried out to ensure:

- Sufficient reinforcement to resist bending moments.
- Punching shear strength.
- Beam shear strength.

The moments and shear forces should be assessed using the STR combination:

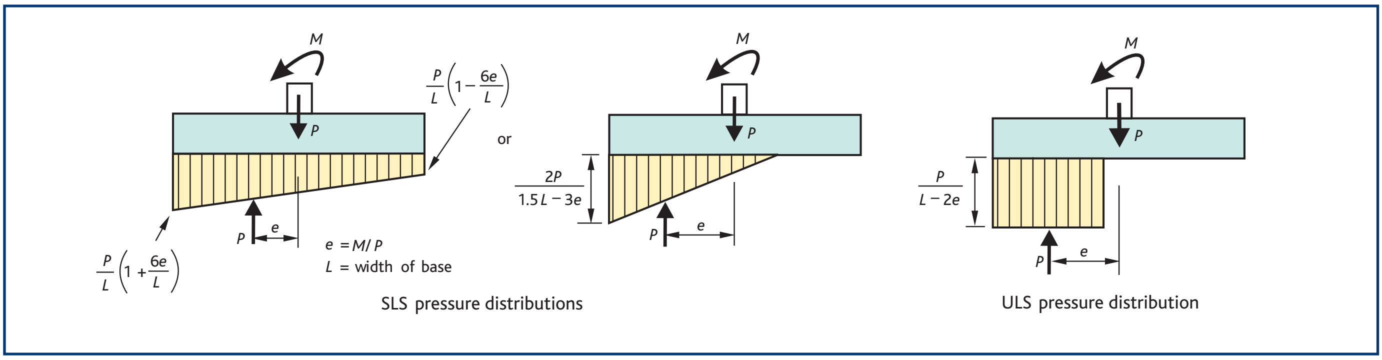
$1.35\ G_k + 1.5\ Q_k$ STR combination 1 (Exp. (6.10))

However, there may be economies to made from using Expressions (6.10a) or (6.10b) from the Eurocode.

The critical bending moments for design of bottom reinforcement are located at the column faces. Both beam shear and punching shear should then be checked at the locations shown in Figure 3. For punching shear the ground reaction within the perimeter may be deducted from the column load (Expression (6.48), Eurocode 2–1–1[6]). It is not usual for a pad foundation to contain shear reinforcement, therefore it is only necessary to ensure that the concrete shear stress capacity without shear reinforcement ($v_{Rd,c}$ – see Table 6) is greater than applied shear stress ($v_{Ed} = V_{Ed}/(bd)$).

If the basic shear stress is exceeded, the designer may increase the depth of the base. Alternatively, the amount of main reinforcement could be increased or, less desirably, shear links could be provided. (See Chapter 4, originally published as *Beams*[8] for an explanation of how to design shear reinforcement.)

Figure 2
Pressure distributions for pad foundations



Design for punching shear

Eurocode 2 provides specific guidance on the design of foundations for punching shear, and this varies from that given for slabs. In Eurocode 2 the shear perimeter has rounded corners and the forces directly resisted by the ground should be deducted (to avoid unnecessarily conservative designs). The critical perimeter should be found iteratively, but it is generally acceptable to check at d and $2d$. Alternatively, a spreadsheet could be used (e.g. spreadsheet TCC81 from *Spreadsheets for concrete design to BS 8110 and Eurocode 2*[7]). The procedure for determining the punching shear requirements is shown in Figure 4.

Table 6
$v_{Rd,c}$ resistance of members without shear reinforcement, MPa

ρ_l	Effective depth, d (mm)							
	300	400	500	600	700	800	900	1000[a]
0.25%	0.47	0.43	0.40	0.38	0.36	0.35	0.35	0.34
0.50%	0.54	0.51	0.48	0.47	0.45	0.44	0.44	0.43
0.75%	0.62	0.58	0.55	0.53	0.52	0.51	0.50	0.49
1.00%	0.68	0.64	0.61	0.59	0.57	0.56	0.55	0.54
1.25%	0.73	0.69	0.66	0.63	0.62	0.60	0.59	0.58
1.50%	0.78	0.73	0.70	0.67	0.65	0.64	0.63	0.62
1.75%	0.82	0.77	0.73	0.71	0.69	0.67	0.66	0.65
≥2.00%	0.85	0.80	0.77	0.74	0.72	0.70	0.69	0.68
k	1.816	1.707	1.632	1.577	1.535	1.500	1.471	1.447

Key

a For depths greater than 1000 calculate $v_{Rd,c}$ directly.

Notes

1 Table derived from: $v_{Rd,c} = 0.12\,k\,(100\rho_l\,f_{ck})^{(1/3)} \geq 0.035\,k^{1.5}\,f_{ck}^{0.5}$
where $k = 1 + \sqrt{(200/d)} \leq 2$ and $\rho_l = \sqrt{(\rho_{ly} + \rho_{lz})} \leq 0.02$,
$\rho_{ly} = A_{sy}/(bd)$ and $\rho_{lz} = A_{sz}/(bd)$

2 This table has been prepared for $f_{ck} = 30$;
where ρ_l exceed 0.40% the following factors may be used:

f_{ck}	25	28	32	35	40	45	50
Factor	0.94	0.98	1.02	1.05	1.10	1.14	1.19

Figure 3
Shear checks for pad foundations

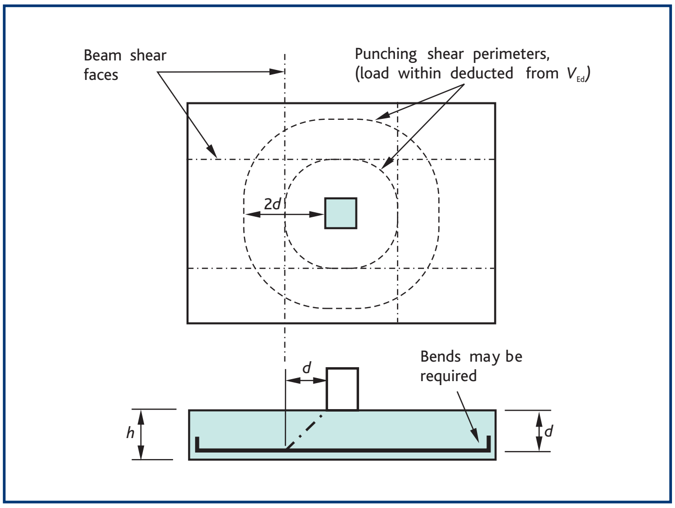


Figure 4
Procedure for determining punching shear capacity for pad foundations

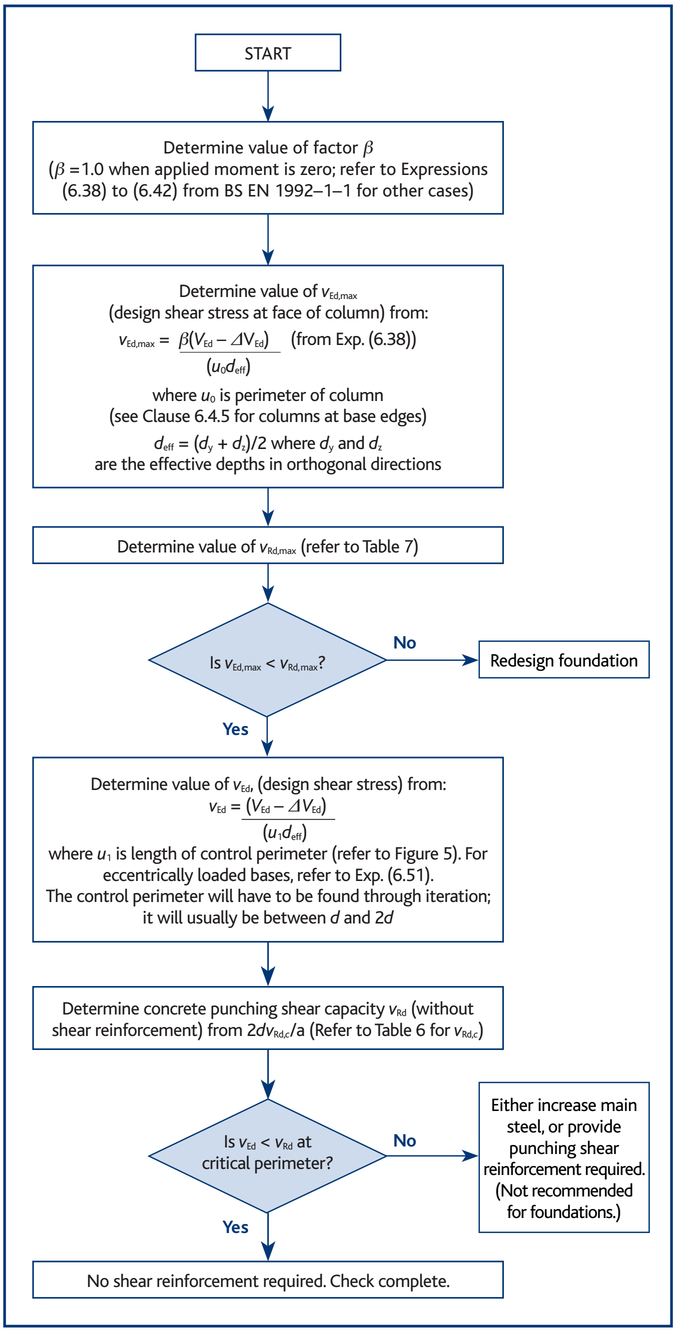
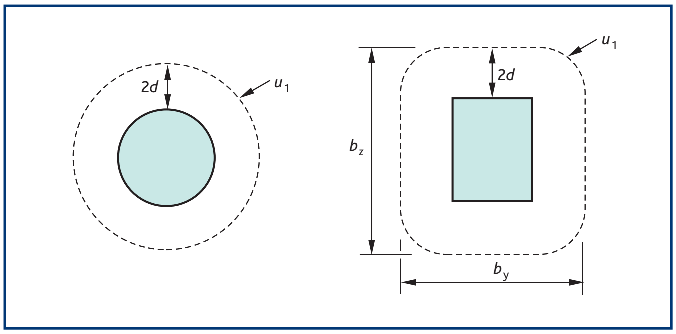


Figure 5
Typical basic control perimeters around loaded areas



Raft foundations

The basic design processes for rafts are similar to those for isolated pad foundations or pilecaps. The only difference in approach lies in the selection of an appropriate method for analysing the interaction between the raft and the ground so as to achieve a reasonable representation of their behaviour. For stiffer rafts (i.e. span-to-thickness greater than 10) with a fairly regular layout, simplified approaches such as yield line or the flat slab equivalent frame method may be employed, once an estimation of the variations in bearing pressure has been obtained from a geotechnical specialist. Whatever simplifications are made, individual elastic raft reactions should equate to the applied column loads.

Thinner, more flexible rafts or those with a complex layout may require the application of a finite element or grillage analysis. For rafts bearing on granular sub-grades or when contiguous-piled walls or diaphragm perimeter walls are present, the ground may be modelled as a series of Winkler springs. However, for cohesive sub-grades, this approach is unlikely to be valid, and specialist software will be required.

Piled foundations

For the purpose of this chapter it is assumed that the pile design will be carried out by a specialist piling contractor. The actions on the piles must be clearly conveyed to the pile designer, and these should be broken down into the unfactored permanent actions and each of the applicable variable actions (e.g. imposed and wind actions). The pile designer can then carry out the structural and geotechnical design of the piles.

Where moments are applied to the pilecap the EQU combination should also be used to check the piles can resist the overturning forces. These EQU loads must also be clearly conveyed to the pile designer and procedures put in place to ensure the piles are designed for the correct forces.

A pilecap may be treated as a beam in bending, where the critical bending moments for the design of the bottom reinforcement are located at the column faces. For further guidance on designing for flexure reference should be made to Chapter 4, originally published as *Beams*[8].

Alternatively, a truss analogy may be used; this is covered in Sections 5.6.4 and 6.5 of Eurocode 2–1–1. The strut angle θ should be at least 21.8° to the horizontal; note that θ should be measured in the plane of the column and pile.

Both beam shear and punching shear should then be checked as shown in Figure 6. For beam shear, the design resistances in Table 6 may be used. If the basic shear stress is exceeded, the designer should increase the depth of the base. Alternatively, the amount of main reinforcement could be increased or, less desirably, shear links could be provided. Care should be taken that main bars are fully anchored. As a minimum, a full anchorage should be provided from the inner face of piles. Large radius bends may be required.

When assessing the shear capacity in a pile cap, only the tension steel placed within the stress zone should be considered as contributing to the shear capacity (see Figure 7).

Table 7
Values for $v_{Rd, max}$

f_{ck}	$v_{Rd,max}$
20	3.68
25	4.50
28	4.97
30	5.28
32	5.58
35	6.02
40	6.72
45	7.38
50	8.00

Figure 6
Critical shear perimeters for piles

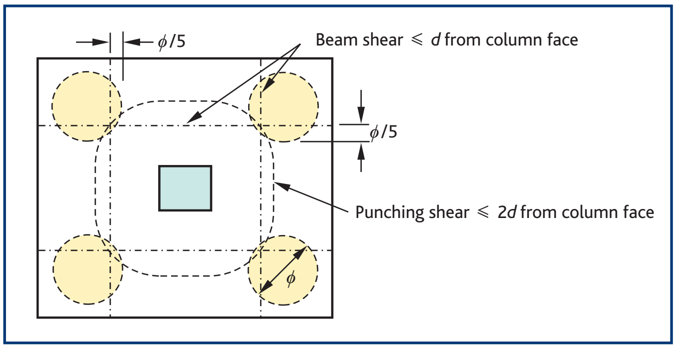


Figure 7
Shear reinforcement for pilecaps

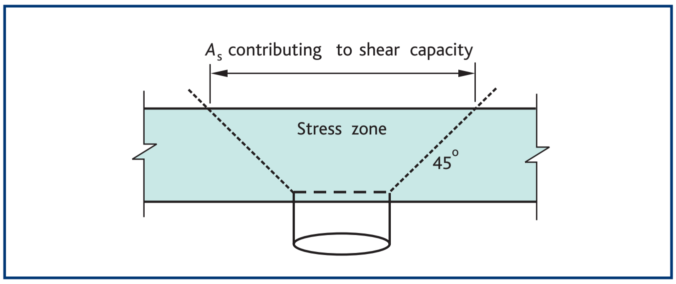
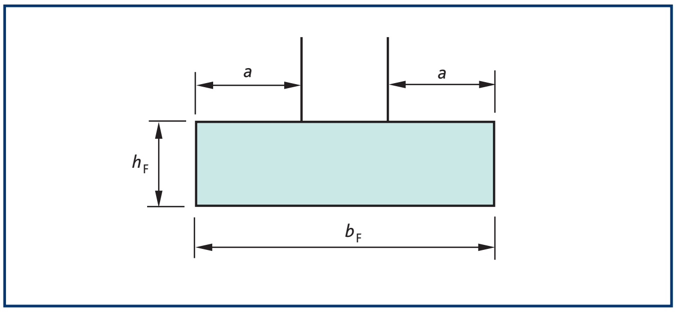


Figure 8
Dimensions for plain foundations



Plain concrete foundations

Strip and pad footings may be constructed from plain concrete provided the following rules are adhered to.

- In compression, the value of α_{cc}, the coefficient taking account of long-term effects applied to design compressive strength (see Cl. 3.1.6), should be taken as 0.6 as opposed to 0.85 for reinforced concrete.
- The minimum foundation depth, h_F, (see Figure 8) may be calculated from:

$$h_F \geq \frac{a}{0.85}\sqrt{\frac{9\sigma_{gd}}{f_{ctd}}}$$

where:

σ_{gd} = the design value of the ground bearing pressure
f_{ctd} = the design concrete tensile strength from Exp. (3.16)

For many situations this is unlikely to offer any savings over the current practice of designing for $h_f \geq a$.

The possibility of splitting forces, as advised in Clause 9.8.4 of Eurocode 2–1–1, may need to be considered.

Eurocode 2 allows plain concrete foundations to contain reinforcement for control of cracking.

Rules for spacing and quantity of reinforcement

Crack control

Refer to Chapter 2, originally published as *Getting started* [9].

Minimum area of principal reinforcement

The minimum area of reinforcement is $A_{s,min} = 0.26\, f_{ctm}\, b_t\, d/f_{yk}$ but not less than $0.0013 b_t d$ (see Table 8).

Maximum area of reinforcement

Except at lap locations, the maximum area of tension or compression reinforcement, should not exceed $A_{s,max} = 0.04\, A_c$

Minimum spacing of reinforcement

The minimum spacing of bars should be the greater of:

- Bar diameter,
- Aggregate size plus 5 mm, or
- 20 mm.

Deep elements

For deep elements the advice in Eurocode 2 for the side faces of deep beams may be followed. The UK National Annex recommends that 0.2% is provided in each face. The distance between bars should not exceed the lesser of twice the beam depth or 300 mm. For pile caps the side face may be unreinforced if there is no risk of tension developing.

Table 8
Minimum percentage of reinforcement required

f_{ck}	f_{ctm}	Minimum % ($0.26\, f_{ctm} / f_{yk}$[a])
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%

Key
a Where f_{yk} = 500 MPa.

Selected symbols

Symbol	Definition	Value
A_c	Cross sectional area of concrete	bh
A_s	Area of tension steel	
$A_{s,\,prov}$	Area of tension steel provided	
$A_{s,\,req'd}$	Area of tension steel required	
d	Effective depth	
d_{eff}	Average effective depth	$(d_y + d_z)$ /2
f_{cd}	Design value of concrete compressive strength	$\alpha_{cc}\, f_{ck}/\gamma_c$
f_{ck}	Characteristic cylinder strength of concrete	
f_{ctm}	Mean value of axial tensile strength	$0.30\, f_{ck}^{2/3}$ for $f_{ck} \leq$ C50/60 (from Table 3.1, Eurocode 2)
G_k	Characteristic value of permanent action	
h	Overall depth of the section	
l_{eff}	Effective span of member	See Section 5.3.2.2 (1)
M	Design moment at the ULS	
Q_k	Characteristic value of a variable action	
$Q_{k,w}$	Characteristic value of a variable wind action	
V_{Ed}	Design value of applied shear force	
v_{Ed}	Design value of applied shear stress	
$V_{Rd,c}$	Design value of the punching shear resistance without punching shear reinforcement	
$v_{Rd,c}$	Design value of the punching shear stress resistance without punching shear reinforcement	
$v_{Rd,max}$	Design value of the maximum punching shear resistance along the control section considered	
x	Depth to neutral axis	$(d - z)/0.4$
x_{max}	Limiting value for depth to neutral axis	$(\delta - 0.4)d$ where $\delta \leq 1.0$
z	Lever arm	
α_{cc}	Coefficient taking account of long term effects on compressive strength and of unfavourable effects resulting from the way load is applied (From UK National Annex)	0.85 for flexure and axial loads, 1.0 for other phenomena
β	Factor for determining punching shear stress	
δ	Ratio of the redistributed moment to the elastic bending moment	
γ_m	Partial factor for material properties	
ρ_0	Reference reinforcement ratio	$f_{ck}/1000$
ρ_l	Required tension reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_s/ bd
ψ_0	Factor for combination value of a variable action	
ψ_1	Factor for frequent value of a variable action	
ψ_2	Factor for quasi-permanent value of a variable action	

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How to design concrete structures using Eurocode 2

7. Flat slabs

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Designing to Eurocode 2

This chapter covers the analysis and design of concrete flat slabs to Eurocode 2[1], a process which is essentially the same as when using BS 8110[2]. However, the layout and content of Eurocode 2 may appear unusual to designers familiar with BS 8110. Eurocode 2 does not contain the derived formulae or specific guidance on determining moments and shear forces. This has arisen because it has been European practice to give principles in the codes and for the detailed application to be presented in other sources such as textbooks.

Chapter 1, originally published as *Introduction to Eurocodes*[3], highlighted the key differences between Eurocode 2 and BS 8110, including terminology.

It should be noted that values from the UK National Annex (NA) have been used throughout this publication, including values that are embedded in derived formulae (derivations can be found at www.eurocode2.info). A list of symbols related to flat slab design is given at the end of this chapter.

Analysis

Using Eurocode 2 for the analysis of flat slabs is similar to using BS 8110. The following methods may be used:

- Equivalent frame method
- Finite element analysis
- Yield line analysis
- Grillage analogy

The Eurocode gives further advice on the equivalent frame method in Annex I and designers used to BS 8110 will find this very familiar. Once the bending moments and shear forces have been determined, the following guidance can be used for the design of flat slabs.

Design procedure

A procedure for carrying out the detailed design of flat slabs is shown in Table 1. This assumes that the slab thickness has previously been determined during conceptual design. Concept designs prepared assuming detailed design would be to BS 8110 may be continued through to detailed design using Eurocode 2. More detailed advice on determining design life, loading, material properties, methods of analysis, minimum concrete cover for durability and bond, and control of crack widths can be found in Chapter 2, originally published as *Getting started*[4].

Fire resistance

Eurocode 2, Part 1–2: *Structural fire design*[5], gives a choice of advanced, simplified or tabular methods for determining the fire resistance. Using tables is the fastest method for determining the minimum dimensions and cover for flat slabs. There are, however, some restrictions and if these apply further guidance can be obtained from specialist literature[6].

Rather than giving a minimum cover, the tabular method is based on nominal axis distance, *a*. This is the distance from the centre of the reinforcing bar to the surface of the member. It is a nominal (not minimum) dimension, so the designer should ensure that

$a \geq c_{nom} + \phi_{link} + \phi_{bar}/2$

The requirements for flat slabs are given in Table 2.

Flexure

The design procedure for flexural design is given in Figure 1; this includes derived formulae based on the simplified rectangular stress block from Eurocode 2. Where appropriate Table 3 may be used to determine bending moments for flat slabs.

Table 1
Flat slab design procedure

Step	Task	Further guidance	
		Chapter in this publication	Standard
1	Determine design life	2: *Getting started*	NA to BS EN 1990 Table NA.2.1
2	Assess actions on the slab	2: *Getting started*	BS EN 1991 (10 parts) and National Annexes
3	Determine which combinations of actions apply	1: *Introduction to Eurocodes*	NA to BS EN 1990 Tables NA.A1.1 and NA.A1.2 (B)
4	Determine loading arrangements	2: *Getting started*	NA to BS EN 1992–1–1
5	Assess durability requirements and determine concrete strength	2: *Getting started*	BS 8500: 2002
6	Check cover requirements for appropriate fire resistance period	2: *Getting started* and Table 2	Approved Document B. BS EN 1992–1–1: Section 5
7	Calculate min. cover for durability, fire and bond requirements	2: *Getting started*	BS EN 1992–1–1 Cl 4.4.1
8	Analyse structure to obtain critical moments and shear forces	2: *Getting started* and Table 3	BS EN 1992–1–1 Section 5
9	Design flexural reinforcement	See Figure 1	BS EN 1992–1–1 Section 6.1
10	Check deflection	See Figure 3	BS EN 1992–1–1 Section 7.4
11	Check punching shear capacity	See Figure 6	BS EN 1992–1–1 Section 6.4
12	Check spacing of bars	2: *Getting started*	BS EN 1992–1–1 Section 7.3
13	Check resistance to moment transfer from column to slab	–	BS EN 1992–1–1 Annex I 1.2(5)

Note
NA = National Annex

Table 2
Minimum dimensions and axis distances for reinforced concrete slabs

Standard fire resistance	Minimum dimensions (mm)	
	Slab thickness, h_s	Axis distance, *a*
REI 60	180	15[a]
REI 90	200	25
REI 120	200	35
REI 240	200	50

Notes
1. This table is taken from BS EN 1992–1–2 Table 5.9.
2. The axis distance is to the centre of the outer layer of reinforcement.
3. The table is valid only if the detailing requirements (see note 4) are observed and, in the normal temperature design, redistribution of bending moments does not exceed 15%.
4. For fire resistance of R90 and above, at least 20% of the total top reinforcement in each direction over intermediate supports required by BS EN 1992–1–1 should be continuous over the full span. This reinforcement should be placed in the column strip.
5. There are three standard fire exposure conditions that may need to be satisfied:

   R Mechanical resistance for load bearing
   E Integrity of separation
   I Insulation

Key
a Normally the requirements of BS EN 1992–1–1 will determine the cover.

Figure 1
Procedure for determining flexural reinforcement

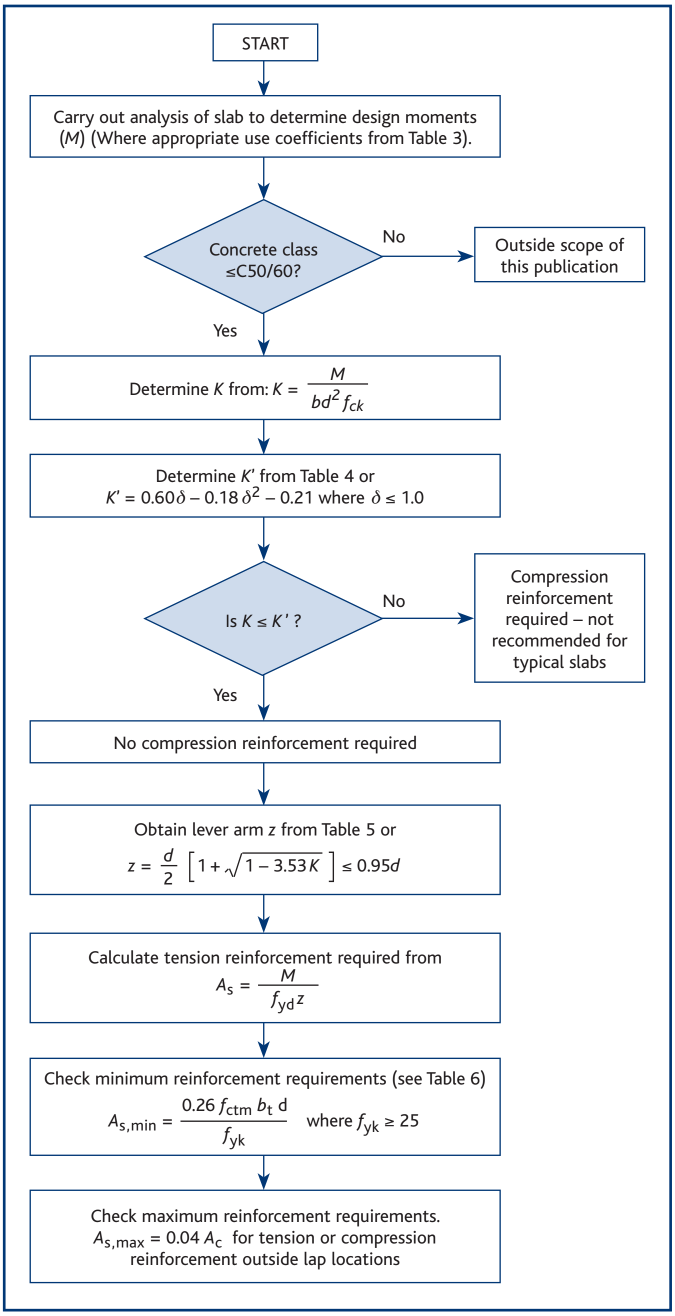


Table 3
Bending moment coefficients for flat slabs

	End support/slab connection				First interior support	Interior spans	Interior supports
	Pinned		Continuous				
	End support	End span	End support	End span			
Moment	0	$0.086Fl$	$-0.04Fl$	$0.075Fl$	$-0.086Fl$	$0.063Fl$	$-0.063Fl$

Notes

1. Applicable to slabs where the area of each bay exceeds 30 m², $Q_k \leq 1.25\,G_k$ and $q_k \leq 5$ kN/m²
2. F is the total design ultimate load, l is the effective span
3. Minimum span > 0.85 longest span, minimum 3 spans
4. Based on 20% redistribution at supports and no decrease in span moments

Whichever method of analysis is used, Cl. 9.4.1 requires the designer to concentrate the reinforcement over the columns. Annex I of the Eurocode gives recommendations for the equivalent frame method on how to apportion the total bending moment across a bay width into column and middle strips to comply with Cl. 9.4.1. Designers using grillage, finite element or yield line methods may also choose to follow the advice in Annex I to meet this requirement.

Eurocode 2 offers various methods for determining the stress-strain relationship of concrete. For simplicity and familiarity the method presented here is the simplified rectangular stress block (see Figure 2), which is similar to that found in BS 8110.

The Eurocode gives recommendations for the design of concrete up to class C90/105. However, for concrete strength greater than class C50/60, the stress block is modified. It is important to note that concrete strength is based on the cylinder strength and not the cube strength (i.e. for class C28/35 the cylinder strength is 28 MPa, whereas the cube strength is 35 MPa).

Table 4
Values for K'

% redistribution	δ (redistribution ratio)	K'
0	1.00	0.208[a]
10	0.90	0.182[a]
15	0.85	0.168
20	0.80	0.153
25	0.75	0.137
30	0.70	0.120

Key
a It is often recommended in the UK that K' should be llimited to 0.168 to ensure ductile failure

Table 5
z/d for singly reinforced rectangular sections

K	z/d	K	z/d
≤ 0.05	0.950[a]	0.13	0.868
0.06	0.944	0.14	0.856
0.07	0.934	0.15	0.843
0.08	0.924	0.16	0.830
0.09	0.913	0.17	0.816
0.10	0.902	0.18	0.802
0.11	0.891	0.19	0.787
0.12	0.880	0.20	0.771

Key
a Limiting z to $0.95d$ is not a requirement of Eurocode 2, but is considered to be good practice

Table 6
Minimum percentage of reinforcement required

f_{ck}	f_{ctm}	Minimum % ($0.26\, f_{ctm} / f_{yk}$[a])
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%

Key
a Where f_{yk} = 500 MPa

Deflection

Eurocode 2 has two alternative methods of designing for deflection; either by limiting span-to-depth ratio or by assessing the theoretical deflection using the Expressions given in the Eurocode. The latter is dealt with in detail in Chapter 8, originally published as *Deflection calculations*[7].

The span-to-depth ratios should ensure that deflection is limited to span/250 and this is the procedure presented in Figure 3. The *Background paper to the UK National Annex*[8] notes that the span-to-depth ratios are appropriate where the structure remains propped during construction or until the concrete attains sufficient strength to support the construction loads. It can generally be assumed that early striking of formwork will not significantly affect the deflection after installing the cladding and/or partitions[9].

Figure 2
Simplified rectangular stress block for concrete up to class C50/60 from Eurocode 2

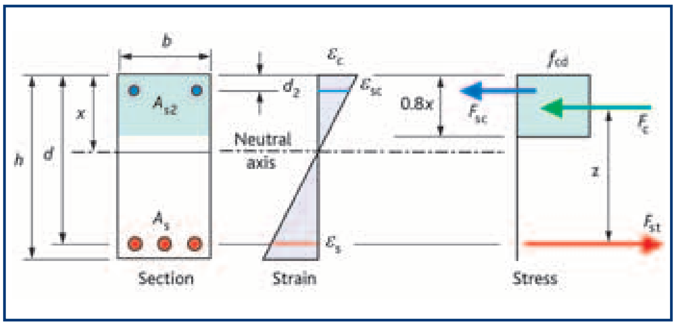
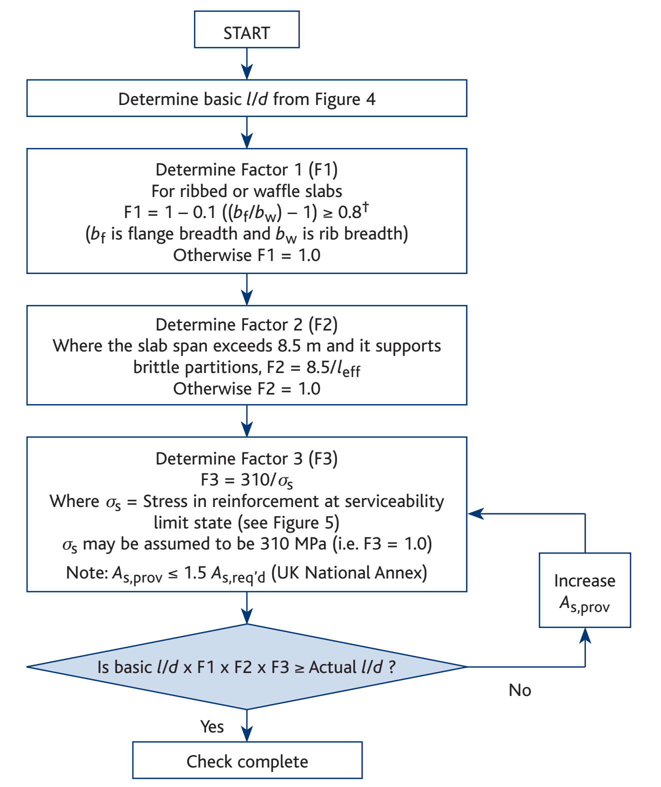


Figure 3
Procedure for assessing deflection



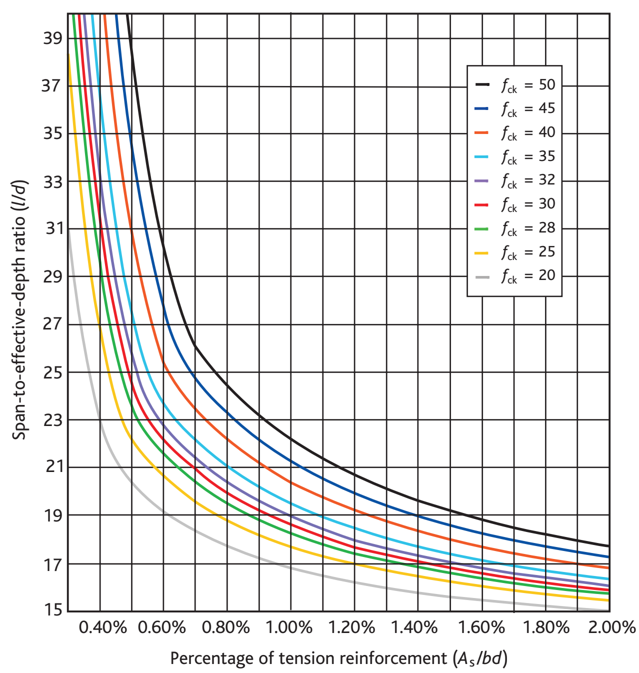
† The Eurocode is ambiguous regarding linear interpolation. It is understood that this was the intention of the drafting committee and is in line with current UK practice.

Punching shear

The design value of the punching shear force, V_{Ed}, will usually be the support reaction at the ultimate limit state. In principle the design for punching shear in Eurocode 2 and BS 8110 is similar. The main differences are as follows.

- Standard factors for edge and corner columns that allow for moment transfer (β) are greater in Eurocode 2. However, β can be calculated directly from Expressions (6.38) to (6.46) of the Eurocode to give more efficient designs.

Figure 4
Basic span-to-effective-depth ratios for flat slabs



Notes

1 This graph assumes a simply supported flat slab (K = 1.2).

2 Compression reinforcement, ρ', has been taken as 0.

3 Curves based on the following expressions:

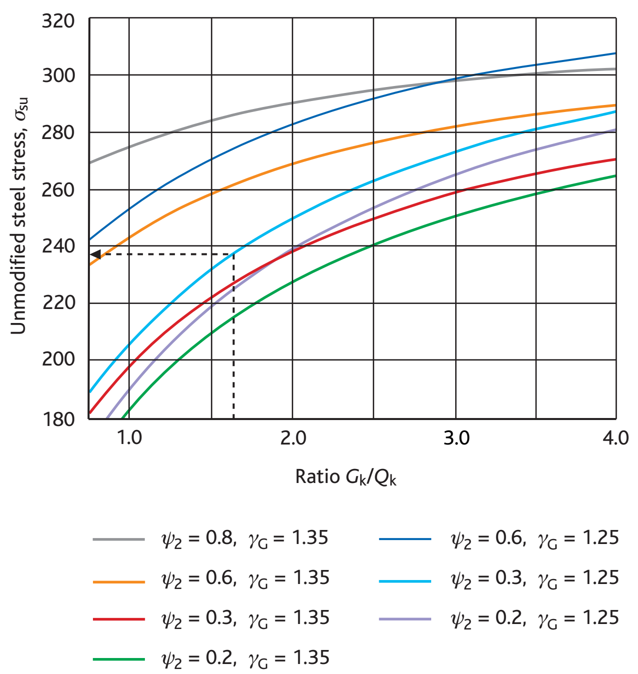
$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{1.5}\right] \quad \text{where } \rho \le \rho_0 \text{ and}$$

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}\,\rho_0}{(\rho - \rho')} + \frac{\sqrt{f_{ck}}}{12}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{where } \rho > \rho_0.$$

- In Eurocode 2 the maximum value of shear at the column face is not limited to 5 MPa, and depends on the concrete strength used.
- With Eurocode 2 the permissible shear resistance when using shear links is higher, although such designs may not be economic or desirable.
- The control perimeters for rectangular columns in Eurocode 2 have rounded corners.
- Where shear reinforcement is required the procedure in Eurocode 2 is simpler; the point at which no shear reinforcement is required can be calculated directly and then used to determine the extent of the area over which shear reinforcement is required.
- It is assumed that the reinforcement will be in a radial arrangement. However, the reinforcement can be laid on a grid provided the spacing rules are followed.

The procedure for determining the punching shear requirements is shown in Figure 6.

Figure 5
Determination of steel stress



To determine stress in the reinforcement (σ_s), calculate the ratio G_k/Q_k, read up the graph to the appropriate curve and read across to determine σ_{su}.

σ_s can be calculated from the expression: $\sigma_s = \sigma_{su} \left(\frac{A_{s,req}}{A_{s,prov}}\right)\left(\frac{1}{\delta}\right)$

ψ_2 is the factor for quasi-permanent value of a variable action. For further explanation refer to *How to design concrete structures using Eurocode 2: Introduction to Eurocodes*[3].

As an alternative to using shear links, proprietary shear stud rails may be used. Eurocode 2 (Figure 6.22) allows them to be laid out in a radial or cruciform pattern and gives spacing requirements for both. Other techniques are available for increasing punching shear resistance and these are covered in a best practice guide[10].

Figure 6
Procedure for determining punching shear capacity

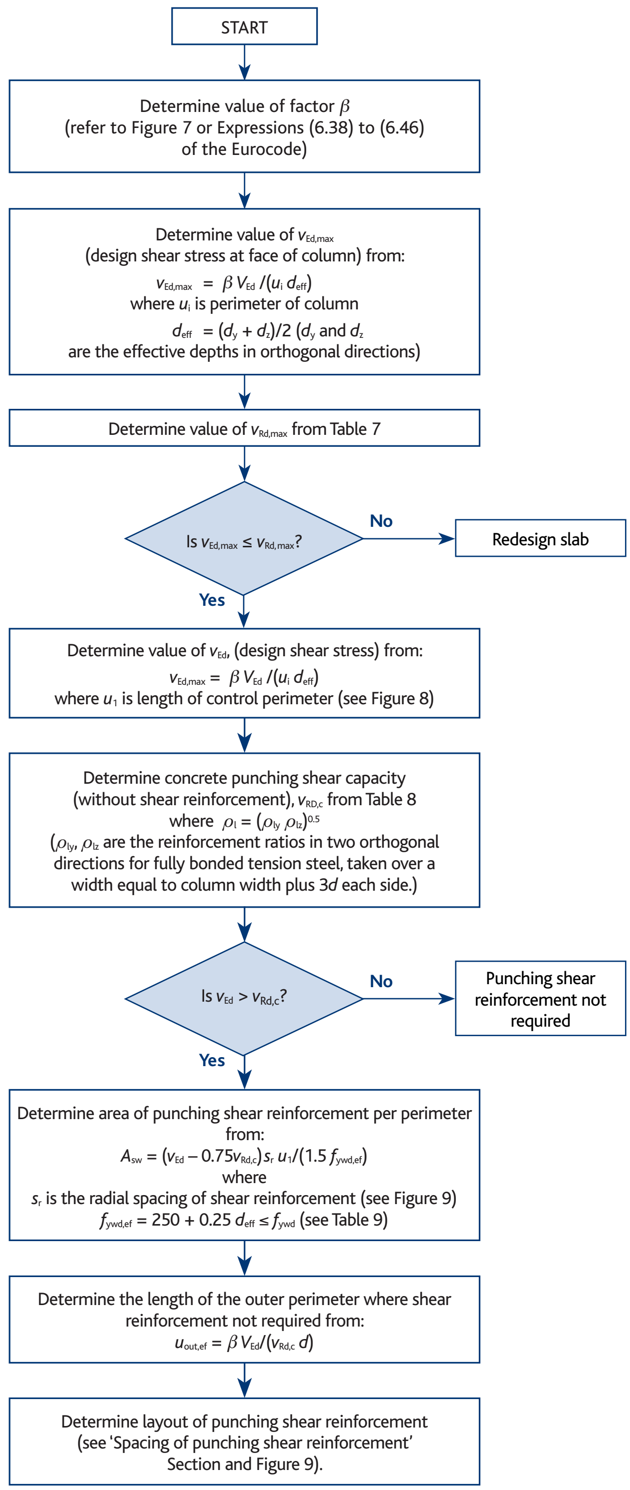


Table 7
Values for $v_{Rd,max}$

f_{ck}	$v_{Rd,max}$
20	3.31
25	4.05
28	4.48
30	4.75
32	5.02
35	5.42
40	6.05
45	6.64
50	7.20

Table 9
Values for $f_{ywd,ef}$

d_{eff}	$f_{ywd,ef}$
150	288
175	294
200	300
225	306
250	313
275	319
300	325
325	331
350	338

Table 8
$v_{Rd,c}$ resistance of members without shear reinforcement, MPa

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

Notes

1 Table derived from: $v_{Rd,c} = 0.12\,k\,(100\rho_l\,f_{ck})^{1/3} \geq 0.035\,k^{1.5}\,f_{ck}^{0.5}$ where $k = 1 + \sqrt{(200/d)} \leq 2$ and $\rho_l = \sqrt{(\rho_{ly} + \rho_{lz})} \leq 0.02$, $\rho_{ly} = A_{sy}/(bd)$ and $\rho_{lz} = A_{sz}/(bd)$

2 This table has been prepared for $f_{ck} = 30$;
Where ρ_l exceeds 0.40% the following factors may be used:

f_{ck}	25	28	32	35	40	45	50
Factor	0.94	0.98	1.02	1.05	1.10	1.14	1.19

Figure 8
Typical basic control perimeters around loaded areas

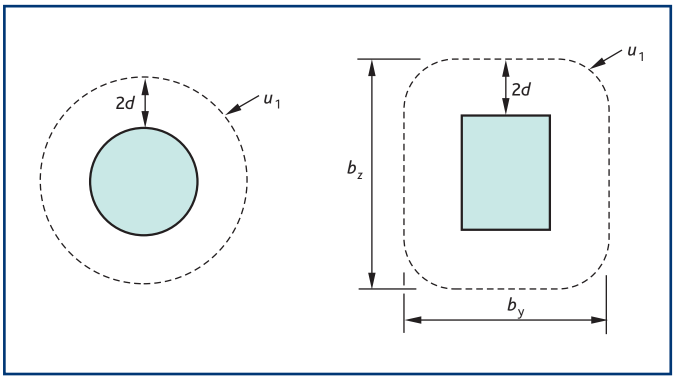
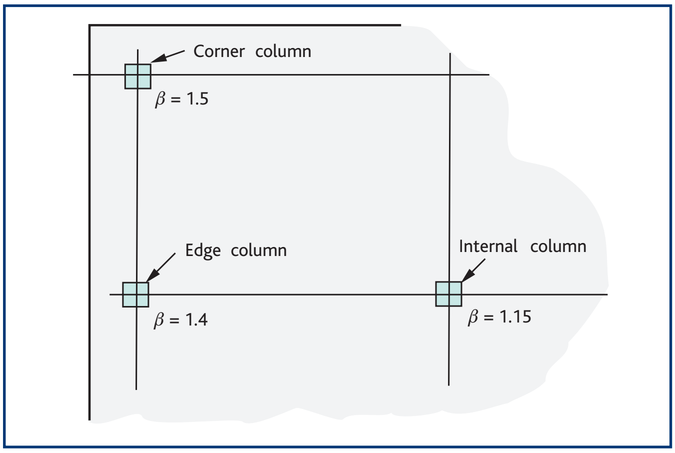


Figure 7
Recommended standard values for β



Rules for spacing and quantity of reinforcement

Minimum area of reinforcement

The minimum area of longitudinal reinforcement in the main direction is $A_{s,min} = 0.26\,f_{ctm}\,b_t\,d/f_{yk}$ but not less than $0.0013b\,d$ (see Table 6).

The minimum area of a link leg for vertical punching shear reinforcement is

$1.5A_{sw,min}/(s_r.s_t) \geq 0.08f_{ck}^{1/2}/f_{yk}$

which can be rearranged as

$A_{sw,min} \geq (s_r.s_t)/F$

where

s_r = the spacing of the links in the radial direction

s_t = the spacing of the links in the tangential direction

F can be obtained from Table 10

Maximum area of reinforcement

Outside lap locations, the maximum area of tension or compression reinforcement should not exceed $A_{s,max} = 0.4\,A_c$

Minimum spacing of reinforcement

The minimum spacing of bars should be the greater of:

- Bar diameter
- Aggregate size plus 5 mm
- 20 mm

Maximum spacing of main reinforcement

For slabs less than 200 mm thick the following maximum spacing rules apply:

- For the principal reinforcement: 3*h* but not more than 400 mm
- For the secondary reinforcement: 3.5*h* but not more than 450 mm

The exception is in areas with concentrated loads or areas of maximum moment where the following applies:

- For the principal reinforcement: 2*h* but not more than 250 mm
- For the secondary reinforcement: 3*h* but not more than 400 mm
  Where *h* is the depth of the slab.

For slabs 200 mm thick or greater, the bar size and spacing should be limited to control the crack width and reference should be made to Section 7.3.3 of the Eurocode or Chapter 2, originally published as *Getting started.*[4]

Spacing of punching shear reinforcement

Where punching shear reinforcement is required the following rules should be observed.

- It should be provided between the face of the column and *kd* inside the outer perimeter where shear reinforcement is no longer required. *k* is 1.5, unless the perimeter at which reinforcement is no longer required is less than 3*d* from the face of the column. In this case the reinforcement should be placed in the zone 0.3*d* to 1.5*d* from the face of the column.
- There should be at least two perimeters of shear links.
- The radial spacing of the links should not exceed 0.75*d* (see Figure 9).
- The tangential spacing of the links should not exceed 1.5*d* within 2*d* of the column face.
- The tangential spacing of the links should not exceed 2*d* for any other perimeter.
- The distance between the face of the column and the nearest shear reinforcement should be less than 0.5*d*.

Figure 9
Punching shear layout

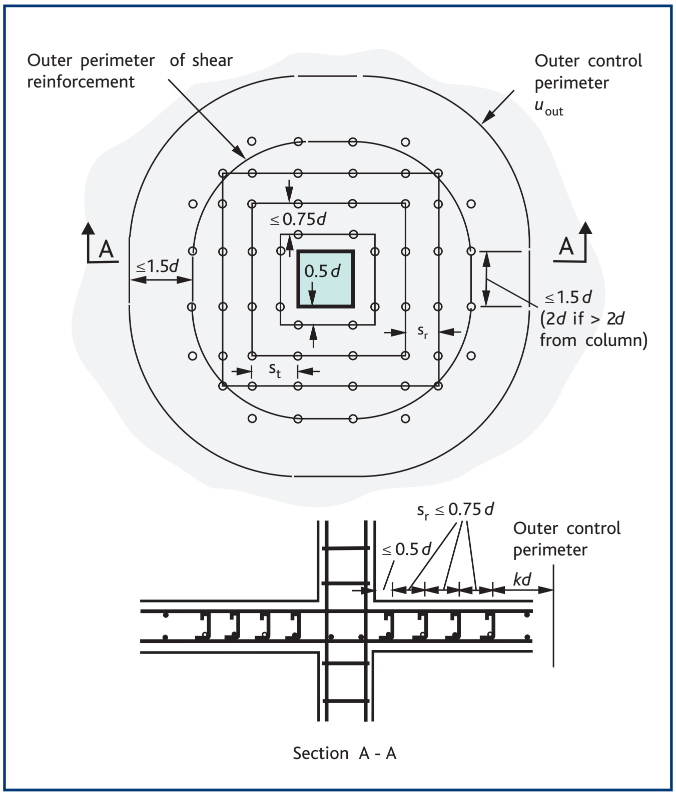


Table 10
Factor, *F*, for determining $A_{sw,min}$

f_{ck}	Factor, *F*
25	1875
28	1772
30	1712
32	1657
35	1585
40	1482
45	1398
50	1326

Note
f_{ck} has been taken as 500 MPa

Selected symbols

Symbol	Definition	Value
A_c	Cross sectional area of concrete	*bh*
A_s	Area of tension steel	
A_{s2}	Area of compression steel	
$A_{s, prov}$	Area of tension steel provided	
$A_{s, req'd}$	Area of tension steel required	
b	Width of slab	
d	Effective depth	
d_2	Effective depth to compression reinforcement	
f_{cd}	Design value of concrete compressive strength	$\alpha_{cc} f_{ck}/\gamma_c$
f_{ck}	Characteristic cylinder strength of concrete	
f_{ctm}	Mean value of axial tensile strength	$0.30 f_{ck}^{2/3}$ for $f_{ck} \leq$ C50/60 (from Table 3.1, Eurocode 2)
h_s	Slab thickness	
K	Factor to take account of the different structural systems	See Table N 7.4 in UK National Annex
l_{eff}	Effective span of member	See Section 5.3.2.2 (1)
l/d	Limiting span-to-depth ratio	
M	Design moment at the ULS	
x	Depth to neutral axis	$(d - z)/0.4$
x_{max}	Limiting value for depth to neutral axis	$(\delta - 0.4)d$ where $\delta \leq 1.0$
z	Lever arm	
α_{cc}	Coefficient taking account of long term effects on compressive strength and of unfavourable effects resulting from the way load is applied	0.85 for flexure and axial loads. 1.0 for other phenomena (From UK National Annex)
δ	Ratio of the redistributed moment to the elastic bending moment	
γ_m	Partial factor for material properties	1.15 for reinforcement (γ_s) 1.5 for concrete (γ_c)
ρ_0	Reference reinforcement ratio	$\sqrt{f_{ck}}/1000$
ρ	Required tension reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_s/bd
ρ'	Required compression reinforcement at mid-span to resist the moment due to the design loads (or at support for cantilevers)	A_{s2}/bd

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How to design concrete structures using Eurocode 2

8. Deflection calculations

R Webster CEng, FIStructE **O Brooker** BEng, CEng, MICE, MIStructE

Methods for checking deflection

This chapter describes the use of Eurocode 2[1] to check deflection by calculation. The alternative method for complying with the code requirements is to use the deemed-to-satisfy span-to-effective-depth ratios, which are appropriate and economic for the vast majority of designs. Further guidance on the span-to-effective-depth method is given in Chapters 3, 4 and 7, originally published as *Beams*[2], *Slabs*[3] and *Flat slabs*[4]. However, there are situations where direct calculation of deflection is necessary, as listed below:

- When an estimate of the deflection is required.
- When deflection limits of span/250 for quasi-permanent actions (see reference 5 for Eurocode terminology) or span/500 for partition and/or cladding loads are not appropriate.
- When the design requires a particularly shallow member, direct calculation of deflection may provide a more economic solution.
- To determine the effect on deflection of early striking of formwork or of temporary loading during construction.

Overview

In the past structures tended to be stiff with relatively short spans. As technology and practice have advanced, more flexible structures have resulted. There are a number of reasons for this, including:

- The increase in reinforcement strength leading to less reinforcement being required for the ultimate limit state (ULS) and resulting in higher service stresses in the reinforcement.
- Increases in concrete strength resulting from the need to improve both durability and construction time, and leading to concrete that is more stiff and with higher service stresses.

What affects deflection?

There are numerous factors that affect deflection. These factors are also often time-related and interdependent, which makes the prediction of deflection difficult.

The main factors are:

- **Concrete tensile strength**
- **Creep**
- **Elastic modulus**

Other factors include:

- **Degree of restraint**
- **Magnitude of loading**
- **Time of loading**
- **Duration of loading**
- **Cracking of the concrete**
- **Shrinkage**
- **Ambient conditions**
- **Secondary load-paths**
- **Stiffening by other elements**

- A greater understanding of structural behaviour and the ability to analyse that behaviour quickly by computer.
- The requirement to produce economic designs for slabs whose thicknesses are typically determined by the serviceability limit state (SLS) and which constitute 80% to 90% of the superstructure costs.
- Client requirements for longer spans and greater operational flexibility from their structures.

Factors affecting deflection

An accurate assessment of deflection can only be achieved if consideration is given to the factors that affect it. The more important factors are discussed in detail below.

Tensile strength

The tensile strength of concrete is an important property because the slab will crack when the tensile stress in the extreme fibre is exceeded. In Eurocode 2 the concrete tensile strength, $f_{ctm,}$ is a mean value (which is appropriate for deflection calculations) and increases as the compressive strength increases. This is an advancement when compared with BS 8110 where the tensile strength is fixed for all concrete strengths.

The degree of restraint to shrinkage movements will influence the effective tensile strength of the concrete. A layout of walls with high restraint will decrease the effective tensile strength. Typical examples of wall layouts are given in Figure 1. For a low restraint layout the following expression may be used for the concrete tensile strength:

$f_{ctm,fl} = (1.6 - h/1000)f_{ctm} > f_{ctm}$

where

$f_{ctm,fl}$ = Mean flexural tensile strength of reinforced concrete

f_{ctm} = Mean tensile strength of concrete

It is often recommended that the design value of the concrete tensile strength for a low restraint layout is taken as the average of $f_{ctm,fl}$ and f_{ctm}, to allow for unintentional restraint. For high restraint f_{ctm} should be used.

Creep

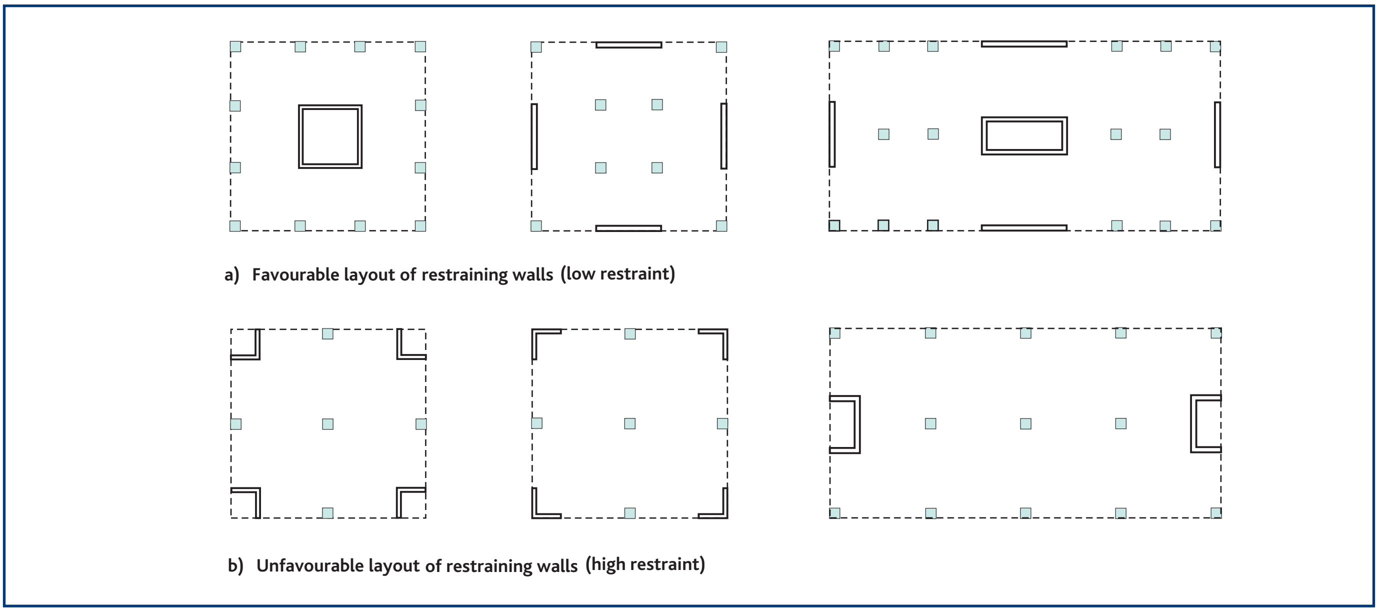
Creep is the time-dependant increase in compressive strain in a concrete element under constant compressive stress. Creep is usually considered in the design by modifying the elastic modulus using a creep coefficient, φ, which depends on the age at loading, size of the member and ambient conditions, in particular relative humidity. Eurocode 2 gives advice on the calculation of creep coefficients in detail in Annex B. It also advises on the appropriate relative humidity to use in Figure 3.1.

The cement strength class is required in the assessment of creep, however, at the design stage it is often not clear which class should be used. Generally, Class R should be assumed. Where the ground granulated blastfurnace slag (ggbs) content exceeds 35% of the cement combination or where fly ash (pfa) exceeds 20% of the cement combination, Class N may be assumed. Where ggbs exceeds 65% or where pfa exceeds 35% Class S may be assumed.

Elastic modulus

The elastic modulus of concrete is influenced by aggregate type, workmanship and curing conditions. The effective elastic modulus under sustained loading will be reduced over time due to the effect of creep. These factors mean that some judgement is required to determine an appropriate elastic modulus. Eurocode 2 gives recommended values for the 28-day secant modulus, E_{cm}, (in Table 3.1) and makes recommendations for adjustments to these values to account for different types of aggregate. The long-term elastic modulus should be taken as:

Figure 1
Typical floor layouts



$E_{c,LT} = E_{c28}/(1 + \varphi)$

where

E_{c28} = 28-day tangent modulus = 1.05 E_{cm}

φ = Creep factor. (Note that with Eurocode 2, φ relates to a 28-day short-term elastic modulus, whereas a 'true' creep factor would be associated with the modulus at the age of loading.)

The assessment of the long-term E-value can be carried out more accurately after the contractor has been appointed because they should be able to identify the concrete supplier (and hence the type of aggregates) and also the construction sequence (and hence the age at first loading).

Loading sequence

The loading sequence and timing may be critical in determining the deflection of a suspended slab because it will influence the point at which the slab will crack (if at all) and is used to calculate the creep factors for the slab. A loading sequence is shown in Figure 2, which shows that in the early stages relatively high loads are imposed while casting the slab above. The loading sequence may vary, depending on the construction method.

Smaller loads are imposed when further slabs are cast above. The loads are then increased permanently by the application of the floor finishes and erection of the partitions. Finally, the variable actions are applied to the structure and, for the purpose of deflection calculation, the quasi-permanent combination should be used. (See Chapter 1, originally published as *Introduction to Eurocodes*[5] for further information on combinations of actions.) However, it is likely that the quasi-permanent combination will be exceeded during the lifetime of the building and, for the purpose of determining whether the slab might have cracked, the frequent combination may be critical.

Commercial pressures often lead to a requirement to strike the formwork as soon as possible and move on to subsequent floors, with the minimum of propping. Tests on flat slabs have demonstrated that as much as 70% of the loads from a newly cast floor (formwork, wet concrete, construction loads) may be carried by the suspended floor below[7]. It can generally be assumed that early striking of formwork will not greatly affect the deflection after installing the cladding and/or partitions. This is because the deflection affecting partitions will be smaller if the slab becomes 'cracked' before, rather than after, the installation of the cladding and/or partitions.

Cracking

Deflection of concrete sections is closely linked to the extent of cracking and the degree to which cracking capacity is exceeded. The point at which cracking occurs is determined by the moments induced in the slab and the tensile strength of the concrete, which increases with age. Often the critical situation is when the slab is struck, or when the load of the slab above is applied. Once the slab has cracked its stiffness is permanently reduced.

It is therefore necessary to find the critical loading stage at which cracking first occurs. This critical loading stage corresponds with the minimum value of K, where:

$$K = f_{ctm} \Big/ \left(W\sqrt{0.5}\right)$$

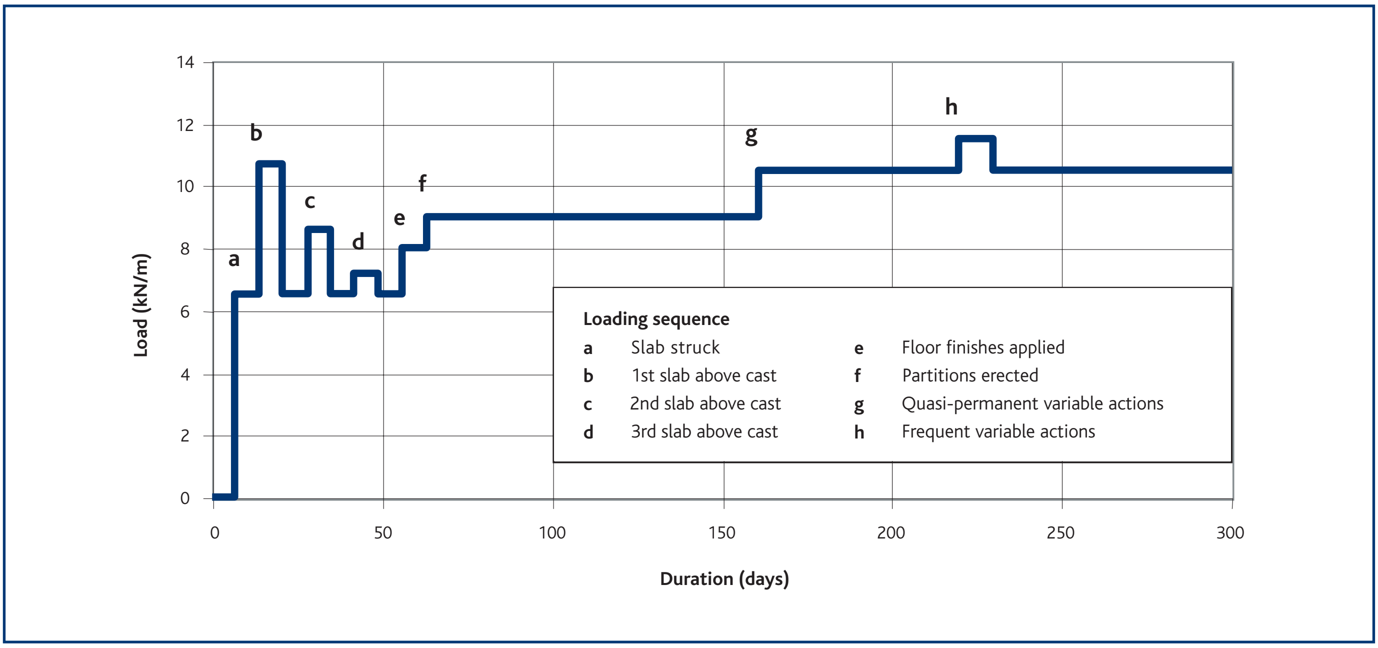
where

W = The serviceability loading applied up to that stage

f_{ctm} = The concrete tensile strength at that stage

Where the frequent combination is the critical load stage, then the degree of cracking (ζ) calculated for the frequent combination should also be used for the quasi-permanent combination, but not for

Figure 2
Loading history for a slab – an example



any of the earlier load stages. If, however, an earlier stage proves critical, the ζ value at that stage should be carried forward to all subsequent stages.

Further information can be found in the best practice guide *Early striking and improved backpropping*[6].

Shrinkage curvature

Shrinkage depends on the water/cement ratio, relative humidity and the size and shape of the member. The effect of shrinkage in an asymmetrically reinforced section is to induce a curvature that can lead to significant deflection in shallow members. This effect should be considered in the deflection calculations.

Methods for calculating deflections

Two methods for calculating deflection are presented below, and these are based on the advice in TR58 *Deflections in concrete slabs and beams*[8].

Rigorous method

The rigorous method for calculating deflections is the most appropriate method for determining a realistic estimate of deflection. However, it is only suitable for use with computer software. The Concrete Centre has produced a number of spreadsheets that use this method to carry out deflection calculations for a variety of slabs and beams[9]. These offer a cost-effective way to carry out detailed deflection calculations, and they include the ability to consider the effect of early age loading of the concrete. Figure 3 illustrates the principles of the method and shows how the factors affecting deflection are considered in the rigorous deflection calculations.

Finite element analysis may also be used to obtain estimates of deflection. In this case the principles in Figure 3 should be applied if credible results are to be obtained.

Panel 1
Determining long term elastic modulus of elasticity

Calculate long-term elastic modulus, E_{LT} from:

$$E_{LT} = \Sigma W \Big/ \left(\frac{W_1}{E_{eff,1}} + \frac{W_2}{E_{eff,2}} + \frac{W_3}{E_{eff,3}} + \frac{W_4}{E_{eff,4}} + \frac{W_5}{E_{eff,5}} \right)$$

where

$E_{eff} = E_{c28}/(1+\varphi)$

W_n = Serviceability load at stage n

φ = Creep coefficient at relevant loading time and duration

Figure 3
Outline of rigorous method for calculating deflection

Repeat at 1/20 points for all three loading stages

Collate input data

- Element dimensions and reinforcement details and arrangements from the ultimate limit state design
- Loading sequence e.g.
  - Striking the formwork
  - Casting the floor above
  - Erection of the partitions and/or cladding
  - Application of finishes

  The sequence will vary from project to project
- Concrete properties (see Table 1)
  - Mean compressive strength (f_{cm})
  - Mean tensile strength (f_{ctm} or $f_{ctm,fl}$)
  - Elastic modulus (E_{c28}) = 1.05 E_{cm}
- Critical arrangement of actions
  (or repeat the calculations for each arrangement to determine the critical case)

↓

Assess whether the element has flexural cracking

- Determine the critical load stage at which cracking first occurs. (See 'Cracking' on page 3)
- Calculate the following properties:
  - Creep coefficients, φ (Annex B of Eurocode 2 or Figure 4)
  - Long term elastic modulus, E_{LT} (see Panel 1)
  - Effective modulus ratio, α_e from: $\alpha_e = E_s/E_{LT}$
  - Neutral axis depth for uncracked condition, x_u (see Panel 2)
  - Second moment of area for uncracked condition, I_u (see Panel 2)
  - Calculate cracking moment, M_{cr} from:
    $M_{cr} = f_{ctm} I_u/(h - x_u)$, using appropriate value for f_{ctm}.
- Does the moment at the critical load stage exceed the cracking moment?
  - If yes, the element is cracked at all subsequent stages.
    $\zeta = 1 - 0.5(M_{cr}/M)^2$ [$\zeta = 0$ for uncracked situation]
    Use these critical values of f_{ctm} and ζ for subsequent stages.
  - If no, the element will not crack at any stage.

↓

Determine the curvature of the slab

- When the slab is cracked calculate the following properties at the load stage being considered, using appropriate values for f_{ctm}, ζ and E_{LT}:
  - Neutral axis depth for cracked section, x_c (see Panel 2)
  - Second moment of area for cracked condition, I_c (see Panel 2)
- Calculate the flexural curvature:

$$\frac{1}{r_{fl}} = \zeta \frac{M_{QP}}{E_e I_c} + (1 - \zeta) \frac{M_{QP}}{E_e I_u}$$

- Calculate the curvature due to shrinkage strain $1/r_{cs}$ (see Panel 2)
- Calculate the total curvature, $1/r_t = 1/r_{fl} + 1/r_{cs}$

↓

Repeat the calculations at frequent intervals (say at 1/20 points) and integrate twice to obtain the overall deflection.

↓

If deflection affecting cladding and/or partitions is required, repeat calculations for frequent combination and for loading at time of installation of partitions and/or cladding.

↓

Estimate deflections:

- Overall deflection (quasi-permanent combination)
- Deflection affecting partitions/cladding (Frequent combination deflection less deflection at time of installation)

Table 1

Concrete properties

f_{ck}	MPa	20	25	28	30	32	35	40	50
$f_{cm} = (f_{ck} + 8)$	MPa	28	33	36	38	40	43	48	58
$f_{ctm} = (0.3\, f_{ck}^{(2/3)} \leq$ C50/60 or $2.12 \ln(1 + (f_{cm}/10)) >$ C50/60)	MPa	2.21	2.56	2.77	2.90	3.02	3.21	3.51	4.07
$f_{ctm}^{*} = (0.3\, f_{cm}^{(2/3)} \leq$ C50/60 or $1.08 \ln(f_{cm}) + 0.1 >$ C50/60)[a]	MPa	2.77	3.09	3.27	3.39	3.51	3.68	3.96	4.50
$E_{cm} = (22\,[(f_{cm})/10]^{0.3}$	GPa	30.0	31.5	32.3	32.8	33.3	34.1	35.2	37.3
$E_{c28} = (1.05\, E_{cm})$	GPa	31.5	33.0	33.9	34.5	35.0	35.8	37.0	39.1
$\varepsilon_{cd,0}$ CEM class R, RH = 50%	microstrain	746	706	683	668	653	632	598	536
$\varepsilon_{cd,0}$ CEM class R, RH = 80%	microstrain	416	394	381	372	364	353	334	299
$\varepsilon_{cd,0}$ CEM class N, RH = 50%	microstrain	544	512	494	482	471	454	428	379
$\varepsilon_{cd,0}$ CEM class N, RH = 80%	microstrain	303	286	275	269	263	253	239	212
$\varepsilon_{cd,0}$ CEM class S, RH = 50%	microstrain	441	413	397	387	377	363	340	298
$\varepsilon_{cd,0}$ CEM class S, RH = 80%	microstrain	246	230	221	216	210	202	189	166
$\varepsilon_{ca}(\infty)$	microstrain	25	38	45	50	55	63	75	100

Key

a f_{ctm}^{*} may be used when striking at less than 7 days or where construction overload is taken into account.

Panel 2

Useful Expressions for a rectangular section

$$x_u = \frac{\dfrac{bh^2}{2} + (\alpha_e - 1)(A_s d + A_{s2} d_2)}{bh + (\alpha_e - 1)(A_s + A_{s2})}$$

$$I_u = \frac{bh^3}{12} + bh\left(\frac{h}{2} - x_u\right)^2 + (\alpha_e - 1)\left[A_s(d - x_u)^2 + A_{s2}(x_u - d_2)^2\right]$$

$$x_c = \left\{\left[\left(A_s\alpha_e + A_{s2}(\alpha_e - 1)\right)^2 + 2b\left(A_s d\alpha_e + A_{s2} d_2(\alpha_e - 1)\right)\right]^{0.5} - \left(A_s\alpha_e + A_{s2}(\alpha_e - 1)\right)\right\}\Big/ b$$

$$I_c = \frac{bx_c^3}{3} + \alpha_e A_s(d - x_c)^2 + (\alpha_e - 1) A_{s2}(d_2 - x_c)^2$$

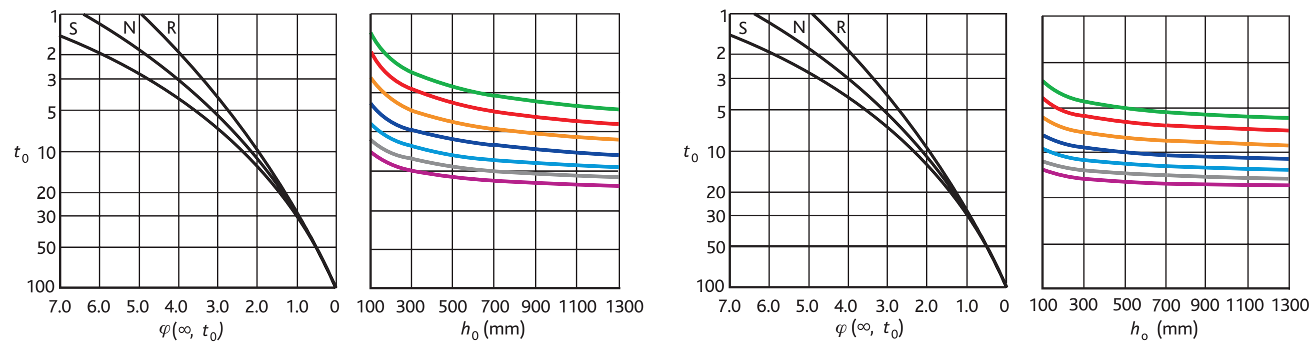
$$\frac{1}{r_{cs}} = \zeta\, \varepsilon_{cs}\alpha_e \frac{S_u}{I_u} + (1 - \zeta)\, \varepsilon_{cs}\, \alpha_e \frac{S_c}{I_c}$$

where

A_s = Area of tension reinforcement
A_{s2} = Area of compression reinforcement
b = Breadth of section
d = Effective depth to tension reinforcement
d_2 = Depth to compression reinforcement
h = Overall depth of section
α_e = Modular ratio
$S_u = A_s(d - x_u) - A_{s2}(x_u - d_2)$
$S_c = A_s(d - x_c) - A_{s2}(x_c - d_2)$

Figure 4

Method for determining creep coefficient $\varphi(\infty, t_0)$



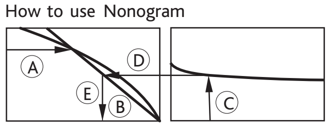
a) Inside conditions - RH = 50%

b) Outside conditions - RH = 80%

Key
- C20/25
- C25/30
- C30/37
- C35/45
- C40/50
- C45/55
- C50/60

Notes
1 t_0 = age of concrete at time of loading
2 $h_0 = 2A_c/u$
3 Intersection point between lines D & E can also be above point A
4 For t_0 > 100 it is sufficiently accurate to assume t = 100

How to use Nonogram



Simplified method

A simplified method for calculating deflection is presented in Figure 5. It is feasible to carry out these calculations by hand, and they could be used to roughly verify deflection results from computer software, or used where a computer is not available.

The major simplification is that the effects of early age loading are not considered explicitly; rather an allowance is made for their effect when calculating the cracking moment. Simplified creep factors are used and deflection from the curvature of the slab is approximated using a factor.

Figure 6
Values for K for various bending moment diagrams

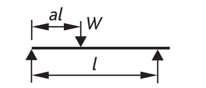
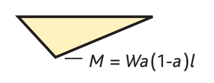
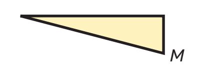
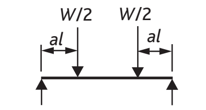
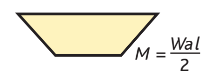
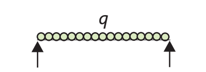
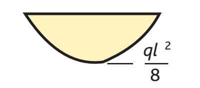
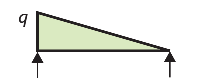
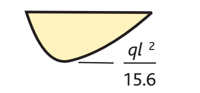
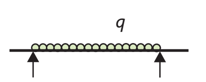
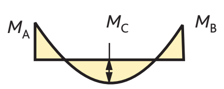
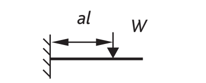
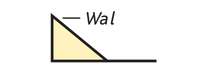
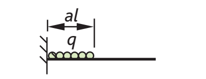
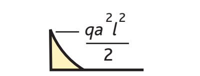
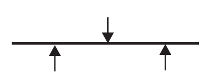
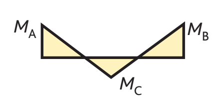
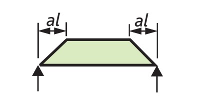
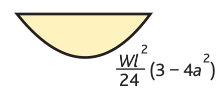
Loading	Bending moment diagram	K
		0.125
		$\frac{3-4a^2}{48(1-a)}$ If $a = \frac{1}{2}$, $K = \frac{1}{12}$
		0.0625
		$0.125 - \frac{a^2}{6}$
		0.104
		0.102
		$K = 0.104\ (1-\frac{\beta}{10})$ $\beta = \frac{M_A + M_B}{M_C}$
		End deflection $= \frac{a\ (3-a)}{6}$ load at end $K = 0.333$
		$\frac{a\ (4-a)}{12}$ if $a = l$, $K = 0.25$
		$K = 0.083\ (1-\frac{\beta}{4})$ $\beta = \frac{M_A + M_B}{M_C}$
		$\frac{1}{80}\frac{(5-4a^2)^2}{3-4a}$

Figure 5
Simplified method for calculating deflection

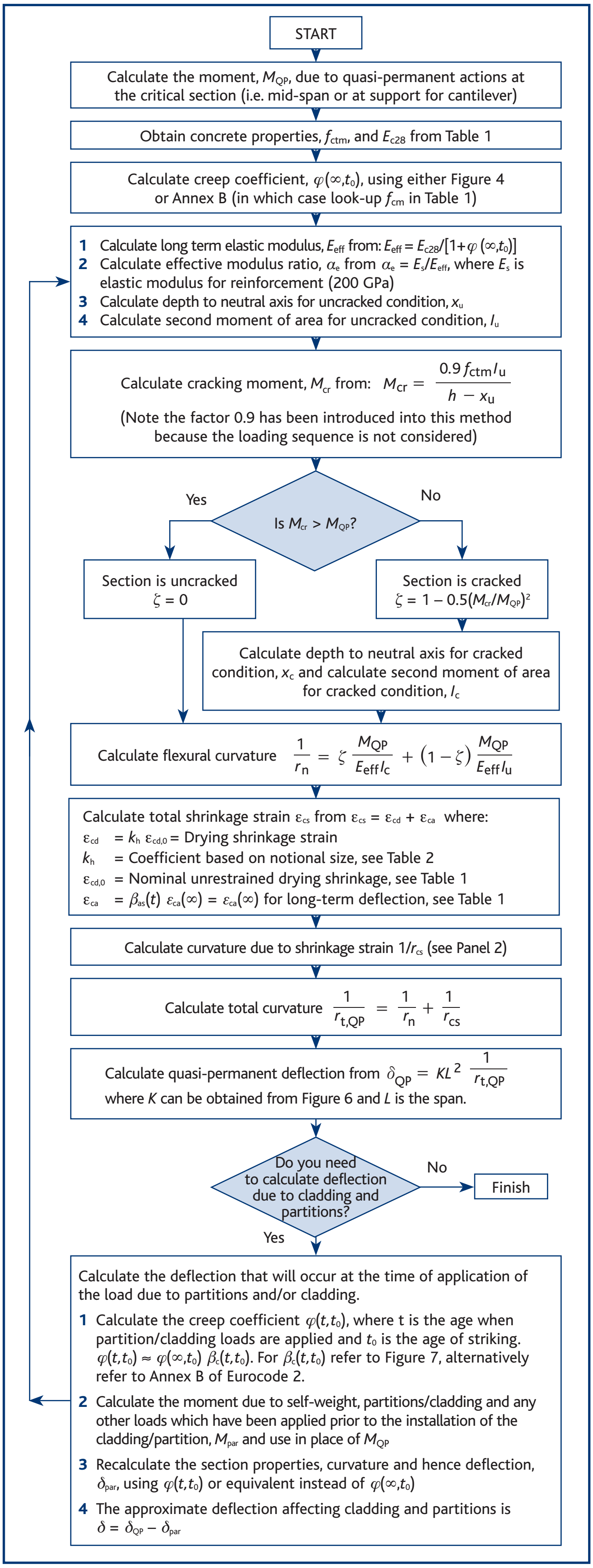
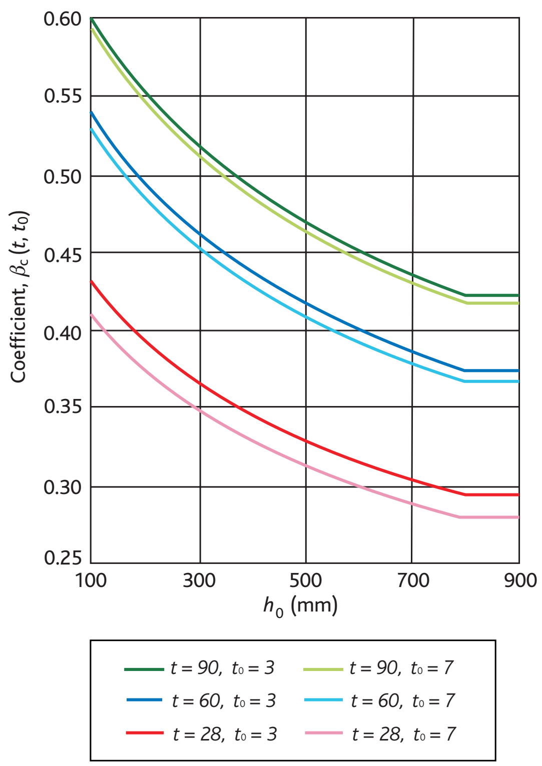


Table 2
Values for K_h

h_0	k_h
100	1.0
200	0.85
300	0.75
≥500	0.70

Notes
h_0 is the notional size (mm) of the cross-section = $2A_c/u$
where
A_c = Concrete cross-sectional area
u = Perimeter of that part of the cross section which is exposed to drying

Figure 7
Coefficient for development of creep with time after loading



Notes
t = Age of concrete when partitions/cladding applied
t_0 = Age of concrete when struck
f_{ck} = 30 (f_{cm} = 38), however the coefficient is not particularly sensitive to concrete class

Precamber

A slab or beam can be precambered to reduce the effect of deflection below the horizontal (see Figure 8). However, in practice too much precamber is generally used and the slab remains permanently cambered. This is because of the difficulty in accurately calculating deflection. A precamber of up to half the quasi-permanent combination deflection could be used, but a lower figure is recommended. Precamber does not reduce the deflections affecting partitions or cladding.

Flat slabs

Flat slabs are very popular and efficient floor systems. However, because they span in two directions, it can be difficult to calculate their deflection. TR58[8] gives several suitable methods for assessing flat slab deflection. Of these, a popular method is to take the average deflection of two parallel column strips and to add the deflection of the middle strip spanning orthogonally to get an approximation of the maximum deflection in the centre of the slab.

The recommended acceptance criteria for a flat slab are shown in Figure 9.

Accuracy

The calculation of deflection in Eurocode 2 using the rigorous method presented here is more advanced than that in BS 8110[10]. It can be used to take account of early-age construction loading by considering reduced early concrete tensile strengths.

However, the following influences on deflections cannot be accurately assessed:

- Tensile strength, which determines the cracking moment.
- Construction loading.
- Elastic modulus.

Therefore any calculation of deflection is only an estimate, and even the most sophisticated analysis can still result in +15% to -30% error. It is advisable to give a suitable caveat with any estimate of deflection that others are relying on.

Figure 8
Precambering of slabs

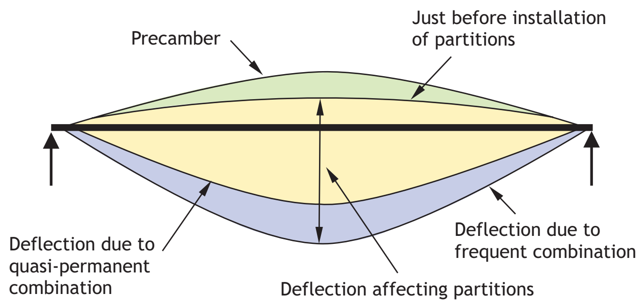
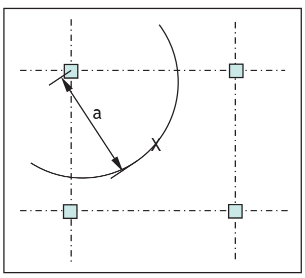


Figure 9
Recommended acceptance criteria for flat slabs



Notes
If maximum permitted $\delta = L/n$ and X is the position of maximum δ
where
L = Span
n = Limiting span-to-depth ratio, e.g. 250
then the deflection at X should not be greater than $2a/n$.
(Maximum deflection on gridlines may be more critical.)

Cladding tolerances

Deflection may affect cladding or glazing in the following ways:

- When a slab deflects, the load on the central fixings will be relieved and shed to outer fixings.
- Manufacturers may say that their glazed systems can only accommodate deflection as low as 5 mm.

There should be open discussions between the designers for the various elements to determine the most cost-effective way of dealing with the interaction of the structure and cladding.

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How to design concrete structures using Eurocode 2

9. Retaining walls

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Introduction

This chapter covers the analysis and design of reinforced concrete retaining walls to Eurocodes 2[1] and 7[2]. It considers retaining walls up to 3 m high and propped basement walls up to two storeys high (7 m). These limits have been chosen so that simplifications can be made in the geotechnical design. The self-weight of these walls, including the self-weight of backfill on them, plays a significant role in supporting the retained material. The chapter does not cover the analysis and design of embedded retaining walls, which rely primarily on passive earth pressure and flexural resistance to support the retained material.

Prior to the publication of the structural Eurocodes, the geotechnical design of retaining walls was covered by BS 8002[3]. Although certain provisions of this code are superseded by Eurocode 7, the former still contains useful qualitative information regarding the design of reinforced concrete walls.

A more general introduction to the Eurocodes is given in Chapters 1 and 2 originally published as *Introduction to Eurocodes*[4] and *Getting started*[5]. The essential features of Eurocode 7 Part 1 are covered in Chapter 6, *Foundations*[6], which includes a discussion of limit states, Geotechnical Categories, methods of design, and the Geotechnical Design Report. It should be read in conjunction with this chapter.

Essential features of Eurocode 7 are presented, along with theoretical models for the analysis of retaining walls. A procedure for analysis and design is given on page 70.

Geotechnical Categories

Eurocode 7 Part 1 defines three Geotechnical Categories that can be used to establish geotechnical design requirements. Simple structures with negliable risk belong in Geotechnical Category 1. Walls that retain soil or water and do not involve exceptional risk or difficult soil or loading conditions belong in Geotechnical Category 2, for which routine procedures for field and laboratory testing and for design and execution may be used. The design of such structures requires quantitative geotechnical data and analysis.

Walls that involve abnormal risk or where there is unusual or exceptionally difficult soil or loading conditions belong in Geotechnical Category 3, for which alternative provisions and rules to those given in Eurocode 7 may be needed; they are outside the scope of this publication.

Limit states

The design of reinforced concrete retaining walls requires verification that the following ultimate limit states are not exceeded (see Figure 1):

- Overall failure of the ground containing the wall.
- Failure of the wall by sliding.
- Failure of the wall by toppling (usually only relevant to walls founded on rock).
- Bearing failure of the ground beneath the wall (which may involve settlement and rotation of the wall).
- Structural failure of the wall.

The resistance available in fine-grained soils, such as clays and silts, depends greatly on how quickly excess pore water pressures in the ground dissipate after loading. The limit states above therefore need to be checked both for short-term (i.e. undrained) behaviour of the ground and for long-term (i.e. drained) behaviour.

Figure 1
Ultimate limit states for reinforced concrete retaining walls

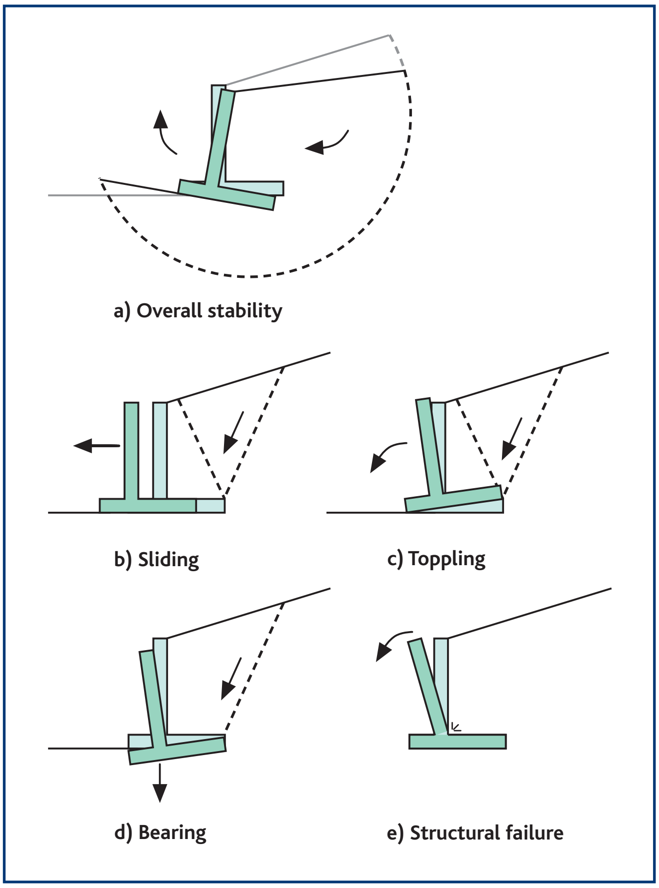


Table 1
Partial factors to be used for retaining wall design according to design approach 1 (UK National Annex)

Combin-ation	Partial factors on actions			Partial factors on material properties of soil			
	γ_G [a]	$\gamma_{G,fav}$	γ_Q	γ_φ [b]	γ_c	γ_{cu}	γ_γ
1	1.35	1.0	1.5	1.0	1.0	1.0	1.0
2	1.0	1.0	1.3	1.25	1.25	1.4	1.0

Key
a γ_G is applied to unfavourable permanent actions
b γ_φ is applied to tan φ'_k

Although Eurocode 7 provides three Design Approaches, the UK National Annex permits only Design Approach 1 to be used in the UK. In this approach, two calculations must be performed with different combinations of partial factors for the STR/GEO limit state (see Table 1).

In calculations for Combination 1, partial factors greater than 1 are applied to actions and structural materials only: to the self-weight of the wall and backfill (treated as permanent actions); to any imposed loads or surcharges at the top of the wall (permanent or variable actions, as appropriate); and to the earth and pore water pressures acting on the wall's boundary (permanent actions).

In calculations for Combination 2, partial factors greater than 1 are applied to variable actions only and to the strength of the ground and structure: to the soil's undrained strength in short-term (i.e. undrained) situations; and to the soil's angle of shearing resistance and effective cohesion in long-term (drained) situations.

The design value, F_d, of an action can be expressed as: $F_d = \gamma_F \, \psi \, F_k$
where

γ_F = partial factor for the action
F_k = characteristic value of the action
ψ = either 1.0, ψ_0, ψ_1 or ψ_2 (see Chapters 1 and 6)

Similarly the design value, X_d, of an action can be expressed as:
$X_d = X_k / \gamma_M$
where

γ_M = partial factor for the action
X_k = characteristic value of the action

It is important to note that the partial factor for φ' applies to tan φ', i.e. tan φ'_d = (tan φ'_k)/γ_φ.

Calculation models for strength limit states

The mechanical behaviour of reinforced concrete cantilever walls is commonly analysed using one of two assumed calculation models, which are explained below.

It is assumed that soils have negligible effective cohesion c', which greatly simplifies the mathematics involved. This is a safe assumption but, in the interests of economy, the effects of effective cohesion may be included in the design.

The beneficial effect of passive earth pressures in front of the wall is ignored in this publication, because its contribution to resistance is often small for reinforced concrete walls and is only mobilized after considerable movement of the wall. Furthermore, Eurocode 7 requires allowance to be made for unplanned excavations in front of retaining walls, which further reduces the effects of passive earth pressures.

In the expressions used here, the self-weights of the wall stem, wall base, and backfill are **favourable** actions when sliding and toppling are considered, but are typically **unfavourable** actions for bearing (but since they reduce the inclination and eccentricity of the total action, they may be favourable – both situations should be checked). In the design calculations, favourable actions are multiplied by different partial factors to unfavourable actions – see Table 1.

Calculation model A

In the first calculation model (see Figure 2), the wall including backfill in block *ABCD* resists sliding and toppling caused by the earth pressures acting on the vertical 'virtual' plane, *BF*. The ground beneath the wall base must also be strong enough to carry the wall's self-weight and any tractions (vertical components of force) on the virtual plane.

An attractive feature of this model is that, provided the wall heel *CD* is large enough, the earth thrust P_a (see Figure 2b) is inclined at an angle to the horizontal equal to the ground slope at the top of the wall (i.e. $\psi = \beta$), provided always that $\psi \leq \varphi'$. The test for the model's applicability is $b_h \geq h_a \tan(45 - \varphi'_d/2)$, which (if met) means that a Rankine active zone forms within the confines of block *ABCD* and the earth pressure coefficient used to calculate the thrust is given by Rankine's formula.

If b_h is too small, then the wall stem interferes with the Rankine zone and ψ should be reduced to $\psi \leq (\varphi'/3$ to $2\varphi'/3)$ – although strictly Rankine's theory is no longer applicable and calculation model B should be used instead.

For sliding and toppling, the most unfavourable location of any imposed surcharge is as shown in Figure 2a), with the edge of the surcharge coincident with point *B*. In that position it increases unfavourable earth pressures acting on the virtual plane *BF* but does not increase favourable vertical forces acting on the wall heel, *DC*. For bearing, the most unfavourable location of the surcharge is when it extends to the back of the wall, point *A*.

Calculation model B

In the second calculation model (see Figure 3), the wall including backfill in block *ACD* resists sliding and toppling caused by earth

Figure 2
Calculation model A

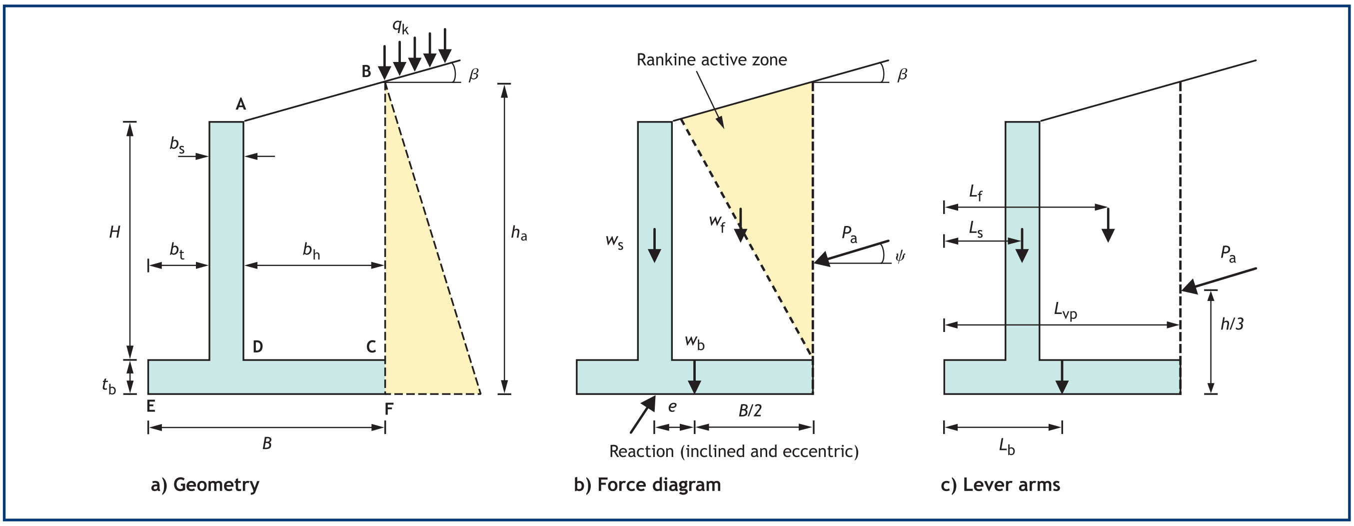
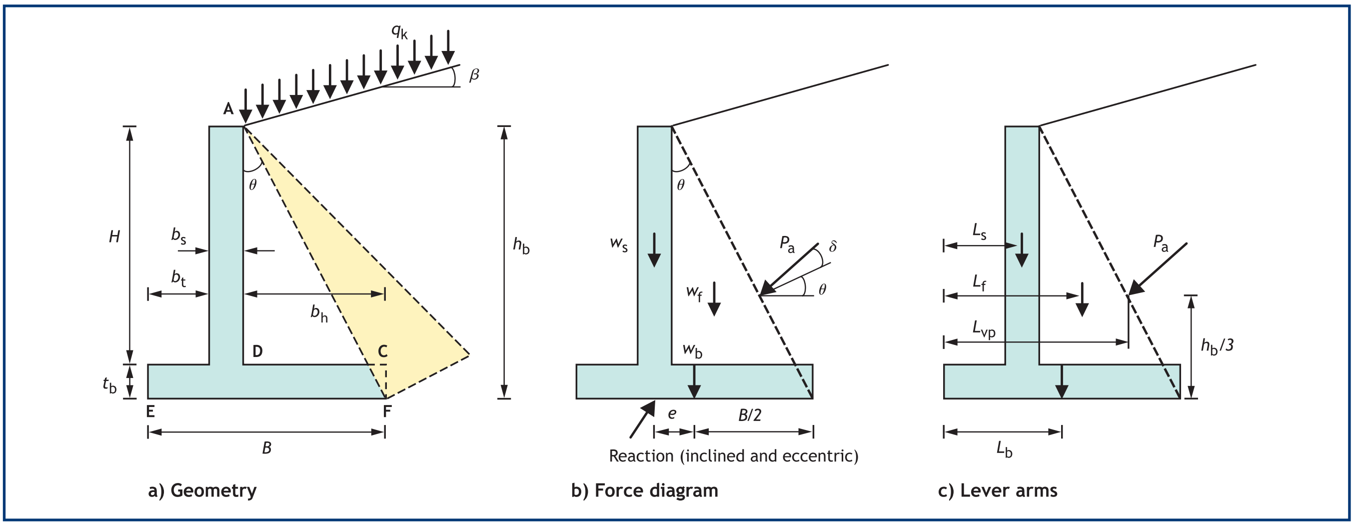


Figure 3
Calculation model B



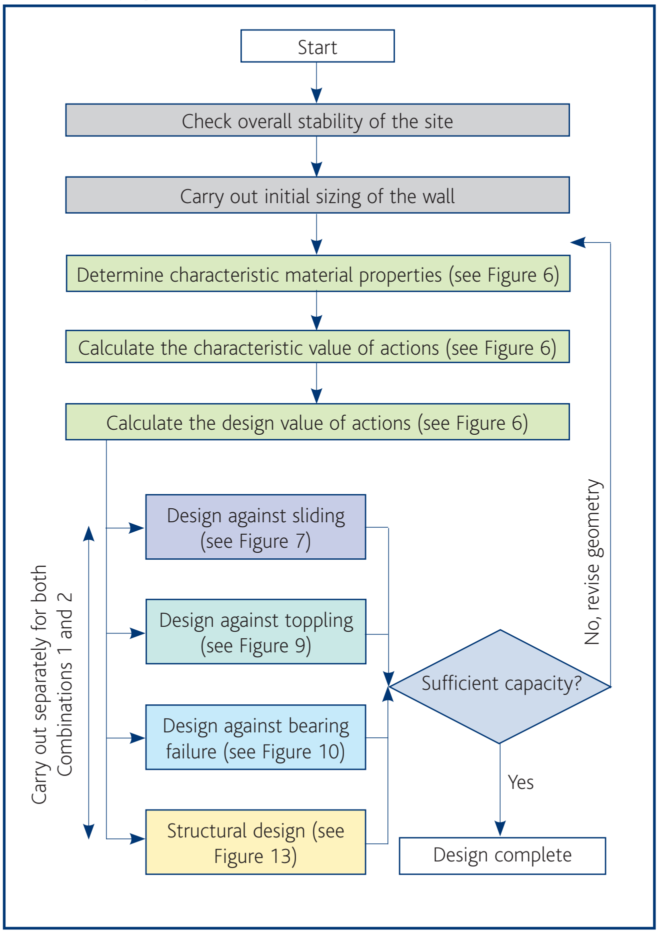
pressures acting on the **inclined** 'virtual' plane, *AF*. The earth thrust P_a (Figure 3b) is inclined at an angle $\theta + \delta$ to the horizontal. (The same model is used to analyse plain concrete gravity walls.) The ground beneath the wall base must also be strong enough to carry the wall's self-weight and the downwards component of the earth thrust on the virtual plane.

A key theoretical advantage of this model is that it can be used consistently for walls of all shapes and sizes. However, its main disadvantage is that it involves careful consideration of trigonometry, with the angle θ playing a significant role, leading to a complicated expression for K_a and values are not so readily obtained from published charts. To prevent the mathematics becoming too involved, various simplifications about the wall's geometry and that of the enclosed backfill are usually made.

Except for situations where b_h is very small, it is reasonable to assume that the angle of friction δ mobilized along the virtual plane *AF* is equal to the angle of shearing resistance of the soil, φ'.

The most unfavourable location of any imposed surcharge is typically when it extends to the back of the wall, point *A* (as shown in Figure 3a). However, since the surcharge also reduces the inclination and eccentricity of the total action, situations with it distant from the wall should also be considered.

Figure 4
Overall design procedure



Design procedure

The overall procedure for the design of reinforced concrete retaining walls is given in Figure 4 and explained in more detail over the next few pages.

Since the sizing of reinforced concrete walls is often controlled by sliding with partial factors from Combination 2, it is sensible to undertake that verification first. The other checks should also be performed for **both combinations.** Bearing can also control the design (depending on the wall geometry and soil properties).

Overall stability of the site

An essential part of the geotechnical design of retaining structures is checking the stability of the site against overall rotational failure (see Figure 1a) and other forms of overall ground failure. Guidance on how to perform the necessary verifications is outside the scope of this document, but can be found elsewhere, e.g. reference 7.

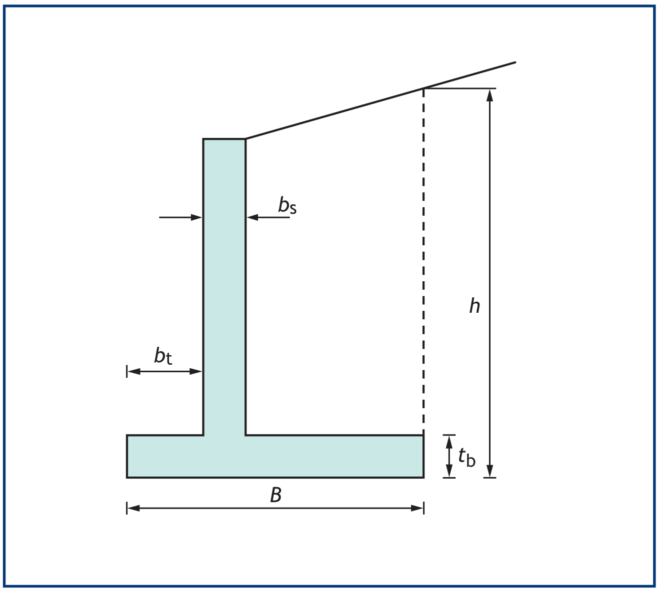
Initial sizing

The base width *B* of a reinforced concrete cantilever retaining wall is usually between 0.5 and 0.7 times its overall height[8], *h* (see Figure 5). The breadth of the wall stem b_s is normally $h/15$ to $h/10$, as is the thickness of the wall base t_b. The breadth of the wall toe b_t is typically equal to $B/4$ to $B/3$.

For basement walls in most structures, the thickness will normally be determined by the waterproofing requirements; often a minimum thickness of 300 mm is used.

For simplicity, it is assumed that adequate drainage systems will be installed behind the retaining wall so that pore water pressures need not be considered. Since this is a potentially unsafe assumption, the expressions given here need to be adjusted if this is not the case.

Figure 5
Symbols for initial sizing



Material properties

The characteristic value of a geotechnical parameter is defined in BS EN 1997–1 Cl. 2.4.5.2(2)P as "a cautious estimate of the value affecting the occurrence of the limit state", determined on the basis of laboratory and field test results complemented by well-established experience and taking account of the zone of soil involved. Characteristic geotechnical parameters should be determined by an experienced geotechnical engineer and recorded in the Geotechnical Design Report.

The properties chosen for the backfill behind the retaining wall are critical to the geotechnical design of the wall and should be selected carefully (see Figure 6). These include its weight density, γ, angle of shearing resistance, φ, and (if cohesive) undrained strength, c_u. It is uncommon for retaining walls to retain cohesive fills, since this can lead to the retention of water (which should be avoided). Further, considering cohesion reduces the calculated earth pressures, which is not conservative. For granular backfill the characteristic angle of shearing resistance is normally taken between 30° and 35°.

The foundation beneath the wall base is critical to the wall's sliding

Figure 6
Procedure for determining material properties, geometry and actions

From Figure 4

↓

Determine characteristic material properties
- Weight density ($\gamma_{k,f}$) and angle of shearing resistance ($\varphi'_{k,f}$) of backfill
- Weight density ($\gamma_{k,fdn}$), undrained strength ($c_{uk,fdn}$), and angle of shearing resistance ($\varphi'_{k,fdn}$) of foundation soil
- Weight density ($\gamma_{k,c}$) and cylinder strength (f_{ck}) of concrete

↓

Determine initial geometry and actions (see Panel 1)
1. Determine initial geometry (see Figure 5)
   - For calculation model B find inclination of virtual plane (θ)
2. Determine lever arms (see Figures 2c or 3c)
3. Calculate forces
   - Characteristic self-weights ($W_{k,s}$, $W_{k,b}$, $W_{k,f}$)
   - Total weight $\Sigma W_{k,i} = W_{k,s} + W_{k,b} + W_{k,f}$
   - Total moment $\Sigma(W_{k,i}\,L_i) = (W_{k,s}\,L_s) + (W_{k,b}\,L_b) + (W_{k,f}\,L_f)$

↓

Carry out separately for both Combinations 1 and 2

↓

Determine design material properties and earth pressures (see Panel 2)
1. Apply partial material factors to obtain design material properties
   - Weight density $\gamma_d = \gamma_k / \gamma_\gamma = \gamma_k$ since $\gamma_\gamma = 1$
   - Angle of shearing resistance $\varphi'_d = \tan^{-1}(\tan \varphi'_k / \gamma_{\varphi'})$
   - Undrained strength $c_{ud} = c_{uk} / \gamma_{cu}$
2. Determine earth pressure
   - Earth pressure coefficient K_{ad} = function of $\varphi'_{d,f}$ and β (depending on calculation model used)
   - Earth pressure is function of vertical load and K_{ad}

↓

To Figure 4

Panel 1
General expressions for geometry and actions

$$W_{k,s} = b_s\,H\,\gamma_{k,c}$$
$$W_{k,b} = t_b\,B\,\gamma_{k,c}$$
$$b_h = B - b_s - b_t$$
$$L_s = b_t + \frac{b_s}{2}$$
$$L_b = \frac{B}{2}$$

For calculation model A:

$$h = t_b + H + b_h \tan\beta$$
$$W_{k,f} = b_h\left[H + \frac{b_h \tan\beta}{2}\right]\gamma_{k,f}$$
$$L_f \approx b_t + b_s + \frac{b_h}{2}$$
$$\Omega = \beta$$
$$L_{vp} = B$$

For calculation model B:

$$h_b = t_b + H$$
$$W_{k,f} \approx \frac{b_h\,H}{2}\,\gamma_{k,f}$$
$$L_f = b_t + b_s + \frac{b_h}{3}$$
$$\theta = \tan^{-1}\left[\frac{b_h}{h_b}\right]$$
$$\Omega = \theta$$
$$L_{vp} = L_t + L_s + \frac{L_f}{3}$$

Panel 2
General expressions for material properties and earth pressures

For calculation model A:

$$K_{ad} = \left[\frac{\cos\beta - \sqrt{\sin^2\varphi_{d,f} - \sin^2\beta}}{\cos\beta + \sqrt{\sin^2\varphi_{d,f} - \sin^2\beta}}\right]\cos\beta$$

$$P_{ad} = K_{ad}\left[\gamma_G \frac{\gamma_{k,f}\,h^2}{2} + \gamma_Q\,q_k h\right]$$

For calculation model B:

$$m_t = \frac{1}{2}\left[\cos^{-1}\left[\frac{\sin\beta}{\sin\varphi'_{d,f}}\right] + \varphi'_{d,f} - \beta\right]$$

$$K_n = \left[\frac{1 - \sin\varphi'_{d,f}}{1 + \sin\varphi'_{d,f}\,\sin(2m_t - \varphi_{d,f})}\right] e^{-2\,(m_t + \beta - \varphi'_{d,f} - \theta)\tan\varphi'_{d,f}}$$

$$K_q = K_n \cos^2\beta$$

$$K_\gamma = K_n \cos\beta\,\cos(\beta - \theta)$$

$$P_{ad} = \gamma_G K_\gamma \frac{\gamma_{k,f}\,h^2}{2} + \gamma_Q\,K_q\,q_k h$$

and bearing resistance and its properties should be chosen carefully. These include its weight density, angle of shearing resistance, and (if cohesive) undrained strength. For consideration of sliding, it is the properties of the soil/concrete interface that are required unless a shear key is provided. For bearing resistance a conservative estimate of the properties of the foundation soils is required.

The weight density of concrete determines part of the wall's self-weight and should be taken as $\gamma_{k,c}$ = 25 kN/m³ for reinforced concrete in accordance with Eurocode 1[9].

Design against sliding

The procedure for designing reinforced concrete retaining walls against sliding is given in Figure 7.

In the short-term (i.e. under undrained conditions), the horizontal component of the thrust on the virtual plane must be resisted by adhesion on the underside of the wall base, $H_{Ed} \le H_{Rd}$, and by friction in the long-term (under drained conditions), $H_{Ed} \le H'_{Rd}$.

Figure 7
Procedure for design against sliding

From Figure 4

↓

Carry out separately for both Combinations 1 and 2:

Determine horizontal thrust on virtual plane (see Panel 2)
- $H_{Ed} = P_{ad} \cos \Omega$

↓

If appropriate determine undrained sliding resistance on underside of wall base
- $H_{Rd} = c_{ud,fdn} B$

↓

Determine drained sliding resistance on underside of wall base
- Calculate vertical action $V_d = \gamma_a \sum_i W_{k,i}$
- Calculate vertical traction from P_{ad} (Panel 2)
- Calculate drainned sliding resistance, H'_{Rd} (Panel 3)

↓

Verify $H_{Ed} \le H_{Rd}$ (if appropriate) and $H_{Ed} \le H'_{Rd}$

↓

If necessary, repeat the above with a shear key

↓

To Figure 4

Panel 3
Expressions for drained sliding resistance

Undrained sliding resistance:

$$H_{Rd} = c_{ud,fdn} B$$

Drained sliding resistance:

$$\delta_{d,fdn} = \varphi_{cvd,fdn}$$

$$H'_{Rd} = V_d \tan \delta_{d,fdn} = \left[\gamma_{G,fav} \sum_i W_{k,i}\right] \tan \varphi_{cvd,fdn}$$

With a shear key:

$$H'_{Rd} \approx \left[\gamma_{G,fav} \sum_i W_{k,i}\right] \tan \varphi'_{d,fdn} \sqrt{1+\left[\frac{\Delta t_b}{B}\right]^2}$$

If air can reach the interface between the base and a clay foundation, then the undrained resistance should be limited to $H_{Rd} \le 0.4V_d$ – see BS EN 1997–1 Cl.6.5.3(12)P. If drainage occurs at the interface, drained conditions may apply even for short-term loading.

The thrust depends largely on the strength properties of the backfill. If necessary, its magnitude can be reduced by using backfill with a greater angle of shearing resistance.

The resistance depends largely on the properties of the soil/structure interface between the wall base and the foundation soil. It is usual to assume full adhesion in calculations for the undrained condition based on total stresses (i.e. to use full c_u) and reduced friction in calculations for the drained condition based on effective stresses (i.e. to use $\delta_{fdn} \le \phi_{fdn}$). For concrete cast against soil, BS EN 1997–1 Cl. 6.5.3(10) recommends $\delta_{fdn} = \phi_{cv,fdn} \le \phi_{fdn}$, where $\phi_{cv,fdn}$ is the constant volume angle of shearing resistance of the foundation soil. Values of $\phi_{cv,fdn}$ are independent of the soil's relative density and typically range from 27° to 33° for granular soils. For cohesive soils the undrained situation will be critical.

If necessary, a shear key can be used to improve the wall's resistance to sliding (Figure 8). The key has three benefits:

- It moves the failure surface away from the interface between the wall and the foundation, so that an improved value of $\delta_{fdn} = \phi_{fdn}$ may be assumed.
- It lengthens the failure surface, from B to $\approx \sqrt{B^2 + \Delta t_b{}^2}$.
- It potentially increases the passive resistance in front of the wall, which is not considered here.

Design against toppling

The procedure for designing reinforced concrete retaining walls against toppling is given in Figure 9. Generally this check is only needed when the retaining wall is founded on rock or very hard soil.

The destabilising moment about the wall toe (point E in Figures 2a and 3a) arising from the thrust on the virtual plane must be resisted

Figure 9
Procedure for design against toppling

From Figure 4

↓

Carry out separately for both Combinations 1 and 2:

Determine destabilising moment M_{Ed} about point E.
(see Panels 1 and 2) $M_{Ed} = P_{ad} \cos \Omega\, h/3$

↓

Determine stabilising moments about point E
- Wall stem $M_{d,s} = W_{d,s} L_s$
- Wall base $M_{d,b} = W_{d,b} L_b$
- Backfill $M_{d,f} = W_{d,f} L_f$
- From active thrust, $M_{d,vp} = P_{ad} \sin \Omega\, L_{vp}$
- $M_{Rd} = M_{d,s} + M_{d,b} + M_{d,f} + M_{d,vp}$

↓

Verify $M_{Ed} \le M_{Rd}$

↓

To Figure 4

by the stabilising moment about the same point arising from the wall's self-weight, i.e. $M_{Ed} \leq M_{Rd}$.

The destabilising moment depends largely on the strength properties of the backfill. If necessary, its magnitude can be reduced by using a backfill with a greater angle of shearing resistance.

The stabilising moment depends largely on the wall's self-weight, including the backfill.

(Somewhat counter-intuitively, the stability of a reinforced concrete retaining wall supporting soil is not governed by limit state EQU, defined in Eurocode 7 as "loss of stability of the structure or the ground, considered as a rigid body", because the strength of the ground is **significant** in providing resistance. Instead, toppling is usually governed by limit state GEO.)

Typically, if the retaining wall has been sized appropriately for sliding, then toppling will not be critical to the design.

Design against bearing failure

The procedure for designing reinforced concrete retaining walls against bearing failure is given in Figure 10.

The bearing pressure acting beneath the wall base must be less than both the foundation's short-term (undrained) bearing capacity, $q_{Ed} \leq q_{Rd}$, and its long-term (drained) value, $q_{Ed} \leq q'_{Rd}$.

The bearing pressure depends on the wall's self-weight, including backfill, and any downwards traction on the virtual plane. Since the centre of action of the resultant bearing force is almost inevitably eccentric to the centre of the wall base, the bearing pressure must be calculated over an effective width B' (see Figure 12).

The bearing capacity depends largely on the strength of the foundation soil. Eurocode 7 gives simple analytical methods for bearing resistance calculations, based on classical bearing capacity theory. Since the resultant bearing force is almost inevitably inclined, dimensionless factors reducing the standard bearing capacity factors

Figure 10
Procedure for design against bearing failure

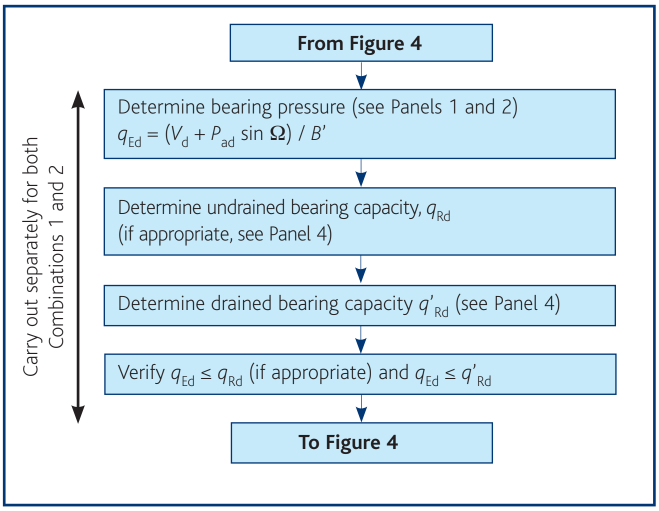
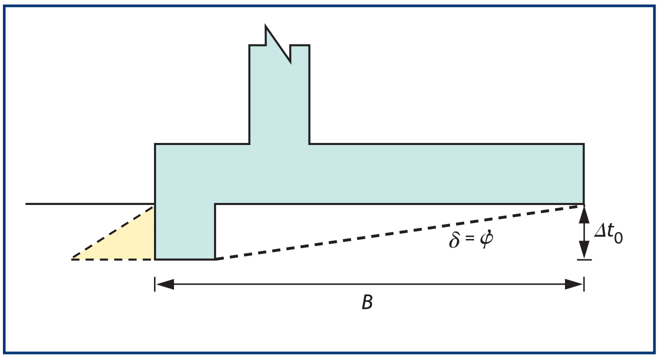


Figure 8
Effects of shear key



Panel 4
Expressions for bearing resistance

$$e = \frac{B}{2} - \left[\frac{\gamma_G \sum_i (W_{k,i}\, L_i) + (P_{ad} \sin \Omega\, L_{vp}) - \left[P_{ad} \cos \Omega \frac{h}{3} \right]}{V_d + (P_{ad} \sin \Omega)} \right]$$

$$B' = B - 2\,e$$

Undrained bearing resistance:

$$q_{Rd} = c_{ud,fdn} N_c i_c + q_d,$$

where $N_c = \pi + 2$

$$i_c = \frac{1}{2}\left[1 + \sqrt{\left(1 - \frac{P_{ad} \cos \Omega}{B'\, c_{ud,fdn}} \right)} \right]$$

Drained bearing resistance:

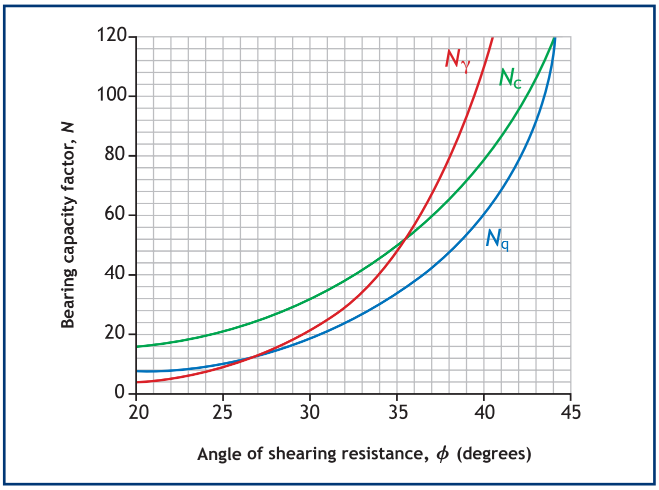
$$q'_{Rd} = q'_d N_q i_q + (\gamma_{d,fdn} - \gamma_w) \frac{B'}{2} N_\gamma i_\gamma$$

$$i_q = \left[1 - \frac{P_{ad} \cos \Omega}{\gamma_G \sum_i W_{k,i} + (P_{ad} \sin \Omega)} \right]^2$$

$$i_\gamma = i_q^{\,1.5}$$

Values for N_c, N_γ and N_q can be obtained from Figure 11

Figure 11
Bearing capacity factors, *N*, from ground properties



are needed. The formulation for the drained bearing capacity given in Figure 10 and Panel 4 assumes that the groundwater level is at the underside of the base of the retaining wall. This will be conservative for situations where groundwater can be guaranteed to be a distance $\geq B$ below the wall base. Note that the overburden pressure used in the expression for bearing capacity is that due to backfill above the wall's toe (and is likely to be a small value).

Structural design

A retaining wall is likely to be much stiffer than the ground it supports, and therefore the stem of a reinforced concrete cantilever wall must be designed to withstand earth pressures greater than the active pressures assumed in the calculations of sliding, toppling, and bearing resistance. Depending on how the wall is constructed it should either be designed for 'compaction pressures'[10] or for 'at-rest' pressures.

As with sliding, toppling, and bearing resistance there are two combinations of partial factors to check at the ultimate limit state.

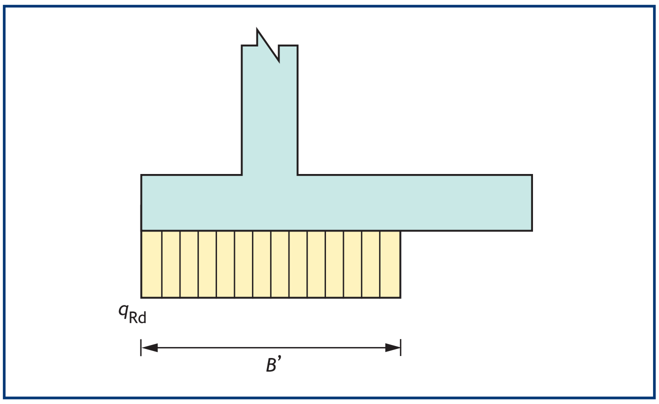
The design procedure is given in Figure 13.

Compaction earth pressures

Walls that are constructed before the placement of backfill should be designed to withstand the compaction earth pressures (see Figure 14). The wall should be constructed with adequate drainage so that water pressures will not build up against the stem in the long-term and so do not need to be considered in the design.

Panel 5 gives the earth pressure σ_h acting at depth z down the wall stem AD owing to the characteristic compaction line load P_k. For static rollers, the line load should be taken as the roller's self-weight. For vibrating compaction equipment, the sum of the self-weight and the centrifugal vibrator force should be used. If the centrifugal force is unknown, then for a vibrating roller twice the self-weight may be used instead. This rule of thumb should be used with caution and is not appropriate for use with vibrating plate compactors. All forces should be entered per unit length along the wall. Details of typical compaction equipment are given in Table 2

Figure 12
Effective base width, B'



This method for compaction earth pressure calculation assumes that the fill is placed using a vibrating roller that is prevented from coming up to the face of the wall AD. If the roller is allowed to reach AD then the depth z_j will be greater than that given in Panel 6, owing to wall yield. Since the wall is not perfectly rigid, the equations given in Panel 6 are conservative.

At-rest earth pressures

Many basement walls are constructed to retain existing ground; they should be designed to withstand the at-rest earth pressures illustrated in Figure 16.

The earth pressure coefficient used to determine the horizontal pressures acting on the wall is given in BS EN 1997–1 Cl. 9.5.2(3) and (4) as: $K_0 = (1 - \sin \varphi')\sqrt{OCR}\,(1 + \sin \beta)$ where OCR is the overconsolidation ratio of the retained soil and β is the angle of

Figure 13
Design procedure for structural design

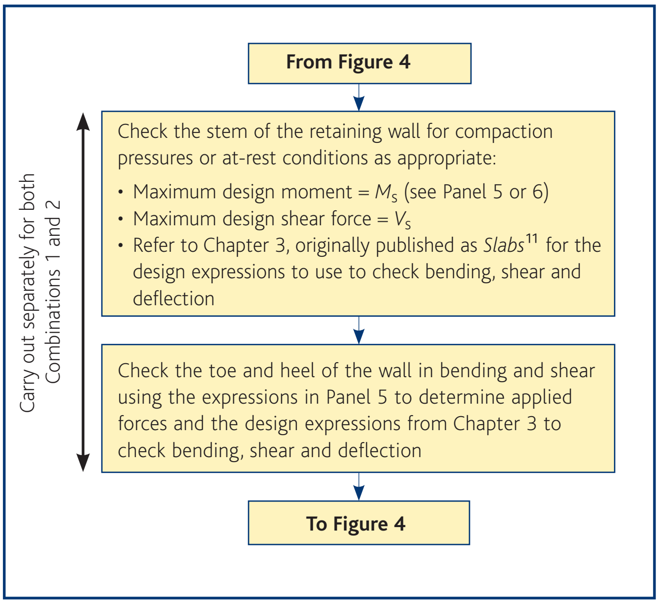
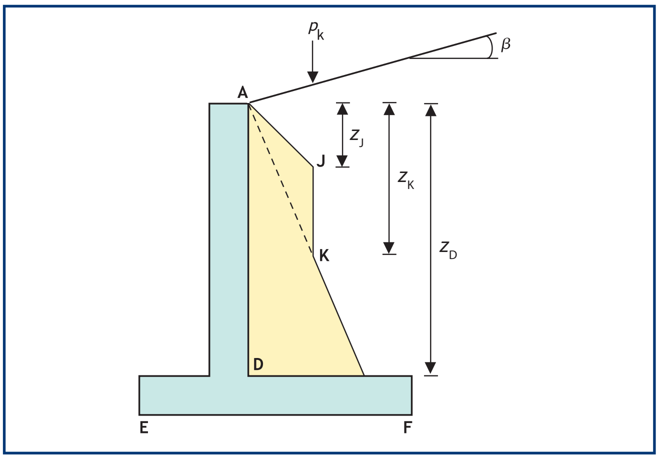


Figure 14
Compaction earth pressures for structural design of cantilever retaining walls



Panel 5

General expressions for designing walls for compaction pressures

Design line load (in units of kN/m), $P_d = \gamma_Q P_k$

$$K_{ad} = \left[\frac{\cos\beta - \sqrt{\sin^2 \varphi'_{d,f} - \sin^2\beta}}{\cos\beta + \sqrt{\sin^2 \varphi'_{d,f} - \sin^2\beta}} \right] \cos\beta$$

$$K_{pd} = \left[\frac{\cos\beta + \sqrt{\sin^2 \varphi'_{d,f} - \sin^2\beta}}{\cos\beta - \sqrt{\sin^2 \varphi'_{d,f} - \sin^2\beta}} \right] \cos\beta$$

Depth of point *J* in Figure 13, $z_J = K_{pd} \sqrt{\dfrac{2P_d}{\pi \gamma_{K,f}}}$

Depth of point *K*, $z_K = K_{ad} \sqrt{\dfrac{2P_d}{\pi \gamma_{K,f}}}$

Between *A* and *J*, horizonal stress (in units of kPa)

$$\sigma_{hd} = K_{pd}\, \gamma_{k,f}\, z$$

Between *J* and *K*

$$\sigma_{hd} = \sqrt{\frac{2P_d \gamma_{K,f}}{\pi}}$$

Betwwen *K* and *D*

$$\sigma_{hd} = K_{ad}\, \gamma_{k,f}\, z$$

Forces for stem design (see Figure 14)

$$\text{Moment, } M_s = \frac{\sigma_{hd,J}\, z_J}{2}\left[z_D - \frac{2z_J}{3} \right] + \frac{\sigma_{hd,J}(z_D - z_J)^2}{2} + \frac{(\sigma_{hd,D} - \sigma_{hd,K})(z_D - z_K)^2}{6}$$

$$\text{Shear, } V_s = \frac{\sigma_{hd,J}\, z_J}{2} + \sigma_{hd,J}(z_D - z_J) + (\sigma_{hd,D} - \sigma_{hd,K})(z_D - z_K)/2$$

where

$\sigma_{hd,J} = \sigma_{hd}$ at *J* etc.

Forces for heel design (see Figure 15b)

$$\text{Moment, } M_h \approx \frac{\gamma_G\, W_{K,f}\, b_h}{2} - \frac{q_{Ed}\,(b_h - B + B')^2}{2}$$

$$\text{Shear, } V_h \approx \gamma_G\, W_{K,f} - q_{Ed}\,(b_h - B + B')$$

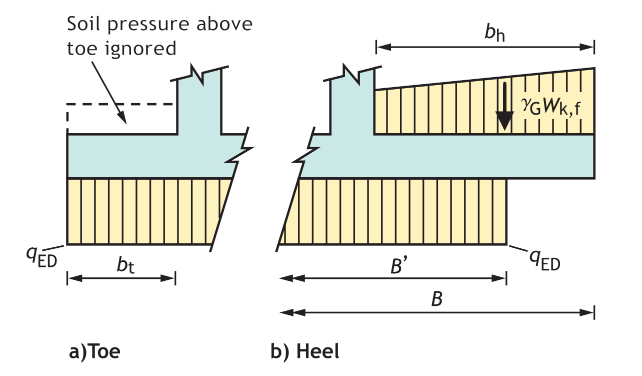
Forces for toe design (see Figure 15a)

$$\text{Moment, } M_t = \frac{q_{Ed}\, b_t^{\,2}}{2}$$

$$\text{Shear, } V_t = q_{Ed}\, b_t$$

Figure 15

Pressure diagram for design of reinforced concrete base



ground surface. The OCR is the ratio of maximum past level of vertical stress to the current level of vertical effective stress.

OCR is typically obtained from an oedometer test and/or knowledge of the geological history of the site. It is not conservative to ignore it.

Many basements involve one or two levels of propping and so verification of sliding and toppling resistance is unnecessary (at least for persistent design situations – they may still be relevant while the wall is being constructed).

Verification of the structural resistance of basement walls follows the same procedure as that for cantilever walls (see Figure 13), with suitable allowance for any water pressures that may build up on the back of the wall *AD*, especially if no provision is made for drainage.

Table 2

Typical compaction equipment[12]

Description of compaction equipment	Mass (kg)	Centrifugal vibrator force (kN)	Design force (kN)
80 kg vibrating plate compactor	80	14	14.3
180 kg vibrating plate compactor	180	80	81.8
350 kg pedestrian-operated vibrating roller	350	12	15.4
670 kg vibrating plate compactor	670	96	102.6
1.5 t pedestrian-operated vibrating roller	1530	58	73.0
2.8 t smooth wheeled roller	2800	N/A	27.5
6.9 t double vibrating roller	6900	118	185.7
8.6 t smooth wheeled roller	8600	N/A	84.4

Panel 6

Expressions for the structural design of basement walls for 'at-rest' pressures

$$K_{0d} = (1 - \sin \varphi'_{d,f}) \sqrt{\text{OCR}}\; (1 + \sin\beta)$$

$$\sigma_{v,d} = \gamma_{d,f}\, z + q_d$$

Between *A* and *W*

$$u_d = 0$$

$$\sigma_{h,d} = \sigma'_{h,d} = K_{0d}\, \sigma_{v,d}$$

Between *W* and *D*

$$u_d = \gamma_w\,(z_D - z_w)$$

$$\sigma'_{h,d} = K_{0d}\,(\sigma_{v,d} - u_d)$$

$$\sigma_{h,d} = \sigma'_{h,d} + u_d$$

Forces for stem design

$$\text{Moment, } M_s = \frac{\sigma_{hd,A}\, z_W}{2}\left[z_D - \frac{z_W}{3} \right] + \frac{\sigma_{hd,W}\, z_W}{2}\left[z_D - \frac{2z_W}{3} \right] + \frac{2\sigma_{hd,W}\,(z_D - z_W)^2}{6} + \frac{\sigma_{hd,D}\,(z_D - z_W)^2}{6}$$

$$\text{Shear, } V_s = \frac{\sigma_{hd,A}\, z_W}{2} + \frac{\sigma_{hd,W}\, z_W}{2} + \frac{\sigma_{hd,W}\,(z_D - z_W)}{2} + \frac{\sigma_{hd,D}\,(z_D - z_W)}{2}$$

See Panel 5 for forces for base design.

When the structure retains:

- Sands and gravels (i.e. high permeability soils) without a reliable drainage system installed, or
- Silts and clays (i.e. low permeability soils)

then BS EN 1997–1 Cl. 2.4.6.1(11) requires the water table to be taken at the "maximum possible level, which may be the ground surface ... unless the adequacy of the drainage system can be demonstrated and its maintenance ensured".

The relevant expressions for the total earth pressure σ_h (= effective earth pressure, σ'_h + pore water pressure, u) acting at depth z down the wall *AD* are given in Panel 6 for a water table at depth d_w below the top of the wall. These expressions assume that the wall is rigid (owing to the propping) and no stress relief occurs during wall installation. Both of these assumptions are conservative.

Detailing

Control of cracking

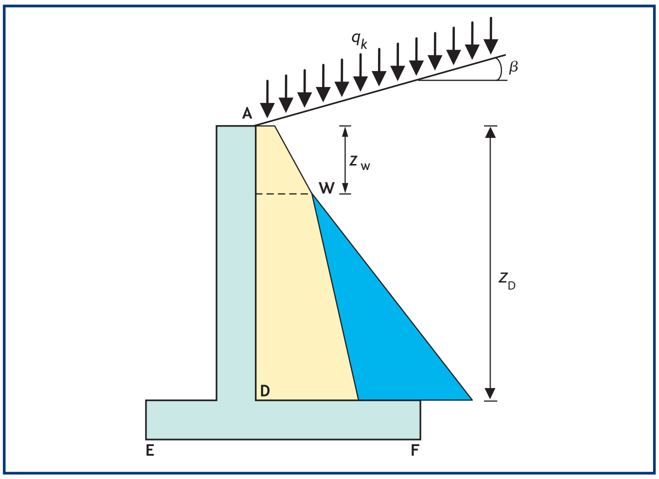
It may be necessary to control the cracking of a reinforced concrete wall, e.g. for aesthetic reasons or to minimise water ingress for a basement wall. For the latter, detailed guidance on the design of water-resisting basements is given in CIRIA Report 139[13]. Strictly, this report has been written for use alongside BS 8110 and BS 8007[14], but the principles can be used with BS EN 1992–1–1 and BS EN 1992–3[3].

Control of cracking may either be assessed by using the deemed-to-satisfy method, which is presented here, or by calculating the crack widths directly (refer to Cl 7.3.4 of BS EN 1992–1–1).

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

- Provide a minimum amount of reinforcement, $A_{s,min}$, so that the reinforcement does not yield immediately upon formation of the first crack.
- Where **restraint** is the main cause of cracking, limit the bar diameter to that shown in Table 3. In this case any level of steel stress may be chosen but the selected value must then be used in the calculation of $A_{s,min}$ and the size of the bar should be limited as shown.
- Where **loading** is the main cause of cracking, limit the bar diameter or the bar spacing to that shown in Table 3.

Figure 16
Earth and pore water pressures for structural design of retaining walls subject to 'at-rest' conditions



In the absence of specific requirements (e.g. water-tightness), the limiting calculated crack width w_{max} may be restricted to 0.3 mm in all exposure classes under quasi-permanent load combinations (see Chapter 1).

The minimum area of reinforcement in tensile zones to control cracking should be calculated for each part of the member as follows:

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

where

k_c = 1.0 for pure tension and 0.4 for pure bending (allows for the nature of the stress distribution within the section immediately prior to cracking and for the change of the lever arm as a result of cracking)

k = 1.0 where the wall is less than 300 mm and K = 0.65 where it exceeds 800 mm thick. For intermediate conditions interpolation may be used. Factor allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces

$f_{ct,eff}$ = mean value of the tensile strength of concrete effective at the time cracks may be first expected to occur at the appropriate age $f_{ct,eff} = f_{ct,m}$ (see table 3.1 of BS EN 1992-1-1[1])

A_{ct} = area of concrete in that part of the section which is calculated to be in the tension zone i.e. in tension just before the formation of the first crack

Table 3
Maximum bar size or spacing to lmit crack width (mm)

Steel stress (σ_s) MPa	W_{max} = 0.3			W_{max} = 0.2		
	Maximum bar size (mm)		Maximum bar spacing (mm)	Maximum bar size (mm)		Maximum bar spacing (mm)
160	32	OR	300	25	OR	200
200	25		250	16		150
240	16		200	12		100
280	12		150	8		50
320	10		100	6		–
360	8		50	5		–

Note

The steel stress may be estimated from the expression below

$$\sigma_s = \frac{f_{yk}\, m A_{s,req}}{\gamma_{ms} n A_{s,prov}\, \delta}$$

where:

f_{yk} = the characteristic reinforcement yields stress
γ_{ms} = the partial factor for reinforcement steel
m = the total load from quasi-permanent combination
n = the total load from ULS combination
$A_{s,req}$ = the area of reinforcement at the ULS
$A_{s,prov}$ = the area of reinforcement provided
δ = the ratio of redistribution moment to elastic moment

σ_s = absolute value of the maximum stress permitted in the reinforcement immediately after the formation of the crack. The value should be chosen bearing in mind the limits on bar size and spacing indicated in Table 3

Large radius bends

It is often necessary to provide large radius bends to the main reinforcement at the base of the stem because of the high tensile forces in the bars. The minimum mandrel diameter, $\phi_{m.min}$ should be assessed as follows:

$$\phi_{m.min} \geq F_{bt}(1/a_b+1/(2\phi))/f_{cd}$$

where

F_{bt} = tensile force from ultimate loads in a bar at the start of the bend

a_b = half the pitch of the bars or nominal cover plus $\phi/2$

ϕ = bar diameter

f_{cd} = design value of concrete compressive strength

The *Standard method of detailing structural concrete*[15], appendix B, contains some useful tables that will assist in determining the minimum mandrel size.

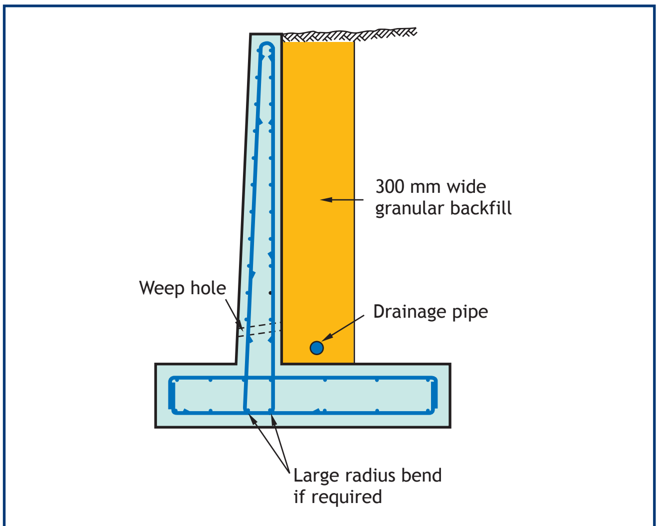
Rules for spacing and quantity of reinforcement

Vertical reinforcement

Where axial forces dominate, the minimum area of vertical reinforcement is $0.002A_c$; half this area should be placed in each face. Otherwise, the minimum percentage of reinforcement can be obtained from Table 6 of Chapter 3. Outside lap locations, the maximum area of vertical reinforcement is $0.04A_c$; this may be doubled at lap locations.

The distance between two adjacent vertical bars should not exceed the lesser of either three times the wall thickness or 400 mm.

Figure 17
Typical drainage layout for a retaining wall



For walls with a high axial load (eg basement walls), the main reinforcement placed nearest to the wall faces should have transverse reinforcement in the form of links with at least four per m^2 of wall area. Where welded mesh and bars of diameter $\phi \leq 16$ mm are used with cover larger than 2ϕ, transverse reinforcement is not required.

Horizontal reinforcement

The minimum area of horizontal reinforcement is the greater of either 25% of vertical reinforcement or 0.001 A_c. However, where crack control is important, early age thermal and shrinkage effects should be considered.

Where flexural forces dominate, these requirements may be relaxed to 20% of the vertical reinforcement area.

Practical issues

Design for movement

Concrete shrinks due to early thermal effects immediately after casting. The base of a retaining structure is usually restrained by the soil on which it is bearing, which induces strains in the concrete. Therefore, the reinforcement detailing, pour size and sequence of construction should be planned to control the resultant cracking.

Typically pour sizes are up to one storey high, to limit hydrostatic pressures on formwork. The *National structural concrete specification*[16] recommends pour sizes with a maximum area of 25 m^2 and maximum dimension 5 m for water-resisting walls. The contractor will seek to use the largest pour size possible and will want to ensure that the minimum volume of concrete for a pour is 6 m^3 (i.e. a full load of a ready-mixed concrete). It is usual to cast alternate bays in a wall to reduce the effects of early shrinkage. Further advice on calculating strains and crack widths for restrained walls can be found in appendices L and M of BS EN 1992–3[1].

For long lengths of retaining walls, it is generally accepted practice to have expansion joints every 20 – 30 m; even so the base of the wall will still be restrained. For a temperature range of 40°C, the joint should allow for 10 mm of movement for every 10 m length of wall.

Drainage

To prevent pore water pressures from building up, it is important to provide drainage material behind a retaining wall. The zone immediately behind the wall should be of a free-draining granular material, which should be protected to prevent it becoming blocked with fines. A drainage pipe, laid to falls, should be provided at the base of the free-draining material. The system should be designed for the anticipated rainfall, and rodding points provided for maintenance of the drainage systems. Weepholes should be provided as a back-up as they act as an overflow and as a visual warning that maintenance is required should the primary system become blocked. A typical drainage system is illustrated in Figure 17.

Construction

It is particularly important when designing retaining walls to consider how they will be constructed. Often some form of temporary works is required and these may impact on the design. The simplest form of construction is in an open excavation with the sides battered back to a safe angle to allow the construction of the wall. In this case there is little impact on the design, although the loads imposed during compaction of the backfill may be onerous.

Where there is insufficient space for opening an excavation, either a king post wall or sheet pile wall will be used as temporary supports, and these may require propping, especially for a two-storey basement.

The designer should ensure there is sufficient space to install the temporary works. It is possible to use contiguous piles, secant piles or diaphragm walling as temporary supports but it is more usual for these to be installed as part of the permanent works.

The designer should consider how the temporary works will affect the installation of the water-proofing for a basement wall. For instance, a sheet pile wall may be used to affix an external waterproofing system, but this would prevent the sheet piles from being removed at the end of construction. Similarly, external waterproofing should not be used where temporary propping would penetrate it, because it will not be possible to complete the waterproofing when the props are removed.

The temporary works may affect the soil pressures acting on the wall and the construction sequence could give rise to temporary actions that exceed the permanent actions imposed on the wall. The designer should consider this aspect and prepare a method statement indicating the assumptions made so the temporary works designer can work to the assumptions made or make alternative proposals.

References

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How to design concrete structures using Eurocode 2

10. Detailing

O Brooker BEng, CEng, MICE, MIStructE

Introduction

This chapter is intended for use by detailers to prepare reinforcement drawings for projects that have been designed using Eurocode 2[1]. It provides a summary of the requirements of the Eurocode and simplifies them where appropriate.

To pave the way for the introduction of Eurocode 2, other supporting standards have been introduced or revised, including:

- BS 8500: *Concrete – Complementary standard to BS EN 206–1*[2], which replaced BS 5328: *Concrete* on 1 December 2003 and should already be familiar.
- BS 4449: *Specification for carbon steel bars for the reinforcement of concrete*[3] was revised in 2005 so that it became a complementary standard to BS EN 10080: *Steel for the reinforcement of concrete*[4] and Normative Annex C of Eurocode 2.
- BS 8666: 2005, *Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete – specification*[5] has also been revised, introducing Eurocode 2 requirements and a greater range of shape codes.

In this publication a number of assumptions have been made. Many of the derived expressions and figures assume UK values for the National Determined Parameters (NDPs). It is assumed that bars sizes are 40 mm or less and that the bars will not be bundled together. It is also assumed that the concrete class will not exceed C50/60. For additional requirements outside these limits, reference should be made to Eurocode 2.

This chapter focuses on detailing the reinforcement to comply with Eurocode 2; guidance on procedures, responsibilities and preparation of drawings can be found in other documents[6,7,8].

Type and grade of reinforcement

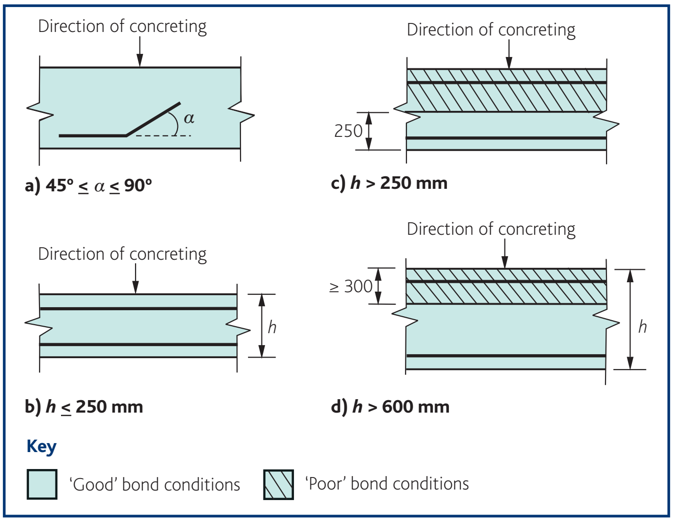
The 2005 revision of BS 4449 introduced a characteristic yield strength of 500 MPa. There are now three grades of reinforcement, A, B and C (see Table 1), which offer increasing ductility. It is expected that for the design of UK buildings all three grades A, B or C will be appropriate and in this case the bar size can be prefixed with an H (e.g. H12). Grade A is not suitable for use where redistribution above 20% has been assumed in the design and a grade B bar should be specified in these circumstances (e.g. B12). Grade C is for seismic conditions or where additional ductility is required.

Table 1
Notation for steel reinforcement

Type of steel reinforcement	Notation
Grade B500A, Grade B500B or Grade B500C conforming to BS 4449:2005	H
Grade B500A conforming to BS 4449: 2005	A
Grade B500B or Grade B500C conforming to BS 4449: 2005	B
Grade B500C conforming to BS 4449: 2005	C
Reinforcement of a type not included in the above list having material properties that are defined in the design or contract specification.	X

Note
In the Grade description B500A, etc., 'B' indicates reinforcing steel.

Figure 1
Description of bond conditions



Cover

The nominal cover should generally be specified by the designer and full details of how to determine this are given in Chapter 2, originally published as, *Getting started*[9]. The nominal cover should be shown on the drawings and should refer to the reinforcement nearest to the surface of the concrete e.g. the links in a beam.

Also, the cover to the main bar should be at least equal to the size of that bar, plus the allowance for deviations, Δc_{dev}. Where there are no links the nominal cover should be at least equal to the size of the bar plus Δc_{dev}, this may be significant for bar diameters greater than 12 mm. Δc_{dev} may be 5 or 10 mm depending on the quality assurance system assumed for the project. If the cover needs to be increased to meet these requirements, the detailer should consult with the designer.

Anchorage and lap lengths

Eurocode 2 introduces a range of factors (α_1 to α_6) for use when calculating the appropriate anchorage and lap lengths. A number of assumptions can be made that enable Table 2 to be developed for anchorage and lap lengths. If the conditions noted in the table are not met then reference should be made to Eurocode 2. Lap lengths for unequal size bars may be based on the smaller bar, although this is not stated in the Code.

Anchorage of bars and links

All reinforcement should be anchored so that the forces in it are safely transmitted to the surrounding concrete by bond without causing

Table 2
Anchorage and lap lengths for concrete class C25/30 (mm)

		Bond condition, (see Figure 1)	Reinforcement in tension, bar diameter, ϕ (mm)								Reinforce-ment in compres-
			8	10	12	16	20	25	32	40	
Anchorage length, l_{bd}	Straight bars only	Good	230	320	410	600	780	1010	1300	1760	40ϕ
		Poor	330	450	580	850	1120	1450	1850	2510	58ϕ
	Other bars	Good	320	410	490	650	810	1010	1300	1760	40ϕ
		Poor	460	580	700	930	1160	1450	1850	2510	58ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	320	440	570	830	1090	1420	1810	2460	57ϕ
		Poor	460	630	820	1190	1560	2020	2590	3520	81ϕ
	100% lapped in one location (α_6 = 1.15)	Good	340	470	610	890	1170	1520	1940	2640	61ϕ
		Poor	490	680	870	1270	1670	2170	2770	3770	87ϕ

Notes
1 Nominal cover to all sides and distance between bars ≥ 25 mm (i.e. $\alpha_2 < 1$).
2 $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$.
3 Design stress has been taken as 435 MPa. Where the design stress in the bar at the position from where the anchorage is measured, σ_{sd}, is less than 435 MPa the figures in this table can be factored by $\sigma_{sd}/435$. The minimum lap length is given in cl 8.7.3 of Eurocode 2.
4 The anchorage and lap lengths have been rounded up to the nearest 10 mm.
5 Where 33% of bars are lapped in one location, decrease the lap lengths for '50% lapped in one location' by a factor of 0.82.
6 The figures in this table have been prepared for concrete class C25/30; refer to Table 13 for other classes or use the following factors for other concrete classes.

Concrete class	C20/25	C28/35	C30/37	C32/40	C35/45	C40/50	C45/55	C50/60
Factor	1.16	0.93	0.89	0.85	0.80	0.73	0.68	0.63

cracking or spalling. The design anchorage length, l_{bd}, (which can be obtained from Table 2) is measured along the centreline of the bar (see Figure 2). The anchorage of links is shown in Figure 3.

Arrangement of laps

Where possible laps in a member should be staggered (see Figure 4) and not located in areas of high stress. The arrangement of lapped bars should comply with Figure 5, as set out below:

1. The clear distance between lapped bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear distance.
2. The longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_0. Where this is not the case, the bars should be considered as being lapped in one section.
3. In case of adjacent laps, the clear distance between adjacent bars should not be less than 2ϕ or 20 mm.

When the provisions comply with 1 and 3 above, the permissible percentage of lapped bars in tension may be 100% where the bars are all in one layer. In this case the design lap length, l_0, from Table 2, must be increased (see note 5). Where the bars are in several layers no more than 50% should be lapped in any one layer.

All bars in compression and secondary (distribution) reinforcement may be lapped in the same location.

Transverse reinforcement

Bars in tension

Transverse tensile stresses occur at the ends of lapped bars. Where the diameter, ϕ, of the lapped bars is less than 20 mm, or the percentage of lapped bars in any section is less than 25%, then any transverse reinforcement or links necessary for other reasons may be assumed sufficient for the transverse tensile forces without further justification.

Where the diameter, ϕ, of the lapped bars is greater than or equal to 20 mm, the transverse reinforcement should have a total area, A_{st} (sum of all legs parallel to the layer of the spliced reinforcement – see Figure 6a) of not less than the area A_s of one lapped bar ($\Sigma A_{st} \geq 1.0A_s$). The transverse bar should be placed perpendicular to the direction of the lapped reinforcement and between that and the surface of the concrete. Table 3 gives recommended minimum areas for transverse reinforcement. Where the lapped bars in a slab are located in an area of low stress, e.g. near the point of contraflexure, it is generally acceptable to assume that transverse reinforcement is not required.

If more than 50% of the bars are lapped in one location and the distance, a, between adjacent laps at a section is ≤ 10ϕ (see Figure 5) transverse reinforcement should be formed by links or U-bars anchored into the body of the section.

Bars in compression

In addition to the rules for bars in tension (Figure 6a), one bar of the transverse reinforcement should be placed outside each end of the lap length of bars in compression and within 4ϕ of the ends of the lap length (see Figure 6b).

Figure 2
Design anchorage length l_{bd}, for any shape measured along the centreline

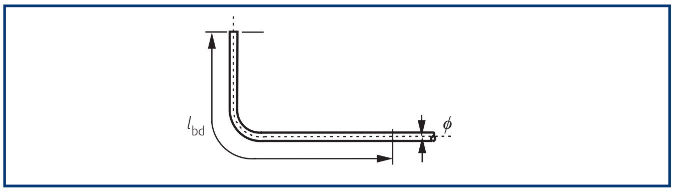


Figure 3
Anchorage of links

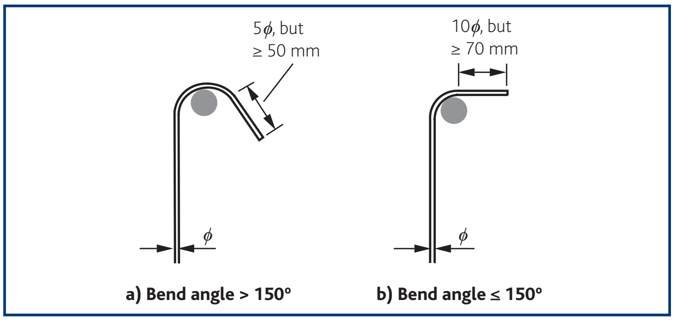


Figure 4
Percentage of lapped bars in one lapped section

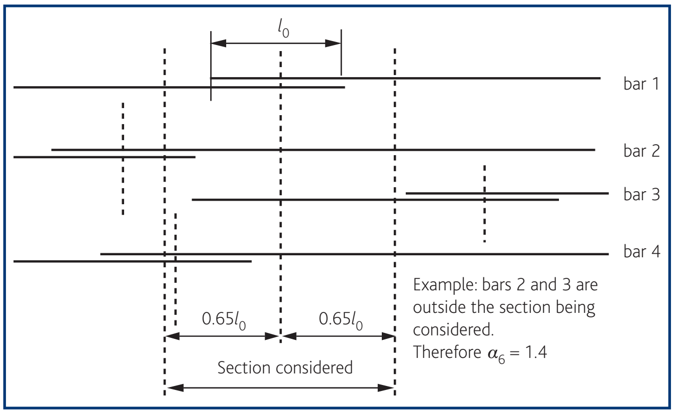
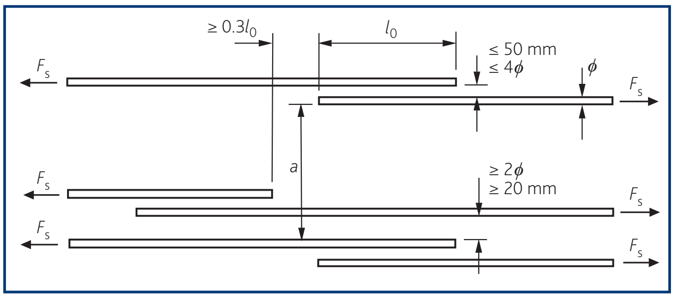


Figure 5
Arranging adjacent lapping bars



Beams

Curtailment

Unless the additional tensile force in the longitudinal reinforcement due to shear has been calculated, the curtailment length of the longitudinal reinforcement should be extended beyond the point at which it is required for flexural strength (this is known as the 'shift rule') using the following expression (see also Figure 7):

$a_l = z \cot \theta /2$ for vertical shear reinforcement.

where

z = lever arm

θ = angle of compression strut

This can conservatively be taken as:

$a_l = 1.125d$

For beams designed using the co-efficients given in Table 3 of Chapter 4, originally published as *Beams,*[10] the simplified rules shown in Figure 8 may be used. However, the simplifications are conservative and economies can be achieved by curtailing bars to suit the actual moments.

Figure 6

Transverse reinforcement for lapped splices

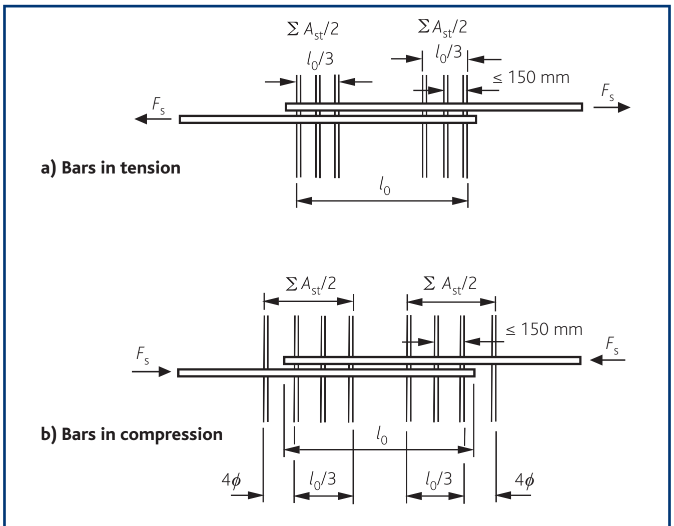


Table 3

Bar sizes for transverse reinforcement

Lap length (mm), for transverse bars at 150 mm centres[a]	Number of bars at each lap	Bar size (mm)			
		20	25	32	40
		A_s = 314	A_s = 491	A_s = 804	A_s = 1260
≤450	2	10	16	16	25
451 – 900	3	10	12	16	20
901 – 1350	4	8	10	12	16
1351 – 1800	5	8	8	12	16
1801 – 2250	6	8	8	10	12
2251 – 2700	7	N/A	8	10	12

Key

a For transverse bars at less than 150 mm centres use the following expression to calculate the required number of bars and hence the required transverse bar diameter:

Number of bars required = 1 + $l_0/(3s)$ where s = spacing of the transverse bars.

Figure 7

Illustration of curtailment of longitudinal reinforcement

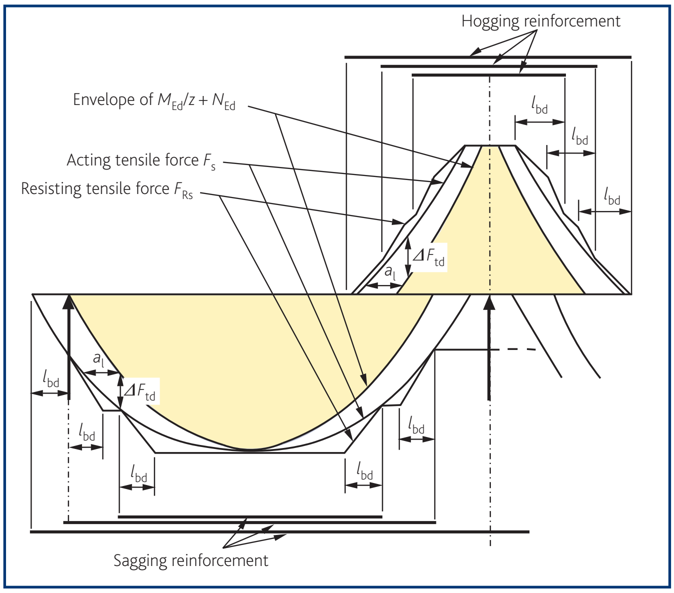
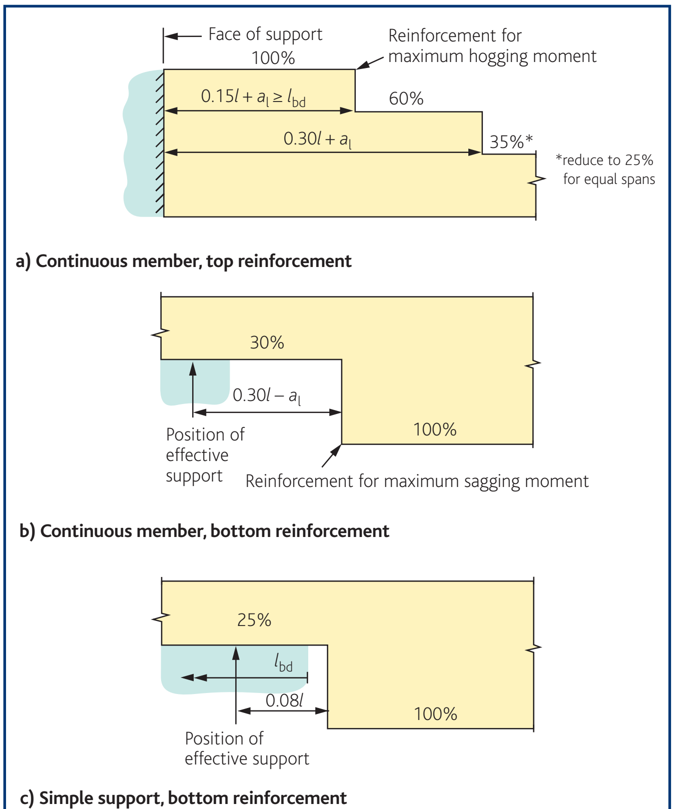


Figure 8

Simplified detailing rules for beams



Notes

1. l is the effective length.
2. a_l is the distance to allow for tensile force due to shear force.
3. l_{bd} is the design anchorage length.
4. $Q_k \leq G_k$.
5. Minimum of two spans required.
6. Applies to uniformly distributed loads only.
7. The shortest span must be greater than or equal to 0.85 times the longest span.
8. Applies where 15% redistribution has been used.

Reinforcement in end supports

In monolithic construction, even when simple supports have been assumed in design, the section at supports (top reinforcement) should be designed for a bending moment arising from partial fixity of at least 25% of the maximum bending moment in the span (i.e. provide 25% of mid-span bottom reinforcement).

The area of bottom reinforcement provided at supports with little or no end fixity assumed in design, should be at least 25% of the area of steel provided in the span. The bars should be anchored to resist a force , F_E.

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

where

$|V_{Ed}|$ = absolute value of shear force

N_{Ed} = the axial force if present

The anchorage, l_{bd}, should be measured beyond the line of contact between the beam and support.

Provided σ_{sd} is taken as 435 MPa in the calculation of the anchorage length (which is assumed in Tables 2 and 13) then it should not be necesary to calculate F_E.

Flanged beams

At supports the tension reinforcement to resist hogging moments should be distributed across the full width of the effective flange as shown in Figure 9; part of it may be concentrated over the web.

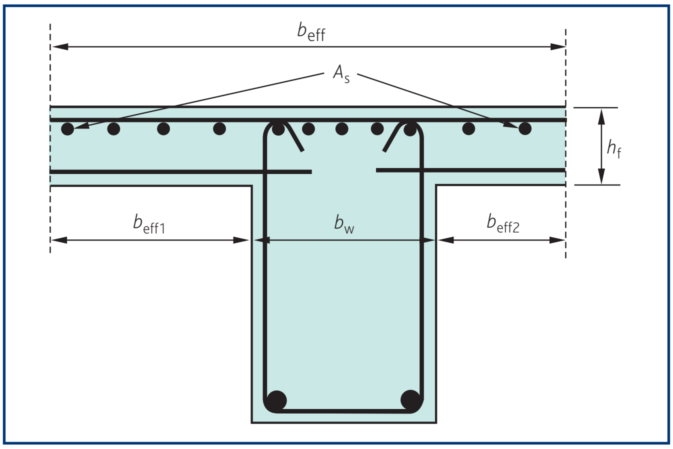
Minimum area of longitudinal reinforcement

The minimum area of reinforcement is $A_{s,min} = 0.26\, f_{ctm}\, b_t\, d/f_{yk}$ but not less than $0.0013 b_t d$, where b_t is the mean width of the tension zone (see Table 4). For a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value of b_t.

Maximum area of longitudinal reinforcement

Outside lap locations, the maximum area of tension or compression reinforcement should not exceed $A_{s,max} = 0.04\, A_c$.

Figure 9
Placing of tension reinforcement in flanged cross section



Minimum spacing of reinforcement

The minimum clear distance between bars should be the greater of:

- Bar diameter
- Aggregate size plus 5 mm
- 20 mm

Shear reinforcement

The longitudinal spacing of vertical shear reinforcement should not exceed $0.75d$. Note the requirement for a maximum spacing of 150 mm where the shear reinforcement acts as a transverse reinforcement at laps in the longitudinal bars. The transverse spacing of the legs in a series of shear links should not exceed:

$s_{t,max} = 0.75d \leq 600$ mm

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

$\frac{A_{sw}}{s\, b_w} \geq \rho_{w,min}$ where $\rho_{w,min}$ can be obtained from Table 4.

Slabs

Curtailment

The curtailment rules for beams should be followed, except that a value of $a_l = d$ may be used.

For slabs designed using the co-efficients given in Table 3 of Chapter 3, originally published as *Slabs*[11], the simplified rules shown in Figure 10 may be used.

Reinforcement in end supports

In simply supported slabs, the area of reinforcement may be reduced to half the calculated span reinforcement and continued up to the support, otherwise 100% of the reinforcement may be continued to the support. Beyond the face of the support 15% of the area of maximum reinforcement should be provided (see Figure 10c). The bars should be anchored to resist a force, F_E, as given in the section on beams.

Table 4
Minimum percentage of reinforcement required

f_{ck}	f_{ctm}	Minimim percentage ($0.26\, f_{ctm} / f_{yk}$[a])	$\rho_{w,min} \times 10^{-3}$
25	2.6	0.13%	0.80
28	2.8	0.14%	0.85
30	2.9	0.15%	0.88
32	3.0	0.16%	0.91
35	3.2	0.17%	0.95
40	3.5	0.18%	1.01
45	3.8	0.20%	1.07
50	4.1	0.21%	1.13

Key
a Assuming f_{yk} = 500 MPa

Minimum spacing requirements

The minimum clear distance between bars (horizontal or vertical) should not be less than the bar size, b, (d_g + 5 mm), or 20 mm, where d_g is the maximum size of aggregate.

Maximum spacing of reinforcement

For slabs less than 200 mm thick the following maximum spacing rules apply (h is the depth of the slab):

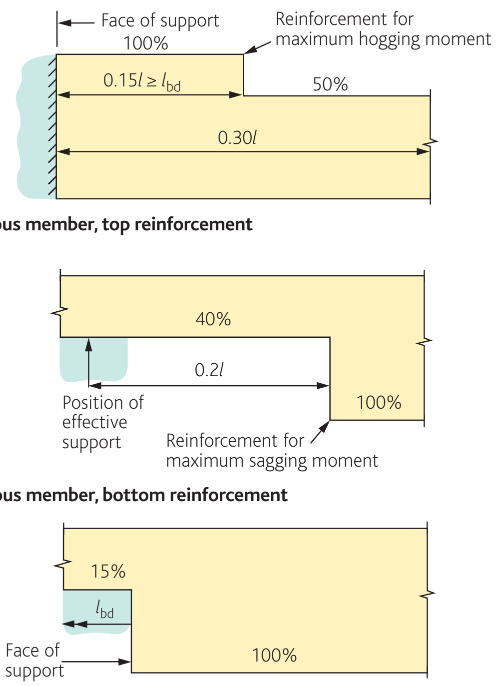
- For the principal reinforcement: 3h but not more than 400 mm.
- For the secondary reinforcement: 3.5h but not more than 450 mm.

The exception is in areas with concentrated loads or areas of maximum moment where the following applies:

- For the principal reinforcement: 2h but not more than 250 mm.
- For the secondary reinforcement: 3h but not more than 400 mm.

For slabs 200 mm thick or greater, the spacing requirements are given in Table 5. Where the designer has not specified the required spacing or provided the steel stress, σ_s, it can generally be assumed that σ_s will not exceed 320 MPa for a typical slab. Where the slab supports office or residential areas it is unlikely that σ_s will exceed 280 MPa.

Figure 10
Simplified detailing rules for slabs



a) Continuous member, top reinforcement

b) Continuous member, bottom reinforcement

c) Simple support, bottom reinforcement

Notes

1 l is the effective length.
2 l_{bd} is the design anchorage length.
3 Q_k ≤ 1.25 G_k and q_k ≤ 5kN/m^2.
4 Minimum of two spans required.
5 Applies to uniformly distributed loads only.
6 The shortest span must be greater than or equal to 0.85 times the longest span.
7 Applies where 20% redistribution has been used.

Minimum areas of reinforcement

The minimum area of reinforcement to be provided varies with the concrete strength (see Table 4).

Maximum area of longitudinal reinforcement

Outside lap locations, the maximum area of tension or compression reinforcement, should not exceed $A_{s,max}$ = 0.04A_c. At lap locations $A_{s,max}$ = 0.08A_c.

Edge reinforcement

Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown in Figure 11.

Flat slabs

A flat slab should be divided into column and middle strips (see Figure 12); the division of the moments between the column and middle strips is given in Table 6.

Figure 11
Edge reinforcement for slab

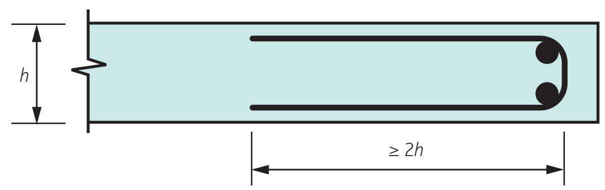


Table 5
Values for $\rho_{w,min}$

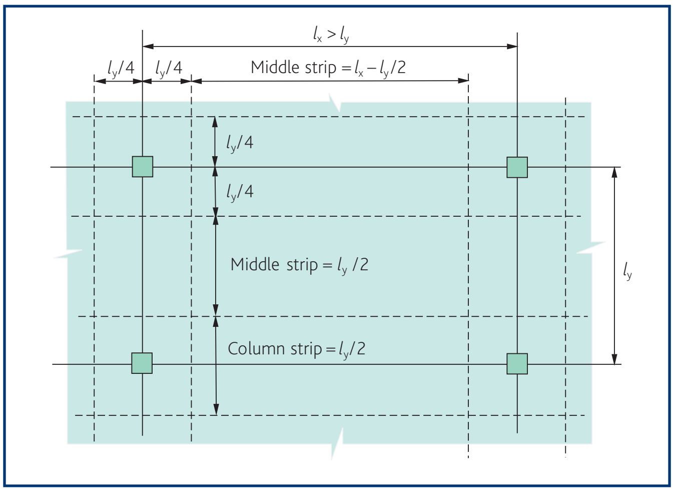
Steel stress (ε_s) MPa	w_{max} = 0.4 mm			w_{max} = 0.3 mm		
	Maximum bar size (mm)		Maximum bar spacing (mm)	Maximum bar size (mm)		Maximum bar spacing (mm)
160	40	OR	300	32	OR	300
200	32		300	25		250
240	20		250	16		200
280	16		200	12		150
320	12		150	10		100
360	10		100	8		50

Table 6
Apportionment of bending moments in flat slabs – equivalent frame method

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

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

Figure 12
Division of panels in flat slabs



At internal columns, unless rigorous serviceability calculations are carried out, top reinforcement of area 0.5 A_t should be placed in a width equal to the sum of 0.125 times the panel width on either side of the column. A_t represents the area of reinforcement required to resist the full negative moment from the sum of the two half panels each side of the column. It is also advisable to apply this requirement to perimeter columns as far as is possible.

At internal columns at least two bars of bottom reinforcement in each orthogonal direction should be provided and they should pass between the column reinforcement. Whilst it is not a code requirement it is considered good practice to provide two bars running parallel to the slab edge between the reinforcement of an external column.

Reinforcement perpendicular to a free edge required to transmit bending moments from the slab to an edge or corner column should be placed within the effective width, b_e, shown in Figure 13.

Punching shear reinforcement

Where punching shear reinforcement is required the following rules should be observed.

- It should be provided between the face of the column and kd inside the outer perimeter where shear reinforcement is no longer required. k is 1.5, unless the perimeter at which reinforcement is no longer required is less than $3d$ from the face of the column. In this case the reinforcement should be placed in the zone $0.3d$ to $1.5d$ from the face of the column.
- There should be at least two perimeters of shear links.
- The radial spacing of the links, s_r should not exceed $0.75d$ (see Figure 14).
- The tangential spacing of the links, s_d should not exceed $1.5d$ within $2d$ of the column face.
- The tangential spacing of the links should not exceed $2d$ for any other perimeter.
- The distance between the face of the column and the nearest shear reinforcement should be between $0.3d$ and $0.5d$

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

The minimum area of a link leg for vertical punching shear reinforcement is

$1.5A_{sw,min}/(s_r s_t) \geq 0.08\sqrt{f_{ck}}/f_{yk}$

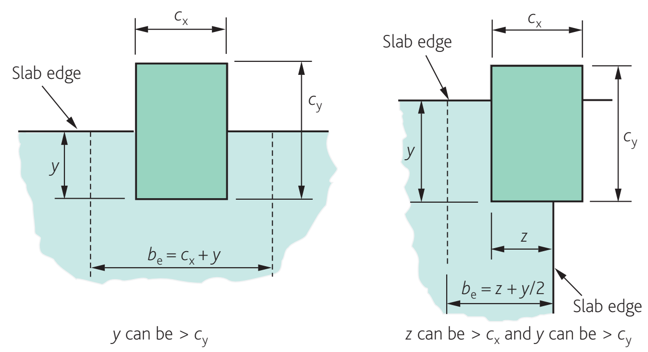
which can be rearranged as:

$A_{sw,min} \geq (s_r s_t)/F$

where

s_r = the spacing of the links in the radial direction
s_t = the spacing of the links in the tangential direction
F = factor obtained from Table 7

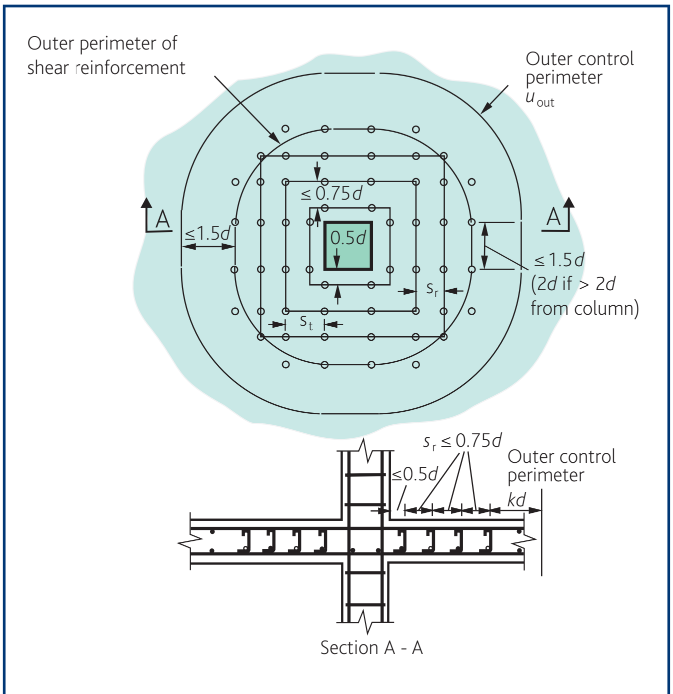
Figure 13
Effective width, b_e of a flat slab



a) Edge column **b) Corner column**

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

Figure 14
Punching shear layout



Columns and walls

Maximum areas of reinforcement

In Eurocode 2 the maximum nominal reinforcement area for columns and walls outside laps is 4% compared with 6% in BS 8110. However, this area can be increased provided that the concrete can be placed and compacted sufficiently. Self-compacting concrete may be used for particularly congested situations, where the reinforcing bars should be spaced to ensure that the concrete can flow around them. Further guidance can be found in *Self-compacting concrete*[12].

Minimum reinforcement requirements

The recommended minimum diameter of longitudinal reinforcement in columns is 12 mm. The minimum area of longitudinal reinforcement in columns is given by: $A_{s,min} = 0.10\, N_{Ed}/f_{yd} \geq 0.002A_c$. The diameter of the transverse reinforcement (link) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars (see Table 8). No longitudinal bar should be more than 150 mm from a transverse bar.

Particular requirements for walls

The minimum area of vertical reinforcement in walls is given by: $A_{s,vmin} = 0.002A_c$ (see also Table 9). Half the area should be provided in each face.

The distance between two adjacent vertical bars should not exceed the lesser of either three times the wall thickness or 400 mm.

The minimum area of horizontal reinforcement in each face of a wall is the greater of either 25% of vertical reinforcement or $0.001A_c$. However, where crack control is important, early age thermal and shrinkage effects should be considered.

There is no advice given in the Code on provision of reinforcement to control cracking in plain walls, but reinforcement may be provided if required.

Table 7
Factor, *F*, for determining $A_{sw,min}$

f_{ck}	25	28	30	32	35	40	45	50
Factor, *F*	1875	1772	1712	1657	1585	1482	1398	1326

Note
f_{yk} has been taken as 500 MPa

Table 8
Requirements for column reinforcement

Bar dia. (mm)	12	16	20	25	32	40
Max spacing[a] (mm)	144[b]	192[b]	240[b]	240[b]	240[b]	240[b]
Min link dia. (mm)	6[c]	6[c]	6[c]	8	8	10

Key
- **a** At a distance greater than the larger dimension of the column above or below a beam or slab, dimensions can be increased by a factor of 1.67.
- **b** But not greater than minimum dimension of the column.
- **c** 6 mm bars are not readily available in the UK.

Lapping fabric

Unless 'flying end' fabric is being specified, laps of fabric should be arranged as shown in Figure 15. When fabric reinforcement is lapped by layering, the following should be noted:

- Permissible percentage of fabric main reinforcement that may be lapped in any section is 100% if $(A_s/s) \leq 1200$ mm²/m (where s is the spacing of bars) and 60% if $A_s/s > 1200$ mm²/m.
- All secondary reinforcement may be lapped at the same location and the minimum lap length $l_{0,min}$ for layered fabric is as follows:

  ≥ 150 mm for $\phi \leq 6$ mm
  ≥ 250 mm for 6 mm < $\phi \leq 8.5$ mm
  ≥ 350 mm for 8.5 mm < $\phi \leq 12$ mm

There should generally be at least two bar pitches within the lap length. This could be reduced to one bar pitch for $\phi \leq 6$ mm.

Tolerances

The tolerances for cutting and/or bending dimensions are given in Table 10 and should be taken into account when completing the bar schedule.

Where the reinforcement is required to fit between two concrete faces (e.g. links) then an allowance should be made for deviations in the member size and bending tolerances. There is no guidance given in Eurocode 2, but Table 11 gives guidance on the deductions to be made for deviations.

Table 9
Minimum area of vertical reinforcement in walls (half in each face)

Wall thickness (mm)	$A_{s,min}$/m length of wall (mm²)
200	400
250	500
300	600
350	700
400	800

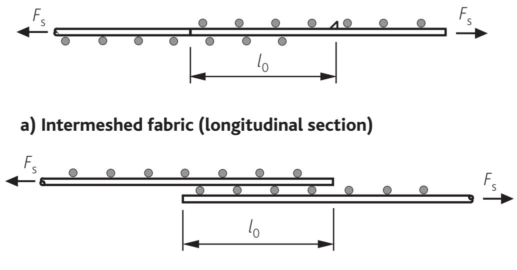
Table 10
Tolerance

Cutting and bending processes	Tolerance (mm)
Cutting of straight lengths (including reinforcement for subsequent bending)	+25, –25
Bending:	
≤ 1000 mm	+5, –5
> 1000 mm to ≤ 2000 mm	+5, –10
> 2000 mm	+5, –25

Table 11
Deductions to bar dimensions to allow for deviations between two concrete faces

Distance between concrete faces (mm)	Type of bar	Total deduction (mm)
0 – 1000	Links and other bent bars	10
1000 – 2000	Links and other bent bars	15
Over 2000	Links and other bent bars	20
Any length	Straight bars	40

Figure 15
Lapping of welded fabric



a) Intermeshed fabric (longitudinal section)

b) Layered fabric (longitudinal section)

Tying requirements

At each floor and roof level an effectively continuous peripheral tie should be provided within 1.2 m from the edge; this need not be additional reinforcement. In practice, for most buildings the tie should resist a tensile force of 60 kN. An area of reinforcement of 138 mm² is sufficient to resist this force.

Internal ties should be provided at each floor and roof level in two directions approximately at right angles. They should be effectively continuous throughout their length and should be anchored to the peripheral ties at each end, unless continuing as horizontal ties to columns or walls. The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. In walls they should be within 0.5 m from the top or bottom of floor slabs. In each direction, internal ties should be capable of resisting a design value of tensile force $F_{tie,int}$ (in kN per metre width):

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

where

$(q_k + g_k)$ = sum of the average permanent and variable floor loads (in kN/m²)

l_r = the greater of the distances (in m) between the centres of the columns, frames or walls supporting any two adjacent floor spans in the direction of the tie under consideration

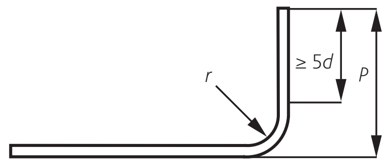
$F_t = (20 + 4n_0) \leq 60$ kN (n_0 is the number of storeys)

The maximum spacing of internal ties is $1.5l_r$.

Minimum radii and end projections

The minimum radii for bends and length of end projections are given in Table 12.

Table 12
Minimum scheduling radii and bend allowances



Nominal size of bar, *d* (mm)	Minimum radius for scheduling, *r* (mm)	Mimimum end projection, *P*	
		General (min 5*d* straight), including links where bend ≥ 150° (mm)	Links where bend < 150° (min 10*d* straight) (mm)
8	16	115[a]	115[a]
10	20	120[a]	130
12	24	125[a]	160
16	32	130	210
20	70	190	290
25	87	240	365
32	112	305	465
40	140		580

Key
a The minimum end projections for smaller bars is governed by the practicalities of bending bars.

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Table 13
Anchorage and lap lengths

		Bond condition (see Figure 1)	Reinforcement in tension, bar diameter, ϕ (mm)								Reinforcement in compression
			8	10	12	16	20	25	32	40	
Concrete class C20/25											
Anchorage length, l_{bd}	Straight bars only	Good	270	370	480	690	910	1180	1500	2040	47ϕ
		Poor	380	520	680	990	1290	1680	2150	2920	67ϕ
	Other bars	Good	370	470	570	750	940	1180	1500	2040	47ϕ
		Poor	530	670	810	1080	1340	1680	2150	2920	67ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	370	510	660	970	1270	1640	2100	2860	66ϕ
		Poor	530	730	950	1380	1810	2350	3000	4080	94ϕ
	100% lapped in one location (α_6 = 1.5)	Good	400	550	710	1030	1360	1760	2250	3060	70ϕ
		Poor	510	790	1010	1480	1940	2320	3220	4370	100ϕ
Concrete class C25/30											
Anchorage length, l_{bd}	Straight bars only	Good	230	320	410	600	780	1010	1300	1760	40ϕ
		Poor	330	450	580	850	1120	1450	1850	2510	58ϕ
	Other bars	Good	320	410	490	650	810	1010	1300	1760	40ϕ
		Poor	460	580	700	930	1160	1450	1850	2510	58ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	320	440	570	830	1090	1420	1810	2460	57ϕ
		Poor	460	630	820	1190	1560	2020	2590	3520	81ϕ
	100% lapped in one location (α_6 = 1.5)	Good	340	470	610	890	1170	1520	1940	2640	61ϕ
		Poor	490	680	870	1270	1670	2170	2770	3770	87ϕ
Concrete class C28/35											
Anchorage length, l_{bd}	Straight bars only	Good	210	300	380	550	730	940	1200	1630	37ϕ
		Poor	300	420	540	790	1030	1340	1720	2330	53ϕ
	Other bars	Good	300	380	450	600	750	940	1200	1630	37ϕ
		Poor	420	540	650	860	1070	1340	1720	2330	53ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	300	410	530	770	1010	1320	1680	2280	52ϕ
		Poor	420	590	760	1100	1450	1880	2400	3260	75ϕ
	100% lapped in one location (α_6 = 1.5)	Good	320	440	570	830	1090	1410	1800	2450	56ϕ
		Poor	450	630	810	1180	1550	2010	2570	3470	80ϕ
Concrete class C30/37											
Anchorage length, l_{bd}	Straight bars only	Good	210	280	360	530	690	900	1150	1560	36ϕ
		Poor	290	400	520	750	990	1280	1640	2230	51ϕ
	Other bars	Good	290	360	430	580	720	900	1150	1560	36ϕ
		Poor	410	520	620	820	1030	1280	1640	2230	51ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	290	390	510	740	970	1260	1610	2180	50ϕ
		Poor	410	560	720	1050	1380	1790	2290	3110	72ϕ
	100% lapped in one location (α_6 = 1.5)	Good	310	420	540	790	1040	1350	1720	2340	54ϕ
		Poor	430	600	780	1130	1480	1920	2460	3340	77ϕ
Concrete class C32/40											
Anchorage length, l_{bd}	Straight bars only	Good	200	270	350	510	660	860	1100	1490	34ϕ
		Poor	280	380	500	720	950	1230	1570	2130	49ϕ
	Other bars	Good	270	350	420	550	690	860	1100	1490	34ϕ
		Poor	390	490	590	790	980	1230	1570	2130	49ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	270	380	490	710	930	1200	1540	2090	48ϕ
		Poor	390	540	690	1010	1320	1720	2200	2980	69ϕ
	100% lapped in one location (α_6 = 1.5)	Good	290	400	520	760	990	1290	1650	2240	51ϕ
		Poor	420	570	740	1080	1420	1840	2350	3200	73ϕ

		Bond condition (see Figure 1)	Reinforcement in tension, bar diameter, ϕ (mm)								Reinforcement in compression
			8	10	12	16	20	25	32	40	
Concrete class C35/45											
Anchorage length, l_{bd}	Straight bars only	Good	190	260	330	480	630	810	1040	1410	32ϕ
		Poor	260	360	470	680	890	1160	1480	2010	46ϕ
	Other bars	Good	260	330	390	520	650	810	1040	1410	32ϕ
		Poor	370	470	560	740	930	1160	1480	2010	46ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	260	360	460	670	870	1130	1450	1970	45ϕ
		Poor	370	510	650	950	1250	1620	2070	2810	65ϕ
	100% lapped in one location (α_6 = 1.5)	Good	280	380	490	710	940	1210	1550	2110	48ϕ
		Poor	390	540	700	1020	1340	1730	2220	3010	69ϕ
Concrete class C40/50											
Anchorage length, l_{bd}	Straight bars only	Good	170	230	300	440	570	740	950	1290	30ϕ
		Poor	240	330	430	620	820	1060	1350	1840	42ϕ
	Other bars	Good	240	300	360	480	600	740	950	1290	30ϕ
		Poor	340	430	510	680	850	1060	1350	1840	42ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	240	330	420	610	800	1040	1330	1800	41ϕ
		Poor	340	460	600	870	1140	1480	1890	2570	59ϕ
	100% lapped in one location (α_6 = 1.5)	Good	250	350	450	650	860	1110	1420	1930	44ϕ
		Poor	360	500	640	930	1220	1590	2030	2760	63ϕ
Concrete class C45/55											
Anchorage length, l_{bd}	Straight bars only	Good	160	220	280	400	530	690	880	1190	27ϕ
		Poor	220	310	400	580	760	980	1250	1700	39ϕ
	Other bars	Good	220	280	330	440	550	690	880	1190	27ϕ
		Poor	310	390	470	630	780	980	1250	1700	39ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	220	300	390	560	740	960	1230	1670	38ϕ
		Poor	310	430	550	800	1060	1370	1750	2380	55ϕ
	100% lapped in one location (α_6 = 1.5)	Good	230	320	420	600	790	1030	1310	1780	41ϕ
		Poor	330	460	590	860	1130	1470	1880	2550	58ϕ
Concrete class C50/60											
Anchorage length, l_{bd}	Straight bars only	Good	150	200	260	380	490	640	820	1110	25ϕ
		Poor	210	290	370	540	700	910	1170	1580	36ϕ
	Other bars	Good	220	280	330	440	550	690	880	1190	27ϕ
		Poor	310	390	470	630	780	980	1250	1700	39ϕ
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	220	300	390	560	740	960	1230	1670	38ϕ
		Poor	310	430	550	800	1060	1370	1750	2380	55ϕ
	100% lapped in one location (α_6 = 1.5)	Good	230	320	420	600	790	1030	1310	1780	41ϕ
		Poor	330	460	590	860	1130	1470	1880	2550	58ϕ

Notes

1. Cover to all sides and distance between bars ≥ 25 mm (i.e. $\alpha_2 < 1$).
2. $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$.
3. Design stress has been taken at 435 MPa. Where the design stress in the bar at the position from where the anchorage is measured, σ_{sd}, is less than 435 MPa the figures in this table can be factored by $\sigma_{sd}/435$. The mimimum lap length id given in cl 8.7.3 of Eurocode 2.
4. The anchorage and lap lengths have been rounded up to the nearest 10 mm.
5. Where 33% of bars are lapped in one location, decrease the lap lengths for '50% lapped in one location' by a factor of 0.82.

Table 14
Sectional areas of groups of bars (mm²)

Bar size (mm)	Number of bars									
	1	2	3	4	5	6	7	8	9	10
8	50.3	101	151	201	251	302	352	402	452	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	565	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	942	1260	1570	1880	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

Table 15
Sectional areas per metre width for various spacings of bars (mm²)

Bar size (mm)	Spacing of bars (mm)									
	75	100	125	150	175	200	225	250	275	300
8	670	503	402	335	287	251	223	201	183	168
10	1050	785	628	524	449	393	349	314	286	262
12	1510	1130	905	754	646	565	503	452	411	377
16	2680	2010	1610	1340	1150	1010	894	804	731	670
20	4190	3140	2510	2090	1800	1570	1400	1260	1140	1050
25	6540	4910	3930	3270	2800	2450	2180	1960	1780	1640
32	10700	8040	6430	5360	4600	4020	3570	3220	2920	2680
40	16800	12600	10100	8380	7180	6280	5590	5030	4570	4190

Table 16
Mass of groups of bars (kg per metre run)

Bar size (mm)	Number of bars									
	1	2	3	4	5	6	7	8	9	10
8	0.395	0.789	1.184	1.578	1.973	2.368	2.762	3.157	3.551	3.946
10	0.617	1.233	1.850	2.466	3.083	3.699	4.316	4.932	5.549	6.165
12	0.888	1.776	2.663	3.551	4.439	5.327	6.215	7.103	7.990	8.878
16	1.578	3.157	4.735	6.313	7.892	9.470	11.048	12.627	14.205	15.783
20	2.466	4.932	7.398	9.865	12.331	14.797	17.263	19.729	22.195	24.662
25	3.853	7.707	11.560	15.413	19.267	23.120	26.974	30.827	34.680	38.534
32	6.313	12.627	18.940	25.253	31.567	37.880	44.193	50.507	56.820	63.133
40	9.865	19.729	29.594	39.458	49.323	59.188	69.052	78.917	88.781	98.646

Table 17
Mass in kg per square metre for various spacings of bars (kg per m²)

Bar size (mm)	Spacing of bars (mm)									
	75	100	125	150	175	200	225	250	275	300
8	5.261	3.946	3.157	2.631	2.255	1.973	1.754	1.578	1.435	1.315
10	8.221	6.165	4.932	4.110	3.523	3.083	2.740	2.466	2.242	2.055
12	11.838	8.878	7.103	5.919	5.073	4.439	3.946	3.551	3.228	2.959
16	21.044	15.783	12.627	10.522	9.019	7.892	7.015	6.313	5.739	5.261
20	32.882	24.662	19.729	16.441	14.092	12.331	10.961	9.865	8.968	8.221
25	51.378	38.534	30.827	25.689	22.019	19.267	17.126	15.413	14.012	12.845
32	84.178	63.133	50.507	42.089	36.076	31.567	28.059	25.253	22.958	21.044
40	131.528	98.646	78.917	65.764	56.369	49.323	43.843	39.458	35.871	32.882

How to design concrete structures using Eurocode 2

11. BS 8500 for building structures

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Introduction

BS 8500 Concrete – Complementary British Standard to BS EN 206–1[1] was revised in December 2006 principally to reflect changes to *Special Digest 1*[2] and bring it into line with other standards.

The guidelines given in BS 8500 for durability are based on the latest research and recommends strength, cover, cement content and water/cement ratios for various exposure conditions.

Concrete design information

Exposure classification

Initially the relevant exposure condition(s) should be identified. In BS 8500 exposure classification is related to the deterioration processes of carbonation, ingress of chlorides, chemical attack from aggressive ground and freeze/thaw (see Table 1). All of these deterioration processes are sub-divided. The recommendations for XD and XS exposure classes are sufficient for exposure class XC and it is only necessary to check each face of the concrete element for either XC, XD or XS exposure class.

Selecting concrete strength and cover

Having identified the relevant exposure condition(s), a recommended strength class and cover should be chosen. Table 2 indicates the minimum cover and strengths required to meet common exposure conditions for a 50-year working life; further explanation is given below. Table 2 is not intended to cover all concrete exposure situations and reference should be made to BS 8500 for those cases not included, and where a 100-year working life is required.

Compressive strength

BS 8500 uses 'compressive strength class' to define concrete strengths; the notation used gives the cylinder strength as well as the cube strength (see Table 3). It is important to quote the compressive strength class in full to avoid confusion.

Cover to reinforcement

The durability guidance given in BS 8500 is based on the assumption that the minimum cover for durability is achieved. An allowance should be made in the design for deviations from the minimum cover (Δc_{dev}). This should be added to the minimum cover to obtain the nominal cover.

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Table 1
Exposure Classes

Class	Class description	Informative example applicable to the United Kingdom
No risk of corrosion or attack (XO class)		
X0	For concrete without reinforcement or embedded metal where there is no significant freeze/thaw, abrasion or chemical attack.	Unreinforced concrete surfaces inside structures. Unreinforced concrete completely buried in soil classed as AC-1 and with hydraulic gradiant not greater than 5. Unreinforced concrete permanently submerged in non-aggressive water. Unreinforced concrete in cyclic wet and dry conditions not subject to abrasion, freezing or chemical attack. **NOTE:** *For reinforced concrete, use at least XC1.*
Corrosion induced by carbonation (XC classes)[a] *(Where concrete containing reinforcement or other embedded metal is exposed to air and moisture.)*		
XC1	Dry or permanently wet.	Reinforced and prestressed concrete surfaces inside enclosed structures except areas of structures with high humidity. Reinforced and prestressed concrete surfaces permanently submerged in non-aggressive water.
XC2	Wet, rarely dry.	Reinforced and prestressed concrete completely buried in soil classed as AC-1 and with a hydraulic gradient not greater than 5. For other situations see 'chemical attack' section below.
XC3 & **XC4**	Moderate humidity or cyclic wet and dry.	External reinforced and prestressed concrete surfaces sheltered from, or exposed to, direct rain. Reinforced and prestressed concrete surfaces inside structures with high humidity (e.g. poorly ventilated, bathrooms, kitchens). Reinforced and prestressed concrete surfaces exposed to alternate wetting and drying.
Corrosion induced by chlorides other than from sea water (XD classes)[a] *(Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water.)*		
XD1	Moderate humidity.	Concrete surfaces exposed to airborne chlorides. Parts of structures exposed to occasional or slight chloride conditions.
XD2	Wet, rarely dry.	Reinforced and prestressed concrete surfaces totally immersed in water containing chlorides[b].
XD3	Cyclic wet and dry.	Reinforced and prestressed concrete surfaces directly affected by de-icing salts or spray containing de-icing salts (e.g. walls; abutments and columns within 10 m of the carriageway; parapet edge beams and buried structures less than 1 m below carriageway level, pavements and car park slabs).
Corrosion induced by chlorides from sea water (XS classes)[a] *(Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water.)*		
XS1	Exposed to airborne salt but not in direct contact with sea water.	External reinforced and prestressed concrete surfaces in coastal areas.
XS2	Permanently submerged.	Reinforced and prestressed concrete completely submerged and remaining saturated, e.g. concrete below mid-tide level[b].
XS3	Tidal, splash and spray zones.	Reinforced and prestressed concrete surfaces in the upper tidal zones and the splash and spray zones [c].
Freeze/thaw attack (XF classes) *(Where concrete is exposed to significant attack from freeze/thaw cycles whilst wet.)*		
XF1	Moderate water saturation without de-icing agent.	Vertical concrete surfaces such as facades and columns exposed to rain and freezing. Non-vertical concrete surfaces not highly saturated, but exposed to freezing and to rain or water.
XF2	Moderate water saturation with de-icing agent.	Elements such as parts of bridges, which would otherwise be classified as XF1 but which are exposed to de-icing salts either directly or as spray or run-off.
XF3	High water saturation without de-icing agent.	Horizontal concrete surfaces, such as parts of buildings, where water accumulates and which are exposed to freezing. Elements subjected to frequent splashing with water and exposed to freezing.
XF4	High water saturation with de-icing agent or sea water [d].	Horizontal concrete surfaces, such as roads and pavements, exposed to freezing and to de-icing salts either directly or as spray or run-off. Elements subjected to frequent splashing with water containing de-icing agents and exposed to freezing.
Chemical attack (ACEC classes) *(Where concrete is exposed to chemical attack.) Note: BS 8500-1 refers to ACEC classes rather than XA classes used in BS EN 206-1*		

Key

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

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

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

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

Table 2

Selected[a] recommendations for normal-weight reinforced concrete quality for combined exposure classes and cover to reinforcement for at least a 50-year intended working life and 20 mm maximum aggregate size

Exposure conditions			Cement/ combination designations[b]	Strength class[c], maximum w/c ratio, minimum cement or combination content (kg/m³), and equivalent designated concrete (where applicable)							
Typical example	Primary	Secondary		Nominal cover to reinforcement[d]							
				$15 + \Delta c_{dev}$	$20 + \Delta c_{dev}$	$25 + \Delta c_{dev}$	$30 + \Delta c_{dev}$	$35 + \Delta c_{dev}$	$40 + \Delta c_{dev}$	$45 + \Delta c_{dev}$	$50 + \Delta c_{dev}$
Internal mass concrete	**X0**	—	All	Recommended that this exposure is not applied to reinforced concrete							
Internal elements (except humid locations)	**XC1**	—	All	C20/25, 0.70, 240 or RC20/25	<<<	<<<	<<<	<<<	<<<	<<<	<<<
Buried concrete in AC-1 ground conditions[e]	**XC2**	**AC-1**	All	—	—	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<	<<<	<<<
Vertical surface protected from direct rainfall	**XC3** & **XC4**	—	All except IVB	—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	C25/30, 0.65, 260 or RC25/30	<<<	<<<	<<<
Exposed vertical surfaces		**XF1**	All except IVB	—	C40/50, 0.45, 340 or RC40/50	C30/37, 0.55, 300 or RC30/37	C28/35, 0.60, 280 or RC28/35	<<<	<<<	<<<	<<<
Exposed horizontal surfaces		**XF3**	All except IVB	—	C40/50,0.45, 340[g] or RC40/50XF[g]	<<<	<<<	<<<	<<<	<<<	<<<
		XF3 (air entrained)	All except IVB	—	—	C32/40, 0.55, 300 plus air[g,h]	C28/35, 0.60, 280 plus air[g,h] or PAV2	C25/30, 0.60, 280 plus air[g,h,j] or PAV1	<<<	<<<	<<<
Elements subject to airborne chlorides	**XD1**[f]	—	All	—	—	C40/50, 0.45, 360	C32/40, 0.55, 320	C28/35, 0.60, 300	<<<	<<<	<<<
Car park decks and areas subject to de-icing spray	**XD3**[f]	—	IIB-V, IIIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C28/35, 0.50, 340
			CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
			IIIB, IVB-V	—	—	—	—	—	C32/40, 0.40, 380	C28/35, 0.45, 360	C25/30, 0.50, 340
Vertical elements subject to de-icing spray and freezing		**XF2**	IIB-V, IIIA	—	—	—	—	—	C35/45, 0.40, 380	C32/40, 0.45, 360	C32/40, 0.50, 340
			CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380	C35/45, 0.45, 360
			IIIB, IVB-V	—	—	—	—	—	C32/40, 0.40, 380	C32/40 0.45, 360	C32/40, 0.50, 340
Car park decks, ramps and external areas subject to freezing and de-icing salts		**XF4**	CEM I, IIA, IIB-S, SRPC	—	—	—	—	—	See BS 8500	C40/50, 0.40, 380[g]	<<<
		XF4 (air entrained)	IIB-V, IIIA, IIIB	—	—	—	—	—	C28/35, 0.40, 380[g,h]	C28/35 0.45, 360[g,h]	C28/35, 0.50, 340[g,h]
Exposed vertical surfaces near coast	**XS1**[f]	**XF1**	CEM I, IIA, IIB-S, SRPC	—	—	—	See BS 8500	C35/45, 0.45, 360	C32/40, 0.50, 340	<<<	<<<
			IIB-V, IIIA	—	—	—	See BS 8500	C32/40, 0.45, 360	C28/35, 0.50, 340	C25/30, 0.55, 320	<<<
			IIIB	—	—	—	C32/40, 0.40, 380	C25/30, 0.50, 340	C25/30, 0.50, 340	C25/30, 0.55, 320	<<<
Exposed horizontal surfaces near coast		**XF4**	CEM I, IIA, IIB-S, SRPC	—	—	—	See BS 8500	C40/50, 0.45, 360[g]	<<<	<<<	<<<

Key

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

b See BS 8500-2, Table 1. (CEM I is Portland cement, IIA to IVB are cement combinations.)

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

d Δc_{dev} is an allowance for deviations.

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

f Also adequate for exposure class XC3/4.

g Freeze/thaw resisting aggregates should be specified.

h Air entrained concrete is required.

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

— Not recommended

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

Eurocode 2[4] recommends that Δc_{dev} is taken as 10 mm, unless the fabrication is subjected to a quality assurance ystem where it is permitted to reduce Δc_{dev} to 5 mm, or 0 mm if the element can be rejected if it is out of tolerance (e.g. precast elements).

Cement types and minimum cement content

Table 4 may be used to understand the cement/combination designations. It should be noted from Table 2 that the strength, water/cement ratio and minimum cement content may vary depending on the cement type used. In the UK, all cement/combinations are available (except SRPC), although in most concrete production plants either ground granulated blastfurnace slag or flyash (pfa) is available; not both. When using a designated concrete (see section below), it is not necessary to specify the types of cement/combinations.

Explanation of the compressive strength class notation

C	40/	50
'C' for normal weight concrete[A] 'LC' for lightweight concrete	Cylinder strength[B]	Cube strength[C]

A Includes heavyweight concrete

B Minimum characteristic 150 mm diameter by 300 mm cylinder strength, N/mm²

C Minimum characteristic cube strength, N/mm²

Air content

Where air entrainment is required for exposure classes XF3 and XF4 the minimum air content by volume of 3.0%, 3.5% or 5.5% should be specified for 40 mm, 20 mm and 10 mm maximum aggregate size respectively.

Freeze/thaw aggregates

For exposure conditions XF3 and XF4 freeze/thaw resisting aggregates should be specified. The producer is then obliged to conform to the requirements given in BS 8500–2: 2006, Cl.4.3.

Aggressive ground

Where plain or reinforced concrete is in contact with the ground further checks are required to ensure durability. An aggressive chemical environment for concrete class (ACEC class) should be assessed for the site. BRE *Special Digest* 1[2] gives guidance on the assessment of the ACEC class and this is normally carried out as part of the interpretive reporting for a ground investigation. Knowing the ACEC class, a design chemical class (DC class) can be obtained from Table 5.

For designated concretes, an appropriate foundation concrete (FND designation) can be selected using Table 6; the cover should be determined from Table 2 for the applicable exposure classes. A FND concrete has the strength class of C25/30, therefore, where a higher strength is required a designed concrete should be specified. For designed concretes, the concrete producer should be advised of the DC-class (see section on specification).

Table 3
Compressive strength class for normal and heavyweight concrete

Example Compressive strength classes (BS 8500)	Designated concrete (BS 8500)	Previous Grade of concrete (BS 5328[3] & BS 8110[5])
C20/25	RC20/25	C25
C25/30	RC25/30	C30
C28/35	RC28/35	C35
C30/37	RC30/37	–
C32/40	RC32/40	C40
C35/45	RC35/45	C45
C40/50	RC40/50	C50
C45/55	–	–
C50/60	–	C60

NOTE: Refer to BS 8500-1: 2006, Table A.20 for full list of Compressive strength classes.

Table 4
Cement and combination type[a]

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

Key

a There are a number of cements and combinations not listed in this table that may be specified for certain specialist applications. See BRE Special Digest 1[2]. for the sulfate-resisting characteristics of other cements and combinations.

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

c When IIA or IIA-D is specified, CEM I and silica fume may be combined in the concrete mixer using the *k*-value concept; see BS EN 206-1:200, 5.2.5.2.3.

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

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

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

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

Table 5
Selection of the DC-class and the number of Addition Protection Measures (APMs) where the hydrostatic head of groundwater is not more than five times the section width [a, b, c, d, e]

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

Key
- **a** Where the hydrostatic head of groundwater is greater than five times the section width, refer to BS 8500.
- **b** For guidance on precast products see Special Digest 1[2].
- **c** For structural performance outside these values refer to BS 8500.
- **d** For section widths < 140 mm refer to BS 8500.
- **e** Where any surface attack is not acceptable e.g. with friction piles, refer to BS 8500.
- **f** This should include APM3 (surface protection), where practicable, as one of the APMs; refer to BS 8500.

Table 6
Guidance on selecting designated concrete for reinforced concrete foundations

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

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

Fire design
Having selected concrete cover and strength to meet the durability recommendations of BS 8500, the nominal cover should be checked in accordance with Eurocode 2[4], for fire cover.

Concrete cast against uneven surfaces
The nominal cover (i.e. minimum cover plus fixing tolerance) should be a minimum of 75 mm for concrete cast directly against the earth and 50 mm for concrete cast against blinding.

Abrasion
BS 8500 does not contain abrasion classes; instead reference should be made to BS 8204-2[6] or Concrete Society Technical Report 34[7]. Table 7 summarises the factors that affect the abrasion resistance of floors.

Specification

Method of specifying
There are various methods of specifying concrete to BS 8500 (see Table 8). The most popular are designated and designed. BS 8500 also introduces a new method 'proprietary concrete'.

The specifier
Figures 1 and 2 show standard specification forms produced by the Quarry Products Association for designated and designed concretes[8]. Similar tables are included in the National Structural Concrete Specification[9] (NSCS). In BS 8500 the 'specifier' is the person or body responsible for the final compilation of the technical requirements, called the specification, which is passed to the concrete producer. This will generally be the contractor, however, the designer will want to ensure their requirements are incorporated and this will normally be through their own specification for the works (e.g. with the NSCS). Figures 1 and 2 have been annotated to indicate which information is typically provided by the designer and contractor. The designer should require that any reported non-conformities are passed to them for assessment.

Consistence
The term 'workability' has been replaced by the term 'consistence' and a series of consistence classes has been introduced. Table 9 gives the slump and flow classes and the likely target slump/flow.

Chloride Class
Concrete that is to be prestressed, pre-tensioned or heat cured should normally be specified as chloride class Cl0,10. Reinforced concrete should be specified as class Cl0,40 except for concrete made with cement conforming to BS 4027[10] (SRPC), which should be specified as class Cl0,20. Post-tensioned elements in an internal building environment may also be specifed as class Cl0,10.

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Table 7
Factors affecting the abrasion resistance of concrete floors

Factor	Effect
Power floating	Power finishing and, in particular, repeated power trowelling is a significant factor in creating abrasion resistance, however, excessive repetitions of the process do not necessarily further enhance performance.
Curing	Prompt and efficient curing is essential in order to retain sufficient water in the surface zone to complete hydration and the development of concrete strength at and close to the surface.
Cement content	Cement content should not be less than 325 kg/m^3. Cement contents above 360 kg/m^3 are unlikely to enhance abrasion resistance and excessive cement content can impair the power finishing process.
Water/cement ratio	Water/cement ratio is of great importance. It should not exceed 0.55. Reducing to 0.50 is likely to increase abrasion resistance but lowering further is unlikely to give further enhancement.
Aggregates	Coarse aggregate usually has no direct effect on abrasion resistance, except in floors in very aggressive environments where the surface is expected to be worn away. Coarse and fine aggregates should not contain soft or friable materials.
Dry shake finishes	Dry shake finishes can be used to enhance the surface properties in high abrasion locations.

Figure 1
Example specification of Designated Concrete

Schedule for the specification requirements of designated concretes for use on contract

Contract Title: New Office

Contract period: June - Dec '04

BS 8500-1 reference	Requirement	Schedule			
4.2.2a)	The concretes below shall be supplied as Designated Concretes in accordance with this specification and the relevant clauses of BS 8500-2[A]				
4.2.2b) D	Concrete designation	FND2z	RC25/30	RC32/40	D
4.2.2c) D	Maximum aggregate size when other than 20 mm	—	—	10	D
4.2.2d) C	Consistence (Ring the class required when other than the default classes of S3 for the GEN, FND and RC series and S2 for the PAV series. Use a separate column for different consistence with the same designated concrete) Other (specify)	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5 C
4.2.3 C D	Additional requirements	—	—	—	C D
Exchange of information					
BS EN 206-1, 7.1 C	Total volume required Anticipated peak delivery rate Any access limitations	48 m^3 6 m^3/day	1200 m^3 18 m^3/hr	72 m^3 6 m^3/day	C
5.1a) C	Intended method of placing, e.g. pumping, and finishing, e.g. power floating, the concrete	Skip + tamped	Pumping + float	Skip + tamped	C
5.1b) C	Where identity testing is routine: Type of test Volume of concrete in assessment Number of tests on this volume Whether a non-accredited laboratory will be used	N/A	N/A	N/A	C
5.1 & BS EN 206-1, 7.1 C	Other information from the specifier to the producer	—	—	—	C
5.2 & BS EN 206-1, 7.2 C	Information required from the producer	—	—	—	C

A There is no need to cite BS EN 206-1 as BS 8500-2 has a clause that requires conformity to BS EN 206-1.

KEY

D Designer specifies concrete designation, maximum aggregate size and any additional requirements

C Contractor specifies consistence, any additional requirements and completes exchange of information section

Red text Example specification

Table 8
Methods of specifying concrete

BS 8500	BS 5328 (superseded by BS 8500 1 Dec 2003)
Designated concrete	Designated mix
Designed concrete	Designed mix
Prescribed concrete	Prescribed mix
Standardized prescribed concrete	Standard mix
Proprietary concrete	No equivalent

Figure 2
Example specification of Designed Concrete

Schedule for the specification requirements of designed concretes for use on contract

Contract Title: New Office

Contract period: June - Dec '04

BS 8500-1 reference	Requirement	Schedule			
4.3.2a)	The concretes below shall be supplied as designed concretes in accordance with this specification and the relevant clauses of BS 8500-2[A]				
D	Concrete reference, if any	Pads	Slab	Cols	D
4.3.2b) D	Compressive strength class	C28/35	C25/30	C32/40	D
4.3.2c) D	For sulfate resisting concrete, design chemical class	DC-2z	DC-	DC-	DC- D
D	For other concretes, limiting values of composition: Maximum w/c ratio Minimum cement/combination content, kg/m^3		0.70 240	0.55 300	D
4.3.2d) & 4.3.3a) D	Cement or combination types (delete those not permitted) Other special property, e.g. white, low heat, +SR (specify)	CEM 1, SRPC, IIA, IIB IIIA, IIIB, IVB	CEM 1, SRPC, IIA, IIB IIIA, ~~IIIB~~, ~~IVB~~	CEM 1, SRPC, IIA, IIB IIIA, ~~IIIB~~, ~~IVB~~	CEM 1, SRPC, IIA, IIB IIIA, IIIB, IVB D
4.3.2e) D	Maximum aggregate size, mm	20	20	10	D
4.3.2f) D	Chloride class (ring the one required) Prestressed or heat cured reinforced concrete Reinforced [B] Unreinforced with no embedded metal	Cl 0,10 RC Cl 1,0	Cl 0,10 RC Cl 1,0	Cl 0,10 RC Cl 1,0	Cl 0,10 RC Cl 1,0 D
4.3.2g) & h) D	For lightweight and heavyweight concrete, target density				D
4.3.2i) C D	Consistence (Ring the class required. Use separate columns for the same basic concretes with different consistence) Other (specify)	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5	S1, S2, S3, S4 F2, F3, F4, F5 C D
4.3.2 Note 2 D	UKAS or equivalent accredited third party product conformity certification (delete if not required)	Yes	Yes	Yes	Yes D
4.3.3b) to n) C D	Additional requirements				C D
Exchange of information					
BS EN 206-1, 7.1 C	Volume required Anticipated peak delivery rate Any access limitations	48 m^3 6 m^3/day	1200 m^3 18 m^3/hr	72 m^3 6 m^3/day	C
5.1a) C	Intended method of placing, e.g. pumping, and finishing, e.g. power floating, the concrete	Skip + tamped	Pumping + float	Skip + tamped	C
5.1b) C	Where identity testing is routine: Type of test Volume of concrete in assessment Number of tests on this volume Whether a non-accredited laboratory will be used	N/A	N/A	N/A	C
5.1 & BS EN 206-1, 7.1 C	Other information from the specifier to the producer	—	—	—	C
5.2 & BS EN 206-1, 7.2 C	Information required from the producer	—	—	—	C

A There is no need to cite BS EN 206-1 as BS 8500-2 has a clause that requires conformity to BS EN 206-1.

B Where RC is ringed, the chloride class shall be Cl 0.40 except where SRPC is used. In this case the chloride class shall be Cl 0,20.

KEY

D Designer specifies compressive strength class, design chemical class, maximum water/cement ratio, minimum cement content, cement or combination types (unless design chemical class is specified), maximum aggregate size, chloride class, target density (excluding normal weight concrete), requirement for third party product conformity certification (recommended) and any additional requirements

C Contractor specifies consistence, any additional requirements and completes exchange of information section

Red text Example Specification

Table 9a
Consistence slump classes and likely target values

Slump class	Target slump (mm)
S1	20
S2	70
S3	130
S4	190

Table 9b
Consistence flow classes and likely target values

Flow class	Target flow (mm)
F2	380
F3	450
F4	520
F5	590

Conformity
Under BS 8500, the concrete producer is now required to follow a formal procedure called 'conformity' to verify that the concrete is in accordance with the specification. It is, therefore, recommended that the concrete supplier should have third party certification. Where this is not adopted, the specifier is advised to adopt adequate identity testing to ensure the concrete is as specified.

Identity testing
The specifier is responsible for organising any identity testing, which is in all but in name acceptance testing. Identity testing can include strength, consistence and air content. There are a number of situations where it is recommended:
- where the producer does not hold third party certification
- in cases of doubt
- for critical elements, e.g. high strength columns
- for spot checks on the producer.

Exchange of information
To enable the concrete producer to design and produce a suitable concrete, certain information must be provided in addition to the specification, e.g. where the concrete needs to be pumped or a high quality finish is required.

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How to Design Concrete Structures using Eurocode 2

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CCIP-006
Published December 2006
ISBN 1-984818-47-1
Price Group P

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