The Institution of Structural Engineers
The Institution of Civil Engineers

October 1985

Manual for the design of reinforced concrete building structures

Published by the Institution of Structural Engineers

The Institution of Structural Engineers
The Institution of Civil Engineers

October 1985

Manual for the design of reinforced concrete building structures

Published by the Institution of Structural Engineers

Constitution

D. J. Lee, BScTech, DIC, FEng, FIStructE, FICE, FIHT, *Chairman*
S. J. Alexander, MA, CEng, MIStructE, MICE
P. Beckmann, MSc(Eng), CEng, FIStructE, MICE, HonRIBA, MIngF
P. G. Cobb, CEng, MICE
B. H. Fisher, BSc, CEng, FIStructE, FICE
R. S. Narayanan, BE, MSc, DIC, CEng, FIStructE
R. F. D. Povey, CEng, FIStructE
H. C. Symonds, MA, CEng, FIStructE, MICE
F. Walley, CB, MSc, PhD, FEng, FIStructE, FICE
D. J. Wilson, BSc(Eng), CEng, MIStructE
K. R. Wilson, MA, CEng, MICE
P. F. Winfield, CEng, FIStructE
R. J. W. Milne, BSc, *Secretary to the* ad hoc *Committee*

Contents

page number

Tables

Foreword

In 1982 the Institution of Structural Engineers formed a Committee to prepare a *Manual for the design of reinforced concrete building structures* which would be compatible with British Standard BS 8110. Happily the Institution of Civil Engineers has joined in this task and this document is the result. It has been written by and for practising designers and thus reflects the logical sequence of operations which a designer follows.

The *Manual* covers the majority of reinforced concrete buildings, but with the deliberate exclusion of some items. For example, prestressed and lightweight concretes are not covered and the range of structures is limited to those not dependent on the bending of columns for resistance against horizontal forces. The first limitation does not imply a bias against the use of prestressed or lightweight concrete in buildings while the second limitation recognizes that buildings are usually designed to be braced by strongpoints such as shear walls, infill panels and the like.

Users will note that the recommendations given in this *Manual* fall within the wider range of options in BS8110.

The Committee has aimed at clarity and logical presentation of reinforced concrete design practice in writing the *Manual*. It is hoped that the concise format will be welcomed.

The *Manual* offers practical guidance on how to design safe, robust and durable structures. The initial design section is a novel feature of the *Manual*, and the guidance given will make a positive contribution to design practice. If these initial design procedures are followed, the final calculations can be carried out expeditiously. The information has been laid out for hand calculation but the procedures are suited for electronic computations as well.

The preparation of the *Manual* has proceeded concurrently with, but independently of, BS 8110. Helpful comment has been received from members of the BS 8110 Committee, including the Chairman, Dr. D. D. Matthews, Dr. A. W. Beeby and Mr. H. B. Gould. Indeed there has been a valuable two-way exchange which has had an impact on BS 8110.

During the preparation many people have commented, and I would be grateful if any further comment could be forwarded to the Institution.

Lastly I would like to express my thanks to the members of the Committee and their organizations and also to our Secretary, Mr. R. J. W. Milne, for the enthusiasm and harmonious relations which have characterised our work.

D. J. LEE
Chairman

1 Introduction

1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of reinforced concrete building structures. Structures designed in accordance with this *Manual* will normally comply with BS 8110.[1]

1.2 Scope of the *Manual*

The range of structures and structural elements covered by the *Manual* is limited to building structures, using normal weight concrete and which do not rely on bending in columns for their resistance to horizontal forces. This will be found to cover the vast majority of all reinforced concrete building structures. For detailing rules the *Standard method of detailing structural concrete*[2] should be used.

For structures or elements outside this scope BS 8110[1] should be used.

1.3 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
reinforcement estimating
final design of members.

2 General principles

This section outlines the general principles that apply to both initial and final design 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.

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' structure, i.e. one in which the columns will not be subject to 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 200mm solid blockwork properly tied and pinned 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 must be ensured that floors can 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 4.11.

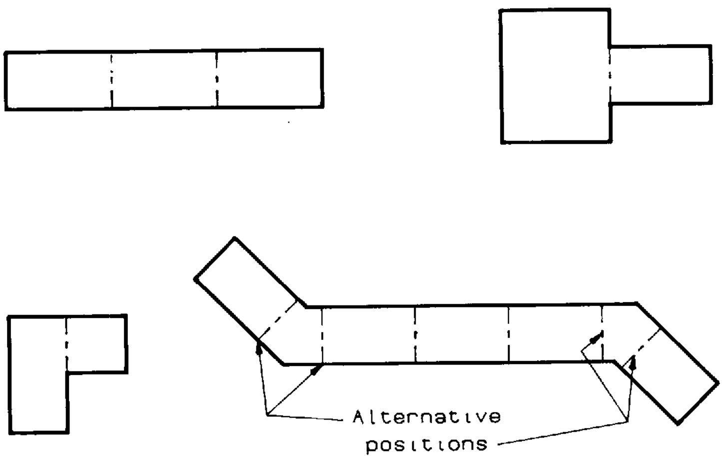
Elements whose failure would cause collapse of more than a limited part of the structure adjacent to them should be avoided. Where this is not possible, alternative load paths should be identified or the element in question strengthened.

2.4 Movement joints

Movement joints should 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. 1.



1 Location of movement joints

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, 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.

Attention should be drawn to the necessity of ensuring that joints are incorporated in the finishes and in the cladding at the movement joint locations.

2.5 Fire resistance and durability

In order for a structural member to be able to carry its load during and after a fire its size may need to be greater than that which is dictated by purely structural considerations. Similarly, the cover to reinforcement necessary to ensure durability may dictate the lower limit of the cross-sectional dimensions.

2.6 Loading

This *Manual* adopts the limit-state principle and the partial factor format of BS 8110. The loads to be used in calculations are therefore:

(a) Characteristic dead load, G_k: the weight of the structure complete with finishes, fixtures and fixed partitions (BS 648[3])
(b) Characteristic imposed load, Q_k (BS 6399, Part 1[4])
(c) Characteristic wind load, W_k (CP 3, Chapter V, Part 2[5])
(d) **Nominal earth load, E_n (CP 2004[6])**
(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) the wind load derived from CP 3, Chapter V, Part 2,[5] multiplied by the appropriate partial safety factor.

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

The design loads are obtained by multiplying the characteristic loads by the appropriate partial safety factor γ_f from Table 1.

Table 1 Partial safety factors for loads

Load combination (including earth and water loading where present)	Load type					
	dead, G_k		imposed, Q_k		**earth and water, E_n**	**wind W_k**
	adverse	beneficial	adverse	beneficial		
1. dead and imposed	1.4	1.0	1.6	0	1.4*	—
2. dead and wind	1.4	1.0	—	—	1.4*	1.4
3. dead, wind and imposed	1.2	1.2	1.2	1.2	1.2	1.2

*For pressures arising from an accidental head of water at ground level a partial factor of 1.2 may be used.

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

2.7 Serviceability limit states

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

2.8 Material design stresses

Design stresses are given in the appropriate sections of the *Manual*. The partial safety factors for strength of materials, γ_m, are the same as those given in BS 8110.[1]

3 Initial design

3.1 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.

3.2 Loads

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

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.

Dead loading on plan 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	$0.5 kN/m^2$
demountable lightweight partitions	$1.0 kN/m^2$
blockwork partitions	$2.5 kN/m^2$

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

The design ultimate load should be obtained as follows:

(a) dead load + imposed load

1.4 × characteristic dead load + 1.6 × characteristic imposed load

(b) dead load + wind load

1.0 × characteristic dead load + 1.4 × characteristic wind load or
1.4 × characteristic dead load + 1.4 × characteristic wind load

(c) dead load + imposed load + wind load

1.2 × all characteristic loads

3.3 Material properties

For normal construction in the UK, a characteristic concrete strength of $30N/mm^2$ should be assumed for the initial design. In areas with poor aggregates this may have to be reduced. In the final design a higher grade concrete may have to be specified to meet durability requirements.

In the UK a characteristic strength of $460N/mm^2$ should be used for high-tensile reinforcement and $250N/mm^2$ for mild steel. European and American steel may, for some time to come, have different yield strengths, and corresponding values should be used.

3.4 Structural form and framing

The following measures should be adopted:

(i) provide stability against lateral forces and ensure braced construction by arranging suitable shear walls deployed symmetrically wherever possible
(ii) adopt a simple arrangement of slabs, beams and columns so that loads are carried to the foundations by the shortest and most direct routes
(iii) allow for movement joints (see subsection 2.4)
(iv) choose an arrangement that will limit the span of slabs to 5–6m and beam spans to 8–10m on a regular grid; for flat slabs restrict column spacings to 8m
(v) adopt a minimum column size of 300 × 300mm or equivalent area
(vi) ensure robustness of the structure, particularly if precast construction is envisaged.

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.

3.5 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 2 shows the minimum practical member sizes for different periods of fire resistance and the cover to the main reinforcement required for continuous members in mild and moderate environments. For severe exposures, covers should be increased. For simply supported members, sizes and covers should be increased (see Section 4).

Table 2 Minimum member sizes and cover for initial design of continuous members

Member	Minimum dimension, mm	Fire rating 4h	2h	1h
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

3.6 Stiffness

3.6.1 Slabs

To ensure adequate stiffness, the depths of slabs and the waist of stairs should not be less than those derived from Table 3.

Table 3 Span/effective depth ratios for initial design of slabs

Characteristic imposed loading (including finishes) kN/m²	One-way spanning			Two-way spanning		Flat slab without drops
	simply sup-ported	contin-uous	canti-lever	simply supported	continuous	
5.0	27	31	11	30	40	36
10.0	24	28	10	28	39	33

The ratios for two-way slabs have been calculated for a square panel. For a 2 × 1 panel, the ratio for a one-way panel 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 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 3 should be used.

3.6.2 Beams
Beams should be of sufficient depth to avoid the necessity for excessive compression reinforcement and to ensure that an economical amount of tension and shear reinforcement is provided. This will also facilitate the placing of concrete. For initial sizing the effective depth should therefore be determined from Table 4. If other considerations demand shallower construction, reference should be made to subsection 4.4.

Table 4 Span/effective depth ratios for initial design of beams

cantilever	6
simply supported	12
continuous	15

For spans greater than 10m the effective depth ratios should be multiplied by 10/(span in metres).

3.7 Sizing

3.7.1 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.

3.7.2 Loading
Ultimate loads, i.e. characteristic loads multiplied by the appropriate partial safety factors, should be used throughout. At this stage it may be assumed that all spans are fully loaded, unless the members concerned are sensitive to unbalanced loading.

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.

3.7.3 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 $f_{cu} \geqslant 30$N/mm^2:

$$\text{width of beam (in mm)} = \frac{1000V}{2d} \qquad \text{width of rib (in mm)} = \frac{1000V}{0.6d}$$

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.

For f_{cu} <30N/mm^2 the width should be increased in proportion.

3.7.4 Sizes and reinforcement of columns
Stocky columns should be used, i.e. columns for which the ratio of the effective height to the least lateral dimension does not exceed 15, where the effective height equals 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 following factors:

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

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.

The ultimate loads that can be carried by columns of different sizes and different reinforcement percentages p may be obtained from Table 5 for f_{cu}=30N/mm^2 and f_y=460N/mm^2.

Table 5 Ultimate loads for stocky columns

Column size* mm × mm	Cross-sectional area, mm^2	p=1% kN	p=2% kN	p=3% kN	p=4% kN
300 × 300	90 000	1213	1481	1749	2016
300 × 350	105 000	1415	1728	2040	2353
350 × 350	122 500	1651	2016	2380	2745
400 × 350	140 000	1887	2304	2720	3137
400 × 400	160 000	2156	2633	3109	3585

*Provided that the smallest dimension is not less than 200mm, any shape giving an equivalent area may be used.

The values of the cross-sectional areas in Table 5 are obtained by dividing the total ultimate load, factored as above, by a 'stress' that is expressed as:

$$0.35f_{cu} + \frac{p}{100}(0.67f_y - 0.35f_{cu})$$

where f_{cu} is the characteristic concrete strength in N/mm^2
f_y the characteristic strength of reinforcement in N/mm^2 and
p the percentage of reinforcement.

3.7.5 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 walls have returns at the compression end, they should be treated as flanged beams.

3.7.6 Shear in flat slabs at columns

Check that:

$$\frac{1250\ w\ (\text{area supported by column})}{(\text{column perimeter} + 9h)d} \leq 0.6\text{N/mm}^2$$

where w is the total 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, and areas are in m^2.

Check also that:

$$\frac{1250\ w\ (\text{area supported by column})}{(\text{column perimeter})\ d} \leq 0.8\sqrt{f_{cu}} \text{ or } 5\text{N/mm}^2$$

whichever is the lesser.

3.7.7 Adequacy of chosen sections to accommodate the reinforcement, bending moments 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 6 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.

Table 6 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.

Alternatively, bending moments and shear forces may be obtained by moment distribution.

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:

$$w\,\frac{l_x\,l_y}{24}\ \text{kNm per metre}$$

where w is the 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:

$$w \frac{l_x\, l_y}{24} c \text{ kNm per rib}$$

where c is the rib spacing in metres.
If $l_y > 1.5l_x$ 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 6. For the column strips increase this by 15%.

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

$$A_s = \frac{M}{0.87 f_y \times 0.8d}$$

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

Compression reinforcement
If, for a rectangular section, $M > 0.15 f_{cu}bd^2$, compression reinforcement is required:

$$A'_s = \frac{M - 0.15 f_{cu}bd^2}{0.87 f_y (d - d')}$$

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 its effective depth.*

If, for flanged sections, $M > 0.4 f_{cu} b_f h_f (d - 0.5h_f)$ 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 $0.5d$.

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 nor more than 8 bars. In slabs, the bar spacing should not be less than 150mm nor more than 300mm; the bars should not be less than size 10 nor normally more than size 20.

3.8 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 comments, 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 in the light of comments received. The amended design should form the basis for the architect's drawings and may also be used for preparing reinforcement estimates for budget costings.

---

*Consistent units need to be used in the formula.

3.9 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 (SMM)*.[7] 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), size 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 1

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 $15.0m^3$.

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*[7] 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 sizes 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 final 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 10–15% should be added to cater for some changes and for possible omissions.

4 Final design

4.1 Introduction

Section 3 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 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.

4.1.1 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 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.

4.1.2 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.

4.1.3 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:

Grid lines
Establish grid lines 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
Mark on the preliminary drawings the loads that are to be carried by each slab. 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.

4.1.4 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 remaining sections of the *Manual* have been laid out in the following order, which should be followed in most cases:

slabs
structural frames
beams
columns
walls
staircases
retaining walls, basements
foundations
robustness and
detailing.

There will be occasions when this sequence cannot be adhered to, e.g. when the foundation drawings are required before the rest of the structural drawings are completed. In such instances extra care is required in assessing the loads and other requirements of the superstructure design.

4.2 Slabs

4.2.1 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 size of 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, flat slabs, and ribbed and coffered slabs. 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 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 from a moment distribution analysis. These moments may then be redistributed up to a maximum of 30%, although normally 15% is considered a reasonable limit. The following criteria should be observed:

(a) Equilibrium must be maintained
(b) The redistributed design moment at any section should not be less than 70% of the elastic moment.

The general procedure to be adopted is as follows:

1. Check that the section complies 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. Make final check on span/depth ratios
5. Calculate reinforcement
6. For flat slabs check shear around columns and calculate shear reinforcement if found to be necessary.

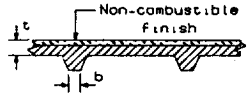
4.2.2 Fire resistance and durability

4.2.2.1 Fire resistance

The member sizes and reinforcement covers required to provide fire resistance are given in Table 7. The covers in the Table may need to be increased to ensure durability (see clause 4.2.2.2).

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[1]).

Table 7 Fire resistance requirements for slabs

Fire resistance h	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** 	
	Simply supported	Continuous	Simply supported	Continuous
			t/b	*t/b*
1	95	95	90/90	90/80
1½	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
1½	25	20	35	25
2	35	25	45	35
3	45	35	55	45
4	55	45	65	55

If the width of the rib is more than the minimum in Table 7 the cover may be decreased as below:

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

4.2.2.2 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 thickness of cover to the reinforcement
(d) good compaction and
(e) adequate curing.

Values for *(a)*, *(b)* and *(c)* which, in combination, will be adequate to ensure durability are given in Table 8 for various environments.

As *(a)* and *(b)* at present cannot be checked by methods that are practical for use during construction, Table 8 gives, in addition, the characteristic strengths that have to be specified in the UK to ensure that requirements *(a)* and *(b)* are satisfied.

Table 8 Durability requirements for slabs

Conditions of exposure (For definitions see Appendix C)	Cover to *all* reinforcement mm	mm	mm
Mild	25	20	20
Moderate	—	35	30
Severe	—	—	40
Very severe	—	—	50
Maximum free water/cement ratio	0.65	0.60	0.55
Minimum cement content, kg/m^3	275	300	325
Characteristic concrete strength in the UK, N/mm^2	30	35	40

Notes to Table 8
1. **The cover to *all* reinforcement should not be less than the nominal maximum size of the aggregate.**
2. **The cover in mm to the *main* reinforcement should not be less the the bar size.**

The characteristic strengths quoted in Table 8 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.

4.2.3 Bending moments and shear forces

4.2.3.1 General

Slabs should be designed to withstand the most unfavourable arrangements of design loads.

Design for a single load case of maximum design ultimate load on all spans or panels will be sufficient provided that the following conditions are met:

(a) In a one-way spanning slab the area of each bay exceeds $30m^2$
In this context, a bay means a strip across the full width of a structure bounded on the other two sides by lines of supports (see Fig. 2)
(b) The variation in the spans does not exceed 15% of the longest span
(c) The ratio of the characteristic imposed load to the characteristic dead load does not exceed 1.25
(d) The characteristic imposed load does not exceed $5kN/m^2$, excluding partitions
(e) In the analysis the elastic support moments other than at a cantilever support should be reduced by 20%, with a consequential increase in the span moments. The resulting bending moment envelope should satisfy the following provisions:
  (i) **Equilibrium must be maintained**
  (ii) The redistributed moment at any section should not be less than 70% of the elastic moment.

Where a cantilever of a length exceeding one-third of the adjacent span occurs, the condition of maximum load on the cantilever and minimum load on the adjacent span must be checked.

Concentrated loads

The effective width of solid slabs assumed to resist the bending moment arising from a concentrated load may be taken to be:

$$\text{width} = l_w + 2.4\left(1 - \frac{x}{l}\right)x$$

where l_w = load width
x = the distance to the nearer support from the section under consideration
l = the span

For loads near an unsupported edge see BS 8110.[1]

4.2.3.2 One-way spanning slabs of approximately equal span

Where the conditions in clause 4.2.3.1 are met, the moments and shear forces in continuous one-way spanning slabs may be calculated using the coefficients given in Table 9. Allowance has been made in these coefficients for the 20% reduction mentioned above.

Table 9 Bending moments and shear forces for one-way slabs

	end support	end span	penultimate support	interior spans	interior supports
moment	0	$0.086Fl$	$-0.086Fl$	$0.063Fl$	$-0.063Fl$
shear	$0.4F$	—	$0.6F$	—	$0.5F$

where F is the total design ultimate load ($1.4G_k + 1.6Q_k$) for each span and l is the span.

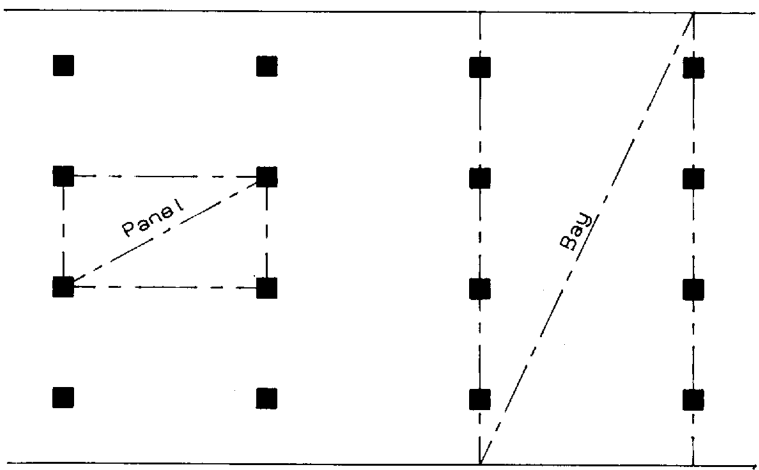
4.2.3.3 Two-way spanning slabs on linear supports

Bending moments in two-way slabs may be calculated by yield-line analysis. Alternatively, the following coefficients may be used to obtain bending moments in the two directions for slabs whose ratio of the long span to the short span is 1.5 or less and with edge conditions described in Table 10:

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

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

where β_{sx} and β_{sy} are the coefficients given in Table 10 and l_x is the shorter span.



2 Definition of panels and bays

Table 10 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			Long-span coefficients β_{sy} for all values of l_y/l_x
	1.0	1.25	1.5	
1 Interior panels				
Negative moment at continuous edge	0.031	0.044	0.053	0.032
Positive moment at midspan	0.024	0.034	0.040	0.024
2 One short edge discontinuous				
Negative moment at continuous edge	0.039	0.050	0.058	0.037
Positive moment at midspan	0.029	0.038	0.043	0.028
3 One long edge discontinuous				
Negative moment at continuous edge	0.039	0.059	0.073	0.037
Positive moment at midspan	0.030	0.045	0.055	0.028
4 Two adjacent edges discontinuous				
Negative moment at continuous edge	0.047	0.066	0.078	0.045
Positive moment at midspan	0.036	0.049	0.059	0.034

The distribution of the reactions of two-way slabs on to their supports can be derived from Fig. 3.

4.2.3.4 Flat slabs

If a flat slab has at least three spans 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 may be obtained from Table 11.

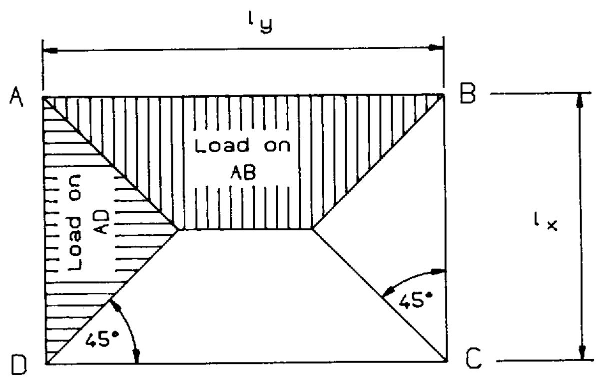
Where the conditions above do not apply, bending moments in flat slabs have to be obtained by frame analysis (see subsection 4.3). A single load case may be applicable subject to satisfying the conditions in clause 4.2.3.1. The structure should then be considered as being divided longitudinally and transversely 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.

Table 11 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	$-0.040Fl$*	$-0.02Fl$	$0.083Fl$†	$-0.063Fl$	$0.071Fl$	$-0.055Fl$
shear	$0.45F$	$0.4F$	—	$0.6F$	—	$0.5F$
total column moments	$0.040Fl$	—	—	$0.022Fl$	—	$0.022Fl$

where F is the total design ultimate load on a panel bounded by four columns and l is the effective span.

*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.

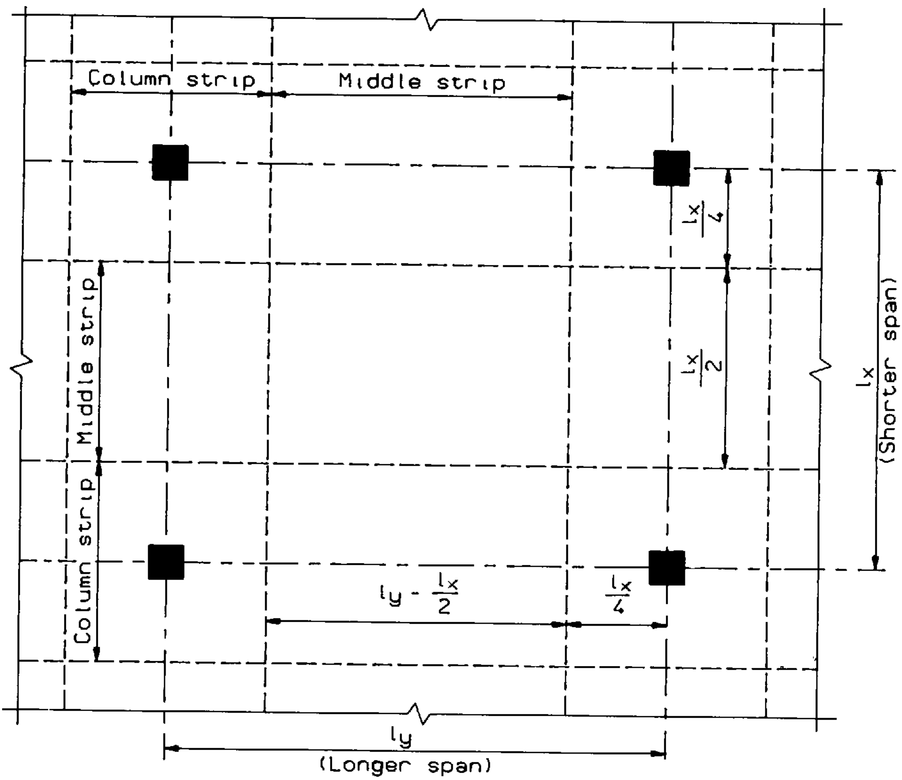


Notes

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.

3 Distribution of reactions from two-way slabs on to supports

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. 4). 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.

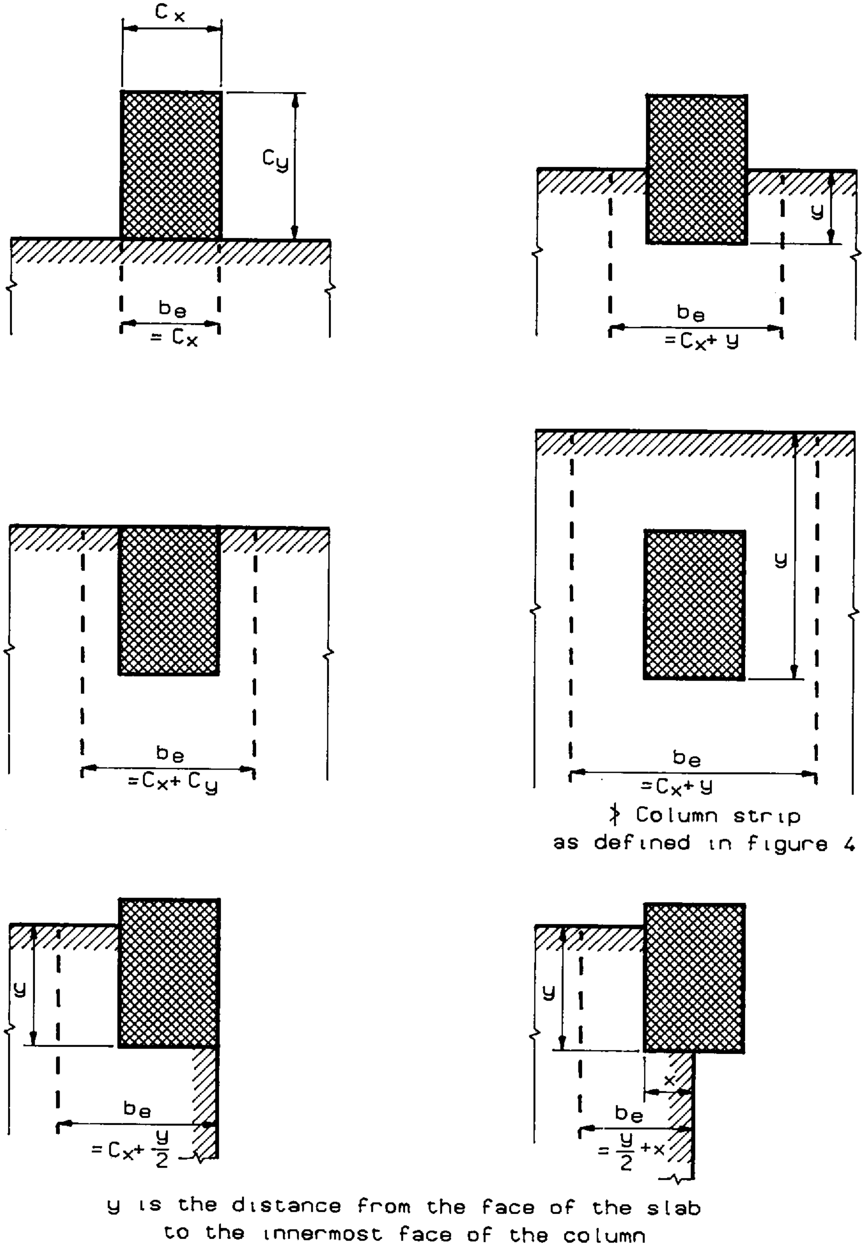


4 Division of panel without drops into strips

Division of moments between column and middle strips
The design moments obtained from analysis of the frames or from Table 11 should be divided between the column and middle strips in the proportions given below:

	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. b_e should never be taken as greater than the column strip width appropriate for an



5 Definition of breadth of effective moment transfer strip, b_e

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

$$M_{max} = 0.15 \, f_{cu} b_e d^2$$

where d is the effective depth for the top reinforcement in the column strip. The moments obtained from Table 11 or a frame analysis should be adjusted at the columns to the above values and the midspan moments increased accordingly.

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 clause 4.2.5.2 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

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.

For internal columns with unequal spans

$$V_{eff} = V_t + \frac{1.5 \, M_t}{x}$$

where x is the side of the column perimeter parallel to the axis of bending and M_t is the design moment transmitted to the column.

At corner columns and at edge columns bent about an axis parallel to the free edge, the design effective shear is $V_{eff} = 1.25 \, V_t$.

For edge columns bent about an axis perpendicular to the edge, the design effective shear is $1.4 \, V_t$ for approximately equal spans. For edge columns with unequal spans

$$V_{eff} = 1.25 \, V_t + \frac{1.5 \, M_t}{x}$$

4.2.4 Span/effective depth ratios

Compliance with the ratios below will generally limit total deflections to span/250.

4.2.4.1 Slabs on linear supports

The span/effective depth should not exceed the appropriate value in Table 12 multiplied by the modification factor in Table 13.

Table 12 Span/effective depth ratios for solid slabs

cantilever	7
simply supported	20
continuous	26

Table 13 Modification factors for M/bd^2 for slabs

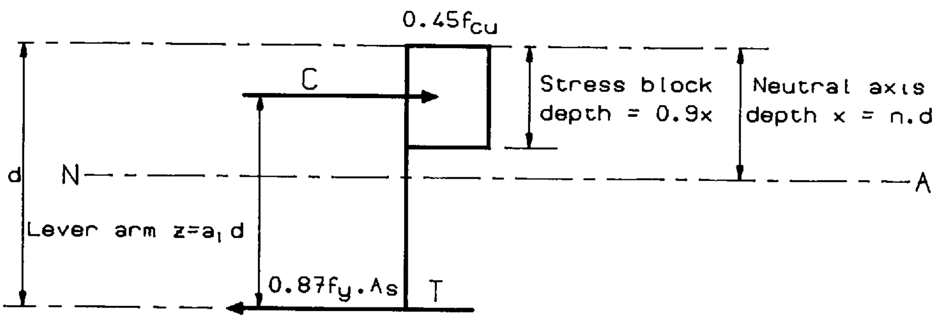
Steel stress N/mm^2	M/bd^2 0.50	0.75	1.00	1.50	2.00	3.00
(f_y = 250) 156	2.00	2.00	1.96	1.66	1.47	1.24
(f_y = 460) 288	1.68	1.50	1.38	1.21	1.09	0.95

Notes to Tables 12 and 13
1. For spans in excess of 10m, the above ratios should be multiplied by 10/(span in metres).
2. M in the Table is the design ultimate moment at the centre of the span or for a cantilever at the support.
3. For two-way slabs the ratio refers to the shorter span, and the short span moment should be used for M.

4.2.4.2 Flat slabs without drops
The ratio of the longer span to the corresponding effective depth should not exceed the values for slabs on linear supports multiplied by 0.90.

4.2.5 Section design – solid slabs

4.2.5.1 Bending
(a) Check that the applied moment is less than the moment of resistance using the formulas that are based on the stress diagram in Fig 6.



6 Stress diagram

For concrete the moment of resistance $M_u = K' f_{cu}bd^2$ where K' is obtained from below:

% moment redistribution	0 to 10	15	20	25	30
Values K'	0.156	0.144	0.132	0.119	0.104

The area of tension reinforcement is then given by:

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

where z is obtained from Table 14.

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

(b) The spacing of main bars should not exceed the lesser of:

$$3d,\ 300\text{mm, or } \frac{75\,000\beta}{pf_y}$$

where p is the reinforcement percentage and $0.3 < p < 1.0$ and

$$\beta \text{ is the ratio: } \frac{\text{moment after redistribution}}{\text{moment before redistribution.}}$$

If $p \geqslant 1$ use $p = 1$ in formula above.

Spacing of distribution bars should not exceed the lesser of:

$3d$ or 400mm.

Main bars in slabs should be not less than size 10.

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

one-quarter of the area of main reinforcement
or 0.001 3bh in the case of high yield steel
or 0.002 4bh in the case of mild steel
or, if control of shrinkage and temperature cracking is critical, 0.0025bh high yield steel or 0.003bh mild steel

where h is the overall depth of the slab in mm.

Table 14 Lever arm and neutral axis depth factors for slabs

$K = M/bd^2f_{cu}$	0.05	0.06	0.07	0.08	0.09	0.100	0.104	0.110	0.119	0.130	0.132	0.140	0.144	0.150	0.156
$a_1 = (z/d)$	0.94	0.93	0.91	0.90	0.89	0.87	0.87	0.86	0.84	0.82	0.82	0.81	0.80	0.79	0.775
$n = (x/d)$	0.13	0.16	0.19	0.22	0.25	0.29	0.30	0.32	0.35	0.39	0.40	0.43	0.45	0.47	0.50
	30%							25%		20%		15%		0–10%	

Limit of Table for various % of moment redistribution

(c) Two-way slabs on linear supports

The reinforcement calculated from the bending moments obtained from clause 4.2.3.3 should be provided for the full width in both directions.

At corners where the slab is not continuous, torsion reinforcement equal to three-quarters of the reinforcement in the shorter span should be provided in the top and bottom of the slab in each direction for a width in each direction of one-fifth of the shorter span.

(d) Flat slabs

Column and middle strips should be reinforced to withstand the design moments obtained from clause 4.2.3.4. 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.

The minimum amounts of reinforcement and the maximum bar spacing should be as stated in *(b)*.

4.2.5.2 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 should be checked first at the column perimeter:

$$v = \frac{1000V_{\text{eff}}}{U_{\text{c}}d}\ \text{N/mm}^2$$

where V_{eff} is the effective shear force in kN (see clause 4.2.3.4), d is the average effective depth in mm of both layers and U_{c} **is the column perimeter in mm.**

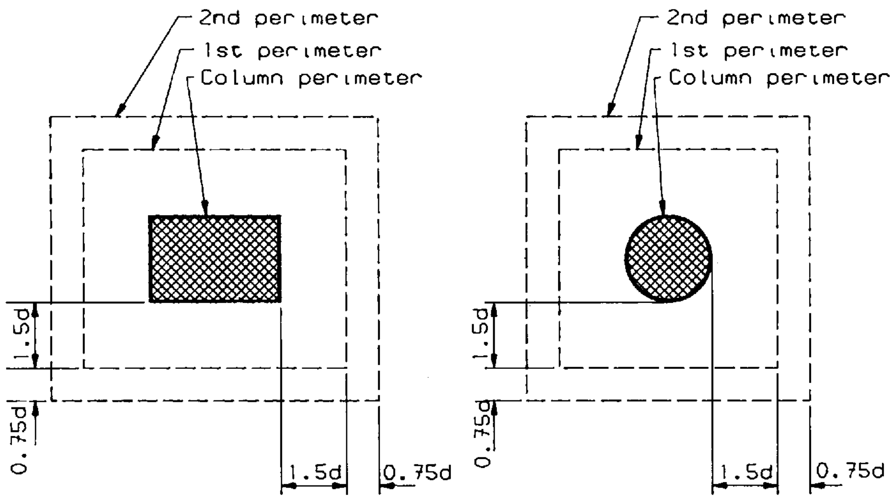
v must in this case not exceed $0.8\sqrt{f_{\text{cu}}}$ or 5N/mm^2, whichever is the lesser.

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

$$v = \frac{1000V_{\text{eff}}}{Ud}\ \text{N/mm}^2$$

where U is the shear perimeter in mm as defined in Figs. 7 and 8.

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



7 Shear perimeters for internal columns

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.

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. 9.

The first perimeter is checked. If the shear stress here is less than the permissible ultimate shear stress v_{c} in Table 15, no further checks are required. If $v > v_{\text{c}}$, successive perimeters have to be checked until one is reached where $v < v_{\text{c}}$.

Table 15 Ultimate shear stress v_c for flat slabs

$\frac{100 A_s}{bd}$	Effective depth, mm						
	150	175	200	225	250	300	≥400
0.25	0.54	0.52	0.50	0.49	0.48	0.46	0.42
0.50	0.68	0.66	0.64	0.62	0.59	0.57	0.53
0.75	0.76	0.75	0.72	0.70	0.69	0.64	0.61
1.00	0.86	0.83	0.80	0.78	0.75	0.72	0.67
1.50	0.98	0.95	0.91	0.88	0.86	0.83	0.76

Note to Table 15
The tabulated values apply for f_{cu} = 30N/mm^2
For f_{cu} = 25N/mm^2 the tabulated values should be divided by 1.062.
For f_{cu} = 35N/mm^2 the tabulated values should be multiplied by 1.053.
For f_{cu} = 40N/mm^2 the tabulated values should be multiplied by 1.10.

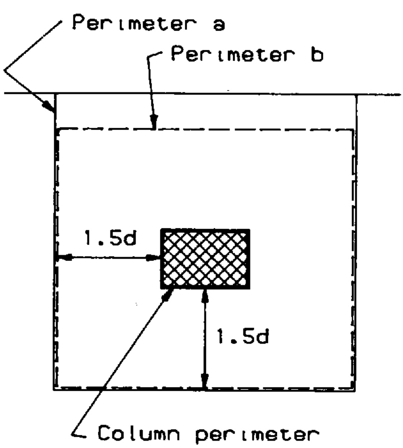
If the shear stress exceeds v_c, shear reinforcement will be necessary, unless column heads or drop panels can be incorporated in the structure. Shear reinforcement should, however, not be used in slabs thinner than 250mm.

Shear reinforcement should consist of vertical links and the total area required is:

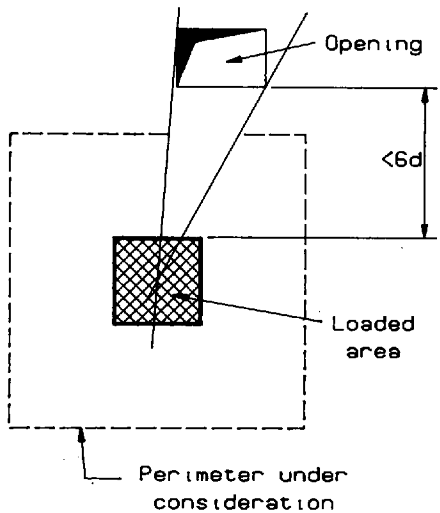
$$A_{sv} = \frac{(v - v_c)\, Ud}{0.87\, f_{yv}} \text{ mm}^2$$

where U is the perimeter in mm as previously defined
d is in mm and
f_{yv} is the characteristic strength of the shear reinforcement in N/mm^2.

$v - v_c$ should not be taken as less than 0.4N/mm^2. This reinforcement should be evenly distributed along two perimeters, the one on which v is calculated and the one $0.75d$ nearer the column face. In assessing the reinforcement required on a perimeter, any shear reinforcement on that perimeter which is derived from calculation on another perimeter may be taken into account.



8 Shear perimeter for edge column



9 Effect of opening on shear perimeter

4.2.5.3 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.

4.2.6 Section design — ribbed and coffered slabs

4.2.6.1 Bending

The bending moments per metre width obtained for solid slabs from clause 4.2.3 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 4.4. The area of tension reinforcement should be obtained from the same subsection. Structural topping should contain the minimum reinforcement indicated for solid slabs.

4.2.6.2 Span/effective depth ratios

(a) **Ribbed or coffered slabs on linear supports**

The span/effective depth ratio should not exceed the appropriate value from Table 16, multiplied by the modification factor in Table 13.

Table 16 Span/effective depth ratios for ribbed and coffered slabs

	$b_w/b=1$	$b_w/b \leq 0.3$
cantilever	7	5.6
simply supported	20	16.0
continuous	26	20.8

Notes to Table 16
1. For spans in excess of 10m, the ratios should be multiplied by 10/(span in metres).
2. b_w is the average width of the ribs.
3. b is the effective flange width.
4. For values of b_w/b between 1 and 0.33, interpolate linearly between the values in the Table.

(b) **Coffered slabs on column supports**

The ratio of the longer span to the corresponding effective depth should not exceed the values for slabs on linear supports multiplied by 0.90.

4.2.6.3 Shear

The shear force per metre width obtained from clause 4.2.3 should be multiplied by the spacing of the ribs to obtain the shear force per rib.

The shear stress should be calculated from $v = \dfrac{1000V}{b_w d}$

where v = design shear stress in N/mm^2

V = 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 exceeds the permissible shear stress v_c in Table 15 then one of the following should be adopted:

1. Increase width of rib
2. Reduce spacing of ribs
3. Provide solid concrete at supports
4. 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.

4.2.6.4 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.

4.2.7 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. The notes below may be helpful to the designer:

1. The use of a structural topping should be considered but 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 ensure a minimum bearing when constructed of 75mm on concrete beams and walls, but in cases where this bearing cannot be achieved reference should be made to BS 8110 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 4.11)
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.

4.3 Structural frames

4.3.1 Division into sub-frames

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 sub-frames. Each sub-frame 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 sub-frames should be used. Other than for end spans of a frame, sub-frames 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 concrete section ignoring reinforcement.

For the purpose of calculating the stiffness of flanged beams the flange width of T-beams should be taken as 0.14 times the effective span plus the web width and for L-beams 0.07 times the effective span plus the web width. If the actual flange width is less, this should be used.

4.3.2 Elastic analysis

The loading to be considered in the analyses should be that which provides the greater values of moments and shears for the following two cases:

all spans with maximum ultimate load ($1.4G_k + 1.6Q_k$)

alternate spans with maximum ultimate load and all other spans with minimum ultimate loads ($1.0G_k$).

The elastic bending moments should now be calculated.

4.3.3 Redistribution of moments

The moments obtained from the elastic analysis of the frames may be redistributed up to a maximum of 30% to produce members that are convenient to detail and construct. 'Whether to redistribute and by how much to redistribute are thus matters of engineering judgment, not analysis'[8]. Normally 15% redistribution could be taken as a reasonable limit.

The criteria to be observed are:

(a) Equilibrium must be maintained for each load case

(b) The design redistributed moment at any section should not be less than 70% of the elastic moment

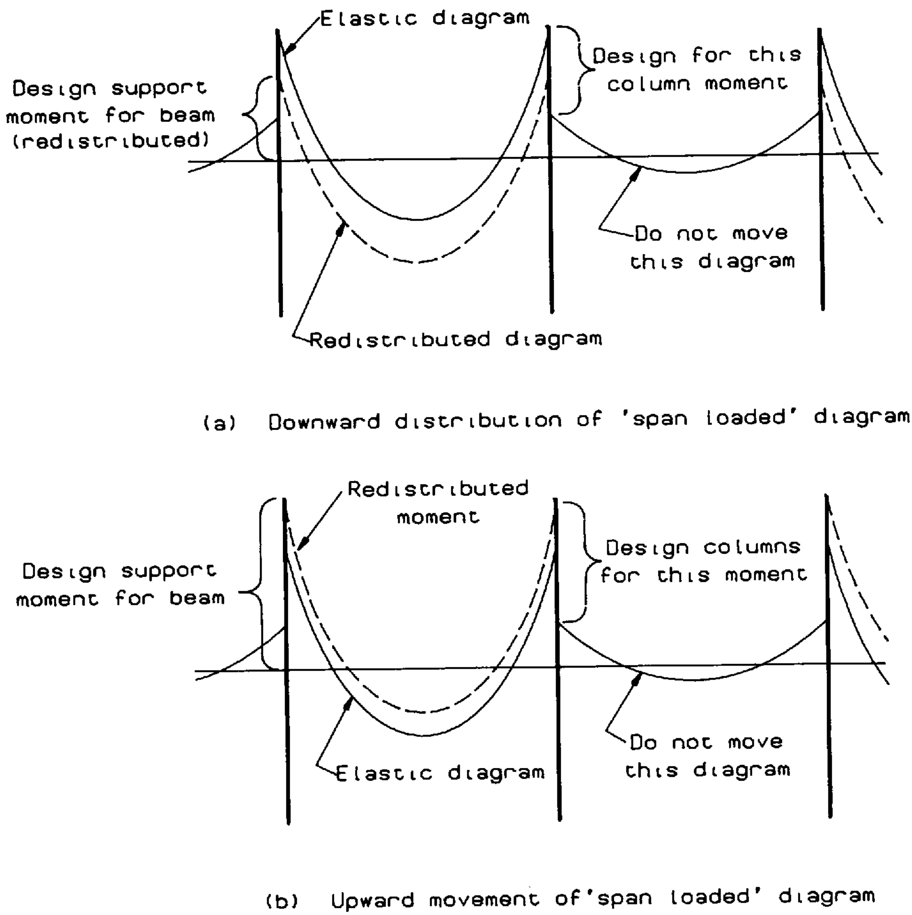
(c) The design moment for the columns should be the greater of the redistributed moment or the elastic moment prior to redistribution.

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. 10).
2. **All spans loaded**
   Move the moment diagram of the loaded spans up or down by the percentage redistribution required.

4.3.4 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 4.3.2 and any redistribution carried out in accordance with clause 4.3.3.



10 Redistribution procedures for frames

4.4 Beams

4.4.1 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 comply with durability requirements
3. calculate bending moments and shear forces according to subsection 4.3 or clause 4.4.3*(b)*
4. check span/depth ratio and determine the compression steel (if any) reauired to limit deflection
5. calculate reinforcement.

The effective span of a simply supported beam should be taken as the smaller of the following:

(*a*) the distance between the centres of bearings, or
(*b*) the clear distance between supports plus the effective depth d of the beam.

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

The effective length of a cantilever beam should normally be taken as its length to the face of the support plus half its effective depth. Where, however, it forms the end of a continuous beam, the length to the centre of the support should be used.

Slenderness: The clear distance between adequate lateral restraints to a beam should not exceed the lesser of

$60b_c$ or $250\ b_c^2/d$

where b_c is the width of the compression flange midway between the restraints. (This is not usually a limitation on beams for which a slab provides the compression flange at midspan.) For cantilevers, the length should not exceed the lesser of

$25\ b_c$, or $100b_c^2/\mathrm{d}$.

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 the arrangement of the structure is such that loads are imposed mainly on one face of a beam without corresponding rotational restraints being provided, torsion may be a problem. BS 8110[1] should be consulted for design for torsion.

4.4.2 Fire resistance and durability

4.4.2.1 Fire resistance

The member sizes and reinforcement covers required to provide fire resistance are shown in Table 17.

Table 17 Fire resistance and cover for beams

Fire resistance h	Minimum width, mm		Cover to main steel, mm	
	simply supported	con- tinuous	simply supported	con- tinuous
1	120	120	30	20
1½	150	120	40	35
2	200	150	50	50
3	240	200	70	60
4	280	240	80	70

If the width of the beam is more than the minimum in Table 17 the cover may be decreased as below:

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 (see BS 8110, Part 2[1]).

4.4.2.2 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, and
(e) adequate curing.

Values for (a), (b) and (c) which, in combination, will be adequate to ensure durability are given in Table 18 for various environments.

As (a) and (b) at present cannot be checked by methods that are practical for use during construction, Table 18 gives, in addition, the characteristic strengths that have to be specified in the UK to ensure that requirements (a) and (b) are satisfied.

Table 18 Durability requirements for beams

Conditions of exposure (For definitions see Appendix C)	Cover to *all* reinforcement mm	mm	mm
Mild	25	20	20
Moderate	—	35	30
Severe	—	—	40
Very severe	—	—	50
Maximum free water/cement ratio	0.65	0.60	0.55
Minimum cement content, kg/m^3	275	300	325
Characteristic concrete strength in the UK, N/mm^2	30	35	40

Notes to Table 18
1. The cover to *all* reinforcement should not be less than the nominal maximum size of the aggregate.
2. The cover in mm to the *main* reinforcement should not be less the the bar size.

The strengths quoted in Table 18 will often require cement contents that are higher than those given in Table. The potential problems of increased shrinkage arising from high cement and water contents should be considered in the design.

4.4.3 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 4.3 or
(b) as a beam that is continuous over its supports and capable of free rotation about them.

For beams that support substantially uniformly distributed loads over three or more spans that do not differ in length by more than 15% of the longest span, and for which the characteristic imposed load does not exceed the characteristic dead load, the values of the ultimate bending moments and shear forces should be obtained from Table 19. No redistribution of moments should be made when using values obtained from this Table.

Table 19 Design ultimate 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	$0.09Fl$	$-0.11Fl$	$0.07Fl$	$-0.08Fl$
Shear	$0.45F$	—	$0.6F$	—	$0.55F$

where l is the effective span and
$F = 1.4\ G_k + 1.6\ Q_k$

Where a cantilever of a length exceeding one-third of the adjacent span occurs, the condition of maximum load on the cantilever and minimum load on the adjoining span must be checked.

4.4.4 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value in Table 20 multiplied by the modification factor in Table 21. Compliance with these ratios will normally ensure that the total deflection does not exceed span/250.

Table 20 Span/effective depth ratios for beams

	$b_w/b = 1$	$b_w/b \leq 0.3$
cantilever	7	5.6
simply supported	20	16.0
continuous	26	20.8

Table 21 Modification factors for M/bd^2 for beams

Steel stress N/mm^2	M/bd^2 0.50	0.75	1.00	1.50	2.00	3.00	5.00
(f_y = 250) 156	2.00	2.00	1.96	1.66	1.47	1.24	1.00
(f_y = 460) 288	1.68	1.50	1.38	1.21	1.09	0.95	0.82

Notes to Tables 20 and 21

1. For spans in excess of 10m, the above ratios should be multiplied by 10/(span in metres). For exceptionally long spans the span/depth ratio may be exceeded if calculations of deflections are carried out according to BS 8110, Part 2[1].
2. M in the tables is to be taken as the moment at midspan, or for a cantilever at the support.
3. b is the effective width of the compression flange of a flanged beam or the width of a rectangular beam.
4. b_w is the average web width of the beam.
5. For values of b_w/b between 1 and 0.3, interpolate linearly between the values in the Table.

If the section is found to be inadequate, the span/depth ratio can be further modified using Table 22 which determines the percentage of compression steel required to limit deflections. If this percentage is impractical, the section should be redesigned. Any compression reinforcement determined at this stage may have to be increased to provide adequate strength (see clause 4.4.5.1).

Table 22 Modification factors for compression reinforcement for beams.

Factor	1.00	1.05	1.08	1.10	1.14	1.20	1.25	1.33	1.40	1.45	1.50
$\frac{100A'_s}{bd}$	0.00	0.15	0.25	0.35	0.50	0.75	1.0	1.5	2.0	2.5	3.0

4.4.5 Section design

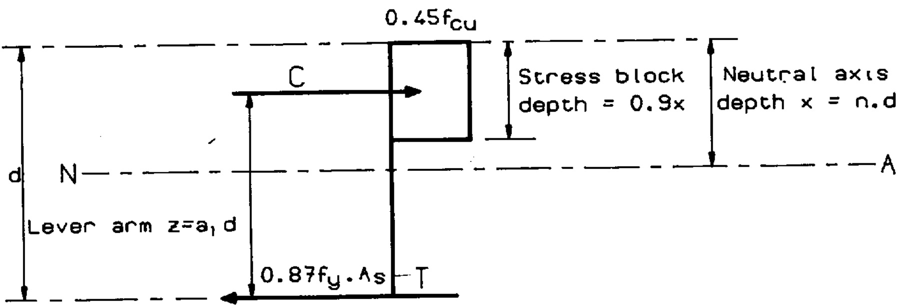
4.4.5.1 Bending

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 resistance 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 and Tables that are based on the stress diagram in Fig 11. The lever arm is assumed to be not greater than $0.95d$.

The effect of any small axial load on the beam can be ignored if the design ultimate axial force is less than $0.1\, f_{cu}bd$.



11 Stress diagram

Rectangular beams

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

(a) Calculate M_u for concrete $= K'f_{cu}bd^2$
where K' is obtained from Table 23.

Table 23 K' Factors for beams

% moment redistribution	0 to 10	15	20	25	30
Values K'	0.156	0.144	0.132	0.119	0.104

(b) If $M \leq M_u$ for the concrete, the area of tension reinforcement A_s is calculated from:

$$A_s = \frac{M}{(0.87 f_y)\, z}$$

where z is obtained from Table 24 for different values of K.

Table 24 Lever arm and neutral axis depth factors for beams

$K = M/bd^2f_{cu}$	0.05	0.06	0.07	0.08	0.09	0.100	0.104	0.110	0.119	0.130	0.132	0.140	0.144	0.150	0.156
$a_1 = (z/d)$	0.94	0.93	0.91	0.90	0.89	0.87	0.87	0.86	0.84	0.82	0.82	0.81	0.80	0.79	0.775
$n = (x/d)$	0.13	0.16	0.19	0.22	0.25	0.29	0.30	0.32	0.35	0.39	0.40	0.43	0.45	0.47	0.50
	30%							25%		20%		15%		0–10%	

Limit of Table for various % of moment redistribution

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

$$A'_s = \frac{M - M_u}{0.87 f_y (d - d')}$$

where d' is the depth of the compression steel from the compression face.

If $d' > \left(1 - \frac{f_y}{800}\right) x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.87f_y$.

The area of tension reinforcement A_s is calculated from:

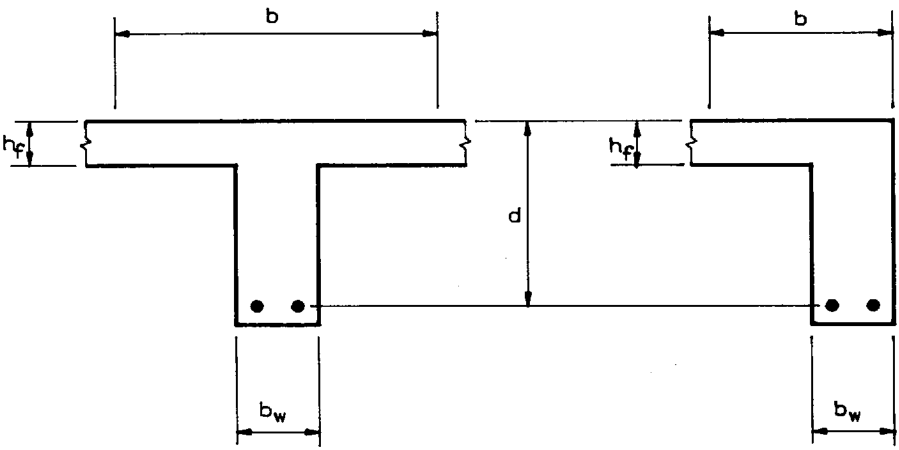
$$A_s = \frac{M_u}{0.87 f_y z} + A'_s$$

Flanged beams

For section design the effective width b of a flanged beam (see Fig. 12) should be taken as:

for T-beams: web width plus $0.2l_z$ or actual flange width if less
for L-beams: web width plus $0.1l_z$ or actual flange width if less

where l_z is the distance between points of zero moment. For a continuous beam this may be taken as 0.7 times the effective span.



12 Beam sections

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

(a) Check the position of the neutral axis by determining

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

using flange width b and selecting values of n and z from Table 24. Calculate $x = nd$.

(b) If $0.9x \leqslant h_f$ the neutral axis lies within the flange and A_s is determined as for a rectangular beam, i.e.

$$A_s = \frac{M}{0.87 f_y z}$$

(c) If $0.9x > h_f$ then the neutral axis lies outside the flange. Calculate the ultimate resistance moment of the flange M_{uf} from

$$M_{uf} = 0.45 f_{cu} (b - b_w) h_f (d - 0.5 h_f)$$

(d) Calculate $K_f = \dfrac{M - M_{uf}}{f_{cu} b_w d^2}$

where b_w is the breadth of the web.

If $K_f \leqslant K'$, obtained from Table 23, then select value of a_1 from Table 24 and calculate A_s from

$$A_s = \frac{M_{uf}}{0.87 f_y (d - 0.5 h_f)} + \frac{M - M_{uf}}{0.87 f_y z}$$

If $K_f > K'$, redesign the section or consult BS 8110[1] for design of compression steel.

4.4.5.2 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 and/or excessive cracking.

Tension reinforcement

The minimum areas of tension reinforcement are given in Table 25.

Table 25 Minimum areas of tension reinforcement for beams

	f_y=250N/mm^2	f_y=460N/mm^2
Rectangular beams with overall dimensions b and h	0.0024 bh	0.002 bh
Flanged beams (web in tension) $b_w/b<0.4$ $b_w/b \geqslant 0.4$	 0.0035 b_wh 0.0024 b_wh	 0.002 b_wh 0.002 b_wh
Flanged beams (flange in tension over a continuous support) T-beam L-beams	 0.0048 b_wh 0.0036 b_wh	 0.0026 b_wh 0.0020 b_wh
Transverse reinforcement in flanges of flanged beams (may be slab reinforcement)	0.0015 h_f per metre width	0.0015 h_f per metre width

Compression reinforcement
The minimum areas of compression reinforcement should be:

rectangular beam	$0.002\,bh$
flanged beam web in compression	$0.002\,b_w h$

Maximum area of reinforcement
Neither the area of tension reinforcement, nor the area of compression reinforcement should exceed $0.04\ b_w h$.

Main bars in beams should normally be not less than size 16.

Minimum area of bars in the side face of beams (to control cracking)
Where the overall depth of the beam exceeds 500mm, longitudinal bars should be provided at a spacing not exceeding 250mm. The size of the bars should not be less than

$0.75\ \sqrt{b_w}$ for high yield bars (f_y=460N/mm^2)

$1.00\ \sqrt{b_w}$ for mild steel bars (f_y=250N/mm^2)

where b_w is the width of the web for flanged beams and the beam width for rectangular beams. b_w need not be assumed to be greater than 500 mm.

Maximum spacing of tension bars
The clear space between main bars should not exceed the values in Table 26.

Table 26 Clear distance between bars in mm according to percentage redistribution

f_y N/mm^2	Redistribution to or from section considered						
	−30%	−20%	−10%	0%	+10%	+20%	+30%
250	210	240	270	300	300	300	300
460	115	130	145	160	180	195	210

Minimum spacing
The horizontal distance between bars should not be less than the bar size or the maximum size of the aggregate plus 5mm.

Where there are two or more rows the gaps between corresponding bars in each row should be vertically in line, and the vertical distance between bars should not be less than

two-thirds the maximum size of the aggregate or

when the bar size is greater than the maximum aggregate size plus 5mm, a spacing less than the bar size should be avoided.

4.4.5.3 Shear
The shear stress v at any point should be calculated from:

$$v = \frac{1000V}{b_w d}\ \text{N/mm}^2$$

where V is the ultimate shear force in kN
b_w is the width of the beam web in mm and
d is the effective depth in mm.

In no case should v exceed $0.8\ \sqrt{f_{cu}}$ or 5N/mm^2 even if shear reinforcement is provided.

The shear stress, v_c, which the concrete on its own can be allowed to resist, is given in Table 27 for various percentages of bending reinforcement and various effective depths for 30N/mm^2 concrete.

Table 27 Ultimate shear stresses v_c (N/mm^2) for beams

$\frac{100\,A_s}{b_w d}$	Effective depth, mm						
	150	175	200	225	250	300	≥400
≤0.15	0.46	0.44	0.43	0.41	0.40	0.38	0.36
0.25	0.54	0.52	0.50	0.49	0.48	0.46	0.42
0.50	0.68	0.66	0.64	0.62	0.59	0.57	0.53
0.75	0.76	0.75	0.72	0.70	0.69	0.64	0.61
1.00	0.86	0.83	0.80	0.78	0.75	0.72	0.67
1.50	0.98	0.95	0.91	0.88	0.86	0.83	0.76
2.00	1.08	1.04	1.01	0.97	0.95	0.91	0.85
≥3.00	1.23	1.19	1.15	1.11	1.08	1.04	0.97

Note to Table 27

The tabulated values apply for f_{cu} = 30N/mm^2

For f_{cu} = 25N/mm^2 the tabulated values should be divided by 1.062.

For f_{cu} = 35N/mm^2 the tabulated values should be multiplied by 1.053.

For f_{cu} = 40N/mm^2 the tabulated values should be multiplied by 1.10.

The term A_s relates to that area of longitudinal tension reinforcement that continues for a distance d beyond the section being considered. At supports the full area of tension reinforcement at the section may be considered, provided that the normal rules for curtailment and anchorage are met.

Shear reinforcement in the form of vertical links should be provided in accordance with the minimum areas shown in Table 28.

The spacing of links in the direction of the span should not exceed 0.75d. At right-angles to the span the horizontal spacing should be such that no longitudinal tension bar is more than 150mm from a tension leg of a link; this spacing should in any case not exceed d.

Table 28 Minimum provision of links in beams

value of v N/mm^2	Area of shear reinforcement	
Less than $0.5v_c$	**Grade 250 (mild steel) links equal to 0.18% of the horizontal section throughout the beam, except in members of minor structural importance such as lintels**	
$0.5v_c < v < (v_c + 0.4)$	Minimum links for whole length of beam	$A_{sv} > \dfrac{0.4\,b_w\,S_v}{0.87\,f_{yv}}$
$(v_c + 0.4) < v$	Links only provided	$A_{sv} > b_w \dfrac{S_v\,(v - v_c)}{0.87\,f_{yv}}$

where b_w is the width in mm of (the web of) the beam

S_v is the spacing of the links in mm

A_{sv} is the total cross-section of the link(s) in mm^2 (2 legs for a single closed link, 4 legs for double closed links) and

f_{yv} is the characteristic strength of the links in N/mm^2

Enhanced shear strength of sections close to supports
For beams carrying a generally uniform load or where the principal load is located further than $2d$ from the face of the support, the shear stress may be calculated at a section a distance d from the face of the support. If the corresponding amount of shear reinforcement is provided at sections closer to the support, then no further check for shear at such sections is required.

Arrangement of links
For compression reinforcement in an outer layer, every corner bar and alternate bar should be supported by a link passing round the bar and having an included angle of not more than 135°. No bar within a compression zone should be further than 150mm from a restrained bar.

Where slabs are supported at the bottom of the beams, the links should be designed to carry the reaction from the slab in tension in addition to any shear forces.

Openings
In locations where the design shear stress is less than the permissible stress, 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.

4.5 Columns

4.5.1 Introduction
This subsection describes the final design of stocky columns resisting axial loads and bending moments. A method is given for biaxial bending.

The general procedure to be adopted is as follows:

1. check that the column is not slender
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 4.5.3
5. design section and reinforcement.

4.5.2 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 and durability. To facilitate concreting the minimum dimension of a column should not be less than 200mm.

4.5.2.1 Slenderness
The ratio of the effective height of a stocky column to its least cross-sectional dimension should be 15 or less. The effective height should be obtained by multiplying the clear height between the lateral restraints at the two ends of the column by the factor obtained from Table 29.

4.5.2.2 Fire resistance
Minimum dimensions and covers are given in Table 30.

Table 29 Effective height 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.

Table 30 Fire resistance requirements for columns

Fire rating h	Minimum dimension mm: Fully exposed	50% exposed	One side exposed	Cover to main reinforcement mm
1	200	200	200	25
1½	250	200	200	30
2	300	200	200	35
3	400	300	200	35
4	450	350	240	35

4.5.2.3 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 to the thickness of the cover to the reinforcement
(d) good compaction and
(e) adequate curing.

Values for *(a)*, *(b)* and *(c)* that, in combination, will be adequate to ensure durability are given in Table 31 for various environments.

As *(a)* and *(b)* at present cannot be checked by methods that are practical for use during construction, Table 31 gives, in addition, the characteristic strengths that have to be specified in the UK to ensure that requirements *(a)* and *(b)* are satisfied.

The strengths quoted in Table 31 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 31 Durability requirements for columns

Conditions of exposure (For definitions see Appendix C)	Cover to *all* reinforcement mm	mm	mm
Mild	25	20	20
Moderate	—	35	30
Severe	—	—	40
Very severe	—	—	50
Maximum free water/cement ratio	0.65	0.60	0.55
Minimum cement content, kg/m^3	275	300	325
Characteristic concrete strength in the UK, N/mm^2	30	35	40

Notes to Table 31
1. **The cover to *all* reinforcement should not be less than the nominal maximum size of the aggregate.**
2. The cover in mm to the *main* reinforcement should not be less than the bar size.

4.5.3 Axial loads and moments

The minimum design moment for any column in any plane should be obtained by multiplying the ultimate design axial load by an eccentricity, which should be taken as 0.05 times the overall column dimension in the relevant plane but not exceeding 20mm.

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 the loadings on them are grossly dissimilar.

(b) The moments in the columns may be obtained using the subframes shown in Fig. 13, subject to the minimum design moments above.

Alternatively, axial loads and moments may be obtained from the frame analysis outlined in subsection 4.3.

4.5.4 Section design

Sections should normally be designed using the charts in Appendix D. Alternatively, the following simplified procedures may be adopted where applicable:

(a) In the case of columns where only the minimum design moment (see clause 4.5.3) applies, the ultimate axial load capacity in N of the column may be taken as

$$0.4\, f_{cu}A_c + 0.75\, f_yA_{sc}$$

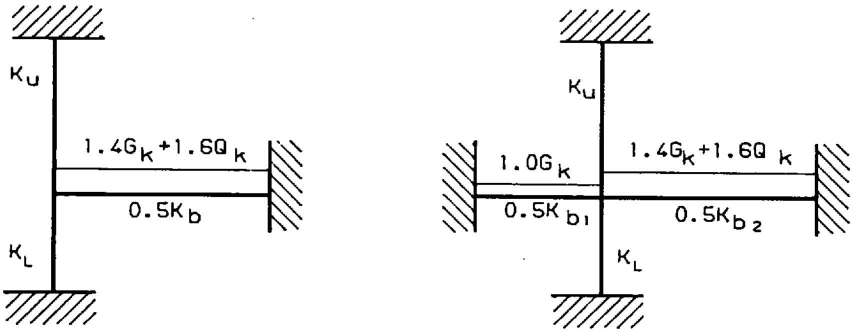
where f_{cu} = characteristic concrete cube strength in N/mm^2
A_c = area of concrete in mm^2
A_{sc} = area of longitudinal reinforcement in mm^2
f_y = characteristic strength of reinforcement in N/mm^2

(b) In the case of columns supporting an approximately symmetrical arrangement of beams (i.e. where adjacent spans do not differ by more than 15%), subject to uniformly distributed loads, the ultimate axial load capacity of the column may be taken as:

$$0.35\, f_{cu}A_c + 0.67\, f_yA_{sc}$$

where the terms have the same definitions as above.

External columns | Internal columns



$$MF_u = M_e \frac{K_u}{K_L+K_u+0.5K_b} \qquad MF_u = M_{es} \frac{K_u}{K_L+K_u+0.5K_{b1}+0.5K_{b2}}$$

$$MF_L = M_e \frac{K_L}{K_L+K_u+0.5K_b} \qquad MF_L = M_{es} \frac{K_L}{K_L+K_u+0.5K_{b1}+0.5K_{b2}}$$

M_e =Fixed end beam moment
M_{es} =Total out of balance fixed end beam moment
MF_u =Framing moment in upper column
MF_L =Framing moment in lower column
K_u =Stiffness of upper column
K_L =Stiffness of lower column
K_{b1} =Stiffness of left hand beam
K_{b2} =Stiffness of right hand beam

13 Subframes for column moments

4.5.5 Biaxial bending

Where it is necessary to consider bending about both axes, a symmetrically reinforced rectangular column section may be designed by increasing the moment about one of the axes using the procedure outlined below.

When $\dfrac{M_x}{M_y} \geqslant \dfrac{h'}{b'}$ the increased moment about the x–x axis is $M_x + \beta \dfrac{h' M_y}{b'}$

If $\dfrac{M_x}{M_y} < \dfrac{h'}{b'}$ the increased moment about the y–y axis is $M_y + \beta \dfrac{b' M_x}{h'}$

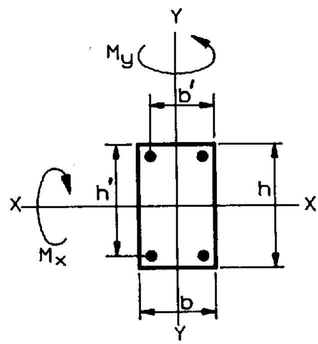
where b' and h' are the effective depths (see Fig. 14) and
β is obtained from Table 32.

Table 32 Enhancement coefficients for biaxial bending

$\frac{N}{bh\,f_{cu}}$	0	0.1	0.2	0.3	0.4	0.5	≥0.6
β	1.00	0.88	0.77	0.65	0.53	0.42	0.30

where N is the design ultimate axial load in N and
b and h are in mm (see Fig. 14).

The section should then be designed by the charts in Appendix D for the combination of N and the relevant enhanced moment.



14 Biaxial bending in columns

4.5.6 Reinforcement

Minimum area of reinforcement should be 0.4% of the gross cross-sectional area of concrete.

Longitudinal bars should not be less than size 12.

Maximum area of reinforcement (other than at laps) should be 6% of the gross cross-sectional area of concrete but 4% is generally preferable. At laps the maximum total percentage should be 10%.

Maximum spacing of main bars should not exceed 250mm.

Columns should be provided with links whose size should be the greater of:

one-quarter the size of the largest longitudinal bar or size 6*.

Every corner bar and each alternate bar in an outer layer of reinforcement should have a link passing around it. The included angle of the links should not be more than 135° except for hoops or spirals in circular columns. No bar within a compression zone should be further than 150mm from a bar restrained by a link. The maximum spacing of links should be 12 times the size of the smallest compression bar but not more than the smallest cross-sectional dimension of the column.

*This bar size may not be freely available.

4.6 Walls

4.6.1 Introduction

This subsection describes the final design of stocky reinforced concrete walls that may provide the lateral stability to reinforced concrete framed buildings.

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 that walls within any storey height are not slender
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 4.6.3
6. design section and reinforcement.

4.6.2 Slenderness, fire resistance and durability

4.6.2.1 Slenderness

The ratio of the effective height of stocky walls to their thickness should be 15 or less. The thickness should not be less than 150mm, but to facilitate concreting 180mm is preferable. The effective height should be obtained by multiplying the clear height between floors by the factor obtained from Table 33.

Table 33 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.

4.6.2.2 Fire resistance

The minimum dimensions and covers should be obtained from Table 34.

4.6.2.3 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 thickness of the cover to the reinforcement
(d) good compaction, and
(e) adequate curing.

Values for *(a)*, *(b)* and *(c)* that, in combination, will be adequate to ensure durability are given in Table 35 for various environments.

As *(a)* and *(b)* at present cannot be checked by methods that are practical for use during construction, Table 35 gives, in addition, the characteristic strengths that have to be specified in the UK to ensure that requirements *(a)* and *(b)* are satisfied.

Table 34 Fire resistance requirements for walls

Fire rating h	Minimum dimension mm	Cover to vertical reinforcement mm
1	150†	25
1½	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 35 Durability requirements for walls above ground

Conditions of exposure (For definitions see Appendix C)	Cover to *all* reinforcement mm	mm	mm
Mild	25	20	20
Moderate	—	35	30
Severe	—	—	40
Very severe	—	—	50
Maximum free water/cement ratio	0.65	0.60	0.55
Minimum cement content: kg/m^3	275	300	325
Characteristic concrete strength in the UK (N/mm^2)	30	35	40

Notes to Table 35
1. The cover to *all* reinforcement should not be less than the nominal maximum size of the aggregate.
2. The cover in mm to the *main* reinforcement should not be less the the bar size.

The strengths quoted in Table 35 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.

4.6.3 Axial loads and moments

4.6.3.1 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 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 2.6(*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.

4.6.3.2 Bending at right-angles to the walls
The axial loads and in-plane moments should be determined as in clause 4.6.3.1. 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.

4.6.4 Section design

4.6.4.1 Walls resisting in-plane moments and axial loads
The stresses on the walls from the loads and moments should be obtained from the following expression:

$$\text{extreme fibre stresses, } f_f = \frac{N}{Lh} \pm \frac{M}{hL^2/6} \text{N/mm}^2$$

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

The ultimate compressive load per unit length equals hf_f N/mm. This should be equal to or less than the ultimate load capacity

$$0.35f_{cu}A_c + 0.67f_y A_{sc}$$

where f_{cu} = characteristic concrete cube strength in N/mm^2
A_c = area of concrete in mm^2 per mm length of wall
A_{sc} = area of vertical reinforcement in mm^2 per mm length of wall
f_y = characteristic strength of reinforcement in N/mm^2

The area of tension reinforcement if required should be obtained by calculating the total tensile force from the following expression:

$$\text{total tension} = 0.5f_t L_t h$$

where f_t is the extreme fibre stress in tension in N/mm^2 and L_t is the length of the wall in mm where tension occurs.

The area of tension reinforcement should be placed within $0.5L_t$ from the end of the wall where the maximum tensile stress occurs.

The section should generally be designed on the assumption that the in-plane moments can act in both directions and should be reinforced accordingly.

4.6.4.2 Walls resisting in-plane moments, axial loads and transverse moments
The section should firstly be designed for the case in clause 4.6.4.1. The section should then be checked for the transverse moments, treating each unit length as a column and additional reinforcement provided if necessary.

4.6.4.3 Intersecting walls
Where two walls intersect to form a core the interface shear may need to be checked.

4.6.5 Reinforcement

The minimum area of vertical reinforcement in the wall should be 0.4% of the gross cross-sectional area of the concrete on any unit length, 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 in a metre length.

When the vertical reinforcement does not exceed 2% of the gross cross-sectional area, the area of horizontal reinforcement should not be less than 0.3% for steel of f_y = 250 N/mm^2 and 0.25% for f_y = 460 N/mm^2.

The vertical bars should not be less than size 10, and the horizontal bars should not be less than size 6* or one-quarter of the size of the vertical bars, whichever is the greater.

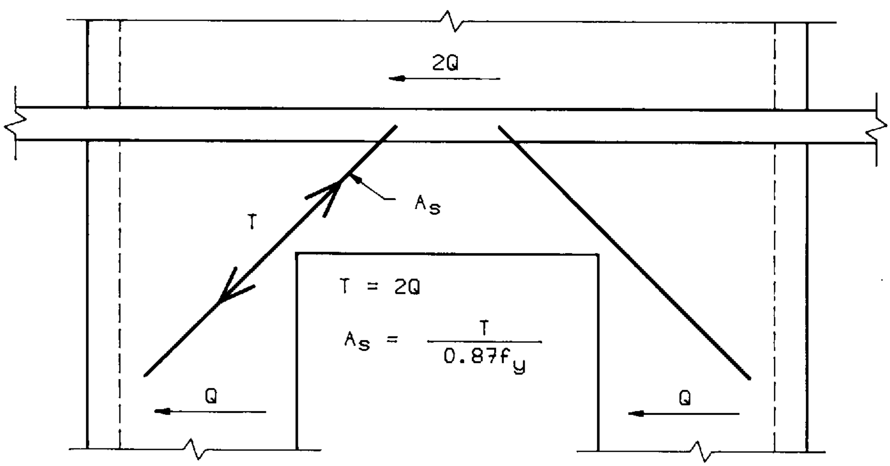
The maximum spacing of vertical bars should not exceed 250mm for steel of f_y = 250 N/mm^2 and 200mm for f_y = 460 N/mm^2.

The maximum spacing of horizontal bars should not exceed 300mm.

For walls with vertical reinforcement exceeding 2% of the gross cross-sectional area the recommendations in BS 8110[1] should be used.

4.6.6 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. This reinforcement should take the form of diagonal bars positioned at the corners of the openings as illustrated in Fig. 15. 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 size 16 bars across each corner of the opening.



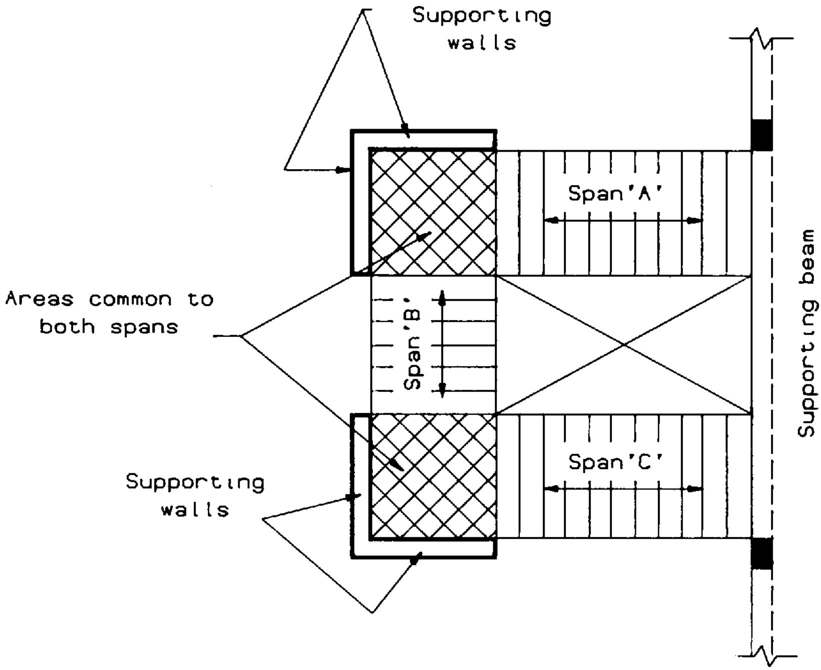
15 Reinforcement at openings in walls

*This bar size may not be freely available.

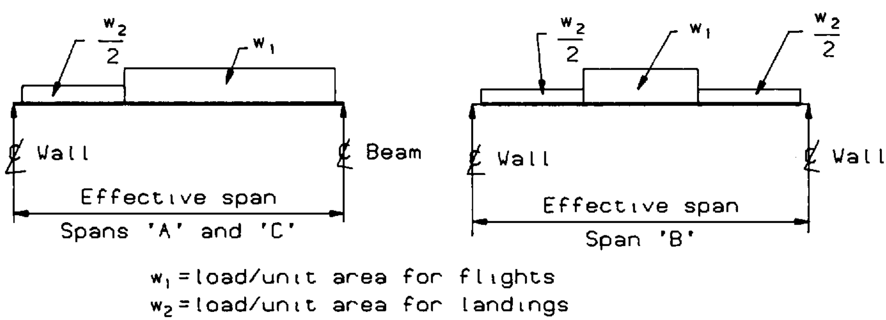
4.7 Staircases

4.7.1 Introduction

The reinforced concrete slab supporting the stair flights and landings should be designed generally in accordance with the design information in subsection 4.2, except as indicated otherwise in this subsection.



16 Stairs with open wells



17 Loading diagram

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.

4.7.2 Fire resistance, durability and concrete grades

The member sizes, reinforcement covers and concrete grades to provide fire resistance and durability should be obtained from Tables 7 and 8.

4.7.3 Bending moments and shear forces

Staircase slabs and landings should be designed to support the most unfavourable arrangements of design loads. Normally this requirement will be satisfied if staircase slabs and landings are designed to resist the moments and shear forces arising from the single-load case of maximum design ultimate load on all spans.

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.

4.7.4 Effective spans

4.7.4.1 Stairs spanning between beams or walls

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

4.7.4.2 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.

4.7.4.3 Stairs with open wells

The effective span and loads on each span are as indicated in Figs. 16 and 17. The arrangement of flight supports shown in Figs. 16 and 17 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 clauses 4.7.4.1 and 4.7.4.2.

4.7.5 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value from Table 36 multiplied by the modification factor in Table 37.

Table 36 Span/effective depth ratios for stairs

cantilever	7
simply supported	20
continuous	26

Table 37 Modification factors for M/bd^2 for stairs

Steel stress N/mm^2	M/bd^2 0.50	0.75	1.00	1.50	2.00	3.00
(f_y = 250) 156	2.00	2.00	1.96	1.66	1.47	1.24
(f_y = 460) 288	1.68	1.50	1.38	1.21	1.09	0.95

Notes to Tables 36 and 37
1. For spans in excess of 10m, the above ratios should be multiplied by 10/(span in metres).
2. M in Table 37 is the design ultimate moment at midspan or for a cantilever at the support.
3. Where the stair flight occupies at least 60% of the span the permissible span/depth ratio may be increased by 15%.

4.7.6 Section design

The design of the landing slabs and flights should be carried out in accordance with the methods described in clause 4.2.5.

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.

4.8 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 reference 9.

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 reference 9.

4.9 Guidance for the design of basement walls

4.9.1 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 [10]
2. make the walls at least 300mm thick and ensure that they comply with the slenderness provisions in clause 4.6.2.1

3. check that walls comply with the requirements for fire resistance in clause 4.6.2.2
4. check that walls comply with the requirements for durability in clause 4.6.2.3.

The face exposed to the earth must be considered to be in a moderate environment, unless the soil is aggressive, in which case BRE Digest no. 250[11] should be complied with.

4.9.2 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 2.6*. 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.

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

4.9.3 Section design

The sections should be designed in accordance with subsections 4.2 and 4.4 as appropriate. Where axial loading is significant, the provisions of subsections 4.5 and 4.6 should be followed as appropriate.

4.9.4 Foundation

The foundation or base slab should be designed as a strip footing 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.

4.9.5 Reinforcement

The minimum area of reinforcement in each direction of the wall should be 0.4% of the gross cross-sectional area.[10] The spacing of reinforcement should not exceed 300mm. Diagonal reinforcement should be provided across the corners of any openings in the wall.

4.10 Foundations

4.10.1 Introduction

The design of the foundations is usually the final step. The type of foundation, the sizes and the provisional formation levels depend on the results of the ground investigation.

Until partial factors for bearing pressures and pile resistances are codified it will be necessary to use the dead, imposed and wind loads on their own (i.e. without multiplying them by the partial safety factors from Table 1) for the proportioning of the foundations. The factored loads are, however, required for determining the depths of foundations and for the design of any reinforcement.

---

*For pressures arising from an accidental head of water at ground level a partial safety factor of 1.2 may be used.

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 safety factors in Table 1 and the imposed loading reduction in BS 6399[4] where appropriate**
4. recalculate the loads and moments, if any, on the individual foundations without the partial safety factors in Table 1, using the imposed loading reduction in BS 6399[4] where appropriate; in many cases it may be sufficiently accurate to divide the factored loads and moments calculated in step 3 by 1.45
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 and the reinforcement, if any, using the factored loads.

4.10.2 Durability and cover

All foundations other than those in aggressive soil conditions are considered as being in moderate environments (for definitions see Appendix C). Cover to *all* reinforcement should be 50mm. For reinforced foundations the minimum cement content should be 300kg/m^3 and the maximum water/cement ratio 0.60.

The characteristic strength of the concrete for reinforced bases and pile caps should therefore be not less than 35N/mm^2. For unreinforced bases f_{cu} = 20N/mm^2 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 ground water, the recommendations of BRE Digest no. 250[11] should be followed.

4.10.3 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.

4.10.4 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 foundations at one level throughout.

4.10.5 Design of spread footings

4.10.5.1 Axially loaded unreinforced pad footings

For concrete with f_{cu} = 20N/mm^2 the ratio of the depth h to the projection from the column face a should be not less than that given in Table 38 for different values of **unfactored** pressures, q, in kN/m^2.

Table 38 Depth/projection ratios for unreinforced footings

Unfactored ground pressure q, kN/m^2	h/a
≤200	1.0
300	1.2
400	1.4

For other concrete strengths

$$\frac{h}{a} > 0.15 \left(\frac{q^2}{f_{cu}}\right)^{1/4}$$

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

4.10.5.2 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 overall depth h to the projection from the column face a, given in Table 39 for different values of **unfactored** ground pressures q.
   The effective depth h should not in any case be less than 300mm.
2. Check that the face shear
   $$v = \frac{1000N}{2(c_x + c_y)d}$$
   does not exceed $v_c = 0.8\sqrt{f_{cu}}$ or 5N/mm^2, 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 does exceed v_c increase the depth.
3. With the chosen depth (revised according to stage 2, if necessary) enter Table 39 and obtain the corresponding reinforcement percentage.

Table 39 Reinforcement percentages, depth/projection ratios and ground pressures for reinforced footings for f_y = 460N/mm^2

q kN/m^2	d/a 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				0.25	0.23	0.20	0.17	0.13	0.13	0.13
250					0.28	0.25	0.22	0.15	0.13	0.13
300						0.30	0.26	0.17	0.13	0.13
400							0.35	0.23	0.17	0.13

The stippled areas indicate combinations of q and d/a that should not be used.

The above percentages apply to reinforcement with f_y = 460N/mm^2. For f_y = 250N/mm^2 multiply the above reinforcement percentages by 1.85.

4.10.5.3 Eccentrically loaded footings

The design of eccentrically loaded footings proceeds as follows:

1. determine initial depth of footing from Table 39 using maximum value of **unfactored** ground pressure
2. check punching shear according to clauses 4.2.3.4 and 4.2.5.2
3. check face shear according to stage 2 in clause 4.10.5.2 using V_{eff} from clause 4.2.3.4 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 39 to obtain the reinforcement percentage.

4.10.6 Design of other footings

4.10.6.1 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 clause 4.10.5.2.

4.10.6.2 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 clauses 4.2.3.4 and 4.2.5.2. 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 tied together by links and may be curtailed in accordance with the detailing rules in subsection 4.12.

Where a balanced footing consists of two pad footings joined by a beam, the beam may be designed in accordance with subsection 4.4.

Steps in the top or bottom surface may be introduced if necessary provided that they are taken into account in the design.

4.10.7 Reinforcement

Where reinforcement is required it should be provided in two generally orthogonal directions. The areas in each direction should not be less than 0.0013 bh for Grade 460 or 0.0025 bh for Grade 250 reinforcement, 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 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 Grade 460 nor 300mm for Grade 250. 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.

4.10.8 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. Such methods are outside the scope of this *Manual.*

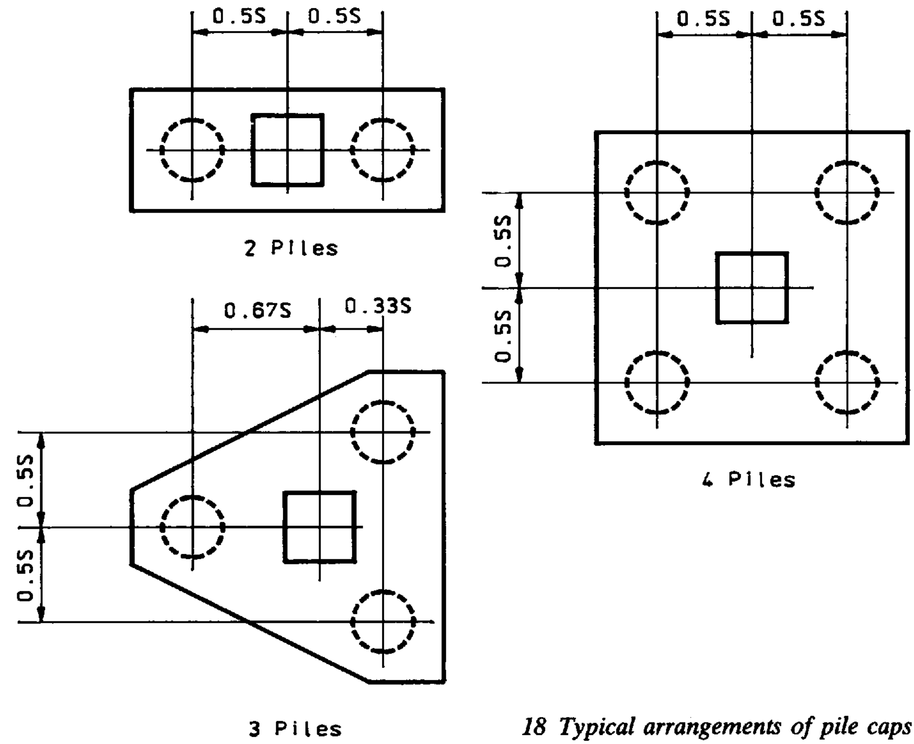
4.10.9 Design of pile caps

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. 18 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.



18 Typical arrangements of pile caps

4.10.10 Reinforcement in pile caps

All pile caps should generally be reinforced in two orthogonal directions on the top and bottom faces with not less than 0.0013 *bh* for Grade 460 or 0.0025 *bh* for Grade 250 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 size 12 at a vertical spacing not more than 250mm, should be provided.

4.11 Robustness

4.11.1 General

If the recommendations of this *Manual* have been followed, a robust structure will have been designed, subject to the reinforcement being properly detailed[2].

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 4.11.2.

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.

4.11.2 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 1.0 F_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 load does not exceed 7.5 kN/m^2, the ties in each direction should be capable of resisting a tie force of 1.0 F_t kN per metre width. If the spans exceed 5m and/or the total load exceeds 7.5 kN/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 1.0 F_t. For floor-to-ceiling heights greater than 2.5m, the tie forces should be increased *pro rata,* up to a maximum of 2.0 F_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 in accordance with the provisions of BS 8110, Part 2.[1]

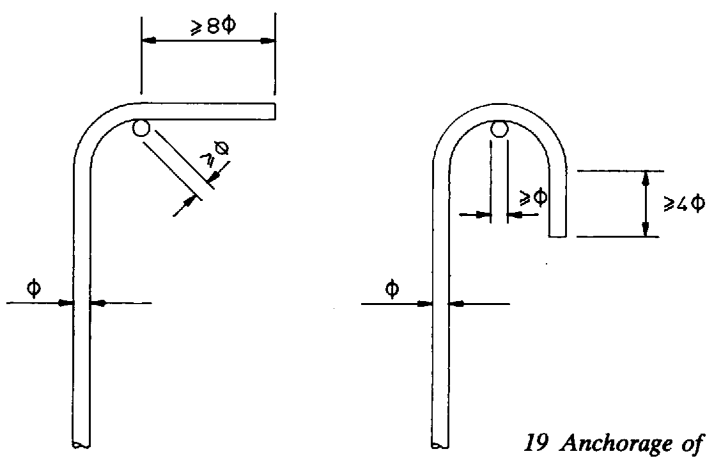
4.12 Detailing

4.12.1 General
Certain aspects of reinforcement detailing may influence the design. The most common of these are outlined below.

4.12.2 Bond and anchorage
Local bond stress may be ignored, provided that the force in the bar can be developed by the appropriate anchorage length (see Table 40).

A link may be considered fully anchored if it is detailed in accordance with Fig. 19.

4.12.3 Laps and splices
Laps and splices should generally be positioned away from zones of high stress and should preferably be staggered. When bars in tension are lapped the length should be at least equal to the design tension anchorage length necessary to develop the required



19 Anchorage of links

stress in the reinforcement. Lap lengths for unequal size bars (or wires in fabric) may be based on the smaller bar. The following provisions also apply:

(a) Where a lap occurs at the top of a section as cast and the minimum cover is less than twice the size of the lapped reinforcement, the lap length should be multiplied by a factor of 1.4

(b) Where a lap occurs at the corner of a section and the minimum cover to either face is less than twice the size of the lapped reinforcement or where the clear distance between adjacent laps is less than 75mm or six times the size of the lapped reinforcement, whichever is the greater, the lap length should be multiplied by a factor of 1.4

(c) In cases where both conditions *(a)* and *(b)* apply, the lap length should be multiplied by a factor of 2.0.

Where bars in compression are lapped the length should be at least 25% greater than the compression anchorage length necessary to develop the required stress in the reinforcement. Lap lengths for unequal size bars (or wires in fabric) may be based on the smaller bar.

Values for lap length are given in Table 40 as multiples of bar size.

Table 40 Ultimate anchorage bond lengths and lap lengths as multiples of bar size

f_{cu}, N/mm^2	25			30			40 and over		
Reinforcement type	250	460*	Fabric†	250	460*	Fabric†	250	460*	Fabric†
Tension anchorage and lap lengths	39	41	31	36	37	29	31	32	25
1.4 × tension lap	55	57	44	50	52	40	43	45	35
2.0 × tension lap	78	81	62	71	74	57	62	64	49
Compression anchorage length	32	32	25	29	29	23	25	26	20
Compression lap length	39	40	31	36	37	29	31	32	25

Note: **These lengths have been calculated assuming the reinforcement is acting at its design strength ($0.87f_y$). If the reinforcement acts at a lower stress the length may be reduced proportionately. The minimum lap length for bar reinforcement should not be less than 15 times the bar size or 300mm, whichever is the greater, and for fabric reinforcement should not be less than 250mm.**

*deformed bars type 2

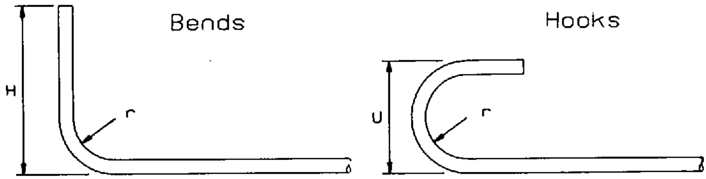
†welded fabric complying with BS4483.[12]

Mechanical splices may be used in lieu of laps in order to reduce congestion of reinforcement. For further information specialist literature should be consulted.

4.12.4 Hooks, bends and bearings

The minimum radii to which reinforcement may be bent may govern certain aspects of design, e.g. depths of bearings and choice of bar size for a given thickness of slab. Table 41[13] gives these dimensions, together with effective anchorage lengths for bends and hooks.

Table 41 Minimum radii, bend and hook sizes and effective anchorage lengths



Bar size	Grade 250 bars r mm	Bend H mm	Bend Effective anchorage length†	Hook U mm	Hook Effective anchorage length†	Grade 460 bars r mm	Bend H mm	Bend Effective anchorage length†	Hook U mm	Hook Effective anchorage length†
6*	12	165	15	40	16	18	170	15	50	24
8	16	170	15	50	16	24	175	15	70	24
10	20	170	13	60	16	30	180	15	85	24
12	24	175	11	75	16	36	185	14	100	24
16	32	185	9	100	16	48	195	12	135	24
20	40	190	8	120	16	60	215	12	165	24
25	50	230	8	150	16	100	310	12	260	24
32	64	275	8	195	16	128	380	12	330	24
40	80	335	8	240	16	160	455	12	410	24

Notes to Table 41
1. *Bar size may not be freely available.
2. Values of *H* are the theoretical maximum values and allow for a 50mm positive cutting and bending tolerance.
3. Values of *U* do not include an allowance for 'springback' after bending. For Grade 460 bars an allowance for the actual circumscribing bar diameter has been included.
4. †Effective anchorage lengths are given as multiples of bar size.

Where a bar is fully stressed through the length of a bend, greater bending radii may be required to limit the compressive stress on the inside of the bend.

4.12.5 Curtailment of reinforcement

In every flexural member except at end supports every bar should extend beyond the point at which it is no longer needed, for a distance at least equal to the greater of:

(a) the effective depth of the member or
(b) twelve times the bar size

and in addition, for a bar in the tension zone, one of the following distances for all arrangements of design ultimate load:

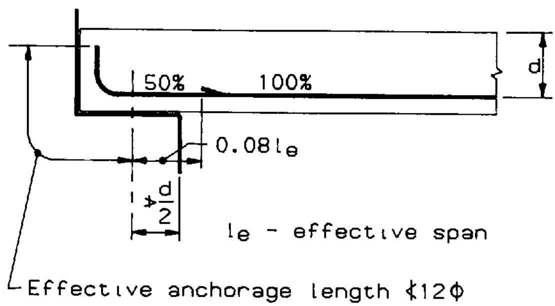
(c) an anchorage length appropriate to its design strength ($0.87f_y$) from the point at which it is no longer required to assist in resisting the bending moment
(d) to the point where the design shear capacity of the section is greater than twice the design shear force at that section or
(e) to the point where other bars continuing past that point provide double the area required to resist the design bending moment at that section.

The point at which a bar is no longer required is the point where the design resistance moment of the section, considering only the continuing bars, is equal to the design moment.

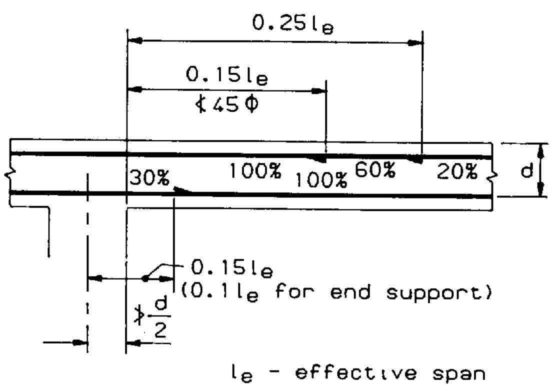
As curtailment of substantial areas of reinforcement at a single section can lead to the development of large cracks at that point, it is essential to stagger the curtailment points.

Alternatively, bars may be curtailed as shown in Figs. 20 to 24 for cases where:

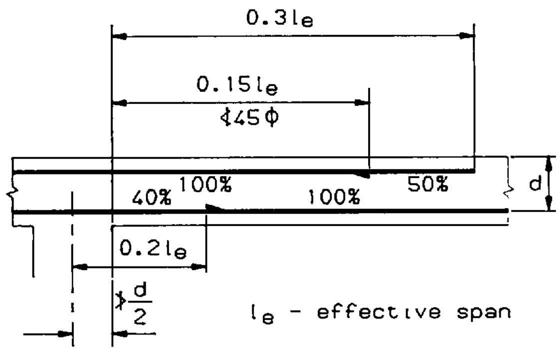
(i) the loading is predominantly uniformly distributed and
(ii) for continuous beams and slabs the spans do not differ by more than 15% of the longest span.



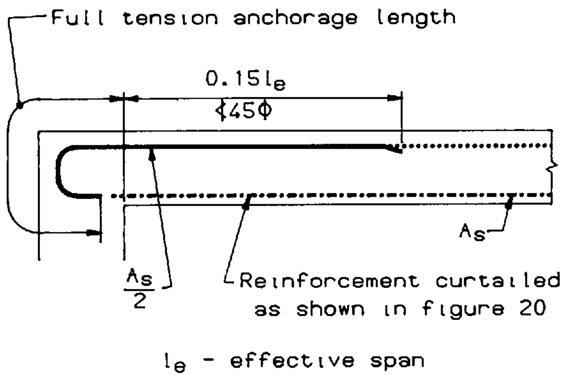
20 Simply supported beams and slabs



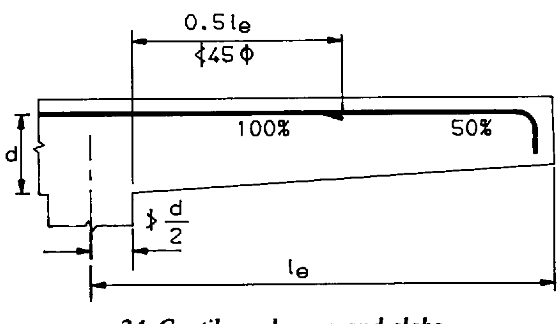
21 Continuous beams



22 *Continuous slab*



23 *Beams and slabs monolithic with support beam or wall (designed as simply supported)*



24 *Cantilever beams and slabs*

4.12.6 Corbels and nibs

These should be designed and detailed in accordance with the appropriate clauses in the precast concrete section of BS 8110.[1] Care should be taken to assess adequately the horizontal forces arising from restrained temperature and moisture movements as these will often govern the design.

References

1. BS 8110: *Structural use of concrete,* Part 1: *Code of practice for design and construction,* Part 2: *Code of practice for special circumstances,* British Standards Institution, London 1985
2. *Standard method of detailing structural concrete,* Joint report of the Institution of Structural Engineers and the Concrete Society, The Institution of Structural Engineers, London August 1985 (draft for discussion)
3. BS 648: *Schedule of weights of building materials,* British Standards Institution, London 1964
4. BS 6399: *Design loadings for buildings,* Part 1: *Code of practice for dead and imposed loads,* British Standards Institution, London 1984
5. CP 3: Chapter V: *Loading,* Part 2: *Wind loads,* British Standards Institution, London 1972
6. CP 2004: *Foundations,* British Standards Institution, London 1972
7. *SMM 6 Standard method of measurement of building works,* 6th edit., RICS and NFBTE, London 1979
8. Beeby, A. W.: *The analysis of beams in plane frames according to CP 110,* Development report no. 1, C & CA, Slough 1978
9. Deacon, R. C.: *Concrete ground floors – their design construction and finish,* 2nd edit., C & CA, Slough 1975
10. Coffin, F. G., Beckmann, P., and Pearce, T.: *Guide to the design of waterproof basements,* CIRIA, London 1978
11. *Concrete in sulphate-bearing soils and ground water,* BRE Digest 250, HMSO, London 1981
12. BS 4483: *Steel fabric for the reinforcement of concrete,* British Standards Institution, London 1969
13. BS 4466: *Specification for bending dimensions and scheduling of reinforcement for concrete,* British Standards Institution, London 1981

Appendix A Reinforcement quantities

This Appendix contains Tables A1, A2, A3, A4 and A5 referred to in method 2 of subsection 3.9.

The factors for converting reinforcement areas into unit weights of reinforcement assume that:

(a) 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.
(b) the detailing is in accordance with reference 2.

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/m^2	Remarks
One-way spanning slabs	$\dfrac{M}{(0.8d)(0.87f_y)}$	Minimum steel or 0.25 A_{sx}	0.0125 A'_{sx}*	M is the maximum bending moment per metre width anywhere in the slab.
Two-way spanning slabs with linear supports	$\dfrac{M_x}{(0.8d)(0.87f_y)}$	$\dfrac{M_y}{0.8(d-20)\ 0.87f_y}$	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	$\dfrac{M_x}{(0.8d)(0.87f_y)}$	$\dfrac{M_y}{0.8\ (d\ -20)\ 0.87f_y}$	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} and A'_{sy} are areas of reinforcement (in mm^2) 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

Minimum reinforcement

Ribs high yield steel – 0.25% $b_w h$

mild steel – 0.50% $b_w h$

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	$\dfrac{M}{0.87 f_y (d - 0.5 h_f)}$	$\dfrac{0.009\, A'_s}{c}$	For fabric reinforcement: 1.25 × 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	$\dfrac{M}{0.87 f_y (d - 10 - 0.5 h_f)}$	$\dfrac{0.02\, A'_s}{c}$	As for one-way spanning slabs	M is the maximum bending moment per rib in the two directions
Coffered slabs on column supports	$\dfrac{M_x}{0.87 f_y (d - 0.5 h_f)}$ and $\dfrac{M_y}{0.87 f_y (d - 20 - 0.5 h_f)}$	$\dfrac{0.013\, (A'_{sx} + A'_{sy})}{c}$	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 – 0.25% b_wh
mild steel – 0.50% b_wh

where b_w is the width of beam and h is the overall depth of beam

Links: mild steel: 0.25% of a horizontal section through the web
high yield steel: 0.12% of horizontal section through web

	A_s required	Weight kg/m	Remarks
Longitudinal steel	At midspan for T- and L-beams (and at supports for upstand beams) $\dfrac{M}{0.87f_y\,(d-0.5h_f)}$ For rectangular beams and at supports for T- and L-beams (and at midspan for upstand-beams) $\dfrac{M}{(0.87f_y)(0.75d)}$	$0.011\,A'_s$	A'_s is the area (in mm^2) of main reinforcement selected at midspan or supports whichever is greater. M is the design ultimate bending moment
Links	Shear stress $v = \dfrac{\text{design ultimate shear force}}{b_w d}$ If $v>0.6N/mm^2$ $\dfrac{A_{sv}}{S_v} = \dfrac{b_w(v-0.6)}{0.87f_y}$ If $v \leq 0.6N/mm^2$ choose A'_{sv} and S_v' to satisfy minimum steel	Single links (i.e. two legs) $0.016\,(B_w + H)\,\dfrac{A'_{sv}}{S'_v}$ Double links (i.e. four legs) $0.016\,(1.5\,B_w + 2H)\,\dfrac{A'_{sv}}{S_v'}$ Treble links (i.e. six legs) $0.016\,(2B_w+3H)\,\dfrac{A'_{sv}}{S'_v}$	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 mm^2. 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:

- size at least one-quarter of the biggest longitudinal bar
- spacing: 12 × size 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	$0.011\ A_s$	A_s area of all vertical bars (mm^2)
Links	peripheral links	b and h are dimensions of column cross-section in metres
	$0.016\ (b + h)\ \dfrac{A}{S_v}$	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	For sausage links (shape code 81) b is the dimension parallel to the link
	$0.016b\ \dfrac{A}{S_v}$	

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 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

Contract **Job no.**

General description, intended use, unusual environmental conditions

Site constraints

Stability provisions

Movement joints

Loading

Fire resistance

Durability

Soil conditions and foundation design

Performance criteria

Materials

Ground slab construction

Other data

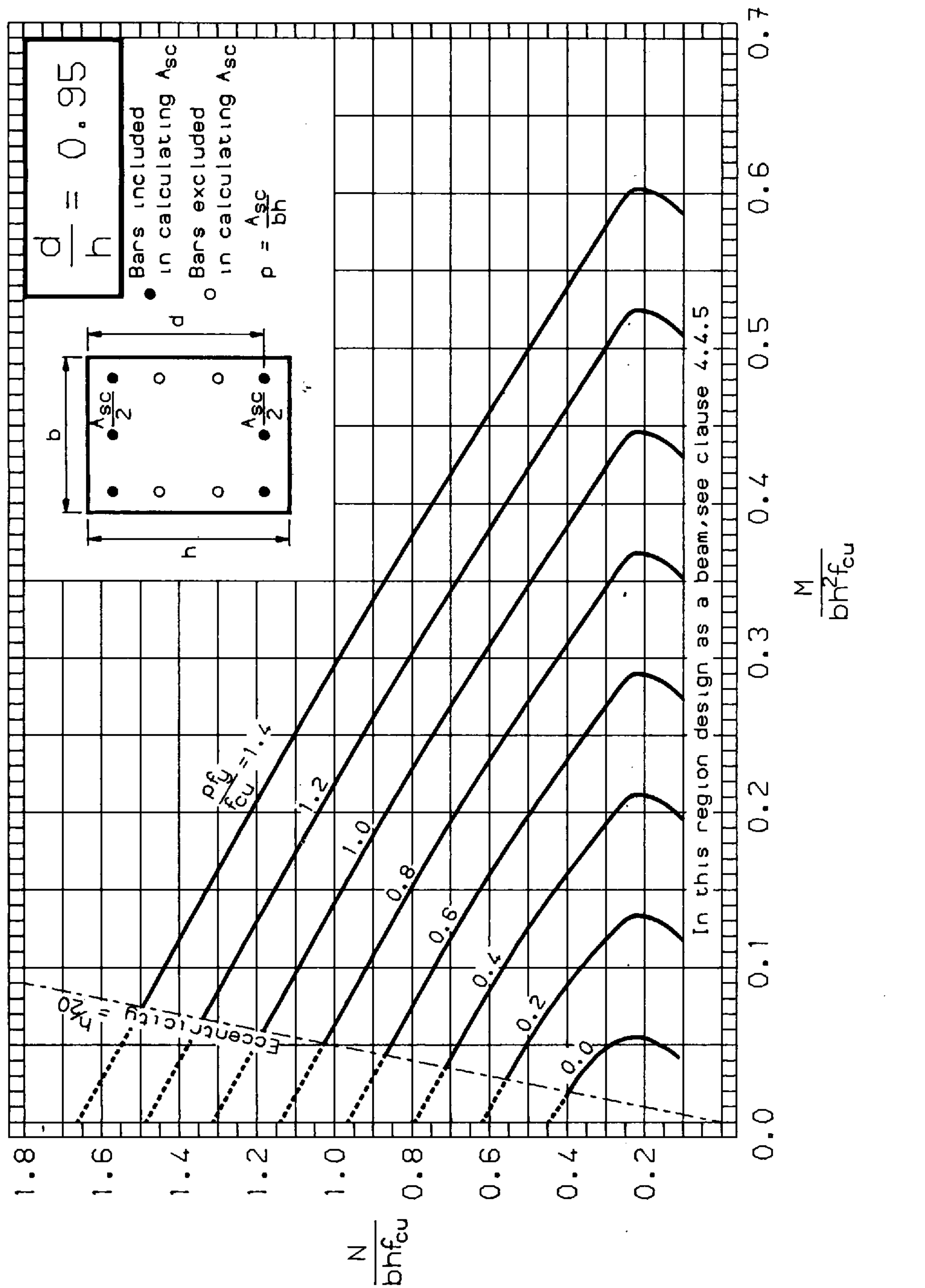
Appendix C Exposure conditions

The exposure conditions in service to be considered when determining the covers and grades of concrete to be used for all members are as follows:

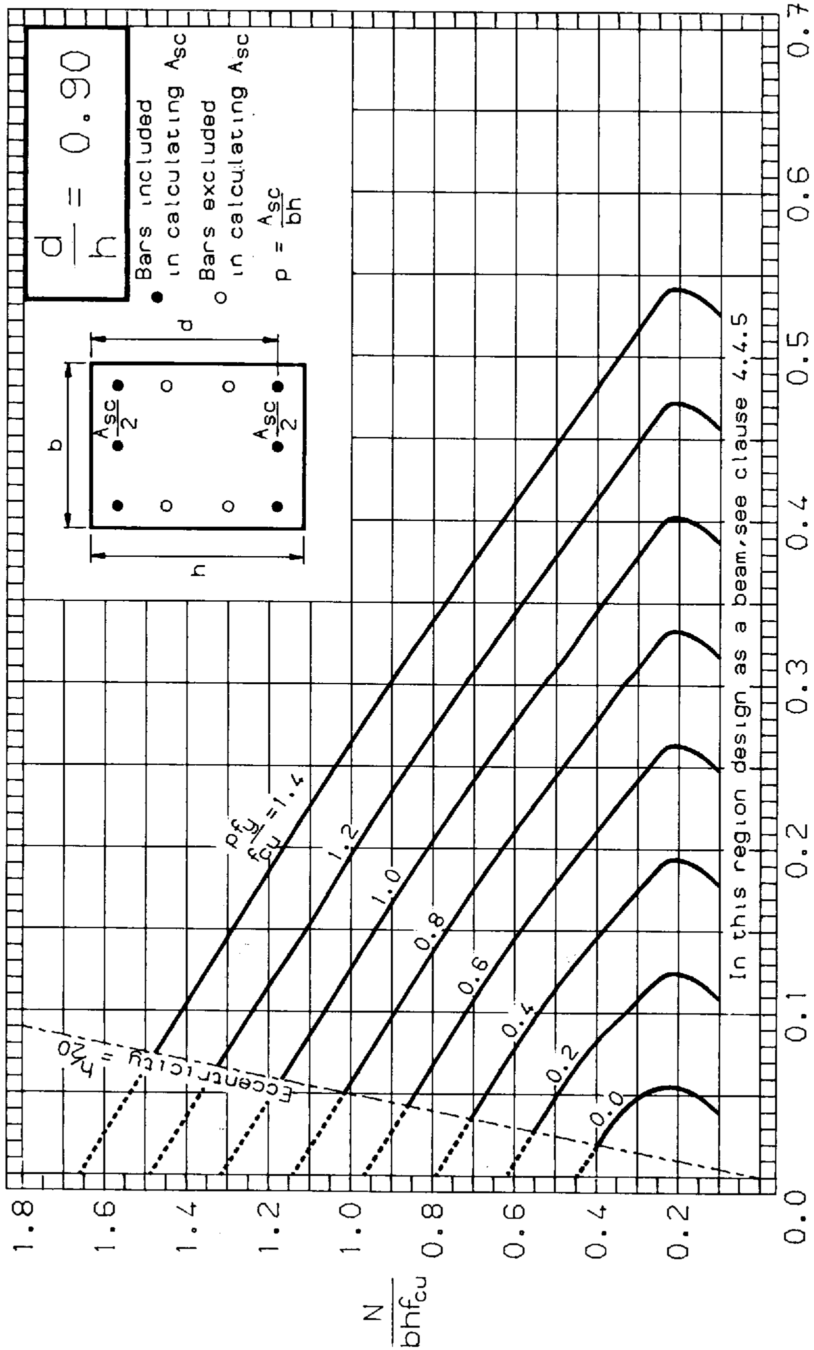
Environment	Definition
mild	concrete surfaces protected against weather or aggressive conditions
moderate	concrete surfaces sheltered from severe rain or freezing while wet concrete subject to condensation concrete surfaces continuously under water concrete in contact with non-aggressive soil
severe	concrete surfaces exposed to: severe rain; alternate wetting and drying or occasional freezing; or severe condensation
very severe	concrete surfaces exposed to: sea water spray de-icing salts, directly or indirectly corrosive fumes or severe freezing conditions while wet
extreme	concrete surfaces exposed to abrasive action by: sea water carrying solids flowing water with pH $\leqslant 4.5$ or machinery or vehicles

Appendix D Column design charts

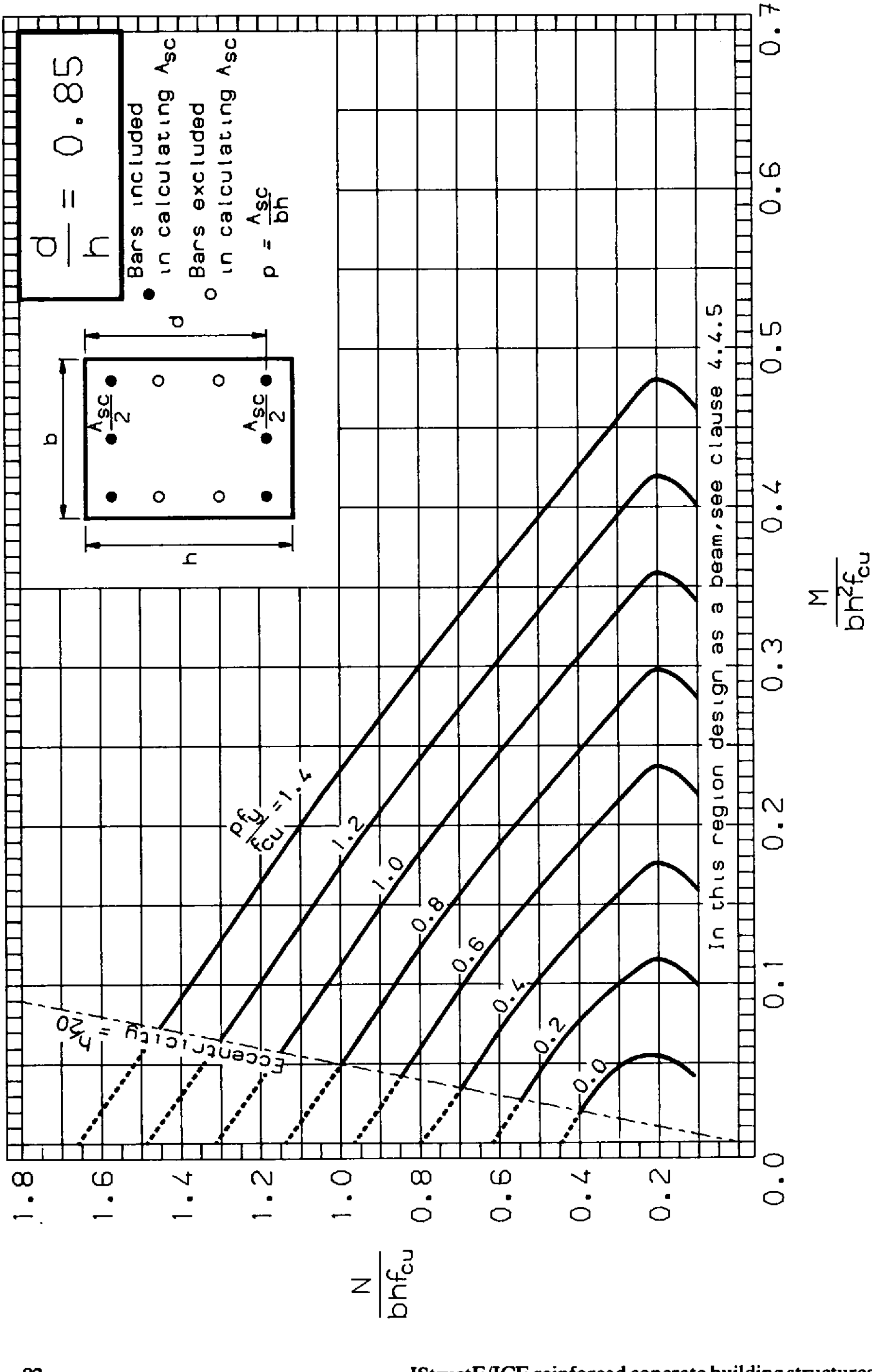
This Appendix contains design charts for stocky symmetrically reinforced rectangular and circular columns. The charts for rectangular columns are drawn for d/h values of 0.95, 0.90, 0.85, 0.80 and 0.75, while those for circular columns are for h_s/h values of 0.9, 0.8, 0.7 and 0.6.



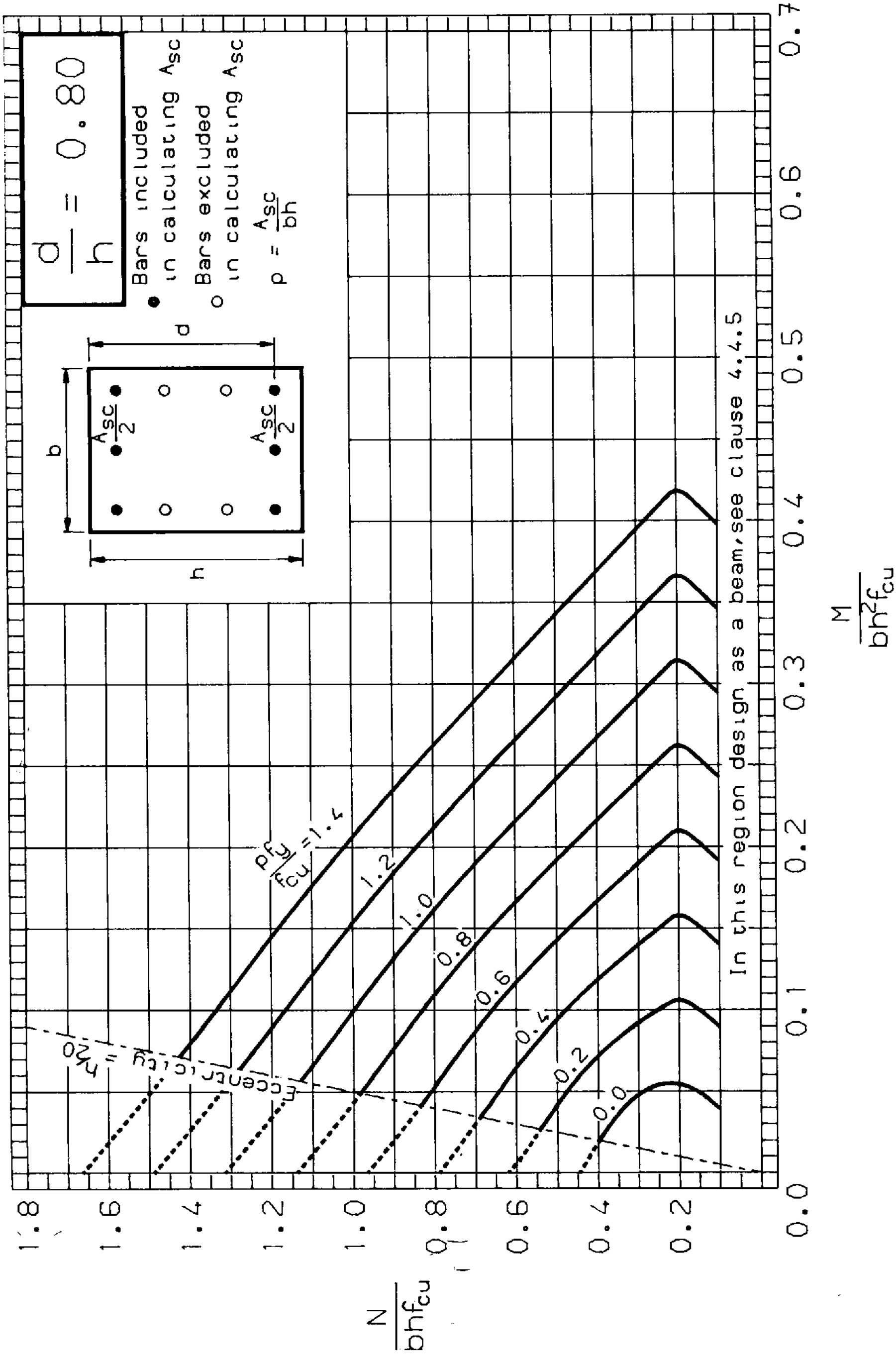
d/h = 0.95
Bars included in calculating Asc
Bars excluded in calculating Asc
p = Asc/bh
b
h
d
Asc/2
Asc/2
In this region design as a beam, see clause 4.4.5
Eccentricity = h/20
pfy/fcu = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
N/bhfcu
M/bh²fcu
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8



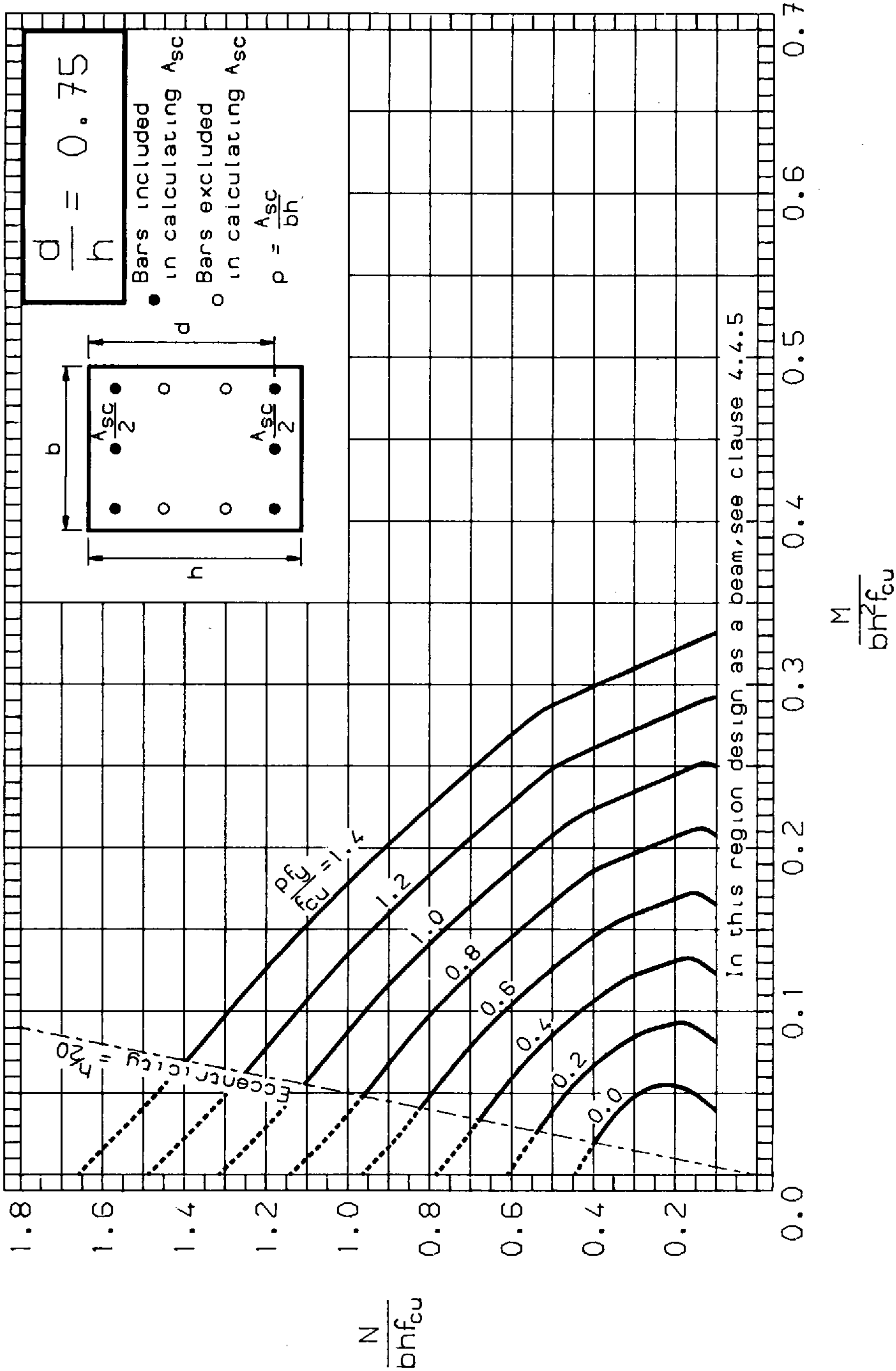
d/h = 0.90
● Bars included in calculating Asc
○ Bars excluded in calculating Asc
p = Asc/bh
Asc/2
Asc/2
b
d
h
pfy/fcu = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity = h/20
In this region design as a beam, see clause 4.4.5
N/bhfcu
M/bh2fcu
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8

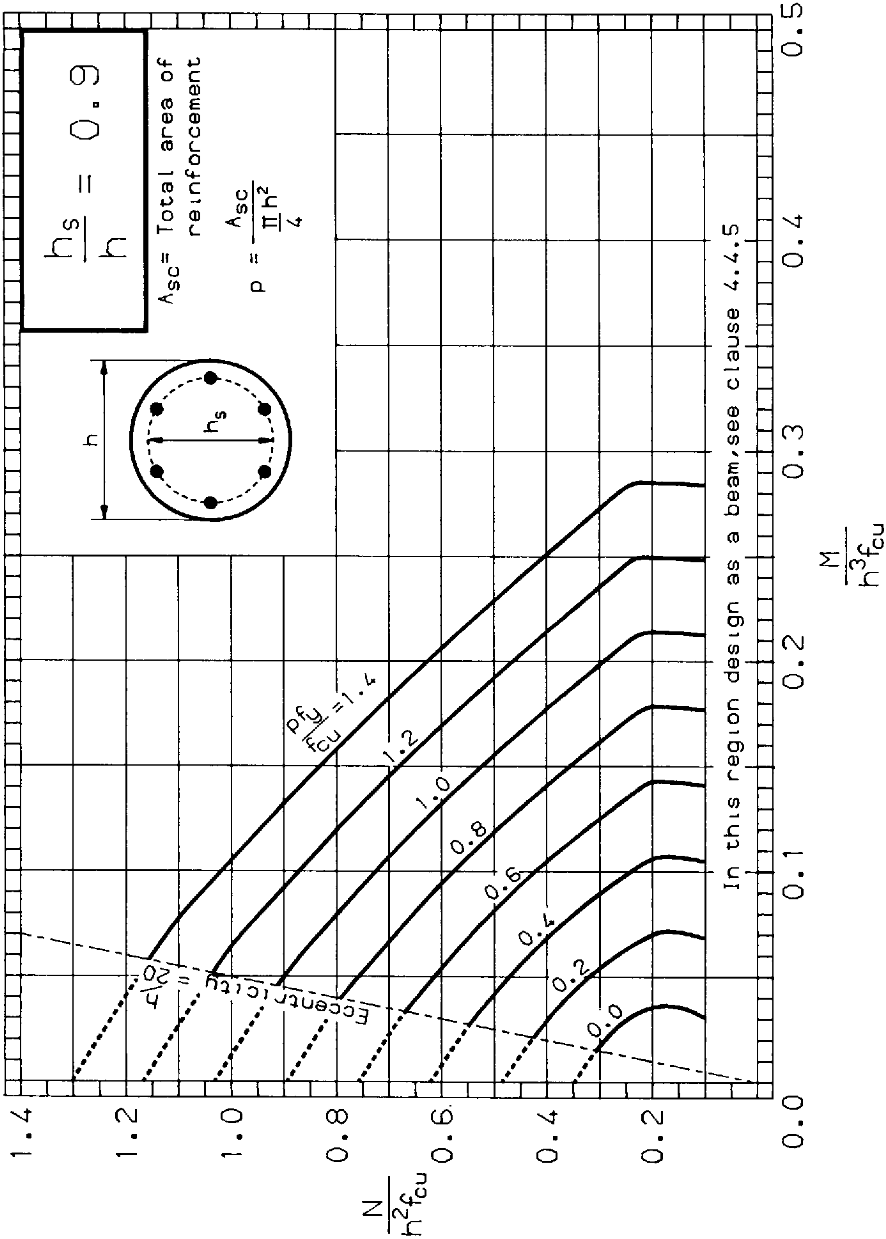


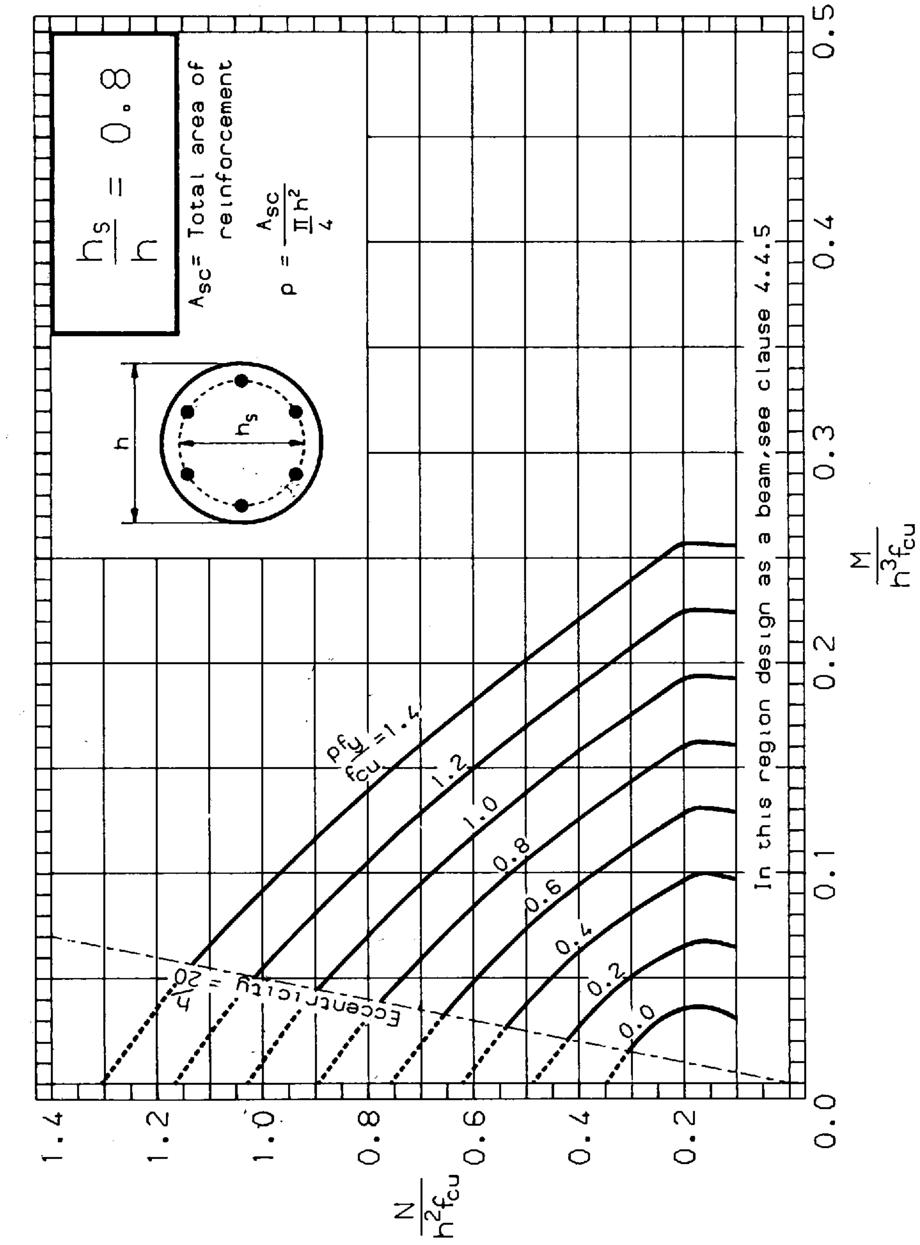
d/h = 0.85
Bars included in calculating Asc
Bars excluded in calculating Asc
p = Asc/bh
Asc/2
b
d
h
pfy/fcu = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity = h/20
In this region design as a beam, see clause 4.4.5
N/bhfcu
M/bh²fcu
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8



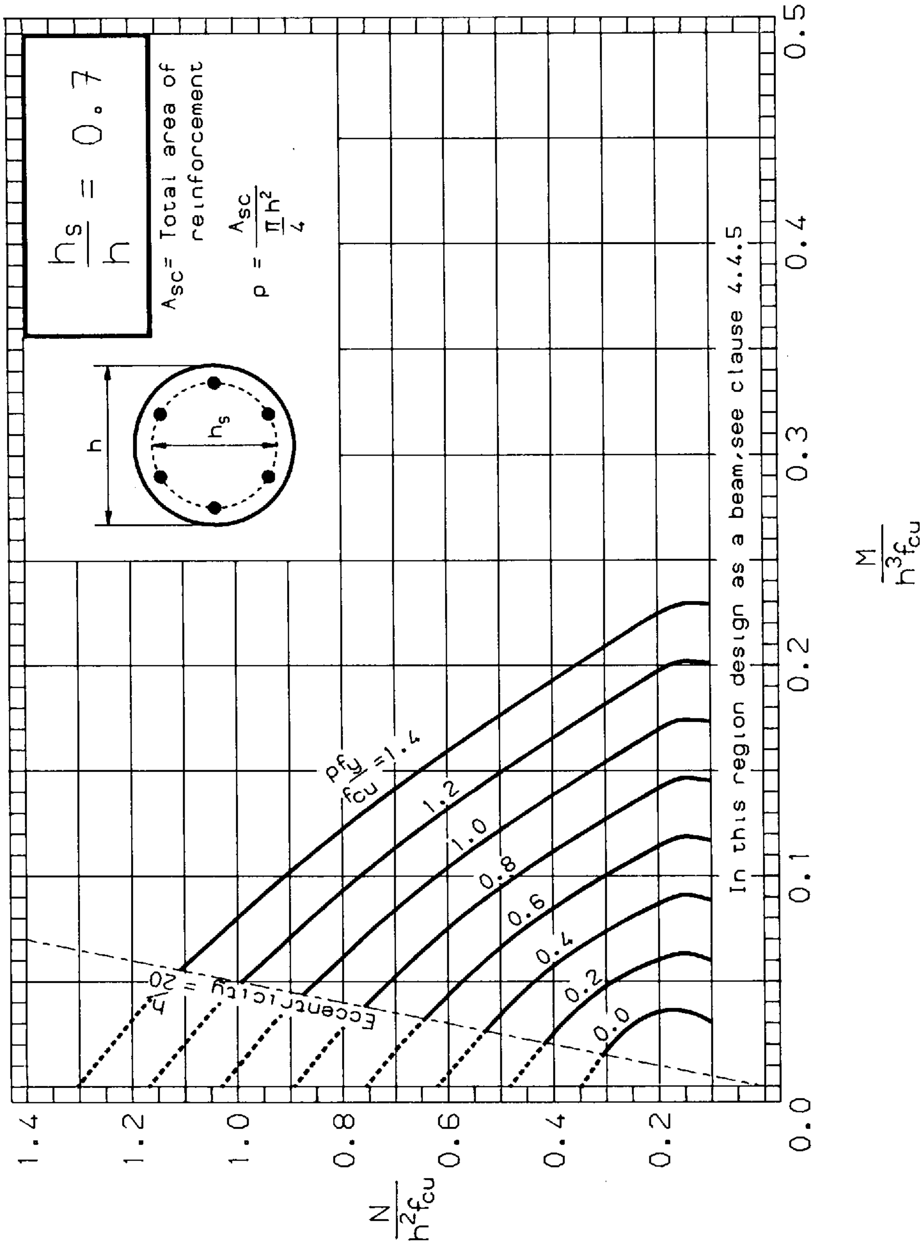
d/h = 0.80
Bars included in calculating Asc
Bars excluded in calculating Asc
p = Asc/bh
Asc/2
b
h
d
In this region design as a beam, see clause 4.4.5
Eccentricity = h/20
pfy/fcu = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
M/bh²fcu
N/bhfcu
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8


d/h = 0.75
● Bars included in calculating Asc
○ Bars excluded in calculating Asc
p = Asc/bh
b
h
d
Asc/2
Asc/2
pfy/fcu = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity = h/20
In this region design as a beam, see clause 4.4.5
M/bh²fcu
N/bhfcu
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8

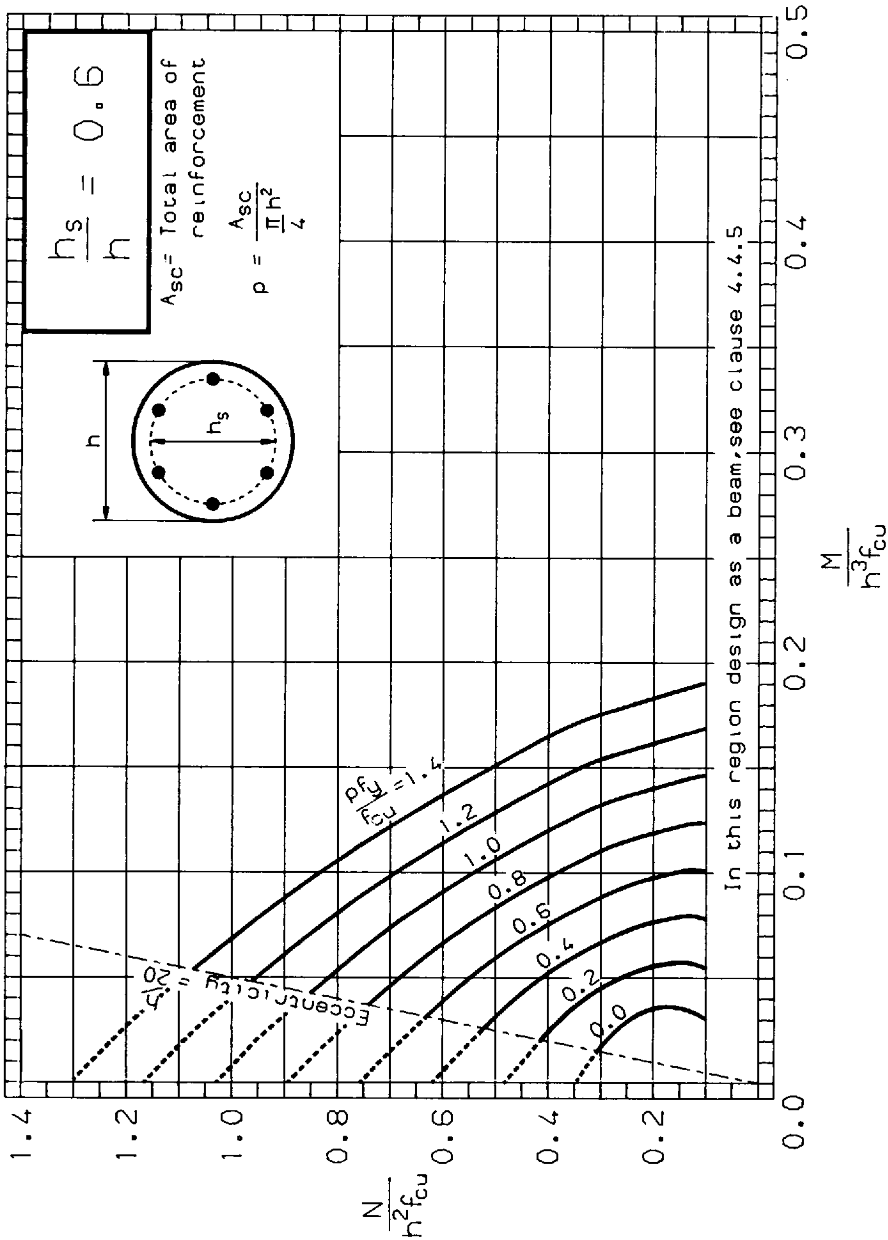

$\frac{h_s}{h} = 0.9$
A_{sc} = Total area of reinforcement
$p = \frac{A_{sc}}{\frac{\pi h^2}{4}}$
h
h_s
$\frac{p f_y}{f_{cu}} = 1.4$
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity = h/20
In this region design as a beam, see clause 4.4.5
$\frac{N}{h^2 f_{cu}}$
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.1
0.2
0.3
0.4
0.5
$\frac{M}{h^3 f_{cu}}$



$\frac{h_s}{h} = 0.8$
A_{sc} = Total area of reinforcement
$p = \frac{A_{sc}}{\frac{\pi h^2}{4}}$
h
h_s
$\frac{pf_y}{f_{cu}} = 1.4$
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity = h/20
In this region design as a beam, see clause 4.4.5
$\frac{N}{h^2 f_{cu}}$
$\frac{M}{h^3 f_{cu}}$
0.0
0.1
0.2
0.3
0.4
0.5
0.2
0.4
0.6
0.8
1.0
1.2
1.4



$\frac{h_s}{h} = 0.7$
A_{sc} = Total area of reinforcement
$p = \frac{A_{sc}}{\frac{\pi}{4}h^2}$
h
h_s
$\frac{pf_y}{f_{cu}} = 1.4$
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Eccentricity $\frac{e}{h} = 20$
In this region design as a beam, see clause 4.4.5
$\frac{N}{h^2 f_{cu}}$
$\frac{M}{h^3 f_{cu}}$
0.0
0.1
0.2
0.3
0.4
0.5
0.2
0.4
0.6
0.8
1.0
1.2
1.4



$\frac{h_s}{h} = 0.6$
A_{sc} = Total area of reinforcement
$p = \frac{A_{sc}}{\frac{\pi h^2}{4}}$
h
h_s
In this region design as a beam, see clause 4.4.5
Eccentricity = h/20
$\frac{pf_y}{f_{cu}}$ = 1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
$\frac{N}{h^2 f_{cu}}$
$\frac{M}{h^3 f_{cu}}$
0.0
0.1
0.2
0.3
0.4
0.5
0.2
0.4
0.6
0.8
1.0
1.2
1.4