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Manual for the design of reinforced concrete building structures to EC2

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Constitution

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Foreword

The *Eurocode for the Design of Concrete Structures* (EC2) is likely to be published as a Euronorm (EN) in the next few years. The prestandard (ENV) for EC2 has now been available since 1992. To facilitate its familiarisation the Institution of Structural Engineers and the Institution of Civil Engineers decided to prepare a *Manual*.

This *Manual* follows in the footsteps of influential guides published by the Institution of Structural Engineers and uses the format of the green book (*Manual* for BS 8110). As with the green book the scope of the *Manual* covers the majority of concrete building structures and has now been extended to cover slender columns and prestressed concrete. An appendix for the structural design of foundations using limit state philosophy (as foreseen by ENV EC7), has also been included. It is hoped that this extended scope will be welcomed by practitioners. It is a helping hand not a substitute for the greater range in EC2.

The mandate to our Task Group was to produce a *Manual*, the results of which would comply with ENV EC2 and the UK National Application Document (NAD) – these are explained in Section 1 of the *Manual*. This latter requirement imposed certain constraints. Thus we had to use the UK boxed values (see section 1), refer to BS loading codes and use BS 8110 for fire design. During the drafting period, further ENV standards have been published, in particular for loading (EC1) and for fire design (EC2, Part 1.2). The designer can use this *Manual* in conjunction with these standards, but it should be noted that these ENV prestandards have their own NADs, which should be followed.

Designers should find this *Manual* concise and useful in practical design. This *Manual* is laid out for hand calculations, but the procedures are equally suitable for computer application.

Special thanks are due to all the members of the drafting Task Group (and their organisations) who have given their valuable time voluntarily, in particularly difficult economic circumstances in the industry. I am also grateful to Bob Milne of the Institution, for acting as my secretary with characteristic enthusiasm. Many members had commented on an earlier draft and these have contributed to its improvement. Users of this *Manual* should forward their comments to the Institution so that they can be taken into account in the next revision.

In 1992 the Committee started work under the chairmanship of David Lee who joins me in commending this *Manual* to the industry.

D. J. Lee
Chairman (until April 1995)

R. S. Narayanan
Chairman (since May 1995)

1 Introduction and scope

1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of reinforced and prestressed concrete building structures. Structures designed in accordance with this *Manual* will normally comply with DD ENV 1992-1-1: 1992[1] (hereinafter referred to as EC2).

1.2 Eurocode system

The structural Eurocodes were initiated by the European Commission but are now produced by the Comité Européen de Normalisation (CEN) which is the European standards organization, its members being the national standards bodies of the EU and EFTA countries, e.g. BSI.

CEN will eventually publish these design standards as full European Standards EN (Euronorms), but initially they are being issued as Prestandards ENV. Normally an ENV has a life of about 3 years to permit familiarization and trial use of the standard by member states. After formal voting by the member bodies, ENVs are converted into ENs taking into account the national comments on the ENV document. At present the following Eurocode parts have been published as ENVs but as yet none has been converted to an EN:

DD ENV 1991-1-1:	Basis of design and actions on structures (EC1)
DD ENV 1992-1-1:	Design of concrete structures (EC2)
DD ENV 1993-1-1:	Design of steel structures (EC3)
DD ENV 1994-1-1:	Design of composite steel and concrete structures (EC4)
DD ENV 1995-1-1:	Design of timber structures (EC5)
DD ENV 1996-1-1:	Design of masonry structures (EC6)
DD ENV 1997-1-1:	Geotechnical design (EC7)
DD ENV 1998-1-1:	Earthquake resistant design of structures (EC8)
DD ENV 1999-1-1:	Design of aluminium alloy structures (EC9)

Each Eurocode is published in a number of parts, usually with 'General rules' and 'Rules for buildings' in Part 1. The various parts of EC2 are:

Part 1.1	General rules and rules for buildings;
Part 1.2	Supplementary rules for structural fire design;
Part 1.3	Supplementary rules for precast concrete elements and structures;
Part 1.4	Supplementary rules for the use of lightweight aggregate concrete;
Part 1.5	Supplementary rules for the use of unbonded and external prestressing tendons;
Part 1.6	Supplementary rules for plain or lightly reinforced concrete structures;
Part 2.0	Reinforced and prestressed concrete bridges;
Part 3.0	Concrete foundations;
Part 4.0	Liquid retaining and containment structures.

All Eurocodes follow a common editorial style. The codes contain 'Principles' and 'Application rules'. Principles are general statements, definitions, requirements and sometimes analytical models. All designs must comply with the Principles, and no alternative is permitted.

Application rules are rules commonly adopted in design. They follow the Principles and satisfy their requirements. Alternative rules may be used provided that compliance with the Principles can be demonstrated.

Some parameters in Eurocodes are designated by | _ | , commonly referred to as boxed values. The boxed values in the Codes are indicative guidance values. Each member state is required to fix the boxed value applicable within its jurisdiction. Such information would be found in the National Application Document (NAD) which is published as part of each ENV.

There are also other purposes for NADs. NAD is meant to provide operational information to enable the ENV to be used. For certain aspects of the design, the ENV may refer to national standards or to CEN standard in preparation or ISO standards. The NAD is meant to provide appropriate guidance including modifications required to maintain compatibility between the documents. Very occasionally the NAD might rewrite particular clauses of the code in the interest of safety or economy. This is however rare.

1.3 Scope of the *Manual*

The range of structures and structural elements covered by the *Manual* is limited to building structures that do not rely on bending in columns for their resistance to horizontal forces and are also non-sway. This will be found to cover the vast majority of all reinforced and prestressed concrete building structures. In using the *Manual* the following should be noted:

- The *Manual* has been drafted to comply with ENV 1992-1-1 together with the UK NAD
- Although British Standards have been referenced as loading codes in Sections 3 and 6, to comply with the UK NAD, the *Manual* can be used in conjunction with other loading codes
- The structures are braced and non-sway
- The concrete is of normal weight
- The structure is predominantly *in situ*
- Prestressed concrete members have bonded or unbonded internal tendons
- The *Manual* can be used in conjunction with all commonly used materials in construction; however the data given are limited to the following:
  – concrete up to characteristic cylinder strength of 50N/mm^2 (cube strength 60N/mm^2)
  – high-tensile reinforcement with characteristic strength of 460N/mm^2
  – mild-steel reinforcement with characteristic strength of 250N/mm^2
  – prestressing tendons with 7-wire low-relaxation (Class 2) strands
- High ductility (Class H) has been assumed for:
  – all ribbed bars and grade 250 bars, and
  – ribbed wire welded fabric in wire sizes of 6mm or over
- Normal ductility (Class N) has been assumed for plain or indented wire welded fabric.

For structures or elements outside this scope EC2 should be used.

1.4 Contents of the *Manual*

The *Manual* covers the following design stages:

- general principles that govern the design of the layout of the structure
- initial sizing of members
- estimating of quantities of reinforcement and prestressing tendons
- final design of members.

2 General principles

This section outlines the general principles that apply to both initial and final design of both reinforced and prestressed concrete building structures, and states the design parameters that govern all design stages.

2.1 General

One engineer should be responsible for the overall design, including stability, and should ensure the compatibility of the design and details of parts and components even where some or all of the design and details of those parts and components are not made by the same engineer.

The structure should be so arranged that it can transmit dead, wind and imposed loads in a direct manner to the foundations. The general arrangement should ensure a robust and stable structure that will not collapse progressively under the effects of misuse or accidental damage to any one element.

The engineer should consider site constraints, buildability[2], maintainability and decommissioning.

The engineer should take account of his responsibilities as a 'Designer' under the *Construction (Design & Management) Regulations*.[3]

2.2 Stability

Lateral stability in two orthogonal directions should be provided by a system of strongpoints within the structure so as to produce a braced non-sway structure, in which the columns will not be subject to significant sway moments. Strongpoints can generally be provided by the core walls enclosing the stairs, lifts and service ducts. Additional stiffness can be provided by shear walls formed from a gable end or from some other external or internal subdividing wall. The core and shear walls should preferably be distributed throughout the structure and so arranged that their combined shear centre is located approximately on the line of the resultant in plan of the applied overturning forces. Where this is not possible, the resulting twisting moments must be considered when calculating the load carried by each strongpoint. These walls should generally be of reinforced concrete not less than 180mm thick to facilitate concreting, but they may be of 215mm brickwork or 190mm solid blockwork properly tied and pinned up to the framing for low- to medium-rise buildings.

Strongpoints should be effective throughout the full height of the building. If it is essential for strongpoints to be discontinuous at one level, provision must be made to transfer the forces to other vertical components.

It is essential that floors be designed to act as horizontal diaphragms, particularly if precast units are used.

Where a structure is divided by expansion joints each part should be structurally independent and designed to be stable and robust without relying on the stability of adjacent sections.

2.3 Robustness

All members of the structure should be effectively tied together in the longitudinal, transverse and vertical directions.

A well-designed and well-detailed cast-*in situ* structure will normally satisfy the detailed tying requirements set out in subsection 5.11.

Elements whose failure would cause collapse of more than a limited part of the structure adjacent to them should be avoided. Where this is not possible, alternative load paths should be identified or the element in question strengthened.

2.4 Movement joints

Movement joints may need to be provided to minimize the effects of movements caused by, for example, shrinkage, temperature variations, creep and settlement.

The effectiveness of movement joints depends on their location. Movement joints should divide the structure into a number of individual sections, and should pass through the whole structure above ground level in one plane. The structure should be framed on both sides of the joint. Some examples of positioning movement joints in plan are given in Fig. 2.1.

Movement joints may also be required where there is a significant change in the type of foundation or the height of the structure.

For reinforced concrete frame structures in UK conditions, movement joints at least 25mm wide should normally be provided at approximately 50m centres both longitudinally and transversely. In the top storey and for open buildings and exposed slabs additional joints should normally be provided to give approximately 25m spacing. Joint spacing in exposed parapets should be approximately 12m.

Joints should be incorporated in the finishes and in the cladding at the movement joint locations.

2.5 Fire resistance and durability

For the required period of fire resistance (prescribed in the Building Regulations), the structure should:

- have adequate loadbearing capacity
- limit the temperature rise on the far face by sufficient insulation, and
- have sufficient integrity to prevent the formation of cracks that will allow the passage of fire and gases.

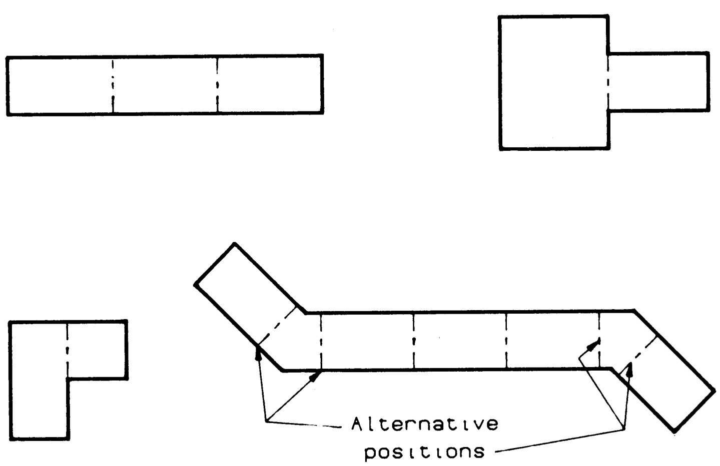


Fig. 2.1 Location of movement joints

The design should take into account the likely deterioration of the structure and its components in their environment having due regard to the anticipated level of maintenance. The following inter-related factors should be considered:

- the required performance criteria
- the expected environmental conditions
- the composition, properties and performance of materials
- the shape of members and detailing
- the quality of workmanship
- any protective measure
- the likely maintenance during the intended life.

Concrete of appropriate quality with adequate cover to the reinforcement should be specified.

The above requirements for durability and fire resistance may dictate sizes for members greater than those required for structural strength alone.

3 Design principles – reinforced concrete

3.1 Loading

The loads to be used in calculations are:

(a) Characteristic dead load, G_k: the weight of the structure complete with finishes, fixtures and fixed partitions (BS 648[4])
(b) Characteristic imposed load, Q_k (BS 6399, Parts 1 and 3[5])
(c) Characteristic wind load, W_k (90% of the load derived from CP 3, Chapter V, Part 2[6])*
(d) Nominal earth load, E_n (BS 8004[7])
(e) At the ultimate limit state the horizontal forces to be resisted at any level should be the greater of:

    (i) 1.5% of the characteristic dead load above that level†, or
    (ii) 90% of the wind load derived from CP 3, Chapter V, Part 2[6], multiplied by the appropriate partial safety factor.

    The horizontal forces should be distributed between the strongpoints according to their stiffness.

In using the above documents the following modifications should be noted:

(f) The imposed floor loads of a building should be treated as one load to which the reduction factors given in BS 6399: Part 1: 1996[5] are applicable.
(g) Snow drift loads obtained from BS 6399: Part 3: 1988[5] should be multiplied by 0.7 and treated in a similar way to an imposed load and not as an accidental load.

3.2 Limit states

This *Manual* adopts the limit-state principle and the partial factor format of EC2.

3.2.1 Ultimate limit state

The design loads are obtained by multiplying the characteristic loads by the appropriate partial factor γ_f from Table 3.1.

The 'adverse' and 'beneficial' factors should be used so as to produce the most onerous condition.

3.2.2 Serviceability limit states

Provided that span/effective depth ratios and bar diameter and spacing rules are observed it will not be necessary to check for serviceability limit states.

*This complies with the UK NAD, although the Code has been revised and issued as BS 6399: Part 2[5].

†This will generally be more conservative than EC2 requirements.

Table 3.1 Partial factors for loads γ_f at the ultimate limit state

Load combination including earth & water where present	Dead load G_k		Imposed, wind snow load Q_k and W_k		Earth and water E_n
	Adverse	Beneficial	Adverse	Beneficial	
1. Dead + imposed	1.35	—	1.50	—	1.35†
2. Dead + wind	1.35	1.0	1.50	—	1.35†
3. Dead + snow	1.35	1.0	1.50	—	1.35†
4. Dead + imposed + wind + snow	1.35	—	1.35	—	1.35†

The Table uses the simplified combination permitted in EC2.

†For pressures arising from an accidental head of water at ground level a partial factor of 1.15 may be used.

3.3 Material and design stresses

Design stresses are given in the appropriate sections of the *Manual*. It should be noted that EC2 specifies concrete strength class by both the cylinder strength and cube strength (for example C25/30 is a concrete with cylinder strength of 25 and cube strength of 30N/mm^2 at 28 days). Standard strength classes are C20/25, C25/30, C30/37, C35/45, C40/50, C45/55 and C50/60. All design equations which include concrete compressive strength use the characteristic 28 day cylinder strength, f_{ck}.

Partial factors for concrete are 1.5 for ultimate limit state and 1.0 for serviceability limit state.

The strength properties of reinforcement are expressed in terms of the characteristic yield strength, f_{yk}.

Partial factors for reinforcement steel are 1.15 for ultimate limit state and 1.0 for serviceability limit state.

4 Initial design – reinforced concrete

4.1 Introduction

In the initial stages of the design of building structures it is necessary, often at short notice, to produce alternative schemes that can be assessed for architectural and functional suitability and which can be compared for cost. They will usually be based on vague and limited information on matters affecting the structure such as imposed loads and nature of finishes, let alone firm dimensions, but it is nevertheless expected that viable schemes be produced on which reliable cost estimates can be based.

It follows that initial design methods should be simple, quick, conservative and reliable. Lengthy analytical methods should be avoided.

This section offers some advice on the general principles to be applied when preparing a scheme for a structure, followed by methods for sizing members of superstructures. Foundation design is best deferred to later stages when site investigation results can be evaluated.

The aim should be to establish a structural scheme that is suitable for its purpose, sensibly economical, and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Sizing of structural members should be based on the longest spans (slabs and beams) and largest areas of roof and/or floors carried (beams, columns, walls and foundations). The same sizes should be assumed for similar but less onerous cases – this saves design and costing time at this stage and is of actual benefit in producing visual and constructional repetition and hence, ultimately, cost benefits.

Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good 'benchmark' at the initial stage.

Loads should be carried to the foundation by the shortest and most direct routes. In constructional terms, simplicity implies (among other matters) repetition; avoidance of congested, awkward or structurally sensitive details and straightforward temporary works with minimal requirements for unorthodox sequencing to achieve the intended behaviour of the completed structure.

Standardized construction items will usually be cheaper and more readily available than purpose-made items.

4.2 Loads

Loads should be based on BS 648[4], BS 6399: Parts 1 and 3[5] and CP 3: Chapter V: Part 2[6].

Imposed loading should initially be taken as the highest statutory figures where options exist. The imposed load reduction allowed in the loading code should not be taken advantage of in the initial design stage except when assessing the load on the foundations.

Loading should be generous and not less than the following in the initial stages:

floor finish (screed)	1.8kN/m^2
ceiling and service load	0.5kN/m^2

Allowance for:

demountable lightweight partitions*	1.0kN/m^2
blockwork partitions†	2.5kN/m^2

Weight of reinforced concrete should be taken as 24kN/m^3.

Design loads should be obtained using Table 3.1.

4.3 Material properties

For normal construction in the UK, a characteristic cylinder concrete strength f_{ck} of 25N/mm^2 should be assumed for the initial design. In areas with poor aggregates this may have to be reduced.

For UK steels a characteristic strength f_{yk} of 460N/mm^2 should be used for high-tensile reinforcement and 250N/mm^2 for mild steel.

4.4 Structural form and framing

The following measures should be adopted:

(a) provide stability against lateral forces and ensure braced construction by arranging suitable shear walls deployed symmetrically wherever possible
(b) adopt a simple arrangement of slabs, beams and columns so that loads are carried to the foundations by the shortest and most direct routes
(c) allow for movement joints (see subsection 2.4)
(d) choose an arrangement that will limit the span of slabs to 5m to 6m and beam spans to 8m to l0m on a regular grid; for flat slabs restrict column spacings to 8m
(e) adopt a minimum column size of 300mm × 300mm or equivalent area
(f) provide a robust structure.

The arrangement should take account of possible large openings for services and problems with foundations, e.g. columns immediately adjacent to site boundaries may require balanced or other special foundations.

4.5 Fire resistance and durability

The size of structural members may be governed by the requirement of fire resistance and may also be affected by the cover necessary to ensure durability. Table 4.1 shows the minimum practical member sizes for different periods of fire resistance and the cover to the main reinforcement required for continuous members in dry and humid environments without frost. For other exposure classes, cover should be increased. For simply supported members, sizes and cover should be increased (see Section 5 and Appendix C).

4.6 Stiffness

To provide adequate stiffness, the effective depths of beams, slabs and the waist of stairs should not be less than those derived from Table 4.2.

Beams should be of sufficient depth to avoid the necessity for excessive compression reinforcement and to ensure that economical amounts of tension and shear reinforcement are provided. This will also facilitate the placing of concrete.

*To be treated as imposed loads.
†To be treated as dead loads when the layout is fixed.

Table 4.1 Minimum member sizes and cover† for initial design of continuous members

Member	Fire resistance	Minimum dimension, mm		
		4 hours	2 hours	1 hour
Columns fully exposed to fire	width	450	300	200
Beams	width cover	240 70	200 50	200 45
Slabs with plain soffit	thickness cover	170 45	125 35	100 35
Slabs with ribbed open soffit and no stirrups	thickness* width of ribs cover	150 150 55	115 110 35	90 90 35

*Thickness of structural topping plus any non-combustible screed.
†Cover is to main reinforcement.

Table 4.2 Basic ratios of span/effective depth for initial design (f_{yk} = 460N/mm^2)

Structural system	Span/effective depth ratio	
	Beam	Slab
1. Simply supported beam	15	
One-way or two-way spanning simply supported slab		21
2. End span of: continuous beam;	20	
one-way continuous slab; or two-way spanning slab continuous over one long side		27
3. Interior span of: beam;	21	
one-way or two-say spanning slab		30
4. Slab supported on columns without beams (flat slab), based on longer span		26
5. Cantilever	6	8

Notes to Table 4.2

1. For two-way spanning slabs (supported on beams), the check on the ratio of span/effective depth should be carried out on the shorter span. For flat slabs, the longer span should be taken.
2. For flanged sections with the ratio of the flange to the rib width greater than 3, the Table value should be multiplied by 0.8.
3. For members, other than flat slab panels, which support partitions liable to be damaged by excessive deflection of the member, and where the span exceeds 7m, the Table value should be multiplied by 7/span.
4. For flat slabs where the greater span exceeds 8.5m, the Table value should be multiplied by 8.5/span.

4.7 Sizing

4.7.1 Introduction

When the depths of slabs and beams have been obtained it is necessary to check the following:

- width of beams and ribs
- column sizes and reinforcement
- shear in flat slabs at columns
- practicality of reinforcement arrangements in beams, slabs and at beam-column junctions.

4.7.2 Loading

Ultimate loads, i.e. characteristic loads multiplied by the appropriate partial factors, should be used throughout. At this stage it may be assumed that all spans are fully loaded, unless the members (e.g. overhanging cantilevers) concerned are sensitive to unbalanced loading (see subclause 4.7.7.1).

For purposes of assessing the self-weight of beams, the width of the downstand can be taken as half the depth but usually not less than 300mm.

4.7.3 Width of beams and ribs

The width should be determined by limiting the shear stress in beams to 2.0N/mm^2 and in ribs to 0.6N/mm^2 for concrete of characteristic strength $f_{ck} \geq 25$N/mm^2:

$$\text{width of beam (in mm)} = \frac{1000V}{2d}$$

$$\text{width of rib (in mm)} = \frac{1000V}{0.6d}$$

where V is the maximum shear force (in kN) on the beam or rib, considered as simply supported and d is the effective depth in mm.

4.7.4 Sizes and reinforcement of columns

Where possible it will generally be best to use stocky columns (i.e. columns for which the ratio of the effective height to the least lateral dimension does not exceed 15) as this will avoid the necessity of designing for the effects of slenderness. Slenderness effects can normally be neglected in non-sway structures where the ratio of the effective height to the least lateral dimension of the column is less than 15. For the purpose of initial design, the effective height of a non-sway column may be taken as 0.85 times the clear storey height.

The columns should be designed as axially loaded, but to compensate for the effect of eccentricities, the ultimate load from the floor immediately above the column being considered should be multiplied by the factors listed below and overleaf:

It is recommended that the columns are made the same size through at least the two topmost storeys, as the above factors may lead to inadequate sizes if applied to top storey columns for which the moments tend to be large in relation to the axial loads.

- For columns loaded by beams and/or slabs of similar stiffness on both sides of the column in two directions at right-angles to each other, e.g. some internal columns 1.25

• For columns loaded in two directions at right-angles to each other by unbalanced beams and/or slabs, e .g. corner columns 2.00
• In all other cases, e.g. facade columns 1.50

For the initial design of columns, the required cross-sectional area may be calculated by dividing the ultimate load by the selected equivalent 'stress' given in Table 4.3. Alternatively, for a known column size the ultimate load capacity may be found by using the selected equivalent stress.

When choosing the column dimensions, care should be taken to see that the column remains stocky, as defined above.

The equivalent 'stresses' given in Table 4.3 are derived from the expression:

$$\text{stress} = 0.44f_{ck} + \frac{\rho}{100}(0.67f_{yk} - 0.44f_{ck})$$

where f_{ck} is the characteristic concrete strength in N/mm^2
f_{yk} the characteristic strength of reinforcement in N/mm^2, and
ρ the percentage of reinforcement.

Where slender columns (i.e. the ratio of the effective height l_e, to the least lateral dimension, b exceeds 15) are used, the ultimate load capacity of the column or equivalent 'stress' should be reduced by the appropriate factor from Table 4.4.

4.7.5 Walls

Walls carrying vertical loads should be designed as columns. Shear walls should be designed as vertical cantilevers, and the reinforcement arrangement should be checked as for a beam. Where the shear walls have returns at the compression end, they should be treated as flanged beams.

Table 4.3 Equivalent 'stress' values

Reinforcement (460N/mm^2) percentage ρ	Equivalent stresses (N/mm^2) for concrete strength classes		
	C25/30	C30/37	C35/45
ρ = 1%	14	16	18
ρ = 2%	17	19	21
ρ = 3%	20	22	24
ρ = 4%	23	25	27

Table 4.4 Reduction factors for slender columns

l_e/b	Capacity reduction factor
15	1.00
20	0.80
25	0.67
30	0.55
35	0.45

4.7.6 Punching shear in flat slabs at columns

Check that:

(a) where shear reinforcement is to be avoided:

$$\frac{1250w\,(A_{\text{supp}})}{(u_c + 9h)d} \leq 0.5\text{N/mm}^2$$

(b) where shear reinforcement may be provided:

$$\frac{1250w\,(A_{\text{supp}})}{(u_c + 9h)d} \leq 0.9\text{N/mm}^2$$

(c) Check also that:

$$\frac{1250w\,(A_{\text{supp}})}{(u_c)d} \leq 0.9\sqrt{f_{ck}}$$

In the above verification

w is the total design ultimate load per unit area in kN/m^2
d is the effective depth of the slab at the column in mm
h is the thickness of the slab at the column in mm
A_{supp} is the area supported by the column in m^2
u_c is column perimeter in mm.

4.7.7 Adequacy of chosen sections to accommodate the reinforcement

4.7.7.1 Bending moment and shear forces

In the initial stage the reinforcement needs to be checked only at midspan and at the supports of critical spans.

Beams and one-way solid slabs

Bending moments and shear forces in continuous structures can be obtained from Table 4.5 when:

(a) the imposed load does not exceed the dead load
(b) there are at least three spans, and
(c) the spans do not differ in length by more than 15% of the longest span.

Alternatively, bending moments and shear forces may be obtained by elastic analysis.

Table 4.5 Ultimate bending moments and shear forces

	Uniformly distributed loads F = total design ultimate load on span	Central point loads W = design ultimate point load
Bending moments		
at support	–0.100 FL	–0.150 WL
at midspan	0.080 FL	0.175 WL
Shear forces	0.65 F	0.65 W

where L is the span.

Two-way solid slabs on linear supports
If the longer span l_y does not exceed 1.5 times the shorter span l_x, the average moment per metre width may be taken as:

$$w \; \frac{l_x l_y}{18} \text{ kNm per metre}$$

where w is the design ultimate load in kN/m^2, and l_x and l_y are in metres.

If $l_y > 1.5 l_x$ the slab should be treated as acting one-way.

Solid flat slabs
Determine the moments per unit width in the column strips in each direction as 1.5 times those for one-way slabs.

One-way ribbed slabs
Assess the bending moments at midspan on a width equal to the rib spacing, assuming simple supports throughout.

Two-way ribbed slabs on linear supports
If the longer span does not exceed 1.5 times the shorter span, estimate the average rib moment in both directions as:

$$w \frac{l_x l_y}{18} \; c \text{ kNm per rib}$$

where c is the rib spacing in metres.

If $l_y > 1.5 l_x$ the slab should be treated as acting one-way.

Coffered slabs on column supports
Assess the average bending moment at midspan on a width equal to the rib spacing using Table 4.5. For the column strips increase this by 15%.

4.7.7.2 Provision of reinforcement
Using the bending moments above reinforcement may be calculated as follows:

Tension reinforcement
Reinforcement can now be calculated by the following formula:

$$A_s = \frac{M}{0.87\, f_{yk}\, 0.8d}$$

where M is the design ultimate bending moment at the critical section and d is the effective depth.

Compression reinforcement
If, for a rectangular section, $M > 0.167\, f_{ck} bd^2$, compression reinforcement is required:

$$A'_s = \frac{M - 0.167\, f_{ck} bd^2}{0.87\, f_{yk}(d - d')}$$

where A'_s is the area of the compression steel, d' is the depth to its centroid, b is the width of the section and d is its effective depth.

If, for flanged sections, $M > 0.567 f_{ck} b_f h_f (d - 0.5 h_f)$ the section should be redesigned. b_f and h_f are the width and the thickness of the flange. h_f should not be taken as more than $0.36d$.

Bar arrangements
When the areas of the main reinforcement in the members have been calculated, check that the bars can be arranged with the required cover in a practicable manner avoiding congested areas.

In beams, this area should generally be provided by not less than 2 or more than 8 bars. In slabs, the bar spacing should not be less than 150mm or more than 300mm; the bars should not be less than 10mm or normally more than 20mm in diameter.

4.8 The next step

At this stage general arrangement drawings, including sections through the entire structure, should be prepared and sent to other members of the design team for comment, together with a brief statement of the principal design assumptions, e.g. imposed loadings, weights of finishes, fire ratings and durability.

The scheme may have to be amended following receipt of comments. The amended design should form the basis for the architect’s drawings and may also be used for preparing reinforcement estimates for budget costings.

4.9 Reinforcement estimates

In order for the cost of the structure to be estimated it is necessary for the quantities of the materials, including those of the reinforcement, to be available. Fairly accurate quantities of the concrete and brickwork can be calculated from the layout drawings. If working drawings and schedules for the reinforcement are not available it is necessary to provide an estimate of the anticipated quantities. The quantities are normally described in accordance with the requirements of the *Standard method of measurement of building works* (SMM)[8]. In the case of reinforcement quantities the basic requirements are, briefly:

1. For bar reinforcement to be described separately by: steel type (e.g. mild or high-yield steel), diameter and weight and divided up according to:
   (a) element of structure, e.g. foundations, slabs, walls, columns, etc., and
   (b) bar ‘shape’, e.g. straight, bent or hooked; curved; links, stirrups and spacers.
2. For fabric (mesh) reinforcement to be described separately by: steel type, fabric type and area, divided up according to 1(a) and 1(b) above.

There are different methods for estimating the quantities of reinforcement; three methods of varying accuracy are given below.

Method l

The simplest method is based on the type of structure and the volume of the reinforced concrete elements. Typical values are, for example:

- warehouses and similarly loaded and proportioned structures: 1 tonne of reinforcement per $10m^3$
- offices, shops, hotels: 1 tonne per $13.5m^3$
- residential, schools: 1 tonne per $l5.0m^3$

However, while this method is a useful check on the total estimated quantity it is the least accurate, and it requires considerable experience to break the tonnage down to SMM[8] requirements.

Method 2

Another method is to use factors that convert the steel areas obtained from the initial design calculations to weights, e.g. kg/m^2 or kg/m as appropriate to the element.

Tables A1 to A5 in Appendix A give factors for the various elements of the structure that should be used for this purpose.

If the weights are divided into practical bar diameters and shapes this method can give a reasonably accurate assessment. The factors, however, do assume a degree of standardization both of structural form and detailing.

This method is likely to be the most flexible and relatively precise in practice, as it is based on reinforcement requirements indicated by the initial design calculations.

Method 3

For this method sketches are made for the 'typical' cases of elements and then weighted. This method has the advantages that:

(a) the sketches are representative of the actual structure
(b) the sketches include the intended form of detailing and distribution of main and secondary reinforcement
(c) an allowance of additional steel for variations and holes may be made by inspection.

This method can also be used to calibrate or check the factors described in method 2 as it takes account of individual detailing methods.

When preparing the reinforcement estimate, the following items should be considered:

(a) *Laps and starter bars*
A reasonable allowance for normal laps in both main and distribution bars, and for starter bars has been made in Tables A1 to A5. It should however be checked if special lapping arrangements are used.
(b) *Architectural features*
The drawings should be looked at and sufficient allowance made for the reinforcement required for such 'non-structural' features.
(c) *Contingency*
A contingency of between 10% and 15% should be added to cater for some changes and for possible omissions.

5 Final design – reinforced concrete

5.1 Introduction

Section 4 describes how the initial design of a reinforced concrete structure can be developed to the stage where preliminary plans and reinforcement estimates may be prepared. The approximate cost of the structure can now be estimated.

Before starting the final design it is necessary to obtain approval of the preliminary drawings from the other members of the design team. The drawings may require further amendment, and it may be necessary to repeat this process until approval is given by all parties. When all the comments have been received it is then important to marshal all the information received into a logical format ready for use in the final design. This may be carried out in the following sequence:

(1) checking of all information
(2) preparation of a list of design data
(3) amendment of drawings as a basis for final calculations.

5.1.1 Checking of all information

To ensure that the initial design assumptions are still valid, the comments and any other information received from the client and the members of the design team, and the results of the ground investigation, should be checked.

Stability
Ensure that no amendments have been made to the sizes and to the disposition of the core and shear walls. Check that any openings in these can be accommodated in the final design.

Movement joints
Ensure that no amendments have been made to the disposition of the movement joints.

Loading
Check that the loading assumptions are still correct. This applies to dead and imposed loading such as floor finishes, ceilings, services, partitions and external wall thicknesses, materials and finishes thereto.

Make a final check on the design wind loading and consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account.

Fire resistance, durability and sound insulation
Establish with other members of the design team the fire resistance required for each part of the structure, the durability classifications that apply to each part and the mass of floors and walls (including finishes) required for sound insulation.

Foundations
Examine the information from the ground investigation and decide on the type of foundation to be used in the final design. Consider especially any existing or future structure adjacent to the perimeter of the structure that may influence not only the location of the foundations but also any possible effect on the superstructure and on adjacent buildings.

Performance criteria
Establish which codes of practice and other design criteria are to be used in the final design.

Materials
Decide on the concrete mixes and grade of reinforcement to be used in the final design for each or all parts of the structure, taking into account the fire-resistance and durability requirements, the availability of the constituents of concrete mixes and any other specific requirements such as water-excluding concrete construction for basements.

5.1.2 Preparation of a list of design data

The information obtained from the above check and that resulting from any discussions with the client, design team members, building control authorities and material suppliers should be entered into a design information data list. A suitable format for such a list is included in Appendix B. This list should be sent to the design team leader for approval before the final design is commenced.

5.1.3 Amendment of drawings as a basis for final calculations

The preliminary drawings should be brought up to date incorporating any amendments arising out of the final check of the information previously accumulated and finally approved.

In addition the following details should be added to all the preliminary drawings as an aid to the final calculations.

Gridlines
Establish gridlines in two directions, mutually at right-angles for orthogonal building layouts: identify these on the plans.

Members
Give all walls, columns, beams and slabs unique reference numbers or a combination of letters and numbers related if possible to the grid, so that they can be readily identified on the drawings and in the calculations.

Loading
Prepare drawings showing the loads that are to be carried by each element, clearly indicating whether the loads are factored or unfactored. It is also desirable to mark on the plans the width and location of any walls or other special loads to be carried by the slabs or beams.

5.1.4 Final design calculations

When all the above checks, design information, data lists and preparation of the preliminary drawings have been carried out the final design calculations for the structure can be commenced. It is important that these should be carried out in a logical sequence. The remainder of this section has been laid out in the following order:

- slabs
- structural frames
- beams
- columns
- walls
- staircases
- retaining walls, basements

• foundations
• robustness, and
• detailing.

5.2 Slabs

5.2.1 Introduction

The first step in preparing the final design is to complete the design of the slabs. This is necessary in order that the final loading is determined for the design of the frame.

The initial design should be checked, using the methods described in this subsection, to obtain the final sizes of the slabs and to calculate the amount and dimensions of the reinforcement.

This subsection gives fire resistance and durability requirements, and bending and shear force coefficients for one-way spanning slabs, two-way spanning slabs on linear supports, and flat slabs using solid, ribbed and coffered construction. The coefficients apply to slabs complying with certain limitations which are stated for each type.

For those cases where no coefficients are provided the bending moments and shear forces for one-way spanning slabs may be obtained by elastic analysis. These moments may then be redistributed, maintaining equilibrium with applied loads, up to a maximum of 30%, although normally 15% is considered a reasonable limit.

The treatment of shear around columns for flat slabs and the check for deflection for all types of slab are given, together with some notes on the use of precast slabs.

The general procedure to be adopted is as follows:

(1) Check that the cross-section and cover comply with requirements for fire resistance.
(2) Check that cover and concrete grade comply with requirements for durability.
(3) Calculate bending moments and shear forces.
(4) Calculate reinforcement.
(5) Make final check on span/depth ratios.
(6) For flat slabs check shear around columns and calculate shear reinforcement as necessary.

The effective span of a simply supported slab should normally be taken as the clear distance between the faces of supports plus one-third of their widths. However, where a bearing pad is provided between the slab and the support, the effective span should be taken as the distance between the centres of the bearing pads.

The effective span of a slab continuous over its supports should normally be taken as the distance between the centres of the supports.

The effective length of a cantilever slab where this forms the end of a continuous slab is the length of the cantilever from the centre of the support. Where the slab is an isolated cantilever the effective length is the length of the cantilever from the face of the support.

5.2.2 Fire resistance and durability

5.2.2.1 Fire resistance

The member size and reinforcement cover required to provide fire resistance are given in Table 5.1. The cover in the Table may need to be increased for durability (see subclause 5.2.2.2).

Where the cover to the outermost reinforcement exceeds 40mm special precautions against spalling may be required, e.g. partial replacement by plaster, lightweight aggregate or the use of fabric as supplementary reinforcement (see BS 8110, Part 2[16]).

5.2.2.2 Durability

The requirements for durability in any given environment are:

(a) an upper limit to the water/cement ratio
(b) a lower limit to the cement content
(c) a lower limit to the cover to the reinforcement
(d) good compaction
(e) adequate curing, and
(f) good detailing

Values for (a), (b) and (c) which, in combination, will be adequate to ensure durability are given in Table 5.2 for various environments.

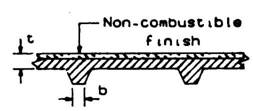
As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 5.2 gives, in addition, the concrete class strengths that have to be specified in the UK so that requirements (a) and (b) are satisfied.

For frost resistance the use of air entrainment of the concrete should be considered; however the effects of air entrainment on the concrete properties should be taken into account.

If the width of the rib is more than the minimum in Table 5.1 the cover may be decreased as over:

Increase in width, mm	Decrease in cover, mm
25	5
50	10
100	15
150	15

Table 5.1 Fire resistance requirements for slabs



	Plain soffit solid slab (including hollow pot, joist + block) **Minimum overall depth, mm**		Ribbed soffit (including T-section + channel section) **Minimum thickness/width, mm/mm**	
Fire resistance hours	Simply supported	Continuous	Simply supported	Continuous
			t/*b*	*t*/*b*
1	95	95	90/90	90/80
1.5	110	110	105/110	105/90
2	125	125	115/125	115/110
3	150	150	135/150	135/125
4	170	170	150/175	150/150
	Cover to main reinforcement, mm			
1	20	20	25	20
1.5	25	20	35	25
2	35	25	45	35
3	45	35	55	45
4	55	45	65	55

Table 5.2 Durability requirements for slabs

Conditions of exposure (for definitions see Appendix C)	Nominal cover to all reinforcement				
	mm	mm	mm	mm	mm
1. Dry environment	20	20	20	20	20
2. Humid environment					
(a) without frost	—	30	30	25	25
(b) with frost	—	—	30	25	25
3. Humid with frost and de-icing salts	—	—	35	30	30
4. Seawater environment					
(a) without frost	—	—	35	30	30
(b) with frost	—	—	35	30	30
The following classes may occur alone or in combination with those above:					
5. Aggressive chemical environment					
(a) slightly aggressive	—	—	30	25	25
(b) moderately aggressive	—	—	—	25	25
(c) highly aggressive*	—	—	—	—	40
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content, kg/m^3	260	280	300†	300	300
Concrete class strength	C25/30	C30/37	C35/45	C40/50	C45/55 or better

Notes to Table 5.2

1. Cover is expressed in terms of nominal values which have been obtained from the minimum values by allowing for a negative tolerance of 5mm.
2. The cover in mm to the main reinforcement should not be less than the bar diameter.

*Protective barrier to prevent direct contact with highly aggressive media should be provided.

†280 kg/m^3 for exposure classes 2b and 5a.

The concrete class strengths quoted in Table 5.2 will often require cement contents that are higher than those in the Table. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

5.2.3 Bending moments and shear forces

5.2.3.1 General

Slabs should be designed to withstand the most unfavourable arrangements of design loads. For continuous slabs subjected to predominantly uniformly distributed loads it will be sufficient to consider only the following arrangements of loads for ultimate state verification:

(a) Alternate spans carrying the maximum design dead and imposed load (i.e. $1.35G_k + 1.5Q_k$), other spans carrying the maximum design dead load (i.e. $1.35G_k$).

(b) Any two adjacent spans carrying the maximum design dead and imposed load (i.e. $1.35G_k + 1.5Q_k$), other spans carrying the maximum design dead load (i.e. $1.35G_k$).

The moments obtained from elastic analysis may be redistributed up to a maximum of 30% except for plain or indented fabric for which the limit is 15%. It should be noted that:

(c) the resulting distribution of moments remains in equilibrium with the applied load
(d) the design redistributed moment at any section should not be less than 70% of the elastic moment, and
(e) there are limitations in the depth of the neutral axis of the section depending on the percentage of redistribution (see subclause 5.2.5.1).

Concentrated loads
The bending moment arising from a concentrated load may be distributed over a width of slab equal to the width of the load plus the lesser of the actual width or $1.2(1 - (x/l))x$ on each side of the load, where x = the distance to the nearer support from the section under consideration, and l = the span.

5.2.3.2 One-way spanning slabs
For continuous slabs with (a) substantially uniform loading (b) dead load greater than or equal to imposed load and (c) at least three spans that do not differ by more than 15% of the longest span, the bending moments and shear forces may be calculated using the coefficients given in Table 5.3, where F is the total design ultimate load ($1.35G_k + 1.5Q_k$) for each span and l is the span. No redistribution should be carried out on the bending moments obtained from Table 5.3.

5.2.3.3 Two-way spanning slabs on linear supports
Bending moments in two-way slabs may be calculated by yield-line analysis. Alternatively, the coefficients in Table 5.4 may be used to obtain bending moments per unit width (m_{sx} and m_{sy}) in the two directions for various edge conditions, i.e.:

$$m_{sx} = ß_{sx} w l_x^2$$
$$m_{sy} = ß_{sy} w l_x^2$$

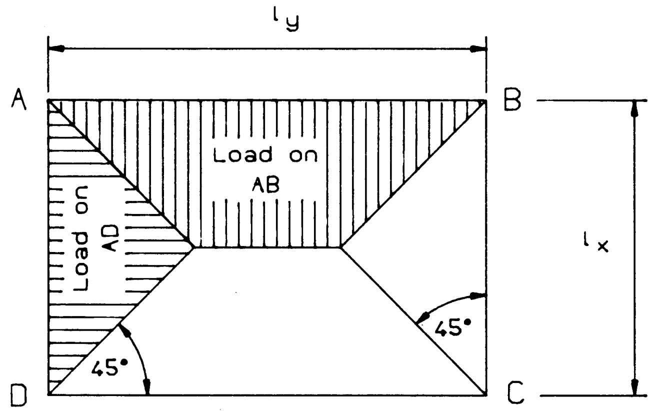
where $ß_{sx}$ and $ß_{sy}$ are the coefficients given in Table 5.4 and l_x is the shorter span.

The distribution of the reactions of two-way slabs on to their supports can be derived from Fig. 5.1.

5.2.3.4 Flat slabs
If a flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exceed 1.2, the maximum values of the bending moments and shear forces in each direction may be obtained from Table 5.5.

Where the conditions above do not apply, bending moments in flat slabs should be obtained by frame analysis (see subsection 5.3). The structure should then be considered as being divided longitudinally and transversely into frames consisting of columns and strips of slab. The width of slab contributing to the effective stiffness should be the full width of the panel. The stiffening effects of drops and column heads may be ignored for the analysis but need to be taken into account when considering the distribution of reinforcement.

Division of panels (except in the region of edge and corner columns)
Flat slab panels should be assumed to be divided into column strips and middle strips (see Fig. 5.2). In the assessment of the widths of the column and middle strips, drops should be ignored if their smaller dimension is less than one-third of the smaller dimension of the panel.



Notes

1. The reactions shown apply when all edges are continuous (or discontinuous)
2. When one edge is discontinuous, the reactions on all continuous edges should be increased by 10% and the reaction on the discontinuous edge may be reduced by 20%.
3. When adjacent edges are discontinuous, the reactions should be adjusted for elastic shear considering each span separately.

Fig. 5.1 Distribution of reactions from two-way slabs onto supports

Table 5.3 Bending moments and shear forces for one-way slabs

	End support	End span	Penultimate support	Interior spans	Interior supports
Moment	0	$0.09Fl$	$-0.11Fl$	$0.07Fl$	$-0.1Fl$
Shear	$0.45F$	—	$0.6F$	—	$0.55F$

Division of moments between column and middle strips

The design moments obtained from analysis of the frames or from Table 5.5 should be divided between the column and middle strips in the proportions given overleaf:

column strip	middle strip	
negative	75%	25%
positive	55%	45%

In general, moments will be able to be transferred only between a slab and an edge or corner column by a column strip considerably narrower than that appropriate for an internal panel. The breadth of this strip, b_e, for various typical cases is shown in Fig. 5.3. b_e should not be taken as greater than the column strip width appropriate for an interior panel.

The maximum design moment that can be transferred to a column by this strip is given by:

$$M_{max} = 0.167 f_{ck} b_e d^2$$

where d is the effective depth for the top reinforcement in the column strip, and $f_{ck} \leq 35\text{N/mm}^2$. The moments obtained from Table 5.5 or a frame analysis should be adjusted at the columns to the above values and the midspan moments increased accordingly.

Table 5.4 Bending moment coefficients for two-way spanning rectangular slabs

Type of panel and moments considered	Short span coefficients β_{sx} Values of l_y/l_x 1.0	1.25	1.5	1.75	2.0	Long-span coefficients β_{sy} for all values of l_y/l_x
1. 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
2. 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
3. One long edge discontinuous						
Negative moment at continuous edge	0.039	0.059	0.073	0.082	0.089	0.037
Positive moment at midspan	0.030	0.045	0.055	0.062	0.067	0.028
4. 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

Table 5.5 Bending moment and shear force coefficients for flat slab panels of three or more equal spans

	Outer support column	Outer support wall	Near middle of end span	At first interior support	At middle of interior span(s)	At internal supports
Moment	$-0.04Fl$*	$-0.2Fl$	$0.09Fl$†	$-0.11Fl$	$0.07Fl$	$-0.1Fl$
Shear	$0.45F$	$0.4F$	–	$0.6F$	–	$0.55F$
Total column moments	$0.04Fl$	–	–	$0.22Fl$	–	$0.22Fl$

F is the total design ultimate load ($1.35G_k + 1.5Q_k$)
*These moments may have to be reduced to be consistent with the capacity to transfer moments to the columns; the midspan moments † must then be increased correspondingly.

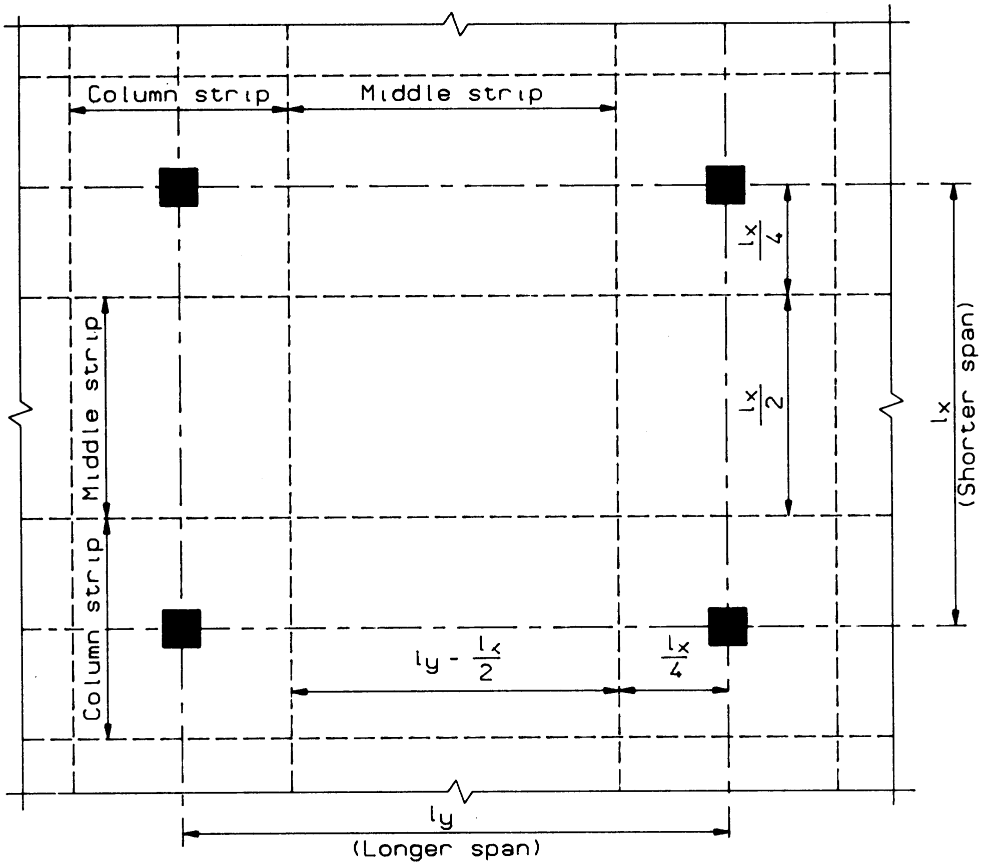


Fig. 5.2 Division of panel without drops into strips

Where the slab is supported by a wall, or an edge beam with a depth greater than 1.5 times the thickness of the slab, the design moments of the half column strip adjacent to the beam or wall should be one-quarter of the design moments obtained from the analysis.

Effective shear forces in flat slabs

The critical consideration for shear in flat slab structures is that of punching shear around the columns. This should be checked in accordance with subclause 5.2.4.2 except that the shear forces should be increased to allow for the effects of moment transfer as indicated below.

After calculation of the design moment transmitted by the connection, the design effective shear force V_{eff} at the perimeter of the column should be taken as:

V_{eff} = 1.15 V_t for internal columns with approximately equal spans
= 1.4 V_t for edge columns
= 1.5 V_t for corner columns

where V_t, is the design shear transferred to the column and is calculated on the assumption that the maximum design load is applied to all panels adjacent to the column considered.

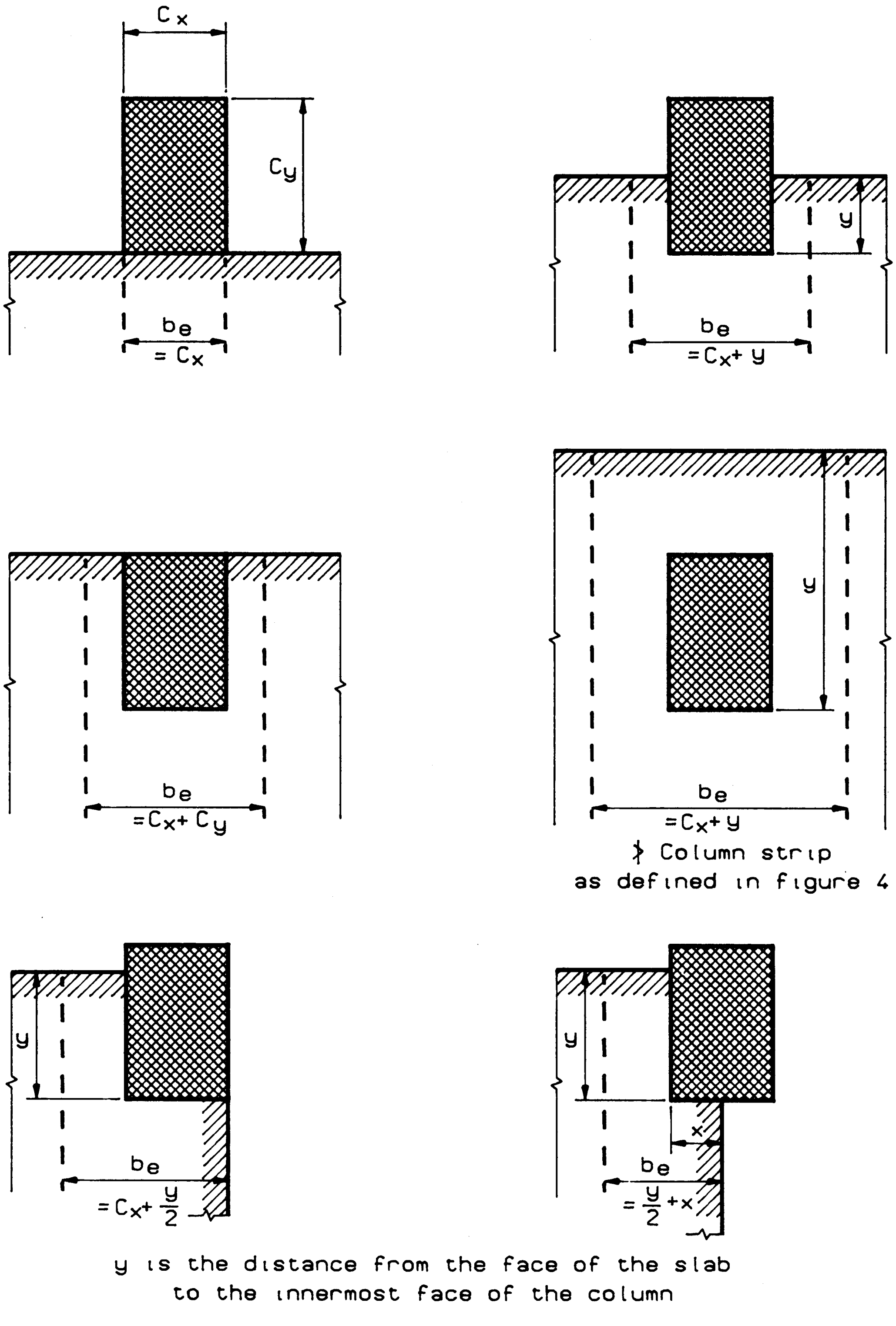


Fig. 5.3 Definition of breadth of effective moment transfer strip, b_e

5.2.4 Section design – solid slabs

5.2.4.1 Bending

(a) Reinforcement

To avoid compression reinforcement in slabs check that the applied moment is less than the limiting moment of resistance $M_u = K_{lim} f_{ck} bd^2$, which is based on Fig. 5.4. The values of K_{lim} should be obtained from Fig. 5.5 for the amount of redistribution carried out.

The area of tension reinforcement is then given by:

$$A_s = \frac{M}{(0.87 f_{yk})z}$$

where z is obtained from Fig. 5.5 for different values of $K = M/bd^2 f_{ck}$.

For two-way spanning slabs, care should be taken to use the value of d appropriate to the direction of the reinforcement.

(b) Detailing

Two-way slabs on linear supports
The reinforcement calculated from the bending moments obtained from subclause 5.2.3.3 should be provided for the full width in both directions.

In the corner area shown in Fig. 5.6:

(a) provide top and bottom reinforcement
(b) in each layer provide bars parallel to the slab edges
(c) in each of the four layers the area of reinforcement per unit length should be equal to 75% of the reinforcement required for the maximum span moment per unit length
(d) the area of reinforcement in (c) can be halved if one edge of the slab in the corner is continuous.

Flat slabs
Column and middle strips should be reinforced to withstand the design moments obtained from subclause 5.2.3.4. In general two-thirds of the amount of reinforcement required to resist the negative design moment in the column strip should be placed in a width equal to half that of the column strip symmetrically positioned about the centreline of the column.

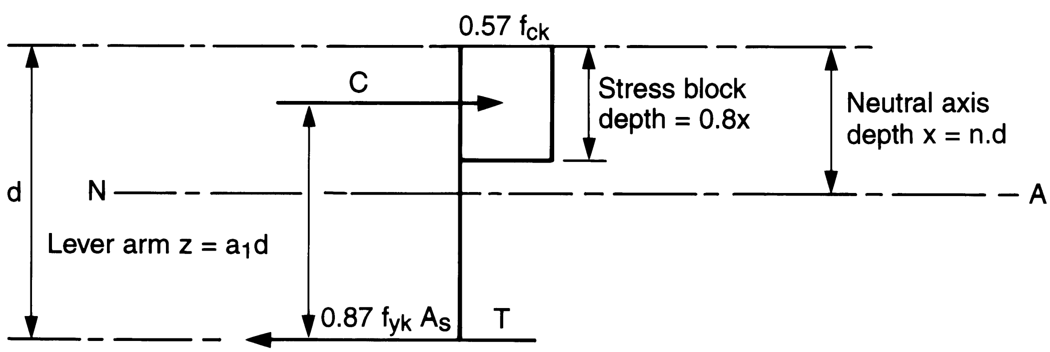


Fig. 5.4 Stress diagram

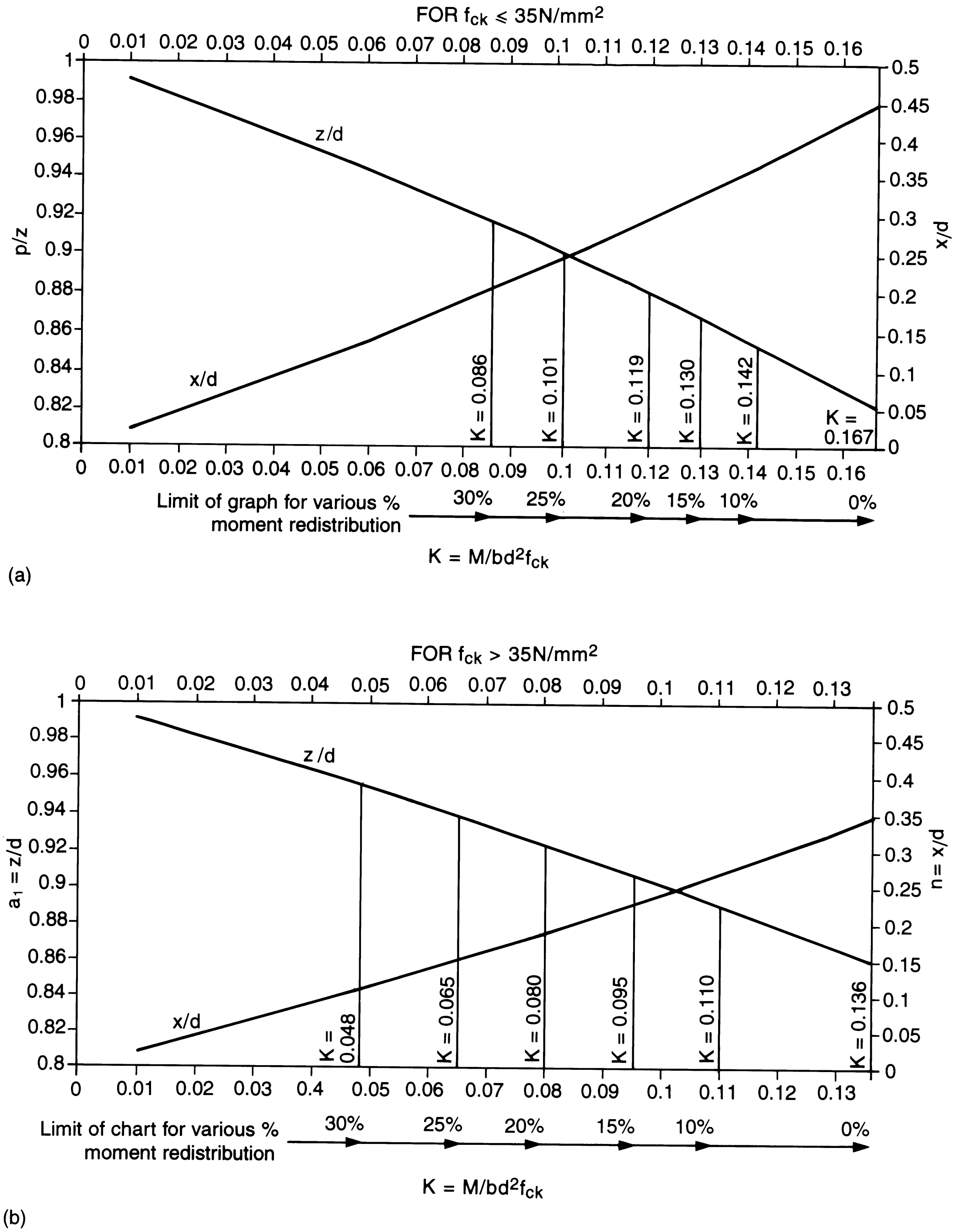


Fig. 5.5 (a) Values of lever arm and N.A. depth for $f_{ck} = \leq 35N/mm^2$
(b) Values of lever arm and N.A. depth for $f_{ck} > 35N/mm^2$

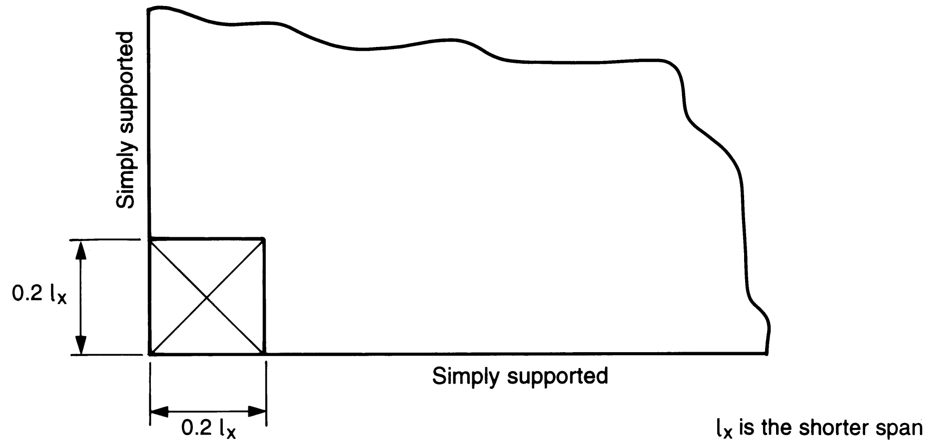


Fig. 5.6 Corner reinforcement: two-way spanning slabs

Minimum and maximum reinforcement

The area of reinforcement in each direction should be not less than the greater of:

- 0.0015bd in the case of high yield steel, or
- 0.0024bd in the case of mild steel, or
- if control of shrinkage and temperature cracking is critical, 0.0065bh high-yield steel or 0.012bh mild steel or
- 20% of the area of main reinforcement

where: d is the effective depth
b is the width for which the reinforcement is calculated
h is the overall depth of the slab in mm.

The area of tension or compression reinforcement in either direction should not exceed 4% of the area of concrete.

Main bars should not be less than 10mm in diameter.

To control flexural cracking the maximum bar spacing or maximum bar diameter of high-bond bars should not exceed the values given in Table 5.6, corresponding to the value of Q_k/G_k. If mild steel bars are used crack control should be checked in accordance with EC2. In any case bar spacings should not exceed the lesser of 3h or 500mm.

5.2.4.2 Shear

In the absence of heavy point loads there is normally no need to calculate shear stresses in slabs on linear supports.

For heavy point loads the punching shear stress should be checked using the method for shear around columns in flat slabs.

In flat slabs, shear stresses at the column perimeter should be checked first:

$$v_{Sd,p} = \frac{1000V_{eff}}{u_c d} \quad \text{N/mm}^2$$

where: V_{eff} is the effective shear force in kN (see subclause 5.2.3.4)
d is the average of the effective depth of the tension reinforcement in both directions, and
u_c is the column perimeter in mm.

$v_{Sd,p}$ should not exceed $0.9\sqrt{f_{ck}}$

Table 5.6 Alternative requirements to control crack widths for members reinforced with high bond bars

Q_k/G_k	Maximum bar spacing, mm		Maximum bar diameter, mm
0.2	175		20
0.3	200		20
0.4	225		20
0.5	225		20
0.6	250		20
0.7	250	OR	25
0.8	275		25
0.9	275		25
1.0	275		25
1.1	300		25
1.2	300		25
1.3	300		32

Note:
It is necessary only to satisfy one of the requirements in the Table, either the maximum bar spacing or the maximum bar diameter.

The shear stresses should then be checked at successive shear perimeters:

$$v_{\mathrm{Sd}} = \frac{1000V_{\mathrm{eff}}}{ud} \quad \mathrm{N/mm^2}$$

where u is the shear perimeter in mm as defined in Figs. 5.7, 5.8 (columns close to a free edge) or 5.9 (openings close to columns).

V_{eff} may be reduced by the load within the perimeter being considered.

Critical shear perimeter (b) in Fig. 5.7 is first checked. If the shear stress here is less than the permissible ultimate shear stress v_{Rd1} in Table 5.7, no further checks are required. If $v_{\mathrm{Sd}} > v_{\mathrm{Rd1}}$, successive perimeters have to be checked until one is reached where $v_{\mathrm{Sd}} \leq v_{\mathrm{Rd1}}$.

In slabs at least 200mm thick shear reinforcement should be provided within the area where the shear stress exceeds v_{Rd1} but is less that $2v_{\mathrm{Rd1}}$. If the shear stress exceeds $2v_{Rd1}$ column heads or drop panels should be incorporated or the slab thickness increased to reduce the shear stress to less than $2v_{\mathrm{Rd1}}$.

Shear reinforcement should consist of vertical links. The total area required is calculated from equations (a) or (b) below, whichever is appropriate. These equations should not be applied where $v_{\mathrm{Sd}} > 2v_{\mathrm{Rd1}}$.

(a) Where

$$v_{\mathrm{Rdl}} < v_{\mathrm{Sd}} \leq 1.6\ V_{\mathrm{Rdl}}$$

$$A_{\mathrm{sw}} = \frac{(v_{\mathrm{Sd}} - v_{\mathrm{Rdl}})ud}{0.87\ f_{\mathrm{ywk}}}$$

(b) Where

$$1.6V_{\mathrm{Rdl}} < v_{\mathrm{Sd}} \leq 2v_{\mathrm{dl}}$$

$$A_{\mathrm{sw}} = \frac{3.83(V_{\mathrm{sd}} - 1.4v_{\mathrm{Rdl}})ud}{f_{\mathrm{ywk}}}$$

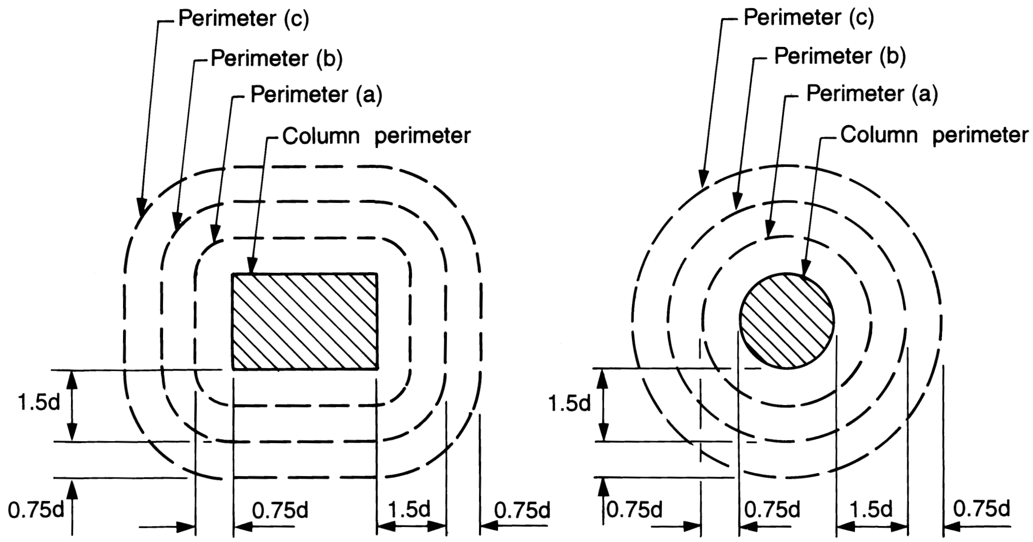
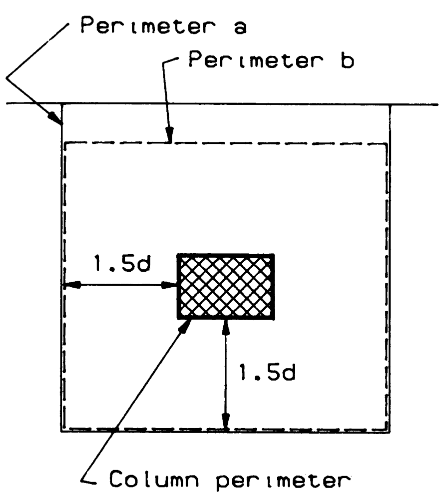


Fig. 5.7 Shear perimeters for internal columns



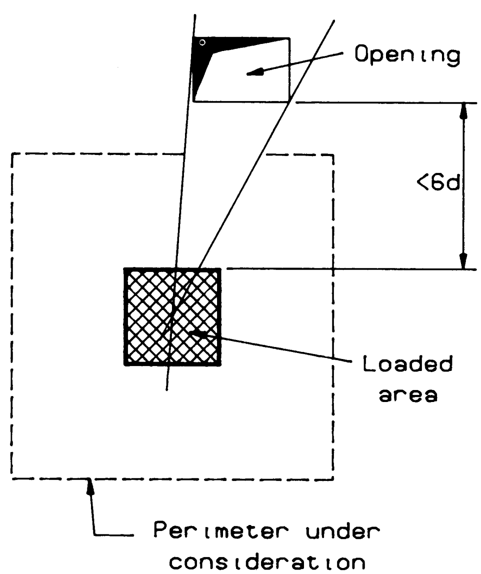


Fig. 5.8 Shear perimeter for edge column *Fig. 5.9 Effect of opening on shear perimeter*

However A_{sw} should not be taken as less than $0.00072ud$ for high tensile bars or $0.0013ud$ for mild steel bars.

The reinforcement should be provided on at least two perimeters between the column perimeter and perimeter (b) (see Figs. 5.7 and 5.10). The first perimeter of reinforcement should be located approximately $0.5d$ from the face of the column area and should contain not less than 40% of A_{sw}. The second perimeter should be located at not more than $0.75d$ from the first. The spacing of the legs of links around any perimeter should not exceed $0.6d$ or 300mm, whichever is less.

Table 5.7 Ultimate shear stress v_{Rd1}, N/mm²

$\frac{100 A_s}{b_w d}$	Effective depth, mm 150	175	200	225	250	300	400	500	>600
< 0.15	0.55	0.54	0.53	0.52	0.51	0.49	0.45	0.42	0.38
0.25	0.57	0.56	0.55	0.54	0.53	0.51	0.47	0.43	0.39
0.50	0.61	0.60	0.59	0.58	0.57	0.55	0.50	0.46	0.42
0.75	0.65	0.64	0.63	0.62	0.61	0.59	0.54	0.50	0.45
1.00	0.70	0.68	0.67	0.66	0.65	0.62	0.58	0.53	0.48
1.50	0.78	0.77	0.76	0.74	0.73	0.70	0.65	0.59	0.54
2.00	0.87	0.86	0.84	0.83	0.81	0.78	0.72	0.66	0.60
> 3.00	1.04	1.03	1.01	0.99	0.97	0.94	0.86	0.79	0.72

Notes to Table 5.7

1. The tabulated values apply for f_{ck} = 25N/mm²
   For f_{ck} = 20N/mm² the tabulated values should be multiplied by 0.87
   For f_{ck} = 30N/mm³² the tabulated values should be multiplied by 1.13
   For f_{ck} = 35N/mm² the tabulated values should be multiplied by 1.23
   For f_{ck} = 40N/mm² the tabulated values should be multiplied by 1.36
2. The Table does not allow for any contribution from axial loads. For an axial compression where stress of $\sigma_{cp} = (N/A_c)$ (N/mm²), the Table values should be increased by $0.15\sigma_{cp}$, where N is the design axial load and A_c is the area of concrete section.

The shear is now checked on perimeter (c). If reinforcement is required then this is provided between perimeter (a) and (c) in an analogous way to that used for the check on the critical perimeter.

5.2.4.3 Openings

When openings in floors or roofs are required such openings should be trimmed where necessary by special beams or reinforcement so that the designed strength of the surrounding floor is not unduly impaired by the opening. Due regard should be paid to the possibility of diagonal cracks developing at the corners of openings. The area of reinforcement interrupted by such openings should be replaced by an equivalent amount, half of which should be placed along each edge of the opening.

For flat slabs, openings in the column strips should be avoided.

5.2.5 Span/effective depth ratios

The span/effective depth ratio should not exceed the appropriate value in Table 5.8.

5.2.6 Section design – ribbed and coffered slabs

5.2.6.1 Bending

The bending moments per metre width obtained for solid slabs from clause 5.2.3 should be multiplied by the spacing of the ribs to obtain the bending moments per rib.

The rib section should be checked to ensure that the moment of resistance is not exceeded by using the methods for beams described in subsection 5.4. The area of tension reinforcement should be obtained from the same subsection. Structural topping should contain the minimum reinforcement indicated for solid slabs.

5.2.6.2 Span/effective depth ratios

The span/effective depth ratio should not exceed the appropriate value from Table 5.8.

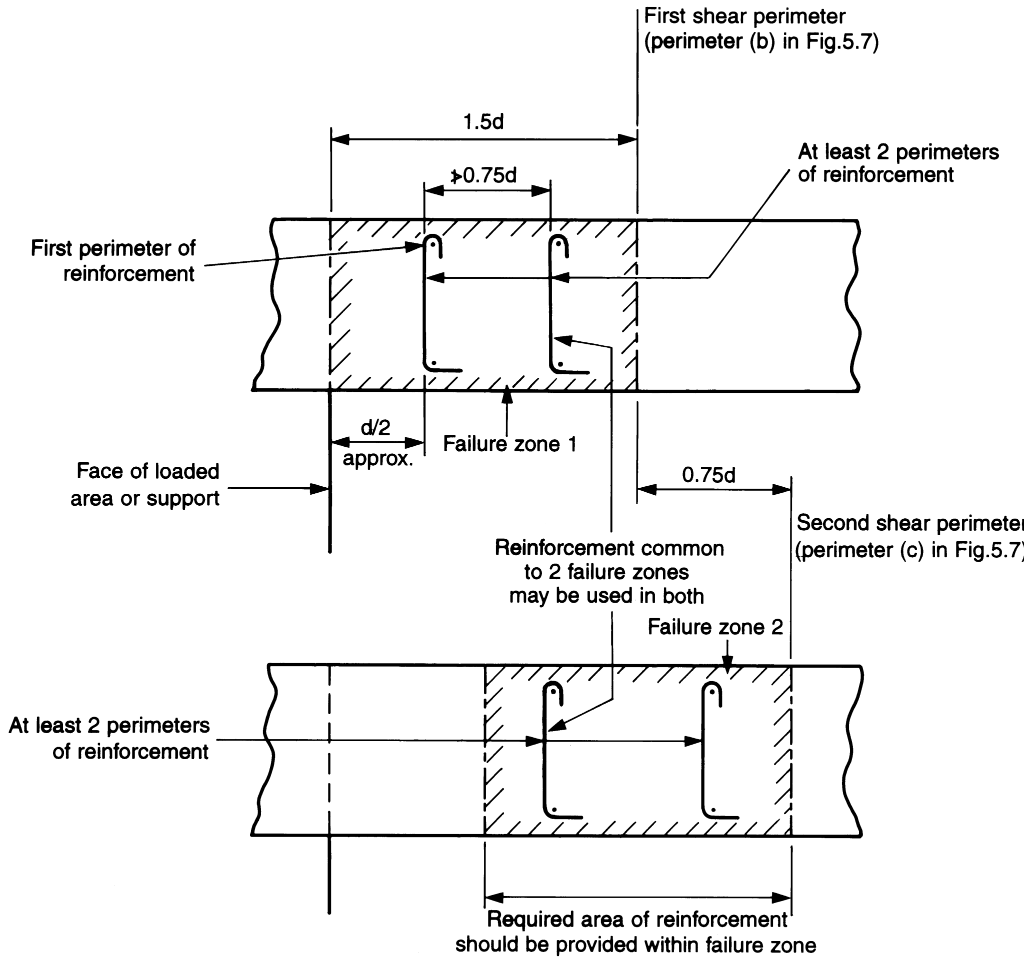


Fig. 5.10 Zones for punching shear reinforcement

Table 5.8 Span/effective depth ratios for slabs

Location	$A_s/bd \geq 1.5\%$	$A_s/bd = 0.5\%$	A_s/bd 0.15%
One-or two-way spanning slab:			
Simply supported	16	22	30
End span	20	28	38
Interior span	22	30	33
Flat slab	18	26	36
Cantilever	6	9	12

Notes to Table 5.8

1. Values may be interpolated.
2. For flanged sections where the ratio of the flange to the rib width exceeds 3, the values should be multiplied by 0.8.
3. For spans exceeding 7m, other than for flat slabs, supporting partitions liable to be damaged by excessive deflections, the value should be multiplied by 7/span (in metres).
4. The above assumes f_{yk} = 460N/mm^2. If other values of f_{yk} are used then multiply the above by 460/f_{yk}.
5. A_s/bd is calculated at the location of maximum span moment.
6. For flat slabs where the greater span exceeds 8.5m, the value should be multiplied by 8.5/span.

5.2.6.3 Shear

The shear force per metre width obtained from clause 5.2.3 should be multiplied by the spacing of the ribs to obtain the shear force per rib.

The shear stress should be calculated from

$$\nu_{\mathrm{Sd}} = \frac{1000\,V_{\mathrm{Sd}}}{b_{\mathrm{w}}d}$$

where: V_{Sd} = design shear stress in N/mm^2
V_{Sd} = design shear force arising from design ultimate loads per rib in kN
b_{w} = average width of the rib in mm
d = effective depth in mm.

If the shear stress v_{Sd} exceeds the permissible shear stress v_{Rd1} in Table 5.7 then one of the following should be adopted:

- Increase width of rib
- Reduce spacing of ribs
- Provide solid concrete at supports
- Provide shear reinforcement only if none of the above is possible.

For ribbed and coffered flat slabs, solid areas should be provided at columns, and the punching shear stress should be checked in a similar manner to the shear around columns in solid flat slabs.

5.2.6.4 Beam strips in ribbed and coffered slabs

Beam strips may be used to support ribbed and coffered slabs. The slabs should be designed as continuous, and the beam strips should be designed as beams spanning between the columns. The shear around the columns should be checked in a similar manner to the shear around columns in solid flat slabs. The shear in the ribs should be checked at the interface between the solid areas and the ribbed areas. If shear reinforcement is required in the ribs, these should be extended into the solid areas for a minimum distance equal to the effective depth.

5.2.7 Notes on the use of precast floors

Use of precast or semi-precast construction in an otherwise *in situ* reinforced concrete building is not uncommon. There are various proprietary precast and prestressed concrete floors on the market. Precast floors can be designed to act compositely with an *in situ* structural topping, although the precast element can carry loads without reliance on the topping. Design using proprietary products should be carried out closely in conjunction with the particular manufacturer and in accordance with ENV 1992–1–3: 1994[22]. The following notes may be helpful to the designer:

1. The use of a structural topping should be considered, particularly to reduce the risk of cracking in the screed and finishes:
   (a) when floors are required to resist heavy concentrated loads such as those due to storage racking and heavy machinery
   (b) when resistance to moving loads such as forklift trucks is required or to provide diaphragm action when a floor is used which would otherwise have insufficient capacity for transmitting in-plane shear.

   When used a structural topping should always incorporate light fabric reinforcement.
2. In selecting a floor, fire rating, durability and acoustic insulation need to be considered as well as structural strength

3. Precast components should be detailed to give a minimum bearing (after allowing for tolerances) of 75mm on concrete beams and walls, but in cases where this bearing cannot be achieved reference should be made to ENV 1992-1-3[22] for more detailed guidance. Mechanical anchorage at the ends should be considered. The design should cater for the tying requirements for accidental loading (see subsection 5.11)
4. Precast floor units, particularly those that are prestressed, have cambers that should be allowed for in the thickness of finishes. When two adjoining units have different spans, any differential camber could also be critical, and this has to be allowed for in the applied finishes (both top and bottom)
5. A ceiling to mask steps between adjoining units may be necessary
6. Holes required for services need to be planned
7. An *in situ* make-up strip should be provided to take up the tolerances between precast units and *in situ* construction.

5.3 Structural frames

5.3.1 Division into subframes

The moments, loads and shear forces to be used in the design of individual columns and beams of a frame supporting vertical loads only may be derived from an elastic analysis of a series of subframes. Each subframe may be taken to consist of the beams at one level, together with the columns above and below. The ends of the columns remote from the beams may generally be assumed to be fixed unless the assumption of a pinned end is clearly more reasonable. Normally a maximum of only five beam spans need be considered at a time. For larger buildings, several overlapping subframes should be used. Other than for end spans of a frame, subframes should be arranged so that there is at least one beam span beyond that beam for which bending moments and shear forces are sought.

The relative stiffness of members may be based on the gross concrete section ignoring reinforcement. For the purpose of calculating the stiffness of flanged beams the flange width of T- and L-beams may be taken from Table 5.9, in which l = length of the span or cantilever and b_w = width of the web.

5.3.2 Elastic analysis

Frames should be analysed for the most unfavourable arrangements of design loads.

For frames subjected to predominantly uniformly distributed loads it will be sufficient to consider the following arrangements of loads only for ultimate limit state verification:

(a) Alternate spans carrying the maximum design dead and imposed load, i.e. ($1.35G_k + 1.5Q_k$), other spans carrying the maximum design dead load, i.e. $1.35G_k$.
(b) Any two adjacent spans carrying the maximum design dead and imposed load, i.e. ($1.35G_k + 1.5Q_k$), other spans carrying the maximum design dead load, i.e. $1.35G_k$.

Maximum column moments should be obtained using the following arrangements of loads:

alternate spans carrying $1.1G_k + 1.5Q_k$, all other spans carrying $0.9G_k$.

5.3.3 Redistribution of moments

The moments obtained from elastic analysis may be redistributed up to a maximum of 30% to produce members that are convenient to detail and construct, noting that:

(a) the resulting distribution of moments remains in equilibrium with the applied load
(b) the design redistribution moment at any section should not be less than 70% of the elastic moment

Table 5.9 Effective widths of flanged beams

	T-beam	L-beam
End span	$b_w + 0.7l$	$b_w + 0.085l$
Interior spans	$b_w + 0.14l$	$b_w + 0.07l$
Cantilever	$b_w + 0.20l$	$b_w + 0.10l$

Notes to Table 5.9

1. The ratio of the adjacent spans should be between 1 and 1.5.
2. The length of the cantilever should be less than half the adjacent span.
3. The actual flange width should be used where it is less than the value obtained from the Table.

(c) there are limitations on the depth of the neutral axis of the section depending on the percentage of redistribution (see subclause 5.4.4.1), and
(d) the design moment for the columns should be the greater of the redistributed moment or the elastic moment prior to redistribution.

A simple procedure may be adopted that will satisfy the above criteria:

1. Alternate spans loaded
Move the moment diagram of the loaded span up or down by the percentage redistribution required; do not move moment diagram of the unloaded span (see Fig. 5.11).

2. Adjacent spans loaded
Reduce the common support moment to not less than the support obtained from the alternate spans loaded case (1 above).

5.3.4 Design shear forces

Shear calculations at the ultimate limit state may be based on the shear forces compatible with the bending moments arising from the load combinations noted in clause 5.3.2 and any redistribution carried out in accordance with clause 5.3.3.

5.4 Beams

5.4.1 Introduction

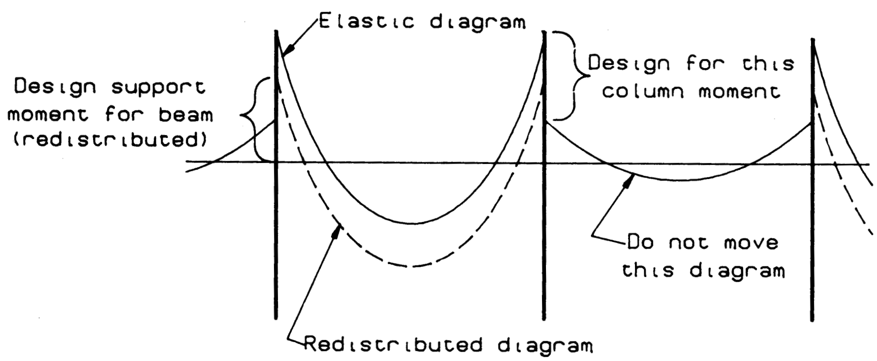
This subsection describes the final design of beams of normal proportions and spans. Deep beams with a clear span less than twice the effective depth are not considered.

The general procedure to be adopted is as follows:

1. Check that the section complies with the requirements for fire resistance
2. Check that cover and concrete quality comply with durability requirements
3. Calculate bending moments and shear forces according to clause 5.4.3
4. Calculate reinforcement required for bending and shear
5. Check span/depth ratio.

The effective span of a simply supported beam should normally be taken as the clear distance between the faces of supports plus one-third of their width. However, where a bearing pad is provided between the slab and the support, the effective span should be taken as the distance between the centres of the bearing pads.

The effective span of a beam continuous over its supports should normally be taken as the distance between the centres of the supports.



(a) Downward distribution of 'span loaded' diagram

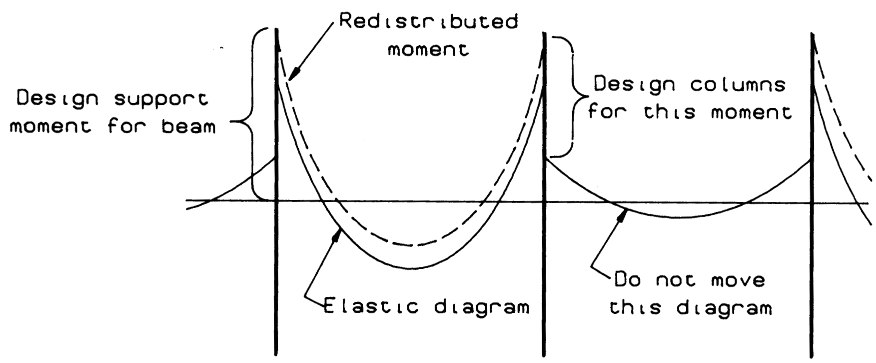


Fig. 5.11 Redistribution procedures for frames

The effective length of a cantilever beam where this forms the end of a continuous beam is the length of the cantilever from the centre of the support. Where the beam is an isolated cantilever the effective length is is the length of the cantilever from the face of the support.

To prevent **lateral buckling**, the length of the compression flange measured between adequate lateral restraints to the beam should not exceed 50*b*, where *b* is the width of the compression flange, and the overall depth of the beam should not exceed 4*b*.

In normal slab-and-beam or framed construction specific calculations for torsion are not usually necessary, torsional cracking being adequately controlled by shear reinforcement. Where torsion is essential for the equilibrium of the structure, e.g. the arrangement of the structure is such that loads are imposed mainly on one face of a beam without corresponding rotational restraints being provided, EC2 should be consulted.

5.4.2 Fire resistance and durability

5.4.2.1 Fire resistance

The member sizes and reinforcement covers required to provide fire resistance are shown in Table 5.10.

If the width of the beam is more than the minimum in Table 5.10 the cover may be decreased as follows:

Increase in width, mm	**Decrease in cover, mm**
25	5
50	10
100	15
150	15

Where the cover to the outermost reinforcement exceeds 40mm special precautions against spalling may be required, e.g. partial replacement by plaster, lightweight aggregate or the use of fabric as supplementary reinforcement. For beams with sloping sides, the width in Table 5.10 should be measured at the centroid of the tensile reinforcement. Table 5.10 applies to all beams that can be exposed to fire on three sides only, i.e. the upper side is insulated by a slab.

5.4.2.2 Durability

The requirements for durability in any given environment are:

(a) an upper limit to the water/cement ratio
(b) a lower limit to the cement content
(c) a lower limit to the thickness of the cover to the reinforcement
(d) good compaction
(e) adequate curing, and
(f) good detailing.

Values for (a), (b) and (c) which, in combination, will give adequate durability are given in Table 5.11 for various environments.

As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 5.11 gives, in addition, the characteristic strengths that have to be specified in the UK to ensure that requirements (a) and (b) are satisfied.

The concrete class strengths quoted in Table 5.11 will often require cement contents that are higher than those given in the Table. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

For frost resistance the use of air entrainment of the concrete should be considered; however the effects of air entrainment on the concrete properties should be taken into account.

5.4.3 Bending moments and shear forces

The maximum values of the bending moments and shear forces at any section of a continuous beam may be obtained by either:

(a) consideration of the beam as part of a structural frame as described in subsection 5.3, or
(b) as a beam that is continuous over its supports and capable of free rotation about them.

Table 5.10 Fire resistance and cover for beams

	Minimum width, mm		Nominal cover to main steel, mm	
Fire resistance hours	Simply supported	Continuous	Simply supported	Continuous
1	120	120	30	20
1.5	150	120	40	35
2	200	150	50	50
3	240	200	70	60
4	280	240	80	70

For beams with (a) substantially uniform loading, (b) one type of imposed load, and (c) three or more spans that do not differ by more than 15%, the bending moments and shear forces may be calculated using the coefficients given in Table 5.12 for ultimate limit state verification.

5.4.4 Section design

5.4.4.1 Bending

The most common beams have flanges at the top. At the supports they are designed as rectangular beams and in the spans as flanged beams. For upstand beams, the reverse applies.

If the applied moment M is less than the limiting moment M_u for the concrete, compression steel will not be needed.

The resistance moments of concrete sections that are required to resist flexure only can be determined from the formulas that are based on the stress diagram in Fig. 5.12.

The effect of any small axial load on the beam can be ignored if the design ultimate axial force is less than $0.08 f_{ck} bh$, where h is the overall depth of the section.

Table 5.11 Durability requirements for beams

Conditions of exposure (For definitions see Appendix C)	Nominal cover to all reinforcement mm	mm	mm	mm	mm
1. Dry environment	20	20	20	20	20
2. Humid environment					
(a) without frost	–	35	35	30	30
(b) with frost	–	–	35	30	30
3. Humid with frost and de-icing salts	–	–	40	35	35
4. Seawater environment					
(a) without frost	–	–	40	35	35
(b) with frost	–	–	40	35	35
The following classes may occur alone or in combination with above:					
5. Aggressive chemical environment					
(a) slightly aggressive	–	–	35	30	30
(b) moderately aggressive	–	–	–	30	30
(c) highly aggressive*	–	–	–	–	45
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content, kg/m^3	260	280	300†	300	300
Concrete class strength N/mm^2 (cylinder/cube)	C25/30	C30/37	C35/45	C40/50	C45/55 or better

Notes to Table 5.11

1 Cover is expressed in terms of nominal values which have been obtained from the minimum values by allowing for a negative tolerance of 5mm.

2 The cover in mm to the main reinforcement should not be less than the bar diameter.

*Protective barrier to prevent direct contact with highly aggressive media should be provided.

†280kg/m^3 for exposure classes 2b and 5a.

Table 5.12 Bending moments and shear forces for beams

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0*	$0.09Fl$	$-0.11Fl$	$0.07Fl$	$-0.1Fl$
Shear	$0.45F$	—	$0.6F$	—	$0.55F$

*See clause 5.4.4.2

F is the total design ultimate load ($1.35G_k + 1.5Q_k$) for each span, and l is the length of the span.

No redistribution of moments should be made when using values obtained from this Table.

Rectangular beams

The procedure for the design of rectangular beams is as follows:

(a) Calculate $M_u = K_{lim} f_{ck} bd^2$ where K_{lim} is obtained from Fig. 5.13 for the amount of redistribution carried out.

(b) If $M < M_u$, the area of tension reinforcement A_s is calculated from:

$$A_s = M/(0.87f_{yk})z$$

where z is obtained from Fig. 5.13 for different values of

$$K = \frac{M}{bd^2 f_{ck}}$$

(c) If $M > M_u$ then compression reinforcement is needed. The area of compression reinforcement A'_s is calculated from:

$$A'_s = (M - M_u)/0.87f_{yk}\ (d - d')$$

where d' is the depth to the centre of the compression reinforcement from the com pression face.

If $d' > (1 - f_{yk}/800)x$, use $700\ (l - d'/x)$ in lieu of $0.87f_{yk}$.

The area of tension reinforcement A_s is calculated from:

$$A_s = (M_u/0.87\ f_{yk}z) + A'_s$$

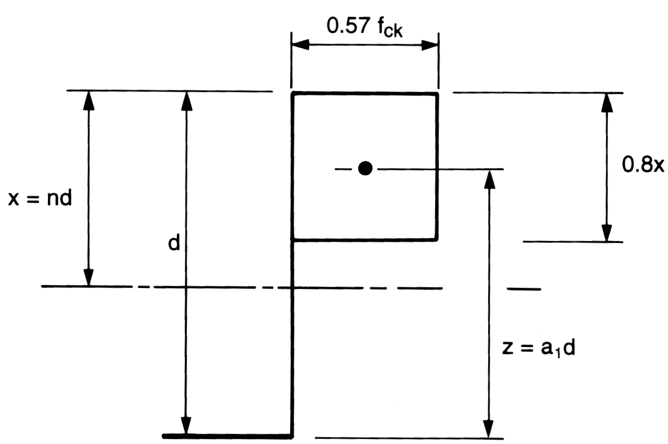


Fig 5.12 Stress diagram

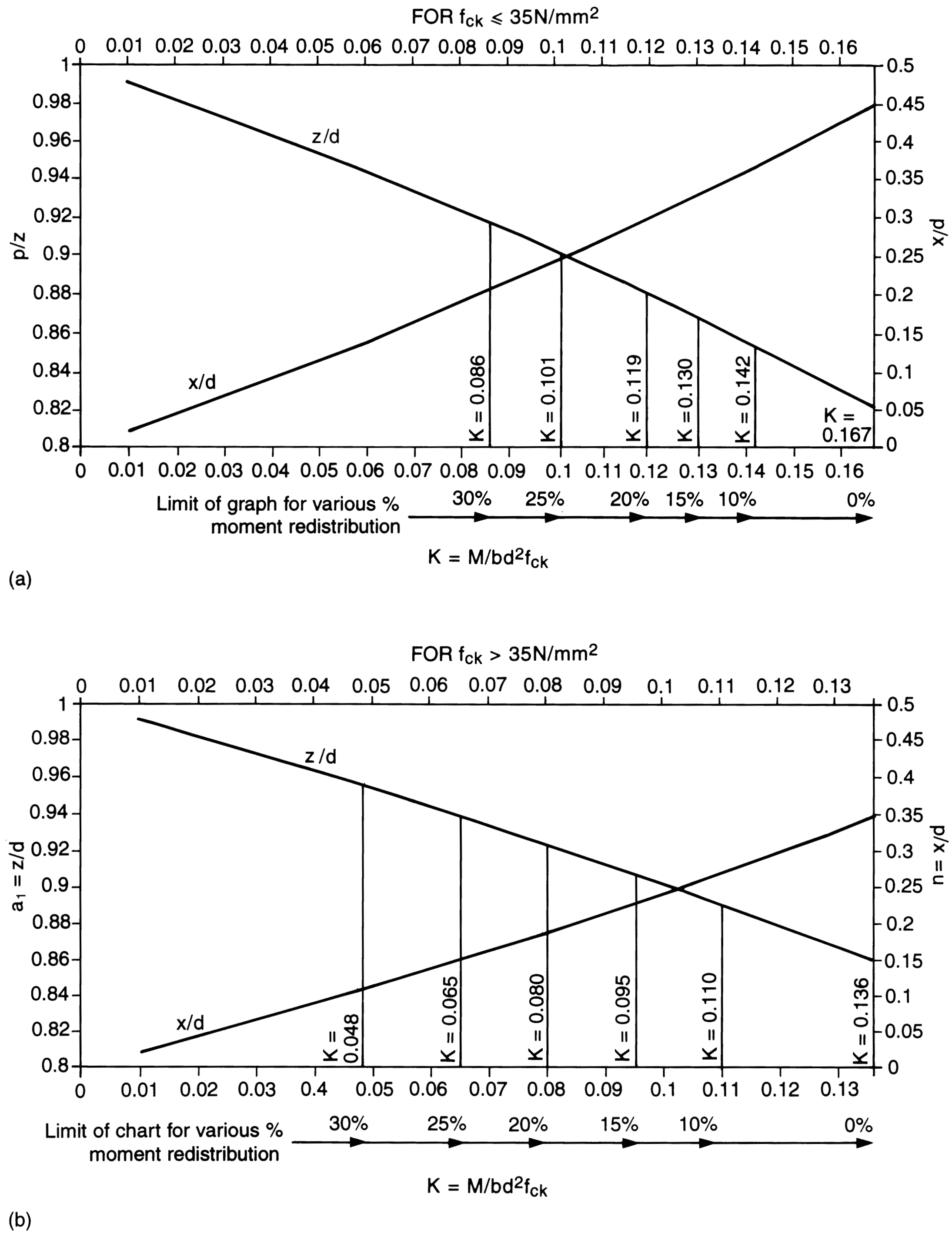


Fig. 5.13 (a) Values of lever arm and N.A. depth for f_{ck} = ≤ 35N/mm²
(b) Values of lever arm and N.A. depth for f_{ck} > 35N/mm²

Flanged beams
For section design the effective width of a flanged beam (see Fig. 5.14) should be obtained from Table 5.13.

The procedure for the design of flanged beams is as follows:

(a) Check the position of the neutral axis by determining $K = M/bd^2f_{ck}$ using effective flange width b and selecting values of x and z from Fig. 5.13.
(b) If $0.8x < h_f$ then A_s is determined as for a rectangular beam of breadth b, i.e

$$A_s = M/(0.87f_{yk})z$$

(c) If $0.8x > h_f$ then the stress block lies outside the flange. Calculate the resistance moment of the flange M_{uf} from:

$$M_{uf} = 0.57f_{ck}(b - b_w)h_f\,(d - 0.5h_f)$$

(d) Calculate $K_f = (M - M_{uf})/f_{ck}b_wd^2$

If $K_f = K_{lim}$, obtained from Fig. 5.13 for the amount of redistribution carried out, then select value of z/d and hence z. Calculate A_s from:

$$A_s = [M_{uf}/0.87f_{yk}(d - 0.5h_f\,)] + (M - M_{uf})/0.87f_{yk}z$$

If $K_f > K_{lim}$, redesign the section.

5.4.4.2 Minimum and maximum amounts of reinforcement
The areas of reinforcement derived from the previous calculations may have to be modified or supplemented in accordance with the requirements below in order to prevent brittle failure (without warning) and/or excessive cracking. Main bars in beams should normally not be less than 16mm in diameter.

Tension reinforcement
The minimum areas of tension reinforcement should be $0.0015b_td$, where b_t is the mean width of the tension zone.

In monolithic construction, even when simple supports have been assumed in design (e.g. the end support of a continuous beam), the section should be designed for a support moment of at least 25% of the maximum bending moment in the span.

At intermediate supports of continuous beams, the total amount of tensile reinforcement A_s of a flanged cross-section may be distributed uniformly over a width of $2b_w$ as shown in Fig. 5.15.

Compression reinforcement
The minimum areas of compression reinforcement where required should be:

- rectangular beam $0.002bh$
- flanged beam web in compression $0.002b_wh$

Table 5.13 Effective widths of flanged beams

	T-beam	L-beam
End span	$b_w + 0.17l$	$b_w + 0.085l$
Interior spans	$b_w + 0.14l$	$b_w + 0.07l$
Cantilever	$b_w + 0.20l$	$b_w + 0.10l$

Notes to Table 5.13
1. The ratio of the adjacent spans should be between 1 and 1.5.
2. The length of the cantilever should be less than half the adjacent span.
3. The actual flange width should be used where it is less than the value obtained from the Table.

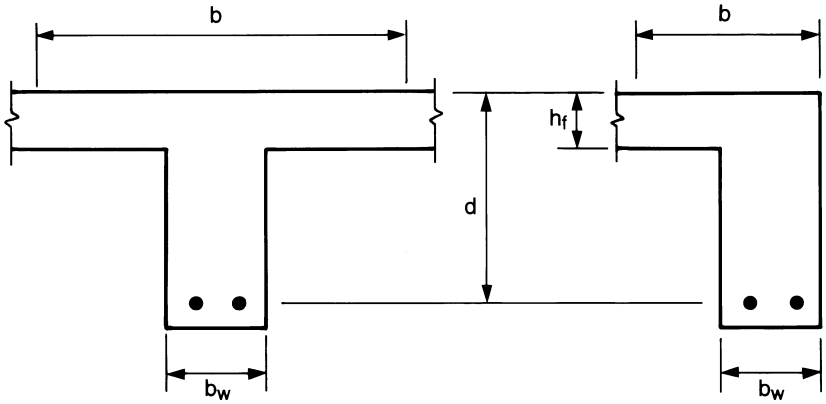


Fig. 5.14 Beam sections

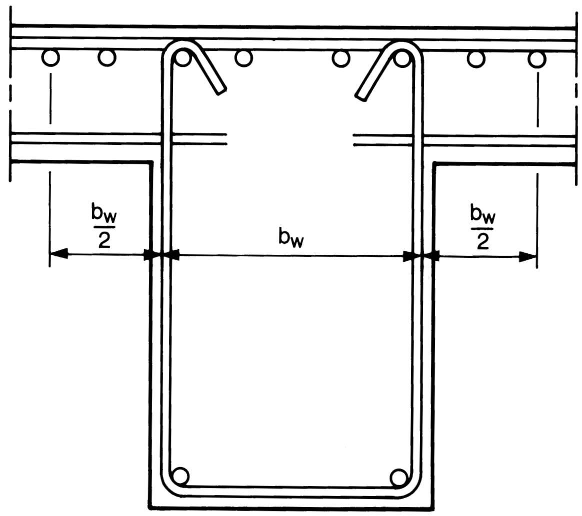


Fig. 5.15 Distribution of reinforcement in flanged beam

Maximum area of reinforcement
Neither the area of the tension reinforcement, nor the area of compression reinforcement should exceed 4% of the area of concrete.

Minimum areas of bars in the side face of beams (to control cracking)
Where the overall depth of the beam exceeds 1000mm, and the main reinforcement is concentrated in only a small proportion of the depth, reinforcement should be provided in the side faces of the beam to control cracking. This longitudinal reinforcement should be placed within the links, and should be evenly distributed between the level of the tension steel and the neutral axis.

The area of steel $A_s = 0.6b\,(0.83d - x)/\sigma_s$, where σ_s is the stress induced into the reinforcement immediately after cracking, b, d and x are in mm and σ_s in N/mm^2. The value of b need not be taken as more than 500mm.

When high-bond bars are used the procedure to be followed is:

(a) arbitrarily select a value for σ_s
(b) calculate A_s from the formula above

(c) obtain the maximum bar spacing corresponding to σ_s from Table 5.14
(d) obtain the maximum bar diameter corresponding σs from Table 5.14
(e) check that reinforcement can be provided that satisfies either
    (b) and (c) or
    (b) and (d)

If either cannot be achieved adjust σ_s and repeat steps (b) to (e).

If mild-steel bars are used crack control should be checked in accordance with EC2.

Maximum spacing for tension bars
To control flexural cracking at serviceability the maximum bar spacing or maximum bar diameters of high-bond bars should not exceed the values in Table 5.14, corresponding to the value of Q_k/G_k. If mild steel bars are used crack control should be checked in accordance with EC2.

Minimum spacing
The horizontal or vertical distance between bars should not be less than the bar diameter or 20mm or aggregate size + 5mm, whichever is the greatest.

Where there are two or more rows the gaps between corresponding bars in each row should be in line vertically, and the space between the resulting columns of bars should permit the passage of an internal vibrator.

Table 5.14 Alternative requirements to control crack widths for members reinforced with high-bond bars

Q_k/G_k steel stress or σ_s N/mm^2	Serviceability spacing mm	Maximum bar		Maximum bar diameter mm
–	450	–		6
–	400	–		8
–	360	50		10
–	320	100		12
–	280	150		16
0.2	247	175		20
0.3	242	200		20
0.4	230	225		20
0.5	219	225	OR	20
0.6	210	250		20
0.7	201	250		25
0.8	195	275		25
0.9	188	275		25
1.0	182	275		25
1.1	177	300		25
1.2	173	300		25
≥ 1.3	168	300		32

Note: It is necessary only to satisfy one of the requirements in the Table, either the maximum bar spacing or the maximum bar diameter.

5.4.4.3 Shear

The design ultimate shear force at the face of the support should not exceed V_{Rd2} where:

$$V_{Rd2} = 0.3\nu f_{ck} b_w d$$

where ν is obtained from Table 5.15.

For the design of shear reinforcement, the design ultimate shear force, V_{Sd} may be taken as that acting at a distance d from the face of the support.

Where V_{Sd} exceeds V_{Rd1}, the shear capacity of the section without shear reinforcement, shear reinforcement will be required. This may be assessed by either the standard method or the variable strut inclination method (see below).

$V_{Rd1} = v_{Rd1} b_w d$ where v_{Rd1} is obtained from Table 5.16
b_w is the width of the beam web
d is the effective depth.

Where loads are supported at the bottom of the beams, the link should be designed to carry the applied load in tension in addition to any shear forces.

Where concentrated loads are located at a distance x from the face of the support, and x is less than or equal to $2.5d$, then the values of v_{Rd1} given in Table 5.16 may be multiplied by a factor β where $\beta = 2.5d/x$, with $1.0 < \beta < 5.0$.

When this enhancement is taken into account, V_{Rd1} and shear reinforcement should be calculated at all critical sections over the length $2.5d$ from the support, with ß = 1.0 on the span side of the relevant concentrated loads. The maximum shear reinforcement so obtained should be provided over this entire length.

The variable-strut inclination method may lead to savings in shear reinforcement, but will be subject to more severe curtailment rules (see clause 5.12.6).

(a) *Standard method*
Shear reinforcement in the form of vertical links should be provided in accordance with

$$A_{sw} = \frac{1.28s(V_{sd} - V_{rd1})}{f_{ywk} d}$$

where: A_{sw} is the total cross-section of the link(s) (2 legs for a single closed link, 4 legs for a double closed link)

s is the longitudinal spacing of the links, and

f_{ywk} is the characteristic strength of the links.

(b) *Variable-strut inclination method*
Calculate the inclination of the strut, θ, from:

$$\cot\theta = \frac{0.3\nu f_{ck}\, b_w\, d}{V_{Sd}} + \left[\left(\frac{0.3\nu f_{ck}\, b_w\, d}{V_{Sd}}\right)^2 - 1\right]$$

but not greater than 1.5.

Table 5.15 Effectiveness factor, *ν*

f_{ck}, N/mm^2	20	25	30	35	≥ 40
ν	0.600	0.575	0.550	0.525	0.500

Table 5.16 Ultimate shear stress v_{Rd1}, N/mm²

$\frac{100\ A_s}{b_w d}$	Effective depth, mm 150	175	200	225	250	300	400	500	> 600
< 0.15	0.55	0.54	0.53	0.52	0.51	0.49	0.45	0.42	0.38
0.25	0.57	0.56	0.55	0.54	0.53	0.51	0.47	0.43	0.39
0.50	0.61	0.60	0.59	0.58	0.57	0.55	0.50	0.46	0.42
0.75	0.65	0.64	0.63	0.62	0.61	0.59	0.54	0.50	0.45
1.00	0.70	0.68	0.67	0.66	0.65	0.62	0.58	0.53	0.48
1.50	0.78	0.77	0.76	0.74	0.73	0.70	0.65	0.59	0.54
2.00	0.87	0.86	0.84	0.83	0.81	0.78	0.72	0.66	0.60
> 3.00	1.04	1.03	1.01	0.99	0.97	0.94	0.86	0.79	0.72

Notes to Table 5.16

1. The tabulated values apply for f_{ck} = 25N/mm²
   For f_{ck} = 20N/mm² the tabulated values should be multiplied by 0.87
   For f_{ck} = 30N/mm² the tabulated values should be multiplied by 1.13
   For f_{ck} = 35N/mm² the tabulated values should be multiplied by 1.23
   For f_{ck} = 40N/mm² the tabulated values should be multiplied by 1.36
2. The Table does not allow for any contribution from axial loads. For an axial compression where stress of $\sigma_{cp} = N/A_c$ (N/mm²), the Table values should be increased by $0.15\sigma_{cp}$, where N is the design axial load and A_c is the area of concrete section.

Shear reinforcement in the form of vertical links should be provided in accordance with

$$A_{sw} = \frac{1.28sV_{sd}}{f_{ywk}d\cot\theta}$$

where: A_{sw} is the total cross-section of the link(s) (2 legs for a single closed link, 4 legs for a double closed link)
s is the longitudinal spacing of the links, and
f_{ywk} is the characteristic strength of the links.

Minimum reinforcement and spans

Whichever method is used minimum shear reinforcement should be provided in accordance with $A_{sw} = \rho_w s b_w$. where ρ_w is obtained from Table 5.17.

The diameter of the shear reinforcement should not exceed 12mm where it consists of plain round bars.

The maximum spacing of shear reinforcement is given in Table 5.18.

Arrangement of links

For compression reinforcement in an outer layer, every corner bar should be supported by a link passing round the bar and having an included angle of not more than 135°. A maximum of 5 bars in or close to each corner can be secured against buckling by any one set of links.

Table 5.17 Shear reinforcement ratios

Concrete class	Ratio ρ_w f_{ywk} = 460 N/mm²	f_{ywk} = 250 N/mm²
C25/30 to C35/45	0.0012	0.0022
C40/50 to C50/60	0.0015	0.0028

Table 5.18 Maximum spacing of shear reinforcement

Design shear force, V_{Sd}	Longitudinal spacing* mm	Transverse spacing mm
$\leq 0.2V_{Rd2}$	$0.8d \leq 300$	$d \leq 800$
$> 0.2V_{Rd2}$ and $\leq 0.67V_{Rd2}$	$0.6d \leq 300$	
$> 0.67V_{Rd2}$	$0.3d \leq 200$	

*When $V_{Sd} > 3V_{Rd1}$ the longitudinal spacing should not exceed the values in Table 5.19 if these are more critical.

Table 5.19 Longitudinal spacing values

$\frac{(V_{sd} - 3V_{Rd1})}{A_{sw}d}$ N/mm^3	Spacing mm
≤ 0.17	300
0.38	200
0.67	150
1.50	100
4.00	50

Table 5.20 Span/effective depth ratios for beams

Location	A_s/bd $\geq 1.5\%$	A_s/bd $= 0.5\%$	A_s/bd $= 0.15\%$
Simply supported beam	16	22	30
End span of continuous beam	20	28	38
Interior span	22	30	33
Cantilever	6	9	12

Notes on Table 5.20

1. Values may be interpolated.
2. For flanged sections where the ratio of the flange width to the rib width exceeds 3, the values should be multiplied by 0.8.
3. For spans exceeding 7m supporting partitions liable to be damaged by excessive deflections, the value should be multiplied by 7/span (in metres)
4. The above assumes $f_{yk} = 460\text{N/mm}^2$. If other values of f_{yk} are used then multiply the above by $460/f_{yk}$.
5. A_s/bd is calculated at the location of maximum span moment.

Openings

In locations where $V_{Sd} < V_{Rd1}$, small openings not exceeding $0.25d$ in diameter can be permitted within the middle third of the depths of beams without detailed calculations. Where these conditions are not met, detailed calculations should be carried out.

5.4.5 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value in Table 5.20.

5.5 Columns

5.5.1 Introduction

This subsection describes the final design of columns in braced, non-sway structures resisting axial loads and bending moments. A method is given for biaxial bending.

The general procedure to be adopted is as follows:

1. Check the slenderness of the column
2. Check that section size and cover comply with requirements for fire resistance
3. Check that cover and concrete comply with requirements for durability
4. Calculate axial loads and moments according to clause 5.5.3 to 5.5.5
5. Design section and reinforcement.

5.5.2 Slenderness, fire resistance and durability

The size of column, concrete grade and the cover to reinforcement should be determined by taking into account the requirements of slenderness, fire resistance and durability. To facilitate concreting the minimum dimension of a column should not be less than 200mm.

5.5.2.1 Slenderness

If the ratio of the effective height of a column to its least cross-sectional dimension exceeds $(l_e/h)_{crit}$, which is obtained from Fig. 5.16, then the column is slender. Fig. 5.16 is obtained from:

$$(l_e/h)_{crit} = 7.21\left(2 - \frac{M_1}{M_2}\right)$$

M_2 is the numerically larger end moment on the column, and M_1 is the numerically smaller end moment.
(Note that for columns in most framed structures M_1 will have the opposite sign to M_2)

The effective height may be obtained by multiplying the clear height between the lateral restraints at the two ends of the column by the factor obtained from Table 5.21.

5.5.2.2 Fire resistance

Minimum dimensions and cover are given in Table 5.22.

5.5.2.3 Durability

The requirements for durability in any given environment are:

(a) an upper limit to the water/cement ratio
(b) a lower limit to the cement content
(c) a lower limit of the cover to the reinforcement
(d) good compaction
(e) adequate curing, and
(f) good detailing.

Table 5.21 Effective height, l_e, factors for columns

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

Condition 1: Column connected monolithically to beams on each side that are at least as deep as the overall depth of the column in the plane considered. Where the column is connected to a foundation this should be designed to carry moment, in order to satisfy this condition.
Condition 2: Column connected monolithically to beams or slabs on each side that are shallower than the overall depth of the column in the plane considered, but generally not less than half the column depth.
Condition 3: Column connected to members that do not provide more than nominal restraint to rotation.

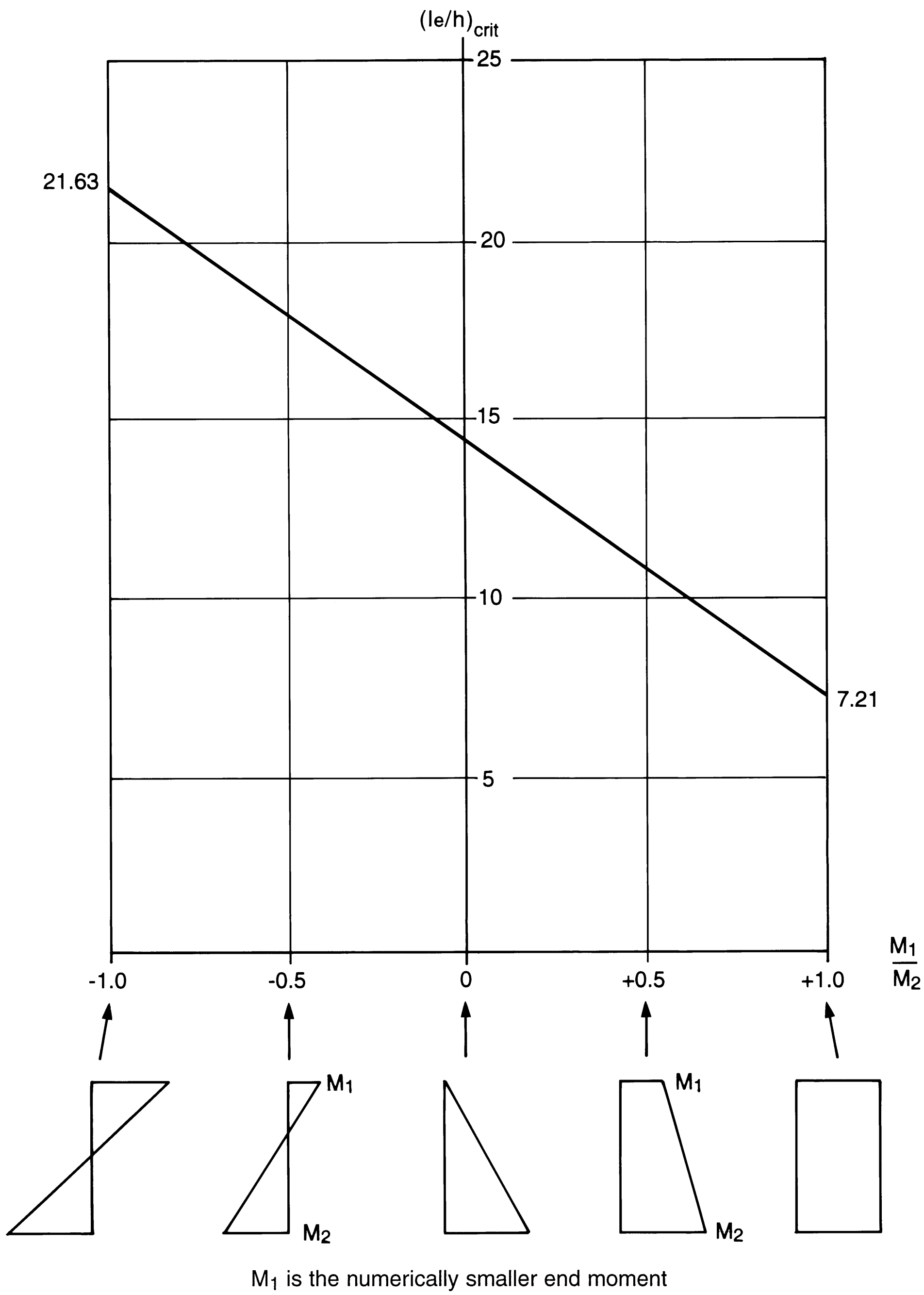


Fig. 5.16 Critical slenderness ratios for braced columns

Values for (a), (b) and (c) that, in combination, will be adequate for durability purposes are given in Table 5.23 for various environments.

As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 5.23 gives, in addition, the concrete class strengths that have to be specified in the UK so that requirements (a) and (b) are satisfied.

Table 5.22 Fire resistance requirements for columns

	Minimum dimension mm			
Fire resistance, hours	Fully exposed	50% exposed	One side exposed	Cover to main reinforcement mm
1	200	200	200	25
1.5	250	200	200	30
2	300	200	200	35
3	400	300	200	35
4	450	350	240	35

The strengths quoted in Table 5.23 will often require cement contents that are higher than those given in the Table. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

For frost resistance the use of air entrainment of the concrete should be considered; however the effects of air entrainment on the concrete properties should be taken into account.

5.5.3 Axial loads and moments – non-slender columns

The minimum design moment for any column in any plane should be obtained by multiplying the ultimate design axial load by an eccentricity of 0.05 times the overall column dimension in the relevant plane.

When column designs are required in the absence of a full frame analysis the following procedure may be adopted:

(a) The axial loads may generally be obtained by increasing by 10% the loads obtained on the assumption that beams and slabs are simply supported. A higher increase may be required where adjacent spans and/or the loadings on them are grossly dissimilar.
(b) The moments in the columns may be obtained using the subframes shown in Fig. 5.17 subject to the minimum design moments mentioned previously.

Alternatively, axial loads and moments may be obtained from the frame analysis outlined in subsection 5.3.

5.5.4 Axial loads and moments – slender columns

5.5.4.1 General

When the slenderness ratio of a braced column in any direction exceeds $(l_e/h)_{crit}$, it is necessary to make allowance in the design for the possible effects of the ultimate deflection of the column. The possible design conditions are the design ultimate axial load combined with the greater of either:

(a) the maximum end moment arising from the first-order (initial) analysis of the structure, or
(b) the moments at around mid-height of the column arising from the first-order analysis combined with additional moments due to:
    (a) the ultimate deflection of the column, e_2, and
    (b) an accidental eccentricity to take account of any 'out of plumb' of the column, e_a.

The moments at around mid-height are given in Table 5.24.

Table 5.23 Durability requirements for columns

Conditions for exposure (for definitions see Appendix C)	Nominal cover to all reinforcement				
	mm	mm	mm	mm	mm
1. Dry environment	20	20	20	20	20
2. Humid environment					
(a) without frost	–	30	30	25	25
(b) with frost	–	–	30	25	25
3. Humid with frost and de-icing salts	–	–	35	30	30
4. Seawater environment					
(a) without frost	–	–	35	30	30
(b) with frost	–	–	35	30	30
The following classes may occur alone or in combination with those above:					
5. Aggressive chemical environment					
(a) slightly aggressive	–	–	30	25	25
(b) moderately	–	–	–	–	25
aggressive	–	–	–	–	40
(c) highly aggressive*					
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content, kg/m^3	260	280	300†	300	300
Concrete class strength	C25/30	C30/37	C35/45	C40/50	C45/55 or both

Notes on Table 5.23

1. Cover is expressed in terms of nominal values which have been obtained from the minimum values by allowing for a negative tolerance of 5mm.
2. The cover in mm to the main reinforcement should not be less than the bar diameter.

*Protective barrier to prevent direct contact with highly aggressive media should be provided.

†280 kg/m^3 for exposure classes 2b and 5a.

5.5.4.2 Calculation of first-order moments around mid-height

The first-order moment (M_{zi} or M_{yi}) at about mid-height of a braced column should be the greater of either:

$$0.6\,M_2 + 0.4\,M_1 \qquad \text{or} \qquad 0.4\,M_2$$

where M_1 is the numerically smaller end moment from first order analysis and M_2 is the

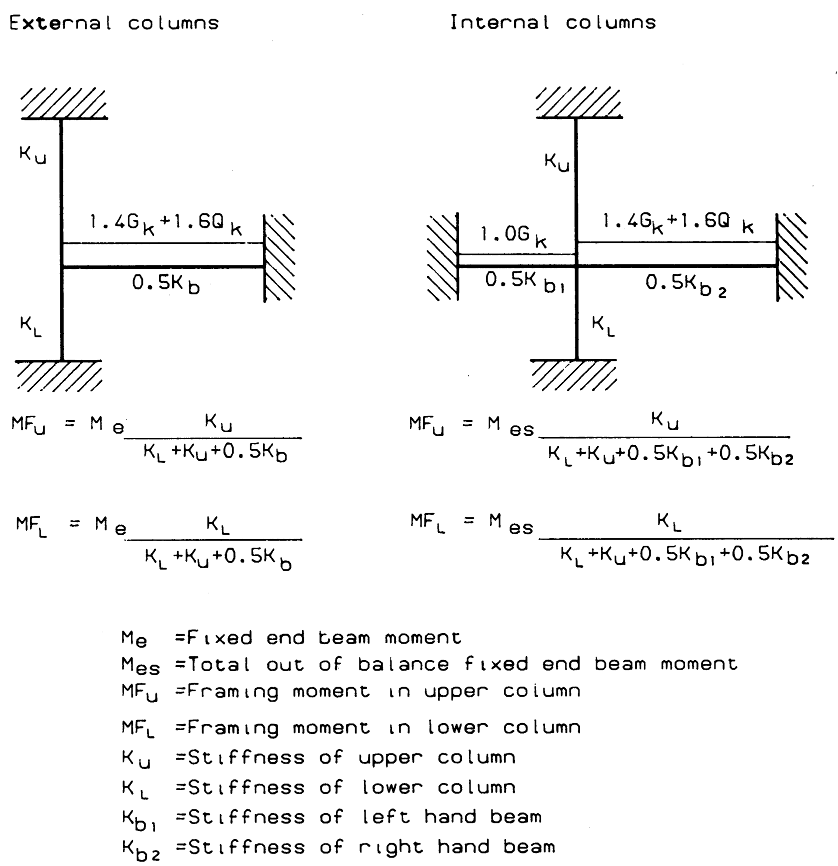


Fig. 5.17 Subframes for column moments – slender columns

numerically larger end moment from first order analysis

Note that algebraically, M_1 and M_2 will commonly have opposite signs.

5.5.4.3 Calculation of the ultimate deflection

The ultimate deflection in mm is given by

$$e_2 = f_{yk}\left(\frac{l_e}{d}\right)10^{-6}\ \text{mm}$$

where: f_{yk} = characteristic strength of the reinforcement in N/mm^2
l_e = effective length of the column in direction considered
d = effective depth of section in direction considered

The above value for e_2 may be reduced by multiplying by the factor K, which is obtained iteratively, given by:

$$K = \frac{(N_u - N)}{(N_u - N_{ba})} \leq 1$$

where: N_u = 'squash' load of column = $0.567 f_{ck}A_c + 0.87A_s f_{yk}$
N_{bal} = balanced load i.e. the axial load that when applied to a section maximizes the ultimate moment capacity. For a symmetrically reinforced section, $N_{bal} = 0.267 f_{ck}A_c$.

Alternatively, K may conservatively be taken as 1.

Table 5.24 Bending moments at around mid-height in slender columns

Moments about y-axis		Moments about z-axis
Case 1	$M_{yi} + N\,(e_{2z} + e_a)$	$M_{zi} + N\,e_a$
Case 2	$M_{yi} + Ne_a$	$M_{zi} + N(e_{2y} + e_a)$

Notes to Table 5.24
The moments for each case should be considered to act simultaneously, i.e. biaxially, except that separate checks about each axis are permitted when

$$\frac{h\,M_{zi}}{b\,M_{yi}} \geq 5 \text{ and } \frac{h\,M_{yi}}{Nh} \leq 0.$$

OR

$$\frac{h\,M_{zi}}{b\,M_{yi}} \leq 0.2 \text{ and } \frac{M_{yi}}{Nh} \leq 0.2$$

M_{yi} and M_{zi} are, respectively, the first-order moments at around mid-height of the columns about the y- and z-axes (see subclause 5.5.4.2 and Fig. 5.18).
e_{2y} and e_{2z}, are, respectively, the ultimate deflections calculated for the y- and z-directions (see subclause 5.5.4.3).
e_a is an eccentricity to allow for accidental 'out of plumb' of the column and is given by

$$e_a = \nu l_o/2 \text{ where } \nu = \text{greater of } \frac{1}{100\sqrt{l_t}} \text{ or } \frac{1}{200}$$

l_o = clear height of column
l_t = overall height of column from foundation to roof in metres
N is the design ultimate axial load.

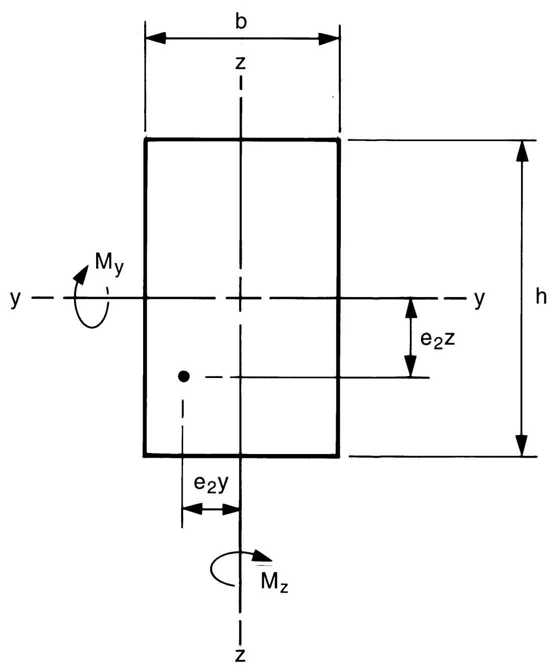


Fig. 5.18 Axis and eccentricities for columns

Initially the area of reinforcement should be obtained by assuming $K = 1$. The value of N_u and hence a new value of K should then be determined leading to a reduced area of reinforcement. This process can be repeated as necessary.

5.5.5 Section design

Sections subject to uniaxial bending should normally be designed using the charts in Appendix D.

When biaxial bending occurs, a symmetrically reinforced rectangular column section may be designed using the charts in Appendix D for the moments given in Table 5.25.

5.5.6 Reinforcement

Minimum area of reinforcement is given by the greater of:

$$\frac{0.18N}{f_{yk}} \quad \text{or} \quad 0.3\% \text{ of the gross cross-sectional area of the concrete.}$$

Longitudinal bars should not be less than 12mm diameter.

Maximum area of reinforcement, even at laps, should not exceed 8% of the gross cross-sectional area.

Columns should be provided with links whose diameter should not be less than one-quarter the diameter of the largest longitudinal bar nor less than 6mm.

Every corner bar should have a link passing round it. The maximum spacing of links should be the lesser of:

- 12 times the diameter of the smallest compression bar
- the least dimension of the column
- 300mm.

The maximum spacing should be reduced to 60% of the value given above:

(a) over a height equal to the larger dimension of the column above and below a beam or slab, and
(b) in the region of lapped joints if the longitudinal bar diameter exceeds 12mm.

A maximum of 5 bars in or close to each corner can be secured against buckling by any one set of links.

5.6 Walls

5.6.1 Introduction

This subsection describes the final design of reinforced concrete walls that may provide the lateral stability to reinforced concrete framed buildings. The design of interconnected shear walls is outside the scope of this *Manual*.

The general procedure to be adopted is as follows:

1. Check that walls providing lateral stability are continuous through the height of the building and that their shear centre coincides approximately with the line of the resultant of the applied horizontal loads in two orthogonal directions; if not, calculate the resulting twisting moments and check that they can be resisted
2. Check the slenderness of the walls within every storey height
3. Check that the section complies with the requirements for fire resistance
4. check that cover and concrete comply with durability requirements
5. calculate axial loads and moments according to clause 5.6.3
6. design section and reinforcement.

Table 5.25 Design moments for biaxial bending

$\frac{h\,M_z}{b\,M_y}$	y-axis	z-axis
$\geq$ 5*	M_y	M_z
$\leq$ 0.2*	M_y	M_z
All other cases		
$\frac{M_z h'}{M_y b'} \leq 1$	$M_y + \beta \frac{h'\,M_z}{b'}$	0
$\frac{M_z h^1}{M_y b^1} > 1$	0	$M_z + \beta \frac{b'\,M_y}{h^1}$

*Separate checks are made about the y- and z-axes where b' and h' are the effective depths (see Fig 5.19), and β is obtained from Table 5.26.

Table 5.26 Coefficients for biaxial bending

$\frac{N}{bhf_{ck}}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7
β	1.00	0.9	0.8	0.7	0.6	0.5	0.4	0.3

where N is the design ultimate axial load in Newtons and b and h are in mm (see Fig. 5.19).

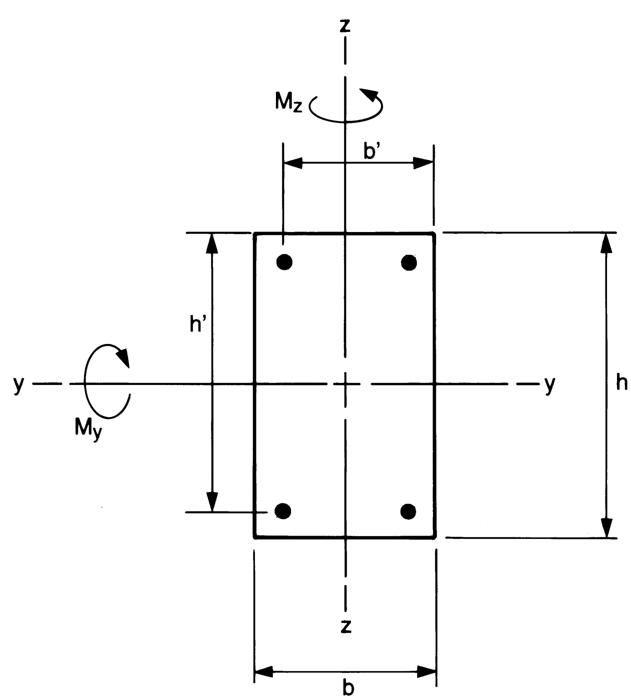


Fig. 5.19 Biaxial bending in columns

The thickness of the wall should not be less than 150mm, but to facilitate concreting 180mm is preferable.

5.6.2 Slenderness, fire resistance and durability

5.6.2.1 Slenderness

If the ratio of the effective height of a wall to its thickness exceeds $(l_e/h)_{crit}$, which is obtained from Fig. 5.16, then the wall is slender. Fig. 5.16 is obtained from

$$\left(l_e/h\right)_{crit} = 7.2l\left(2 - \frac{M_1}{M_2}\right)$$

M_2 is the numerically larger end moment on the wall and M_1 is the numerically smaller end moment.

(Note that for walls in most framed structures M_1 will have the opposite sign to M_2)

The effective height may be obtained by multiplying the clear height between floors by the factor obtained from Table 5.27.

5.6.2.2 Fire resistance

The minimum dimensions and covers should be obtained from Table 5.28.

5.6.2.3 Durability

The requirements for durability in any given environment are:

(a) an upper limit to the water/cement ratio
(b) a lower limit to the cement content
(c) a lower limit to the cover to the reinforcement
(d) good compaction
(e) adequate curing, and
(f) good detailing.

Values for (a), (b) and (c) that, in combination, will be adequate to ensure durability are given in Table 5.29 for various environments.

As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 5.29 gives, in addition, the concrete class strengths that have to be specified in the UK so that requirements (a) and (b) are satisfied.

For frost resistance the use of air entrainment of the concrete should be considered; however the effects of air entrainment on the concrete properties should be taken into account.

The concrete class strengths quoted in Table 5.29 will often require cement contents that are higher than those given in the Table. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

Table 5.27 Effective height factors for walls

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

Condition 1: Wall connected monolithically to slabs on either side that are at least as deep as the overall thickness of the wall. Where the wall is connected to a foundation, this should be designed to carry moment, in order to satisfy this condition.
Condition 2: Wall connected monolithically to slabs on either side that are shallower than the overall thickness of the wall, but not less than half the wall thickness.
Condition 3: Wall connected to members that do not provide more than nominal restraint to rotation.

Table 5.28 Fire resistance requirements for walls

Fire rating hours	Minimum dimension mm	Cover to vertical reinforcement, mm
1	150†	25
1.5	175† 150*	25
2	160*	25
3	200* 150**	25
4	240* 180**	25

† These walls may have less than 0.4% reinforcement.
* These walls to have between 0.4% and 1% reinforcement.
** These walls to have more than 1% reinforcement.

Table 5.29 Durability requirements for walls above ground

Conditions for exposure (for definitions see Appendix C)	Nominal cover to all reinforcement				
	mm	mm	mm	mm	mm
1. Dry environment	20	20	20	20	20
2. Humid environment					
(a) without frost	–	30	30	25	25
(b) with frost	–	–	30	25	25
3. Humid with frost and de-icing salts	–	–	35	30	30
4. Seawater environment					
(a) without frost	–	–	35	30	30
(b) with frost	–	–	35	30	30
The following classes may occur alone or in combination with those above:					
5. Aggressive chemical environment					
(a) slightly aggressive	–	–	30	25	25
(b) moderately aggressive	–	–	–	25	25
(c) highly aggressive*	–	–	–	–	40
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content, kg/m^3	260	280	300†	300	300
Concrete class strength	C25/30	C30/37	C35/45	C40/50	C45/55 or better

Notes to Table 5.29

1 Cover is expressed in terms of nominal values which have been obtained from the minimum values by allowing for a negative tolerance of 5mm.
2 The cover in mm to the main reinforcement should not be less than the bar diameter.

*Protective barrier to prevent direct contact with highly aggressive media should be provided.
†280kg/m^3 for exposure classes 2b and 5a.

Values for (a), (b) and (c) which, in combination, will be adequate to ensure durability are given in Table 5.2 for various environments.

As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 5.2 gives, in addition, the concrete class strengths that have to be specified in the UK so that requirements (a) and (b) are satisfied.

For frost resistance the use of air entrainment of the concrete should be considered; however the effects of air entrainment on the concrete properties should be taken into account.

5.6.3 Axial loads and moments

5.6.3.1 In-plane bending

The axial load on the wall should be calculated to obtain the most onerous conditions using the partial safety factors for loads in Table 3.1, and on the assumption that the beams and slabs transmitting forces into it are simply supported.

The horizontal forces should be calculated in accordance with the provision of clause 3.1(e), and the in-plane moments should be calculated for each lift of wall on the assumption that the walls act as cantilevers. The moment to be resisted by any one wall should be in the same ratio to the total cantilever moment as the ratio of its stiffness to the sum of the total stiffnesses of all the walls resisting the horizontal forces in that direction.

5.6.3.2 Bending at right-angles to the walls

The axial loads and in-plane moments should be determined as in subclause 5.6.3.1. In addition, the moments from horizontal forces acting at right-angles on the walls and from beams and slabs spanning monolithically on to the walls should be calculated assuming full continuity at the intersection with the floor slab.

5.6.3.3 Slender walls

Where the slenderness of a braced wall exceeds $(l_e/h)_{crit}$, the bending moments at right-angles to the wall should be the greater of M_2 or $M_i + N(e_2 + e_a)$. For definitions of symbols see clause 5.5.4.1.

5.6.4 Section design

The extreme fibre stresses due to in-plane moments and axial loads should be obtained from the following expression:

$$\text{extreme fibre stress, } f_t = \frac{N}{Lh} + \frac{6M}{hL^2} \text{ N/mm}^2$$

where N = design ultimate axial load in N
M = ultimate in-plane moment in Nmm
L = length of wall in mm
h = thickness of wall in mm.

This will result in a maximum ultimate compressive load and (possibly) a maximum ultimate tensile load per unit length of wall of $f_t h$ N/mm. This load should then be used together with any transverse moment to calculate the appropriate reinforcement area by treating each unit length of wall as a column. The wall should generally be designed on the assumption that the in-plane forces can act from either direction.

5.6.4.1 Walls not subject to significant bending at right-angles to the wall

Where walls are not likely to be subjected to significant transverse bending such as where they are internal walls supporting approximately symmetrical arrangements of slabs, the following simplified approaches may be adopted:

(a) *for compressive loads*

$$f_t\, h \leq 0.43\, f_{ckh} + 0.67\, f_{yk}\, A_{sc}$$

where f_{ck} = characteristic concrete cylinder strength in N/mm²
f_{yk} = characteristic strength of reinforcement in N/mm²
A_{sc} = area of reinforcement in mm² per mm length of wall.

(b) *for tensile loads*
The total area of tension reinforcement should be calculated from the expression:

$$A_s = \frac{0.5 f_t h L_t}{0.87 f_{yk}}$$

where L_t is the length of the wall in mm over which tension occurs. The area of reinforcement should be placed within $0.5L_t$ from the end of the wall where the maximum tension occurs.

5.6.4.2 Intersecting walls
Where two walls intersect to form a core the interface shear may be calculated from standard elastic methods. The interface shear stress v_{Sd} should be less or equal to v_{Rd3} where

$$v_{Rd3} = 2.5\, \tau_{Rd}\, hf + \frac{A_{sf}}{s_f} f_{yd}$$

5.6.5 Reinforcement

The minimum area of vertical reinforcement in the wall should be 0.4% of the gross cross-sectional area of the concrete and should be equally divided between the two faces of the wall.

The maximum area of vertical reinforcement should not exceed 4% of the gross cross-sectional area of the concrete.

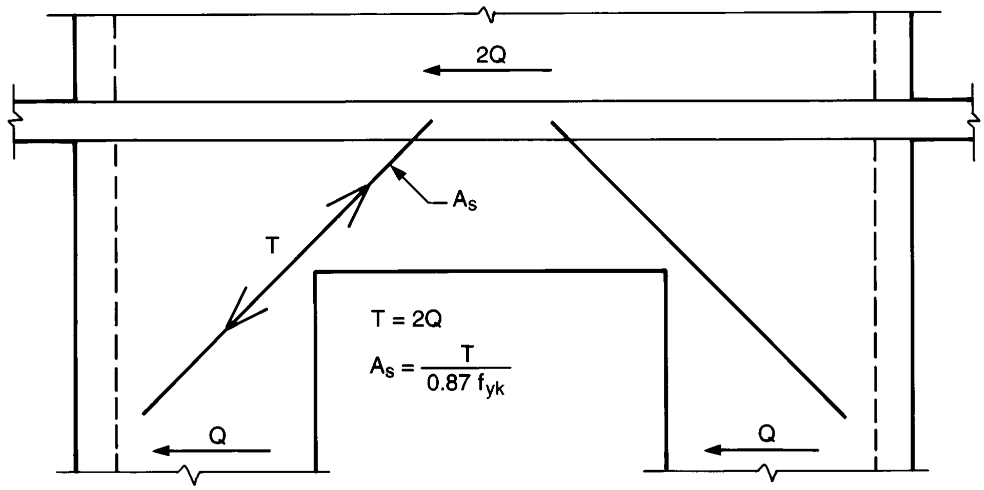
Horizontal reinforcement equal to not less than half the area of vertical reinforcement should be provided between the vertical reinforcement and the wall surface on both faces.

The spacing of the vertical bars should not exceed the lesser of 300mm or twice the wall thickness. The spacing of horizontal bars should not exceed 300mm and the diameter should not be less than one-quarter of the vertical bars.

If the vertical reinforcement exceeds 2% of the gross cross-sectional area of the concrete then links should be provided in accordance with clause 5.5.6.

5.6.6 Openings in shear and core walls

Door and service openings in shear walls introduce weaknesses that are not confined merely to the consequential reduction in cross-section. Stress concentrations are developed at the corners, and adequate reinforcement needs to be provided to cater for these concentrations. This reinforcement should take the form of diagonal bars positioned at the corners of the openings as illustrated in Fig. 5.20. The reinforcement will generally be adequate if it is designed to resist a tensile force equal to twice the shear force in the vertical components of the wall as shown, but should not be less than two 16mm diameter bars across each corner of the opening.



Other reinforcement has been omitted for clarity

Fig. 5.20 Reinforcement at openings in walls

5.7 Staircases

5.7.1 Introduction

The reinforced concrete slab supporting the stair flights and landings should be designed generally in accordance with the design information in subsection 5.2, except as indicated otherwise in this subsection.

When considering the dead loads for the flights, care should be taken to ensure that a sufficient allowance is made to cater for the weight of the treads and finishes as well as the increased loading on plan occasioned by the inclination of the waist.

5.7.2 Fire resistance, durability and concrete grades

The member sizes, reinforcement covers and concrete strength classes to provide fire resistance and durability should be obtained from Tables 5.1 and 5.2.

5.7.3 Bending moments and shear forces

Staircase slabs and landings should be designed to support the most unfavourable arrangements of design loads.

Where a span is adjacent to a cantilever of length exceeding one-third of the span of the slab, the case should be considered of maximum load on the cantilever and minimum load on the adjacent span.

Where staircases with open wells have two intersecting slabs at right-angles to each other, the loads on the areas common to both spans may be divided equally between the spans.

5.7.4 Effective spans

5.7.4.1 Stairs spanning between beams or walls

The effective span is the distance between centre-lines of supporting beams or walls.

5.7.4.2 Stairs spanning between landing slabs

The effective span is the distance between centre-lines of supporting landing slabs, or the distance between the edges of the supporting slabs plus 1.8m, whichever is the smaller.

5.7.4.3 Stairs with open wells

The effective span and loads on each span are as indicated in Figs. 5.21 and 5.22. The arrangement of flight supports shown in Figs. 5.21 and 5.22 is a special case where vertical support is provided at the ends of all flights. Where this condition does not occur, the stair flights should be designed for the full landing loads and the effective spans should be in accordance with subclauses 5.7.4.1 and 5.7.4.2.

5.7.5 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value from Table 5.30.

5.7.6 Section design

The design of the landing slabs and flights should be carried out in accordance with the methods described in clause 5.2.4.

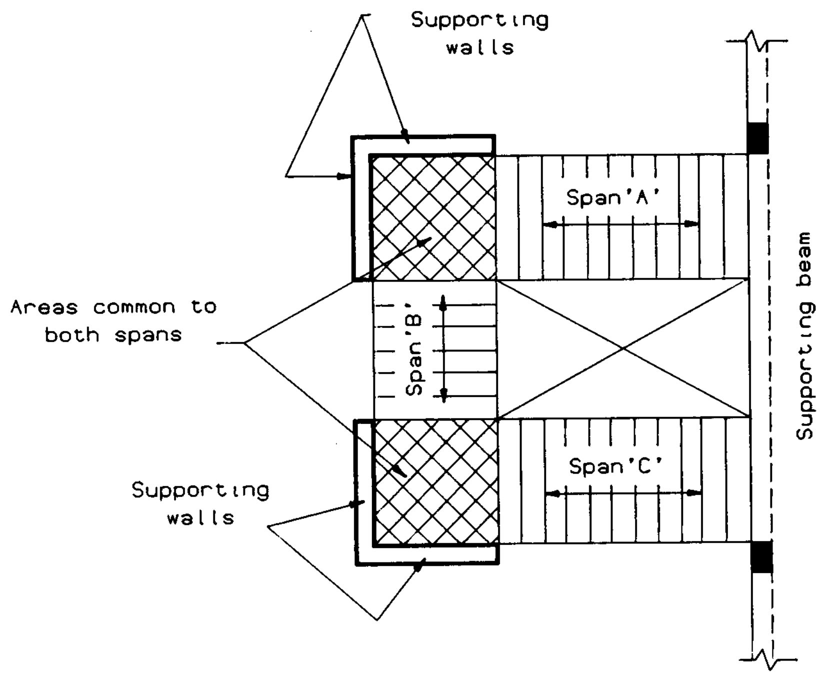


Fig. 5.21 Stairs with open walls

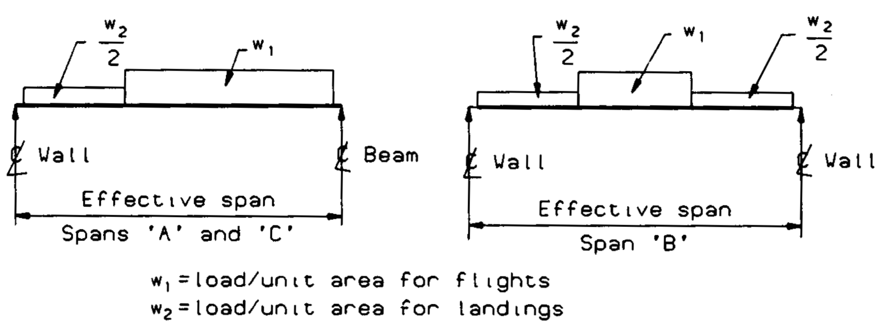


Fig. 5.22 Loading diagram

Table 5.30 Span/effective depth ratios for stairs

Location	A_s/bd $\geq$ 1.5%	A_s/bd = 0.5%	A_s/bd = 0.15%
Simply supported stairs	16	22	30
End span of continuous stairs	20	28	38
Interior span	22	30	33
Cantilever	6	9	12

Notes on Table 5.30

1. Values may be interpolated.
2. For flanged sections where the ratio of the flange to the rib width exceeds 3, the values should be multiplied by 0.8.
3. For spans exceeding 7m supporting partitions liable to be damaged by excessive deflections, the value should be multiplied by 7/span (in metres).
4. The above assumes f_{yk} = 460N/mm^2. If other values of f_{yk} are used then multiply the above by 460/f_{yk}.
5. A_s/bd is calculated at the location of maximum span moment.

The overall depth of the flights should be taken as the minimum waist thickness measured perpendicular to the soffit of the stair flight.

There is normally no need to calculate shear stresses in staircases supported on beams or walls. For stair landings, or beam strips supporting stair flights, the shear around columns should be checked in a similar manner to the shear around columns in solid flat slab construction.

5.8 Design of non-suspended ground floor slabs

Non-suspended ground slabs are generally designed on an empirical basis. Successful design requires attention to practical details. Thermal and moisture movements tend to produce the most critical stresses and cracking particularly when the concrete is still green. Careful planning of joints and provision of suitable reinforcement are essential. Useful guidance can be obtained from references 9 and 10.

The long strip method recommended in reference 9 is suitable for buildings where large areas of the ground floor are free of structural walls (e.g. warehouse floors). Where the layout of the building does not lend itself to long strip construction, the slab can be normally cast in bays not exceeding 50m^2 in area with the longer dimension of the bay limited to 10m. The slab thickness and reinforcement can be obtained from references 9 and 10.

5.9 Guidance for the design of basement walls

5.9.1 General

This subsection describes the design of basement walls that form part of a reinforced concrete structure.

The general procedure to be adopted is as follows:

1. Establish the requirements for the internal environment and follow the appropriate recommendations in the CIRIA guide on waterproof basements[11] and BS 8102[12]
2. Make the walls at least 250mm thick and ensure that they comply with the slenderness provisions in subclause 5.6.2.1
3. Check that walls comply with the requirements for fire resistance in subclause 5.6.2.2
4. Check that walls comply with the requirements for durability in subclause 5.6.2.3.

5.9.2 Bending moments and shear forces

The maximum values of the bending moments and shear forces at any section should be obtained by elastic analysis using the appropriate ultimate loads noted in subsection 3.1*. A minimum vertical surcharge of $10kN/m^2$ should be considered where vehicular traffic could impose lateral loading on the wall.

Construction method and sequence could affect the design and should be considered early in the design process[7].

Any design requirements for temporary works (e.g. propping, sequence of backfilling and construction of floors) should be stated on the drawings.

5.9.3 Section design

The sections should be designed in accordance with clause 5.6.4.

5.9.4 Foundation

The foundation or base slab should be designed as a strip footing in accordance with subsection 5.10 under the action of the axial load and bending moment from the wall. The base should be reinforced to ensure that the bending moments at the base of the wall can be transmitted safely to the base slab.

5.9.5 Reinforcement

Reinforcement should be provided in accordance with clause 5.6.5 except that the minimum horizontal reinforcement should not be less than 0.4% of the gross cross-sectional area of the wall.

5.10 Foundations

5.10.1 Introduction

The type of foundation, the sizes and the provisional formation levels depend on the results of ground investigation. Geotechnical design of foundations is beyond the scope of this *Manual,* and it has been assumed that the designer has access to the relevant expertise. Part 3 of EC2 dealing with the strength design of foundations will be published as an ENV document in due course. The main provisions of EC2, Part 1[1], still apply to the design of foundations.

It will be necessary to use the unfactored** dead, imposed and wind loads for the proportioning of foundations. The factored loads are, however, required for determining the depths of foundation members and for the design of any reinforcement. The general procedure to be adopted is as follows:

1. Evaluate results of ground investigation and decide whether spread or piled foundations are to be used
2. Examine existing and future levels around the structure, and taking into account the bearing strata and ground water levels, determine the provisional formation levels
3. Calculate the loads and moments, if any, on the individual foundations using the partial factors in Table 3.1 and the imposed loading reduction in BS 6399[5] where appropriate

* For pressures arising from an accidental head of water at ground level a partial factor of 1.15 may be used.

** Limit state design of foundations using partial factors has been introduced in EC7[23]. Until it is fully implemented in the UK, it is anticipated that the current practice for sizing the foundations will continue. The *Manual* therefore provides guidance for foundations based on current procedures for sizing. Appendix E sets out the likely steps, when EC7[23] is fully implemented.

4. Recalculate the loads and moments, if any, on the individual foundations without the partial factors in Table 3.1, using the imposed loading reduction in BS 6399[5] where appropriate; in many cases it may be sufficiently accurate to divide the factored loads and moments calculated in step 3 by 1.40
5. Calculate the plan areas of spread footings or the number of piles to be used to support each column or wall using the unfactored loads
6. Calculate the depth required for each foundation member and the reinforcement, if any, using the factored loads.

5.10.2 Durability and cover

All foundations other than those in aggressive soil conditions or exposed to frost are considered as being in exposure class 2a (see Appendix C). Cover to *all* reinforcement should be 50mm. For reinforced foundations the minimum cement content should be 280kg/m^3 and the maximum water/cement ratio 0.60.

The concrete class strength for reinforced bases and pile caps should therefore be not less than C30/37. For unreinforced bases C16/20 may be used, subject to a minimum cement content of 220kg/m^3. Where sulphates are present in significant concentrations in the soil and/or the groundwater, the recommendations of BRE Digest no. 363[13] should be followed.

5.10.3 Types of foundation

The loads and moments imposed on foundations may be supported by any one of the following types:

Pad footing
A square or rectangular footing supporting a single column.

Strip footing
A long footing supporting a continuous wall.

Combined footing
A footing supporting two or more columns.

Balanced footing
A footing supporting two columns, one of which lies at or near one end.

Raft
A foundation supporting a number of columns or loadbearing walls so as to transmit approximately uniform loading to the soil.

Pile cap
A foundation in the form of a pad, strip, combined or balanced footing in which the forces are transmitted to the soil through a system of piles.

5.10.4 Plan area of foundations

The plan area of the foundation should be proportioned on the following assumptions:

1. All forces are transmitted to the soil without exceeding the allowable bearing pressure
2. When the foundation is axially loaded, the reactions to design loads are uniformly distributed per unit area or per pile. A foundation may be treated as axially loaded if the eccentricity does not exceed 0.02 times the length in that direction
3. When the foundation is eccentrically loaded, the reactions vary linearly across the footing or across the pile system. Footings should generally be so proportioned that zero

pressure occurs only at one edge. It should be noted that eccentricity of load can arise in two ways: the columns being located eccentrically on the foundation; and/or the column transmitting a moment to the foundation. Both should be taken into account and combined to give the maximum eccentricity.

4. All parts of a footing in contact with the soil should be included in the assessment of contact pressure
5. It is preferable to maintain a reasonably similar pressure under all foundations to avoid significant differential settlement.

5.10.5 Design of spread footings

5.10.5.1 Axially loaded unreinforced pad footings

The ratio of the depth of the footing, h_f, to the projection from the column face a should be not less than that given in Table 5.31 for different values of unfactored pressures, σ in kN/m^2.

5.10.5.2 Axially loaded reinforced pad footings

The design of axially loaded reinforced pad footings is carried out in three stages:

1. Determine the depth of the footing from the ratios of the effective depth *d* to the projection from the column face *a*, given in Table 5.32 for different values of unfactored ground pressures σ. The effective depth *d* should not in any case be less than 300mm.

Table 5.31 Depth/projection ratios for unreinforced footings

	h_f/a				
Unfactored ground pressure σ, kN/m^2	C16/20	C20/25	C25/30	C30/37	C35/45
≤ 200	1.1	1.0	1.0	1.0	1.0
300	1.3	1.2	1.1	1.0	1.0
400	1.5	1.4	1.3	1.2	1.2

In no case should h_f/a be less than 1, nor should h_f be less than 300mm.

Table 5.32 Reinforcement percentages, depth/projection ratios and ground pressures for reinforced footings for f_{yk} = 460N/mm^2

	d/a									
σ kN/m^2	0.24	0.32	0.37	0.41	0.43	0.46	0.49	0.60	0.70	> 0.80
50	0.18	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13
100		0.20	0.15	0.13	0.13	0.13	0.13	0.13	0.13	0.13
150			0.23	0.19	0.17	0.15	0.13	0.13	0.13	0.13
200						0.20	0.17	0.13	0.13	0.13
250								0.15	0.13	0.13
300									0.17	0.13

The stippled areas indicated combinations of σ and *d/a* that should not be used.

2. Check that the face shear given by:

$$v_{Sd} = \frac{1000N}{2\left(c_x + c_y\right)d}$$

does not exceed $0.9\sqrt{f_{ck}}$, where N is the factored column load in kN, c_x and c_y are the column dimensions in mm and d is the effective depth in mm. If V_{Sd} exceeds this value increase the depth.
3. With the chosen depth (revised according to stage 2, if necessary) enter Table 5.32 and obtain the corresponding reinforcement percentage.

5.10.5.3 Eccentrically loaded footings

The design of eccentrically loaded footings proceeds as follows:

1. Determine initial depth of footing from Table 5.32 using maximum value of unfactored ground pressure
2. Check punching shear according to subclauses 5.2.3.4 and 5.2.4.2
3. Check face shear according to stage 2 in subclause 5.10.5.2 using V_{eff} from clause 5.2.3.4 in lieu of N
4. Increase the depth if necessary to avoid shear reinforcement
5. With the chosen depth (revised according to stage 4, if necessary) enter Table 5.32 to obtain the reinforcement percentage using maximum values of *unfactored* ground pressure.

5.10.6 Design of other footings

5.10.6.1 Strip footings

Strip footings should be designed as pad footings in the transverse direction and in the longitudinal direction at free ends or return corners. If reinforcement is required in the transverse direction it should also be provided in the longitudinal direction and should not be less than that obtained from the procedures in subclause 5.10.5.2.

5.10.6.2 Combined footings and balanced footings

Combined footings and balanced footings should be designed as reinforced pad footings except as extended or modified by the following requirements:

- Punching shear should additionally be checked for critical perimeters encompassing two or more closely spaced columns according to subclauses 5.2.3.4 and 5.2.4.2. Bending moments should additionally be checked at the point of zero shear between the two columns. Reinforcement should be provided in top and bottom faces and may be curtailed in accordance with the detailing rules in subsection 5.12.
- Where a balanced footing consists of two pad footings joined by a beam, the beam may be designed in accordance with subsection 5.4.
- Steps in the top or bottom surface may be introduced if necessary provided that they are taken into account in the design.

5.10.7 Reinforcement

Where reinforcement is required it should be provided in two generally orthogonal directions. The areas in each direction should not be less than $0.0015bh$ for reinforcement with $f_{yk} = 460\text{N/mm}^2$ where b and h are the breadth and overall depth in mm, respectively. All reinforcement should extend the full length of the footing.

If $l_x > 1.5\ (c_x + 3d)$, at least two-thirds of the reinforcement parallel to l_y should be concentrated in a band width $(c_x + 3d)$ centred at the column, where d is the effective depth, l_x

and c_x are the footing and column dimensions in the x-direction and l_y and c_y are the footing and column dimensions in the y-direction. The same applies in the transverse direction with suffixes x and y transposed.

Reinforcement should be anchored each side of all critical sections for bending. It is usually possible to achieve this with straight bars.

The spacing between centres of reinforcement should not exceed 200mm for bars with $f_{yk} = 460\text{N/mm}^2$. Reinforcement need normally not be provided in the side face nor in the top face, except for balanced or combined foundations.

Starter bars should terminate in a 90° bend tied to the bottom reinforcement, or in the case of an unreinforced footing spaced 75mm off the blinding.

5.10.8 Design of rafts

The design of a raft is analogous to that of an inverted flat slab (or beam-and-slab) system, with the important difference that the column loads are known but the distribution of ground bearing pressure is not. A distribution of ground bearing pressure has to be determined that:

(a) satisfies equilibrium by matching the column loads
(b) satisfies compatibility by matching the relative stiffness of raft and soil
(c) allows for the concentration of loads by slabs or beams continuous over supports, and
(d) stays within the allowable bearing pressure determined from geotechnical considerations of strength and settlement.

Provided that such a distribution can be determined or estimated realistically by simple methods, design as a flat slab or beam-and-slab may be carried out. In some cases, however, a realistic distribution cannot be determined by simple methods, and a more complex analysis is required.

5.10.9 Design of pile caps

The design of pile caps should be carried out in accordance with the following general principles:

(a) The spacing of piles should generally be three times the pile diameter
(b) The piles should be grouped symmetrically under the loads
(c) The load carried by each pile is equal to N/(no. of piles). When a moment is transmitted to the pile cap the loads on the piles should be calculated to satisfy equilibrium
(d) Pile caps should extend at least 150mm beyond the theoretical circumference of the piles
(e) For pile caps supported on one or two piles only, a moment arising from a column eccentricity of 75mm should be resisted either by ground beams or by the piles.

The general procedure to be adopted is as follows:

(f) Using the unfactored loads and moments calculate the number of piles required under each column
(g) Proportion the pile caps on plan in accordance with the above general principles. Typical arrangements are shown in Fig. 5.23 where s is the spacing of the piles
(h) Determine the initial depth of the pile cap as equal to the horizontal distance from the centreline of the column to the centreline of the pile furthest away
(i) Check the face shear as for reinforced pad footings, using factored loads, and increase the depth if necessary
(j) Calculate the bending moments and the reinforcement in the pile caps using the factored loads.

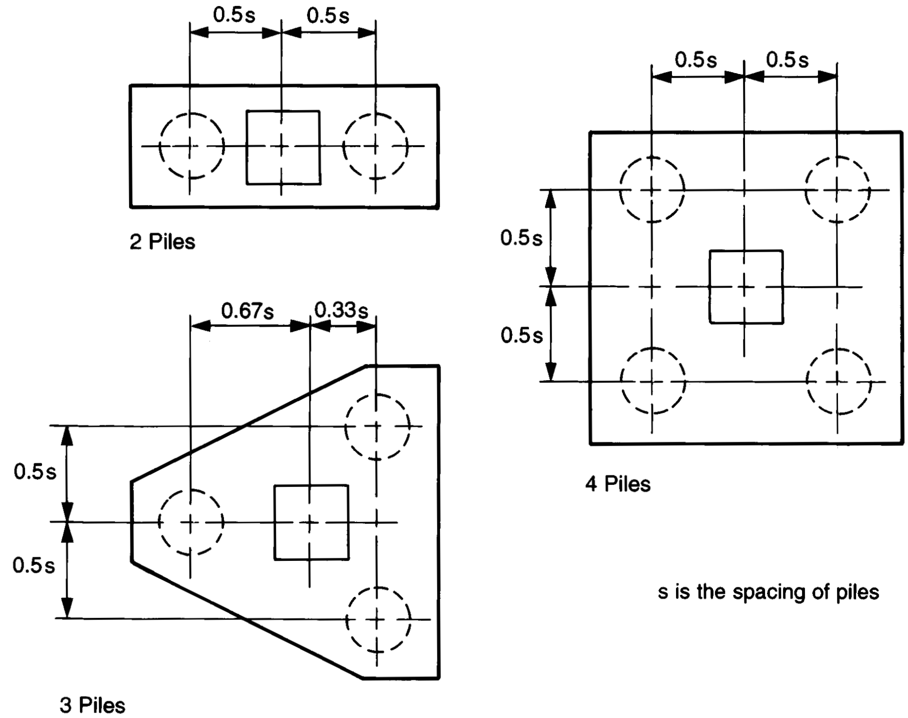


Fig. 5.23 Typical arrangements of pile caps

5.10.10 Reinforcement in pile caps

All pile caps should generally be reinforced in two orthogonal directions on the top and bottom faces. Where bars with f_{yk} = 460N/mm^2 are used the amount of reinforcement should not be less than 0.0015bh in each direction.

The bending moments and the reinforcement should be calculated on critical sections at the column faces, assuming that the pile loads are concentrated at the pile centres. This reinforcement should be continued past the piles and bent up vertically to provide full anchorage past the centreline of each pile.

In addition, fully lapped, circumferential horizontal reinforcement consisting of bars not less than 12mm in diameter at a spacing not more than 250mm, should be provided.

5.11 Robustness

5.11.1 General

If the recommendations of this *Manual* have been followed, a robust structure will have been designed, subject to the reinforcement being properly detailed.

However, in order to demonstrate that the requirements for robustness have been met, the reinforcement already designed should be checked to ensure that it is sufficient to act as:

(a) peripheral ties
(b) internal ties
(c) external column or wall ties
(d) vertical ties.

The arrangement of these (notional) ties and the forces they should be capable of resisting are stated in clause 5.11.2.

Reinforcement considered as part of the above ties should have full tension laps throughout so as to be effectively continuous. For the purpose of checking the adequacy of the ties, this reinforcement may be assumed to be acting at its characteristic strength when resisting the forces stated below, and no other forces need to be considered in this check. Horizontal ties, i.e. (a), (b) and (c) above, should be present at each floor level and at roof level.

5.11.2 Tie forces and arrangements

Forces to be resisted by horizontal ties are derived from a 'tie force coefficient'

$F_t = (20 + 4n)$ kN for $n \leq 10$, or
$F_t = 60$kN for $n > 10$

where n is the number of storeys.

(a) *Peripheral ties*
Peripheral ties should be located in zones within 1.2m from the edges; they should be capable of resisting a tie force of $1.0F_t$ and should be fully anchored at all corners.

(b) *Internal ties*
Internal ties should be present in two directions approximately at right-angles to each other. Provided that the floor spans do not exceed 5m and the total characteristic dead and imposed load does not exceed 7.5kN/m^2, the ties in each direction should be capable of resisting a tie force of $1.0F_t$ kN per metre width. If the spans exceed 5m and/or the total load exceeds 7.5kN/m^2, the tie force to be resisted should be increased *pro rata*. Internal ties may be spread evenly in the slabs or may be concentrated at beams or other locations, spaced at not more than 1.5 times the span. They should be anchored to the peripheral ties at each end.

In spine or crosswall construction the length of the loadbearing wall between lateral supports should be considered in lieu of the spans when determining the force to be resisted by the internal ties in the direction of the wall.

(c) *External column or wall ties*
External columns and loadbearing walls should be tied to the floor structure. Corner columns should be tied in both directions. Provided that the clear floor-to-ceiling height does not exceed 2.5m, the tie force for each column and for each metre length of wall is $1.0F_t$. For floor-to-ceiling heights greater than 2.5m, the tie forces should be increased *pro rata*, up to a maximum of $2.0F_t$. The tie force should in no case be assumed less than 3% of the total design ultimate load carried by the column or wall. This reinforcement should be fully anchored in both vertical and horizontal elements.

(d) *Vertical ties*
Vertical ties should be present in each column and loadbearing wall. They should be capable of resisting a tensile force equal to the maximum total design ultimate load received by the column or wall from any one floor or roof.

Where effectively continuous vertical ties cannot be provided (e.g. in some precast construction), the effect of each column or loadbearing wall being removed in turn should be considered, and alternative load paths should be provided if necessary. In this context:

$\gamma_G = 1.0$
$\gamma_Q = 0.33$
$\gamma_c = 1.3$, and
$\gamma_s = 1.0$.

5.12 Detailing

5.12.1 General

Certain aspects of reinforcement detailing may influence the design. The most common of these are outlined in the following subsections. Additional rules for high-bond bars exceeding 32mm in diameter are given in clause 5.12.5.

5.12.2 Bond conditions

The bond conditions affect the anchorage and lap lengths. Good and poor bond conditions are illustrated in Fig. 5.24.

5.12.3 Anchorage and lap lengths

Anchorage and lap lengths should be obtained from Table 5.33 for high bond bars, Table 5.34 for plain bars, Table 5.35 for welded mesh fabric made with high bond bars and Table 5.36 for welded mesh made with plain bars.

The clear spacing between two lapped bars should be in accordance with Fig. 5.26 (see also clause 5.12.5).

5.12.4 Transverse reinforcement

(a) *Anchorage zones*

Tranverse reinforcement should be provided for all anchorages in compression and, in the absence of transverse compression caused by support reactions, for anchorage in tension.

The minimum total area of transverse reinforcement required within the anchorage zone is 25% of the area of the anchored bar. Transverse reinforcement should be evenly distributed in tension anchorages and concentrated at the ends of compression anchorages.

(b) *Laps*

No special transverse reinforcement is required if the diameter of bars lapped is less than 16mm or fewer than 20% of the bars in the section are lapped. When required the area of total transverse reinforcement, A_{St}, should be equal to or greater than the area of the smaller spliced bar and should be placed as shown in Fig 5.27.

(c) *Anchorage of links*

Links should be anchored using one of the methods shown in Fig. 5.28.

5.12.5 Additional rules for high-bond bars exceeding 32mm in diameter

Bars exceeding 32mm in diameter should be used only in elements with a depth not less than 15 times the bar diameter. Such bars should not be anchored in tension zones. Lapped joints in tension or compression are not permitted, and mechanical devices (e.g. couplers) should be used instead.

The anchorage bond stress for such bars should be reduced by a factor (132 – ϕ)/100. This is allowed for in Tables 5.33 to 5.36.

In the absence of transverse compression transverse reinforcement is required in the anchorage zone (in addition to any shear reinforcement) as shown in Fig. 5.29. The reinforcement should not be less than the following:

- In the direction parallel to the lower face: $A_{st} = n_1 \times 0.25A_s$
- In the direction perpendicular to the lower face: $A_{st} = n_2 \times 0.25A_s$

where n_1 is the number of layers with bars anchored at the same point in the member and n_2 is the number of bars anchored in each layer.

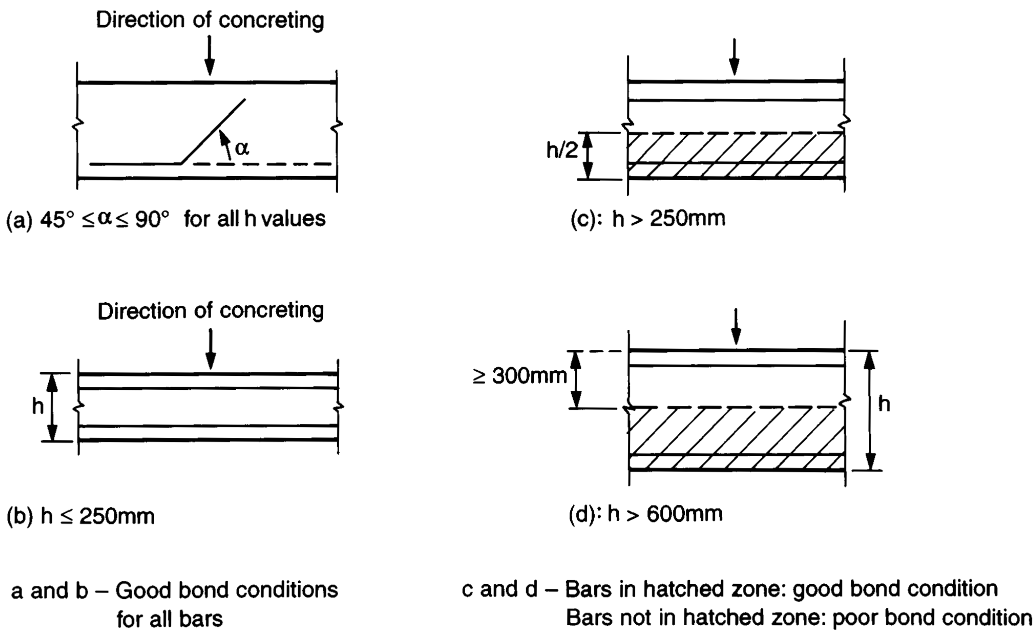


Fig. 5.24 Definition of bond conditions

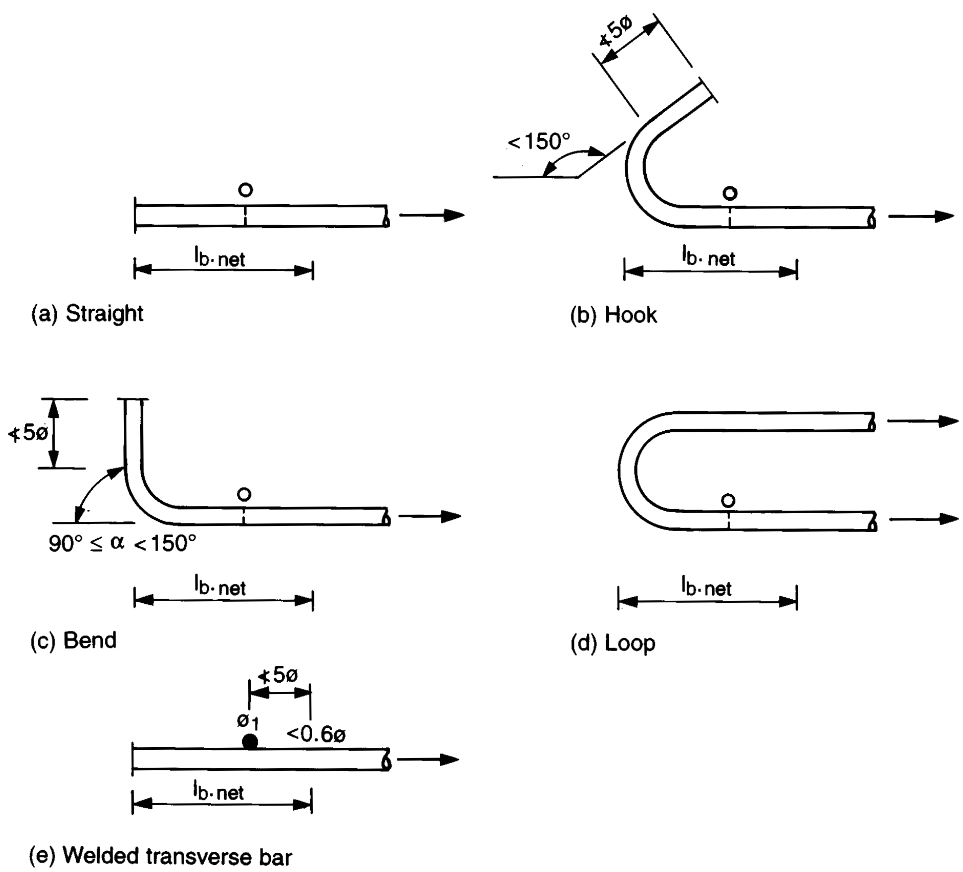


Fig. 5.25 Required anchorage length

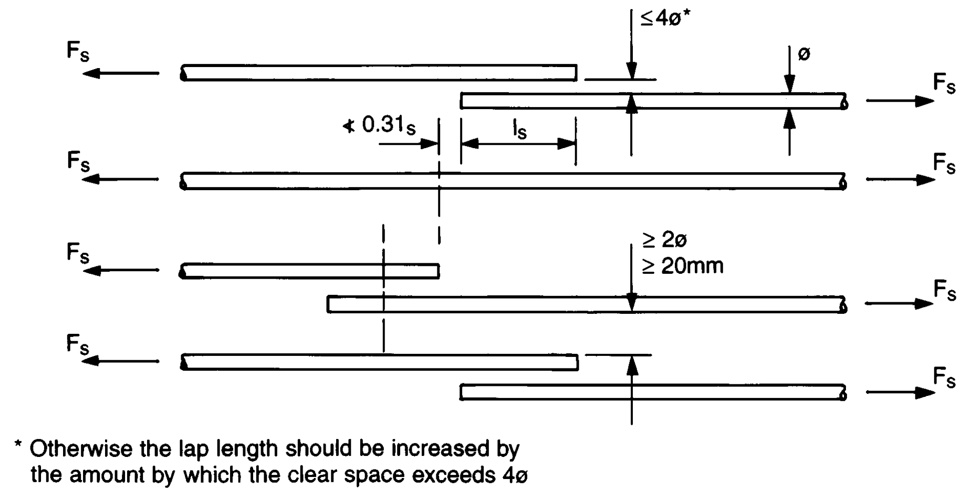


Fig. 5.26 Adjacent laps

Table 5.33 Anchorage and lap lengths as multiples of bar diameter – high bond bars f_{yk} = 460N/mm²

Concrete strength, f_{ck}, N/mm²	20	25	30	35	40
Anchorage – straight bars compression and tension l_{bnet}	44	37	34	30	27
Anchorage – curved bars [5] tension l_{bnet}	31	26	24	21	19
Laps – compression – tension [6] Laps – tension [7]	44 62	37 52	34 48	30 42	27 38
Laps – tension [8]	88	74	68	60	54

Notes to Table 5.33

General

1. For bars with f_{yk} other than 460N/mm², the Table values should be multiplied by (f_{yk}/460).
2. The values in the Table apply to good bond conditions (see Fig. 5.24) where the bar diameter ≤ 32mm.
3. For poor bond conditions (see Fig. 5.24) the Table values should be divided by 0.7.
4. For bar diameter > 32mm the values should be divided by [(132 – ϕ)/100], where ϕ is the bar diameter in mm for anchorage in compression zones.

Specific conditions

5. See Figs. 5.25(b), (c) and (d). In the anchorage region cover perpendicular to the plan of curvature should be at least 3 ϕ.
6. Percentage of bars lapped at the section < 30% and clear spacing between bars ≥ 10 ϕ and side cover to outer bar ≥ 5 ϕ.
7. Percentage of bars lapped at the section < 30% and clear spacing between bars < 10 ϕ or side cover to the outer bar < 5 ϕ.
8. Percentage of bars lapped at the section <30 % and clear spacing between bars < 10 ϕ or side cover to the outer bar < 5 ϕ.

Table 5.34 Anchorage and lap lengths as multiples of bar diameter – plain bars f_{yk} = 250N/mm²

Concrete cylinger strength f_{ck}, N/mm²	20	25	30	35	40
Anchorage – straight bars compression and tension (not applicable to bar diameter > 8mm) l_{bnet}	50	46	41	39	37
Anchorage – curved bars [4] tension l_{bnet}	35	32	29	27	26
Laps – compression – tension [5] Laps – tension [6]	50 70	46 64	41 60	39 56	37 52
Laps – tension [7]	100	92	84	78	74

Notes to Table 5.34

1. For bars with f_{yk} other than 250N/mm², the Table values should be multiplied by (f_{yk}/250).
2. The values in the Table apply to good bond conditions (see Fig. 5.24).
3. For poor bond conditions (see Fig. 5.24) the Table values should be divided by 0.7.

Specific conditions

4. See Fig. 5.25(b) and (d) and Fig. 5.25(c) where the diameter is less than 8mm. In the anchorage region cover perpendicular to the plane of curvature should be at least 3 ϕ.
5. Percentage of bars lapped at the section < 30% and clear spacing between bars ≥ 10 ϕ and side cover to the outer bar ≥ 5 ϕ.
6. Percentage of bars lapped at the section > 30% or clear spacing between bars < 10 ϕ or side covers to the outer bar < 5 ϕ.
7. Percentage of bars lapped at the section > 30% and clear spacing between bars < 10 ϕ or side cover to the outer bar < 5 ϕ.

Table 5.35 Anchorage and lap lengths as multiples of bar diameter-welded mesh fabric made with high bond bars f_{yk} = 460N/mm²

Concrete cylinder strength f_{ck}, N/mm²	20	25	30	35	40
Basic anchorage l_{bnet} and lap lengths	44	37	34	30	27

Notes to Table 5.35

1. For bars with f_{yk} other than 460N/mm² the Table figures should be multiplied by (f_{yk}/460).
2. Where welded transverse bars are present in the anchorage zone the Table values for anchorage may be multiplied by 0.7.
3. The values given in the Table apply to good bond conditions (see Fig. 5.24) and to bar diameters ≤ 32mm.
4. For poor bond conditions (see Fig. 5.24) the values in the Table should be divided by 0.7.
5. For bar diameters > 32mm, the values should be divided by [(132 – ϕ)/100], where ϕ is the diameter of the bar in mm.
6. The Table values should be multiplied by the following factors corresponding to the different values of (A_s/s) values. A_s is the area of the main reinforcement (mm²) and s is the spacing of the bars forming the main reinforcement (m).

A_s/s	≤ 480	680	880	1080	≥ 1 280
Multiplier	1.00	1.25	1.50	1.75	2.00

Table 5.36 Anchorage and lap lengths as multiples of bar diameter – welded mesh fabric made with plain bars f_{yk} = 460N/mm²

Concrete cylinder strength f_{ck} N/mm²	20	25	30	35	40
Straight anchorage; compression	26	24	22	20	19
Straight anchorage: tension	33	30	27	25	23
Laps: compression tension*	33	30	27	25	23
Laps: tension†	46	42	38	34	33
Laps: tension†	66	60	54	49	47

The values in the Table apply to (a) good bond conditions and (b) fabric defined in BS 4449[24], BS 4461[25] and BS 4482[26].
For poor bond conditions the Table should be divided by 0.7.
Bond stresses are based on BS 8110: 1985[16], but modified to allow for γ_c = 1.5.
The values apply provided the fabric is welded in a shear-resistant manner complying with BS 4483[14], and the number of welded intersections within the anchorage is at least equal to 4 × (A_{sreq}/A_{sprov}). If the latter condition is not satisfied, values appropriate to the individual bars/wires should be used.
*The bars lapped at the section < 30%, clear spacing between bars ≥ 10 ϕ and side cover to the outer bar ≥ 5 ϕ.
†The bars lapped at the section > 30%, or clear spacing between bars > 10 ϕ or side cover to the outer bar > 5 ϕ.
+The bars lapped at the section > 30% and clear spacing between bars > 10 ϕ or side cover to the outer bar > 5 ϕ.

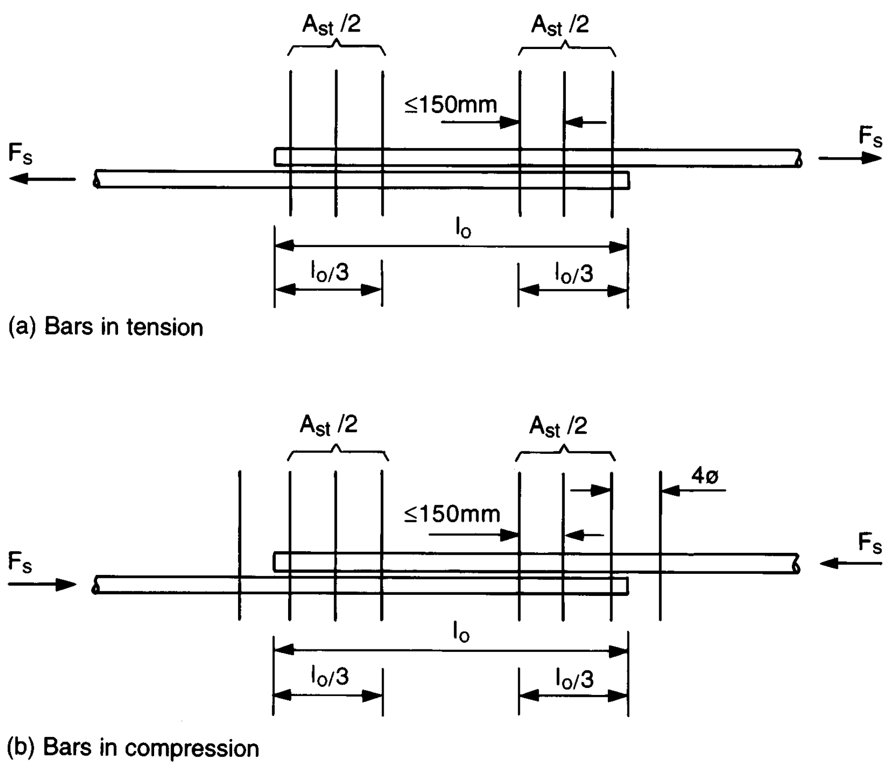


Fig. 5.27 Transverse reinforcement for lapped splices

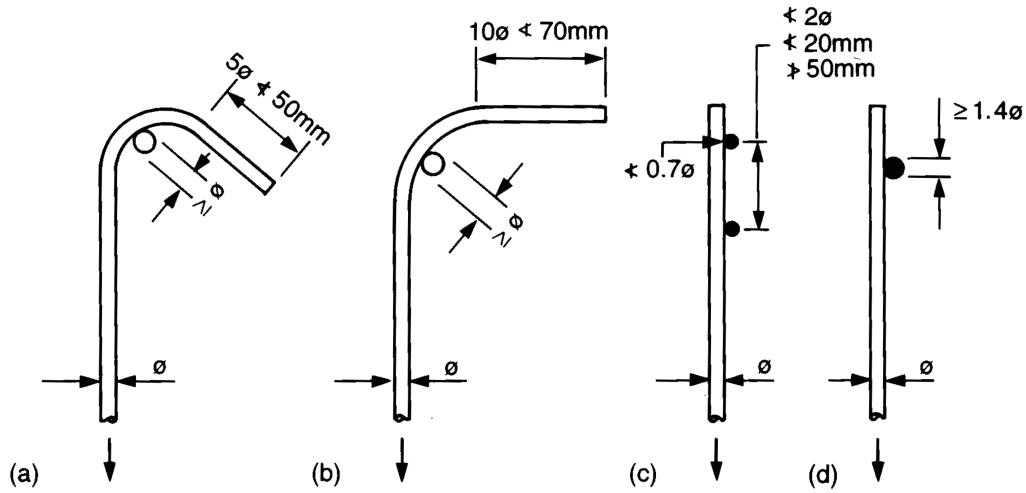


Fig. 5.28 Anchorage of links

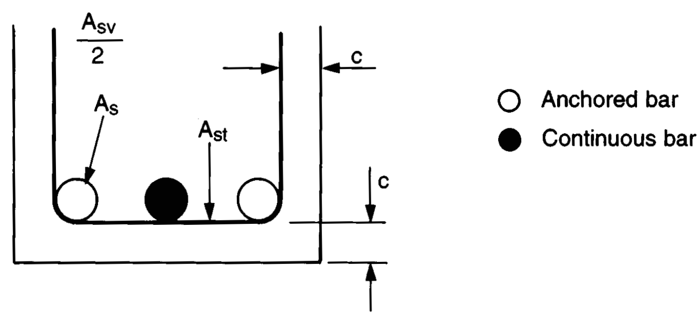


Fig. 5.29 Additional reinforcement in an anchorage zone where the bar diameter is greater than 32mm and there is no transverse compression

5.12.6 Curtailment of bars in flexural members

(a) When a bar is curtailed in a flexural member, it should be anchored beyond the point when it is no longer required, for a length of $l_{b,net}$ or d, whichever is the greater.

In determining the location where a bar is no longer required, the force in the bar should be calculated taking into account:

(i) the bending moment, and
(ii) the effect of a truss model for resisting shear.

This may be assessed by shifting the bending moment diagram in the direction of reducing moment by an amount a_1, where $a_1 = 0.45d \cot \theta$ for beams and $1.0d$ for slabs, where θ is the angle of the concrete truss assumed in shear design ($\cot \theta = 1$ for standard method).

A practical method is:

(i) determine where the bar can be curtailed based on bending moment alone, and
(ii) anchor this bar beyond this location for a distance
(iii) $l_{bnet} + a_1$.

This procedure is illustrated in Fig. 5.30.

(b) At simply supported ends, the bar should be anchored beyond the line of contact between the member and its support by

$0.67\ l_{b,net}$ at a direct support and
$1.00\ l_{b,net}$ at an indirect support.

A direct support is one where the reaction provides compression across the bar being anchored. All other supports are considered indirect. This requirement is illustrated in Fig. 5.31.

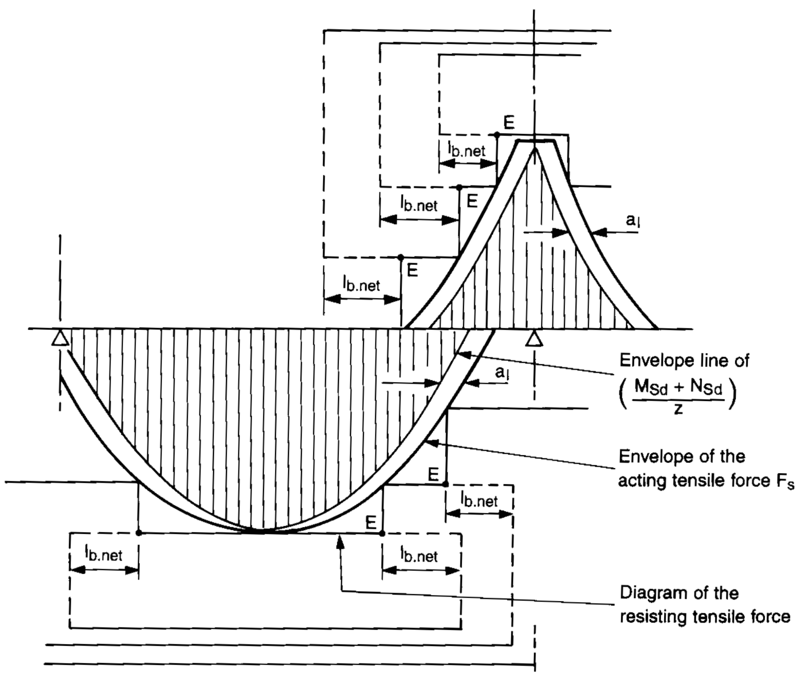


Fig. 5.30 Envelope line for the design of flexural members – anchorage lengths

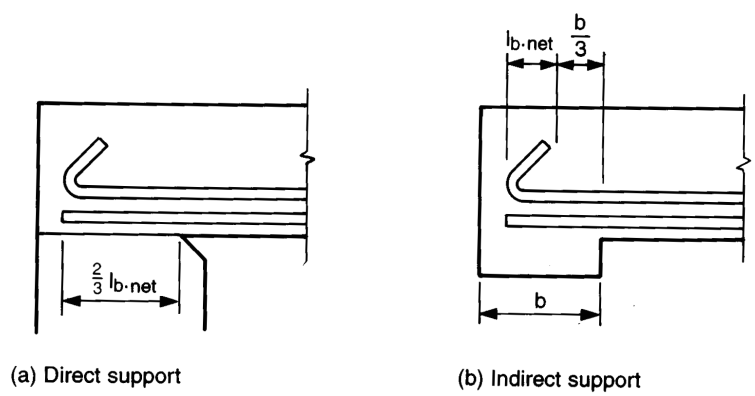


Fig. 5.31 Anchorage at end supports: (a) direct support (b) indirect support

5.12.7 Corbels and nibs

Depending on the relative values of the shear span a_c (see Fig. 5.32) and overall depth h_c (see Fig. 5.32), these members should be designed using strut and tie models when $0.4h_c \leq a_c \leq h_c$ or as cantilevers when $a_c > h_c$. Unless special provision is made to limit the horizontal forces on the support, the corbel (or nib) should be designed for the combined effects of the vertical force V and a horizontal force of $0.2V$.

The area of tie reinforcement, A_s, required could be obtained using the chart in Fig 5.33. In addition to this reinforcement horizontal stirrups with a total area of $0.4A_s$ should be provided as shown in Fig. 5.32.

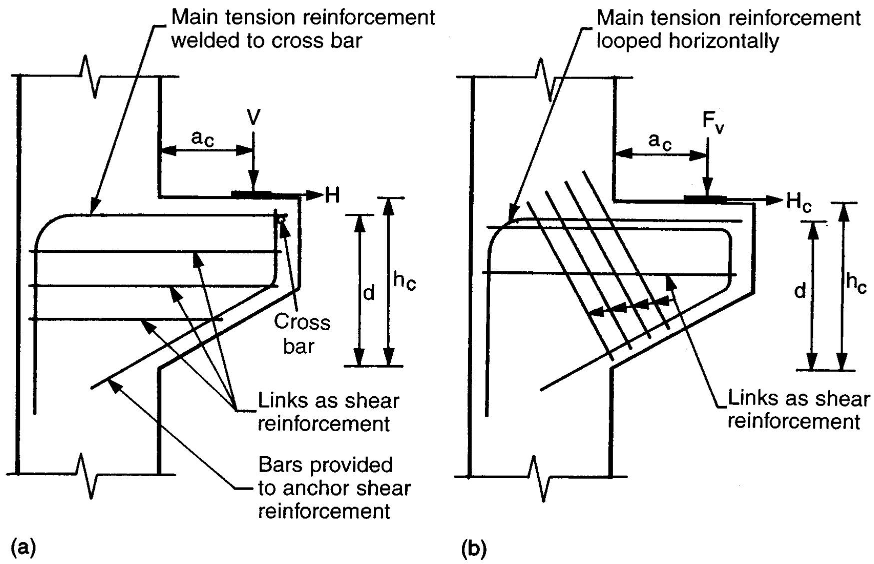


Fig. 5.32 Corbels and nibs

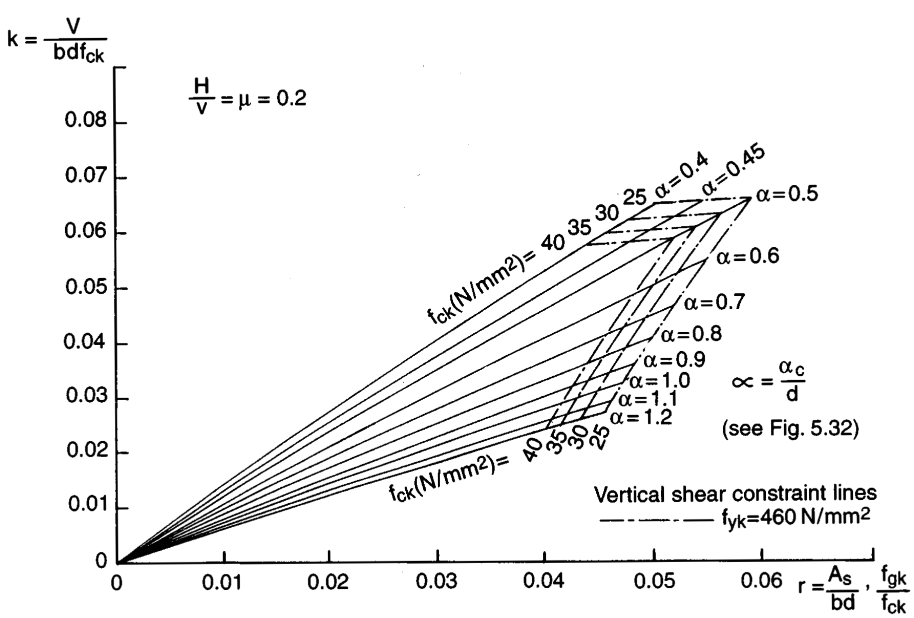


Fig. 5.33 Design chart for corbels

6 Design principles – prestressed concrete

6.1 Introduction

This section outlines the general principles which apply to both the initial and final design of prestressed concrete members and sets out the design parameters that govern all design stages. The general recommendations given in Section 2 are applicable to all concrete building structures and are not repeated here.

This *Manual* can be used for the design of both precast and *in situ* prestressed concrete members, other than those formed from precast segments which are subsequently stressed together (precast segmental construction).

Prestressed concrete is different from ordinary (non-prestressed) reinforced concrete because the tendons apply loads to the concrete as a result of their prestress force, whilst in reinforced concrete the stresses in the reinforcement result from the loads applied to the structure. A proportion of the external loads is therefore resisted by applying a load in the opposite sense through the prestressing whilst the balance has to be resisted by ordinary reinforcement.

Prestressing tendons may be internal, i.e. within the concrete either bonded to the concrete or unbonded, or external, i.e. outside the concrete but inside the envelope of the member, see Fig. 6.1. This *Manual* deals only with prestressed concrete members with internal tendons.

Prestressed members can be either **pretensioned**, i.e. the tendons are stressed **before** the concrete is cast around them and the force transferred to the concrete when it has obtained sufficient strength, or **post-tensioned**, i.e. ducts are cast into the concrete through which the tendons are threaded and then stressed **after** the concrete has gained sufficient strength. This *Manual* deals with both **pre-** and **post-tensioned** members. Table 6.1 compares the advantages and disadvantages of pre- and post-tensioning,

Precast members will generally be pretensioned with the tendons bonded to the concrete whilst *in situ* members will be post-tensioned with the tendons either bonded or unbonded. Table 6.2 lists factors affecting the choice of bonded or unbonded tendons.

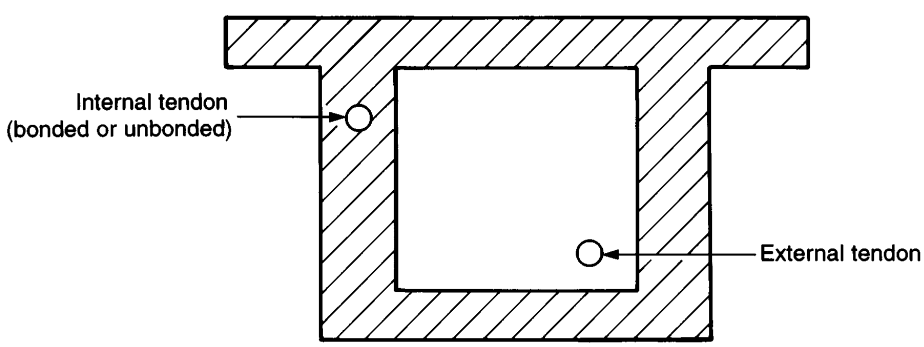


Fig. 6.1 Internal & external tendons

Table 6.1 Advantages and disadvantages of pre- and post-tensioning

Type of construction	Advantages	Disadvantages
Pretensioned	• no need for anchorages • tendons protected by concrete without the need for grouting or other protection • prestress is generally better distributed in transmission zones	• heavy stressing bed required • more difficult to incorporate deflected tendons
Post-tensioned	• no external stressing bed required • more flexibility in tendon layout and profile • draped tendons can be used	• tendons require a protective system • large concentrated forces in end blocks

When a concrete member is prestressed it will deflect and shorten. If the tendon profile is such that the deflected shape of the member is compatible with the restraints acting on the member, the profile is said to be **concordant**. This will always be the case for a statically determinate member. However, tendon profiles in statically indeterminate members will not generally be **concordant**, see Fig. 6.2.

When the tendon profile is **concordant**, the forces induced at any point in the member by the action of prestressing are an axial compression equal to the prestressing force and a moment equal to the product of the prestressing force and its eccentricity relative to the neutral axis of the member. These are the primary prestressing forces.

When the tendon profile is **non-concordant**, additional forces and moments will be induced in the member during prestressing by the restraints acting on it, see Fig. 6.2. These are known as the **secondary** or **parasitic effects**.

Other common nomenclature associated with prestressed concrete is defined in Fig. 6.3.

Concrete is stronger in compression than in tension. Prestress is introduced to pre-compress the areas of concrete which would otherwise be in tension under service loads. The concrete section is therefore stronger and behaves more as an homogeneous section, allowing elastic methods of analysis to be used, although the concrete compressive stresses can be high. When the tendons are bonded to the concrete, their high ultimate strength can be mobilized which generally means that the ultimate flexural capacity of a prestressed concrete member is much greater than the applied ultimate design moment. Therefore, limiting the maximum compressive stresses and crack widths under service loads is generally critical in the design of prestressed concrete members, although this may not be the case for members with unbonded tendons. Therefore, flexural design is normally carried out at the serviceability limit state and then checked at the ultimate limit state. Shear design is carried out at the ultimate limit state.

6.2 Loading

The loads to be used in calculations are:

(a) Characteristic dead load, G_k: the weight of the structure complete with finishes, fixtures and fixed partitions (BS 648[4])
(b) Characteristic imposed load, Q_k (BS 6399, Part 1 and Part 3[5])
(c) Characteristic wind load, W_k (90% of the load derived from CP3, Chapter V, Part 2[6])*
(d) Prestressing loads, P (see Section 8)

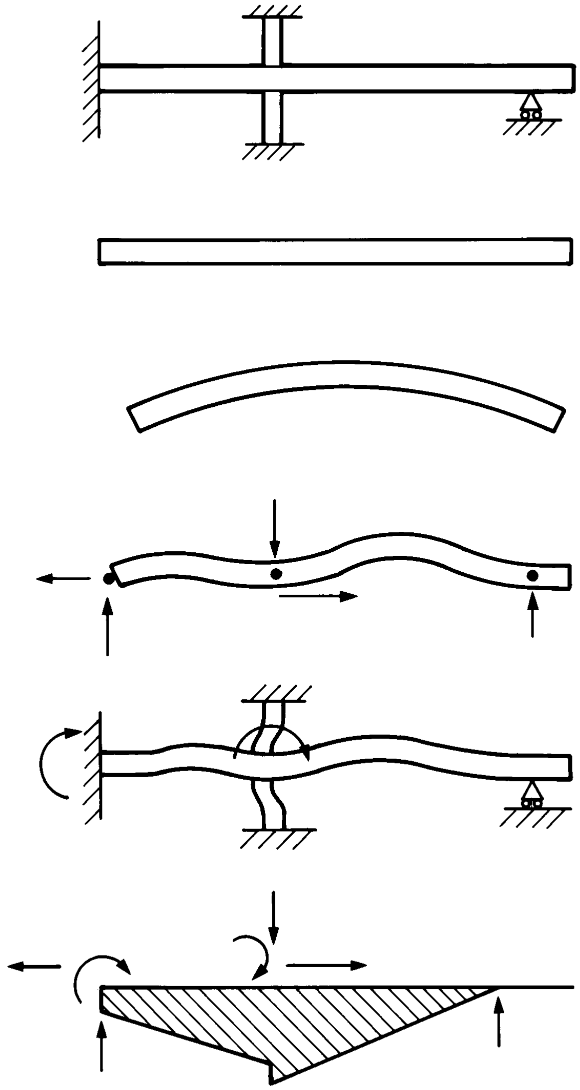


Fig. 6.2 Prestressed concrete member as part of statically indeterminate structure

Table 6.2 Advantages and disadvantages of bonded and unbonded construction

Type of construction	Advantages	Disadvantages
Bonded	• tendons are more effective at ULS • does not depend on the anchorage after grouting • localises the effect of damage	• tendon cannot be inspected or replaced • tendons cannot be re-stressed once grouted
Unbonded	• tendons can be removed for inspection and are replaceable if corroded • reduced friction losses • generally faster construction • tendons can be re-stressed • thinner webs	• less efficient at ULS • relies on the integrity of the anchorages and deviators • effects of any damage are more widespread • less efficient in controlling cracking

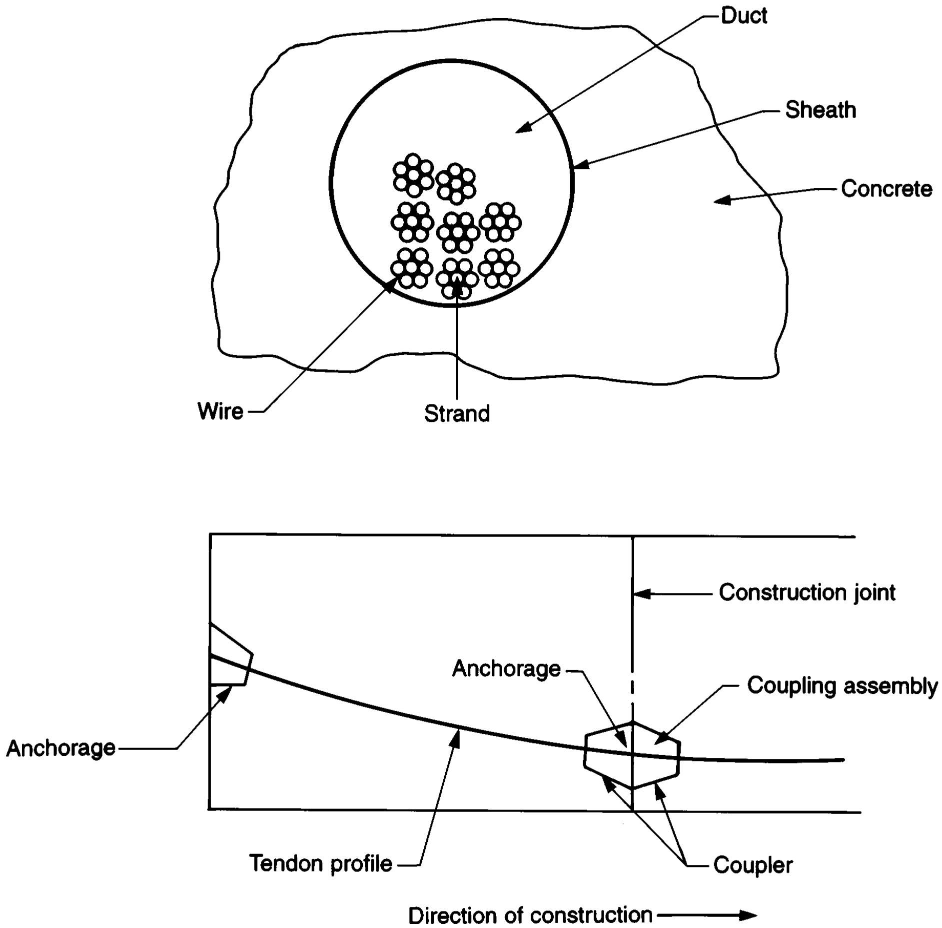


Fig. 6.3 Nomenclature associated with prestressed concrete members

(e) At the ultimate limit state the horizontal forces to be resisted at any level should be the greater of:

(i) 1.5% of the characteristic dead load above that level†, or
(ii) 90% of the wind load derived from CP3, Chapter V, Part 2[16] multiplied by the appropriate partial safety factor.

The horizontal forces should be distributed between strongpoints according to their stiffness.

In using the above documents with this *Manual* the following modifications should be noted:

(f) The imposed floor loads of a building should be treated as one variable action to which the reduction factors given in BS 6399, Part 1, 1996[5] are applicable.
(g) Snow drift loads obtained from BS 6399, Part 3, 1988[5] should be multiplied by 0.7 and treated in a similar way to an imposed load and not as an accidental load.

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* This complies with the UK NAD, although the code has been revised and issued as BS6399, Part 2[5].
† This will generally be more conservative than EC2 requirements.

6.3 Limit states

This *Manual* adopts the limit-state principle and the partial factor format of EC2.

6.3.1 Ultimate limit state

The design loads are obtained by multiplying the characteristic loads by the appropriate partial factor, γ_f, from Table 6.3. The 'adverse' and 'beneficial' factors should be used so as to produce the most onerous condition. The factors to be used for prestress are explained in Table 6.4 and Section 8.

6.3.2 Serviceability limit states

At the serviceability limit state, three basic load combinations, viz. rare, frequent and quasi-permanent, need to be considered depending on the effect being checked. When an imposed load, wind and snow are likely to occur simultaneously, the design values are obtained by multiplying the characteristic values by an appropriate ψ value to allow for the improbability of all the loads acting with their characteristic values. Thus within each combination, further sub-combinations need consideration. Prestressing loads must also be considered with an upper or lower characteristic value to produce the most critical combination.

The appropriate 'multiplier' ($\gamma_f.\psi$) for different combinations are given in Table 6.5. The factors to be used for prestress are explained in Table 6.4 and Section 8.

6.4 Materials, prestressing components and design stresses

In the UK the most common prestressing tendons are comprised of 7-wire low relaxation (Class 2) strands to BS 5896[15] which are generally available in three different types: standard, super and dyform (or drawn); and three nominal diameters: 13mm, 15mm and 18mm. The most common strands are 13mm and 15mm diameter standard and super. These are used singly in pretensioned members and in groups in post-tensioned construction. In most building structures, the size of the members generally limits the number of strands in each tendon to typically 4 or 7 strands, although anchorages for tendons of up to 55 strands are available.

Table 6.6 and Table 6.7 give typical dimensional data for ducts, anchorages, anchorage pockets and jacks for the more commonly used prestressing systems in the UK. For more

Table 6.3 Partial factors for loads, $\gamma_{f,}$ at the ultimate limit state

Load combination	Dead load G_k		Imposed, wind & snow load, Q_k or W_k		Prestress P	
	Adverse	Beneficial	Adverse	Beneficial	Adverse	Beneficial
1. Dead + imposed	1.35	–	1.50	–	1.00 or 1.20*	0.90 or 1.00*
2. Dead + wind	1.35	1.00	1.50	–	1.00 or 1.20*	0.90 or 1.00*
3. Dead + snow	1.35	1.00	1.50	–	1.00 or 1.20*	0.90 or 1.00*
4. Dead + imposed + wind + snow	1.35	–	1.35	–	1.00 or 1.20*	0.90 or 1.00*

Notes to Table 6.3

1. The table uses the simplified combination permitted in EC2.
2. *For selection of partial factors for prestress, see Table 6.4.

Table 6.4 Partial factors to be used for prestress

Effect	Clause	Partial factors			
		Serviceability limit state		Ultimate limit state	
		Primary	Secondary	Primary	Secondary
Flexure					
Serviceability limit state					
Concrete compressive stress	8.3.4.1	1.0	1.0	–	–
Crack control					
– tendons located on tensile side of neutral axis	8.3.4.2 8.3.4.3	0.9	0.9	–	–
– tendons located on compressive side of neutral axis	8.3.4.2 8.3.4.3	1.1	1.1	–	–
Ultimate limit state					
Bonded tendons					
– maximum stress in tendons > $0.87f_{p0.1k}$	8.3.5.1	–	–	1.0	1.0
– maximum stress in tendons ≤ $0.87f_{p0.1k}$	8.3.5.1	–	–	0.9	1.0
Unbonded tendons					
– prestressing force is beneficial	8.3.5.2	–	–	0.9	1.0
Shear					
Inclined tendons reduce design shear force	8.4.1	–	–	0.9	1.0
Inclined tendons increase design shear force	8.4.1	–	–	1.2	1.0
Reduced maximum shear capacity of section, $V_{Rd2.red}$	8.4.1	–	–	1.2	–
Shear capacity of section, V_{Rd1}	8.4.1	–	–	0.9	–
Shear capacity of flat slab, σ_{cp0}	8.4.1	–	–	0.9	–
Anchorage zones					
Cracking of pretensioned members	8.5.2	–	–	0.9	–
Post-tensioned members	8.5.3	–	–	$A_p f_{pk}$*	–

Note to Table 6.4

1. *EC2 only specifies a design load, not a factor.

Table 6.5 'Multiplier' ($\gamma_f.\psi$) to be used at the serviceability limit state

Load combination		Dead load G_k	Imposed load, Q_k			Wind W_k	Snow Q_k	Prestress P
			Dwellings	Offices and stores	Parking			
Rare	(a)	1.0	1.0	1.0	1.0	0.7	0.7	0.9, 1.0 or 1.1*
	(b)	1.0	0.5	0.7	0.7	1.0	0.7	0.9, 1.0 or 1.1*
	(c)	1.0	0.5	0.7	0.7	0.7	1.0	0.9, 1.0 or 1.1*
Frequent	(a)	1.0	0.4	0.6	0.7	0	0	0.9, 1.0 or 1.1*
	(b)	1.0	0.2	0.3	0.6	0.2	0	0.9, 1.0 or 1.1*
	(c)	1.0	0.2	0.3	0.6	0	0.2	0.9, 1.0 or 1.1*
Quasi-permanent		1.0	0.2	0.3	0.6	0	0	0.9, 1.0 or 1.1*

Notes to Table 6.5

1. *For selection of partial factors for prestress, see Table 6.4

precise information and when other prestressing systems are used, the engineer should refer to the manufacturer's literature.

Design stresses are given in the appropriate sections of the *Manual.*

It should be noted that EC2 specifies concrete strength class by both the cylinder strength and cube strength (for example C30/37 is a concrete with a cylinder strength of 30N/mm^2 and a cube strength of 37N/mm^2 at 28 days). Standard strength classes for prestressed concrete are C30/37, C35/45, C40/50, C45/55 and C50/60. **All design equations in this *Manual* which include concrete compressive strength use the characteristic 28 day cylinder strength, f_{ck}.**

Partial factors for concrete are 1.5 for the ultimate limit state and 1.0 for the serviceability limit state.

The strength properties of reinforcement are expressed in terms of the characteristic yield strength, f_{yk}, and those of prestressing strand in terms of the characteristic tensile strength, f_{pk}, and the 0.1% proof stress, $f_{p0.1k}$.

Partial factors for reinforcing steel and prestressing tendons are 1.15 at the ultimate limit state and 1.0 for the serviceability limit state.

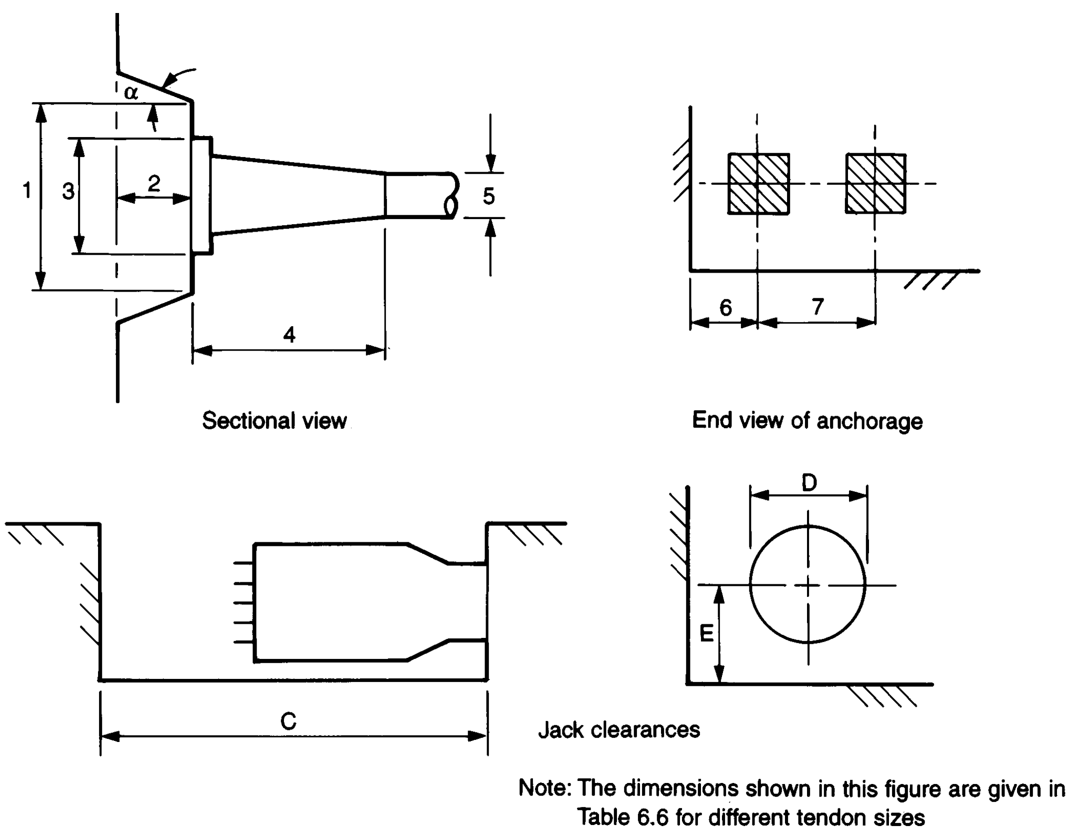


Fig. 6.4 Dimensions of common post-tensioning systems (1–9 strands)

Table 6.6 Typical dimensional data for common post-tensioning systems (1 to 19 strands)

Strand diameter and number of strands	Pocket dimensions, mm			Anchorage dimensions, mm		Duct diameter int/ext*, mm	Anchorage spacing, mm		Jack diameter and clearances, mm		
	1	2	α	3	4	5	6	7	C	D	E
13–1	130	110	30°	70	85	25/30	80	90	1200	140	100
13–3	180	115	30°	120	210	40/45	115	155	1100	200	150
13–4	240	115	30°	135	210	45/50	125	180	1100	248	175
13–7	240	120	30°	175	215	55/60	155	235	1200	342	220
13–12	330	125	30°	230	405	75/82	195	305	1300	405	250
13–19	390	140	30°	290	510	80/87	230	385	2100	490	295
15–1	135	115	30°	75	85	30/35	95	105	1200	140	100
15–3	200	115	30°	150	210	45/50	130	185	1100	210	140
15–4	240	120	30°	157	215	50/55	140	210	1200	342	220
15–7	305	125	30°	191	325	60/67	175	280	1300	405	250
15–12	350	140	30°	270	510	80/87	220	365	1500	490	295
15–19	470	160	30°	340	640	100/107	265	460	2000	585	300

Notes to Table 6.6

1. The dimensions and clearances are illustrated in Fig. 6.4.
2. The values in the table are based on an envelope of the requirements of a number of manufacturers' systems. Where clearances are critical it is recommended that reference is made to the specific manufacturers' catalogues.
3. *Data are for corrugated steel sheaths with bonded tendons.

Table 6.7 Typical dimensional data for common post-tensioning systems for slabs

Strand diameter and number of strands	Pocket dimensions, mm			Achorage dimensions, mm			Duct external dimensions*, mm	Anchorage spacing, mm		Jack clearances, mm		
	1a	1b	2	3a	3b	4	5	x_e, y_e	x_s, y_s	C	E	F
13–1	130	130	110	70	110	70	30 dia.	125, 80	150, 100	1390	100	–
13–4	144	310	103	96	250	130	75 × 20	220, 140	370, 220	1200	90	280
15–1	150	150	115	130	130	95	35 dia.	145, 100	175, 125	1450	100	–
15–4	168	335	127	115	280	240	75 × 20	235, 160	400, 230	1450	70	327

Notes to Table 6.7

1. The dimensions and clearances are illustrated in Fig 6.5.
2. *Data are for corrugated steel sheaths with a thickness of 1.5 mm.
3. The values in the table are based on an envelope of the requirements of a number of manufacturers' systems. Where clearances are critical it is recommended that reference is made to the specific manufacturers' catalogues.

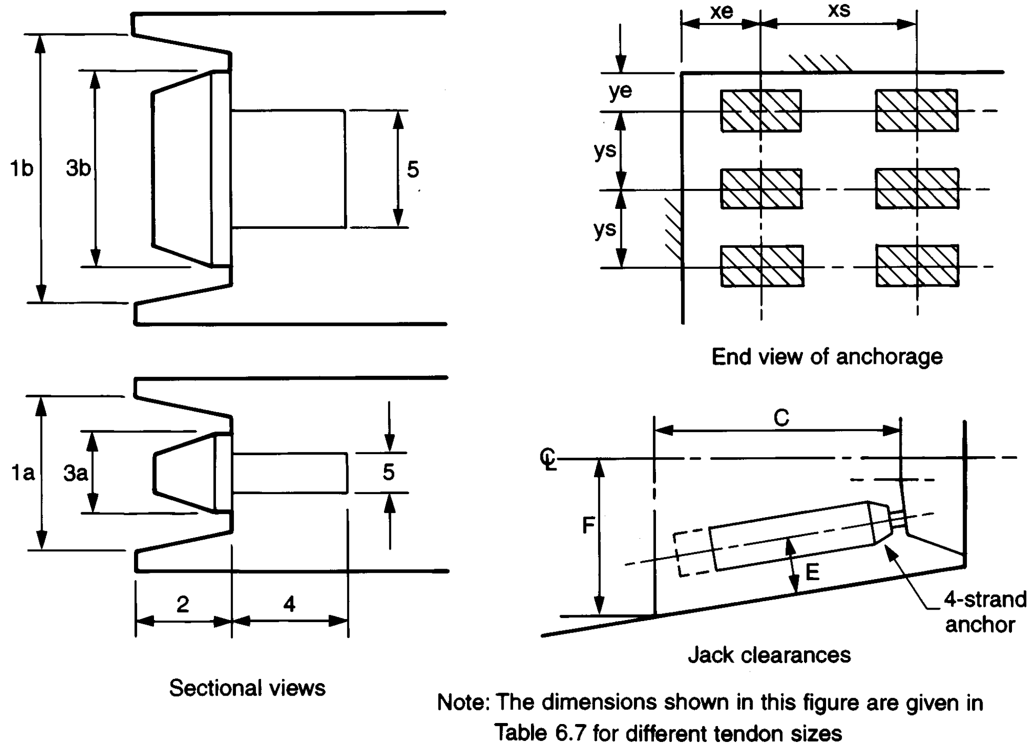


Fig. 6.5 Dimensions of common post-tensioning systems for slabs

7 Initial design – prestressed concrete

7.1 Introduction

In the initial stages of the design of building structures it is necessary, often at short notice, to produce alternative schemes that can be assessed for architectural and functional suitability and which can be compared for cost. They will usually be based on vague and limited information on matters affecting the structure such as imposed loads and nature of finishes, let alone firm dimensions, but it is nevertheless expected that viable schemes be produced on which reliable cost estimates can be based.

It follows that initial design should be simple, quick, conservative and reliable. Lengthy analytical methods should be avoided.

This section offers some advice on the general principles to be applied when preparing a scheme for a structure, followed by methods for sizing prestressed slabs and beams.

The aim should be to establish a structural scheme that is suitable for its purpose, sensibly economical, and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Sizing of structural members should be based on the longest spans (slabs and beams) and on the largest areas of roof and/or floors carried (beams). The same sizes should be assumed for similar but less onerous cases – this saves design and costing time at this stage and is of actual benefit in producing visual and constructional repetition and hence, ultimately, cost benefits.

Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good 'benchmark' at the initial stage.

Loads should be carried to the foundations by the shortest and most direct routes. In constructional terms, simplicity implies (among other matters) repetition; avoidance of congested, awkward or structurally sensitive details and straightforward temporary works with minimal requirements for unorthodox sequencing to achieve the intended behaviour of the completed structure.

Standardised construction items will usually be cheaper and more readily available than purpose-made items.

7.2 Loads

Loads should be based on BS 648[4], BS 6399: Parts 1 and 3[5] and CP3: Chapter V: Part 2[6].

Imposed loading should initially be taken as the highest statutory figures where options exist. The imposed load reduction allowed in the loading code should not be taken advantage of in the initial design stage except when assessing the load on the foundations.

Loading should be generous and not less than the following in the initial stages:

floor finish (screed)	1.8 kN/m^2
ceiling and service load	0.5 kN/m^2

Allowance for:

demountable lightweight partitions* 1.0 kN/m^2
blockwork partitions† 2.5 kN/m^2

Weight of concrete should be taken as 24 kN/m^3.

The initial design of prestressed concrete members should be carried out at the serviceability limit state using the following simplified load combinations (a) to (d):

(a) *dead load + imposed load*
1.0 × characteristic dead load + 1.0 × characteristic imposed load
(b) *dead load + wind load*
1.0 × characteristic dead load + 1.0 × characteristic wind load
(c) *dead load + snow load*
1.0 × characteristic dead load + 1.0 × characteristic snow load
(d) *dead load + imposed load + wind load + snow load*
1.0 × characteristic dead load + 0.9 × characteristic imposed load + 0.9 × characteristic wind load + 0.9 × characteristic snow load.

7.3 Material properties

It is recommended that the minimum concrete grade for post-tensioned construction is C30/37 and for pretensioned members is C40/50. A higher grade concrete may need to be used in order to allow higher compressive stresses to be carried by the concrete and to meet durability requirements. For initial design, concrete grades greater than C40/50 should not be considered.

For initial design, prestress forces after losses should be taken from Table 7.1.

When UK steels are used for ordinary reinforcement a characteristic strength, f_{yk}, of 460N/mm^2 should be adopted for high tensile reinforcement and 250N/mm^2 for mild steel.

7.4 Structural form and framing

The following measures should be adopted:

(a) provide stability against lateral forces and ensure braced construction by arranging suitable shear walls deployed symmetrically wherever possible
(b) adopt a simple arrangement of slabs, beams and columns so that loads are carried to the foundations by the shortest and most direct routes
(c) allow for movement joints (see subsection 2.4)
(d) choose an arrangement that will limit the span of slabs to 8m to 10m and beam spans to 12m to 15m on a regular grid; for flat slabs restrict column spacings to 12m
(e) if using post-tensioned construction, choose an arrangement that provides sufficient space for the stressing jacks (see subsection 6.4) and that allows the prestressed members to shorten as the tendons are stressed

Table 7.1 Strand loads (after losses) to be used for initial design

Strand type	Strand load, kN	
	13mm dia.	15mm dia.
Standard	88	125
Super	100	143

*Treat these as imposed loads.
†Treat these as dead loads when the layout is fixed.

(f) if unbonded tendons are being used, provide sufficient space for access to the tendons to allow for their replacement
(g) adopt a minimum column size of 300 × 300 mm or equivalent area
(h) provide a robust structure
(i) allow for sufficient topping to accommodate cambers induced by prestressing, including any differential cambers between adjoining members.

The arrangement should take account of possible large openings for services and problems with foundations, e.g. columns immediately adjacent to site boundaries may require balanced or other special foundations. Tendon positions should be chosen to avoid locations of any possible future holes.

7.5 Fire resistance and durability

The size of structural members may be governed by the requirement of fire resistance and may also be affected by the cover necessary to ensure durability. Table 7.2 gives the minimum practical member sizes for different periods of fire resistance and the cover to the prestressing tendons required for members in dry humid environments without frost. For other exposure conditions cover should be increased (see Section 8 and Appendix C).

7.6 Stiffness

7.6.1 Slabs

To provide adequate stiffness, the effective depths of slabs and the waist of stairs should not be less than those derived from Table 7.3.

Table 7.2 Minimum member sizes and covers for initial design of prestressed members

Member	Minimum dimension, mm				
	for fire resistance of		4 h	2 h	1 h
Beams	Width		280	200	200
	Cover†	Simply supported	90	60	45
		Continuous	80	45	45
Plain soffit solid slabs and flanges of T-section beams and slabs	Depth (including non-combustible finishes)		170	125	100
	Cover†	Simply supported	65	40	30
		Continuous	55	35	30
Ribs of T-section slabs with no stirrups	Width of ribs		200	150	110
	Cover†	Simply supported	75	55	35
		Continuous	65	45	35

Notes to Table 7.2
1. †Cover is to prestressing tendons.
2. The cover to pretensioned tendons should not be less than twice the diameter of the tendon.
3. In post-tensioned members, the minimum cover to the sheaths not be less than the sheath diameter.

The ratios for two-way slabs have been calculated for a square panel. For a 2 × 1 panel, the ratio for a one-way slab should be used and ratios interpolated for intermediate proportions. The depth should be based on the shorter span.

Flat slab design should be based on the longer span dimension. For exterior panels, 85% of the ratios quoted in Table 7.3 should be used.

Ribbed slabs should be proportioned so that:

- the rib spacing does not exceed 900mm
- the rib width is not less than 125mm
- the rib depth does not exceed four times its width.

The minimum structural topping thickness should preferably be 75mm, but never less than 50mm or one-tenth of the clear distance between ribs, whichever is the greater. For ribbed slabs, 85% of the ratios quoted in Table 7.3 should be used.

Slabs with large span to depth ratios may be susceptible to vibration. In order to avoid this, there should be a run of at least four panels with a minimum depth of 250mm for plain soffit slabs and 400mm for ribbed slabs. In other cases the engineer should consult specialist literature.

7.6.2 Beams

For initial sizing the effective depth of beams should be determined from Table 7.4. If other considerations demand shallower construction, reference should be made to subsection 8.3.

7.7 Sizing

7.7.1 Introduction

When the depths of the slabs and beams have been obtained it is necessary to check the following:

- width of beams and ribs
- column sizes and reinforcement (see clause 4.7.4)
- shear in flat slabs at columns
- practicality of tendon and reinforcement arrangements in beams, slabs and at beam-column junctions.

Table 7.3 Span-effective depth ratios for initial design of slabs

Characteristic imposed loading (including finishes) kN/m^2	One-way spanning			Two-way spanning		Flat slab without drops
	simply supported	continuous	cantilever	simply supported	continuous	
5	30	38	12	32	42	36
10	26	34	11	28	36	30

Table 7.4 Span/effective depth ratios for initial design of beams

cantilever	8
simply supported	18
continuous	22

7.7.2 Loading

Serviceability loads should be used throughout for initial design (see subsection 7.2). For prestressed concrete members it is necessary to calculate the maximum shear force and both the maximum and minimum bending moments at the critical sections. A critical condition may occur at transfer (when the prestressing force is initially applied to the concrete member) as the prestressing force is at its highest value and the applied loading may be significantly less than the maximum load which the member will eventually have to carry.

The loading arrangements to be considered for continuous members are:

- alternate spans carrying the maximum dead and imposed load, other spans carrying the maximum dead load
- any two adjacent spans carrying the maximum dead and imposed load, other spans carrying the maximum dead load.

For purposes of assessing the self-weight of beams, the width of the downstand can be taken as half the depth but usually not less than 300mm.

7.7.3 Width of beams and ribs

The width should be determined by limiting the shear stress in beams to 2.0 N/mm^2 and in ribs to 0.8N/mm^2.

$$\text{width of beam (in mm)} = \frac{1000V}{2d} \quad \text{width of rib (in mm)} = \frac{100V_{\mathrm{SLS}}}{0.8d}$$

where V_{SLS} is the maximum shear force at the serviceability limit state (in kN) on the beam or rib, considered as simply supported, and d is the effective depth in mm.

7.7.4 Punching shear in flat slabs at columns

Check that:

(a) where shear reinforcement is to be avoided

$$\frac{1250 w_{\mathrm{SLS}}\, A_{\mathrm{sup\,p}}}{\left(u_{\mathrm{c}} + 9h\right)\mathrm{d}} \leq 0.5\ \mathrm{N/mm^2}$$

(b) where shear reinforcement is to be provided

$$\frac{1250 W_{SLS}\, A_{\mathrm{sup\,p}}}{u_{\mathrm{c}}\, d} \leq 0.6\sqrt{f_{\mathrm{ck}}}\ \mathrm{N/mm^2}$$

(c) $$\frac{1250 W_{SLS}\, A_{\mathrm{sup\,p}}}{u_{\mathrm{c}}\, d} \leq 0.6\sqrt{f_{\mathrm{ck}}}\ \mathrm{N/mm^2}$$

In the above verifications:

w_{SLS}	is the total design load per unit area at the serviceability limit state in kN/m^2
d	is the effective depth of the slab at the column in mm
h	is the thickness of the slab at the column in mm,
$\mathrm{A}_{\mathrm{supp}}$	is the area supported by the column in m^2
u_{c}	is the column perimeter in mm, and
f_{ck}	is the concrete cylinder strength in N/mm^2.

7.7.5 Adequacy of chosen sections to accommodate the tendons and reinforcement

In the initial stage the number of prestressing tendons needs to be checked only at midspan and at the supports of critical spans.

7.7.5.1 Bending moments and shear forces

Beams and one-way solid slabs

Bending moments and shear forces may be obtained by elastic analysis.

Two-way solid slabs on linear supports

If the longer span l_y does not exceed 1.5 times the shorter span l_x, the average moment per metre width may be taken as:

	Short span	Long span
at mid span:	$\dfrac{w_{SLS}\, l_x\, l_y}{20}$ kNm per metre	$\dfrac{w_{SLS}\, l_x^2}{20}$ kNm per metre
at a continuous support:	$-\dfrac{w_{SLS}\, l_x\, l_y}{20}$ kNm per metre	$-\dfrac{w_{SLS}\, l_x^2}{17}$ kNm per metre

where w_{SLS} is the design load at the serviceability limit state in kN/m^2, and l_x and l_y are in metres.

If $l_y > 1.5 l_x$ the slab should be treated as acting one-way.

Solid flat slabs

Determine the moments per unit width in each direction as for one-way slabs.

Ribbed slabs

Determine the bending moments per rib by multiplying the moments for solid slabs by the rib spacing.

7.7.5.2 Provision of tendons and reinforcement

Determine the number of tendons

An estimate of the size and number of tendons required can be made using ‘load balancing’. In this approach the concrete section is considered loaded by the applied dead and imposed loads which are partially supported (or balanced) by the forces from the prestressing tendons. Any out-of-balance forces have to be resisted by the concrete section and ordinary reinforcement. The exact degree of load balancing required is a matter of experience, judgment and the degree of cracking that is considered acceptable. When draped tendons are used and cracking is acceptable an economical design will generally be obtained when the prestressing tendons balance approximately 50% of the total dead and imposed loading.

Fig. 7.1 shows the forces applied to the concrete by the prestress: (a) at the anchorages, (b) by the curvature of the tendons, and (c) at a change of section. These are referred to as equivalent loads and will automatically generate the primary and secondary effects when applied to the concrete member.

The bending moments, axial forces and corresponding stresses in the concrete member under dead, imposed and the equivalent loads are calculated as described in subclause 7.7.5.1, assuming an homogeneous section. The prestressing force and tendon profile are adjusted until the stresses obtained comply with the limits given in Table 7.5. When checking the stresses at transfer, the prestressing force required for the in-service condition should be increased by 60%, as the long-term (time-dependent) losses will not have

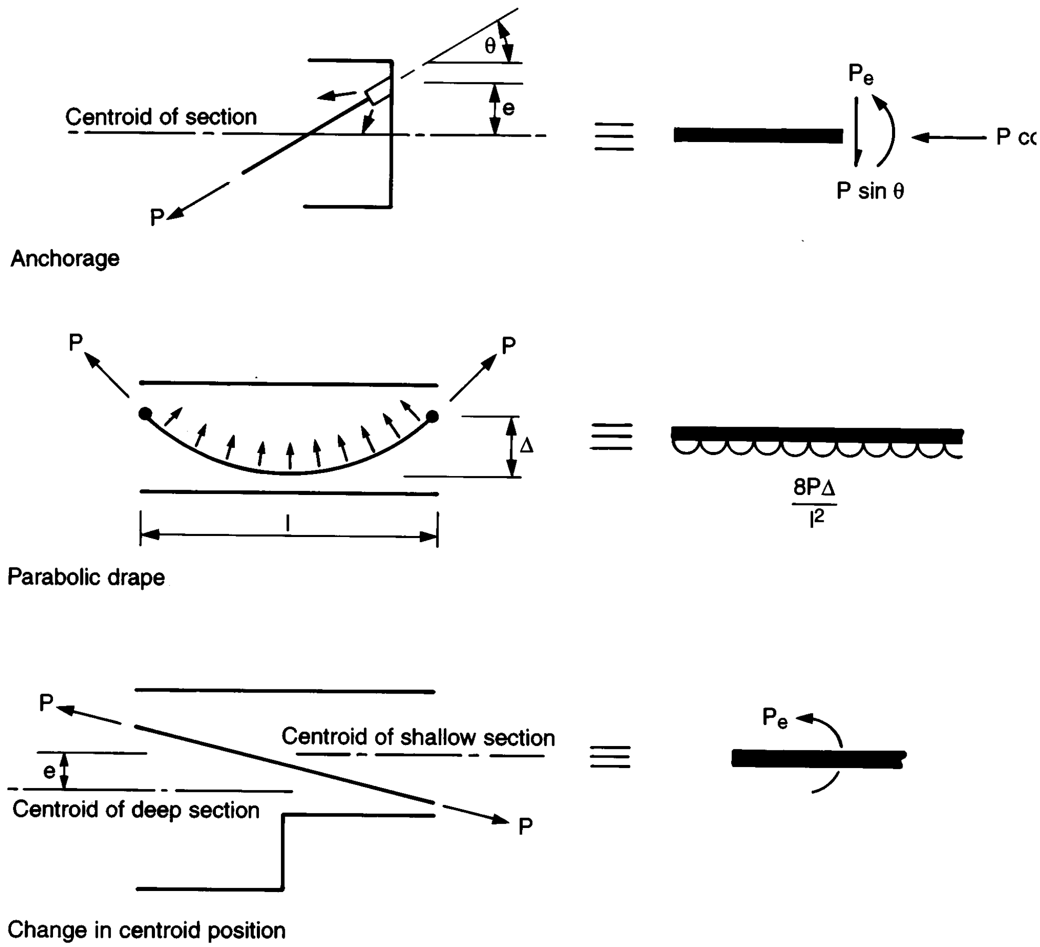


Fig. 7.1 Equivalent loads

occurred, and no tension should be allowed in the concrete.

When the prestressing force has been determined, calculate the size and number of tendons using Table 7.1.

When tension in concrete has been taken into account, ordinary reinforcement should be provided in order to control crack widths and to give adequate ultimate capacity. The area of reinforcement provided should be sufficient to resist the total net tensile force on the section when acting at a stress of $0.6f_{yk}$, see Fig. 7.2, when bonded tendons are used. When unbonded tendons are employed, increase this area by 20%.

Table 7.5 Allowable stresses for initial design

Maximum compressive stress	$0.6f_{ck}$
Maximum tensile stress no tension allowed otherwise	 0 5N/mm^2

f_{ck} is the concrete cylinder strength

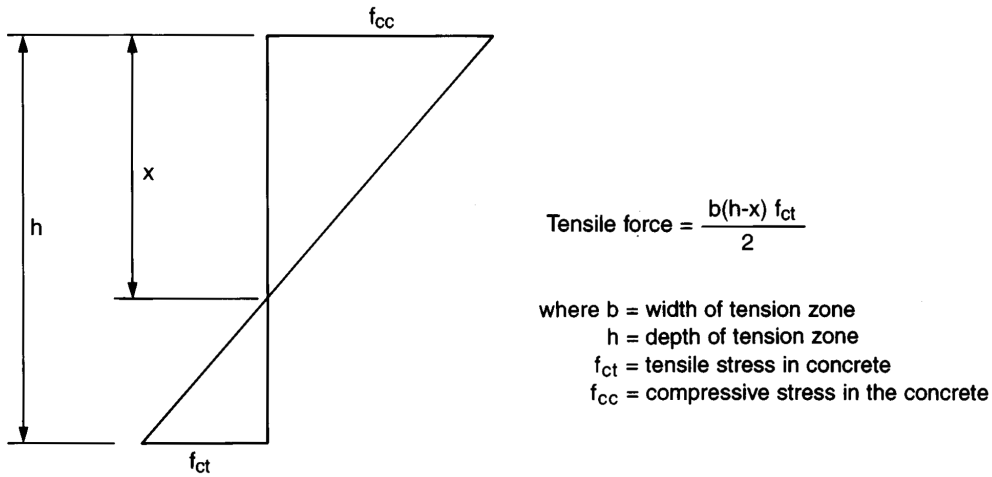


Fig. 7.2 Total net tensile force

Tendon and reinforcement arrangements

When the number and size of the prestressing tendons, ordinary reinforcement and the areas of the main reinforcement in any non-prestressed elements (see clause 4.7.7) have been determined, check that the tendons and bars can be arranged with the required cover in a practicable manner avoiding congested areas.

In flat slabs at least 50% of the tendons in each direction should be placed symmetrically about the column centre-line within a width equal to the column size plus the depth of the slab.

In post-tensioned construction, check that there is sufficient space for the cable anchorages and their associated reinforcement and that the stressing jacks can be located on the ends of the tendons and extended (see subsection 6.4).

7.8 The next step

At this stage general arrangement drawings, including sections through the entire structure, should be prepared and sent to other members of the design team for comment, together with a brief statement of the principal design assumptions, e.g. imposed loading, weights of finishes, fire ratings and durability.

The scheme may have to be amended following receipt of comments. The amended design should form the basis for the architect's drawings and may also be used for preparing reinforcement estimates for budget costing.

7.9 Reinforcement estimates

In order for the cost of the structure to be estimated it is necessary for the quantities of the materials, including those of the prestressing tendons and reinforcement, to be available. Fairly accurate quantities of the concrete, brickwork, prestressing tendon size and length, and number of prestressing anchorages can be calculated from the layout drawings. If working drawings and schedules for the reinforcement are not available it is necessary to provide an estimate of the anticipated quantities.

The quantities of ordinary reinforcement associated with prestressed concrete members can be estimated using the following methods.

Slabs

Pretensioned
The area calculated in subclause 7.7.5.2

Post-tensioned
The area calculated in subclause 7.7.5.2, but not less than 0.15% of cross-sectional area longitudinally, and:

$$(h + b)A_p f_{pk} \times 10^{-5} \text{ kg per end block}$$

where: h is the depth of the end block in metres
b is the width of the end block in metres
A_p is the total area of prestressing tendons anchored within the end block in mm^2, and
f_{pk} strength of the tendons in N/mm^2.

Beams

Shear links
Refer to Table A3, taking the design ultimate shear force as $1.5V_{SLS}$ (see clause 7.7.3)

Pretensioned
The area calculated in subclause 7.7.5.2

Post-tensioned
The area calculated in subclause 7.7.5.2, but not less than 0.15% (high yield) or 0.25% (mild steel) of cross-sectional area longitudinally, and:

$$(h + b)A_p f_{pk} \times 10^{-5} \text{ kg per end block}$$

where: h is the depth of the end block in metres
b is the width of the end block in metres
A_p is the total area of prestressing tendons anchored within the end block in mm^2, and
f_{pk} is the characteristic strength of the tendons in N/mm^2.

When preparing the reinforcement estimate, the following items should be considered:

(a) *Laps and starter bars*
A reasonable allowance for normal laps has been made in the previous paragraphs. It should however be checked if special lapping arrangements are used.

(b) *Architectural features*
The drawings should be looked at and sufficient allowance made for the reinforcement required for such 'non-structural' features.

(c) *Contingency*
A contingency of between 10% and 15% should be added to cater for some changes and for possible omissions.

8 Final design – prestressed concrete

8.1 Introduction

In the initial design phase, the size of the prestressed members and individual tendons will have been selected. The final design is prepared in a similar way to that for ordinary reinforced concrete members described in Section 5, beginning with the steps described in sub-section 5.1.

The slabs should be designed first followed by the beams.

The general procedure to be adopted is as follows:

1. Check that the cross-section and cover comply with requirements for fire resistance
2. Check that cover and concrete grade comply with requirements for durability
3. Select a tendon profile and calculate an initial and final force profile for one tendon
4. Calculate bending moments and shear forces
5. Check concrete compressive stresses and crack widths at the serviceability limit state including transfer. Adjust the prestressing and/or calculate the amount of ordinary reinforcement required
6. Check the capacity of the section at the ultimate limit state and increase the amount of ordinary reinforcement if necessary
7. Calculate the reinforcement required for shear
8. Calculate the reinforcement required in anchorage zones
9. Calculate the expected tendon extensions during jacking.

The effective span of a simply supported member should normally be taken as the clear distance between the faces of the supports plus one third of their widths. However, where a bearing pad is provided between the member and the support, the effective span should be taken as the distance between the centres of the bearing pads.

The effective span of a member continuous over its supports should normally be taken as the distance between centres of supports.

The effective length of a cantilever where this forms the end of a continuous member is the length of the cantilever from the centre of the support. Where the member is an isolated cantilever the effective length is the length of the cantilever from the face of the support.

To prevent **lateral buckling** of beams, the length of the compression flange measured between adequate lateral restraints to the beam should not exceed 50b, where b is the width of the compression flange, and the overall depth of the beam should not exceed 4b.

In normal slab-and-beam or framed construction specific calculations for torsion are not usually necessary, torsional cracking being adequately controlled by shear reinforcement. Where torsion is essential for the equilibrium of the structure, e.g. the arrangement of the structure is such that loads are imposed mainly on one face of a beam without corresponding rotational restraints being provided, EC2 should be consulted.

8.2 Fire resistance and durability

8.2.1 Fire resistance

The member sizes and nominal cover to prestressing tendons required to provide fire resistance are given in Table 8.1. Cover to ordinary reinforcement should satisfy the requirements of Section 5. The cover derived from Table 8.1 may need to be increased for durability (see clause 8.2.2).

Where the cover to the outermost reinforcement exceeds 40mm special precautions against spalling may be required, e.g. partial replacement by plaster, lightweight aggregate or the use of fabric as supplementary reinforcement (see BS 8110: Part 2[16]).

8.2.2 Durability

The requirements for durability in any given environment are:

(a) an upper limit to the water/cement ratio
(b) a lower limit to the cement content
(c) a lower limit to the cover to the reinforcement and prestressing tendons or ducts
(d) a limit on the design crack width
(e) good compaction
(f) adequate curing
(g) good detailing
(h) good grouting of ducts for internal bonded post-tensioned construction
(i) good protection of tendons in unbonded prestressed construction
(j) good protection of anchorages.

Table 8.1 Fire resistance requirements for prestressed concrete members

Member	Minimum dimensions, mm						
		for fire resistance of	1 h	1.5 h	2 h	3 h	4 h
Beams	Simply supported	Width	120	150	200	240	280
		Cover	40	55	70	80	90
	Continuous	Width	100	120	150	200	240
		Cover	30	40	55	70	80
Plain soffit slabs		Thickness	95	110	125	150	170
	Cover	Simply supported	25	30	40	55	65
		Continuous	20	25	35	45	55
Ribbed open soffit slabs		Flange thickness*	90	105	115	135	150
	Simply supported	Width	110	135	150	175	200
		Cover	35	45	55	65	75
	Continuous	Width	75	110	125	150	175
		Cover	25	35	45	55	65

Notes to Table 8.1

1. The table is based on the requirements of BS 8110: Part 2[16]
2. * Including non-combustible finishes.
3. Covers are measured to the prestressing tendons.
4. For beams, the width should be measured at the centroid of the prestressing tendons.
5. If the width of a beam or rib is more than the minimum given in Table 8.1 the cover may be decreased as below:

Increase in width, mm	Decrease in cover, mm
25	5
50	10
100	15
150	15

Values for (a), (b), (c) and (d) which, in combination, will give adequate durability are given in Table 8.2 for various environments. The cover to ordinary reinforcement may be 5mm less than those given in the table. The design crack widths given in the table apply to members prestressed with internal bonded tendons. Crack widths in members prestressed only with unbonded tendons should be limited to 0.3mm for conditions of exposure 1 to 4.

As (a) and (b) cannot be checked by methods that are practical for use during construction, Table 8.2 gives, in addition, the concrete strength classes that have to be specified in the UK so that requirements (a) and (b) are satisfied. These strengths will often require cement contents that are higher than those given in Table 8.2. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

For frost resistance, the use of air-entrained concrete should be considered; however, the effect of air entrainment on the properties of concrete should also be taken into account.

Where tendons are curved, the cover perpendicular to the plane of curvature should be increased to prevent bursting of the cover concrete in accordance with Table 8.3.

Table 8.2 Durability requirements for prestressed concrete members

Conditions of exposure (for definitions, see Appendix C)	Nominal cover to prestressing tendons, mm				Design crack width, mm	
					Pretensioned	Post-tensioned‡
1. Dry environment	25	25	25	25	0.2	0.2
2. Humid environment						
(a) without frost	40	40	35	35	0.2	no tension
(b) with frost	–	40	35	35	0.2	no tension
3. Humid with frost and deicing salts	–	45	40	40	no tension	no tension
4. Sea water environment						
(a) without frost	–	45	40	40	no tension	no tension
(b) with frost	–	45	40	40	no tension	no tension
The following conditions may occur alone or in combination with those above						
5. Aggressive chemical environment						
(a) slightly aggressive	–	40	35	35	special measures may be necessary	
(b) moderately aggressive	–	–	35	35		
(c) highly aggressive*	–	–	-	50		
Maximum free water/cement ratio	0.6	0.55	0.5	0.45		
Minimum cement content, kg/m³	280	300	300†	300		
Minimum concrete strength class	C30/37	C35/45	C40/50	C45/55		

Notes to Table 8.2

1. The table is based on the requirements of ENV206[17] and EC2.
2. *Protective barrier to prevent direct contact with highly aggressive media should be provided.
3. †280 kg/m³ for exposure conditions 2b and 5a.
4. ‡Members with *bonded* tendons only. For members with *unbonded* tendons, the design crack width is 0.3mm for conditions of exposure 1 to 4.
5. Cover is expressed in terms of nominal values which have been obtained from the minimum values by allowing for a negative tolerance of 5mm.
6. The cover may be reduced by 5mm in slabs, for conditions of exposure 2 to 5.
7. The cover to *pretensioned* tendons should not be less than twice the diameter of the tendon.
8. In *post-tensioned* members, the minimum cover to the sheaths should not be less than the duct diameter.
9. The cover to *ordinary* reinforcement may be 5mm less than those given in the table.
10. The cover in mm to the *main* ordinary reinforcement should not be less than the bar diameter.

Table 8.3 Minimum cover to curved ducts, mm

Radius of curvature of duct, m	Duct internal diameter, mm									
	25	300	40	45	50	55	60	75	80	100
	Tendon force, kN									
	159	229	477	636	916	1113	1603	1908	2748	4351
2	30	35	70							95
4			45	50	70	80	120	140		
6					55	60	80	95	135	215
8							65	80	105	160
10									90	130
12									85	115
14	30	35	45	50	55	60	65	80	85	105

Notes to Table 8.3

1. The table is based on reference 18.
2. The tendon force shown is the maximum normally available for the given size of duct.
3. Where tendon profilers or spacers are provided in the ducts, and these are of a type which will concentrate the radial force, the values given in the table will need to be increased.
4. The cover for a given combination of duct internal diameter and radius of curvature shown in the table, may be reduced in proportion to the square root of the tendon force when this is less than the value tabulated.

Grouting of ducts should be carried out in accordance with the model specification in Concrete Society Technical Report No. TR47[19]. Unbonded tendons may be protected by filling the ducts with cement grout or a suitable wax.

Typical details for providing protection to anchorages are given in reference 19.

8.3 Flexural design

8.3.1 Tendon profile

A tendon profile is chosen which satisfies the cover requirements and reflects the bending moment diagram for the applied loads (i.e. high over the supports and low at midspan). For slabs it is common practice to choose the points of contraflexure of the tendon profile so that 90% of the drape occurs over 90% of the half span, see Fig. 8.1. The tendon profile should be sketched at this stage and the leading dimensions indicated.

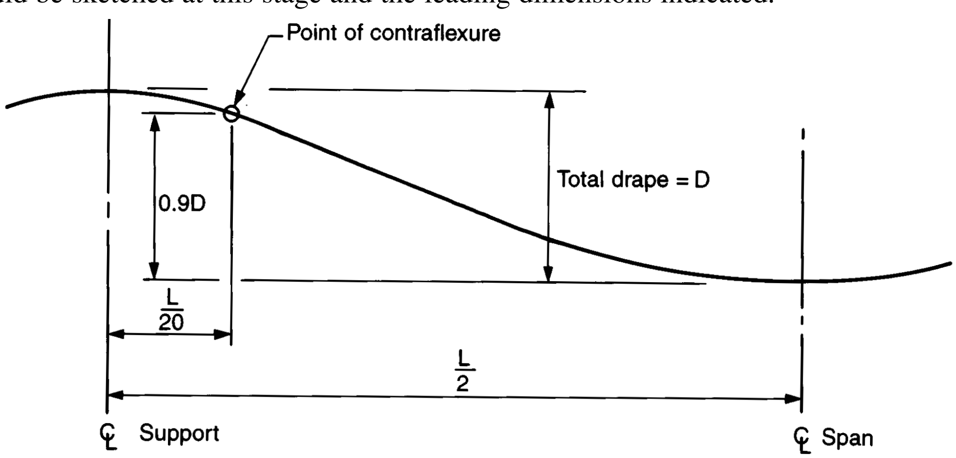


Fig. 8.1 Geometry of tendon profile

8.3.2 Tendon force profile

Determine the variation of the force in one tendon along the length of the member.

8.3.2.1 Initial force ($P_{m,0}$)

The force in the tendon reduces with distance from the jacking point because of friction which arises due to both intentional and unintentional deviations of the tendon. For most building structures with draped tendons it is sufficiently accurate to assume that the unintentional angular deviation is uniform along the member so that the force in the tendon, P_x, becomes:

$$P_x = P_0\left\{1 - (\mu\alpha + K)\,x\right\}$$

where: P_0 is the initial jacking force
m is the coefficient of friction of the duct
a is the angular deviation per unit length in rad/m
K is the wobble factor (which allows for unintentional deviation) in m^{-1}, and
x is the distance from the point at which the tendon is jacked in metres.

Values of μ and K depend on the surface characteristics of the tendon and duct, on the presence of rust, on the elongation of the tendon and on the tendon profile. Information on appropriate values are provided by the manufacturers of prestressing systems, but in the absence of more exact data values of $\mu = 0.25$ and $K = 0.003m^{-1}$ may be used.

With wedge-anchored tendon systems the tendon force profile must be adjusted to allow for the wedge draw-in or anchorage set. This depends on the wedge set movement which can be obtained from the prestressing system manufacturer and is typically 6mm. The length of tendon affected by wedge draw-in, $l_{d,}$ is given by

$$l_d = \sqrt{\frac{\Delta A_{ps} E_s}{m}}$$

where: D is the wedge draw-in
A_{ps} is the area of a tendon
E_s is the elastic modulus of the tendon which may be taken as 190kN/mm^2, and
m is the rate of change of prestressing force due to friction, i.e. $P_0(\mu\alpha + K)$.

The tendon force profile is shown in Fig. 8.2. The maximum force in the tendon after lock-off is $P_{max} = P_0 - ml_d$. The initial jacking force should be chosen so that P_0 and P_{max} do not exceed the values given in Table 8.4, which gives values for commonly used strands.

When the force from pretensioned tendons is transferred to the concrete, the concrete and tendons shorten, reducing the prestress force. The loss of prestress force due to elastic shortening is:

$$\frac{E_s}{E_s} A_{ps} \times \quad \text{(stress in concrete at the level of the tendon due to prestress and the self-weight of the concrete)}$$

where: E_c is the elastic modulus of the concrete given in Table 8.5
E_s is the elastic modulus of the tendon which may be taken as 190kN/mm^2, and
A_{ps} is the total area of the tendons.

In a post-tensioned member the tendons are normally stressed sequentially and the average loss due to elastic shortening is only 50% of the value for a pretensioned member.

Calculate the initial prestressing force, $P_{m,0,}$ by multiplying the force profile for one tendon (Fig. 8.2) by the number of tendons and subtracting the elastic loss.

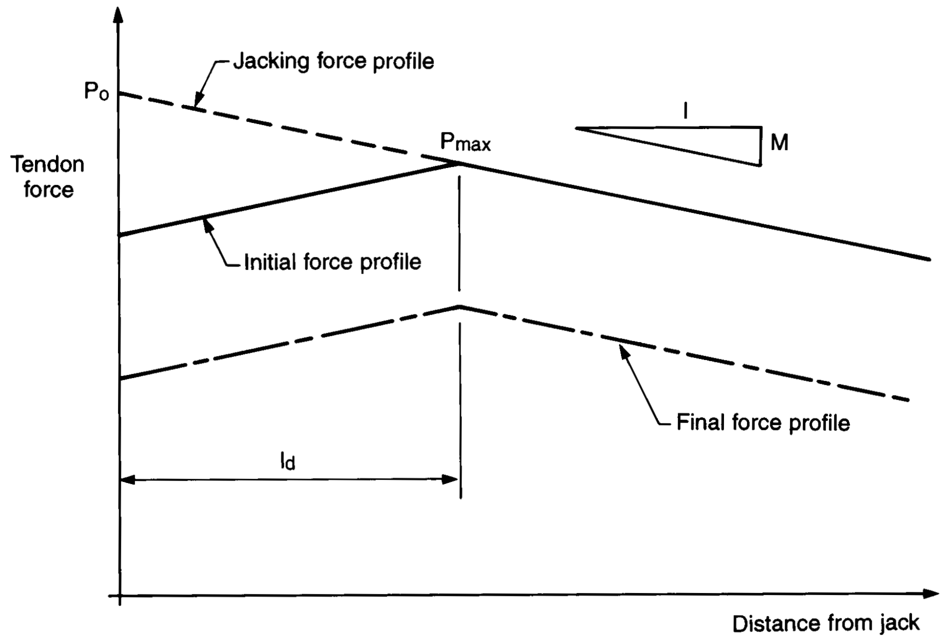


Fig. 8.2 Tendon force profile

Table 8.4 Maximum jacking loads per strand

Strand type	Diameter mm	f_{pk}, N/mm²	$f_{p0.1k}$, N/mm²	P_0, kN	P_{max}, kN
Standard (STD)	15.2 12.5	1670 1770	1420 1500	177 125	167 118
Super (SUP)	15.7 12.9	1770 1860	1500 1580	202 142	191 134
Drawn (DYF)	18.0 15.2 12.7	1700 1820 1860	1450 1545 1580	291 229 159	275 217 150

Note: This table is based on data from BS 5896[15] and EC2.

Table 8.5 Elastic modulus of concrete

Strength class	C30/37	C35/45	C40/50	C50/60
f_{ck}, N/mm²	30.0	35.0	40.0	50.0
E_c, kN/mm²	32.0	33.5	35.0	37.0

8.3.2.2 Final force ($P_{m,\infty}$)

The tendon force will reduce with time due to relaxation of the tendon and creep and shrinkage of the concrete.

The loss due to relaxation depends on the tendon type and the initial stress in the tendon. For low relaxation strands this loss may be taken as 5% of the jacking force.

The loss of force in the tendon due to creep and shrinkage is calculated by multiplying together the strain due to these effects, the elastic modulus of the tendon and the total area of the tendons. Creep and shrinkage strains depend on the type of concrete and its environment. In the absence of more specific data, the shrinkage strain can be obtained from Table 8.6. The strain due to creep is proportional to the stress in the concrete and is calculated by dividing a creep coefficient by the 28-day elastic modulus of the concrete, i.e.:

$$\text{Creep strain} = \frac{\text{creep coefficient}}{E_c} \times \text{(stress in the concrete at transfer at the level of the tendons)}$$

In the absence of more specific data, the creep coefficient can be obtained from Table 8.7. For unbonded tendons the creep strain should be the mean value averaged over the length of the tendon.

Add together the losses due to relaxation, shrinkage and creep to give the long-term (time-dependent) losses and subtract them from the initial force, $P_{m,0}$, to give the final prestressing force, $P_{m,\infty}$.

Table 8.6 Shrinkage strains

Location of the member	Relative humidity, %	Notional size $2A_c/u$, mm	
		≤150	≥600
Indoors	50	-600×10^{-6}	-500×10^{-6}
Outdoors	80	-330×10^{-6}	-280×10^{-6}

where: A_c is the cross-sectional area of concrete, and
u is the perimeter of that area.

Table 8.7 Creep coefficients

Age at loading, days	Notional size $2A_c/u$, mm					
	50	150	600	50	150	600
	Indoors (RH = 50%)			Outdoors (RH = 80%)		
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2.0
28	3.0	2.5	2.0	1.9	1.7	1.5
90	2.4	2.0	1.6	1.5	1.4	1.2

where: A_c is the cross-sectional area of concrete, and
u is the perimeter of that area.

8.3.3 Analysis of the structure

8.3.3.1 General

The structure should be analysed for the following loads as appropriate:

- dead load
- imposed load
- wind load
- snow load
- secondary prestress effects.

The secondary prestress effects can be determined by treating the prestress as a series of equivalent loads (Fig. 8.3) and subtracting the primary effects from the calculated forces in the member (see Section 6 for an explanation of primary and secondary prestress effects).

For members subjected to predominantly uniformly distributed loads, the loading arrangements to be considered are:

- alternate spans carrying the maximum design dead and imposed load, other spans carrying the maximum design dead load
- any two adjacent spans carrying the maximum design dead and imposed load, other spans carrying the maximum design dead load.

The maximum and minimum bending moments and maximum shear force in the prestressed concrete member in its final (in service) state should be determined. In addition the bending moments in the member at the time that the prestressing force is transferred to the member (at transfer) should also be calculated.

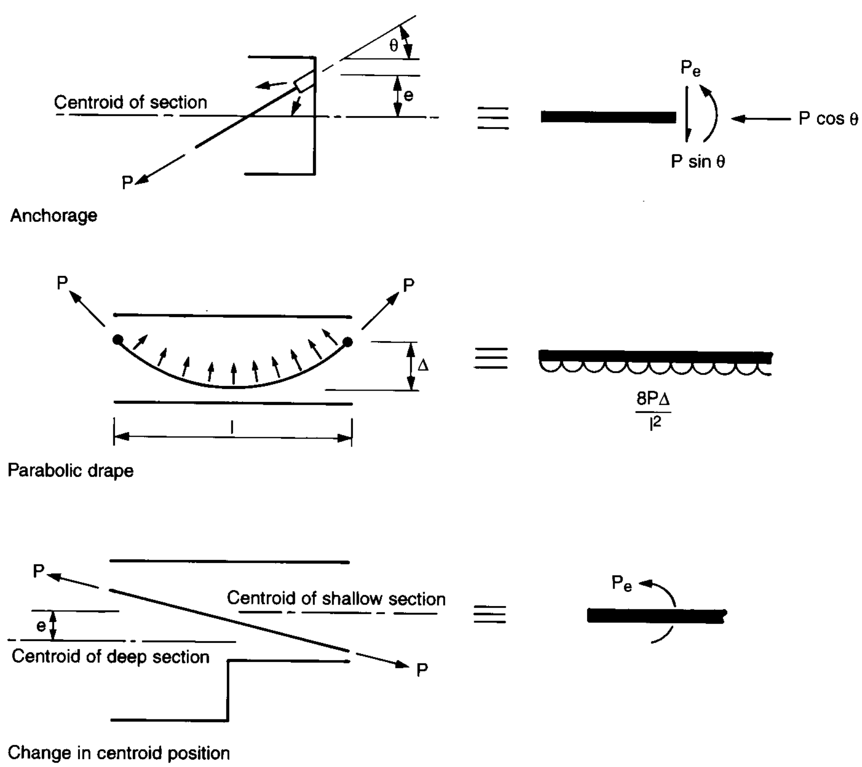


Fig. 8.3 Equivalent loads

The load factors and combinations to be used at the ultimate limit state are given in Table 6.3, while those to be used at the serviceability limit state are given in Table 6.5. The load factors to be applied to the prestressing loads are explained in clauses 8.3.4 and 8.3.5.

8.3.3.2 Beams and one-way solid slabs

Elastic analysis should be used to obtain bending moments and shear forces by either:

(a) considering the beam or slab as part of a structural frame as described in subsection 5.3, or

(b) considering the beam or slab as an individual member.

Redistribution of bending moments may only be carried out at the ultimate limit state as described in clause 5.3.3.

8.3.3.3 Two-way solid slabs on linear supports

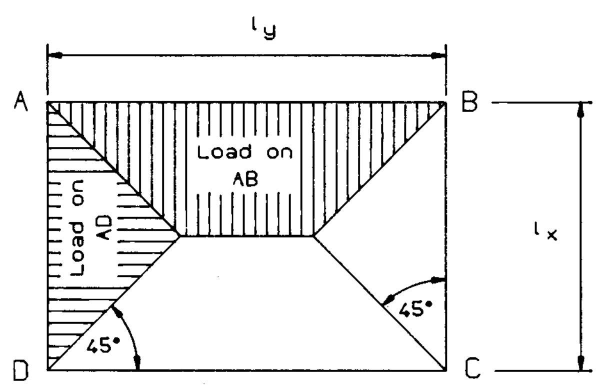
The coefficients in Table 8.8 may be used to obtain bending moments per unit width (m_{sx} and m_{sy}) in the two directions for various edge conditions, i.e:-

$$m_{sx} = b_{sx} w l_x^2$$
$$m_{sy} = b_{sy} w l_{x2}$$

where:	β_{sx} and β_{sy}	are coefficients given in Table 8.8
	w	is the design load per unit area
	l_x	is the shorter span, and
	ly	is the longer span.

No redistribution of moments should be carried out.

The distribution of the reactions of two-way slabs on to their supports can be derived from Fig. 8.4.



Notes

1. The reactions shown apply when all edges are continuous (or discontinuous)
2. When one edge is discontinuous, the reactions on all continuous edges should be increased by 10% and the reaction on the discontinuous edge may be reduced by 20%.
3. When adjacent edges are discontinuous, the reactions should be adjusted for elastic shear considering each span separately.

Fig. 8.4 Distribution of reactions from two-way slabs onto supports

Table 8.8 Bending moment coefficients for two-way rectangular slabs

Type of panel and moments considered	Limit state	Short span coefficients, β_{2x} Values of l_y/l_x					Long span coefficients, β_{syx}, for all values of l_y/l_x
		1.00	1.25	1.50	1.75	2.00	
Interior panels							
Negative moment at continuous	*SLS*	0.033	0.047	0.059	0.071	0.083	0.033
edge	ULS	0.031	0.044	0.053	0.059	0.063	0.032
Positive moment at midspan	*SLS*	0.025	0.036	0.045	0.053	0.062	0.025
	ULS	0.024	0.034	0.040	0.044	0.048	0.024
One short edge discontinuous							
Negative moment at continuous	*SLS*	0.041	0.055	0.065	0.075	0.085	0.041
edge	ULS	0.039	0.050	0.058	0.063	0.067	0.037
Positive moment at midspan	*SLS*	0.031	0.042	0.049	0.056	0.064	0.031
	ULS	0.029	0.038	0.043	0.047	0.050	0.028
One long edge discontinuous							
Negative moment at continuous	*SLS*	0.041	0.055	0.065	0.075	0.085	0.041
edge	ULS	0.039	0.059	0.073	0.082	0.089	0.037
Positive moment at midspan	*SLS*	0.031	0.042	0.049	0.056	0.064	0.031
	ULS	0.030	0.045	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous							
Negative moment at continuous	*SLS*	0.049	0.064	0.073	0.082	0.090	0.049
edge	ULS	0.047	0.066	0.078	0.087	0.093	0.045
Positive moment at midspan	*SLS*	0.037	0.049	0.056	0.062	0.068	0.037
	ULS	0.036	0.048	0.059	0.065	0.070	0.034
Two short edges discontinuous							
Negative moment at continuous	*SLS*	0.057	0.067	0.073	0.077	0.080	–
edge	ULS	0.046	0.056	0.062	0.067	0.070	–
Positive moment at midspan	*SLS*	0.044	0.050	0.055	0.058	0.060	0.044
	ULS	0.034	0.042	0.047	0.050	0.053	0.034
Two long edges continuous							
Negative moment at continuous	*SLS*	–	–	–	–	–	0.056
edge	ULS	–	–	–	–	–	0.045
Positive moment at midspan	*SLS*	0.044	0.063	0.071	0.077	0.080	0.044
	ULS	0.034	0.061	0.078	0.091	0.100	0.034
Three edges discontinuous (one long edge continuous)							
Negative moment at continuous	*SLS*	0.058	0.074	0.085	0.092	0.098	–
edge	ULS	0.057	0.074	0.084	0.092	0.098	–
Positive moment at midspan	*SLS*	0.044	0.056	0.064	0.070	0.074	0.044
	ULS	0.043	0.055	0.063	0.069	0.074	0.044
Three edges discontinuous (one short edge continuous)							
Negative moment at continuous	*SLS*	–	–	–	–	–	0.058
edge	ULS	–	–	–	–	–	0.058
Positive moment at midspan	*SLS*	0.044	0.056	0.064	0.070	0.074	0.044
	ULS	0.042	0.067	0.084	0.096	0.105	0.044
Four edges discontinuous							
Positive moment at midspan	*SLS*	0.050	0.065	0.075	0.082	0.083	0.050
	ULS	0.055	0.078	0.092	0.103	0.111	0.056

Note: This table is based on coefficients given in reference 20.

8.3.3.4 Flat slabs

The structure should be considered as being divided longitudinally and transversely into frames consisting of columns and strips of slab. The width of slab contributing to the effective stiffness should be the full width of the panel for vertical loads and 50% of this value for horizontal loads. The bending moments and shear forces should be obtained by elastic analysis.

In general, moments will be able to be transferred only between a slab and an edge or corner column by a strip of slab considerably narrower than the full panel width. This will limit the maximum design moment that can be transferred to the column to the values given in subclause 5.2.3.4. Where this limiting moment is exceeded, the moments obtained from the frame analysis should be adjusted at the columns to the above values and the midspan moments increased accordingly.

No further redistribution is allowed at the serviceability limit state, but at the ultimate limit state redistribution up to a maximum of 30% may be carried out for braced structures as described in clause 5.3.3.

8.3.3.5 Ribbed slabs

The bending moments and shear force per rib should be obtained by multiplying the values for solid slabs by the rib spacing.

8.3.4 Serviceability limit state

This clause applies to prestressed concrete members with either **bonded** or **unbonded** tendons. For section design of flanged beams, the effective width (see Fig 8.5) should be taken from Table 8.9, in which l is the length of the span or cantilever and b_w is the width of the web.

8.3.4.1 Concrete stresses

Calculate the compressive stresses in the concrete assuming an uncracked section using 1.0 times the mean value of the prestressing force ($P_{m,\infty}$) for both primary and secondary effects and the rare combination of loads, see clause 6.2.2 and Table 8.10, and compare them with the allowable values given in Table 8.11. The limiting tensile stresses are governed by crack control, see subclause 8.3.4.2.

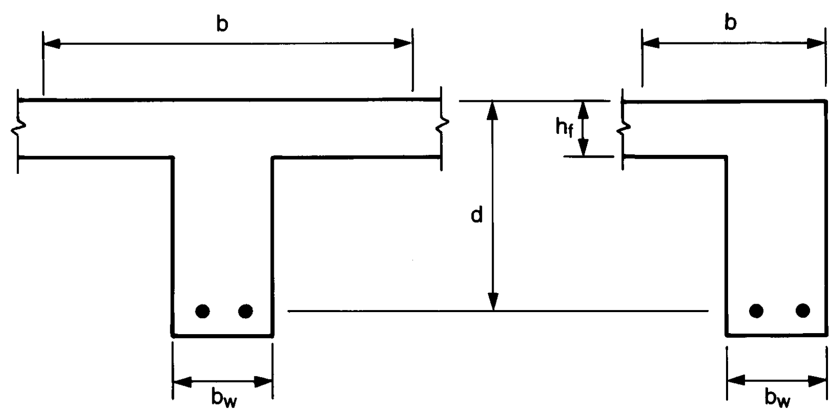


Fig. 8.5 Flanged beam sections

Table 8.9 Effective widths of flanged beams

	T-beam	L-beam
End span	$b_w + 0.17l$	$b_w + 0.085l$
Interior span	$b_w + 0.14l$	$b_w + 0.07l$
Cantilever	$b_w + 0.20l$	$b_w + 0.10l$

Notes to Table 8.9

1. The ratio of adjacent spans should lie between 1.0 and 1.5.
2. The length of the cantilever should be less than half the adjacent span.
3. The actual flange width should be used where it is less than the values obtained from the table.

8.3.4.2 Crack control

(a) A minimum area of reinforcement should be provided in the tensile zone (see sub-clause 8.3.5.3), except where the concrete remains in compression under the rare combination of loads (see clause 6.2.2 and Table 8.10).

(b) Check the more tensile face of prestressed concrete members to ensure that either no tensile stresses arise or that design crack widths are not exceeded, depending on the condition of exposure of the member, see Table 8.2. The checks are carried out using the frequent combination of loads (for members with bonded tendons) or the quasi-permanent combination of loads (for members prestressed with only unbonded tendons, (see clause 6.2.2) and either 0.9 or 1.1 times the mean prestressing force ($P_{m,\infty}$), whichever is more critical, for both primary and secondary effects. Generally a value of 0.9 should be used as the tendon will be located near the more tensile face and will have a beneficial effect on the member.

Calculate the compressive (f_{cc}) and tensile (f_{ct}) stresses in the concrete assuming an uncracked section, see Table 8.10. If f_{ct} is less than the cracking stress (f_{ctm}), see Table 8.11, then the moment on the section should be increased until $f_{ct} = f_{ctm}$. Calculate F^{t}, the tensile force within the tensile zone when $f_{ct} = f_{ctm}$. The stress in the reinforcement should be calculated as follows:

(i) Calculate

$$\beta_1 = 1 + \frac{M}{Pd} - \frac{e}{d} - \frac{h}{2d} \qquad \text{see Fig. 8.6}$$

(ii) Select a value of x/d

(iii) Calculate

$$\beta_2 = \frac{x}{2d}\left(1 - \frac{x}{3d}\right)$$

$$f_{cc} = \frac{\beta_1}{\beta_2}\frac{P}{bd}$$

$$f_{st} = \frac{0.5bxf_{cc} - P}{A_s + A_p}$$

where: A_s is the area of ordinary reinforcement (this could be zero when pretensioned tendons are used, but ordinary reinforcement will be required when post-tensioned tendons are employed)

A_p is the area of bonded prestressing tendons

Table 8.10 Calculation of concrete stresses

Clause	Location†	Stress	
8.3.4.1 and 8.3.4.3	Same	Maximum compressive stress	$f_{cc} = \left[P\left(\frac{1}{A_c}+\frac{e}{Z_c}\right)-\frac{M_s}{Z_c}\right]-\left\{\frac{M_r^- \text{ or } M_t}{Z_c}\right\}$
	Opposite	Maximum compressive stress	$f_{cc} = \left[P\left(\frac{1}{A_c}-\frac{e}{Z_c}\right)+\frac{M_s}{Z_c}\right]+\left\{\frac{M_r^+ \text{ or } M_t}{Z_c}\right\}$
8.3.4.2 (a)	Same	Minimum compressive stress	$f_{cc,min} = 0.9\left[P\left(\frac{1}{A_c}+\frac{e}{Z_c}\right)-\frac{M_s}{Z_c}\right]-\frac{M_r^+}{Z_c}$
	Opposite	Minimum compressive stress	$f_{cc,min} = 1.1^*\left[P\left(\frac{1}{A_c}-\frac{e}{Z_c}\right)+\frac{M_s}{Z_c}\right]+\frac{M_r^-}{Z_c}$
8.3.4.2 (b) and 8.3.4.3	Same	Maximum tensile stress	$f_{ct} = 0.9\left[P\left(\frac{1}{A_c}+\frac{e}{Z_c}\right)-\frac{M_s}{Z_c}\right]-\left\{\frac{M_f^+ \text{ or } M_q^+ \text{ or } M_t}{Z_c}\right\}$
	Opposite	Corresponding compressive stress	$f_{cc} = 0.9\left[P\left(\frac{1}{A_c}-\frac{e}{Z_c}\right)+\frac{M_s}{Z_c}\right]+\left\{\frac{M_f^+ \text{ or } M_q^+ \text{ or } M_t}{Z_c}\right\}$
	Opposite	Maximum tensile stress	$f_{ct} = 1.1^*\left[P\left(\frac{1}{A_c}-\frac{e}{Z_c}\right)+\frac{M_s}{Z_c}\right]+\left\{\frac{M_f^- \text{ or } M_q^- \text{ or } M_t}{Z_c}\right\}$
	Same	Corresponding compressive stress	$f_{cc} = 1.1^*\left[P\left(\frac{1}{A_c}+\frac{e}{Z_c}\right)-\frac{M_s}{Z_c}\right]-\left\{\frac{M_f^- \text{ or } M_q^- \text{ or } M_t}{Z_c}\right\}$

Notes to Table 8.10

1. 'Same' means that the stress is calculated on the same side of the neutral axis as the centroid of the tendons;
   'Opposite' means that the stress is calculated on the opposite side of the neutral axis from the centroid of the tendons.
2. * Normally a factor of 1.1 should be used, but when the term in [ ] is positive the factor should be 0.9.
3. Compressive stresses are positive.
4. P = $P_{m,0}$ when checking at transfer; $P_{m,\infty}$ when checking at the serviceability limit state
   A_c = area of the concrete section
   Z_c = the appropriate section modulus of the concrete section
   M_s = secondary or parasitic moment corresponding to $P_{m,0}$ or $P_{m,\infty}$ as appropriate, positive when it produces tension on the same side of the neutral axis as the centroid of the tendons
   M_t = bending moment due to loads other than prestress acting at the time the prestress force is **transferred** to the concrete member, positive when it produces tension on the same side of the neutral axis as the centroid of the tendons
   M_r^+, M_r^- = maximum and minimum bending moments due to the **rare** combination of loads at the serviceability limit state, positive when it produces tension on the same side of the neutral axis as the centroid of the tendons
   M_f^+, M_f^- = maximum and minimum bending moments due to the **frequent** combination of loads at the serviceability limit state, positive when it produces tension on the same side of the neutral axis as the centroid of the tendons
   M_q^+, M_q^- = maximum and minimum bending moments due to the **quasi-permanent** combination of loads at the serviceability limit state, positive when it produces tension on the same side of the neutral axis as the centroid of the tendons

Table 8.11 Allowable concrete stresses

Strength class or strength at transfer f_{ck}, N/mm^2	C30/37 30.0	C35/45 35.0	C40/50 40.0	C50/60 50.0
Allowable *compressive* stress in service, N/mm^2	18.0	21.0	24.0	30.0
Allowable *compressive* stress at transfer, N/mm^2	13.5	15.8	18.0	22.5
f_{ctm}, N/mm^2	2.9	3.2	3.5	4.1

(iv) Calculate

$$\frac{x}{d} = \frac{1}{1 + \dfrac{f_{st}}{8f_{cc}}}$$

If this value is within 5% of the assumed value for x/d, accept the result for f_{st}. Otherwise, adjust x/d and repeat steps (iii) and (iv).

Calculate the stress in the ordinary reinforcement (σ_s) and the increase in stress in bonded prestressing tendons ($\Delta\sigma_p$).

$$\sigma_s = f_{st} + 0.4 f_{ctm}\left(\frac{1}{eff\rho_p} - \frac{1}{eff\rho_{tot}}\right)$$

$$\Delta\sigma_p = f_{st} - 0.4 f_{ctm}\left(\frac{1}{eff\rho_{tot}} - \frac{\xi_1^2}{eff\rho_{tot}}\right)$$

where

$$eff\rho_p = \frac{A_s + \xi_1^2 A_p}{A_{c.eff}}$$

$$eff\rho_{tot} = \frac{A_s + A_p}{A_{c.eff}}$$

$$A_{c.eff} = 2.5b(h-d) \le \frac{F^1}{f_{ctm}}$$

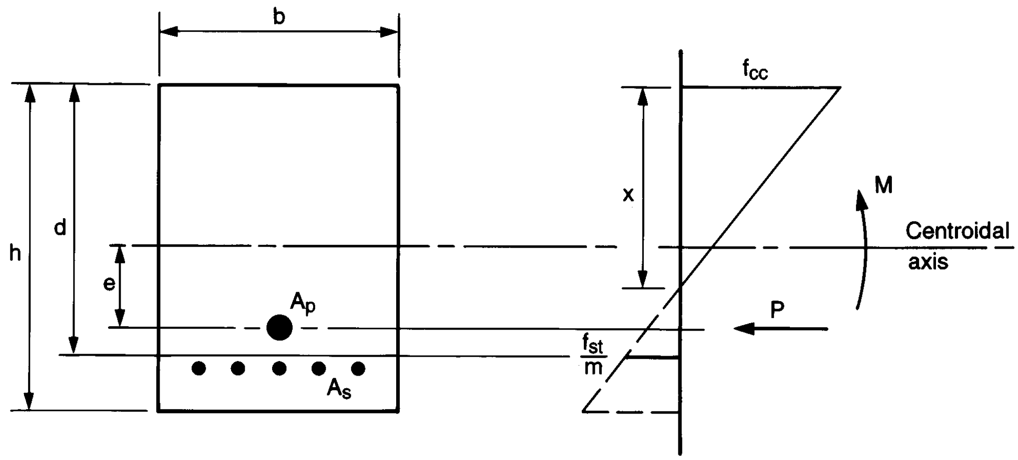
$\xi_1 = \sqrt{\xi \dfrac{\phi_s}{\phi_p}}$ If only prestressing tendons are used to control cracking, then $\xi_1 = 1.0$.

ϕ_s is the diameter of ordinary reinforcement
ϕ_p is the equivalent diameter of prestressing tendons

$\phi_p = 1.6\sqrt{A_p}$ for tendons with several strands, each tendon of area A_p

$\phi_p = 1.75\phi_{wire}$ for single strands with 7 wires, each of diameter f_{wire}, and
ξ is the ratio of bond strength of prestressing steel and high bond reinforcing steel which should be taken as 0.7 for pretensioned members and 0.6 for post-tensioned members.

The maximum allowable bar diameter **or** bar spacing is obtained from Table 8.12 for members with bonded tendons, using σ_s, except when only prestressing strand is used when $\Delta\sigma_p$ should be used. For members prestressed with only unbonded tendons, the maximum allowable bar diameter or bar spacing is obtained from Table 8.13.



d = depth to centroid of all bonded reinforcement (ordinary and prestressed)
e = eccentricity of tendon
P = prestressing force including factor of 0.9 or 1.1, as appropriate
M = applied bending moment, including secondary effects
f_{st} = steel stress
m = modular ratio which should be taken as equal to 8
f_{cc} = compressive stress in concrete

Fig. 8.6 Parameters for calculating reinforcement stresses

Table 8.12 Alternative requirements to control crack widths for members with bonded tendons

Steel stress σ_s or $\Delta\sigma_p$, N/mm^2	Maximum bar diameter ϕ_s or ϕ_p, mm		Maximum bar spacing, mm
100	50		200
120	40		200
140	32		200
160	25		200
200	16	OR	150
240	12		100
280	8		50
320	6		–
360	5		–
400	4		–

Notes to Table 8.12

1. When only prestressing tendons are used to control cracking then the maximum bar diameter (ϕ_p) must be multiplied by ξ.
2. When only prestressing tendons are used to control cracking then the maximum bar spacing must be multiplied by 0.5.
3. It is only necessary to satisfy one of the requirements in the table, either the maximum bar spacing or the maximum bar diameter.

8.3.4.3 Transfer

The condition immediately after the tendons have been stressed (transfer) must also be checked. The loading to be considered is that resulting from the initial tendon force profile ($P_{m,0}$) and the self-weight of the concrete.

Calculate the maximum concrete compressive stress using 1.0 times the mean prestressing force for both primary and secondary effects, see Table 8.10; this should not exceed the value given in Table 8.11, corresponding to the cylinder strength, f_{ck}, of the concrete at the time the prestress force is transferred to the concrete.

Crack widths should also be checked, as described in subclause 8.3.4.2(b). At transfer, the more tensile face of the member will generally be on the far side of the neutral axis from the tendon. The prestressing force will therefore have an adverse effect and 1.1 times the mean value of the prestressing force ($P_{m,0}$) should be used for both primary and secondary effects, see Table 8.10.

8.3.5 Ultimate limit state

Calculate the ultimate design moment due to dead and imposed loads and secondary prestress effects, *M*, using Table 6.3 with a load factor of 1.0 applied to the secondary prestressing effects.

The most common beams have flanges at the top. At supports they are designed as rectangular beams and in the spans as flanged beams. For upstand beams the reverse applies. For section design the effective width of a flanged beam should be taken as the actual width*.

The formulae for the resistance moments of concrete sections are based on the stress diagram of Fig 8.7.

8.3.5.1 Bonded tendons

The resistance moment of concrete sections, prestressed with bonded tendons, that are required to resist flexure only can be determined from the following formulae.

Table 8.13 Alternative requirements to control crack widths for members with unbonded tendons

Steel stress σ_s, N/mm^2	Maximum bar diameter ϕ_s, mm		Maximum bar spacing, mm
160	32		300
200	25		250
240	20		200
280	16	OR	150
320	12		100
360	10		50
400	8		–
450	6		–

Note: It is only necessary to satisfy one of the requirements in the table, either the maximum bar spacing or the maximum bar diameter.

* This is correct and is allowed in EC2, Part 2 as long as the serviceability limit state is also checked; EC2, Part 1 strictly requires the effective width given in Table 8.9 to be used.

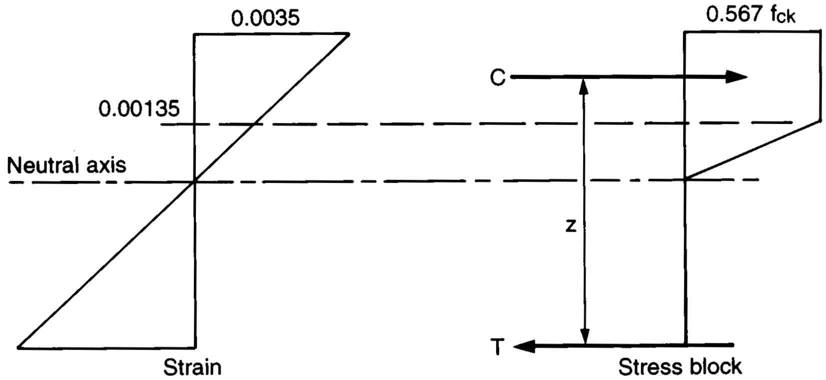


Fig. 8.7 Stress diagram – bilinear stress block

Rectangular beams

The procedure for the design of rectangular beams is as follows:

(a) Calculate the ratio of the stress in the prestressing tendons after all losses to the characteristic strength of the tendons, f_{pe}/f_{pk}.
(b) Calculate $K = M/bd^2 f_{ck}$, where f_{ck} is the concrete cylinder strength.
(c) Determine the limiting value of x/d from Table 8.14 for the appropriate value of f_{ck} and percentage of redistribution (generally zero for prestressed concrete members). Hence determine K_{lim} corresponding to the appropriate value of f_{pe}/f_{pk} from Fig. 8.8.
(d) If $K = K_{lim}$, determine $A_p f_{pk}/bdf_{ck}$ corresponding to the appropriate value of f_{pe}/f_{pk} from Fig. 8.9.
(e) Hence determine the area of prestressing tendons required, A_p.
(f) If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s, can be taken into account by replacing it with an equivalent area of prestressing tendons $A_s f_{yk}/f_{pk}$.
(g) If $K > K_{lim}$ then compression reinforcement is required. The area of ordinary compression reinforcement, A'_s, is calculated from:

$$A'_s = \frac{M - K_{lim}\, bd^2 f_{ck}}{0.87 f_{yk}\,(d - d')}$$

where d' is the depth to the centre of the compression reinforcement from the com pression face.

If $d' > \left(1 - \frac{f_{yk}}{800}\right)x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.87 f_{yk}$.

Table 8.14 Limiting values of *x/d*

Percentage of moment redistribution, %	0	5	10	15	20	25	30
$f_{ck} \leq$ 35N/mm^2	0.45	0.41	0.37	0.33	0.29	0.25	0.21
$f_{ck} \geq$ 40N/mm^2	0.35	0.31	0.27	0.23	0.19	0.15	0.11

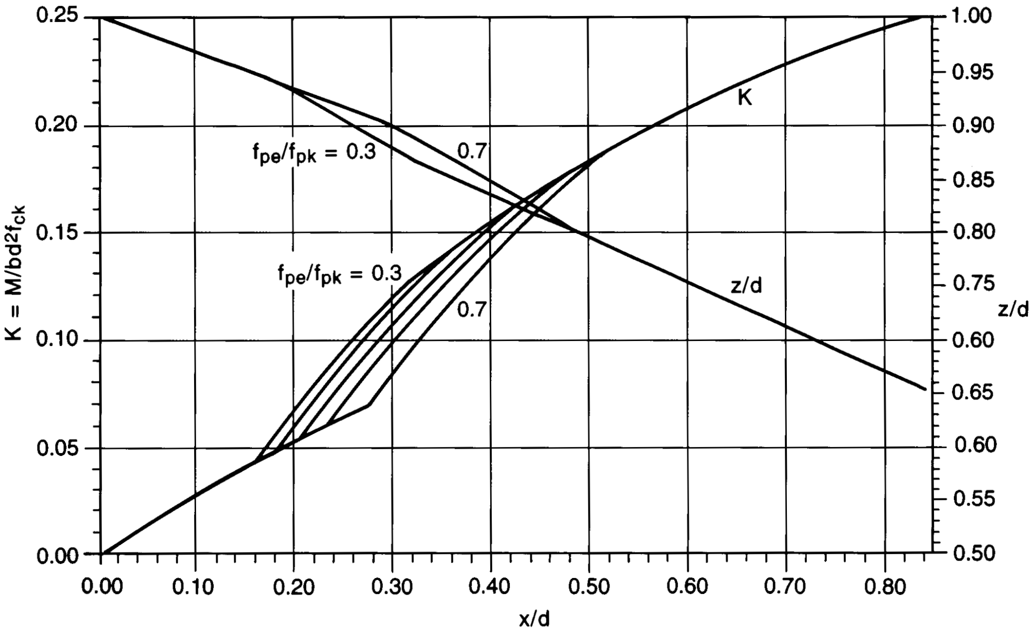


Fig. 8.8 Neutral axis depth and lever arm factors for prestressed concrete rectangular sections with bonded tendons

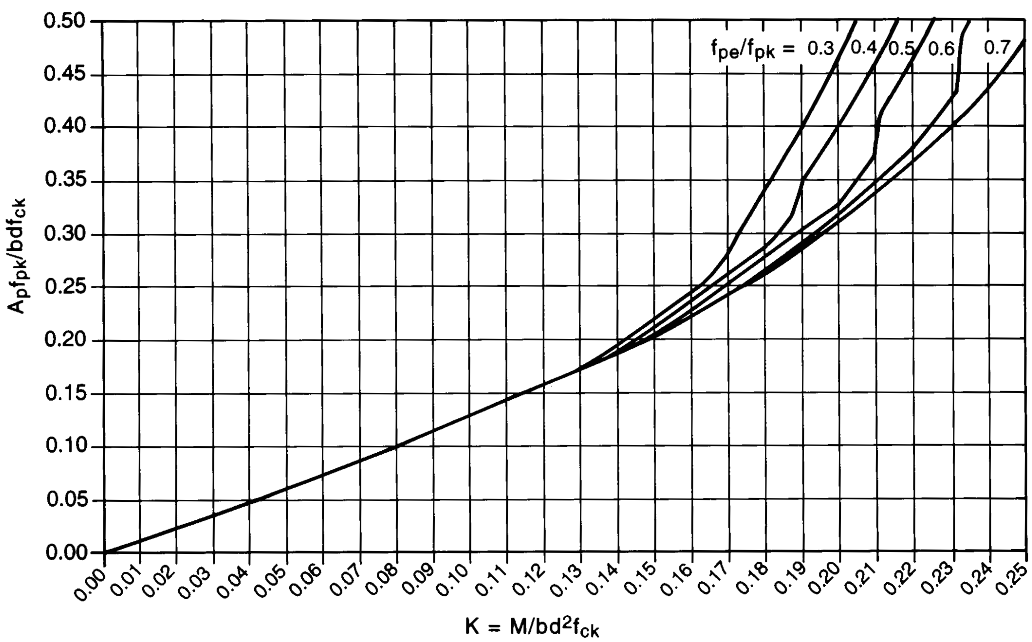


Fig. 8.9 Design chart for prestressed concrete rectangular sections with bonded tendons

(h) Determine A_p corresponding to K_{lim} from Fig. 8.9 and z/d corresponding to the limiting value of x/d from Fig. 8.8. The required area of prestressing tendons in the tension zone is given by:

$$A_p\left(1+\frac{0.87A'_s\,f_{yk}\,z}{k_{lim}\,bd^2\,f_{ck}}\right)$$

(i) If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s, can be taken into account by replacing it with an equivalent area of prestressing tendons $A_s f_{yk}/f_{pk}$.

Flanged beams

The procedure for the design of flanged beams is as follows:

(a) Calculate the ratio of the stress in the prestressing tendons after all losses to the characteristic strength of the tendons, f_{pe}/f_{pk}.

(b) Check the position of the neutral axis by determining $K = M/bd^2f_{ck}$ using flange width b, where f_{ck} is the cylinder strength, and selecting x/d from Fig. 8.8. Check that x/d is less than the limiting value obtained from Table 8.14, otherwise redesign the section. Calculate x.

(c) If $0.8x \leq h_f$, the depth of the flange, then A_p is determined as for a rectangular beam of breadth b.

(d) If $0.8x > h_f$, the stress block lies outside the flange. Calculate the resistance moment of the flange, M_{uf}, from

$$M_{uf} = 0.567f_{ck}(b - b_w)h_f(d - 0.5h_f)$$

(e) Calculate

$$k_w = \frac{(M - M_{uf})}{f_{ck}\,b_w\,d^2}$$

If $K_w \leq K_{lim}$, obtained as for a rectangular beam of width b_w, then determine z/d from Fig. 8.8 and A_pf_{pk}/bdf_{ck} from Fig. 8.9 corresponding to K_w and f_{pe}/f_{pk}; otherwise redesign the section.

(f) Calculate the stress in the prestressing tendons, f_p, at the ultimate limit state from

$$f_p = \frac{K_w}{\left(\frac{z}{d}\right)}\left(\frac{bdf_{ck}}{A_p\,f_{pk}}\right)f_{pk}$$

and determine A_p from

$$A_p = \frac{M_{uf}}{f_p\left(d - 0.5h_f\right)} + \frac{(M - M_{uf})}{f_p\,z}$$

(g) If this is less than the area being provided for serviceability the section is satisfactory at the ultimate limit state. Otherwise ordinary reinforcement must be added in the tension zone of the section. The area of ordinary reinforcement, A_s, can be taken into account by replacing it with an equivalent area of prestressing tendons A_sf_{yk}/f_{pk}.

8.3.5.2 Unbonded tendons

The prestressing force, P_d, should be considered as an external axial load, acting at the appropriate eccentricity, with a value of:

$$P_d = 0.9P_{m,\infty} + \Delta\sigma A_p$$

where: $P_{m,\infty}$ is the prestress force after all losses,
A_p is the area of prestressing tendons, and
$\Delta\sigma$ = 100N/mm^2 when the length of the tendon does not exceed a single span, otherwise 0N/mm^2.

The resistance moment of concrete sections, prestressed with unbonded tendons, that are required to resist flexure only can be determined from the following formulae.

Rectangular beams

The procedure for the design of rectangular beams is as follows:

(a) Calculate $K = M/bd^2 f_{ck}$, where f_{ck} is the concrete cylinder strength.

(b) Determine the limiting value of x/d from Table 8.14 for the appropriate value of f_{ck} and percentage of redistribution (generally zero for prestressed concrete members). Hence determine K_{lim} from Fig. 8.10

(c) If $K \leq K_{lim}$, the area of tension reinforcement, A_s, is calculated from:

$$A_s = \frac{M/z - P_d}{0.87 f_{yk}}$$

where z is obtained from Fig.8.10 corresponding to the appropriate value of K.

(d) If $K > K_{lim}$, then compression reinforcement is needed. The area of ordinary compression reinforcement, A'_s, is calculated from:

$$A'_s = \frac{M - K_{lim}\, bd^2 f_{ck}}{0.87 f_{yk}(d - d')}$$

where d' is the depth to the centre of the compression reinforcement from the compression face.

If $d' > \left(1 - \frac{f_{yk}}{800}\right)x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.87 f_{yk}$.

(e) The area of tension reinforcement, A_s, is calculated from:

$$A_s = A'_s + \frac{\frac{K_{lim}\, bd^2 f_{ck}}{z} - P_d}{0.87 f_{yk}}$$

where z is obtained from Fig. 8.10 corresponding to the value of K_{lim}.

Flanged beams

The procedure for the design of flanged beams is as follows:

(a) Check the position of the neutral axis by determining $K = M/bd^2 f_{ck}$ using flange width b, where f_{ck} is the concrete cylinder strength, and selecting values of x and z from Fig. 8.10. Check that x/d is less than the limiting value obtained from Table 8.14, otherwise redesign the section.

(b) If $0.8x \leq h_f$, the depth of the flange, then A_s is determined as for a rectangular beam of breadth b.

(c) If $0.8x > h_f$, the stress block lies outside the flange. Calculate the resistance moment of the flange, M_{uf}, from:

$$M_{uf} = 0.567 f_{ck}(b - b_w)h_f(d - 0.5h_f)$$

(d) Calculate

$$K_w = \frac{(M - M_{uf})}{f_{ck}\, b_w\, d^2}$$

If $K_w \leq K_{lim}$, obtained from Fig. 8.10 as for a rectangular beam of width b_w, then select z/d corresponding to K_w from Fig. 8.10 and hence determine z. Calculate A_s from:

$$A_s = \frac{\dfrac{M_{uf}}{(d - 0.5h_f)} + \dfrac{M - M_{uf}}{z} - P_d}{0.87f_{yk}}$$

If $K_w > K_{lim}$, redesign the section.

8.3.5.3 Minimum reinforcement

A minimum area of reinforcement should be provided in prestressed members in order to prevent brittle failure (i.e. without warning). The longitudinal tensile reinforcement should not be less than either

$$0.6b_t d/f_{yk} \text{ or } 0.0015b_t d$$

where b_t is the mean width of the tension zone.

In pretensioned members the tendons can be taken into account when calculating the minimum area of reinforcement provided. In post-tensioned members ordinary reinforcement must be provided in order to satisfy this condition.

Where the overall depth of a beam exceeds 1000mm and the main reinforcement is concentrated in only a small proportion of the depth, reinforcement should be provided in the side faces of the beam to control cracking. This longitudinal reinforcement should be placed within the links and should be evenly distributed between the level of the main reinforcement and the neutral axis. The area of reinforcement, A_s, is given by:

$$A_s = \frac{0.6b\,(0.83d - x)}{\sigma_s}$$

where σ_s is the stress induced into the reinforcement immediately after cracking in N/mm^2 and b, d and x are in mm. The value of b need not be taken as more than 500mm.

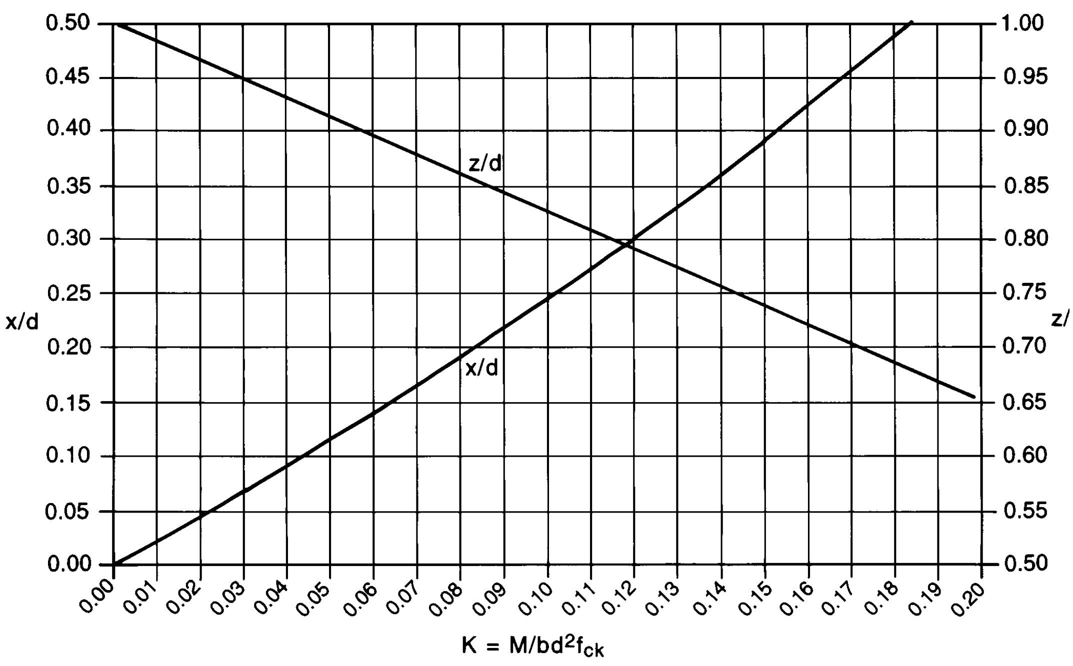


Fig. 8.10 Neutral axis depth and lever arm factors for prestressed concrete rectangular sections with unbonded tendons

When high bond bars are used, the procedure to be followed is:

(a) arbitrarily select a value for σ_s
(b) calculate A_s from the formula above
(c) obtain the maximum bar spacing corresponding to σ_s from Table 8.12 or Table 8.13, as appropriate
(d) obtain the maximum bar diameter corresponding to σ_s from Table 8.12 or Table 8.13, as appropriate
(a) check that reinforcement can be provided which satisfies either:

(b) and (c) **OR** (b) and (d)

If neither can be achieved, adjust σ_s and repeat steps (b) to (e).
If mild steel bars are used, reference should be made to EC2.

8.3.6 Tendon spacing

The minimum clear horizontal distance between pretensioned tendons should not be less than the diameter of the tendon or the maximum size of the aggregate plus 5mm, nor less than 20mm. In post-tensioned construction, the minimum clear horizontal distance between ducts should not be less than the outside diameter of the sheath or 40mm.

Where there are two or more rows the gaps between corresponding tendons or ducts in each row should be vertically in line. In members where internal vibration is being used the horizontal gaps should be wide enough to allow the passage of a poker vibrator. In pretensioned members, the clear vertical distance between tendons should not be less than the diameter of the tendon or the maximum size of the aggregate, nor less than 10mm. In post-tensioned construction, the clear vertical distance should not be less than the outside diameter of the sheath or 50mm.

When tendons are curved, the minimum clear spacing between tendons in the plane of curvature should be increased to prevent local failure of the concrete in accordance with Table 8.15.

In each direction of flat slabs, at least 50% of the tendons required in a panel should be placed symmetrically about the column centre-line within a width equal to the column size plus the depth of the slab. The remaining tendons should be evenly distributed throughout the rest of the panel, see Fig. 8.11.

8.4 Shear design

8.4.1 General

Shear checks for prestressed concrete members are carried out at the ultimate limit state. Calculate the design ultimate shear force, V_{Sd}, using Table 6.3, applying a load factor of 1.0 to the secondary prestressing effects. The effect of inclined prestressing tendons should be taken into account, thus:

$$V_{Sd} = V_{od} - V_{pd}$$

where: V_{od} is the design shear force in the section ignoring the effect of any inclined tendons, and
V_{pd} is the vertical component of the prestressing force, measured positive when acting in the same direction as $V_{od.}$

V_{pd} should be calculated using the prestress force after losses, $P_{m,\infty,}$ multiplied by the appropriate load factor which should normally be taken as 0.9, but when V_{pd} acts in the opposite sense to V_{od}, i.e. V_{Sd} is increased, it should be taken as 1.2.

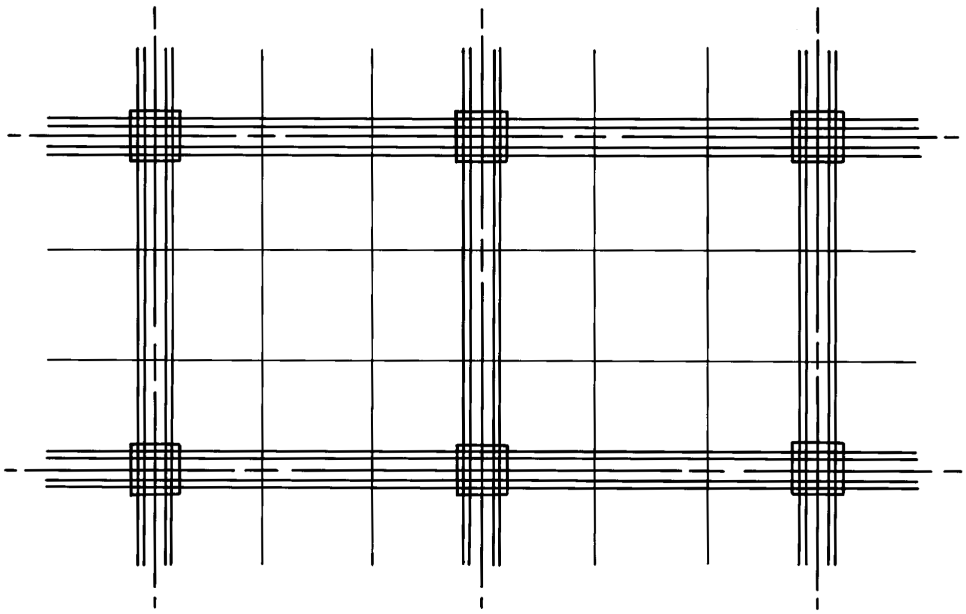


Fig. 8.11 Layout of prestressing tendons in flat slabs

Table 8.15 Minimum distance between centre-lines of ducts in plane of curvature, mm

Radius of curvature of duct, m	Duct internal diameter, mm									
	25	30	40	45	50	55	60	75	80	100
	Tendon force, kN									
	159	229	477	636	916	1113	1603	1908	2748	4351
2	80	95	180	240	Radii not normally used					
4		85	100	125	175	215	300	355		
6			95	100	125	145	205	245	345	535
8					110	125	160	190	265	420
10						120	140	170	215	330
12							130	160	185	280
14									180	245
16									170	230
18										220
20	80	85	95	100	110	120	130	160	170	215

Notes to Table 8.15

1. The table is based on reference 18.
2. The tendon force shown is the maximum normally available for the given size of duct.
3. Where tendon profilers or spacers are provided in the ducts, and these are of a type which will concentrate the radial force, the values given in the table will need to be increased. If necessary reinforcement should be provided between the ducts.
4. The distance for a given combination of duct internal diameter and radius of curvature shown in the table may be reduced in proportion to the tendon force when this is less than the value tabulated.

The effective depth, d, used in checking the shear capacity should be calculated ignoring any inclined tendons.

At any section, V_{Sd} should not exceed the maximum shear capacity, $V_{Rd2.red}$, given by:

$$V_{Rd2.red} = 1.67V_{Rd2}\left(\frac{1 - 1.5\sigma_{cp.eff}}{f_{ck}}\right) \leq V_{Rd2}$$

where: V_{Rd2} $= 0.15f_{ck}b_{w,nom}d$ (This assumes an effectiveness factor, $\nu = 0.5$)
$b_{w,nom}$ $= b_w - 0.5\Sigma\phi$ (for grouted ducts) or $b_w - \Sigma\phi$ (for non-grouted ducts)
b_w is the width of the beam web
ϕ is the diameter of grouted ducts which may be taken as zero when $\phi = b_w/8$, **or** the diameter of non-grouted ducts
$\sigma_{cp.eff}$ $= 1.2P_{m,\infty}/A_c$
$P_{m,\infty}$ is the prestress force after all losses, and
A_c is the total area of the concrete section.

For the design of shear reinforcement, V_{Sd} may be taken as that acting at a distance d from the face of a support.

Where V_{Sd} exceeds V_{Rd1}, the shear capacity of the section without shear reinforcement, shear reinforcement will be required. This may be assessed by either the standard method (see clause 8.4.2) or the variable-strut inclination method (see clause 8.4.3). The latter method may lead to savings in shear reinforcement, but will be subject to more severe curtailment rules (see clause 5.12.6).

V_{Rd1} is calculated from:

$$V_{Rd1} = v_{Rd1}b_wd$$

where: v_{Rd1} is obtained from Fig. 8.12
b_w is the width of the beam web

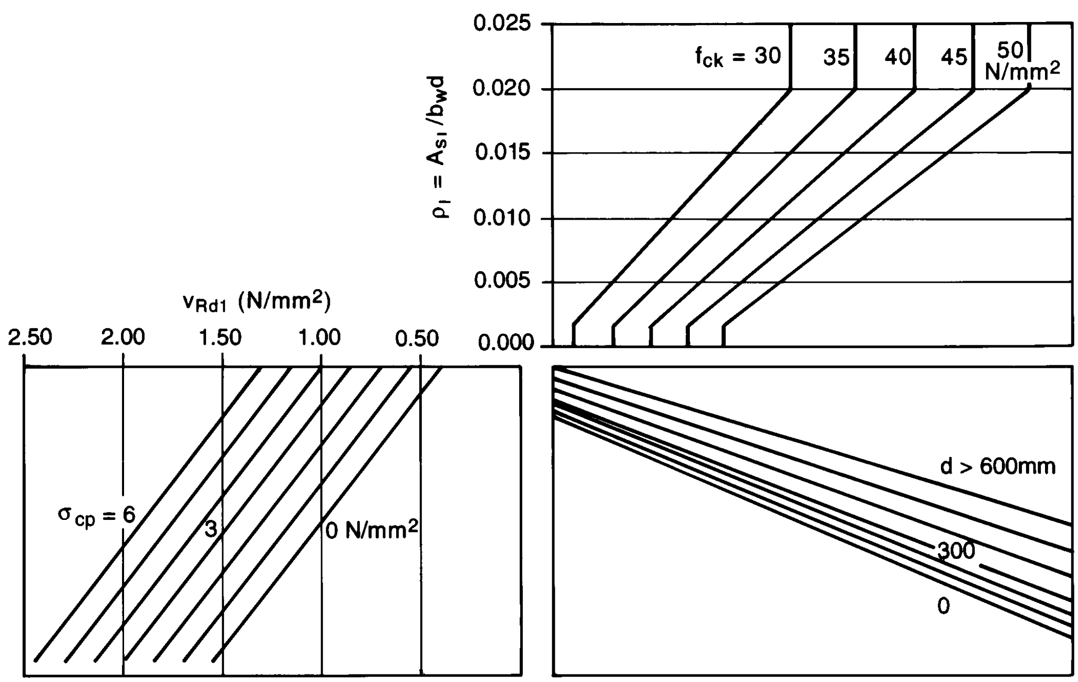


Fig. 8.12 Design chart to detrmine V_{Rd1} for prestressed concrete

A_{sl} is the total area of prestressed and ordinary reinforcement in the tension zone
f_{ck} is the concrete cylinder strength
σ_{cp} = $0.9P_{m,\infty}/A_c$
$P_{m,\infty}$ is the prestress force after all losses, and
A_c is the total area of the concrete.

Where loads are supported at the bottom of beams, the link should be designed to carry the applied loads in tension in addition to any shear forces.

Where concentrated loads are located at a distance x from the face of the support and x is less than or equal to $2.5d$, then the values of v_{Rd1} obtained from Fig. 8.12 may be multiplied by a factor β, where:

$$\beta = 2.5d/x \quad \text{with} \quad 1.0 \leq \beta \leq 5.0$$

When this enhancement is taken into account, V_{Rd1} and shear reinforcement should be calculated at all critical sections over the length $2.5d$ from the face of the support, with $\beta = 1.0$ on the span side of the relevant concentrated loads. The maximum shear reinforcement so obtained should be provided over this entire length.

8.4.2 Standard method

Shear reinforcement in the form of vertical links should be provided in accordance with

$$A_{sw} = \frac{1.28s\left(V_{sd} - V_{Rd1}\right)}{f_{ywk}\, d}$$

where: A_{sw} is the total cross-section of the link(s). (2 legs for a single closed link, 4 legs for a double closed link)
s is the longitudinal spacing of the links, and
f_{ywk} is the characteristic strength of the link.

Where loads are supported at the bottom of beams, the link should be designed to carry the applied loads in tension in addition to any shear forces.

The profile of the prestressing tendons and the curtailment of longitudinal reinforcement should satisfy the requirements of clause 5.12.6.

8.4.3 Variable-strut inclination method

In this method no allowance is made for the contribution of the concrete or the compressive stress due to prestressing. It will therefore be more conservative than the standard method where V_{Sd} is not much larger than V_{Rd1}.

Calculate the inclination of the strut, θ, from

$$\cot\theta = \frac{V_{Rd2.red}}{V_{Sd}} + \sqrt{\left(\frac{V_{Rd2.red}}{V_{Sd}}\right)^2 - 1} \leq 1.5$$

Shear reinforcement in the form of vertical links should be provided in accordance with:

$$A_{sw} = \frac{1.28sV_{sd}}{f_{ywk}\, d\cot\theta}$$

where: A_{sw} is the total cross-section of the link(s). (2 legs for a single closed link, 4 legs for a double closed link)
s is the longitudinal spacing of the links, and
f_{ywk} is the characteristic strength of the link.

Where loads are supported at the bottom of beams, the link should be designed to carry the applied loads in tension in addition to any shear forces.

The profile of the prestressing tendons and the curtailment of longitudinal reinforcement should satisfy the requirements of clause 5.12.6.

8.4.4 Shear in flat slabs

The critical consideration for shear in flat slab structures is that of punching shear around the columns. The shear forces should be increased to allow for the effects of moment transfer to the columns. The design effective shear force, V_{eff}, at the perimeter of the column should be taken as:

$V_{eff} =$ $1.15V_t$ for internal columns with approximately equal spans
$1.4V_t$ for edge columns
$1.5V_t$ for corner columns

where V_t is the design ultimate shear force transferred to the column and is calculated on the assumption that the maximum design load is applied to all panels adjacent to the column considered.

Shear stresses at the column perimeter should be checked first:

$$v_{sd,p} = \frac{V_{eff}}{u_c\, d} \leq 0.9\sqrt{f_{ck}}$$

where: u_c is the column perimeter
f_{ck} is the concrete cylinder strength in N/mm², and
d is the average of the effective depths of reinforcement/tendons in each direction.

The shear force should then be checked at successive perimeters. V_{eff} may be reduced by the load within the perimeter being considered. Where a column is close to a free edge, the effective length of a perimeter should be taken as the lesser of the two illustrated in Fig. 8.13.

When openings are less than six times the effective depth of the slab from the edge of a column then that part of the perimeter that is enclosed by radial projections from the centroid of the column to the openings should be considered ineffective as shown in Fig. 8.14.

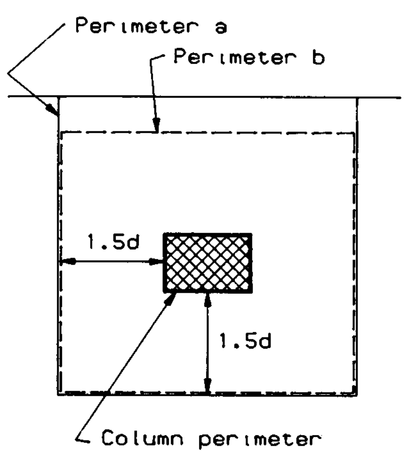


Fig. 8.13 Shear perimeter for edge column

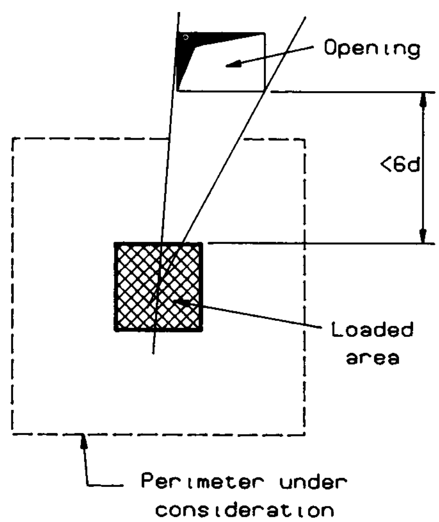


Fig. 8.14 Effect of opening on shear perimeter

Perimeter (b) in Fig. 8.15 is checked first. If the shear force, V_{eff}, is less than V_{Rd1} determined as described below, no further checks are required. If $V_{eff} > V_{Rd1}$, successive perimeters should be checked until one is reached where $V_{eff} \leq V_{Rd1}$.

V_{Rd1} should be calculated from:

$$V_{Rd1} = \sum v_{Rd1} ud$$

where: u is the shear perimeter defined in Fig. 8.13, Fig. 8.14 and Fig. 8.15

d is the average of the effective depths of reinforcement/tendons in each direction

v_{Rd1} is determined from Fig. 8.12 with A_{sl} / b_{wd} taken as equal to:

$$\frac{A_{s1}}{b_w d} = \sqrt{\rho_{1x} \rho_{ly}} + \frac{\sigma_{cp0}}{0.87 f_{yk}}$$

and $\sigma_{cp} = 0$

ρ_{lx}, ρ_{ly} are the longitudinal reinforcement ratios in the x- and y-directions, and $\sqrt{\rho_{1x} \rho_{iy}} \leq 0.015$

σ_{cp0} $= \dfrac{0.9 P_{m,0}}{A_c}$

f_{ck} is the concrete cylinder strength

f_{yk} is the characteristic strength of ordinary reinforcement

$P_{m,0}$ is the initial prestressing force over the full width of the panel, and

A_c is the cross-sectional area of the concrete panel.

Where the prestressing force is different in the x- and y- directions, v_{Rd1} should be determined separately for each direction

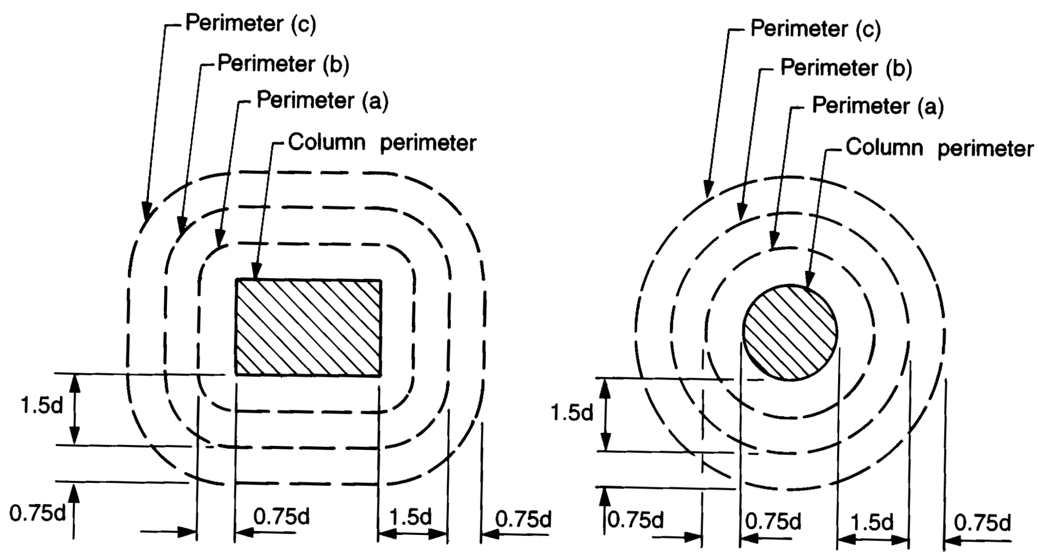


Fig. 8.15 Shear perimeters for internal columns

In slabs at least 200mm thick shear reinforcement should be provided within the area where the shear force V_{eff} exceeds V_{Rd1} but is less than $2V_{Rd1}$. If the shear force exceeds $2V_{Rd1}$, column heads or drop panels should be incorporated or the slab thickness increased so that $2V_{Rd1}$ becomes greater than the shear force.

Shear reinforcement should consist of vertical links. The total area required, A_{sw}, is calculated from the equations given below. These equations should not be applied where $V_{eff} > 2V_{Rd1}$.

where: $V_{Rdl} < V_{eff} \leq 1.6V_{Rdl}$ $\quad A_{sw} = \dfrac{V_{eff} - V_{Rd1}}{0.87f_{ywk}}$

where: $1.6V_{Rdl} < V_{eff} \leq 2V_{Rdl}$ $\quad A_{sw} = \dfrac{3.83\left(V_{eff} - 1.4V_{Rd1}\right)}{f_{ywk}}$

f_{ywk} is the characteristic strength of the link.

The reinforcement should be provided on at least two perimeters between the column perimeter and perimeter (b), Fig. 8.15. The first perimeter of reinforcement should be located approximately 0.5*d* from the face of the column area and should contain not less than 40% of A_{sw}. The second perimeter should be located at not more than 0.75*d* from the first. The spacing of the legs of the links around any perimeter should not exceed 0.6*d*.

The shear is now checked on perimeter (c). If reinforcement is required then this is provided between perimeters (a) and (c) in an analogous way to that used for the check on perimeter (b). The placing of shear reinforcement is shown in Fig. 8.16.

8.4.5 Detailing of shear reinforcement

8.4.5.1 Minimum reinforcement

Minimum shear reinforcement in beams should be provided in accordance with

$A_{sw} = \rho_W s b_w$

where: ρ_W is obtained from Table 8.16

s is the longitudinal spacing of the links, and

b_w is the width of the web.

The minimum area of reinforcement to resist punching shear should also be provided in accordance with Table 8.16, but with sb_w replaced by $(A_{crit} - A_{load})$, where A_{crit} is the area within the critical perimeter and A_{load} is the area under the load or the area of the column.

The diameter of the shear reinforcement should not exceed 12mm where it consists of plain round bars.

8.4.5.2 Maximum spacing

The maximum spacing of shear reinforcement is given in Table 8.17. $V_{Rd2.red}$ is defined in clause 8.4.1.

8.4.5.3 Arrangement of links

For compression reinforcement in an outer layer, every corner bar should be supported by a link passing round the bar and having an included angle of not more than 135°. A maximum of 5 bars in or close to each corner can be secured against buckling by any one set of links.

8.4.5.4 Openings

In locations where $V_{Sd} < V_{Rd1}$, small openings not exceeding 0.25_d in diameter can be permitted within the middle third of the depths of beams without detailed calculations. Where these conditions are not met, detailed calculations should be carried out.

Table 8.16 Minimum shear reinforcement ratio, ρ_w

Concrete class	Ratio, ρ_w f_{ywk} = 460 N/mm^2	f_{ywk} = 250N/mm^2
C30/37 to C35/45	0.0012	0.0022
C40/50 to C50/60	0.0015	0.0028

Table 8.17 Maximum spacing of shear reinforcement

Design shear force, V_{Sd}	Longitudinal spacing, *mm	Transverse spacing, mm
$\leq$ 0.2 $V_{Rd2.red}$	0.8$d \leq$ 300	$d \leq$ 800
> 0.2 $V_{Rd2.red}$ and $\leq$ 0.67 $V_{Rd2.red}$	0.6$d \leq$ 300	
> 0.67 $V_{Rd2.red}$	0.3$\leq$ 200	

*When $V_{Sd} > 3\ V_{Rd1}$ the longitudinal spacing in beams should not exceed the values given in Table 8.18, if these are more critical, in order to control cracking. A_{sw} is defined in clauses 8.4.2 and 8.4.3.

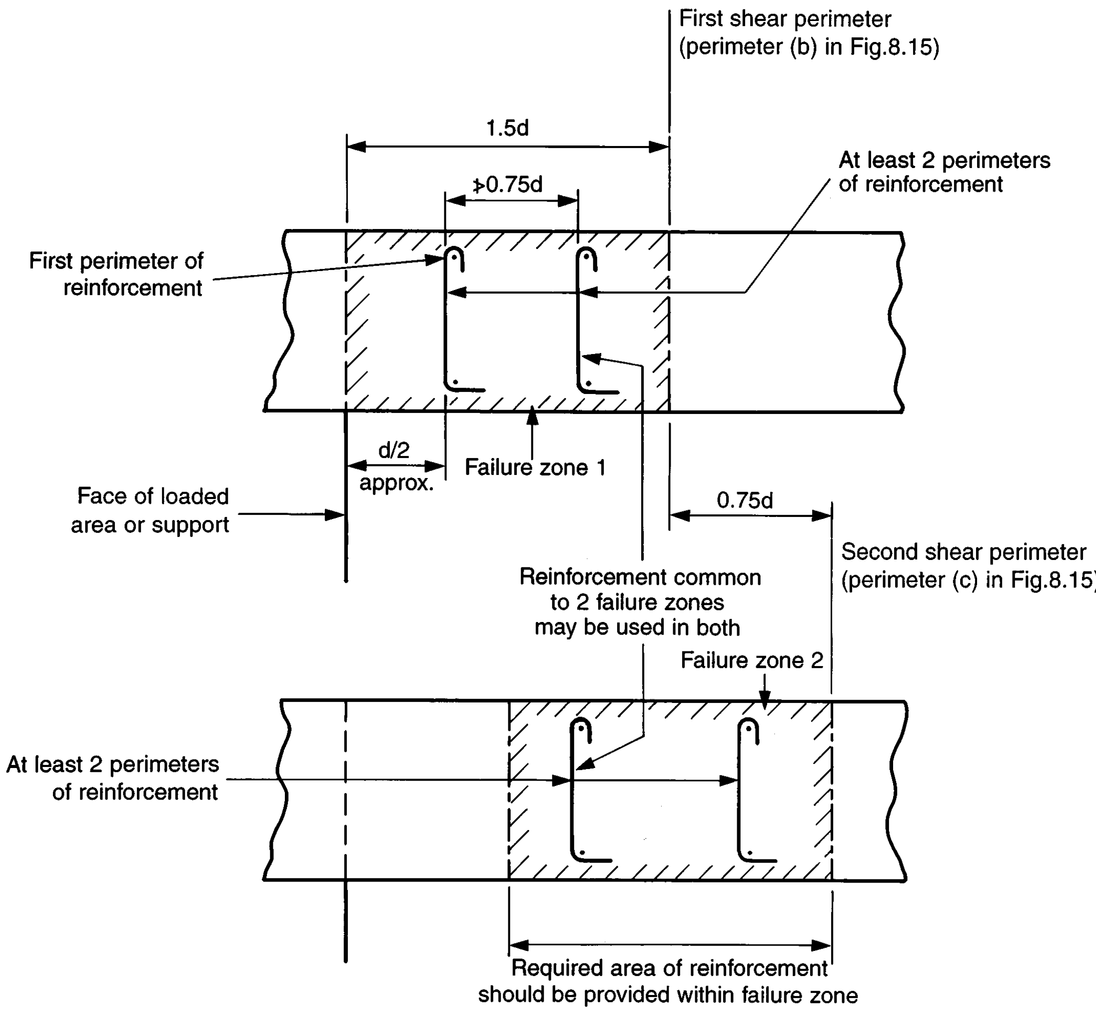


Fig. 8.16 Zones for punching shear reinforcement

Table 8.18 Longitudinal spacing of shear reinforcement in order to control cracking

$\frac{(V_{Sd} - 3V_{Rd1})}{A_{sw}d}$ N/mm³	Longitudinal spacing, mm
≤ 0.17	300
0.38	200
0.67	150
1.50	100
4.00	50

8.5 Anchorage zones

8.5.1 General

Anchorage zones are those areas of prestressed concrete members in which the prestressing forces are initially transferred from the tendons to the concrete and then disperse to an approximately linear distribution across the member. The length of the anchorage zone can generally be considered as being approximately equal to the depth of the member, but where tendon anchorages are well-distributed a shorter length may be more appropriate.

8.5.2 Pretensioned members

In pretensioned members the prestress force is transferred from the tendons to the concrete over a finite transmission length. When the tendons are released from the casting bed the stress within the transmission length is reduced, leading to an increase in the diameter of the tendon due to the Poisson effect. This transmits a radial compressive force into the concrete which is balanced by circumferential tensile forces. The transmission length depends on the type of tendon and the area of adequately anchored reinforcement resisting these tensile forces. The design transmission length, l_{bpd} is given by

$$l_{bpd} = 1.2\ \beta_b \phi$$

where ϕ is the nominal diameter of the tendon, and
β_b is given in Table 8.19.

The anchorage of pretensioned tendons at the ultimate limit state is affected by whether the anchorage zone is cracked or uncracked. The anchorage zone may be considered uncracked where, at any section distance x from the end of the beam, the maximum concrete tensile principal and flexural stresses due to 0.9 times the prestressing force at that point together with the design moment and shear force at the ultimate limit state do not exceed f_{ctm}, obtained from Table 8.11. In this case, no further checks are required.

Table 8.19 Transmission length factor, β_b

Concrete cylinder strength at transfer, f_{ck}, N/mm²	30	35	40	45	50
Tendons enclosed by links*	39	36	34	32	31
Tendons **not** enclosed by links	70	65	60	55	50

Notes to Table 8.19

1. The table is based on the UK NAD for EC2, Part 2[1].
2. *A tendon or group of tendons must be enclosed by links with an area of at least 1000mm²/m.

When the anchorage zone is cracked, a check should be carried out to ensure that the resisting tensile force provided by the tendons, F_{px}, and the ordinary reinforcement is greater than the longitudinal force required to carry the bending moment and shear force at the ultimate limit state, i.e.

$$\text{Resisting force} \geq \frac{M_{sd}}{2} + \frac{V_{sd} \cot\theta}{2}$$

where: M_{Sd} is the design bending moment
V_{Sd} is the design shear force
z is the lever arm (= $0.9d$), and
θ is the angle of the compression strut to the horizontal used in the shear design ($\theta = 45°$ when the standard method is used)

F_{px} can be calculated from

$$F_{px} = \frac{x}{l_{bpd}} P_{m,0} \leq \frac{A_p f_{p0.1k}}{1.15}$$

where $P_{m,0}$ is the force in the tendon immediately after stressing
l_{bpd} is the design transmission length
A_p is the area of the tendons, and
$f_{p0.1k}$ is the 0.1% proof stress of the tendons.

The build up of F_{px} is shown in Fig. 8.17.

Often pretensioned tendons are well distributed across the cross-section of the member so that the forces induced by transfer of the prestressing force from the tendon positions to a linear distribution are low. In slabs these forces can generally be resisted by the concrete alone without the need for ordinary reinforcement, whilst in beams it is generally satisfactory to reduce the stirrup spacing to approximately 75mm throughout the transmission length. Where this is not the case, design reinforcement to maintain overall equilibrium and to carry any spalling forces, using the approach described in clause 8.5.3.

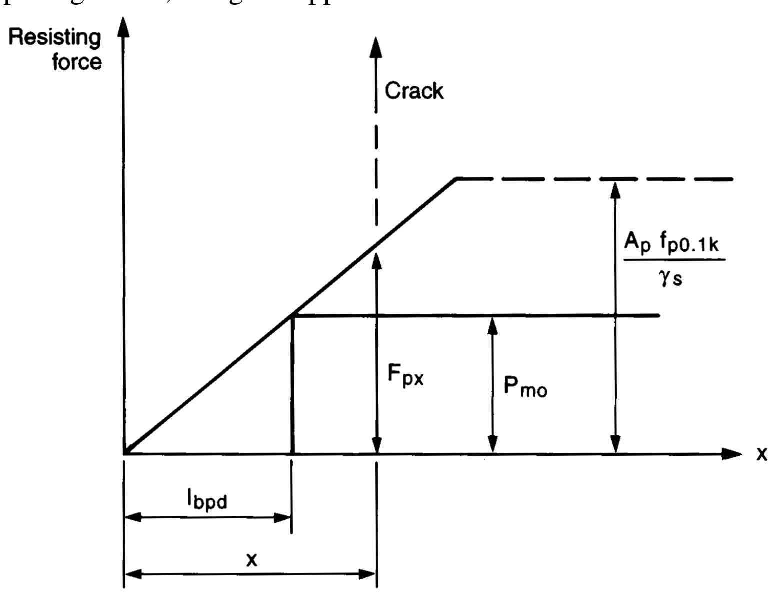


Fig. 8.17 Resisting tensile force provided by pretension tendons

8.5.3 Post-tensioned members

8.5.3.1 General

In post-tensioned members the prestress force is transferred from the tendons to the concrete through anchorage assemblies. The anchorages are supplied as part of the prestressing system and will have been designed by the manufacturer to limit the bearing stress on the concrete to acceptable values, providing the requirements on concrete strength, spacing and the length of straight tendon adjacent to the anchorage specified by the manufacturer are satisfied, see subsection 6.3.

The concentrated loads from the anchorages can induce the following tensile forces in the concrete member:

(a) **Bursting forces** behind the anchorages. These forces act normally to the line of the prestress force in all lateral planes.
(b) Forces required to maintain **overall equilibrium** of the anchorage zone.
(c) **Spalling forces** on the end face of the anchorage zone.

Reinforcement to resist these forces should be designed at the ultimate limit state as described below. At each point in the anchorage zone the area of reinforcement provided should be that required for the most critical effect, i.e. (a) or (b) or (c). The design force from each tendon, P_d, should be taken as equal to its characteristic strength, i.e.

$$P_d = A_p f_{pk}$$

where: A_p = area of the tendon, and
f_{pk} = characteristic strength of the tendon.

The resulting tensile forces should be resisted by reinforcement working at its design strength, $0.87 f_{yk}$.

As well as tensile forces, significant compressive forces occur in anchorage zones and it is essential that the reinforcement is detailed to allow sufficient room for the concrete to be placed and properly compacted.

8.5.3.2 Bursting

The design bursting tensile force, F_{bst}, can be derived from Table 8.20. For rectangular anchorages and/or rectangular end blocks, the bursting force should be calculated in each of the two principal directions using the appropriate value of y_{p0}/y_0 for each direction (see Fig. 8.18), where

y_0 is the half the side of the end block
y_{p0} is the half the side of the loaded area, and
P_d is the design force in the tendon.

Circular bearing plates should be treated as square plates of equivalent area.

Reinforcement to resist this force should be distributed in a region extending from $0.2y_0$ to $2y_0$ from the loaded face. The reinforcement should be provided in the form of closed hoops or spirals and be positioned as near as possible to the outer edge of the largest prism whose cross section is similar to and concentric with that of the anchor plate, having regard

Table 8.20 Design bursting tensile forces in anchorage zones

y_{p0}/y_0	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_d	0.23	0.23	0.20	0.17	0.14	0.11

Note: The table is taken from BS 8110[16].

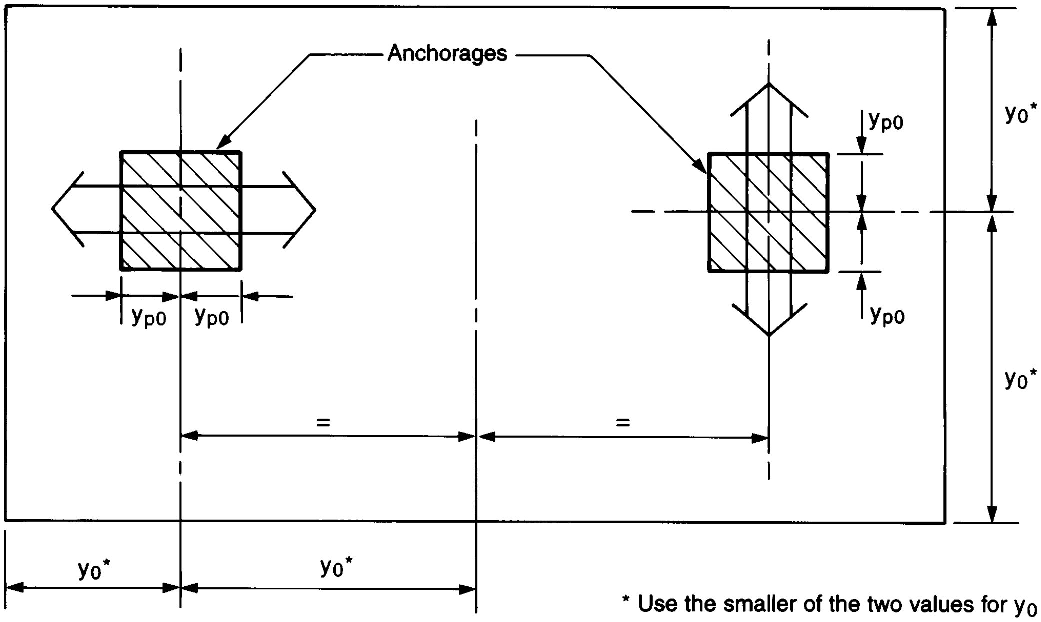


Fig. 8.18 Definition of y_{p0} and y_0 for end blocks

to the direction in which the load is spreading, and at least 50mm outside the edge of the anchor plate. Where spirals are provided as part of the anchorage system additional reinforcement may still be required to resist the bursting forces.

8.5.3.3 Overall equilibrium

Determine the tensile forces required to maintain overall equilibrium of the anchorage zone using strut-and-tie models. Separate models can be used when considering equilibrium in the horizontal and vertical directions, or a three-dimensional model may be used. Construct the models by sketching the flow of forces within the anchorage zone and providing notional concrete struts to carry the compressive forces and ties to carry the tensile forces. Calculate the forces in the resulting truss by considering equilibrium. The model should be chosen to minimise the length of the ties and with the angles between the struts and ties not less than 45° and preferably approximately equal to 60°. Models for some common anchorage zone arrangements are shown in Fig. 8.19. For more detailed information the engineer should refer to specialist texts.

Reinforcement to resist the tensile forces should be distributed over a length of $z/2$ where it is adjacent to a free edge, see Fig. 8.19(a), (c), (d) and (e). Where the tensile force occurs within the body of the member, see Fig. 8.19(b), the reinforcement should be distributed over a length equal to z, where z is defined in Fig. 8.19.

When tendons are to be stressed sequentially, overall equilibrium should be checked at each stage as the tendons are stressed.

8.5.3.4 Spalling[*]

Effectively bonded reinforcement should be placed as close to the loaded face of the anchorage zone as cover requirements allow to resist a force of $0.04P_d$ in either direction. The reinforcement should be distributed as uniformly as possible over the end face.

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[*] These requirements are taken from reference 21.

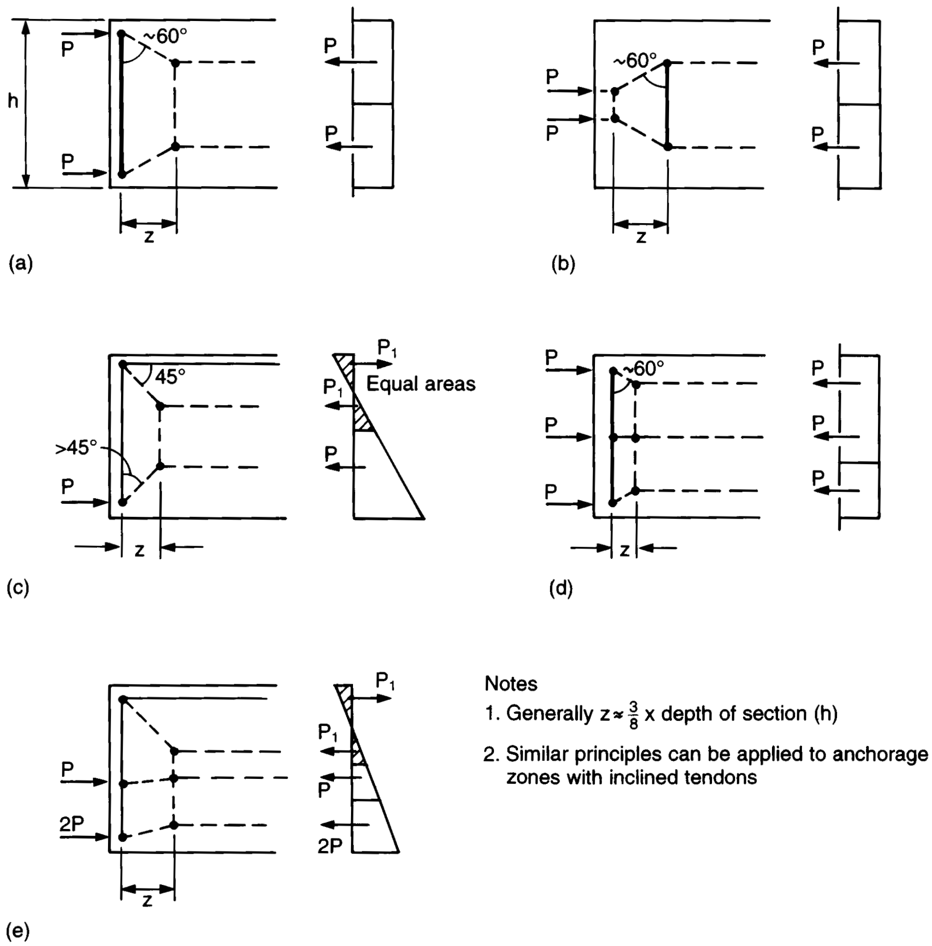

Fig. 8.19 Strut-and-tie models for the design of anchorage zones

When the configuration of the anchorages is such that the prestressing force acts on an unsymmetrical prism, see Fig. 8.20, and $d_1 > 2d_2$, additional spalling stresses are set up and additional reinforcement should be provided close to the loaded face to carry a force equal to:

$$0.2\left(\frac{d_1 - d_2}{d_1 + d_2}\right)^3 P_\mathrm{d}$$

This reinforcement need only be provided in the plane of the unsymmetrical prism and not at right angles to it.

8.5.4 Couplers

A coupler is an anchorage assembly which allows a tendon to be extended. It consists of two parts – an anchorage and the coupling assembly (see Fig 6.3).

The spacing and alignment of tendons at coupler locations should be in accordance with the prestressing system manufacturer's recommendations.

The zone behind the anchorage-side of the coupler should be designed in accordance with clause 8.5.3.

The placing of couplers on more than 50% of the tendons at any cross-section should be avoided. Tendons which are not coupled at a section should not be coupled within 1.5m of that section.

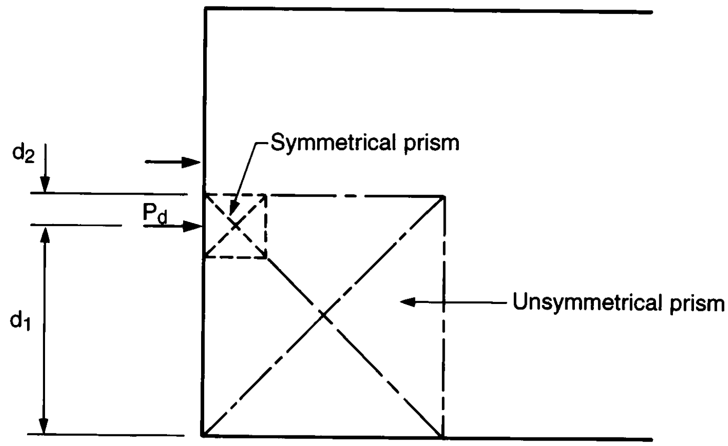

Fig. 8.20 Dimensions used in calculating spalling forces

8.6 Tendon extensions

Tendon extensions during stressing should be calculated and specified on the drawings. These are used during the stressing operation to check that the tendons are stressed throughout their length and have not become jammed in the duct during stressing.

The tendon extension is given by:

$$\delta = \frac{PL}{A_p E_s}$$

where: L is the length from the point of minimum force in the tendon up to the point of attachment of the jack

P is the average force in the tendon over that length **before** it is locked off

A_p is the area of the prestressing tendon, and

E_s is the elastic modulus of the prestressing tendon, which may be taken as 190kN/mm^2.

8.7 Robustness and detailing

The provisions of subsections 5.11 and 5.12 apply to prestressed members as appropriate. In satisfying the requirements for ties, prestressing tendons can be taken into account with a limiting tensile capacity equal to the prestressing force after all losses, $P_{m,\infty}$.

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Appendix A Reinforcement quantities

This Appendix contains Tables A1, A2, A3, A4 and A5 referred to in method 2 of subsection 4.9.

The factors for converting reinforcement areas into unit weights of reinforcement assume that the reinforcement areas are those of practical bar arrangements, e.g. standard sizes at realistic spacings in beams; an even number of bars in columns.

Table A1 Solid slabs and stairs

Minimum reinforcement:
high yield bars – 0.13% of gross cross-section
mild steel bars – 0.24% of gross cross-section

Type of slab	A_{sx} required	A_{sy} required	Weight kg/m²	Remarks
One-way spanning slabs	$\dfrac{M}{(0.8d)(0.87f_{yk})}$	Minimum steel or 0.25 A_{sx}	$0.0125A'_{sx}$*	M is the maximum bending moment per metre width anywhere in the slab
Two-way spanning slabs with linear supports	$\dfrac{M_x}{(0.8d)(0.87f_{yk})}$	$\dfrac{M_y}{0.8(d–20)\ 0.87f_{yk}}$	$0.011(A_{sx} + A'_{sy})$	M_x and M_y are the maximum bending moments per metre width in each direction
Flat slabs on column supports	$\dfrac{M_x}{(0.8d)(0.87f_{yk})}$	$\dfrac{M_y}{0.8(d–20)\ 0.87f_{yk}}$	$0.011(A'_{sx} + A'_{sy})$	M_x and M_y are the mean (of the column and middle strip) maximum bending moments per metre width in each direction

*This includes weight of distribution steel.

Notes to Table A1

1 All the bending moments are the design ultimate moments.
2 A_{sx} and A_{sy} are areas of reinforcement required in two orthogonal directions.
3 $A'_{sx} + A'_{sy}$ are areas of reinforcement (in mm²) selected per metre width in two orthogonal directions.
4 Consistent units must be used in the formulas for obtaining areas of reinforcement.

Table A2 Ribbed and coffered slabs

Minimim reinforcement
Ribs high yield steel — 0.13% $b_w h$
mild steel — 0.24% $b_w h$
where b_w is the average width of the ribs and h is the overall depth of the slab

Structural topping
high-yield steel — 0.13% of gross cross-section of topping
mild steel — 0.24% of gross cross-section of topping

Type of slab	A_s required (in each direction for two-way and flat slabs), mm²	Weight kg/m²		Remarks
		Ribs	Structural topping	
One-way spanning slabs	$\dfrac{M}{0.87 f_{yk} (d - 0.5h_f)}$	$\dfrac{0.009A'_s}{c}$	For fabric reinforcement: 1.25 x wt/m² of fabric For loose bar reinforcement: 0.009 (sum of bar areas per m width in each direction)	M is the maximum bending moment per rib anywhere in the slab
Two-way spanning slabs on linear supports	$\dfrac{M}{0.87 f_{yk} (d - 10 - 0.5h_f)}$	$\dfrac{0.02A'_s}{c}$	As for one-way spanning slabs	M is the maximum bending moment per rib in the two directions
Coffered slabs on column supports	$\dfrac{M_x}{0.87 f_{yk} (d - 10 - 0.5h_f)}$ and $\dfrac{M_y}{0.87 f_{yk} (d - 10 - 0.5h_f)}$	$\dfrac{0.013(A'_{sx} + A'_{sy})}{c}$	As for one-way spanning slabs	M_x and M_y are the mean (of the column and middle strips) maximum bending moments per rib in each direction

Notes to Table A2
1 All bending moments are the design ultimate moments.
2 c is the spacing of ribs in metres.
3 Consistent units should be used in the formulas for obtaining areas of reinforcement.
4 A'_s, A'_{sx} and A'_{sy} are the areas in (mm²) of bars selected per rib.

Table A3 Beams

Minimum reinforcement:

Longitudinal steel: high yield – 0.13% $b_w h$
mild steel – 0.24% $b_w h$

where b_w is the width of beam and h is the overall depth of beam

Links: mild steel – 0.28% of a horizontal section through the web
high yield – 0.15% of horizontal section through web

	A_s required	Weight kg/m	
Longitudinal steel	At midspan for T- and L-beams (and at support for upstand beams) $$\frac{M}{0.87 f_{yh}(d - 0.5h_f)}$$ For rectangular beams and at supports for T- and L-beams (and at midspan for upstand beams) $$\frac{M}{0.87 f_{yk}(0.75d)}$$	$0.011A'_s$	A'_s is the area (in mm²) of main reinforcement selected at midspan or supports, whichever is greater M is the design ultimate bending moment
Links	Shear stress $v = \frac{\text{design ultimate shear force}}{b_w d}$ If $v > 0.6\text{N/mm}^2$ $$\frac{A_{sv}}{S_v} = \frac{b_w(v - 0.6)}{0.87 f_{yk}}$$ If $v \leq 0.6\text{N/mm}^2$ chose A'_{sv} and $S'v$ to satisfy minimum steel	Single links (i.e. two legs) $0.016\ (B_w + H)\ \frac{A'_{sv}}{S'_v}$ Double links (i.e. four legs) $0.016(1.5\ B_w + 2H)\ \frac{A'_{sv}}{S'_v}$ Treble links (i.e. six legs) $0.016(2B_w + 3H)\ \frac{A'_s}{S'_v}$	B_w is the width of beam in metres H is the depth of beam in metres A'_{sv} is the area selected for one leg of a link in mm² S'_v is the selected spacing of links in metres

Table A4 Columns

Minimum reinforcement:

Longitudinal steel: 1% of the necessary concrete area
Links - make the choice to satisfy the following:

- diameter at least one-quarter of the biggest longitudinal bar
- spacing: 12 × diameter of smallest longitudinal bar but not more than 300mm
- every corner and each alternate longitudinal bar should be restrained by a link in each direction

Weight kg per m height of column	Remarks
Main steel $0.011A_s$	A_s of all vertical bars (mm^2)
Links peripheral links $0.016(b+h)\dfrac{A}{S_v}$	b and h are dimensions of column cross-section in metres A is the cross-sectional area of one leg of a link in mm^2 S_v is the spacing of links in metres
sausage links $0.016b\dfrac{A_s}{S_v}$	For sausage links (shape code 81) b is the dimension parallel to the link

Table A5 Walls

Minimum reinforcement:

Vertically 0.4% of cross-sectional area
Horizontally 0.2% of cross-sectional area

Weight of reinforcement in kg/m^2 of wall elevation
$0.011(A_{sv} + A_{sh})$
where A_{sv} and A_{sh} are the areas of reinforcement in mm^2 selected per metre width and height.

Note to Table A5
Consistent units must be used in obtaining areas of reinforcement

Appendix B Design data

Design data should include:

- general description, intended use, unusual environmental conditions
- site constraints
- design assumptions
- stability provisions
- movement joints
- loading
- fire resistance
- durability
- soil conditions and foundation design
- performance criteria
- materials
- ground slab construction
- method statement assumed in design
- quality plan
- method of analysis
- computer programs used
- responsible designer
- responsible approver
- other data

Appendix C Exposure classes related to environmental conditions

Exposure class		Examples of environmental conditions
1. Dry environment		Interior of building for normal habitation or offices*
2. Humid environment	(a)	Interior of buildings with high humidity (e.g. laundries) Exterior components Components in non-aggressive soil and/or water
	(b)	As (a) above but with exposure to frost
3. Humid environment with frost and de-icing salts		Interior and exterior components exposed to frost and de-icing agents
4. Seawater environment	(a)	Components completely or partially submerged in seawater or in the splash zone Components in saturated salt air
	(b)	As (a) above but with frost
5. Aggressive chemical environment†	(a)	Slightly aggressive chemical environment (gas, liquid or solid) Aggressive industrial atmosphere
	(b)	Moderately aggressive chemical environment (gas, liquid or solid)
	(c)	Highly aggressive chemical environment (gas, liquid or solid)

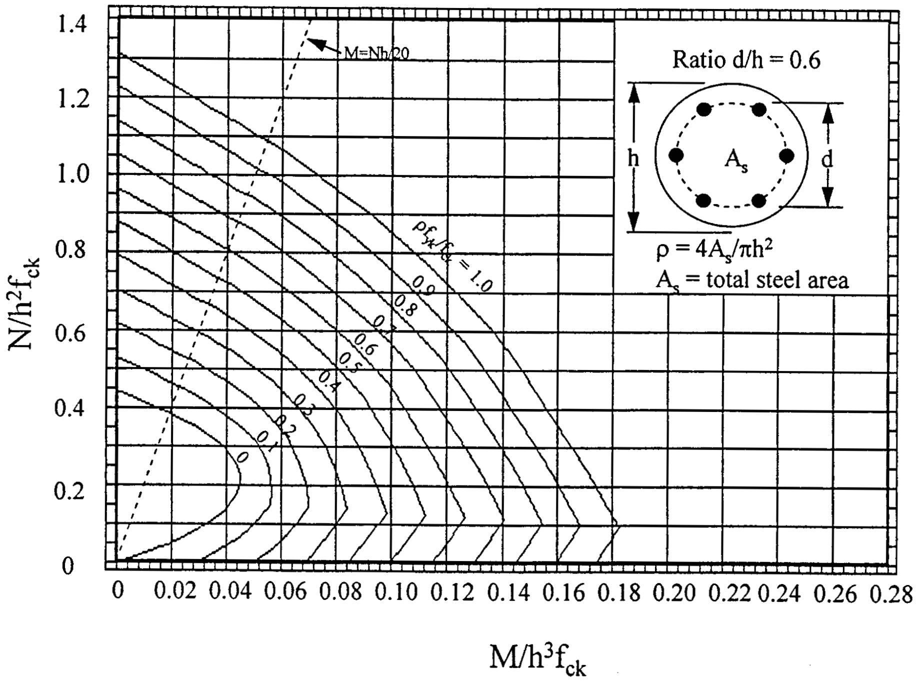
* This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time.

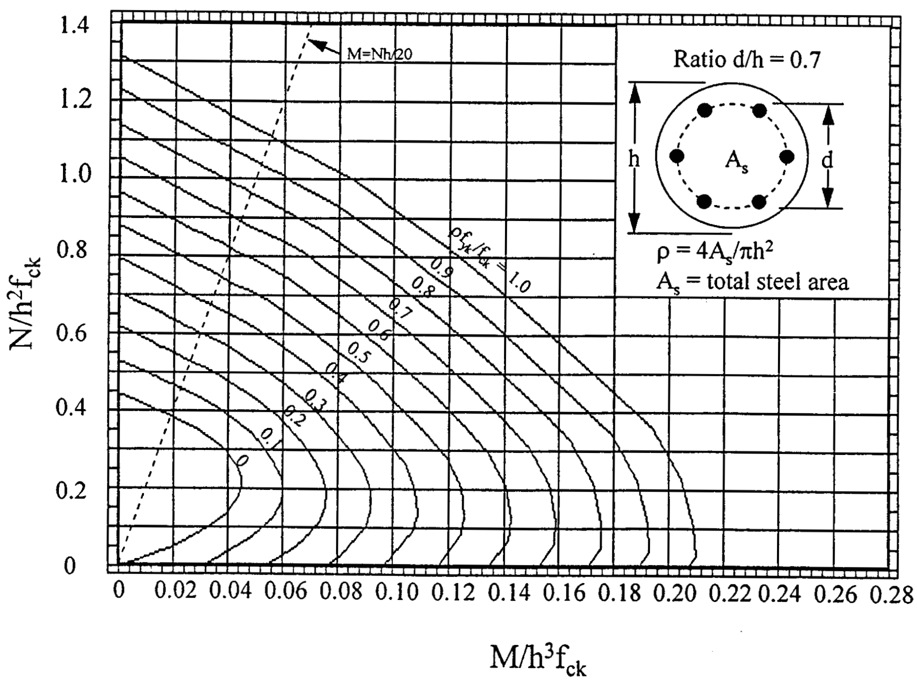
† Chemically aggressive environments are classified in ISO/DP 9690. The following equivalent exposure conditions may be assumed:

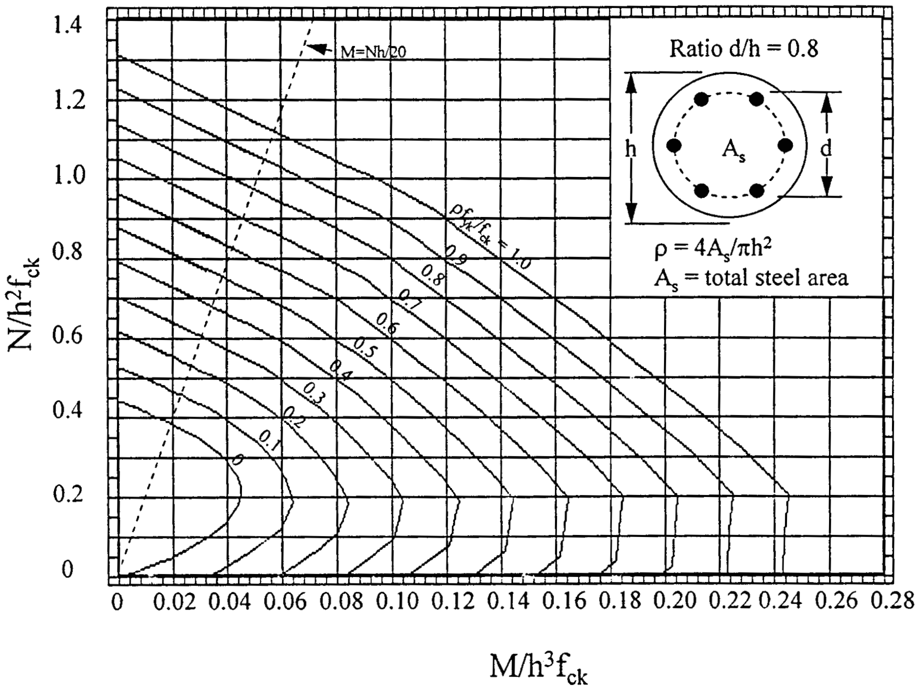
Exposure class 5a : ISO classification A1G, A1L, A1S
Exposure class 5b : ISO classification A2G, A2L, A2S
Exposure class 5c : ISO classification A3G, A3L, A3S

Appendix D Column design charts

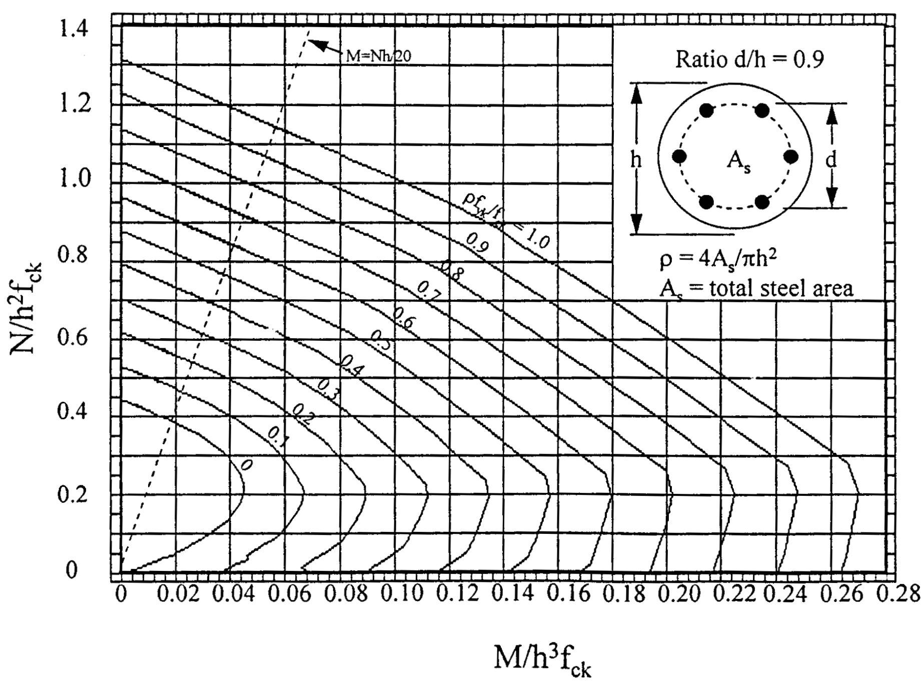
This Appendix contains design charts for circular columns with d/h = 0.6, 0.7, 0.8 and 0.9, and rectangular columns with d/h = 0.8 0.85, 0.9 and 0.95.



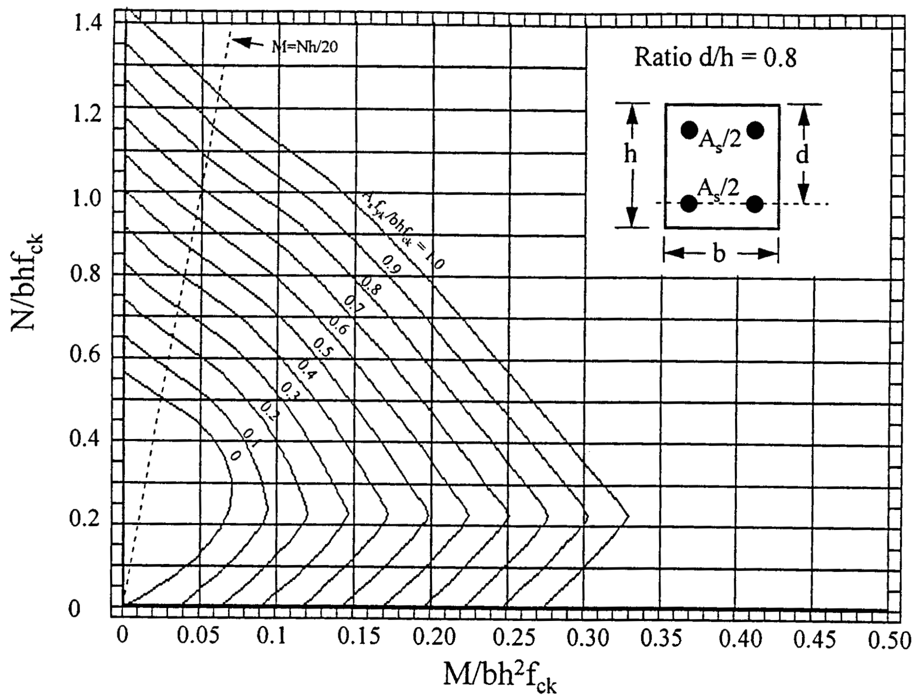


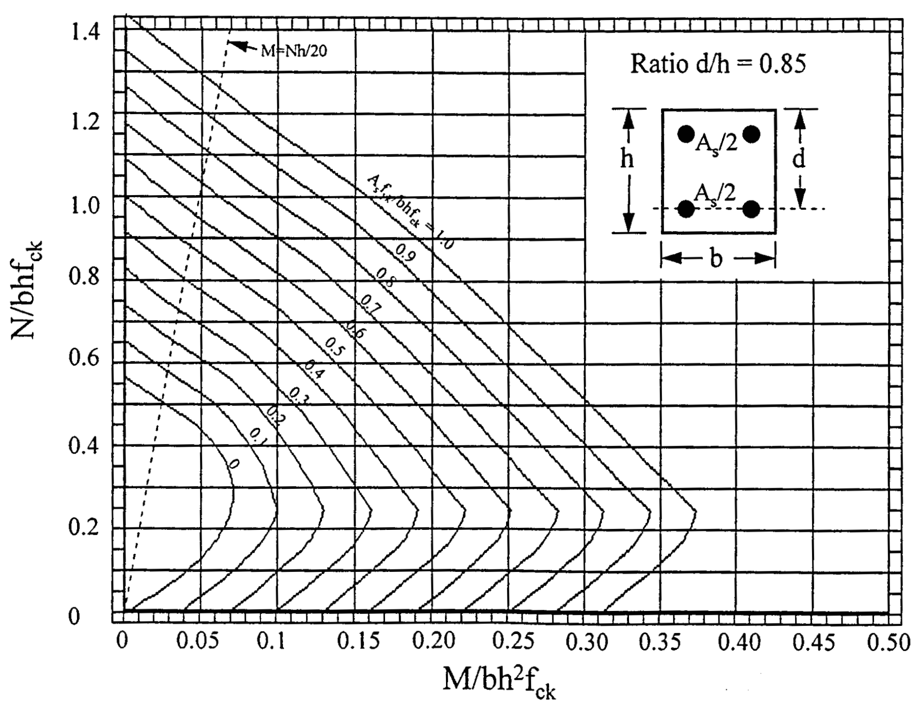


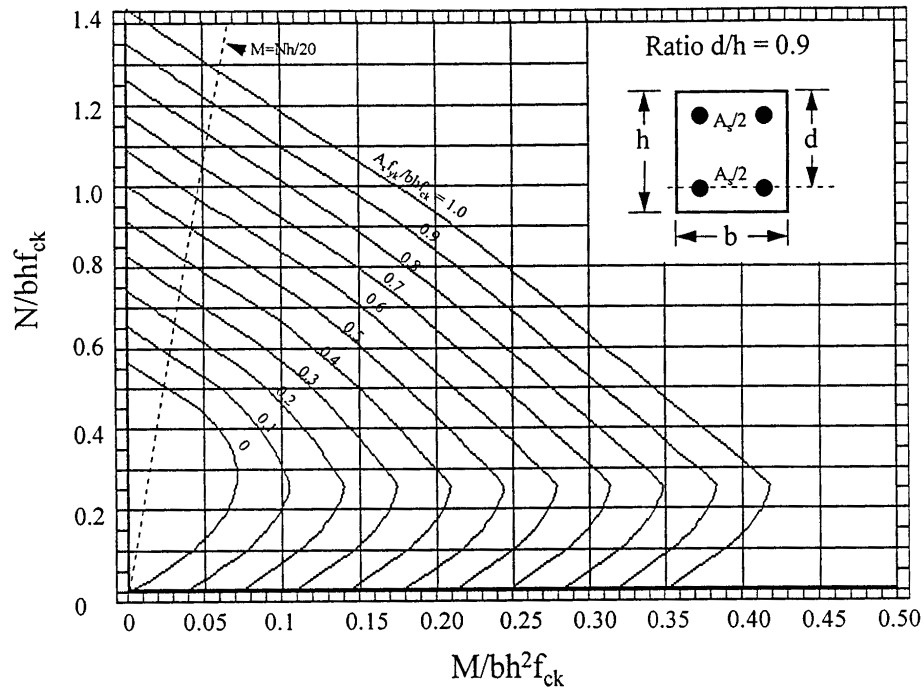
Ratio d/h = 0.8
M=Nh/20
h
d
A_s
$\rho f_{yk}/f_{ck}$ = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
$\rho = 4A_s/\pi h^2$
A_s = total steel area
N/h^2f_{ck}
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
0 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0.16 0.18 0.20 0.22 0.24 0.26 0.28
M/h^3f_{ck}



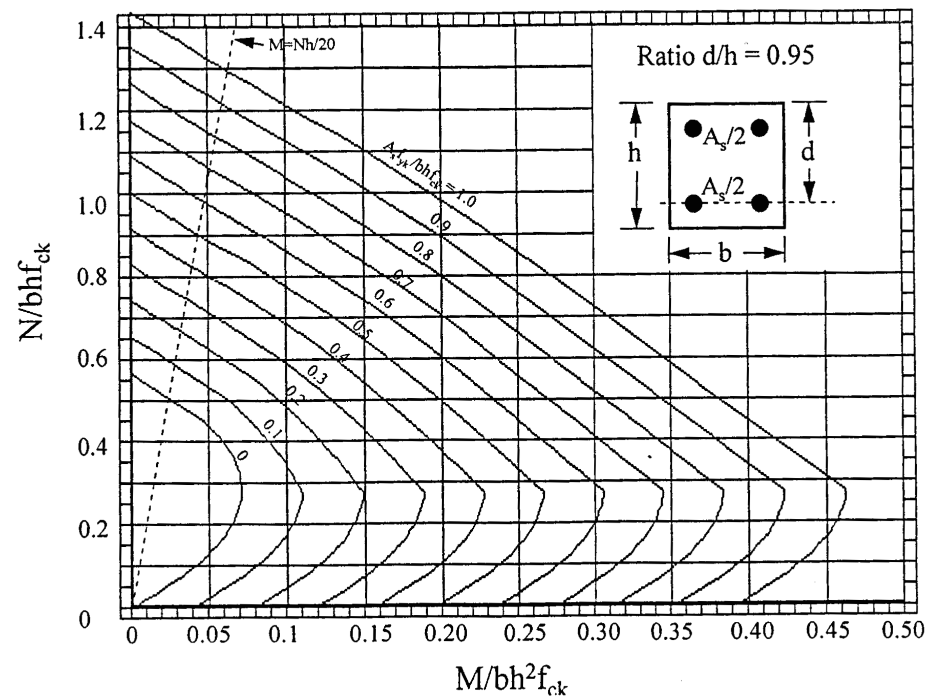
Ratio d/h = 0.9
M=Nh/20
h
d
A_s
$\rho f_{yk}/f_{ck}$ = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
$\rho = 4A_s/\pi h^2$
A_s = total steel area
N/h^2f_{ck}
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
0 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0.16 0.18 0.20 0.22 0.24 0.26 0.28
M/h^3f_{ck}


M=Nh/20
Ratio d/h = 0.8
$A_s/2$
$A_s/2$
h
d
b
A_sf_{yk}/bhf_{ck} = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
N/bhf$_{ck}$
M/bh^{2}f$_{ck}$
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
0.05
0.1
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50


M=Nh/20
Ratio d/h = 0.85
$A_s/2$
$A_s/2$
h
d
b
A_sf_{yk}/bhf_{ck} = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
N/bhf$_{ck}$
M/bh^{2}f$_{ck}$
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
0.05
0.1
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50



Ratio d/h = 0.9
M=Nh/20
$A_s f_{yk}/bhf_{ck}$ = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
$A_s/2$
$A_s/2$
h
d
b
N/bhf_{ck}
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
0.05
0.1
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
M/bh^2f_{ck}



Ratio d/h = 0.95
M=Nh/20
$A_s f_{yk}/bhf_{ck}$ = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.1
0
$A_s/2$
$A_s/2$
h
d
b
N/bhf_{ck}
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
0.05
0.1
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
M/bh^2f_{ck}

Appendix E Foundations

E.1 Introduction

The type of foundation, the sizes and the provisional formation levels depend on the results of ground investigation. Geotechnical design of foundations is beyond the scope of this *Manual*, and it has been assumed that the designer has access to the relevant expertise. Part 3 of EC2 dealing with the strength design of foundations will be published as an ENV document in due course. The main provisions of EC2, Part 1, still apply to the design of foundations.

The foundations should be sized and designed in accordance with EC7 and EC2, Part 3, and noting that three load cases (A, B and C) need consideration. This *Manual* provides a procedure to comply with the above requirements for those structures within its scope. The partial factors to be applied to loads are shown in Table E1. The relevant ground property for each load case will vary, and EC7 should be consulted. In all these cases it should be verified that the appropriate ground pressure is not exceeded and the structural elements are satisfactory. It will also be necessary to check the foundation sizes for serviceability so that the settlement is within acceptable limits (see EC7).

The general procedure to be adopted is as follows:

1. evaluate results of ground investigation and decide whether spread or piled foundations are to be used
2. examine existing and future levels around the structure, and taking into account the bearing strata and ground water levels, determine the provisional formation levels
3. calculate the plan areas of footings or the number of piles to be used to support each column or wall so that the appropriate ground property is not exceeded for all cases
4. calculate the loads and moments, if any, on the individual foundations using the partial factors in Table E1 for each load case
5. calculate the depth required for each foundation member and the reinforcement, if any, using the factored loads.

E.2 Durability and cover

All foundations other than those in aggressive soil conditions or exposed to frost are considered as being in exposure class 2a (see Appendix C). Cover to *all* reinforcement should be 50mm. For reinforced foundations the minimum cement content should be 280kg/m^3 and the maximum water/cement ratio 0.60.

The concrete class strength for reinforced bases and pile caps should therefore be not less than C30/37. For unreinforced bases C16/20 may be used, subject to a minimum cement content of 220kg/m^3. Where sulphates are present in significant concentrations in the soil and/or the groundwater, the recommendations of BRE Digest no. 363[13] should be followed.

E.3 Types of foundation

The loads and moments imposed on foundations may be supported by any one of the following types:

Pad footing
A square or rectangular footing supporting a single column.

Strip footing
A long footing supporting a continuous wall.

Table E1 Partial factors for loads

Load case	Load combination	Dead load G_k		Imposed, wind or snow loads G_k or W_k	Earth or water E_n	
		Adverse	Beneficial	Adverse	Adverse	Beneficial
A	1. Dead + imposed	1.1	0.9	1.5	1.1	0.9
	2. Dead + wind	1.1	0.9	1.5	1.1	0.9
	3. Dead + snow	1.1	0.9	1.5	1.1	0.9
	4. Dead + imposed + wind + snow	1.1	0.9	1.35	1.1	0.9
B	1. Dead + imposed	1.35	–	1.5	1.35	–
	2. Dead + wind	1.35	1.0	1.5	1.35	1.0
	3. Dead + snow	1.35	1.0	1.5	1.35	1.0
	4. Dead + imposed + wind + snow	1.35	–	1.35	1.35	–
C	1. Dead + imposed	1.0	1.0	1.3	1.0	1.0
	2. Dead + wind	1.0	1.0	1.3	1.0	1.0
	3. Dead + snow	1.0	1.0	1.3	1.0	1.0
	4. Dead + imposed + wind + snow	1.0	1.0	1.3	1.0	1.0

Combined footing
A footing supporting two or more columns.

Balanced footing
A footing supporting two columns, one of which lies at or near one end.

Raft
A foundation supporting a number of columns or loadbearing walls so as to transmit approximately uniform loading to the soil.

Pile cap
A foundation in the form of a pad, strip, combined or balanced footing in which the forces are transmitted to the soil through a system of piles.

E.4 Plan area of foundations

The plan area of the foundation should be proportioned on the following assumptions:

1. all forces are transmitted to the soil without exceeding the allowable bearing pressure
2. when the foundation is axially loaded, the reactions to design loads are uniformly distributed per unit area or per pile. A foundation may be treated as axially loaded if the eccentricity does not exceed 0.02 times the length in that direction
3. when the foundation is eccentrically loaded, the reactions vary linearly across the footing or across the pile system. Footings should generally be so proportioned that zero pressure occurs only at one edge. It should be noted that eccentricity of load can arise in two ways: the columns being located eccentrically on the foundation; and/or the column transmitting a moment to the foundation. Both should be taken into account and combined to give the maximum eccentricity.
4. all parts of a footing in contact with the soil should be included in the assessment of contact pressure
5. it is preferable to maintain a reasonably similar pressure under all foundations to avoid significant differential settlement.

E.5 Design of spread footings

E.5.1 Axially loaded unreinforced pad footings

The ratio of the depth of the footing h_f to the projection from the column face a should be not less than that given in Table E2 for different values of unfactored pressures, q in kN/m^2. q is the ground pressure that is in equilibrium with the loads applied to the superstructure in load cases A, B and C of Table E1.

In no case should h_f/a be less than 1, nor should h_f be less than 300mm.

E.5.2 Axially loaded reinforced pad footings

The design of axially loaded reinforced pad footings is carried out in three stages:

1. Determine the depth of the footing from the ratios of the effective depth d to the projection from the column face a, given in Table E2 for different values of unfactored ground pressures σ. The effective depth d should not in any case be less than 300mm.
2. Check that the face shear given by:

$$V_{Sd} = \frac{1000N}{2(c_x + c_y)d}$$

does not exceed $0.9\sqrt{f_{ck}}$, where N is the factored column load in kN, c_x and c_y are the column dimensions in mm and d is the effective depth in mm. If V_{Sd} exceeds this value increase the depth.

Table E2 Depth/projection ratios for unreinforced footings

	h_f/a				
Ground pressure q, kN/m^2	C16/20	C20/25	C25–30	C30/35	C35/45
≤ 300	1.1	1.0	1.0	1.0	1.0
450	1.3	1.2	1.1	1.0	1.0
600	1.5	1.4	1.3	1.2	1.2

Table E3 Reinforcement percentages, depth/projection ratios and ground pressures for reinforced footings for f_{yk} = 460N/mm²

q kN/m²	d/a 0.24	0.32	0.37	0.41	0.43	0.46	0.49	0.60	0.70	> 0.80
75	0.18	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13
150		0.20	0.15	0.13	0.13	0.13	0.13	0.13	0.13	0.13
225			0.23	0.19	0.17	0.15	0.13	0.13	0.13	0.13
300						0.20	0.17	0.13	0.13	0.13
375								0.15	0.13	0.13
450									0.17	0.13

3. With the chosen depth (revised according to stage 2, if necessary) enter Table E3 and obtain the corresponding reinforcement percentage. The stippled areas indicate combinations of s and d/a that should not be used. q is the ground pressure that is in equilibrium with the loads applied to the superstructure in load cases A, B and C.

E.5.3 Eccentrically loaded footings

The design of eccentrically loaded footings proceeds as follows:

1. determine initial depth of footing from Table E2 using maximum value of unfactored ground pressure
2. check punching shear according to subclauses 5.2.3.4 and 5.2.4.2
3. check face shear according to stage 2 in subclause E.5.2 using V_{eff} from clause 5.2.3.4 in lieu of N
4. increase the depth if necessary to avoid shear reinforcement
5. with the chosen depth (revised according to stage 4, if necessary) enter Table E3 to obtain the reinforcement percentage using maximum values of unfactored ground pressure.

E.6 Design of other footings

E.6.1 Strip footings

Strip footings should be designed as pad footings in the transverse direction and in the longitudinal direction at free ends or return corners. If reinforcement is required in the transverse direction it should also be provided in the longitudinal direction and should not be less than that obtained from the procedures in clause E.5.2.

E.6.2 Combined footings and balanced footings

Combined footings and balanced footings should be designed as reinforced pad footings except as extended or modified by the following requirements:

Punching shear should additionally be checked for critical perimeters encompassing two or more closely spaced columns according to subclauses 5.2.3.4 and 5.2.4.2. Bending moments should additionally be checked at the point of zero shear between the two columns. Reinforcement should be provided in top and bottom faces and may be curtailed in accordance with the detailing rules in subsection 5.12.

Where a balanced footing consists of two pad footings joined by a beam, the beam may be designed in accordance with subsection 5.4.

Steps in the top or bottom surface may be introduced if necessary provided that they are taken into account in the design.

E.7 Reinforcement

Where reinforcement is required it should be provided in two generally orthogonal directions. The areas in each direction should not be less than $0.0015bh$ for reinforcement with $f_{yk} = 460\text{N/mm}^2$ where b and h are the breadth and overall depth in mm, respectively. All reinforcement should extend the full length of the footing.

If $l_x > 1.5\ (c_x + 3d)$, at least two-thirds of the reinforcement parallel to l_y should be concentrated in a band width $(c_x + 3d)$ centred at the column, where d is the effective depth, l_x and c_x are the footing and column dimensions in the x-direction and l_y and c_y are the footing and column dimensions in the y-direction. The same applies in the transverse direction with suffixes x and y transposed.

Reinforcement should be anchored each side of all critical sections for bending. It is usually possible to achieve this with straight bars.

The spacing between centres of reinforcement should not exceed 200mm for bars with $f_{yk} = 460\text{N/mm}^2$. Reinforcement need normally not be provided in the side face nor in the top face, except for balanced or combined foundations.

Starter bars should terminate in a 90° bend tied to the bottom reinforcement, or in the case of an unreinforced footing spaced 75mm off the blinding.

E.8 Design of rafts

The design of a raft is analogous to that of an inverted flat slab (or beam-and-slab) system, with the important difference that the column loads are known but the distribution of ground bearing pressure is not. A distribution of ground bearing pressure has to be determined that:

(a) satisfies equilibrium by matching the column loads
(b) satisfies compatibility by matching the relative stiffness of raft and soil
(c) allows for the concentration of loads by slabs or beams continuous over supports, and
(d) stays within the allowable bearing pressure determined from geotechnical considerations of strength and settlement.

Provided that such a distribution can be determined or estimated realistically by simple methods, design as a flat slab or beam-and-slab may be carried out. In some cases, however, a realistic distribution cannot be determined by simple methods, and a more complex analysis is required.

E.9 Design of pile caps

The design of pile caps should be carried out in accordance with the following general principles:

(a) The spacing of piles should generally be three times the pile diameter
(b) The piles should be grouped symmetrically under the loads
(c) The load carried by each pile is equal to N/(no. of piles). When a moment is transmitted to the pile cap the loads on the piles should be calculated to satisfy equilibrium
(d) Pile caps should extend at least 150mm beyond the theoretical circumference of the piles
(e) For pile caps supported on one or two piles only, a moment arising from a column eccentricity of 75mm should be resisted either by ground beams or by the piles.

The general procedure to be adopted is as follows:

(f) Using the unfactored loads and moments calculate the number of piles required under each column

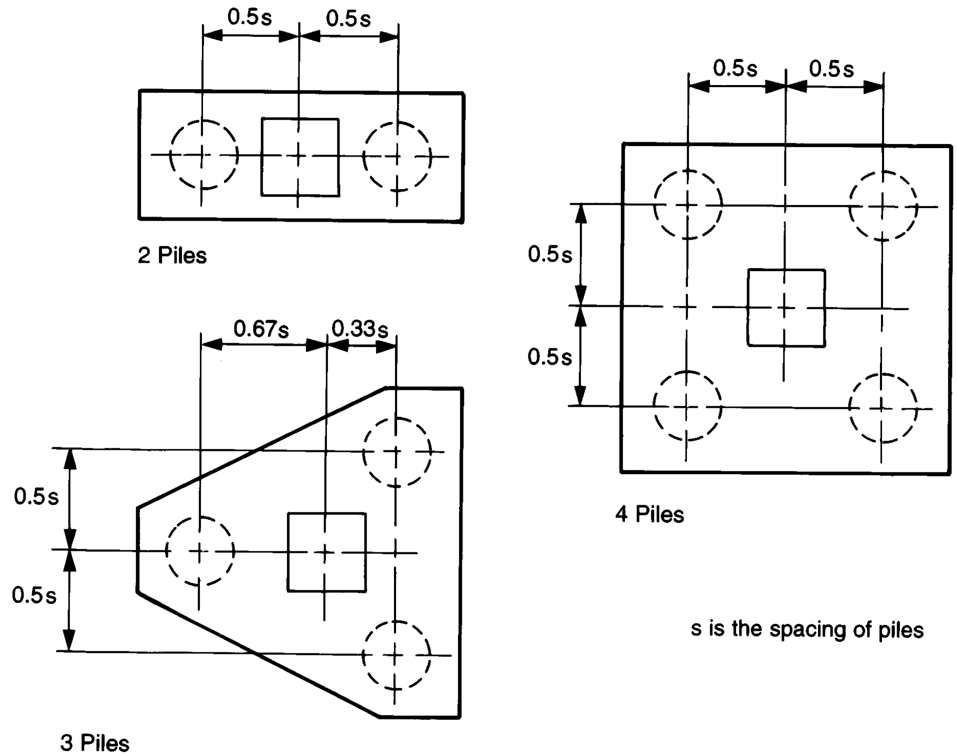


Fig. E1 Typical pile arrangements

(g) Proportion the pile caps on plan in accordance with the above general principles. Typical arrangements are shown in Fig. E1 where s is the spacing of the piles
(h) Determine the initial depth of the pile cap as equal to the horizontal distance from the centreline of the column to the centreline of the pile furthest away
(i) Check the face shear as for reinforced pad footings, using factored loads, and increase the depth if necessary
(j) Calculate the bending moments and the reinforcement in the pile caps using the factored loads.

E.10 Reinforcement in pile caps

All pile caps should generally be reinforced in two orthogonal directions on the top and bottom faces. Where bars with f_{yk} = 460N/mm^2 are used the amount of reinforcement should not be less than 0.0015bh in each direction.

The bending moments and the reinforcement should be calculated on critical sections at the column faces, assuming that the pile loads are concentrated at the pile centres. This reinforcement should be continued past the piles and bent up vertically to provide full anchorage past the centreline of each pile.

In addition, fully lapped, circumferential horizontal reinforcement consisting of bars not less than 12mm in diameter at a spacing not more than 250mm, should be provided.