



The Institution of Structural Engineers

# Manual for the design of concrete building structures to Eurocode 2

September 2006

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## Constitution

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**Notation**

<i>Latin upper case letters</i>			
$A$	Accidental action	$EI$	Bending stiffness
$A$	Cross sectional area	$EQU$	Static equilibrium
$A_c$	Cross sectional area of concrete	$F$	Action
$A_p$	Area of a prestressing tendon or tendons	$F_d$	Design value of an action
$A_s$	Cross sectional area of reinforcement	$F_k$	Characteristic value of an action
$A_{s,min}$	minimum cross sectional area of reinforcement	$G_k$	Characteristic permanent action
$A_{sw}$	Cross sectional area of shear reinforcement	$I$	Second moment of area of concrete section
$D$	Diameter of mandrel	$L$	Length
$D_{Ed}$	Fatigue damage factor	$M$	Bending moment
$E$	Effect of action	$M_{Ed}$	Design value of the applied internal bending moment
$E_c, E_{c(28)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at 28 days	$N$	Axial force
$E_{c,eff}$	Effective modulus of elasticity of concrete	$N_{Ed}$	Design value of the applied axial force (tension or compression)
$E_{cd}$	Design value of modulus of elasticity of concrete	$P$	Prestressing force
$E_{cm}$	Secant modulus of elasticity of concrete	$P_0$	Initial force at the active end of the tendon immediately after stressing
$E_c(t)$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at time $t$	$Q_k$	Characteristic variable action
$E_p$	Design value of modulus of elasticity of prestressing steel	$Q_{fat}$	Characteristic fatigue load
$E_s$	Design value of modulus of elasticity of reinforcing steel	$R$	Resistance
		$S$	Internal forces and moments
		$S$	First moment of area
		SLS	Serviceability limit state
		$T$	Torsional moment
		$T_{Ed}$	Design value of the applied torsional moment
		ULS	Ultimate limit state
		$V$	Shear force
		$V_{Ed}$	Design value of the applied shear force

<i>Latin lower case letters</i>		
$a$	Distance	$f_{0,2k}$
$a$	Geometrical data	$f_t$
$\Delta a$	Deviation for geometrical data	$f_{tk}$
$b$	Overall width of a cross-section, or actual flange width in a T or L beam	$f_y$
$b_w$	Width of the web on T, I or L beams	$f_{yd}$
$d$	Diameter; Depth	$f_{yk}$
$d$	Effective depth of a cross-section	
$d_g$	Largest nominal maximum aggregate size	$f_{ywd}$
$e$	Eccentricity	$h$
$f_c$	Compressive strength of concrete	$h$
$f_{cd}$	Design value of concrete compressive strength	$i$
$f_{ck}$	Characteristic compressive cylinder strength of concrete at 28 days	$k$
$f_{cm}$	Mean value of concrete cylinder compressive strength	$l$
$f_{ctk}$	Characteristic axial tensile strength of concrete	$l_0$
$f_{ctm}$	Mean value of axial tensile strength of concrete	$m$
$f_{cu}$	Characteristic compressive cube strength of concrete at 28 days	$r$
$f_p$	Tensile strength of prestressing steel	$1/r$
$f_{pk}$	Characteristic tensile strength of prestressing steel	$t$
$f_{p0,1}$	0,1% proof-stress of prestressing steel	$t_0$
$f_{p0,1k}$	Characteristic 0,1% proof-stress of prestressing steel	$u$
		$x$
		$x,y,z$
		$z$
		Characteristic 0,2% proof-stress of reinforcement
		Tensile strength of reinforcement
		Characteristic tensile strength of reinforcement
		Yield strength of reinforcement
		Design yield strength of reinforcement
		Characteristic yield strength of reinforcement
		Design yield of shear reinforcement
		Height
		Overall depth of a cross-section
		Radius of gyration
		Coefficient; Factor
		(or L) Length; Span
		effective length or lap length
		Mass
		Radius
		Curvature at a particular section
		Thickness
		Time being considered
		The age of concrete at the time of loading
		Perimeter of concrete cross-section, having area $A_c$
		Components of the displacement of a point
		Neutral axis depth
		Coordinates
		Lever arm of internal forces

<i>Greek lower case letters</i>	
$\alpha$	Angle ; ratio
$\beta$	Angle ; ratio; coefficient
$\gamma$	Partial factor
$\gamma_A$	Partial factor for accidental actions $A$
$\gamma_C$	Partial factor for concrete
$\gamma_F$	Partial factor for actions, $F$
$\gamma_{F,fat}$	Partial factor for fatigue actions
$\gamma_{C,fat}$	Partial factor for fatigue of concrete
$\gamma_G$	Partial factor for permanent actions, $G$
$\gamma_M$	Partial factor for a material property, taking account of uncertainties in the material property itself, in geometric deviation and in the design model used
$\gamma_P$	Partial factor for actions associated with prestressing, $P$
$\gamma_Q$	Partial factor for variable actions, $Q$
$\gamma_S$	Partial factor for reinforcing or prestressing steel
$\gamma_{S,fat}$	Partial factor for reinforcing or prestressing steel under fatigue loading
$\gamma_f$	Partial factor for actions without taking account of model uncertainties
$\gamma_g$	Partial factor for permanent actions without taking account of model uncertainties
$\gamma_m$	Partial factors for a material property, taking account only of uncertainties in the material property
$\delta$	Increment/redistribution ratio
$\zeta$	Reduction factor/distribution coefficient
$\varepsilon_c$	Compressive strain in the concrete
$\varepsilon_{c1}$	Compressive strain in the concrete at the peak stress $f_c$
$\varepsilon_{cu}$	Ultimate compressive strain in the concrete
$\varepsilon_u$	Strain of reinforcement or prestressing steel at maximum load
$\varepsilon_{uk}$	Characteristic strain of reinforcement or prestressing steel at maximum load
$\theta$	Angle
$\lambda$	Slenderness ratio
$\mu$	Coefficient of friction between the tendons and their ducts
$\nu$	Poisson's ratio
$\nu$	Strength reduction factor for concrete cracked in shear
$\xi$	Ratio of bond strength of prestressing and reinforcing steel
$\rho$	Oven-dry density of concrete in kg/m <sup>3</sup>
$\rho_{1000}$	Value of relaxation loss (in %), at 1000 hours after tensioning and at a mean temperature of 20°C
$\rho_l$	Reinforcement ratio for longitudinal reinforcement
$\rho_w$	Reinforcement ratio for shear reinforcement
$\sigma_c$	Compressive stress in the concrete
$\sigma_{cp}$	Compressive stress in the concrete from axial load or prestressing
$\sigma_{cu}$	Compressive stress in the concrete at the ultimate compressive strain $\varepsilon_{cu}$
$\tau$	Torsional shear stress
$\phi$	Diameter of a reinforcing bar or of a prestressing duct
$\phi_n$	Equivalent diameter of a bundle of reinforcing bars
$\varphi(t,t_0)$	Creep coefficient, defining creep between times $t$ and $t_0$ , related to elastic deformation at 28 days
$\varphi(\infty,t_0)$	Final value of creep coefficient
$\psi$	Factors defining representative values of variable actions:
	$\psi_0$ for combination values
	$\psi_1$ for frequent values
	$\psi_2$ for quasi-permanent values

## 1 Introduction

### 1.1 Aims of the Manual

This *Manual* provides guidance on the design of reinforced and prestressed concrete building structures. Structures designed in accordance with this *Manual* will normally comply with BS EN 1992-1-1: 2004<sup>1</sup> and BS EN 1992-1-2: 2004<sup>2</sup>.

It is primarily related to those carrying out hand calculations and not necessarily relevant to computer analysis. However it is good practice that such hand analysis methods are used to verify the output of more sophisticated methods.

### 1.2 Eurocode system

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

CEN is publishing the design standards as full European Standards EN (Euronorms):

BS EN 1990: Eurocode: Basis of design (EC0)

BS EN 1991: Eurocode 1: Actions on structures (EC1)

Part 1-1: General actions – Densities, self-weight and imposed loads

Part 1-2: General actions on structures exposed to fire

Part 1-3: General actions – Snow loads

Part 1-4: General actions – Wind loads

Part 1-6: Actions during execution

Part 1-7: Accidental actions from impact and explosions

Part 2: Traffic loads on bridges

Part 3: Actions induced by cranes and machinery

Part 4: Actions in silos and tanks

BS EN 1992: Eurocode 2: Design of concrete structures (EC2)

Part 1-1: General rules and rules for buildings (EC2 Part 1-1)

Part 1-2: General rules - Structural fire design (EC2 Part 1-2)

Part 2: Reinforced and prestressed concrete bridges (EC2 Part 2)

Part 3: Liquid retaining and containing structures (EC2 Part 3)

BS EN 1993: Eurocode 3: Design of steel structures (EC3)

BS EN 1994: Eurocode 4: Design of composite steel and concrete structures (EC4)

BS EN 1995: Eurocode 5: Design of timber structures (EC5)

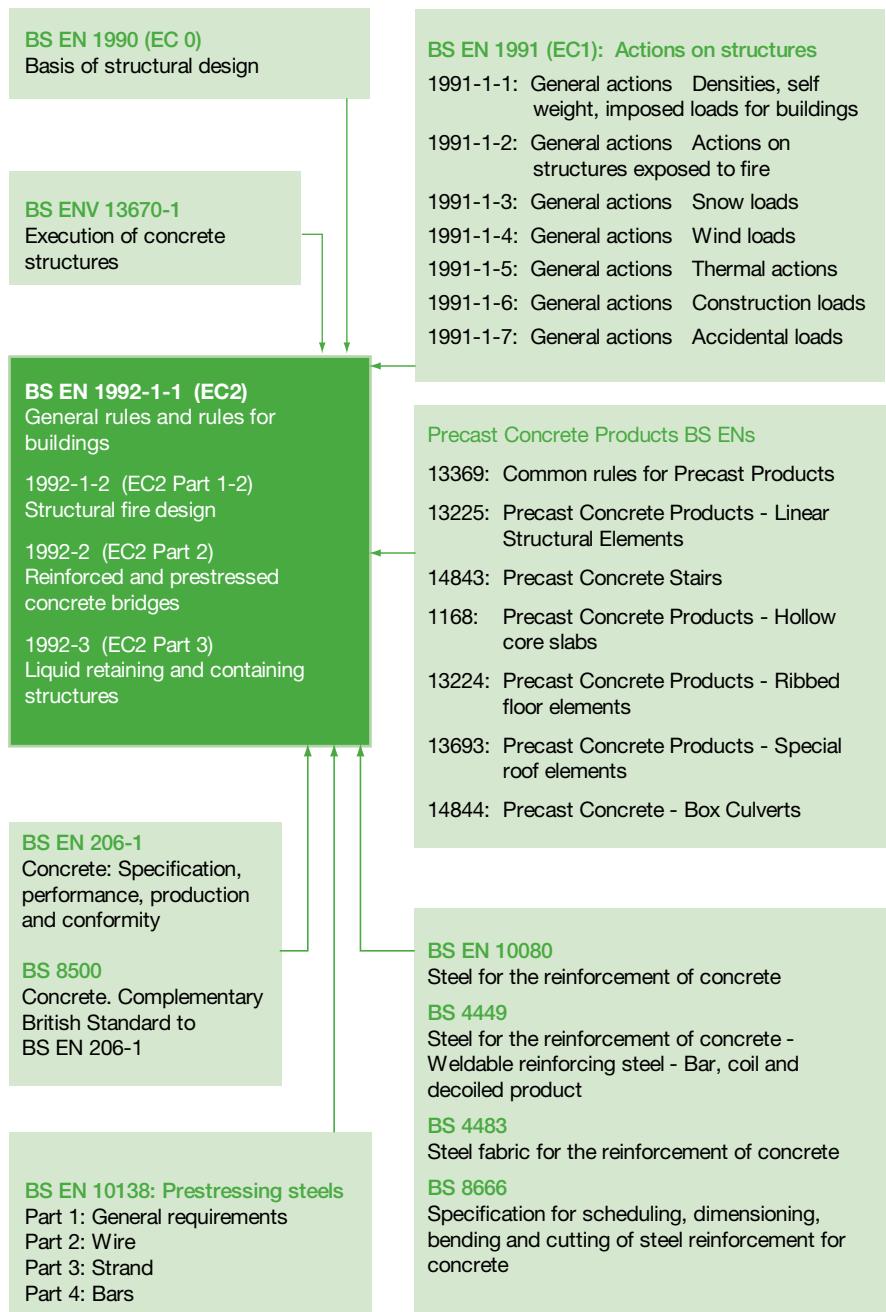
BS EN 1996: Eurocode 6: Design of masonry structures (EC6)

BS EN 1997: Eurocode 7: Geotechnical design (EC7)

BS EN 1998: Eurocode 8: Earthquake resistant design of structures (EC8)

BS EN 1999: Eurocode 9: Design of aluminium alloy structures (EC9)

The European and British Standards relating to EC2 are shown in Figure 1.1.



**Fig 1** Flow chart of standards for Eurocode 2

All Eurocodes follow a common editorial style. The codes contain ‘Principles’ and ‘Application rules’. Principles are identified by the letter P following the paragraph number. Principles are general statements and definitions for which there is no alternative, as well as, requirements and analytical models for which no alternative is permitted unless specifically stated.

Application rules are generally recognised rules which comply with the Principles and satisfy their requirements. Alternative rules may be used provided that compliance with the Principles can be demonstrated, however the resulting design cannot be claimed to be wholly in accordance with the Eurocode although it will remain in accordance with Principles.

Each Eurocode gives values with notes indicating where national choice may have to be made. These are recorded in the National Annex for each Member State as Nationally Determined Parameters (NDPs).

### 1.3 Scope of the Manual

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

In using the *Manual* the following should be noted:

- The *Manual* has been drafted to comply with BS EN 1992-1-1<sup>1</sup> (EC2 Part 1-1) and BS EN 1992-1-2<sup>2</sup> (EC2 Part 1-2) together with the UK National Annexes.
- The assumed design working life of the structure is 50 years (see BS 8500<sup>3</sup>).
- The structures are braced and non-sway.
- The concrete is of normal weight concrete (see Appendix D for properties).
- The structure is predominantly *in-situ*. For precast concrete, reference should be made to the EC2 manual for precast concrete<sup>4</sup>.
- Normal structure/cladding and finishes interfaces are assumed. For sensitive cladding or finishes reference should be made to the deflection assessment methods in EC2<sup>1</sup>.
- Only initial design information is given with regard to prestressed concrete.
- Prestressed concrete members have bonded or unbonded internal tendons.
- Only tabular methods of fire design are covered.
- The use of mild steel reinforcement is not included. Refer to other standards if its use is required.
- Structures requiring seismic resistant design are not covered. Refer to BS EN 1998<sup>5</sup> (Eurocode 8).
- For elements of foundation and substructure the *Manual* assumes that appropriate section sizes and loads have been obtained from BS EN 1997<sup>6</sup>.
- The *Manual* can be used in conjunction with all commonly used materials in construction; however the data given assumes the following:
  - concrete up to characteristic cylinder strength of 50MPa (cube strength 60MPa)
  - high-tensile reinforcement with characteristic strength of 500MPa with Class B ductility
  - ribbed wire fabric reinforcement with characteristic strength of 500MPa with Class A ductility. Moment redistribution is limited to 20% and yield line design is excluded, except where noted
  - prestressing tendons with 7-wire low-relaxation (Class 2) strands.

For structures or elements outside this scope EC2<sup>1,2,7</sup> should be used.

## 1.4 Contents of the Manual

The *Manual* covers the following design stages:

- i) general principles that govern the design of the layout of the structure
- ii) initial sizing of members
- iii) estimating of quantities of reinforcement and prestressing tendons
- iv) final design of members (except for prestressed concrete members).

## 1.5 Notation and terminology

The notation and terminology follow the Eurocode system.

### Axes

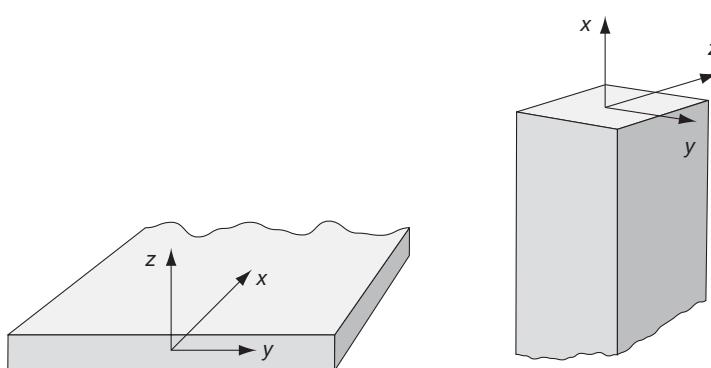
The definitions of the axes are as shown in Figure 1.2.

### Actions

Actions include both loads (permanent or variable) and imposed deformations (e.g. temperature, shrinkage etc.).

### Combination of actions

- *Quasi-permanent combination of actions*: The combination of permanent and variable loads which is most likely to be present most of the time during the design working life of the structure.
- *Frequent combination of actions*: The most likely highest combination of permanent and variable loads which is likely to occur during the design working life of the structure.



**Fig 1.2** Notation for geometric axes

## 2 General principles

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

### 2.1 General

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

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

The permanent and variable load factors to be used for the proportioning of foundations should be obtained from EC0<sup>8</sup> and EC7<sup>6</sup> (see also Section 3.2.1). The factored loads are, however, required for determining the size of foundation members and for the design of any reinforcement.

The engineer should consider site constraints, buildability<sup>9</sup>, maintainability and decommissioning.

The engineer should take account of their responsibilities as a ‘Designer’ under the Construction (Design & Management) Regulations<sup>10</sup>.

### 2.2 Stability

Unbraced structures (‘sway frames’) are not covered by this *Manual* and reference should be made to EC2<sup>1</sup> for their design.

Lateral stability in two orthogonal directions should be provided by a system of strongpoints within the structure so as to produce a braced non-sway structure, which is stiff enough that the columns will not be subject to significant sway moments, nor the building subject to significant global second order effects (see Section 4.8.5). 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 150mm thick to facilitate concreting. For low rise buildings they may be of 215mm brickwork or 190mm solid blockwork properly tied and pinned up to the framing.

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

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

Where a structure is divided by movement 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 Section 5.11.

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

### 2.4 Movement joints

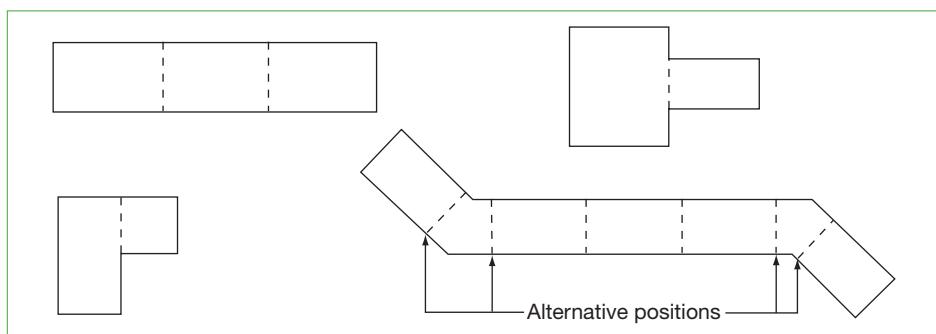
Movement joints may need to be provided to reduce the effects of movements caused by, for example, early age shrinkage, temperature variations, creep and settlement.

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 Figure 2.1.

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

For reinforced concrete frame structures in UK conditions, movement joints at least 25mm wide should normally be provided at approximately 50m centres both longitudinally and transversely. In the top storey with an exposed slab and for open buildings joints should normally be provided to give approximately 25m spacing. Where any joints are placed at over 30m centres the effects of movement (see above) should be included in the global analysis (which is outside the scope of this *Manual*). Joint spacing in exposed parapets should be approximately 12m.

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



**Fig 2.1** Suggested location of movement joints

## 2.5 Fire resistance

For the required period of fire resistance (prescribed in the Building Regulations<sup>11</sup>), the structure should:

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

This *Manual* uses the tabular method given in EC2 Part 1-2<sup>2</sup>. However, there may be benefits if the more advanced methods given in that code are used.

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

## 2.6 Durability

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

- the required performance criteria
- the expected environmental conditions and possible failure mechanism
- the composition, properties and performance of materials
- the shape of members and detailing
- the quality of workmanship/execution
- any protective measure
- the accessibility and location of elements together with likely maintenance during the intended life.

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

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

### 3 Design principles – reinforced concrete

#### 3.1 Loading

The loads to be used in calculations are:

- Characteristic permanent action (dead load),  $G_k$ : the weight of the structure complete with finishes, fixtures and fixed partitions.
- The characteristic variable actions (live loads)  $Q_{ki}$ ; where variable actions act simultaneously a leading variable action is chosen  $Q_{k1}$ , and the other actions are reduced by the appropriate combination factor. Where it is not obvious which should be the leading variable action, each action should be checked in turn and the worse case taken.

For typical buildings these loads are found in:

- BS EN 1991: Eurocode 1: Actions on structures (EC1)
  - Part 1-1: General actions – Densities, self-weight and imposed loads<sup>12</sup>
  - Part 1-3: General actions – Snow loads<sup>13</sup>
  - Part 1-4: General actions – Wind loads<sup>14</sup>
- BS EN 1997: Eurocode 7: Geotechnical design (EC7<sup>6</sup>)

At the ultimate limit state the horizontal forces to be resisted at any level should be the sum of:

- The horizontal load due to the vertical load being applied to a structure with a notional inclination. This inclination can be taken from Table 3.1. This notional inclination leads to all vertical actions having a corresponding horizontal action. This horizontal action should have the same load factor and combination factor as the vertical load it is associated with.
- The wind load derived from BS EN 1991-1-4<sup>14</sup> multiplied by the appropriate partial safety factor.

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

**Table 3.1 Notional inclination of a structure**

<b>Building height (m)</b>	<b>Number of columns stabilised by bracing system</b>			
	<b>1</b>	<b>5</b>	<b>10</b>	<b>≥ 20</b>
≥10	1/300	1/390	1/410	1/410
7	1/270	1/340	1/360	1/370
4	1/200	1/260	1/270	1/280

**Note**

These values are derived from Expression (5.1) of EC2<sup>1</sup>.

### 3.2 Limit states

This *Manual* adopts the limit-state principle and the partial factor format common to all Eurocodes and as defined in BS EN 1990<sup>8</sup> (EC0).

#### 3.2.1 Ultimate limit state (ULS)

The design loads are obtained by multiplying the characteristic loads by the appropriate partial factor  $\gamma_f$  from Table 3.2.

When more than one live load (variable action) is present the secondary live load may be reduced by the application of a combination factor  $\psi_0$  (see Table 3.4).

The basic load combination for a typical building becomes:

$$\gamma_G G_k + \gamma_Q Q_{k1} + \sum \gamma_Q \psi_0 Q_{ki}$$

Where:  $Q_{k1}$ ,  $Q_{k2}$  and  $Q_{k3}$  etc. are the actions due to vertical imposed loads, wind loads and snow etc.,  $Q_{k1}$  being the leading action for the situation considered.

EC0<sup>8</sup> allows alternative combinations which, whilst more complex, may allow for greater economy.

The ‘unfavourable’ and ‘favourable’ factors should be used so as to produce the most onerous condition. Generally permanent actions from a single load source may be multiplied by either the ‘unfavourable’ or the ‘favourable’ factor. For example, all actions originating from the self weight of the structure may be considered as coming from one source and there is no requirement to consider different factors on different spans. Exceptions to this are where overall equilibrium is being checked and the structure is very sensitive to variations in permanent loads (see EC0<sup>8</sup>).

**Table 3.2 Partial factors for loads  $\gamma_f$  at the ultimate limit state**

Permanent Action (Dead load) $G_k$		Variable Actions (Imposed, wind and snow load) $Q_{ki}$		Earth <sup>b</sup> and water <sup>d</sup> (these can generally be considered as permanent actions and factored accordingly)
$\gamma_{G,sup}$	$\gamma_{G,inf}$	$\gamma_Q$ (unfav)	$\gamma_Q$ (fav)	
1.35	1.00	1.50	0.00	1.35

#### Notes

- a Alternative values may be required to check overall equilibrium of structures sensitive to variation in dead weight (see EC0<sup>8</sup>).
- b This assumes that combination 1 of Case 1 (see EC7<sup>6</sup>) is critical for the structural design. This is normal for typical foundations when sized to EC7<sup>6</sup>. For certain structures, such as retaining walls, combination 2 may be more onerous for the structural design. In this combination  $\gamma_G = 1.0$ ,  $\gamma_Q = 1.3$ , and reduction factors are applied to the soil strength. Reference should be made to EC7<sup>6</sup>.
- c For the design of piles and anchors reference should be made to EC7<sup>6</sup>.
- d If the water pressure calculated is the most unfavourable value that could occur during the life of the structure a partial factor of 1.0 may be used.

Further guidance on the use of the use of load combinations is given in *Worked Examples for the design of concrete buildings to Eurocode 2*<sup>19</sup> being prepared by The Concrete Centre.

### 3.2.2 Serviceability limit states (SLS)

The appropriate serviceability limit state should be considered for each specific case. EC2<sup>1</sup> provides specific checks under characteristic, frequent and quasi-permanent loads; the check required varies depending on the effect considered. The corresponding load cases are given in Table 3.3 and are obtained by multiplying the characteristic variable actions by appropriate reduction factors ( $\psi_1$  or  $\psi_2$ ). The values of  $\psi_1$  and  $\psi_2$  are given in Table 3.4. The effects of these factors have been included, where appropriate, in the formulae and tables presented in the *Manual*.

**Table 3.3 Serviceability load cases**

Combination	Permanent Actions	Variable Actions	
	$G_{k,\text{sup}}$	Leading $Q_{kl}$	Others $Q_{ki}$
Characteristic	1.0	1.0	$\psi_0$
Frequent	1.0	$\psi_1$	$\psi_2$
Quasi-permanent	1.0	$\psi_2$	$\psi_2$

**Table 3.4  $\psi$  factors for buildings**

Action	$\psi_0$	$\psi_1$	$\psi_2$
Domestic, residential area	0.7	0.5	0.3
Office area	0.7	0.5	0.3
Congregation areas	0.7	0.7	0.6
Shopping areas	0.7	0.7	0.6
Storage areas	1.0	0.9	0.8
Traffic area	0.7	0.7	0.6
Vehicle $\leq 30\text{kN}$			
Traffic area $30\text{kN} \leq \text{Vehicle} \leq 160\text{kN}$	0.7	0.5	0.3
Roofs	0.7	0.0	0.0
Snow loads	0.7	0.5	0.2
$H \geq 1000\text{m}$ above sea level (a.s.l)			
Snow loads $H \leq 1000\text{m}$ a.s.l	0.5	0.2	0.0
Wind loads	0.5	0.2	0.0
Temperature (non fire)	0.6	0.5	0.0

## 4 Initial design – reinforced concrete

### 4.1 Introduction

In the initial stages of the design of building structures it is necessary, often at short notice, to produce alternative schemes that can be assessed for architectural and functional suitability and which can be compared for cost. They will usually be based on vague and limited information on matters affecting the structure such as imposed loads and nature of finishes, and without 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 of slabs and beams and largest areas of roof and/or floors carried by 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 paths. 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.

The health and safety aspects of the scheme need to be assessed and any hazards identified and designed out wherever possible<sup>10</sup>.

### 4.2 Loads

Loads should be based on BS EN 1991<sup>12-18</sup> (see also Section 3.1).

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

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

- floor finish (screed) 1.8kN/m<sup>2</sup>
- ceiling and service load 0.5kN/m<sup>2</sup>

- Allowance for:
- demountable lightweight partitions       $1.0\text{kN/m}^2$   
– to be treated as imposed loads.
  - blockwork partitions       $2.5\text{kN/m}^2$   
– to be treated as dead loads when the layout is fixed.

Loading from reinforced concrete should be taken as  $25\text{kN/m}^3$ .

### 4.3 Material properties

Design stresses are given in the appropriate sections of the *Manual*. It should be noted that EC2<sup>1</sup> specifies concrete strength class by both the cylinder strength and cube strength (for example C25/30 is a concrete with a characteristic cylinder strength of 25MPa and cube strength of 30MPa at 28 days). Standard strength classes given in EC2<sup>1</sup> are C20/25, C25/30, C30/37, C35/45, C40/50, C45/55 and C50/60. BS 8500<sup>3</sup> gives the following additional classes C28/35 and C32/40 which are not included in either EC2<sup>1</sup> or Appendix D; however interpolation of values is generally applicable. All design equations which include concrete compressive strength use the 28 day characteristic cylinder strength,  $f_{ck}$ . Appendix D gives the strength and deformation properties for concrete.

The partial factor,  $\gamma_c$ , for concrete is 1.5 for ultimate limit state and 1.0 for serviceability limit state. It should also be noted that, for the ultimate limit state  $f_{cd}$  should also be multiplied by  $\alpha_{cc}$ , hence the design strength,  $f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$ . The coefficient  $\alpha_{cc}$  takes account of long term effects on the compressive strength and unfavourable effects resulting from the way the load is applied. In the UK the value of  $\alpha_{cc}$  is generally taken as 0.85, except for shear resistance, where it is taken as 1.0.

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

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

For normal construction in the UK, a concrete strength C30/37MPa should normally be assumed for the initial design.

For UK steels a characteristic strength  $f_{yk}$  of 500MPa should be used.

### 4.4 Structural form and framing

The following measures are recommended for braced structures:

- provide stability against lateral forces and ensure braced construction by arranging suitable shear walls deployed symmetrically wherever possible
- adopt a simple arrangement of slabs, beams and columns so that the load path to the foundations is the shortest and most direct route
- allow for movement joints (see Section 2.4)
- choose a regular grid arrangement that will limit the maximum span of slabs (including flat slabs) to between 6m and 9m and beam spans to between 8m and 12m
- adopt a minimum column size of 300mm x 300mm or equivalent area or as required by fire considerations
- provide a robust structure.

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

#### 4.5 Fire resistance

The size of structural members may be governed by the requirements of fire resistance. Table 4.1 shows the minimum practical member sizes for different periods of fire resistance and the axis distance,  $a$ , from the surface of the concrete to the centre of the main reinforcing bars, required for continuous members (where the moment redistribution is limited to 15%). For simply supported members (and greater redistribution of moments), sizes and axis distance should be increased (see Section 5 and Appendix B).

**Table 4.1 Fire resistance requirements for the initial design of continuous members**

Member	$\mu_{fi}^a \leq 0.5$	width	Minimum dimension (mm)					
			R 30	R 60	R 90	R 120	R 180	R 240
Columns fully exposed to fire	$\mu_{fi}^a \leq 0.5$		200	250	300	450	500	500
	$\mu_{fi}^a \leq 0.7$		200	350	500	500	600	>600 <sup>b</sup>
Columns partly exposed to fire	$\mu_{fi}^a \leq 0.7$	width	155	155	155	175	230	293
Walls exposed on two sides		width	120	140	170	220	270	350
Walls exposed on one side		width	120	130	140	160	210	270
Beams		width	80	150	200	200	240	280
		axis distance	20 <sup>d</sup>	25 <sup>d</sup>	35 <sup>d</sup>	50	60	75
Continuous slabs with plain soffit		thickness	60	80	100	120	150	175
		axis distance	15 <sup>d</sup>	15 <sup>d</sup>	20 <sup>d</sup>	20 <sup>d</sup>	30 <sup>d</sup>	40
Continuous slabs with ribbed open soffit and no stirrups		thickness <sup>c</sup>	80	80	100	120	150	175
		width of ribs	80	100	120	160	310	450
		axis distance	15 <sup>d</sup>	25 <sup>d</sup>	35 <sup>d</sup>	45	60	70
Flat slabs		thickness	150	180	200	200	200	200
		axis distance	15 <sup>d</sup>	15 <sup>d</sup>	25 <sup>d</sup>	35 <sup>d</sup>	45	50

#### Notes

- a**  $\mu_{fi}$  is 'the design axial load in the fire situation' divided by 'the design resistance of the column at normal temperature conditions'. A value of 0.5 should only be assumed if it is lightly loaded. It is unlikely that it will exceed 0.7.
- b** Particular assessment for buckling required.
- c** Thickness of structural topping plus any non-combustible screed.
- d** For practical purposes the axis distance should be such that the minimum cover is 20mm.

## 4.6 Durability

The size of structural members may be affected by the cover necessary to ensure durability (see Appendix B).

## 4.7 Stiffness

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

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

**Table 4.2 Basic ratios of span/effective depth for initial design ( $f_{yk} = 500\text{MPa}$ )**

Structural system	Span/Effective Depth Ratio	
	Beam	Slab
Simply supported beam	14	
One-way or two-way spanning simply supported slab		20
End span of: Continuous beam	18	
One-way continuous slab; or two-way spanning slab continuous over one long side		26
Interior span of: Beam	20	
One-way or two-way spanning slab		30
Slab supported on columns without beams (flat slab), based on longer span		24
Cantilever	6	8
Notes		
a	For two-way spanning slabs (supported on beams), the check on the ratio of span/effective depth should be carried out on the shorter span. For flat slabs, the longer span should be taken.	
b	For flanged sections with the ratio of the flange to the rib width greater than 3, the Table value for beams should be multiplied by 0.8.	
c	For members, other than flat slab panels, which support partitions liable to be damaged by excessive deflection of the member, and where the span exceeds 7m, the Table value should be multiplied by 7/span.	
d	For flat slabs where the greater span exceeds 8.5m, the Table value should be multiplied by 8.5/span.	
e	The values may not be appropriate when the formwork is struck at an early age or when the construction loads exceed the design load. In these cases the deflection may need to be calculated using advice in specialist literature.	

## 4.8 Sizing

### 4.8.1 Introduction

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

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

### 4.8.2 Loading

Ultimate loads, i.e. characteristic loads multiplied by the appropriate partial factors, should be used throughout. At this stage it may be assumed that all spans are fully loaded, unless the members (e.g. overhanging cantilevers) concerned are sensitive to unbalanced loading (see Section 4.8.7.1).

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

### 4.8.3 Width of beams and ribs

The width should be determined by limiting the shear stress in beams to 2.0MPa and in ribs to 0.6MPa for concrete of characteristic strength  $f_{ck}/f_{cu} \geq 25/30\text{MPa}$ :

- width of beam (in mm) =  $1000V / 2d$
- width of rib (in mm) =  $1000V / 0.6d$

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

### 4.8.4 Sizes and reinforcement of columns

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

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

- |  |      |
|--|------|
| • 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. façade 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.

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

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

**Table 4.3 Equivalent ‘stress’ values**

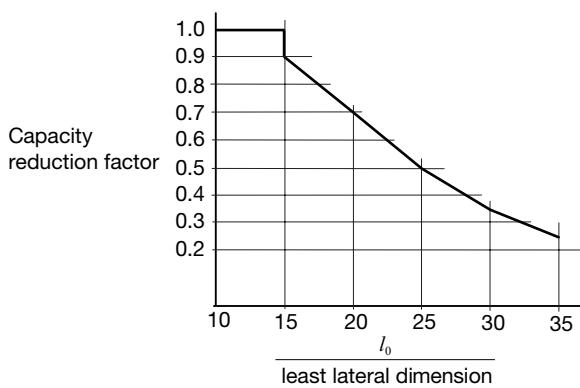
Reinforcement (500MPa) percentage $\rho$	Equivalent stresses (MPa) for concrete strength classes		
	C25/30	C30/37	C35/45
$\rho = 1\%$	14	17	19
$\rho = 2\%$	18	20	22
$\rho = 3\%$	21	23	25
$\rho = 4\%$	24	27	29

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

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

Where:  $f_{ck}$  is the characteristic concrete strength in MPa  
 $f_y$  the characteristic strength of reinforcement in MPa  
 $\rho$  the percentage of reinforcement.

Where slender columns (i.e. the ratio of the effective height  $l_0$ , to the least lateral dimension,  $b$ , exceeds 15) are used, the ultimate load capacity of the column or equivalent ‘stress’ should be reduced by the appropriate factor from Figure 4.1. In braced frames  $l_0$  may be taken as the clear floor to soffit height.



**Fig 4.1** Reduction factors for slender columns

#### 4.8.5 Walls ( $h \geq 4b$ )

Walls carrying vertical loads can initially be designed as columns. Shear walls should be designed as vertical cantilevers, and the reinforcement arrangement should be checked as for a beam. Where the shear walls have returns at the compression end, they should be treated as flanged beams. This *Manual* assumes that shear walls are sufficiently stiff that global second order effects do not need to be considered. The walls should be sized such that:

$$F_{V,Ed} \leq 0.517 \frac{n_s}{(n_s + 1.6)} \frac{\sum E_{cm} I_c}{L^2}$$

Where:  $F_{V,Ed}$  is the total vertical load (on the whole structure stabilised by the wall)

$n_s$  is the number of storeys

$L$  is the total height of building above level of moment restraint

$E_{cm}$  is the mean modulus of elasticity

$I_c$  is the second moment of area (uncracked concrete section) of the wall(s).

This assumes that:

- torsional instability is not governing, i.e. structure is reasonably symmetrical
- global shear deformations are negligible (as in a bracing system mainly consisting of shear walls without large openings)
- base rotations are negligible
- the stiffness of the wall is reasonably constant throughout the height
- the total vertical load increases by approximately the same amount per storey.

In the above equation for  $F_{V,Ed}$  it should be noted that the value 0.517 should be halved if the wall is likely to be cracked.

#### 4.8.6 Punching shear in flat slabs at columns

Check that:

- i) where shear reinforcement is to be avoided or slabs are less than 200mm thick

$$\frac{1250w(A_{supp})}{(u_c + 12h)h} \leq 0.6 \text{ MPa}$$

- ii) where shear reinforcement may be provided

$$\frac{1250w(A_{supp})}{(u_c + 12h)h} \leq 1.0 \text{ MPa}$$

- iii) Check also that in the above verification

$$\frac{1250w(A_{supp})}{(u_c)h} \leq 0.15f_{ck}$$

Where:  $w$  is the total design ultimate load per unit area in kN/m<sup>2</sup>

$h$  is the thickness of the slab at the column in mm.

$A_{supp}$  is the area supported by the column in m<sup>2</sup>

$u_c$  is column perimeter in mm.

It should be noted that for slabs less than 200mm thick shear reinforcement is not effective.

## 4.8.7 Adequacy of chosen sections to accommodate the reinforcement

The actual bar arrangement should be considered at an early stage particularly where the design is close to reinforcement limits.

### 4.8.7.1 Bending moment and shear forces

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

#### Beams and one-way solid slabs

Bending moments and shear forces in continuous structures can be obtained from Table 4.4 when:

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

**Table 4.4 Ultimate bending moments and shear forces**

	<b>Uniformly distributed loads</b> $F$ = total design ultimate load on span	<b>Central point loads</b> $W$ = design ultimate point load
Bending moments at support at midspan	$-0.100 FL$ $0.080 FL$	$-0.150 WL$ $0.175 WL$
Shear forces	$0.65 F$	$0.65 W$
<i>Note</i> $L$ is the span.		

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

#### Two-way solid slabs on linear supports

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

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

Where:  $w$  is the design ultimate load in  $\text{kN/m}^2$ , 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 (see Figure 4.2) in each direction as 1.5 times those for one-way slabs.

### One-way ribbed slabs

Assess the bending moments at midspan on a width equal to the rib spacing, assuming simple supports throughout.

### Two-way ribbed slabs on linear supports

If the longer span does not exceed 1.5 times the shorter span, estimate the average rib moment in both directions as:

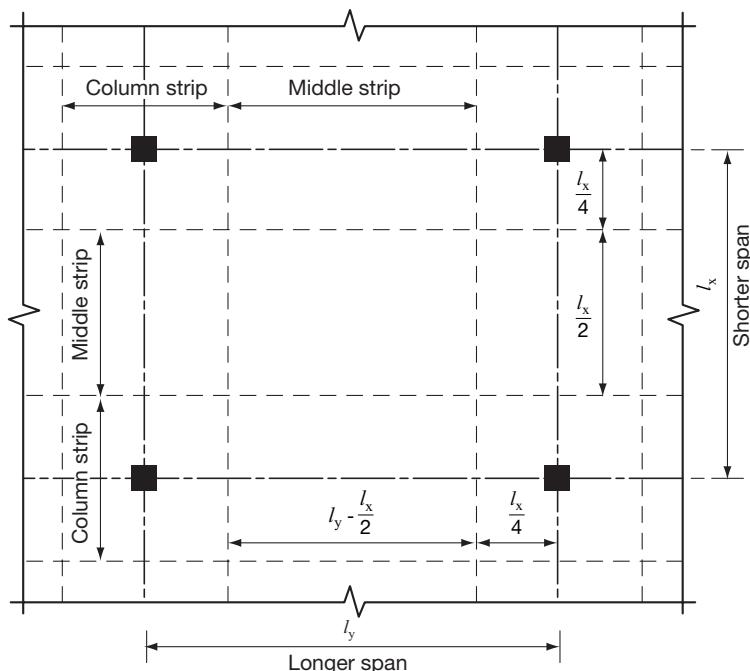
$$w \frac{l_x l_y}{18} c \text{ kNm per rib}$$

Where:  $c$  is the rib spacing in metres.

If  $l_y > 1.5l_x$  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 4.4. For the column strips increase this by 15%.



**Fig 4.2** Division of panel without drops into strips

#### 4.8.7.2 Provision of reinforcement

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

##### Tension reinforcement

Reinforcement can now be calculated by the following formula:

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

Where:  $M$  is the design ultimate bending moment at the critical section  
 $d$  is the effective depth.

##### Compression reinforcement

If, for a rectangular section,  $M > 0.167f_{ck}bd^2$ , assuming no redistribution, compression reinforcement is required:

$$A_{s2} = \frac{M - 0.167 f_{ck} bd^2}{0.87 f_{yk} (d - d_2)}$$

Where:  $A_{s2}$  is the area of the compression steel  
 $d_2$  is the depth to its centroid  
 $b$  is the width of the section  
 $d$  is its effective depth.

If, for flanged sections,  $M > 0.567f_{ck} b_f h_f (d - 0.5h_f)$  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 0.36d.

It should be noted that where compression reinforcement is required transverse reinforcement should be provided to restrain the main reinforcement from buckling.

##### Shear reinforcement

$$A_{sw} = \frac{V}{f_{ywd} \cot \theta} \left( \frac{s}{0.9d} \right) \text{ (for initial sizing } \cot \theta = 1\text{)}$$

Where:  $V$  is the design ultimate shear force at the critical section  
 $s$  is the spacing of shear reinforcement  
 $d$  is the effective depth  
 $f_{ywd}$  is the design yield strength of the shear reinforcement.

##### 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 10mm nor normally more than 20mm in diameter.

## 4.9 The next steps

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

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

## 4.10 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.

In the case of reinforcement quantities the basic requirements are, briefly:

- for bar reinforcement to be described separately by: steel type, diameter and weight and divided up according to:
  - a) element of structure, e.g. foundations, slabs, walls, columns, etc.
  - b) bar 'shape', e.g. straight, bent or hooked; curved; links, stirrups and spacers.
- for fabric (mesh) reinforcement to be described separately by: steel type, fabric type and area, divided up according to a) and 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  $10\text{m}^3$
- offices, shops, hotels: 1 tonne per  $13.5\text{m}^3$
- residential, schools: 1 tonne per  $15\text{m}^3$ .

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.

### Method 2

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

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

Reference should be made to standard tables and spreadsheets available from suitable organisations (e.g. The Concrete Centre).

### Method 3

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

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

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

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

- Laps and starter bars – A reasonable allowance should be made for normal laps in both main and distribution bars, and for starter bars. This should be checked if special lapping arrangements are used.
- Architectural features – The drawings should be looked at and sufficient allowance made for the reinforcement required for such ‘non-structural’ features.
- Contingency – A contingency of between 10% and 15% should be added to cater for some changes and for possible omissions.

## 5 Final design - reinforced concrete

### 5.1 Introduction

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

- i) checking of all information
- ii) preparation of a list of design data
- iii) amendment of drawings as a basis for final calculations.

#### 5.1.1 Checking of all information

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

##### *Stability*

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

##### *Movement joints*

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

##### *Loading*

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

Make a final check on the design wind loading and consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account. In general the load case including permanent, imposed, and wind load will be most onerous for all elements, however it is not normally considered necessary to include wind load for members that do not form part of the direct wind resistance system as the wind load effects will be small and can be neglected. However local effects do need to be checked.

### *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 resisting construction for basements.

### *Hazards*

Identify any hazard resulting from development of the scheme design. Explore options to mitigate<sup>10</sup>.

#### **5.1.2 Preparation of a list of design data**

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

#### **5.1.3 Amendment of drawings as a basis for final calculations**

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

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

- Gridlines – Establish gridlines in two directions, mutually at right-angles for orthogonal building layouts, consistent with that adopted by the rest of the design team: identify these on the plans.
- Members – Give all walls, columns, beams and slabs unique reference numbers or a combination of letters and numbers related if possible to the grid, so that they can be readily identified on the drawings and in the calculations.

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

#### 5.1.4 Final design calculations

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

- slabs (Section 5.2)
- structural frames (Section 5.3)
- beams (Section 5.4)
- columns (Section 5.5)
- walls (Section 5.6)
- staircases (Section 5.7)
- non-suspended ground floor slabs (Section 5.8)
- retaining walls, basements (Section 5.9)
- foundations (Section 5.10)
- robustness (Section 5.11)
- detailing (Section 5.12).

## 5.2 Slabs

### 5.2.1 Introduction

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

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

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

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

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

The general procedure to be adopted is as follows:

- i) Check that the cross section and cover comply with requirements for fire resistance.
- ii) Check that cover and concrete grade comply with requirements for durability.

- iii) Calculate bending moments and shear forces.
- iv) Calculate reinforcement.
- v) Make final check on span/depth ratios.
- vi) For flat slabs check shear around columns and calculate shear reinforcement as necessary.

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

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

*The effective length of a cantilever slab* where this forms the end of a continuous slab is the length of the cantilever from the centre of the support.

## 5.2.2 Fire resistance and durability

### 5.2.2.1 Fire resistance

The member size and reinforcement cover required to provide fire resistance are given in Table 5.1. When using this table, the redistribution of moments in continuous slabs should be limited to 15% although the bending moments given in Tables 5.2, 5.3 and 5.4 of this manual may be used. The cover in the Table may need to be increased for durability (see Section 5.2.2.2).

### 5.2.2.2 Durability

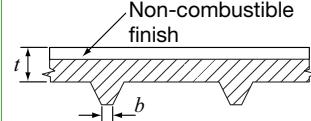
The requirements for achieving durability in any given environment are:

- an upper limit to the water/cement ratio
- a lower limit to the cement content
- a lower limit to the nominal cover to the reinforcement
- good compaction
- adequate curing
- good detailing.

For a given value of nominal cover (expressed as minimum cover plus an allowance for deviation,  $\Delta c_{dev}$ ) Table B.2 (50 years) and Table B.3 (100 years) of Appendix B give values of concrete class, an upper limit to the water cement ratio and cement content which, in combination, will be adequate to ensure durability for various environments.

Where it is specified that only a contractor with a recognised quality system shall do the work (e.g. member of SpeCC, the Specialist Concrete Contractors certification scheme)  $\Delta c_{dev} = 5\text{mm}$ , otherwise  $\Delta c_{dev} = 10\text{mm}$ .

**Table 5.1 Fire resistance requirements for slabs**

Standard fire resistance (R) in minutes	Plain soffit solid slab (including joist + block) Minimum overall depth (mm)	Ribbed soffit (including T-section + channel section)		Flat slabs Minimum overall depth (mm)	
		 <b>Non-combustible finish</b>			
		<b>Minimum thickness/width, (mm/mm)</b>			
		Simply Supported	Continuous		
		<i>t/b</i>	<i>t/b</i>		
R 60	80	80/120	80/100	180	
R 90	100	100/160	100/120	200	
R 120	120	120/190	120/160	200	
R 180	150	150/260	150/310	200	
R 240	175	175/350	175/450	200	
<b>Axis distance to reinforcement, <i>a</i> (mm)</b>					
	Simply supported	Continuous	Simply supported	Continuous	Flat slabs
R 60	20 <sup>a</sup>	10 <sup>a</sup>	25 <sup>a</sup>	25 <sup>a</sup>	15 <sup>a</sup>
R 90	30 <sup>a</sup>	15 <sup>a</sup>	40	35 <sup>a</sup>	25 <sup>a</sup>
R 120	40	20 <sup>a</sup>	55	45	35 <sup>a</sup>
R 180	55	30 <sup>a</sup>	70	60	45
R 240	65	40	75	70	50

#### Notes

- a** For practical purposes the axis distance should be such that the minimum cover is 20mm.
- b** The axis distance, *a* is measured from the surface of the concrete to the centre of the main reinforcing bars.
- c** The axis distance, *a* should be increased by 10mm for prestressing bars and 15mm for prestressing wires or strands.
- d** For other combinations of rib width and axis distance see EC2, Part 1-2<sup>2</sup>.
- e** Where *a* is 70mm or more refer to EC2, Part 1-2<sup>2</sup> for additional requirements.

### 5.2.3 Bending moments and shear forces

#### 5.2.3.1 General

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

- Alternate spans carrying the maximum design dead and imposed load (i.e.  $1.35G_k + 1.5Q_k$ ), other spans carrying the maximum design dead load (i.e.  $1.35G_k$ ).
- All spans carrying the maximum design dead and imposed load (i.e.  $1.35G_k + 1.5Q_k$ ). The moments obtained from elastic analysis may be redistributed up to a maximum of 30% except for plain or indented fabric for which the limit is 15%.

It should be noted that:

- the resulting distribution of moments should remain in equilibrium with the applied load
- the design redistributed moment at any section should not be less than 70% of the elastic moment
- there are limitations in the depth of the neutral axis of the section depending on the percentage of redistribution (see Section 5.2.4.1).

#### Concentrated loads

The bending moment arising from a concentrated load may be distributed over a width of slab equal to the width of the load plus the lesser of the actual width or  $1.2(1 - (x/l))x$  on each side of the load (see Figure 5.1), where  $x$  is the distance to the nearer support from the section under consideration, and  $l$  is the span.

#### 5.2.3.2 One-way spanning slabs

For continuous slabs with a) substantially uniform loading b) dead load greater than or equal to imposed load and c) at least three spans that do not differ by more than 15%, the bending moments and shear forces may be calculated using the coefficients given in Table 5.2.

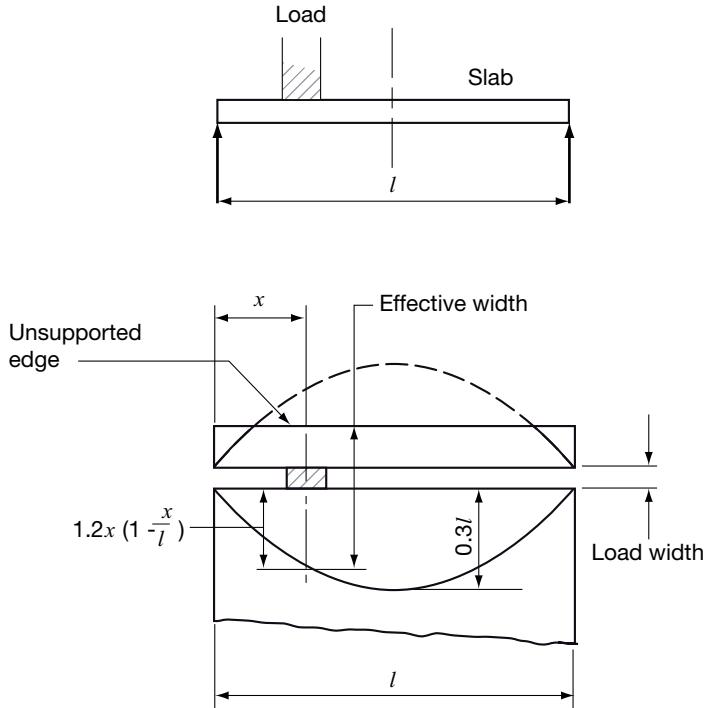
**Table 5.2 Bending moments and shear forces for one-way slabs**

	Simple		Continuous		Penultimate support	Interior spans	Interior supports
	End support	End span	End support	End span			
Moment	0	$0.086Fl$	$-0.04Fl$	$0.075Fl$	$-0.086Fl$	$0.063Fl$	$-0.063Fl$
Shear	$0.4F$	–	$0.046F$	–	$0.6F$	–	$0.5F$

#### Notes

- a F is the total design ultimate load ( $1.35G_k + 1.5Q_k$ ) for each span.
- b l is the span.

Allowance has been made in the coefficients in Table 5.2 for 20% redistribution of moments.



**Fig 5.1** Effective width of solid slab carrying a concentrated load near an unsupported edge

### 5.2.3.3 Two-way spanning slabs on linear supports

Bending moments in two-way slabs may be calculated by any valid method provided the ratio between support and span moments are similar to those obtained by the use of elastic theory with appropriate redistribution. In slabs where the corners are prevented from lifting, the coefficients in Table 5.3 may be used to obtain bending moments per unit width ( $m_{sx}$  and  $m_{sy}$ ) in the two directions for various edge conditions, i.e.:

$$m_{sx} = \beta_{sx} n l_x^2$$

$$m_{sy} = \beta_{sy} n l_x^2$$

Where:  $\beta_{sx}$  and  $\beta_{sy}$  are the coefficients given in Table 5.3  
 $n$  is the total design ultimate load per unit area ( $1.35 G_k + 1.5 Q_k$ )  
 $l_x$  is the shorter span.

**Table 5.3 Bending moment coefficients for two-way spanning rectangular slabs**

Type of panel and moments considered	Short span coefficients $\beta_{sx}$ Values of $\frac{l_y}{l_x}$					Long-span coefficients $\beta_{sy}$ for all values of $\frac{l_y}{l_x}$
	1.00	1.25	1.50	1.75	2.00	
1 Interior panels:						
Negative moment at continuous edge	0.031	0.044	0.053	0.059	0.063	0.032
Positive moment at midspan	0.024	0.034	0.040	0.044	0.048	0.024
2 One short edge discontinuous:						
Negative moment at continuous edge	0.039	0.050	0.058	0.063	0.067	0.037
Positive moment at midspan	0.029	0.038	0.043	0.047	0.050	0.028
3 One long edge discontinuous:						
Negative moment at continuous edge	0.039	0.059	0.073	0.082	0.089	0.037
Positive moment at midspan	0.030	0.045	0.055	0.062	0.067	0.028
4 Two adjacent edges discontinuous:						
Negative moment at continuous edge	0.047	0.066	0.078	0.087	0.093	0.045
Positive moment at midspan	0.036	0.049	0.059	0.065	0.070	0.034

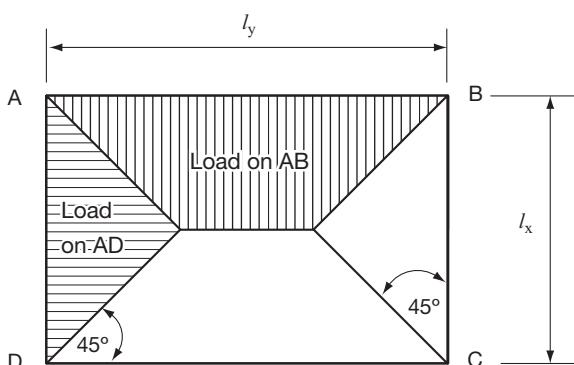
The distribution of the reactions of two-way slabs on to their supports can be derived from Figure 5.2. It should be noted that reinforcement is required in the panel corners to resist the torsion forces (see Section 5.2.4.1 (i)).

Class A reinforcement is assumed to have sufficient ductility for use with this simplified design method or yield line analysis of two way slabs.

#### 5.2.3.4 Flat slabs

If a flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exceed 1.2, the maximum values of the bending moments and shear forces in each direction may be obtained from Table 5.4. This assumes 20% redistribution of bending moments.

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



#### Notes

- a** The reactions shown apply when all edges are continuous (or discontinuous).
- b** 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%.
- c** When adjacent edges are discontinuous, the reactions should be adjusted for elastic shear considering each span separately.

**Fig 5.2** Distribution of reactions from two-way slabs onto supports

**Table 5.4 Bending moment and shear force coefficients for flat slab panels of three or more equal spans**

	Outer support	Near middle of end span	At first interior support	At middle of interior span(s)	At internal supports
Moment	- 0.040 $Fl^b$	0.086 $Fl^c$	- 0.086 $Fl$	0.063 $Fl$	- 0.063 $Fl$
Shear	0.460 $F$	–	0.600 $F$	–	0.500 $F$
Total column moments <sup>d</sup>	0.040 $Fl$	–	0.022 $Fl$	–	0.022 $Fl$

#### Notes

- a  $F$  is the total design ultimate load ( $1.35G_k + 1.5Q_k$ ).
- b These moments may have to be reduced to be consistent with the capacity to transfer moments to the columns; the midspan moments c must then be increased correspondingly.
- d The total column moment should be distributed equally between the columns above and below.
- e Moments at supports may be reduced by  $0.15Fh_c$  where  $h_c$  is the effective diameter of the column or column head.

#### 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 Figure 4.2). In the assessment of the widths of the column and middle strips, drops should be ignored if their smaller dimension is less than one-third of the smaller dimension of the panel.

#### Division of moments between column and middle strips

The design moments obtained from analysis of the frames or from Table 5.4 should be divided between the column and middle strips in the proportions given in Table 5.5.

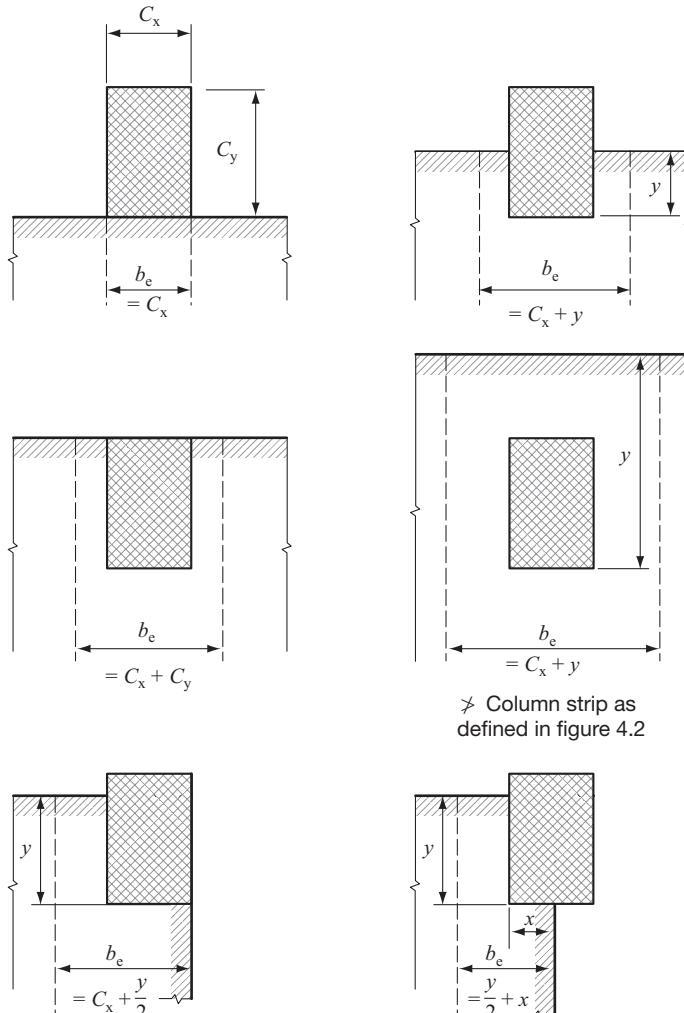
**Table 5.5 Distribution of design moments of flat slabs**

Design moment	Column strip %	Middle strip %
Negative	75	25
Positive	55	45

#### Note

For the case where the width of column strip is taken as equal to that of the drop and the middle strip is thereby increased in width, the design moments to be resisted by the middle strip should be increased in proportion to its increased width. The design moments to be resisted by the column strip may be decreased by an amount such that the total positive and the total negative design moments resisted by the column strip and middle strip together are unchanged.

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 Figure 5.3.  $b_e$  should not be taken as greater than the column strip width appropriate for an interior panel.



**Note**

$y$  is the distance from the face of the slab to the innermost face of the column.

**Fig 5.3** Definition of width of effective moment transfer strip,  $b_e$  on plan

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

$$M_{\max} = 0.17 f_{ck} b_e d^2$$

Where:  $d$  is the effective depth for the top reinforcement in the column strip, and  $f_{ck} \leq 35 \text{ MPa}$ .

Where the transfer moment at an edge column obtained from Table 5.4 is greater than  $M_{\max}$  a further moment redistribution  $\leq 10\%$  may be carried out.

Where the elastic transfer moment at an edge column obtained from a frame analysis is greater than  $M_{\max}$  moment redistribution  $\leq 50\%$  may be carried out.

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 then:

- the total design load to be carried by the beam or wall should include those loads directly on the wall or beam plus a uniformly distributed load equal to one-quarter of the total design load on the panel; and
- the design moments of the half-column strip adjacent to the beam or wall should be one-quarter of the design moments obtained from analysis.

#### *Effective shear forces in flat slabs*

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

The design effective shear force  $V_{\text{eff}}$  at the perimeter of the column should be taken as:

$$\begin{aligned} V_{\text{eff}} &= 1.15 V_{\text{Ed}} \text{ for internal columns with approximately equal spans} \\ &= 1.4 V_{\text{Ed}} \text{ for edge columns} \\ &= 1.5 V_{\text{Ed}} \text{ for corner columns} \end{aligned}$$

Where:  $V_{\text{Ed}}$  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.

Where the adjacent spans differ by more than 25% or the lateral stability depends on frame action  $V_{\text{eff}}$  should be calculated in accordance with EC2<sup>1</sup>, Clause 6.4.3.

## 5.2.4 Section design – solid slabs

### 5.2.4.1 Bending

#### (i) Reinforcement

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

The area of tension reinforcement is then given by:

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

Where:  $z$  is obtained from Figure 5.5 for different values of  $K = \frac{M}{bd^2 f_{ck}}$

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

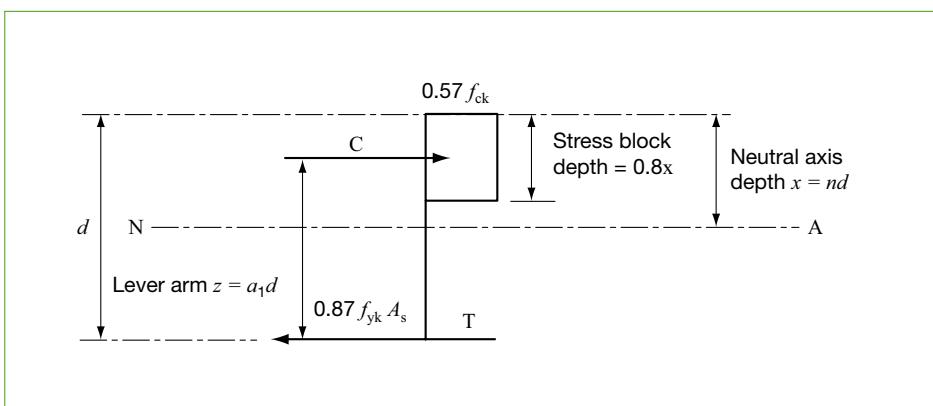
#### (ii) Detailing

Two-way slabs on linear supports

The reinforcement calculated from the bending moments obtained from Section 5.2.3.3 should be provided for the full width in both directions.

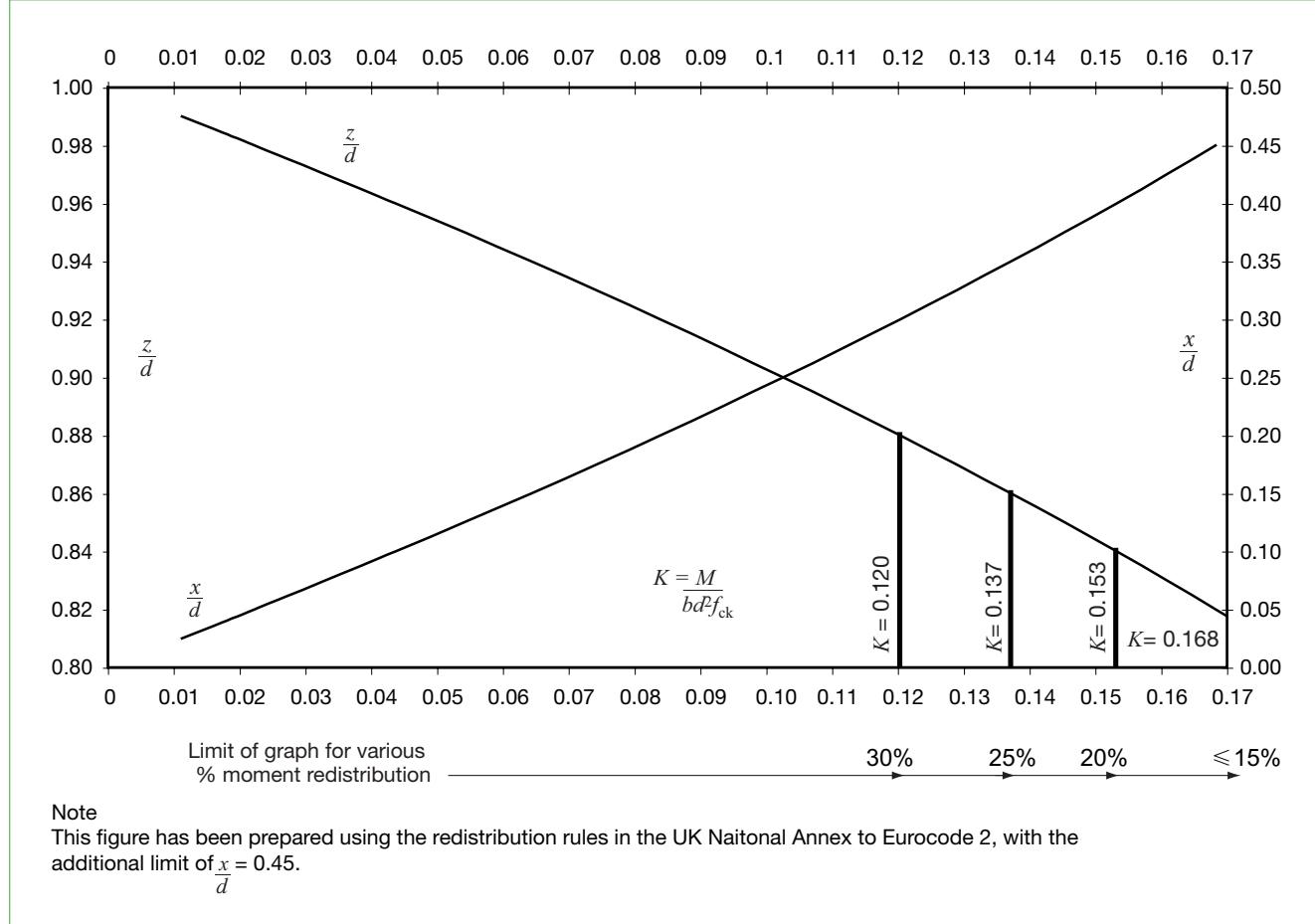
In the corner area shown in Figure 5.6:

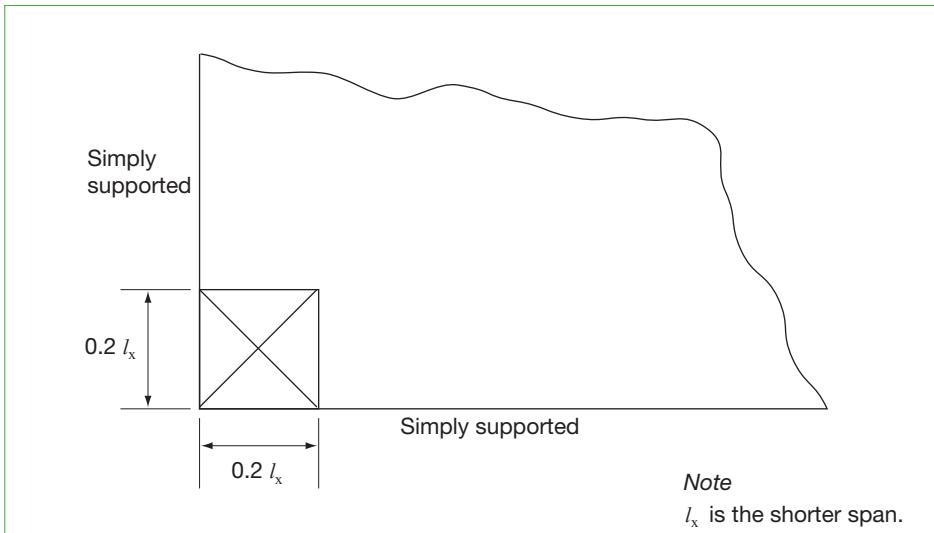
- i) provide top and bottom reinforcement
- ii) in each layer provide bars parallel to the slab edges
- iii) in each of the four layers the area of reinforcement per unit length should be equal to 75% of the reinforcement required for the maximum moment in the span per unit length
- iv) the area of reinforcement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



**Fig 5.4** Stress diagram

**Fig 5.5** Values of lever arm and neutral axis depth





**Fig 5.6** Corner reinforcement: two-way spanning slabs

#### Flat slabs

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

#### Minimum and maximum reinforcement

The area of reinforcement in each direction should not be less than  $0.00014 f_{ck}^{2/3} b h$  or  $0.0015 b h$

Where:  $h$  is the overall depth of the slab (taken as  $d/0.87$ )

$b$  is the width for which the reinforcement is calculated.

If control of shrinkage and temperature cracking is critical, the area of reinforcement should not be less than  $0.0065 b h$  or 20% of the area of main reinforcement.

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

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

To control flexural cracking the maximum bar spacing or maximum bar diameter of high-bond bars should not exceed the values given in Table 5.6, corresponding to the stress in the bar. In any case bar spacings should not exceed the lesser of  $3h$  or 500mm.

**Table 5.6. Alternative requirements to control crack widths to 0.3mm for members reinforced with high bond bars**

Maximum bar diameter (mm)	Stress range (MPa)	OR	Maximum bar spacing (mm)	Stress range (MPa)
40	150 – 165		300	≤ 160
32	165 – 190		275	160 – 180
25	190 – 210		250	180 – 200
20	210 – 230		225	200 – 220
16	230 – 260		200	220 – 240
12	260 – 290		175	240 – 260
10	290 – 320		150	260 – 280
8	320 - 360		125	280 – 300
			100	300 – 320
			75	320 – 340
			50	340 – 360

#### Note

The stress in the reinforcement may be estimated from the relationship:

$$\sigma_s = \left( \frac{f_{yk}}{\gamma_{ms}} \right) \left\{ \frac{\psi_2 Q_k + G_k}{1.5Q_k + 1.35G_k} \right\} \left( \frac{A_{s,req}}{A_{s,prov}} \right) \left( \frac{1}{\delta} \right).$$

Where:  $\psi_2$  should be obtained from Table 3.4 for the particular type of loading considered.

$\frac{f_{yk}}{\gamma_{ms}}$  may be taken as 435 for 500MPa reinforcement.

$A_{s,req}$  is the area of tension reinforcement required at the section considered for the ultimate limit state.

$A_{s,prov}$  is the area of reinforcement actually provided.

$\delta$  is the ratio of the redistributed ultimate moment to the elastic ultimate moment at the section considered ( $\leq 1$ ).

#### 5.2.4.2 Shear

It should be noted that for slabs less than 200mm thick shear reinforcement is not effective.

##### Solid single-way and two-way slabs

Shear reinforcement is not normally required provided the design ultimate shear force  $V_{Ed}$  does not exceed  $V_{Rd,c}$ .

$$V_{Rd,c} = 0.12k (100of_{ck})^{1/3} b_w d$$

but not less than:

$$V_{Rd,c} = 0.035\sqrt{f_{ck}} k^{3/2} b_w d$$

$$\text{Where: } k = 1 + \sqrt{\frac{200}{d}} \leq 2 \quad \text{and} \quad \rho = \frac{A_{sl}}{b_w d} \leq 0.02$$

Where:  $A_{sl}$  is the area of tensile reinforcement, which extends beyond the section considered taking account of the ‘shift rule’ (see Section 5.12.6).

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

### Flat slabs

The shear stress at the column perimeter should be checked first:

$$v_{Ed} = \frac{1000V_{\text{eff}}}{u_0 d} \text{ MPa} \leq 0.2 \left( 1 - \left( \frac{f_{ck}}{250} \right) \right) f_{ck}$$

Where:  $V_{\text{eff}}$  is the effective shear force in kN (the shear force magnified by the effect of moment transfer, see Section 5.2.3.4)

$d$  is the average of the effective depth of the tension reinforcement in both directions

$u_0$  is the column perimeter in mm.

For an interior column  $u_0$  = length of the column perimeter

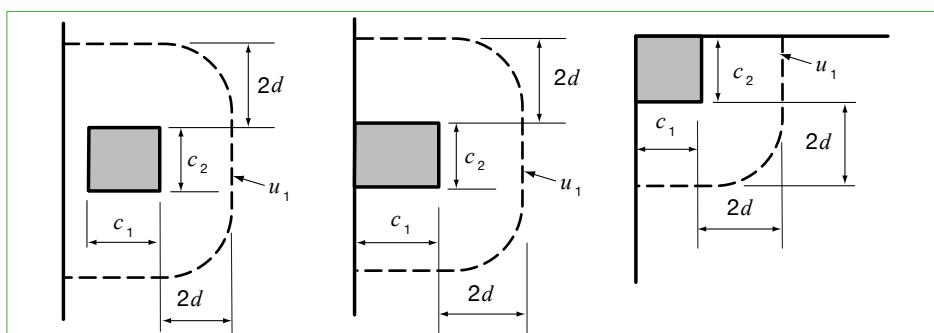
For an edge column  $u_0 = c_2 + 3d \leq c_2 + 2c_1$

For a corner column  $u_0 = 3d \leq c_2 + c_1$ .

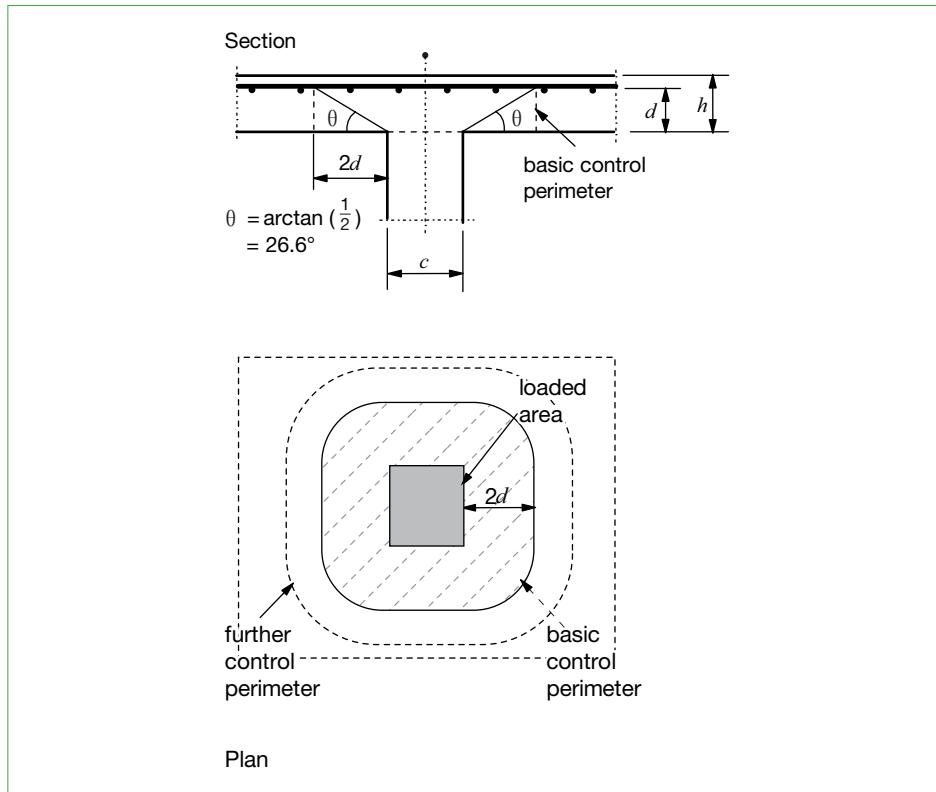
The shear stresses should then be checked at the basic control perimeter,  $2d$  from the column perimeter:

$$v_{Ed} = \frac{1000V_{\text{eff}}}{u_1 d} \text{ MPa}$$

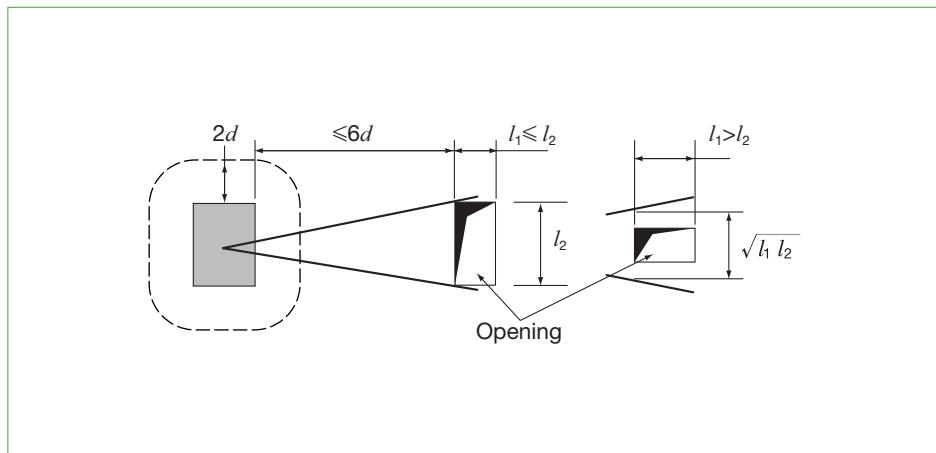
Where:  $u_1$  is the length of the basic control perimeter in mm as defined in Figure 5.7 (columns close to a free edge), Figure 5.8 or Figure 5.9 (openings close to columns).



**Fig 5.7** Basic control perimeters for loaded areas close to or at an edge



**Fig 5.8** Shear perimeters for internal columns



**Fig 5.9** Control perimeter near an opening

If the applied shear stress at the basic control perimeter is less than the permissible ultimate shear stress  $v_{Rd,l}$  (see Table 5.7) no further checks are required. If  $v_{Ed} > v_{Rd,c}$ , the outer control perimeter,  $u_{out}$ , at which  $v_{Ed} \leq v_{Rd,c}$  is then determined.

**Table 5.7 Ultimate shear stress  $v_{Rd,c}$  MPa**

$\frac{100 A_s}{b_w d}$	Effective depth (mm)											
	150	175	200	225	250	275	300	400	500	750	1000	2000
≤0.25	0.54	0.54	0.54	0.52	0.50	0.48	0.47	0.43	0.40	0.36	0.34	0.31
0.5	0.59	0.59	0.59	0.57	0.56	0.55	0.54	0.51	0.48	0.45	0.43	0.39
0.75	0.68	0.68	0.68	0.66	0.64	0.63	0.62	0.58	0.55	0.51	0.49	0.45
1.00	0.75	0.75	0.75	0.72	0.71	0.69	0.68	0.64	0.61	0.57	0.54	0.49
1.25	0.80	0.80	0.80	0.78	0.76	0.74	0.73	0.69	0.66	0.61	0.58	0.53
1.5	0.85	0.85	0.85	0.83	0.81	0.79	0.78	0.73	0.70	0.65	0.62	0.56
≥2.00	0.94	0.94	0.94	0.91	0.89	0.87	0.85	0.80	0.77	0.71	0.68	0.62

#### Notes

- a The tabulated values apply for  $f_{ck} = 30\text{MPa}$ . Approximate values for other concrete strengths may be used:  
 For  $f_{ck} = 25\text{MPa}$  the tabulated values should be multiplied by 0.95  
 For  $f_{ck} = 35\text{MPa}$  the tabulated values should be multiplied by 1.05  
 For  $f_{ck} = 40\text{MPa}$  the tabulated values should be multiplied by 1.1  
 For  $f_{ck} = 45\text{MPa}$  the tabulated values should be multiplied by 1.15.
- b The Table does not allow for any contribution from axial loads. For an axial compression where stress of  $\sigma_{cp} = (N/A_c)$  MPa, the Table values should be increased by  $0.1\sigma_{cp}$ , where  $N$  is the design axial load and  $A_c$  is the area of concrete section.

Shear reinforcement should be provided within the area between the column face and  $1.5d$  inside the outer control perimeter (see Figure 5.10) such that:

$$v_{Rd,cs} = 0.75v_{Rd,c} + \left(1.5 \frac{d}{s_r}\right) A_{sw} f_{ywd,ef} \left(\frac{1}{u_l d}\right) \sin \alpha$$

Where:  $A_{sw}$  is the area of one perimeter of shear reinforcement around the column

$s_r$  is the radial spacing perimeters of shear reinforcement

$f_{ywd,ef}$  is the effective design strength of the punching shear reinforcement, according to:

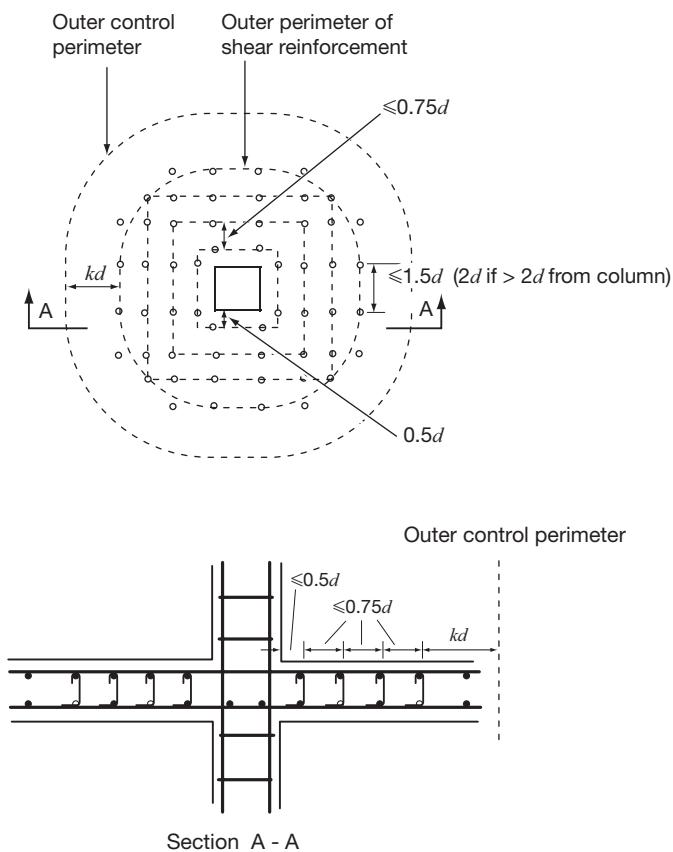
$$f_{ywd,ef} = 250 + 0.25d \leq \frac{f_y}{1.15}$$

$d$  is the mean of the effective depths in the orthogonal directions

$\alpha$  is the angle between the reinforcement and the plane of the slab.

The reinforcement should be provided in at least two perimeters of links. The spacing of link perimeters (see Figure 5.10) should not exceed  $0.75d$ .

The spacing of link legs around a perimeter should not exceed  $1.5d$  within the basic control perimeter ( $2d$  from the column face) and should not exceed  $2d$  for perimeters outside the basic control perimeter where that part of the perimeter is assumed to contribute to the shear capacity (see Figure 5.10).



#### Note

$k = 1.5$ , unless the perimeter at which reinforcement is no longer required is less than  $3d$  from the face of the loaded area/column. In this case the reinforcement should be placed in the zone  $0.3d$  and  $1.5d$  from the face of the column.

**Fig 5.10** Layout of flat slab shear reinforcement

The area of a link leg (or equivalent),  $A_{sw,min}$ , is given by:

$$A_{sw,min} \frac{1.5}{s_r s_t} \geq 0.08 \frac{\sqrt{f_{ck}}}{f_{yk}}$$

Where:  $s_r$  is the spacing of shear links in the radial direction  
 $s_t$  is the spacing of shear links in the tangential direction.

The distance between the face of a support, or the circumference of a loaded area, and the nearest shear reinforcement taken into account in the design should not exceed  $d/2$ . This distance should be taken at the level of the tensile reinforcement.

Where proprietary products are used as shear reinforcement,  $V_{Rd,cs}$  should be determined by testing in accordance with the relevant European Technical Approval.

#### 5.2.4.3 Openings

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

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

#### 5.2.5 Span/effective depth ratios

The span/effective depth ratio should not normally exceed the appropriate value in Table 5.8. The depth of slab may be further optimised by reference to EC2<sup>1</sup>.

#### 5.2.6 Section design - ribbed and coffered slabs

Ribbed or waffle slabs need not be treated as discrete elements for the purposes of analysis, provided that the flange or structural topping and transverse ribs have sufficient torsional stiffness. This may be assumed provided that:

- the rib spacing does not exceed 1500mm
- the depth of the rib below the flange does not exceed 4 times its width
- the depth of the flange is at least 1/10 of the clear distance between ribs or 50mm, whichever is the greater
- transverse ribs are provided at a clear spacing not exceeding 10 times the overall depth of the slab.

**Table 5.8 Span/effective depth ratios for slabs**

<b>Location</b>	$\frac{A_{s,\text{req}}}{bd} \geq 1.5\%$	$\frac{A_{s,\text{req}}}{bd} = 0.5\%$	$\frac{A_{s,\text{req}}}{bd} \leq 0.35\%$
One-or two-way spanning slab:			
Simply supported	14	20	30
End span	18	26	39
Interior span	20	30	45
Flat slab	17	24	36
Cantilever	6	8	12
<b>Notes</b>			
<p><b>a</b> Values may be interpolated.</p> <p><b>b</b> For flanged sections where the ratio of the flange to the rib width exceeds 3, the values should be multiplied by 0.8.</p> <p><b>c</b> For spans exceeding 7m, other than for flat slabs, supporting partitions liable to be damaged by excessive deflections, the value should be multiplied by 7/span (in metres).</p> <p><b>d</b> The above assumes <math>f_{yk} = 500\text{MPa}</math>. If other values of <math>f_{yk}</math> are used then multiply the above by <math>500/f_{yk}</math>.</p> <p><b>e</b> <math>A_{s,\text{req}}/bd</math> is calculated at the location of maximum span moment. The values given in the Table may be increased by the ratio of <math>A_{s,\text{prov}}/A_{s,\text{req}}</math>.</p> <p><b>f</b> For flat slabs, supporting brittle partitions, where the greater span exceeds 8.5m, the value should be multiplied by 8.5/span.</p>			

### 5.2.6.1 Bending

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

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

### 5.2.6.2 Span/effective depth ratios

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

### 5.2.6.3 Shear

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

$$\tau_{Ed} = \frac{1000V_{Ed}}{b_w d} \text{ MPa}$$

Where:  $V_{Ed}$  is the design shear force arising from design ultimate loads per rib in kN  
 $b_w$  is the average width of the rib in mm  
 $d$  is the effective depth in mm.

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

- increase width of rib
- reduce spacing of ribs
- provide solid concrete at supports
- provide shear reinforcement only if none of the above is possible.

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

#### 5.2.6.4 Beam strips in ribbed and coffered slabs

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

#### 5.2.7 Notes on the use of precast floors

Use of precast or semi-precast construction in an otherwise *in-situ* reinforced concrete building is not uncommon. There are various proprietary precast and prestressed concrete floors on the market. Precast floors can be designed to act compositely with an *in-situ* structural topping, although the precast element can carry loads without reliance on the topping. Design using proprietary products should be carried out closely in conjunction with the particular manufacturer and in accordance with EC2, Section 10<sup>1</sup>. The following points may be helpful to the designer:

- The use of a structural topping should be considered, particularly to reduce the risk of cracking in the screed and finishes:
  - when floors are required to resist heavy concentrated loads such as those due to storage racking and heavy machinery
  - 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.
- In selecting a floor, fire rating, durability and acoustic insulation need to be considered as well as structural strength.

- Precast components should be detailed to give a minimum bearing (after allowing for tolerances) of 75mm on concrete beams and walls, but in cases where this bearing cannot be achieved reference should be made to EC2, Section 10<sup>1</sup> for more detailed guidance. Mechanical anchorage at the ends should be considered. The design should cater for the tying requirements for accidental loading (see Section 5.11).
- 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).
- A ceiling to mask steps between adjoining units may be necessary.
- Holes required for services need to be planned.
- An *in-situ* make-up strip should be provided to take up the tolerances between precast units and *in-situ* construction.

### 5.3 Structural frames

#### 5.3.1 Division into subframes

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

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

**Table 5.9 Effective widths of flanged beams**

	T-beam	L-beam
End span	$b_w + 0.170l$	$b_w + 0.085l$
Interior spans	$b_w + 0.140l$	$b_w + 0.070l$
Cantilever	$b_w + 0.200l$	$b_w + 0.100l$

**Notes**

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

### 5.3.2 Elastic analysis

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

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

- i)    a)    Alternate spans carrying the maximum design permanent and variable load, i.e.  $(1.35G_k + 1.5Q_k)$ , other spans carrying the maximum design permanent load, i.e.  $1.35G_k$  and
  - b)    Any two adjacent spans carrying the maximum design permanent and variable load, i.e.  $(1.35G_k + 1.5Q_k)$ , other spans carrying the maximum design permanent load, i.e.  $1.35G_k$ .
- ii)   a)    All spans carrying the design permanent and variable load, i.e.  $(1.35G_k + 1.5Q_k)$  and
  - b)    Alternate spans carrying the design permanent and variable load, i.e.  $(1.35G_k + 1.5Q_k)$ , other spans carrying only the design permanent load, i.e.  $1.35G_k$ .
- iii)   For slabs only: All spans loaded condition, i.e.  $(1.35G_k + 1.5Q_k)$ , provided that:
  - a)    In a one-way spanning slab the area of each bay exceeds  $30m^2$  and
  - b)     $Q_k/G_k \leq 1.25$  and
  - c)     $Q_k \leq 5kN/m^2$ .

Where analysis is carried out for the single load case of all spans loaded, the resulting support moments except those at the supports of cantilevers should be reduced by 20% with a consequential increase in the span moments. In this context a bay means a strip across the full width of a structure bounded on the other two sides by lines of support.

The above simplifications may be applied using Expression 6.10 or 6.10a and 6.10b of BS EN 1990<sup>8</sup>.

### 5.3.3 Redistribution of moments

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

- the resulting distribution of moments remains in equilibrium with the applied load
- the design redistribution moment at any section should not be less than 70% of the elastic moment
- there are limitations on the depth of the neutral axis of the section depending on the percentage of redistribution (see Section 5.4.4.1), and
- 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:

- i) Adjacent spans loaded – Reduce the common support moment to not less than the support moment obtained from the alternate spans loaded case.
- ii) 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 Figure 5.11).

### 5.3.4 Design shear forces

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

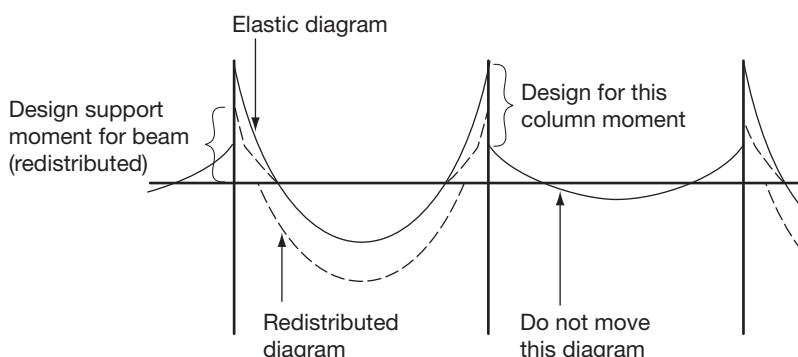
## 5.4 Beams

### 5.4.1 Introduction

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

The general procedure to be adopted is as follows:

- i) check that the section complies with the requirements for fire resistance
- ii) check that cover and concrete quality comply with durability requirements
- iii) calculate bending moments and shear forces according to Section 5.4.3.
- iv) calculate reinforcement required for bending and shear
- v) check span/depth ratio.



#### Note

It is not recommended to redistribute the span moment upwards.

**Fig 5.11** Redistribution procedures for frames

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

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

*The effective length of a cantilever beam* where this forms the end of a continuous beam is the length of the cantilever from the centre of the support.

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

In normal slab-and-beam or framed construction specific calculations for torsion are not usually necessary, torsional cracking being adequately controlled by shear reinforcement. Where torsion is essential for the equilibrium of the structure, e.g. the arrangement of the structure is such that loads are imposed mainly on one face of a beam without corresponding rotational restraints being provided, EC2<sup>1</sup> should be consulted.

## 5.4.2 Fire resistance and durability

### 5.4.2.1 Fire resistance

The member sizes and reinforcement axis distances required to provide fire resistance are shown in Table 5.10 and 5.11. When using these tables, continuous beams should be treated as simply supported if the redistribution of bending moments for normal temperature design exceeds 15%.

**Table 5.10 Fire resistance requirements for simply supported beams**

Standard fire resistance (R) in minutes	Possible combinations of $a$ the average axis distance and $b_{\min}$ the width of beam				Web thickness $b_w$
	Minimum dimensions (mm)				
R30	$b_{\min} = 80$ $a = 25$	120 20 <sup>a</sup>	160 15 <sup>a</sup>	200 15 <sup>a</sup>	80
R60	$b_{\min} = 120$ $a = 40$	160 35 <sup>a</sup>	200 30 <sup>a</sup>	300 25 <sup>a</sup>	100
R90	$b_{\min} = 150$ $a = 55$	200 45	300 40	400 35 <sup>a</sup>	110
R120	$b_{\min} = 200$ $a = 65$	240 60	300 55	500 50	130
R180	$b_{\min} = 240$ $a = 80$	300 70	400 65	600 60	150
R240	$b_{\min} = 280$ $a = 90$	350 80	500 75	700 70	170

**Notes**

- a** For practical purposes the axis distance should be such that the minimum cover is 20mm.
- b** The axis distance,  $a$ , is measured from the surface of the concrete to the centre of the main reinforcing bars.
- c** The axis distance,  $a$ , should be increased by 10mm for prestressing bars and 15mm for prestressing wires or strands.
- d** For other combinations of rib width and axis distance see EC2, Part 1-2<sup>2</sup>.
- e** Where  $a$  is 70mm or more refer to EC2, Part 1-2<sup>2</sup> for additional requirements.
- f**  $a_{sd}$  is the axis distance to the side of beam for the corner bars (or tendon or wire) of beams with only one layer of reinforcement ( $a_{sd} = a + 10\text{mm}$ ). For values of  $b_{\min}$  greater than that given in the highlighted column no increase of  $a_{sd}$  is required.

**5.4.2.2 Durability**

The requirements for durability in any given environment are:

- i) an upper limit to the water/cement ratio
- ii) a lower limit to the cement content
- iii) a lower limit to the thickness of the cover to the reinforcement
- iv) good compaction
- v) adequate curing
- vi) good detailing.

Values for i), ii) and iii) which, in combination, will give adequate durability are given in Appendix B for various environments.

**Table 5.11 Fire resistance requirements for continuous beams**

Standard fire resistance (R) in minutes	Possible combinations of $a$ the average axis distance and $b_{\min}$ the width of beam				Web thickness $b_w$
	Minimum dimensions (mm)				
R60	$b_{\min} = 120$ $a = 25^a$	200 12 <sup>a</sup>			100
R90	$b_{\min} = 150$ $a = 35^a$	250 25 <sup>a</sup>			110
R120	$b_{\min} = 200$ $a = 45$	300 35 <sup>a</sup>	450 35 <sup>a</sup>	500 30 <sup>a</sup>	130
R180	$b_{\min} = 240$ $a = 60$	400 50	550 50	600 40	150
R240	$b_{\min} = 280$ $a = 75$	500 60	650 60	700 50	170

**Notes**

- a** For practical purposes the axis distance should be such that the minimum cover is 20mm.
- b** The axis distance,  $a$ , is measured from the surface of the concrete to the centre of the main reinforcing bars.
- c** The axis distance,  $a$ , should be increased by 10mm for prestressing bars and 15mm for prestressing wires or strands.
- d** For other combinations of rib width and axis distance see EC2, Part 1-2<sup>2</sup>.
- e** Where  $a$  is 70mm or more refer to EC2, Part 1-2<sup>2</sup> for additional requirements.
- f**  $a_{sd}$  is the axis distance to the side of beam for the corner bars (or tendon or wire) of beams with only one layer of reinforcement ( $a_{sd} = a + 10\text{mm}$ ). For values of  $b_{\min}$  greater than that given in the highlighted column no increase of  $a_{sd}$  is required.

**5.4.3 Bending moments and shear forces**

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

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

For beams with a) substantially uniform loading, b) one type of imposed load, and c) three or more spans that do not differ by more than 15%, the bending moments and shear forces may be calculated using the coefficients given in Table 5.12 for ultimate limit state verification. No redistribution of moments should be made when using values obtained from this Table.

**Table 5.12 Bending moments and shear forces for beams at ultimate limit state**

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0 <sup>a</sup>	0.09Fl	- 0.11Fl	0.07Fl	- 0.10Fl
Shear	0.45F	-	0.60F	-	0.55F

#### Notes

- a** See Section 5.4.4.2.
- b**  $F$  is the total design ultimate load ( $1.35G_k + 1.5Q_k$ ) for each span, and  $l$  is the length of the span.
- c** No redistribution of the moments calculated from this Table should be made.

## 5.4.4 Section design

### 5.4.4.1 Bending

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

The effective width of flange should be based on the distance  $l_0$  between points of zero moment, which may be obtained from Figure 5.12.

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

The resistance moments of concrete sections that are required to resist flexure only can be determined from the formulae that are based on the stress diagram in Figure 5.4.

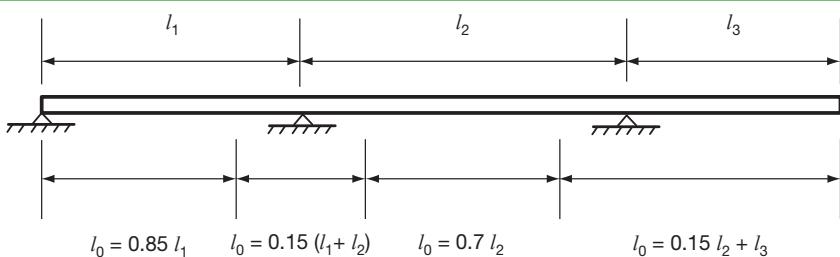
The effect of any small axial compressive load on the beam can be ignored if the design ultimate axial force is less than  $0.08f_{ck}bh$ , where  $h$  is the overall depth of the section.

#### Rectangular beams

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

- Calculate  $M_u = K_{\text{lim}} f_{ck} bd^2$  where  $K_{\text{lim}}$  is the limiting value of  $K$  as obtained from Figure 5.5 for the amount of redistribution carried out.
- If  $M < M_u$ , the area of tension reinforcement  $A_s$  is calculated from:

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



**Fig 5.12** Definition of  $l_0$  for calculation of effective flange width

Where:  $z$  is obtained from Figure 5.5 for different values of  $K$

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

- iii) If  $M > M_u$  then compression reinforcement is needed. The area of compression reinforcement  $A_{s2}$  is calculated from:

$$A_{s2} = \frac{M - M_u}{0.87f_{yk}(d - d_2)}$$

Where:  $d_2$  is the depth to the centre of the compression reinforcement from the compression face.

$$\text{If } d_2 > \left(1 - \frac{f_{yk}}{800}\right)x, \text{ use } 700\left(l - \frac{d_2}{x}\right) \text{ in lieu of } 0.87f_{yk}$$

The area of tension reinforcement  $A_s$  is calculated from:

$$A_s = \frac{M_u}{0.87f_{yk}z} + A_{s2}$$

#### Flanged beams

For section design, provided that the ratio of adjacent spans is between 1 and 1.5, the effective width of a flanged beam (see Figure 5.13) may be derived as:

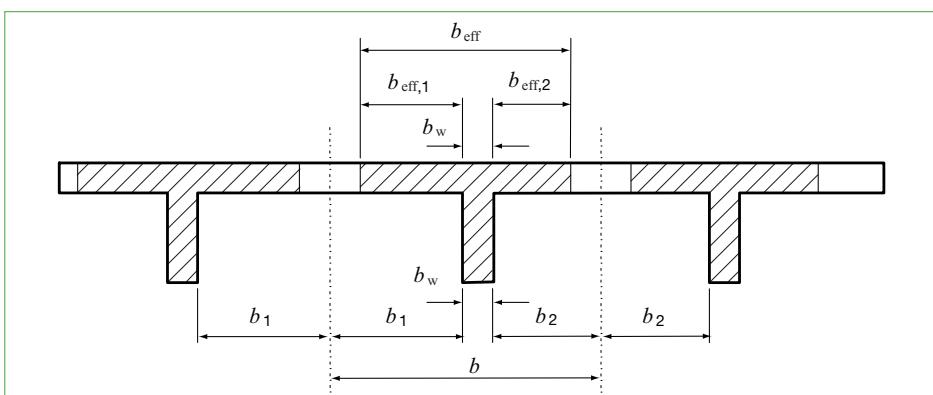
$$b_{\text{eff}} = \sum b_{\text{eff},i} + b_w \leq b$$

Where:  $b_{\text{eff},i} = 0.2b_i + 0.1l_0 \leq 0.2l_0$

and

$b_{\text{eff},i} \leq b_i$  ( $b_i$  is either  $b_1$  or  $b_2$ ).

It should be noted that the flange width at the support will be different from that at midspan. The length of the cantilever should be less than half the adjacent span.



**Fig 5.13** Effective flange width parameters

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

- Check the position of the neutral axis by determining

$$K = \frac{M}{b_{\text{eff}} d^2 f_{ck}}$$

using effective flange width  $b_{\text{eff}}$  and selecting values of  $x$  and  $z$  from Figure 5.5.

- If  $0.8x < h_f$  then  $A_s$  is determined as for a rectangular beam of breadth  $b_{\text{eff}}$ , i.e.

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

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

$$M_{uf} = 0.57 f_{ck} (b_{\text{eff}} - b_w) h_f (d - 0.5h_f)$$

- Calculate  $K_f = \frac{M - M_{uf}}{f_{ck} b_w d^2}$

If  $K_f \leq K_{\text{lim}}$ , obtained from Figure 5.5 for the amount of redistribution carried out, then select value of  $z/d$  and hence  $z$ . Calculate  $A_s$  from:

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

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

#### 5.4.4.2 Minimum and maximum amounts of reinforcement

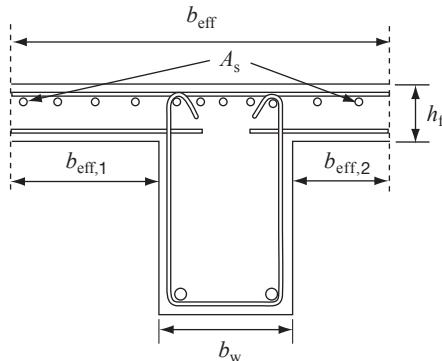
The areas of reinforcement derived from the previous calculations may have to be modified or supplemented in accordance with the requirements below in order to prevent brittle failure (without warning) and/or excessive cracking. Main bars in beams should normally not be less than 16mm in diameter.

##### Tension reinforcement

The area of reinforcement should not be less than  $0.00016 f_{ck}^{2/3} b_t d$  or  $0.0013 b_t d$ , where  $b_t$  is the mean width of the tension zone.

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

At intermediate supports of continuous beams, the total amount of tensile reinforcement  $A_s$  of a flanged cross-section may be distributed uniformly over the effective width,  $b_{\text{eff}}$ , as shown in Figure 5.14.



**Fig 5.14** Distribution of reinforcement in flanged beams

#### *Compression reinforcement*

The minimum areas of compression reinforcement where required should be:

rectangular beam  $0.002bh$

flanged beam web in compression  $0.002b_w h$

#### *Maximum area of reinforcement*

Neither the area of the tension reinforcement, nor the area of compression reinforcement should exceed  $0.04A_c$ .

#### *Bars along the side of beams (to control cracking)*

Where the overall depth of the beam is 750mm or more, 16mm diameter bars should be placed along the sides, below any flange, at a maximum pitch of 250mm.

#### *Maximum spacing for tension bars*

To control flexural cracking at serviceability the maximum bar spacing or maximum bar diameters of high-bond bars should not exceed the values in Table 5.6, corresponding to the stress in the bar.

#### *Minimum spacing*

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

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

#### 5.4.4.3 Shear

Shear reinforcement is not required where the design ultimate shear force  $V_{Ed}$  does not exceed  $V_{Rd,c}$ .

$$V_{Rd,c} = 0.12k (100\varrho f_{ck})^{1/3} b_w d \quad \text{but not less than: } V_{Rd,c} = 0.035 \sqrt{f_{ck}} k^{3/2} b_w d$$

Where:  $k = 1 + \sqrt{200/d} \leq 2$

$\varrho = A_{sl}/b_w d \leq 0.02$  where  $A_{sl}$  is the area of flexural tensile reinforcement provided, which extends beyond the section.

Where  $V_{Ed}$  exceeds  $V_{Rd,c}$  shear reinforcement is required.

This is assessed by the variable strut inclination method as shown in Figure 5.15.

$$\cot\theta = y/z \quad (\text{where } z \text{ may be taken as } 0.9d) \text{ and } 1 \leq \cot\theta \leq 2.5$$

$$V_{Rd,min} = 0.16 d b_w \sqrt{f_{ck}}$$

$$A_{sw}/s = 0.08 b_w \sqrt{f_{ck}} / f_{yw}$$

$$V_{Rd,sy} = A_{sw} z f_{yw} \cot\theta / s \quad (\text{shear reinforcement control})$$

$$V_{Rd,max} = 0.18 d b_w f_{ck} (1-f_{ck}/250) \sin 2\theta \quad (\text{concrete strut control})$$

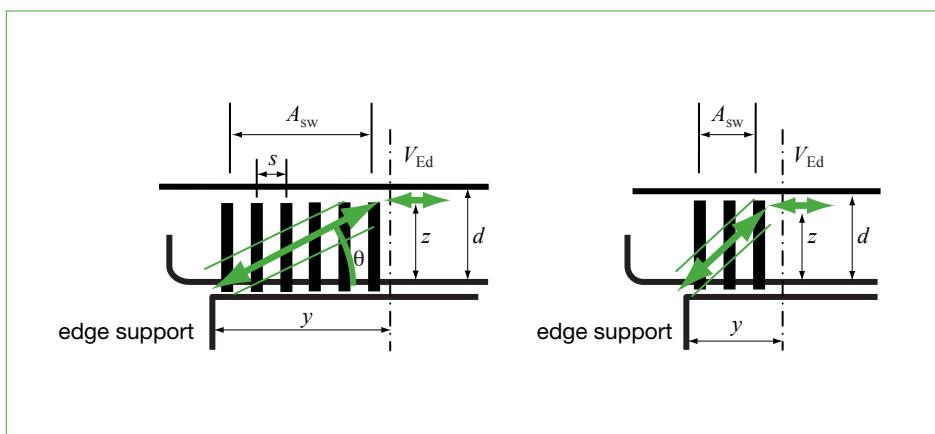
$$\text{for strut at } (\cot\theta = 2.5): V_{Rd,max} = 0.13 d b_w (1-f_{ck}/250) f_{ck}$$

$$\text{for strut at } (\cot\theta = 1): V_{Rd,max} = 0.18 d b_w (1-f_{ck}/250) f_{ck}$$

#### Procedure for design

- Calculate shear capacity for minimum reinforcement.

$$V_{min} = 0.15d b_w \sqrt{f_{ck}} \quad (\text{assuming } \cot\theta = 2.5)$$



**Fig 5.15** Variable strut inclination method

- ii) Calculate angle  $\theta$ , of strut for full shear force at end of beam  
(assuming  $\alpha_{cc} = 1$  and  $z = 0.9d$ ).

$$\theta = 0.5 \sin^{-1} \left[ \frac{5.56 V_{Ed}}{b_w d (1 - f_{ck}/250) f_{ck}} \right]$$

If there is no solution then this implies that the strut has failed.

- iii) If  $\cot\theta < 1$  then the beam should be resized,  $f_{ck}$  increased or the load reduced.
- iv) If  $\cot\theta \geq 1$  then check reinforcement required at  $z \cot\theta$  from end of beam with shear force at that point.
- v) If  $\cot\theta \geq 2.5$  then assume  $\cot\theta = 2.5$  for the calculation of shear reinforcement.

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

Where a concentrated load is applied on the upper side of the beam within a distance  $0.5d \leq a_v \leq 2d$  from the face of the support (or centre of bearing where flexible bearings are used), the contribution of this load to the shear force  $V_{Ed}$  may be reduced by a factor  $\beta$  where  $\beta = a_v / 2d$ . This reduction is only valid provided that the longitudinal reinforcement is fully anchored at the support. Where  $a_v \leq 0.5d$ , the value of  $a_v$  should be taken as  $0.5d$ .

#### *Arrangement of links*

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

#### *Openings*

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

### 5.4.5 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value in Table 5.13.

## 5.5 Columns ( $h \leq 4b$ )

### 5.5.1 Introduction

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

The general procedure to be adopted is as follows:

- i) calculate axial loads and moments according to Sections 5.5.3 to 5.5.5
- ii) check the slenderness of the column
- iii) check that section size and cover comply with requirements for fire resistance
- iv) check that cover and concrete comply with requirements for durability
- v) design section and reinforcement.

**Table 5.13 Span/effective depth ratios for beams**

<b>Location</b>	$\frac{A_s}{bd} \geq 1.5\%$	$\frac{A_s}{bd} = 0.5\%$	$\frac{A_s}{bd} \leq 0.35\%$
Simply supported beam	14	20	30
End span of continuous beam	18	26	39
Interior span	20	30	45
Cantilever	6	8	12

Notes

- a Values may be interpolated.
- b For flanged sections where the ratio of the flange width to the rib width exceeds 3, the values should be multiplied by 0.8.
- c For spans exceeding 7m supporting partitions liable to be damaged by excessive deflections, the value should be multiplied by 7/span (in metres).
- d The above assumes  $f_{yk} = 500\text{MPa}$ . If other values of  $f_{yk}$  are used then multiply the above by  $500/f_{yk}$ .
- e  $A_s/bd$  is calculated at the location of maximum span moment.

## 5.5.2 Slenderness, fire resistance and durability

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

### 5.5.2.1 Slenderness

The slenderness of a column is defined by the ratio of the effective height to the dimension of the section perpendicular to the axis of bending considered. Using the notation given in Figure 5.16, a column is slender if:

$$\frac{l_{oz}}{b} > 4.38 \left( 1.7 - \frac{M_{1z}}{M_{2z}} \right) \xi \quad \text{OR} \quad \frac{l_{oy}}{h} > 4.38 \left( 1.7 - \frac{M_{1y}}{M_{2y}} \right) \xi$$

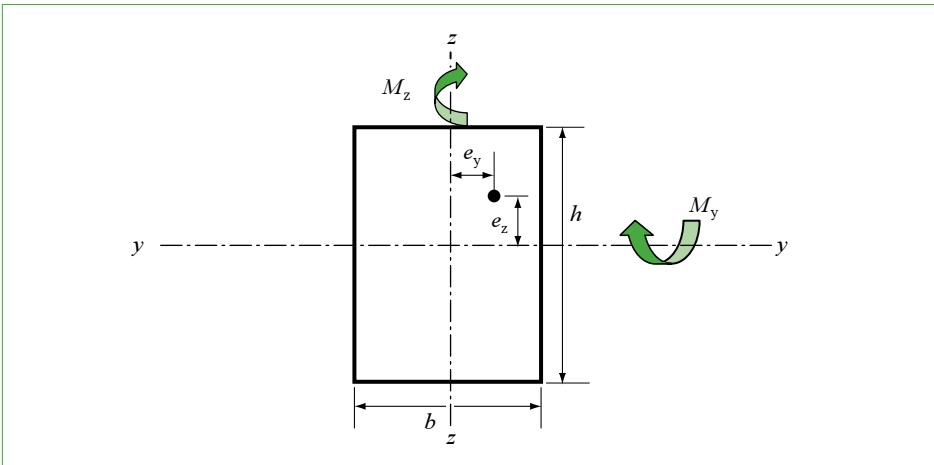
$$(\text{for circular columns, diameter } D: \frac{l_o}{D} > 3.79 \left( 1.7 - \frac{M_{1z}}{M_{2z}} \right) \xi)$$

Where:  $M_{1z}$  and  $M_{1y}$  are the numerically smaller end moments about the z- and y- axes respectively  
 $M_{2z}$  and  $M_{2y}$  are the numerically larger end moments about the z- and y- axes respectively

$$\xi = 0.69 \sqrt{\frac{(1 + 2\omega)(A_c f_{ck})}{N_{Ed}}} \geq 1.0$$

$$\omega = 1.53 \frac{A_s f_{yk}}{A_c f_{ck}} \quad (\text{When } A_s \text{ is not known } \omega_{\min} \text{ may be taken as } 0.003 \frac{f_{yk}}{f_{ck}})$$

It should be noted that, for most columns in framed structures,  $M_1$  will have the opposite sign to  $M_2$ .



**Fig 5.16** Axes and eccentricities for columns

This figure has been deleted as the slenderness limits indicated by it have been found to be generally less conservative than EC2. The equations on page 58 should be used instead.

**Fig 5.17** Slenderness limits for braced columns

For columns where the stiffness of any vertically adjacent column does not vary significantly, the effective height may be obtained by multiplying the clear height between the lateral restraints at the two ends of the column by the factor obtained from Table 5.14. More detailed methods of calculation are available in EC2<sup>1</sup>.

**Table 5.14 Effective height,  $l_0$ , 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

#### Notes

- a 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.
- b 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.
- c Condition 3 Column connected to members that do not provide more than nominal restraint to rotation.

#### 5.5.2.2 Fire resistance

Table 5.15 is based on Method A given in EC2 Part 1.2<sup>2</sup> (Clause 5.3.2) and provides minimum dimensions and axis distances for columns in braced structures. Table 5.15 is valid for the following conditions:

- for intermediate floors the actual length of the column (centre to centre)  $\leq 6\text{m}$
- for the upper floor the actual length of the column (centre to centre)  $\leq 4.5\text{m}$
- the first order eccentricity under fire conditions  $\leq 0.15h$  (or  $b$ )
- the amount of reinforcement  $< 0.04 A_c$ .

Where these conditions are not met reference should be made to EC2, Part 1-2<sup>2</sup>.

#### 5.5.2.3 Durability

The requirements for durability in any given environment are:

- i) an upper limit to the water/cement ratio
- ii) a lower limit to the cement content
- iii) a lower limit of the cover to the reinforcement
- iv) good compaction
- v) adequate curing
- vi) good detailing.

Values for i), ii) and iii) which, in combination will give adequate durability, are given in Appendix B for various environments.

**Table 5.15 Fire resistance requirements for columns with rectangular or circular section**

Standard fire resistance (R) in minutes	Minimum dimensions (mm)			
	Column width $b_{min}$ /axis distance, $a$ , of the main bars			Exposed on one side
	$\mu_{fi}^a = 0.2$	$\mu_{fi} = 0.5$	$\mu_{fi} = 0.7$	$\mu_{fi} \leq 0.7$
R 30	200/25	200/25	200/32 300/27	155/25
R 60	200/25	200/36 300/31	250/46 350/40	155/25
R 90	200/31 300/25	300/45 400/38	350/53 450/40 <sup>b</sup>	155/25
R 120	250/40 350/35	350/45 <sup>b</sup> 450/40 <sup>b</sup>	350/57 <sup>b</sup> 450/51 <sup>b</sup>	175/35
R 180	350/45 <sup>b</sup>	350/63 <sup>b</sup>	450/70 <sup>b</sup>	230/55
R 240	350/61 <sup>b</sup>	450/75 <sup>b</sup>	—	295/70

**Notes**

- a**  $\mu_{fi}$  is the ratio of design axial load in the fire situation / the design resistance of the column at normal temperature conditions. It is unlikely to exceed 0.7.
- b** Minimum 8 bars with a bar at the centre of each face.
- c** The axis distance,  $a$ , is measured from the surface of the concrete to the centre of the main reinforcing bars.
- d** The axis distance,  $a$ , should be increased by 10mm for prestressing bars and 15mm for prestressing wires or strands.

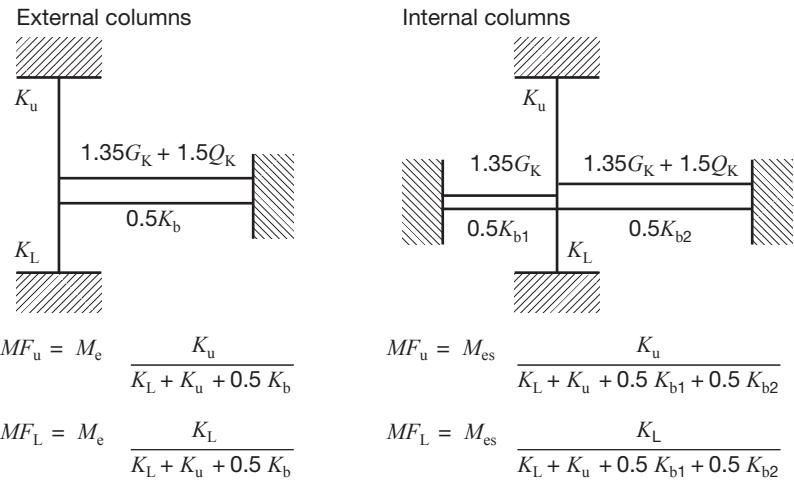
### 5.5.3 Axial loads and moments - columns

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

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

- The axial loads may generally be obtained by increasing by 10% the loads obtained on the assumption that beams and slabs are simply supported. A higher increase may be required where adjacent spans and/or the loadings on them are grossly dissimilar.
- The moments in the columns may be obtained using the subframes shown in Figure 5.18 subject to the minimum design moments above.

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



**Note**

$M_e$  = 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

**Fig 5.18** Sub-frames for column moments

#### 5.5.4 Axial loads and moments - slender columns

##### 5.5.4.1 General

When a column is found to be slender in accordance with the rules set out in Section 5.5.2.1, it is necessary to make allowance in the design for the possible effects of the ultimate deflection of the column. The possible design conditions are the design ultimate axial load ( $N_{Ed}$ ) combined with the most critical of:

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

The moments at around mid-height are given in Table 5.16.

**Table 5.16 Bending moments at around mid-height in slender columns**

<b>Case</b>	<b>Moments about the y-axis</b>	<b>Moments about the z-axis</b>
1	$M_{yi} + N_{Ed}(e_{2z} + e_a)$	$M_{zi} + N_{Ed}e_a$
2	$M_{yi} + N_{Ed}e_a$	$M_{zi} + N_{Ed}(e_{2y} + e_a)$

## Notes

- a The moments for each case should be considered to act simultaneously, i.e. biaxially, except that separate checks about each axis are permitted when the following conditions are satisfied:

$$\left[ \begin{array}{l} \left( \frac{l_y}{b} \right) \leq 2.0 \text{ AND } \left( \frac{l_{oz}}{h} \right) \leq 2.0 \\ \left( \frac{l_{oy}}{h} \right) \leq 2.0 \end{array} \right] \text{ AND } \left[ \frac{hM_{zi}}{bM_{yi}} \geq 5 \text{ OR } \frac{hM_{zi}}{bM_{yi}} \leq 0.2 \right]$$

If these conditions are satisfied then design may be carried out only for the following two cases:

$M_{yi} + N_{Ed}(e_{2z} + e_a)$	about the y axis
$M_{zi} + N_{Ed}(e_{2y} + e_a)$	about the z axis

- b Where the dominant bending is about the minor axis of the section, it is only necessary to check the reinforcement for bending about the minor axis.
- c In the above,  $M_{yi}$  and  $M_{zi}$  are, respectively, the first order moments at around mid-height of the columns about the y and z axes (see Section 5.5.4.2 and Figure 5.19).
- d  $e_{2y}$  and  $e_{2z}$  are, respectively, the ultimate deflections calculated in the y- and z- directions (see Section 5.5.4.3 and Figure 5.16).
- e  $e_a$  is an eccentricity to allow for accidental misalignment of the column and is given by:  $e_a = \theta_i l_o / 2$  where  $l_o$  is the effective length of the column and  $\theta_i$  is a notional inclination given in Table 3.1.
- f  $l_{oy}$  and  $l_{oz}$  are, respectively, the effective lengths of the column in relation to bending about the y- and z- axes.
- g  $N_{Ed}$  is the design ultimate axial load.

#### 5.5.4.2 Calculation of first-order moments around mid height

The first-order moment ( $M_{yi}$  or  $M_{zi}$ ) at about mid-height of a braced column should be the greater of:  
either  $0.6 M_2 + 0.4 M_1$   
or  $0.4 M_2$

Where:  $M_1$  is the numerically smaller end moment from first order analysis

$M_2$  is the numerically larger end moment from first order analysis

Algebraically,  $M_1$  and  $M_2$  will commonly have opposite signs.

#### 5.5.4.3 Calculation of the ultimate deflection

The ultimate deflection in mm is given by:

$$e_2 = f_{yk} \frac{l_0^2}{d} 10^{-6} \text{mm}$$

Where:  $f_{yk}$  is the characteristic strength of the reinforcement in MPa

$l_0$  is the effective length of the column in direction considered

$d$  is the effective depth of section in direction considered.

The expression for  $e_2$  has been derived assuming  $f_{ck} = 30$  MPa. EC2<sup>1</sup> requires an increase in the second order eccentricity for higher strength concretes to allow for creep. In order to take this into account  $e_2$  for an edge or corner column should be increased by:

- 10% for  $f_{ck} = 35$  MPa and
- 20% for  $f_{ck} = 40$  MPa.

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

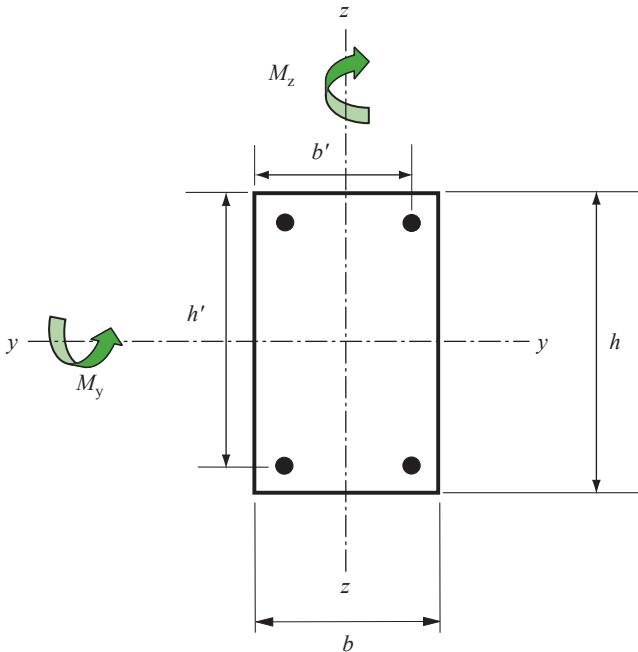
$$K = \frac{(N_u - N)}{(N_u - N_{bal})} \leq 1 \text{ alternatively, } K \text{ may conservatively be taken as 1.}$$

Where:  $N_u$  is the ultimate axial load of column =  $0.567 f_{ck} A_c + 0.87 A_s f_{yk}$

$N_{bal}$  is the balanced load i.e. the axial load that when applied to a section maximizes the ultimate moment capacity. For a symmetrically reinforced section,

$$N_{bal} = 0.267 f_{ck} A_c.$$

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



**Fig 5.19** Biaxial bending in columns

### 5.5.5 Section design

Sections subject to uniaxial bending should normally be designed using the charts in Appendix C. When biaxial bending occurs, a symmetrically reinforced rectangular column section may be designed using the charts in Appendix C for the moments given in Table 5.17.

**Table 5.17 Design moments for biaxial bending**

		<b>y - axis</b>	<b>z - axis</b>
$\frac{hM_{zi}}{bM_{yi}} \geq 5$ OR $\frac{hM_{zi}}{bM_{yi}} \leq 0.2$	consider both: (i) (ii)	0 $M_y$	$M_z$ 0
All other cases:	If $\frac{M_z h'}{M_y b'} \leq 1$	$M_y + \frac{\beta h' M_z}{b'}$	0
	If $\frac{M_z h'}{M_y b'} \geq 1$	0	$M_z + \frac{\beta b' M_y}{h'}$

#### Notes

- a b' and h' are the effective depths (see Figure 5.19).
- b  $\beta$  is obtained from Table 5.18.

**Table 5.18 Coefficients for biaxial bending**

$\frac{N}{bhf_{ck}}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7
$\beta$	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3

**Notes**

- a N is the design ultimate axial load in Newtons.
- b b and h are in mm (see Figure 5.16).

### 5.5.6 Reinforcement

Minimum area of reinforcement is given by the greater of:

$$\frac{0.12N}{f_yk} \text{ or } 0.2\% \text{ of the gross cross-sectional area of the concrete.}$$

Longitudinal bars should not be less than 12mm diameter.

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

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

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

- 20 times the diameter of the smallest compression bar
- the least dimension of the column
- 400mm.

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

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

## 5.6 Walls

### 5.6.1 Introduction

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

The general procedure to be adopted is as follows:

- i) 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. Confirm these walls are sufficient for global second order effects to be ignored (see Section 4.8.5)
- ii) check the slenderness of the walls within every storey height
- iii) check that the section complies with the requirements for fire resistance

- iv) check sufficient stiffness of shear walls for second order effects
- v) check that cover and concrete comply with durability requirements
- vi) calculate axial loads and moments according to Section 5.6.3
- vii) design section and reinforcement.

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

## 5.6.2 Slenderness, fire resistance and durability

### 5.6.2.1 Slenderness

The slenderness of a wall is defined by the ratio of the effective height to the thickness of the wall,  $h$ .

$$\frac{l_{oz}}{h} > 4.38 \left( 1.7 - \frac{M_{1z}}{M_{2z}} \right) \xi$$

Where:  $M_{1z}$  and  $M_{2z}$  are the numerically smaller and larger end moments respectively  
 $N_{Ed}$  is the effective axial load per metre length of the wall calculated according to Section 5.6.4 below.

$$\xi = 0.69 \sqrt{\frac{(1 + 2\omega)(1000hf_{ck})}{N_{Ed}}} \geq 1.0$$

$$\omega = 1.53 \frac{A_s f_{yk}}{1000 h f_{ck}}$$

$A_s$  is the area of steel ( $\text{mm}^2$  per m).

(When  $A_s$  is not known  $\omega_{\min}$  may be taken as  $0.003 \frac{f_{yk}}{f_{ck}}$ )

It should be noted that, for most walls in framed structures,  $M_{1z}$  will have the opposite sign to  $M_{2z}$ .

The effective height may be obtained by multiplying the clear height between floors by the factor obtained from Table 5.19.

**Table 5.19 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

Notes

- a 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.
- b 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.
- c Condition 3 Wall connected to members that do not provide more than nominal restraint to rotation.

### 5.6.2.2 Fire resistance

The minimum dimensions and axis distances should be obtained from Table 5.20. It should be noted that for the higher fire ratings the thickness of walls may be controlled by the fire requirement. It may be possible to achieve thinner walls by using the simplified calculation methods given in EC2<sup>2</sup>.

**Table 5.20 Fire resistance requirements for walls**

Standard fire resistance (R) in minutes	Minimum dimensions (mm) Wall thickness/axis distance, <i>a</i>	
	wall exposed on one side	wall exposed on two sides
R 30	120/10 <sup>b</sup>	120/10 <sup>b</sup>
R 60	130/10 <sup>b</sup>	140/10 <sup>b</sup>
R 90	140/25 <sup>b</sup>	170/25 <sup>b</sup>
R 120	160/35 <sup>b</sup>	220/35 <sup>b</sup>
R 180	210/50	270/55
R 240	270/60	350/60

**Notes**

- a** The axis distance, *a*, is measured from the surface of the concrete to the centre of the main reinforcing bars.
- b** For practical purposes the axis distance should be such that the minimum cover is 20mm.
- c** The axis distance, *a*, should be increased by 10mm for prestressing bars and 15mm for prestressing wires or strands.

### 5.6.2.3 Durability

The requirements for durability in any given environment are:

- i) an upper limit to the water/cement ratio
- ii) a lower limit to the cement content
- iii) a lower limit of the cover to the reinforcement
- iv) good compaction
- v) adequate curing, and
- vi) good detailing.

Values for i), ii) and iii) which, in combination, will give adequate durability are given in Appendix B for various environments.

### 5.6.3 Axial loads and moments

#### 5.6.3.1 In-plane bending

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

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

#### 5.6.3.2 Bending at right-angles to the walls

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

#### 5.6.3.3 Slender walls

Where the slenderness ratio of a braced wall exceeds the limit given in Figure 5.17, the bending moment at right-angles to the wall should be taken as the greater of:

$$M_2 \text{ or } M_{zi} + N_{Ed}(e_{2z} + e_a)$$

$M_{zi}$  should be calculated in accordance with Section 5.5.4.2 and  $e_{2z}$  in accordance with Section 5.5.4.3.

#### 5.6.4 Section design

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

$$f_t = \frac{N}{Lh} \pm \frac{6M}{hL^2} \text{ MPa}$$

Where:  $N$  is the design ultimate axial load in Newtons  
 $M$  is the ultimate in-plane moment in Nmm  
 $L$  is the length of wall in mm  
 $h$  is the width of the wall in mm.

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

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

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

- i) for compressive loads:

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

Where:  $f_{ck}$  is the characteristic concrete cylinder strength in MPa

$f_{yk}$  is the characteristic strength of reinforcement in MPa

$A_{sc}$  is the area of reinforcement in  $\text{mm}^2$  per  $\text{mm}$  length of wall.

- ii) for tensile loads:

The total area of tension reinforcement should be calculated from the expression:

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

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

#### 5.6.4.2 Intersecting walls

Where the composite action of intersecting walls to form a core is assumed the interface shear should be checked in accordance with EC2, Clause 6.2.4<sup>1</sup>.

#### 5.6.5 Reinforcement

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

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

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

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

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

#### 5.6.6 Openings in shear and core walls

Door and service openings in shear walls introduce weaknesses that are not confined merely to the consequential reduction in cross-section. Stress concentrations are developed at the corners, and adequate reinforcement needs to be provided to cater for these concentrations. This reinforcement

should take the form of diagonal bars positioned at the corners of the openings. The reinforcement will generally be adequate if it is designed to resist a tensile force equal to twice the horizontal shear force in the vertical components of the wall, but should not be less than two 16mm diameter bars across each corner of the opening. Alternatively strut and tie methods in accordance with EC2<sup>1</sup> may be used.

## 5.7 Staircases

### 5.7.1 Introduction

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

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

### 5.7.2 Fire resistance and durability

#### 5.7.2.1 Fire resistance

The requirements for fire resistance should be as for slabs (see Section 5.2.2.1).

#### 5.7.2.2 Durability

The requirements for durability in any given environment are:

- i) an upper limit to the water/cement ratio
- ii) a lower limit to the cement content
- iii) a lower limit of the cover to the reinforcement
- iv) good compaction
- v) adequate curing, and
- vi) good detailing.

Values for i), ii) and iii) which, in combination, will give adequate durability are given in Appendix B for various environments.

### 5.7.3 Bending moments and shear forces

Staircase slabs and landings should be designed to support the most unfavourable arrangements of design loads. Where a span is adjacent to a cantilever of length exceeding one-third of the span of the slab, the case should be considered of maximum load on the cantilever and minimum load on the adjacent span.

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

### 5.7.4 Effective spans

#### 5.7.4.1 Stairs spanning between beams or walls

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

#### 5.7.4.2 Stairs spanning between landing slabs

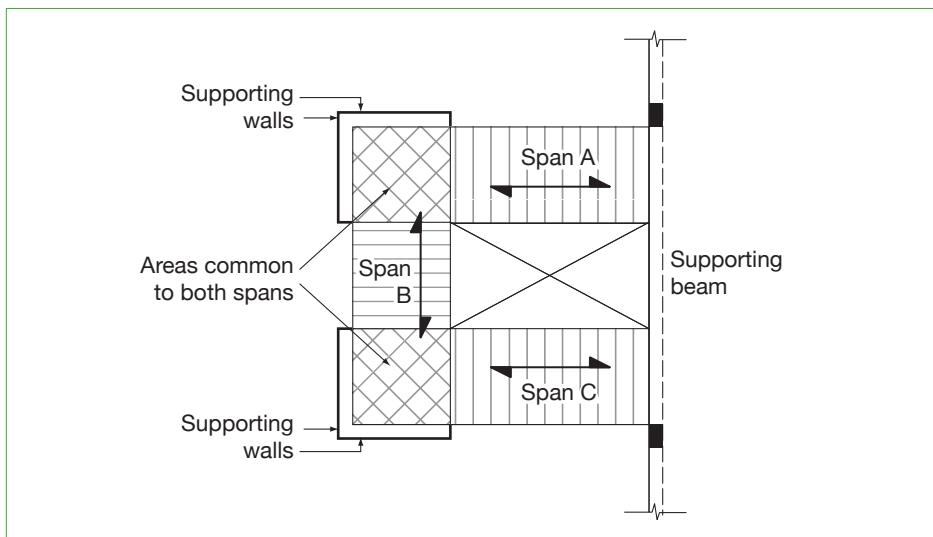
The effective span is the distance between centre-lines of supporting landing slabs, or the distance between the edges of the supporting slabs plus 1.8m, whichever is the smaller.

#### 5.7.4.3 Stairs with open wells

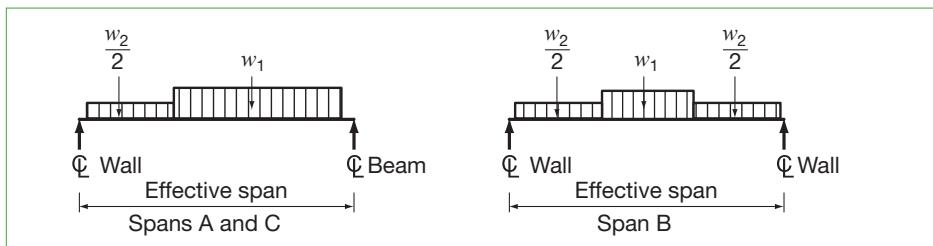
The effective span and loads on each span are as indicated in Figure 5.20 and Figure 5.21. The arrangement of flight supports shown in Figure 5.20 and Figure 5.21 is a special case where vertical support is provided at the ends of all flights. Where this condition does not occur, the stair flights should be designed for the full landing loads and the effective spans should be in accordance with Sections 5.7.4.1 and 5.7.4.2.

#### 5.7.5 Span/effective depth ratios

The span/effective depth should not exceed the appropriate value from Table 5.21.



**Fig 5.20** Stairs with open wells



**Fig 5.21** Loading diagram

**Table 5.21 Span/effective depth ratios for stairs**

Location	$\frac{A_s}{bd} \geq 1.5\%$	$\frac{A_s}{bd} = 0.5\%$	$\frac{A_s}{bd} \leq 0.35\%$
Simply supported stairs	14	20	30
End span of continuous stairs	18	26	39
Interior span	20	30	45
Cantilever	6	8	12

**Notes**

- a Values may be interpolated.
- b For flanged sections where the ratio of the flange to the rib width exceeds 3, the values should be multiplied by 0.8.
- c The above assumes  $f_{yk} = 500\text{MPa}$ . If other values of  $f_{yk}$  are used then multiply the above by  $500/f_{yk}$ .
- d  $\frac{A_s}{bd}$  is calculated at the location of maximum span moment.

### 5.7.6 Section design

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

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

For stair landings, or beam strips supporting stair flights, the shear around columns should be checked in a similar manner to the shear around columns in solid flat slab construction.

## 5.8 Design of non-suspended ground floor slabs

Non-suspended ground slabs are generally designed on an empirical basis. Successful design requires attention to practical details. Thermal and moisture movements tend to produce the most critical stresses and cracking particularly when the concrete is still green. Careful planning of joints and provision of suitable reinforcement are essential. Useful guidance can be obtained from reference 20. The adoption of ground bearing floor slabs needs to be considered in conjunction with the geotechnical investigation for the site. The design of these slabs should take into account possible differential movement between the floor slab and the rest of the structure.

The long strip method recommended 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  $50\text{m}^2$  in area with the longer dimension of the bay limited to 10m. The slab thickness and reinforcement can be obtained from reference 20.

## 5.9 Guidance for the design of basement walls

### 5.9.1 General

This subsection describes the design of basement walls that form part of a reinforced concrete structure. The general procedure to be adopted is as follows:

- i) Consideration should be given to potential hydrostatic forces on both the walls and floors. In certain circumstances it may be necessary to consider flotation effects both for the temporary and permanent works.
- ii) Establish the requirements for the internal environment and, if the structure is to be water resisting, reference should be made to EC2, Part 3<sup>7</sup>, the CIRIA guide on waterproof basements<sup>21</sup> and BS 8102<sup>22</sup>.
- iii) Make the walls at least 250mm thick and ensure that they comply with the slenderness provisions in Section 5.6.2.1.
- iv) Check that walls comply with the requirements for fire resistance in Section 5.6.2.2.
- v) Check that walls comply with the requirements for durability in Section 5.6.2.3.

### 5.9.2 Bending moments and shear forces

The maximum values of the bending moments and shear forces at any section should be obtained by elastic analysis using the appropriate ultimate loads noted in Section 3.1. A minimum vertical surcharge of 10kN/m<sup>2</sup> 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.

### 5.9.3 Section design

The sections should be designed in accordance with Section 5.6.4.

### 5.9.4 Foundation

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

### 5.9.5 Reinforcement

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

## 5.10 Foundations

### 5.10.1 Introduction

The type of foundation, the sizes and the provisional formation levels depend on the results of ground investigation. Geotechnical design of foundations is beyond the scope of this *Manual*, and reference should be made to EC7<sup>6</sup>. EC2<sup>1</sup> deals with the strength design of foundations.

The dead, imposed and wind load factors to be used for the proportioning of foundations should be obtained from EC7<sup>6</sup> (see also Section 3.2.1). The factored loads are, however, required for determining the depths of foundation members and for the design of any reinforcement.

Concrete should be specified using BS 8500<sup>3</sup>.

Care is required in the choice of concrete where freeze thaw conditions apply and /or aggressive ground conditions exist. For exposure conditions XF3 and XF4 freeze thaw aggregates should be specified. BRE Special Digest 1<sup>23</sup> gives guidance for where aggressive ground conditions exist.

The general procedure to be adopted is as follows:

- i) Evaluate results of ground investigation and decide whether spread or piled foundations are to be used.
- ii) Examine existing and future levels around the structure, and taking into account the bearing strata and ground water levels, determine the provisional formation levels.
- iii) Calculate the loads and moments, if any, on the individual foundations using the partial factors in Table 3.2.
- iv) Recalculate the loads and moments, if any, on the individual foundations without the partial factors in Table 3.2, using the imposed loading reduction as appropriate.
- v) 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.
- vi) Calculate the depth required for each foundation member and the reinforcement, if any, using the factored loads.

### 5.10.2 Durability and cover

Foundations exposed to cyclic wet and dry conditions, but not in aggressive conditions or exposed to frost, should be considered as being in exposure Class XC4 (see Appendix B). The concrete strength class for reinforced bases and pile caps should therefore be not less than C28/35.

For unreinforced bases C16/20 may be used, subject to a minimum cement content of 220kg/m<sup>3</sup>.

Where sulphates are present in significant concentrations in the soil and/or the groundwater, the recommendations of BS 8500 – 1<sup>3</sup> and BRE Special Digest No. 1<sup>23</sup> should be followed.

### 5.10.3 Types of foundation

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

- Spread/Pad footing – A square or rectangular footing supporting a single column.
- Strip footing – A long footing supporting a continuous wall.
- Combined footing – A footing supporting two or more columns.
- Balanced footing – A footing supporting two columns, one of which lies at or near one end.
- Raft – A foundation supporting a number of columns or loadbearing walls so as to transmit approximately uniform loading to the soil.
- Pile cap – A foundation in the form of a pad, strip, combined or balanced footing in which the forces are transmitted to the soil through a system of piles.

#### 5.10.4 Plan area of foundations

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

- All forces are transmitted to the soil without exceeding the allowable bearing pressure.
- 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.
- 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.
- All parts of a footing in contact with the soil should be included in the assessment of contact pressure.
- It is preferable to maintain a reasonably similar pressure under all foundations to avoid significant differential settlement.

#### 5.10.5 Design of spread footings

##### 5.10.5.1 Axially loaded unreinforced spread footings

The ratio of the depth of the footing  $h_f$  to the projection from the column face  $a$  should be not less than that given in Table 5.22 for different values of unfactored pressures,  $\sigma$  in kN/m<sup>2</sup>.

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

**Table 5.22 Depth/projection ratios for unreinforced footings**

Unfactored ground pressure $\sigma$ (kN/m <sup>2</sup> )	$\frac{h_f}{a}$			
	C20/25	C25/30	C30/37	C35/45
≤ 200	1.2	1.1	1.1	1.0
300	1.5	1.4	1.3	1.2
400	1.7	1.6	1.5	1.4

##### 5.10.5.2 Axially loaded reinforced spread footings

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

- i) Determine the depth of the footing from the ratios of the effective depth  $d$  to the projection from the column face  $a$ , given in Table 5.23 for different values of unfactored ground pressures  $\sigma$ . The effective depth  $d$  should not in any case be less than 300mm.

- ii) Check that the shear at the column perimeter does not exceed:

$$v_{Ed} = \frac{1000V_{\text{eff}}}{u_c d} \text{ MPa} \leq 0.2 \left(1 - \frac{f_{ck}}{250}\right) f_{ck}$$

Where:  $V_{\text{eff}}$  is the effective shear force in kN (see Sections 5.2.3.4 and 5.2.4.2)  
 $d$  is the average of the effective depth of the tension reinforcement in both directions, and  
 $u_c$  is the column perimeter in mm.

- iii) With the chosen depth (revised according to stage ii), if necessary) enter Table 5.23 and obtain the corresponding reinforcement percentage. Table 5.23 has been derived from a shear and flexure check across full width at face of column with an upper limit on the reinforcement percentage of 0.25.

**Table 5.23 Reinforcement percentages, depth/projection ratios and unfactored ground pressures for reinforced footings for  $f_{ck} = 25\text{MPa}$**

Unfactored ground pressure, $\sigma$ ( $\text{kN/m}^2$ )	$d/a$									
	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.70	$\geq 0.80$
50	0.15	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13
100			0.16	0.13	0.13	0.13	0.13	0.13	0.13	0.13
150					0.17	0.13	0.13	0.13	0.13	0.13
200						0.22	0.13	0.13	0.13	0.13
250								0.16	0.13	0.13
300									0.13	0.13

*Note*

The shaded areas indicate combinations of  $\sigma$  and  $d/a$  that should not be used.

### 5.10.5.3 Eccentrically loaded footings

The design of eccentrically loaded footings proceeds as follows:

- Determine initial depth of footing from Table 5.23 using maximum value of unfactored ground pressure.
- Check punching shear according to Sections 5.2.3.4 and 5.2.4.2.
- Check face shear according to stage ii) in Section 5.10.5.2.
- Increase the depth if necessary to avoid shear reinforcement.
- With the chosen depth (revised according to stage iv), if necessary) enter Table 5.23 to obtain the reinforcement percentage using maximum values of unfactored ground pressure.

## 5.10.6 Design of other footings

### 5.10.6.1 Strip footings

Strip footings should be designed as spread 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 Section 5.10.5.2.

### 5.10.6.2 Combined footings and balanced footings

Combined footings and balanced footings should be designed as reinforced spread 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 Sections 5.2.3.4 and 5.2.4.2. Bending moments should additionally be checked at the point of zero shear between the two columns. Reinforcement should be provided in top and bottom faces and may be curtailed in accordance with the detailing rules in Section 5.12.
- Where a balanced footing consists of two spread footings joined by a beam, the beam may be designed in accordance with Section 5.4.
- Steps in the top or bottom surface may be introduced if necessary provided that they are taken into account in the design.

## 5.10.7 Reinforcement in footings

Where reinforcement is required it should be provided in two generally orthogonal directions. The areas in each direction should not be less than  $0.0015bh$  for reinforcement with  $f_y = 500\text{MPa}$  where  $b$  and  $h$  are the breadth and overall depth in mm, respectively. All reinforcement should extend the full length of the footing.

If  $l_x > 1.5(c_x + 3d)$ , at least two-thirds of the reinforcement parallel to  $l_y$  should be concentrated in a band width  $(c_x + 3d)$  centred at the column, where  $d$  is the effective depth,  $l_x$  and  $c_x$  are the footing and column dimensions in the x-direction and  $l_y$  and  $c_y$  are the footing and column dimensions in the y-direction (see Figure 5.22). The same applies in the transverse direction with suffixes x and y transposed.

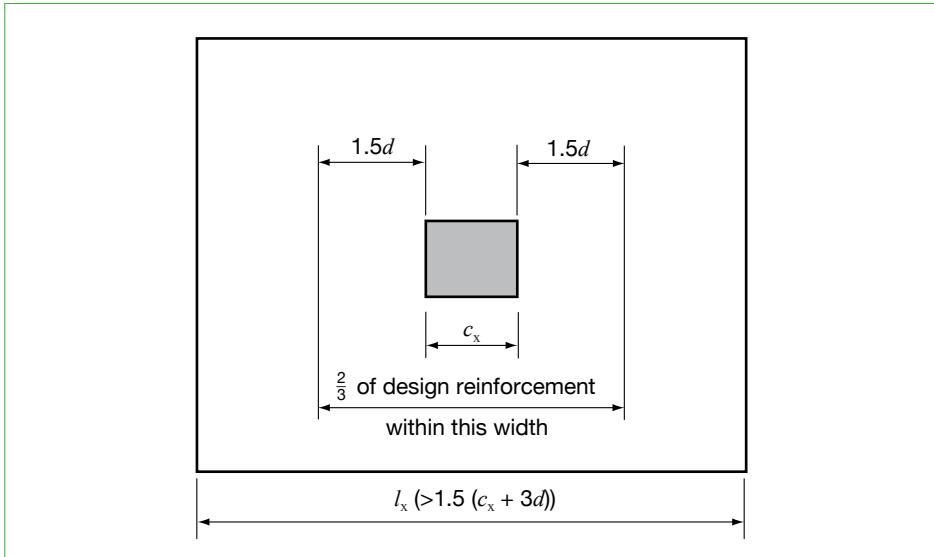
Reinforcement should be anchored each side of all critical sections for bending. It is usually possible to achieve this with straight bars.

The spacing between centres of reinforcement should not exceed 200mm for bars with  $f_y = 500\text{MPa}$ . 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  $90^\circ$  bend tied to the bottom reinforcement, or in the case of an unreinforced footing spaced 75mm off the blinding.

## 5.10.8 Design of rafts

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



**Fig 5.22** Layout of reinforcement in a pad footing

A distribution of ground bearing pressure has to be determined that:

- satisfies equilibrium by matching the column loads
- satisfies compatibility by matching the relative stiffness of raft and soil
- allows for the concentration of loads by slabs or beams continuous over supports, and
- 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 many cases, however, a realistic distribution cannot be determined by simple methods, and a more complex analysis is required.

### 5.10.9 Design of pile caps

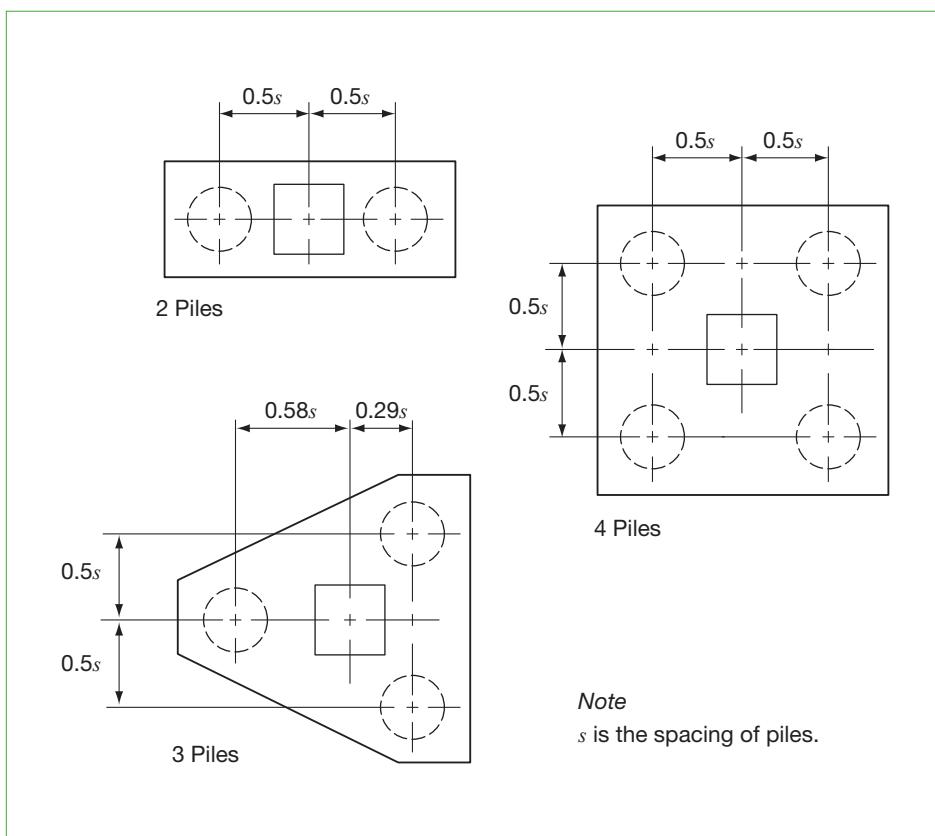
The design of pile caps should be carried out in accordance with the following general principles:

- The spacing of piles should generally be three times the pile diameter.
- The piles should be grouped symmetrically under the loads.
- The load carried by each pile is equal to  $N/\text{(no. of piles)}$ . When a moment is transmitted to the pile cap the loads on the piles should be calculated to satisfy equilibrium.
- Pile caps should extend at least 150mm beyond the theoretical circumference of the piles.
- 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:

- i) Using the unfactored loads and moments calculate the number of piles required under each column.
- ii) Proportion the pile caps on plan in accordance with the above general principles. Typical arrangements are shown in Figure 5.23 where  $s$  is the spacing of the piles.
- iii) 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.
- iv) Check the face shear as for reinforced spread footings, using factored loads, and modify the depth if necessary.
- v) Calculate the bending moments and the reinforcement in the pile caps using the factored loads.

As an alternative to iv) and v) use the 'strut and tie' methods to determine capacity and reinforcement for the pile cap.



**Fig 5.23** Typical arrangement of pile caps

### 5.10.10 Reinforcement in pile caps

All pile caps should generally be reinforced in two orthogonal directions on the top and bottom faces. Where bars with  $f_y = 500\text{MPa}$  are used the amount of reinforcement should not be less than  $0.0015bh$  in each direction.

The bending moments and the reinforcement should be calculated on critical sections at the column faces, assuming that the pile loads are concentrated at the pile centres. This reinforcement should be continued past the piles and bent up vertically to provide full anchorage past the centreline of each pile. In addition, fully lapped, circumferential horizontal reinforcement consisting of bars not less than 12mm in diameter at a spacing not more than 250mm, should be provided.

## 5.11 Robustness

### 5.11.1 General

If the recommendations of this *Manual* for an *in-situ* structure have been followed, a robust structure will have been designed, subject to the reinforcement being properly detailed. However the requirements of the Building Regulations (Approved Document A)<sup>24</sup> should be checked.

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:

- i) peripheral ties
- ii) internal ties
- iii) external column or wall ties
- iv) vertical ties.

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

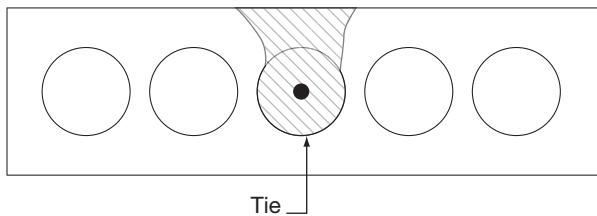
Reinforcement considered as part of the above ties should have full tension laps throughout so as to be effectively continuous. For the purpose of checking the adequacy of the ties, this reinforcement may be assumed to be acting at its characteristic strength when resisting the forces stated below, and no other forces need to be considered in this check. The minimum dimension of any *in-situ* concrete section in which tie bars are provided should not be less than the sum of the bar size (or twice the bar size at laps) plus twice the maximum aggregate size plus 10mm.

Horizontal ties, i.e. i), ii) and iii) above, should be present at each floor level and at roof level.

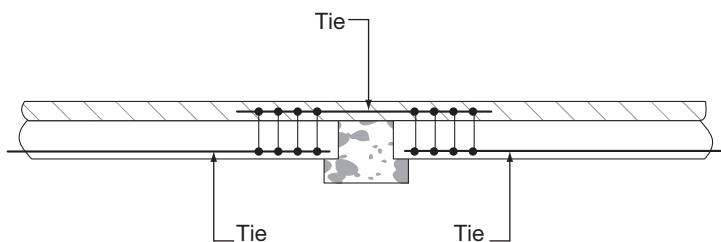
In precast concrete construction ties may be provided wholly within *in-situ* concrete toppings or connections partly within *in-situ* concrete and partly within precast members or wholly within precast members but they should be effectively continuous. In addition the tie should also satisfy one of the following conditions:

- A bar or tendon in a precast member lapped with a bar in *in-situ* connecting concrete bounded on two opposite sides by rough faces of the same precast member (see Figure 5.24).
- A bar or tendon in a precast concrete member lapped with a bar in *in-situ* topping or connecting concrete anchored to the precast member by enclosing links. The ultimate

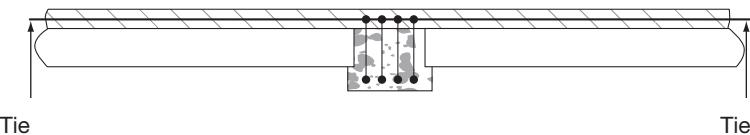
- tensile resistance of the links should be not less than the ultimate tension in the tie (see Figure 5.25).
- Bars projecting from the ends of precast members joined by one of the following methods provided that the full strength of the bar can be shown to exist through the joint:
    - lapping of bars
    - grouting into an aperture
    - overlapping reinforcement loops
    - sleeving
    - threaded reinforcement.
  - Bars lapped within *in-situ* topping or connecting concrete to form a continuous reinforcement with projecting links from the support of the precast floor or roof members to anchor such support to the topping or connecting concrete (see Figure 5.26).



**Fig 5.24** Continuity of ties: bars in precast member lapped with bar in *in-situ* concrete



**Fig 5.25** Continuity of ties: anchorage by enclosing links



**Fig 5.26** Continuity of ties: bars lapped within *in-situ* concrete

In addition precast roof, floor and stair members not containing ties should be anchored back to the parts of the structure containing the ties. The capacity of this anchorage should be capable of carrying the dead weight of the member.

### 5.11.2 Tie forces and arrangements

Forces to be resisted by horizontal ties are derived from a ‘tie force coefficient’:

$$F_t = (20 + 4n) \text{ kN} \quad \text{for } n \leq 10, \text{ or}$$

$$F_t = 60 \text{ kN} \quad \text{for } n > 10$$

Where:  $n$  is the number of storeys.

#### *Peripheral ties*

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

The calculation of  $F_t$  above, for peripheral ties, is based on previous UK practice. While this contradicts expression 9.15 of BSEN 1992-1-1, it is recommended that the guidance given in this Manual is adopted — as it is considered to reflect the UK Committee’s intentions with respect to tie forces.

#### *Internal ties*

Internal ties should be present in two directions approximately at right-angles to each other. Provided that the floor spans do not exceed 5m and the total characteristic dead and imposed load does not exceed  $7.5\text{kN/m}^2$ , the ties in each direction should be capable of resisting a tie force of  $1.0F_t$  kN per metre width at each storey level. If the spans exceed 5m and/or the total load exceeds  $7.5\text{kN/m}^2$ , 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.

#### *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 float-to-ceiling height does not exceed 2.5m, the tie force for each column and for each metre length of wall is  $1.0F_t$  per storey. For floor-to-ceiling heights greater than 2.5m, the tie forces should be increased pro rata, up to a maximum of  $2.0F_t$ . The tie force should in no case be assumed less than 3% of the total design ultimate load carried by the column or wall. This reinforcement should be fully anchored in both vertical and horizontal elements.

#### *Vertical ties*

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

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

$$\gamma_G = 1.0$$

$$\gamma_Q = 0.33$$

$$\gamma_c = 1.3, \text{ and}$$

$$\gamma_s = 1.0.$$

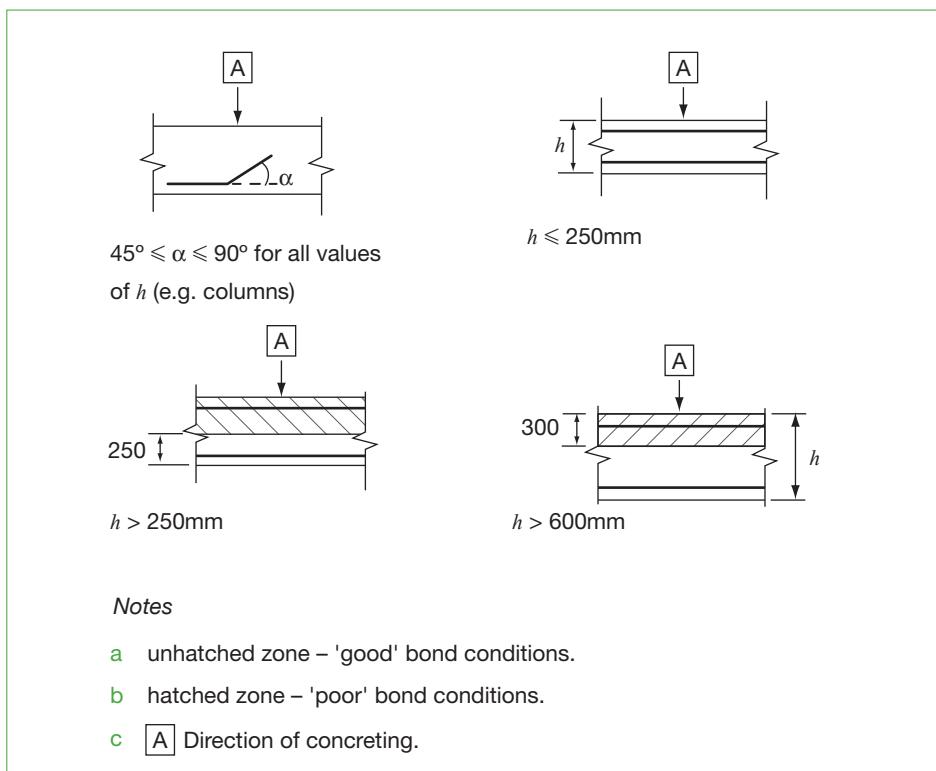
## 5.12 Detailing

### 5.12.1 General

Certain aspects of reinforcement detailing may influence the design. The most common of these are outlined below. Additional rules for bars exceeding 40mm in diameter are given in Section 5.12.5.

### 5.12.2 Bond conditions

The bond conditions affect the anchorage and lap lengths. Good and poor bond conditions are illustrated in Figure 5.27.



**Fig 5.27** Definition of bond conditions in sectional elevations

### 5.12.3 Anchorage and lap lengths

Anchorage and lap lengths should be obtained from Table 5.24 to Table 5.27 for bars and welded mesh fabric (see Figure 5.28). The values apply to reinforcing steel as specified in BS 4449<sup>25</sup> and BS 4483<sup>26</sup>.

The clear spacing between two lapped bars should be in accordance with Figure 5.29 (see also Section 5.12.5).

It should be noted that where the distance between lapped bars is greater than 50mm or  $4\phi$  the lap length should be increased by the amount by which the clear space exceeds 50mm or  $4\phi$ .

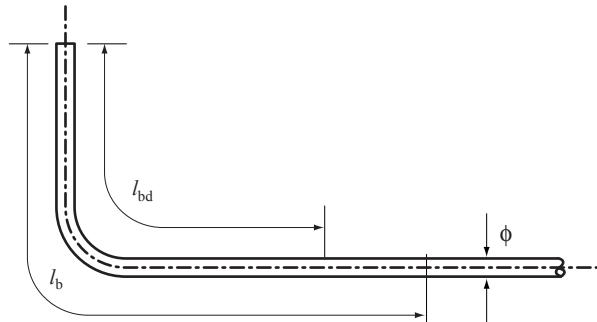


Fig 5.28 Design anchorage length,  $l_{bd}$

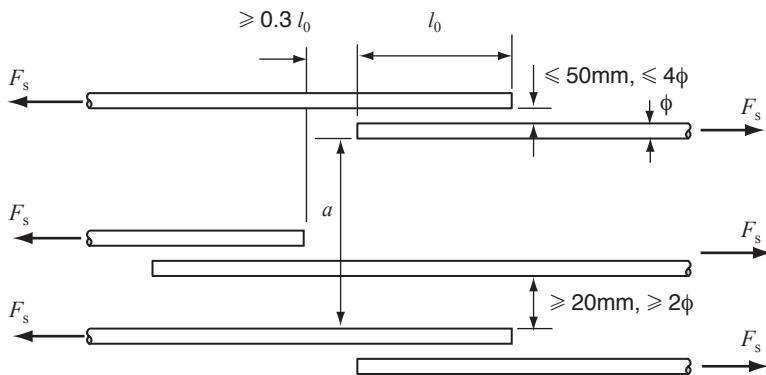


Fig 5.29 Adjacent laps

**Table 5.24 Typical values of anchorage and lap lengths for slabs**

	<b>Bond conditions</b>	<b>Length in bar diameters</b>			
		$f_{ck}/f_{cu} = 25/30$	$f_{ck}/f_{cu} = 28/35$	$f_{ck}/f_{cu} = 30/37$	$f_{ck}/f_{cu} = 32/40$
Full tension and compression anchorage length, $l_{bd}$	good	40	37	36	34
	poor	58	53	51	49
Full tension and compression lap length, $l_0$	good	46	43	42	39
	poor	66	61	59	56

**Notes**

- a** The following is assumed:
  - bar size is not greater than 32mm. If >32 then the anchorage and lap lengths should be divided by a factor  $(132 - \text{bar size})/100$
  - normal cover exists
  - no confinement by transverse pressure
  - no confinement by transverse reinforcement
  - not more than 33% of the bars are lapped at one place.
- b** Lap lengths provided (for nominal bars, etc.) should not be less than 15 times the bar size or 200mm, whichever is greater.

**Table 5.25 Typical values of anchorage and lap lengths for beams**

	<b>Bond conditions</b>	<b>Length in bar diameters</b>			
		$f_{ck}/f_{cu} = 25/30$	$f_{ck}/f_{cu} = 28/35$	$f_{ck}/f_{cu} = 30/37$	$f_{ck}/f_{cu} = 32/40$
Full tension and compression anchorage length, $l_{b,rqd}$	good	36	34	32	31
	poor	48	45	43	41
Full tension and compression lap length, $l_0$	good	42	39	37	35
	poor	56	52	49	47

**Note**

The following is assumed:

- bar size is not greater than 32mm. If >32 then the anchorage and lap lengths should be divided by a factor  $(132 - \text{bar size})/100$
- normal cover exists
- no confinement by transverse pressure
- confinement by links – factor = 0.9
- not more than 33% of the bars are lapped at one place.

**Table 5.26 Typical values of anchorage and lap lengths for columns**

	<b>Bond conditions</b>	<b>Length in bar diameters</b>			
		$f_{ck}/f_{cu} = 25/30$	$f_{ck}/f_{cu} = 28/35$	$f_{ck}/f_{cu} = 30/37$	$f_{ck}/f_{cu} = 32/40$
Full tension and compression anchorage length, $l_{b,rqd}$	good	36	34	32	31
	poor	48	45	43	41
Full tension and compression lap length, $l_0$	good	54	51	48	46
	poor	73	67	64	62

**Note**  
The following is assumed:

- bar size is not greater than 32mm. If >32 then the anchorage and lap lengths should be divided by a factor  $(132 - \text{bar size})/100$
- normal cover exists
- no confinement by transverse pressure
- confinement by links – factor = 0.9
- more than 50% of the bars are lapped at one place – factor = 1.5.

**Table 5.27 Typical values of anchorage and lap lengths for walls**

	<b>Bond conditions</b>	<b>Length in bar diameters</b>			
		$f_{ck}/f_{cu} = 25/30$	$f_{ck}/f_{cu} = 28/35$	$f_{ck}/f_{cu} = 30/37$	$f_{ck}/f_{cu} = 32/40$
Full tension and compression anchorage length, $l_{bd}$	good	40	37	36	34
	poor	54	50	48	46
Full tension and compression lap length, $l_0$	good	61	56	54	51
	poor	81	75	71	68

**Notes**

- a The following is assumed:
  - bar size is not greater than 32mm. If >32 then the anchorage and lap lengths should be divided by a factor  $(132 - \text{bar size})/100$
  - normal cover exists
  - no confinement by transverse pressure
  - more than 50% of the bars are lapped at one place – factor = 1.5.
- b Lap lengths provided (for nominal bars, etc.) should not be less than 15 times the bar size or 200mm, whichever is greater.

## 5.12.4 Transverse reinforcement

### Anchorage zones

Links should be provided and placed in the anchorage zones of beams with a minimum area of 25% of the area of a single anchored bar.

Transverse reinforcement should be evenly distributed in tension anchorages and concentrated at the ends of compression anchorages.

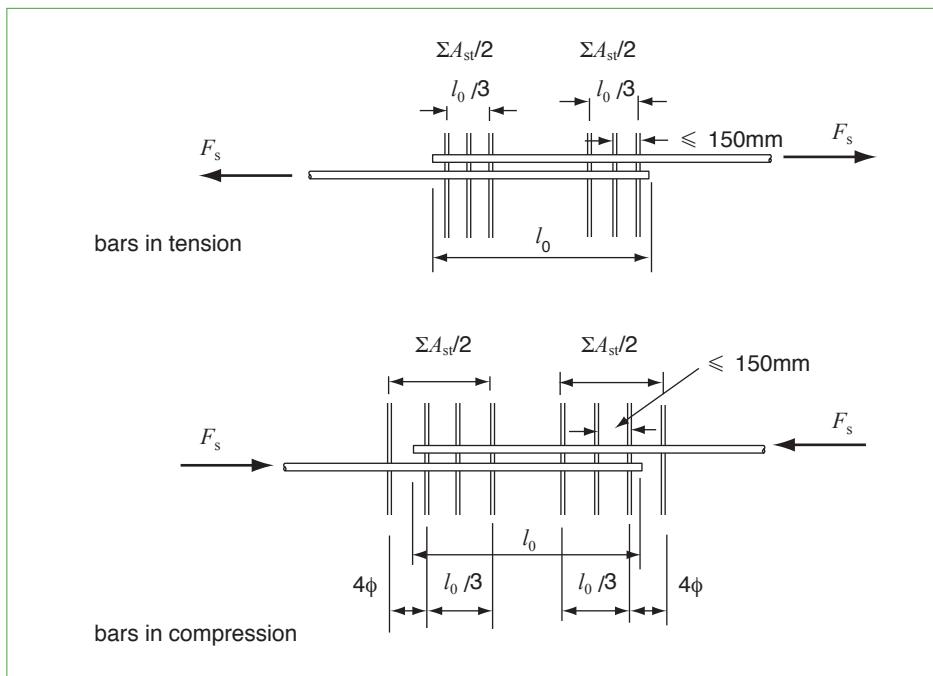
### Laps

Transverse reinforcement is required in the lap zone to resist transverse tension forces. Where the diameter of bars lapped is less than 20mm or the area of bars lapped is less than 25% of the total area of tension reinforcement then any transverse reinforcement or links necessary for other reasons may be assumed sufficient.

Otherwise the area of total transverse reinforcement,  $A_{st}$ , should be equal to or greater than the area of one lapped bar. The transverse bars should be placed perpendicular to the direction of the lapped reinforcement and between that and the surface of the concrete. The transverse reinforcement should be placed as shown in Figure 5.30.

### Anchorage of links

Links should be anchored using one of the methods shown in Figure 5.31.



**Fig 5.30** Transverse reinforcement for lapped splices

### 5.12.5 Additional rules for large diameter bars

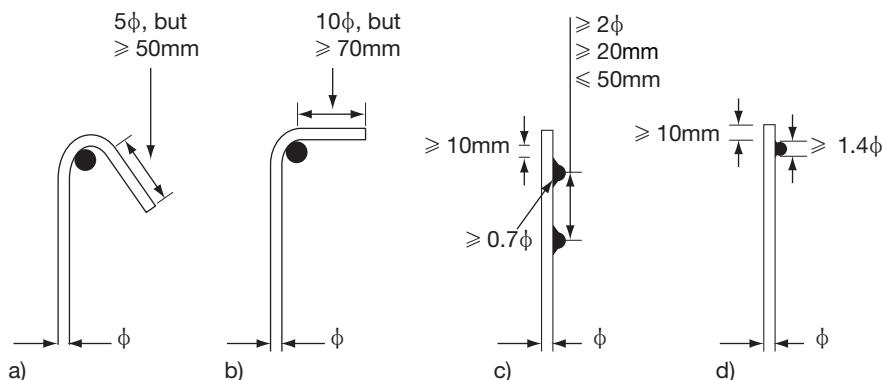
Bars exceeding 40mm in diameter should be used only in elements with a depth not less than 15 times the bar diameter. Such bars should not be anchored in tension zones. Lapped joints in tension or compression are not permitted, and mechanical devices (e.g. couplers) should be used instead.

Transverse reinforcement, additional to that for shear, is required in the anchorage zone where transverse pressure is not present as shown in Figure 5.32. The reinforcement should not be less than the following:

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

Where:  $n_1$  is the number of layers with bars anchored at the same point in the member

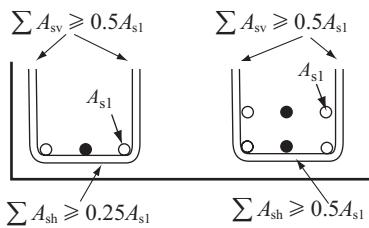
$n_2$  is the number of bars anchored in each layer.



*Note*

For c) and d) the cover should not be less than either  $3\phi$  or 50mm.

Fig 5.31 Anchorage of links



*Note*

- Anchored bar
- Continuing bar

*Example:*

In the left hand case  $n_1 = 1, n_2 = 2$   
and in the right hand case  $n_1 = 2, n_2 = 2$ .

Fig 5.32 Additional reinforcement in an anchorage zone for large diameter bars where there is no transverse compression

### 5.12.6 Curtailment of bars in flexural members

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

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

- the bending moment and
- the effect of a truss model for resisting shear, ('shift rule').

This may be assessed by the 'shift rule', i.e. by shifting the bending moment diagram in the direction of reducing moment by an amount  $a_1$ , where  $a_1 = 0.45d \cot\theta$  for beams and  $1.0d$  for slabs, where  $\theta$  is the angle of the concrete truss assumed in shear design.

A practical method is:

- determine where the bar can be curtailed based on bending moment alone, and
- anchor this bar beyond this location for a distance  $l_{bd} + a_1$ .

This procedure is illustrated in Figure 5.33.

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

- Direct support for slabs: 0.7 times the value given in Table 5.24
- Direct support for beams: 0.8 times the value given in Table 5.25
- All indirect supports: 1.0 times the values given in Tables 5.24 and 5.25

A 'direct' support is one where the reaction provides compression across the bar being anchored. All other supports are considered 'indirect' (see Figure 5.34).

### 5.12.7 Corbels and nibs

Corbels and nibs should be designed using strut and tie models when  $0.4h_c \leq a_c \leq h_c$  or as cantilevers when  $a_c > h_c$  where  $a_c$  is the shear span and  $h_c$  is the overall depth. Unless special provision is made to limit the horizontal forces on the support, the corbel (or nib) should be designed for the combined effects of the vertical force  $F_{Ed}$  and a minimum horizontal force of  $0.2 F_{Ed}$ .

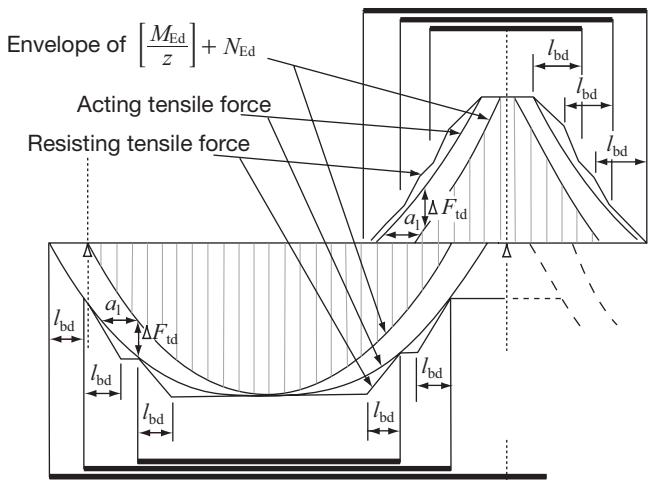
The minimum effective depth of corbel is determined by the maximum resistance of the concrete compressive strut (see Figure 5.35(a)). This may be calculated as follows:

$$\text{Limiting } \frac{z_0}{a_c} \text{ to 1.0 gives } d \geq \frac{F_{Ed}}{0.34v'bf_{ck}} + a_c'$$

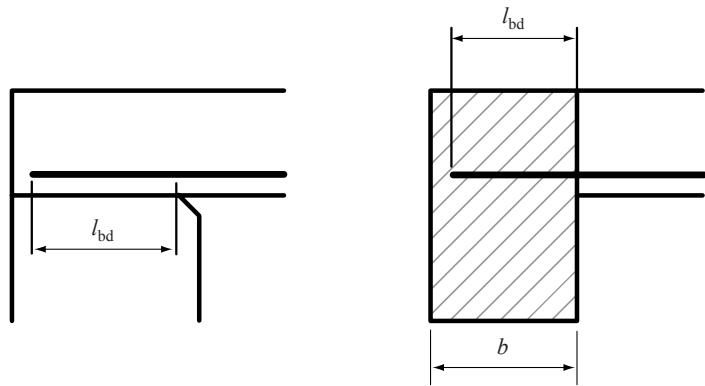
$$\text{Where: } v' = 1 - \frac{f_{ck}}{250}$$

$$\text{Limiting } \frac{z_0}{d} \text{ to } \left(\frac{z_0}{d}\right)_{\min} \text{ gives } d \geq \frac{a_c'}{\sqrt{\frac{0.68a_c'v'bf_{ck}}{F_{Ed}}\left(\left(\frac{z_0}{d}\right)_{\min} - \left(\frac{z_0}{d}\right)^2_{\min}\right) - \left(\frac{z_0}{d}\right)^2_{\min}}}$$

The value of  $(z_0/d)_{\min}$  should not normally be taken less than 0.75.



**Fig 5.33** Envelope design for the design of flexural members - curtailment lengths

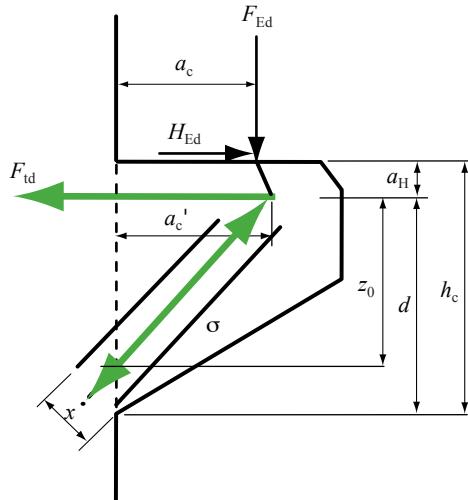


**Fig 5.34** Anchorage at end supports

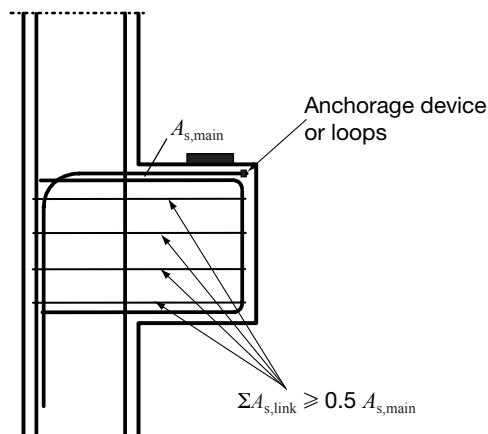
The bearing stress in the corbels should be limited to  $0.48(1-f_{ck}/250)f_{ck}$

The tie force,  $F_{td} = F_{td}' + H_{Ed}$  where  $F_{td}'$  and the value of  $z_0/d$  may be determined from Figure 5.36(a) and Figure 5.36(b) respectively. The value of  $(z_0/d)_{min}$  is assumed to be 0.75.

In addition to the reinforcement required to resist  $F_{td}$ , horizontal stirrups with a total area of  $0.5A_s$  should be provided as shown in Figure 5.35(b).

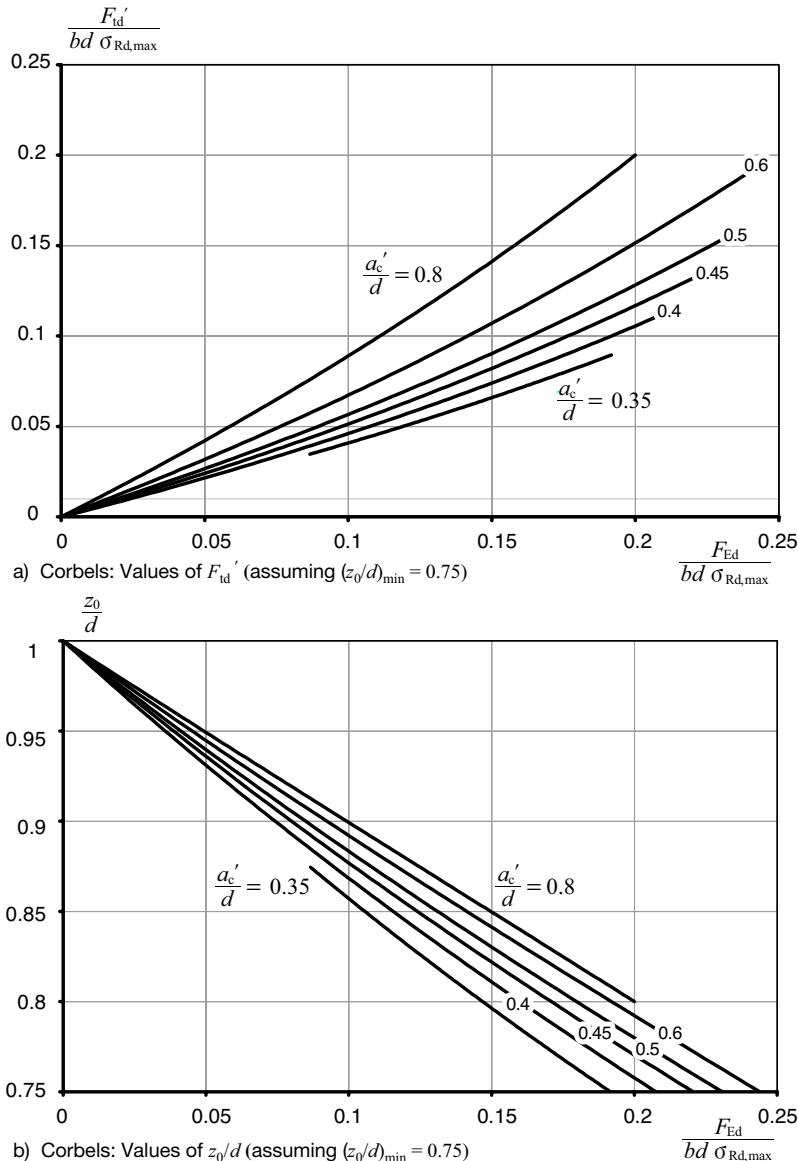


a) Strut and tie layout for corbels



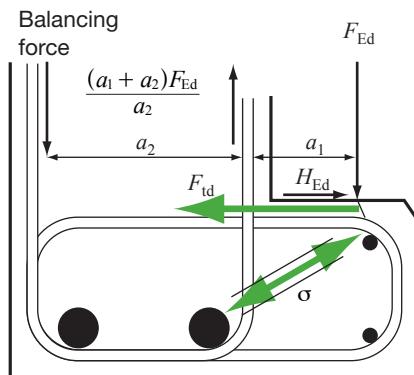
b) Link requirement for corbels

**Fig 5.35** Corbels and nibs



**Fig 5.36** Design chart for corbels

The arrangement of strut and ties for nibs should take account of the position of the bar in the main beam where the strut forces are picked up (see Figure 5.37). It should also be noted that the force in the leg of the beam links close to the nib is increased by a value more than the force on the nib.



**Fig 5.37** Strut and tie layout for nibs

## 6 Design principles - prestressed concrete

### 6.1 Introduction

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

Detail design is often carried out by specialists using automated software. For further information reference should be made, for example, to the Concrete Society Technical Report *Post-tensioned Concrete Floors - Design Handbook*<sup>27</sup>.

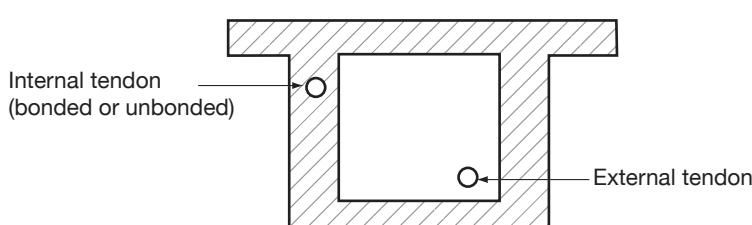
### 6.2 Design principles

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

Prestressed tendons may be internal, i.e. within the concrete, either bonded to the concrete or unbonded (single strand in a plastic tube filled with grease), or external, i.e. outside the concrete but inside the envelope of the member, see Figure 6.1. This *Manual* deals only with prestressed concrete members with internal tendons.

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

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



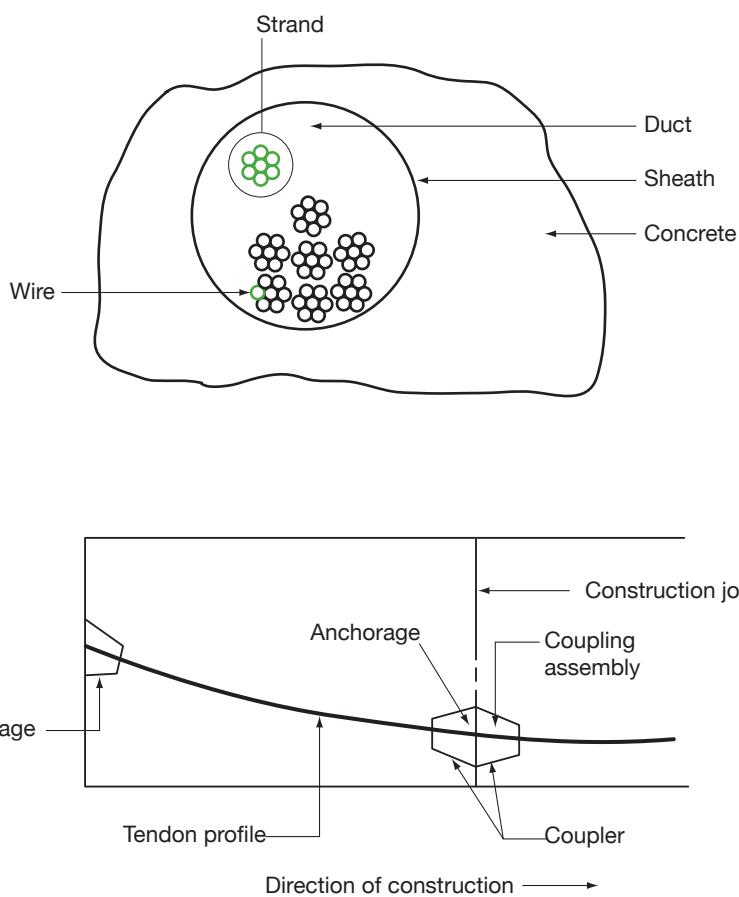
**Fig 6.1** Internal and external tendons

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

Type of construction	Advantages	Disadvantages
Pretensioned	<ul style="list-style-type: none"><li>• no need for anchorages</li><li>• tendons protected by concrete without the need for grouting or other protection</li><li>• prestress is generally better distributed in transmission zones</li><li>• factory produced precast units</li></ul>	<ul style="list-style-type: none"><li>• heavy stressing bed required</li><li>• more difficult to incorporate deflected tendons</li></ul>
Post-tensioned	<ul style="list-style-type: none"><li>• no external stressing bed required</li><li>• more flexibility in tendon layout and profile</li><li>• draped tendons can be used</li><li>• <i>in-situ</i> on site</li></ul>	<ul style="list-style-type: none"><li>• tendons require a protective system</li><li>• large concentrated forces in end blocks</li></ul>

**Table 6.2 Advantages and disadvantages of bonded and unbonded construction**

Type of construction	Advantages	Disadvantages
Bonded	<ul style="list-style-type: none"><li>• tendons are more effective at ULS</li><li>• does not depend on the anchorage after grouting</li><li>• localises the effects of damage</li><li>• the prestressing tendons can contribute to the concrete shear capacity</li></ul>	<ul style="list-style-type: none"><li>• tendon cannot be inspected or replaced</li><li>• tendons cannot be re-stressed once grouted</li></ul>
Unbonded	<ul style="list-style-type: none"><li>• tendons can be removed for inspection and are replaceable if corroded</li><li>• reduced friction losses</li><li>• generally faster construction</li><li>• tendons can be re-stressed</li><li>• thinner webs and larger lever arm</li></ul>	<ul style="list-style-type: none"><li>• less efficient at ULS</li><li>• relies on the integrity of the anchorages and deviators</li><li>• a broken tendon causes prestress to be lost for the full length of that tendon</li><li>• less efficient in controlling cracking</li><li>• careful attention is required in design to ensure against progressive collapse</li></ul>



**Fig 6.2** Nomenclature associated with prestressed concrete members

Figure 6.2 shows some common features and nomenclature associated with prestressed concrete.

### 6.3 Loading

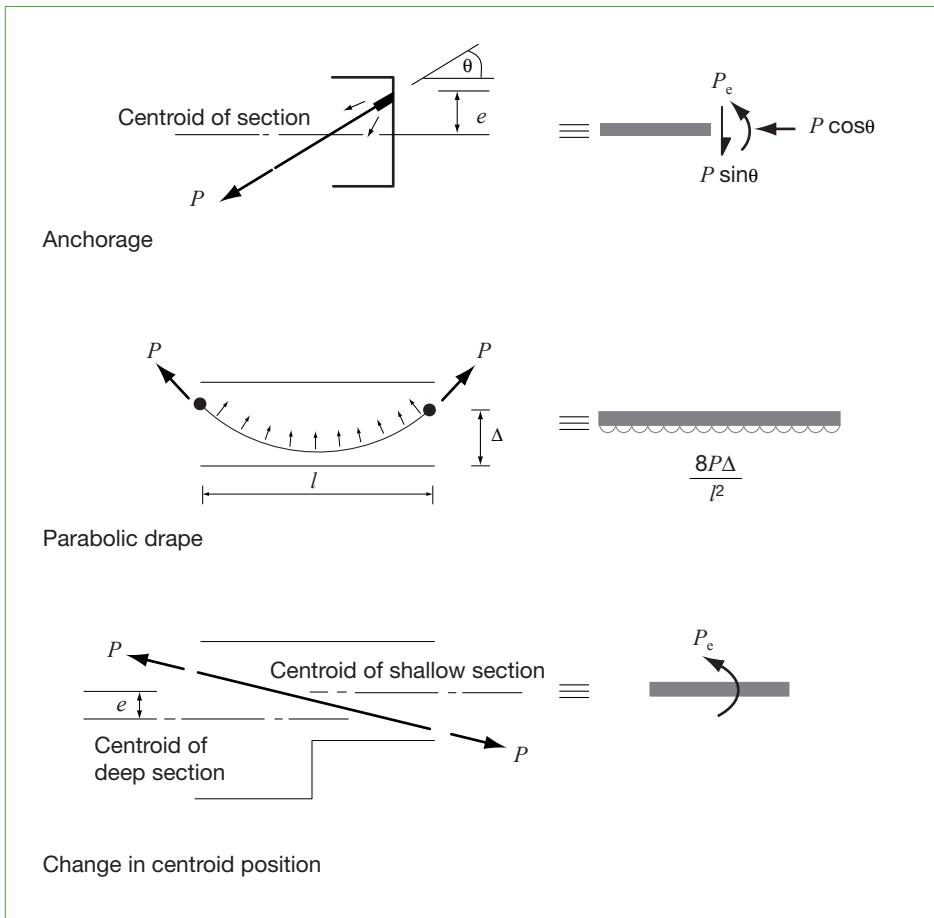
The loads and load combinations to be used in calculations are given in Section 3.

In this *Manual* prestressing forces are considered as variable loads for serviceability limit states and as resistances for the ultimate limit state.

### 6.3.1 Serviceability limit state (SLS)

The effects of prestressing at SLS are normally taken into account by equivalent loads (see Figure 6.3). The analysis using equivalent loads automatically takes account of secondary effects ('parasitic effects') (see Figure 6.4).

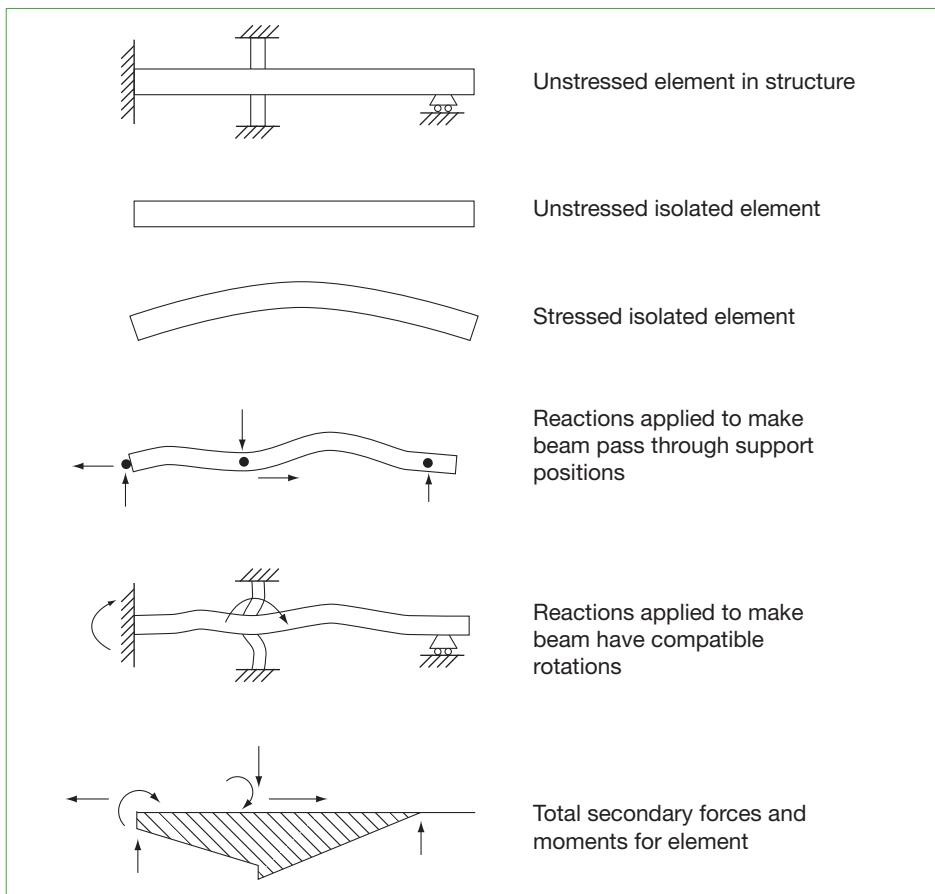
For SLS the dead load and post-tensioning effects, including the effect of losses due to creep, long term shrinkage and relaxation of the prestressing steel, should be considered acting with those combinations of variable loads which result in the maximum stresses. Unless there are specific abnormal loads present, it will generally be sufficient to consider the post-tensioning effects in combination with the variable loads as given in Section 3. For flat slabs it is normally satisfactory to apply the combinations of loading to alternate full width strips of the slab in each direction. However it will normally be satisfactory to obtain the moments and forces under the single load case using the frequent load values.



**Fig 6.3** Equivalent loads

**Deflection limit state** – Where the analysis is used to determine deflections, span/500 after finishes have been added is normally an appropriate limit using the quasi-permanent load combination ( $G_k + P_{BL} + \psi_2 Q_k$ , where  $P_{BL}$  is the prestress equivalent load). It may be necessary to consider other limits and loads depending on the requirements for the slab.

**Crack width limit state** – For analysis to determine crack widths the frequent load combination ( $G_k + P_{BL} + \psi_1 Q_k$ ) should be used for bonded tendons and the quasi-permanent load combination ( $G_k + P_{BL} + \psi_2 Q_k$ ) for unbonded tendons. Unless otherwise agreed with the client the crack width limit should be 0.2mm, except for water retaining structures where the limit should be 0.1mm.



**Fig 6.4** Secondary effects from prestressing in a statically indeterminate structure

At transfer of prestress the permanent loads present during stressing, together with the post-tensioning effects and the effects of early thermal shrinkage, should be considered in obtaining stresses.

Where the applied loads change significantly during construction or phased stressing is employed, the various stages should each be checked for transfer stress limits.

### 6.3.2 Ultimate limit state (ULS)

At the ULS the following load combinations should be used to arrive at the maximum moments and shears at any section (see also Section 3):

$$1.35G_k + 1.5Q_{kl} + \sum 1.05Q_{ki} + \gamma_p P$$

Where:  $G_k$  is the permanent load

$Q_{kl}$  is the primary variable action (load)

$Q_{ki}$  any other variable action

$P$  is the prestressing force.

$\gamma_p$  should be taken as  $\gamma_{p,fav} = 0.9$  when beneficial

or  $\gamma_{p,unfav} = 1.1$  when unfavourable.

When checking flexural stresses, secondary effects of prestressing should be included in the applied loads with a load factor of 1.0.

## 6.4 Materials, prestressing components

In the UK the most common prestressing tendons are comprised of 7-wire low relaxation (Type 2) strands to BS 5896<sup>28</sup> (this standard will be replaced by EN 10138<sup>29</sup>) which are generally available in three different types: standard, super and dyform (or drawn); and two nominal diameters: 12.9mm and 15.7mm (18mm is also available). In post-tensioned normal building construction these are used singly with unbonded tendons and in groups with bonded tendons.

*Unbonded tendons* – Unbonded tendons are protected by a layer of grease inside a plastic sheath. Under normal conditions, the strand is supplied direct from the manufacturer already greased and sheathed. In no circumstances should PVC be used for the plastic sheath, as it is suspected that chloride ions can be released in certain conditions.

*Bonded tendons* – Bonded tendons are placed in metal or plastic ducts, which can be either circular or oval in form. Metal ducts are made from either spirally wound or seam folded galvanised metal strip. The use of plastic ducts should be considered when designing car parks. The oval duct is used in conjunction with an anchorage, which ensures that up to four strands are retained in the same plane in order to achieve maximum eccentricity.

For beams, up to 19 strands may be used in a tendon. This should normally be considered the limit.

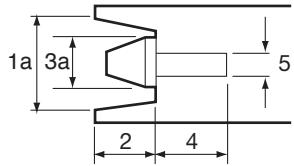
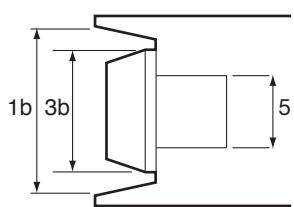
Table 6.3 (slabs) and Table 6.4 (beams) give typical dimensional data for ducts, anchorages, anchorage pockets and jacks for the more commonly used prestressing systems in the UK. For more precise information and when other prestressing systems are used, the engineer should refer to the manufacturer's literature.

**Table 6.3 Typical dimensional data for common post-tensioning systems for slabs**

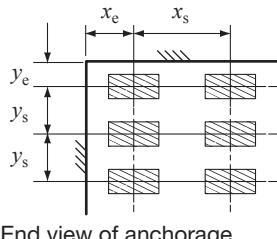
Strand diameter - number of strands	Pocket dimensions (mm)			Anchorage dimensions (mm)		Duct external dimensions <sup>b</sup> (mm)	Anchorage spacing (mm)				Jack clearances (mm)			
	1a	1b	2	3a	3b	4	$x_e$	$y_e$	$x_s$	$y_s$	C	E	F	
12.9 - 1	130	130	110	70	110	70	30 dia.	125	80	150	100	1390	100	-
12.9 - 4	144	310	103	96	250	130	75 × 20	220	140	370	220	1200	90	280
15.7 - 1	150	150	115	130	130	95	35 dia.	145	100	175	125	1450	100	-
15.7 - 4	168	335	127	115	280	240	75 × 20	235	160	400	230	1450	70	327

Notes

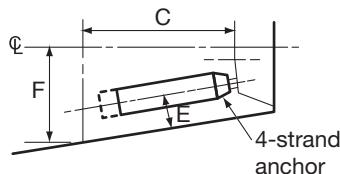
- a The dimensions and clearances are illustrated in Figure 6.5.
- b Data are for corrugated steel sheaths with a thickness of 1.5mm.
- c The values in the Table are based on an envelope of the requirements of a number of manufacturers' systems. Where clearances are critical it is recommended that reference is made to the specific manufacturers' catalogues.



Sectional views



End view of anchorage



Jack clearances

Note

The dimensions shown in this figure are given in Table 6.3 for different tendon sizes.

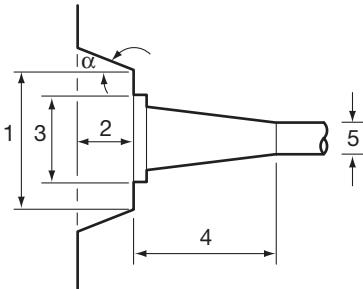
**Fig 6.5** Dimensions of common post-tensioning systems (1 - 9 strands)

**Table 6.4 Typical dimensional data for common post-tensioning systems (1 to 19 strands)**

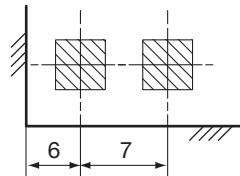
Strand diameter - number of strands	Pocket dimensions			Anchorage dimensions		Duct diameter int/ext <sup>c</sup>	Anchorage spacing		Jack diameter and clearances		
	(mm)	(mm)	(mm)	3	4		(mm)	(mm)	C	D	E
12.9 - 1	130	110	30°	70	85	25/30	80	90	1200	140	100
12.9 - 3	180	115	30°	120	210	40/45	115	155	1100	200	150
12.9 - 4	240	115	30°	135	210	45/50	125	180	1100	248	175
12.9 - 7	240	120	30°	175	215	55/60	155	235	1200	342	220
12.9 - 12	330	125	30°	230	405	75/82	195	305	1300	405	250
12.9 - 19	390	140	30°	290	510	80/87	230	385	2100	490	295
15.7 - 1	135	115	30°	75	85	30/35	95	105	1200	140	100
15.7 - 3	200	115	30°	150	210	45/50	130	185	1100	210	140
15.7 - 4	240	120	30°	157	215	50/55	140	210	1200	342	220
15.7 - 7	305	125	30°	191	325	60/67	175	280	1300	405	250
15.7 - 12	350	140	30°	270	510	80/87	220	365	1500	490	295
15.7 - 19	470	160	30°	340	640	100/107	265	460	2000	585	300

**Notes**

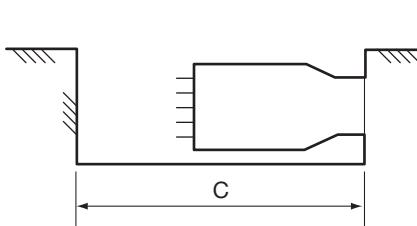
- a The dimensions and clearances are illustrated in Figure 6.6.
- b The values in the Table are based on an envelope of the requirements of a number of manufacturers' systems. Where clearances are critical it is recommended that reference is made to the specific manufacturers' catalogues.
- c Data are for corrugated steel sheaths with bonded tendons.



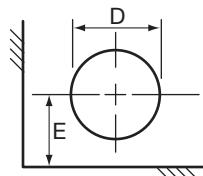
Sectional view



End view of anchorage



Jack clearances



*Note*

The dimensions shown in this figure are given in Table 6.4 for different tendon sizes.

**Fig 6.6** Dimensions of common post-tensioning systems for slabs

## 7 Preliminary design - prestressed concrete

### 7.1 Introduction

#### 7.1.1 General

The following information should enable engineers to do the initial sizing of members.

It is assumed that the final design is carried out by a specialist.

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

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

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

#### 7.1.2 Effective lengths

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

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

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

#### 7.1.3 Lateral buckling

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

#### 7.1.4 Torsion

In normal slab-and-beam or framed construction specific calculations for torsion are not usually necessary, torsional cracking being adequately controlled by shear reinforcement. Where torsion is essential for the equilibrium of the structure, e.g. the arrangement of the structure is such that loads are imposed mainly on one face of a beam without corresponding rotational restraints being provided, EC2<sup>1</sup> should be consulted.

## 7.2 Loads

The loads and load combinations to be used in calculations are given in Section 3.

In this *Manual* prestressing forces are considered as variable loads for serviceability limit states and as resistances for the ultimate limit state.

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

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

- floor finish (screed)       $1.8 \text{ kN/m}^2$
- ceiling and service load     $0.5 \text{ kN/m}^2$

Allowance for:

- demountable lightweight partitions       $1.0 \text{ kN/m}^2$ 
  - treat these as variable actions.
- blockwork partitions                   $2.5 \text{ kN/m}^2$ 
  - treat these as permanent actions when the layout is fixed.

Loading of concrete should be taken as  $25 \text{ kN/m}^3$ .

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

- i) permanent action + variable action:  
 $1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic variable load}$
- ii) permanent action + wind action:  
 $1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic wind load}$
- iii) permanent action + snow action:  
 $1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic snow load}$
- iv) permanent action + variable action + wind action + snow action:  
 $1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic variable load}$   
 $+ 0.7 \times \text{characteristic wind load} + 0.7 \times \text{characteristic snow load}$

Where appropriate the other combinations should be checked:

$1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic wind load}$   
 $+ 0.7 \times \text{characteristic variable load} + 0.7 \times \text{characteristic snow load}$   
and

$1.0 \times \text{characteristic permanent load} + 1.0 \times \text{characteristic snow load}$   
 $+ 0.7 \times \text{characteristic wind load} + 0.7 \times \text{characteristic variable load.}$

### 7.3 Material properties

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

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

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

Strand type	Strand load (kN)	
	12.9mm dia.	15.7mm dia.
Standard	88	125
Super	100	143

When UK steels are used for reinforcement a characteristic strength,  $f_{yk}$ , of 500MPa should be adopted.

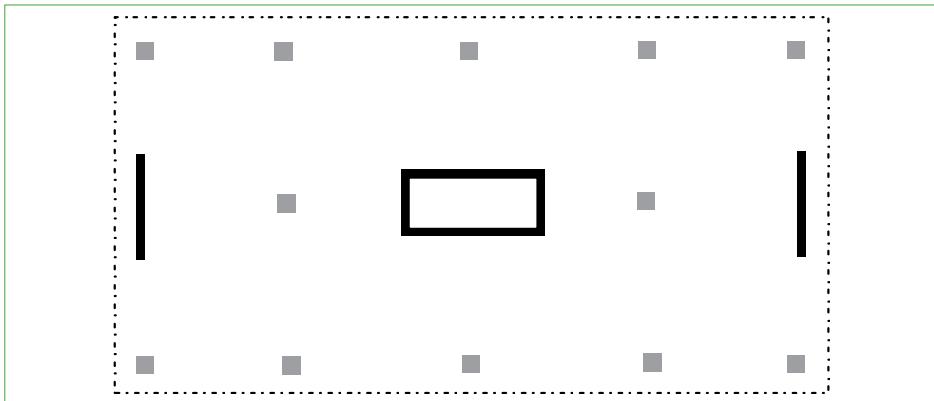
### 7.4 Structural form and framing

In choosing column and wall layouts and spans for a prestressed floor the designer should provide stability against lateral forces and ensure braced construction. Several possibilities may be considered to optimise the design, which include:

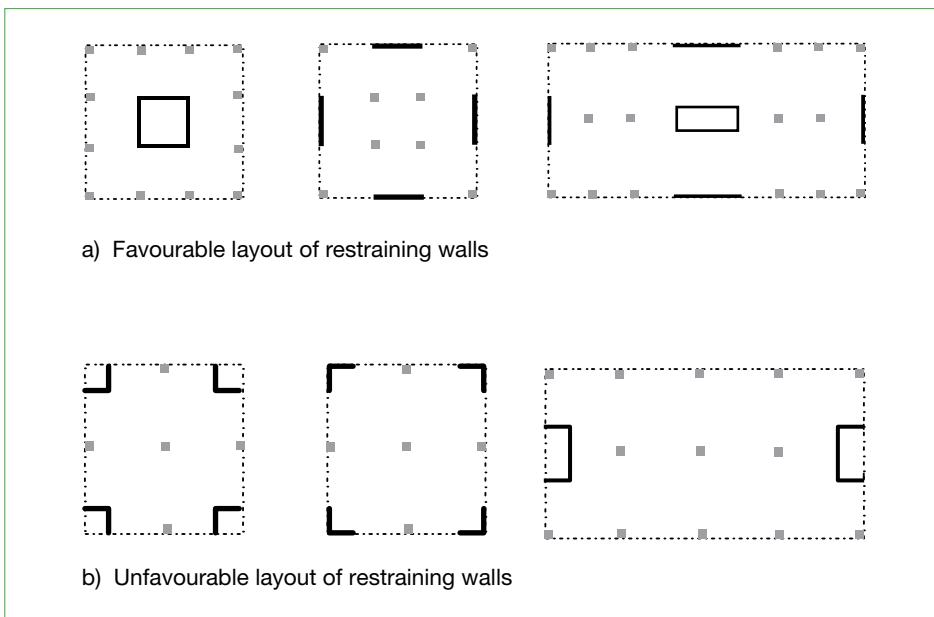
- Reducing the length of the end spans or, if the architectural considerations permit, inset the columns from the building perimeter to provide small cantilevers (see Figure 7.1). Consequently, end span bending moments will be reduced and a more equitable bending moment configuration obtained. It also moves the anchorages away from the columns.
- Reducing, if necessary, the stiffness of the columns or walls in the direction of the prestressing to minimise the prestress lost and resulting cracking in overcoming the restraint offered to floor shortening. Figure 7.2 shows some typical floor layouts. Favourable layouts (see Figure 7.2a)) allow the floors to shorten towards the stiff walls. Unfavourable layouts (see Figure 7.2b)) restrain the floors from shortening.
- Where span lengths vary, adjust the tendon profiles and the number of tendons to provide the uplift required for each span. Generally this will be a similar percentage of the dead load for each span.

Once the layout of columns and walls has been determined, the next consideration is the type of floor to be used. This again is determined by a number of factors such as span lengths, magnitude of loading, architectural form and use of the building, special requirements such as services, location of building, and the cost of materials available.

The slab thickness must meet two primary functional requirements – structural strength and deflection. Vibration should also be considered where there are only a few panels. The selection of thickness or type (e.g. flat slabs with or without drops, coffered or waffle, ribbed or even beam and slab) is also influenced by concrete strength and loading and architectural intent. There are likely to be several alternative solutions to the same problem and a preliminary costing exercise may be necessary in order to choose the most economical. More information can be found in the Concrete Society Technical report TR43<sup>27</sup>.



**Fig 7.1** Typical floor layout to maximise prestressing effects



**Fig 7.2** Layout of shear walls to reduce loss of prestress and cracking effects

Allowance for sufficient topping should be made to accommodate cambers induced by prestressing, including any differential cambers between adjoining members.

The arrangement should take account of possible large openings for services. Tendon positions should be chosen to avoid locations of any possible future holes. Sufficient space should be provided to allow for jacking and for the possible replacement of unbonded tendons.

## 7.5 Fire resistance and durability

The size of structural members may be governed by the requirement of fire resistance. Table 7.2 gives the minimum practical member sizes and cover to the prestressing tendons required for different periods of fire resistance. For beams and ribbed slabs, width can be traded against axis distance to give the necessary fire resistance. Thus Table 7.2 gives two sets of data for each hourly rating, one based on minimum width and the other based on minimum axis distance.

**Table 7.2 Minimum member sizes and axis distances for prestressed members in fire**

Member	Minimum dimension (mm) for standard fire resistance (mins) of:	R240	R120	R60	
<b>Beams</b>	Width Axis distance, $a$ : Simply supported Continuous <sup>a</sup>	280 700 105 85 90 65	200 500 80 65 60 45	120 300 55 40 40 27 <sup>c</sup>	
<b>Plain soffit slabs:</b>	Depth (including non-combustible finishes)	175	120	80	
	Axis distance, $a$ : Simply supported Continuous <sup>b</sup>	80 55	55 35	35 25	
	Two-way	Simply supported Continuous <sup>b</sup>	65 55	40 35	30 25
<b>Ribbed slabs:</b>	Width of rib, $b_r$ Axis distance <sup>b</sup> , $a$ $a_{sd} = a + 10$	280 500 105 85 175 55	160 300 85 55 120 35	100 200 50 30 95 25	
	Continuous Axis distance to side of rib $a_{sd} = a + 10$	450 700 85 75 175 55	160 300 60 45 120 35	100 200 40 25 80 25	
<b>Flat slabs</b>	Depth (including non-combustible finishes) Axis distance <sup>b</sup> , $a$	200 65	200 50	180 30	

**Notes**

- a** Where more than 15% moment redistribution has been assumed the values for simply supported beams should be used.
- b** Where more than 15% moment redistribution has been assumed the values for single way simply supported slabs should be used.
- c** Normally the cover required for bond and durability at normal temperature will control.

The values given in Table 7.2 will also ensure that there will be sufficient cover for exposure condition XC1. For other exposure conditions reference should be made to Appendix B.

Unless otherwise stated grouting of ducts should be carried out in accordance with the National Structural Concrete Specification<sup>30</sup>. Reference should also be made to the Concrete Society Technical Report No. TR47<sup>31</sup> with regard to grouting of ducts and protection of anchorages.

Where tendons are curved, the cover perpendicular to the plane of curvature should be increased to prevent bursting of the cover concrete in accordance with Table 7.3.

**Table 7.3 Minimum cover to curved ducts (mm)**

Radius of curvature of duct (m)	Duct internal diameter (mm)										
	25	30	40	45	50	55	60	75	80	100	
	Tendon force (kN)										
(m)	159	229	477	636	916	1113	1603	1908	2748	4351	
2	30	35	70	95			Radii not normally used				
4			45	50	70	80	120	140			
6					55	60	80	95	135	215	
8							65	80	105	160	
10									90	130	
12									85	115	
14	30	35	45	50	55	60	65	80	85	105	

#### Notes

- a The Table is based on reference 32.
- b The tendon force shown is the maximum normally available for the given size of duct.
- c Where tendon profilers or spacers are provided in the ducts, and these are of a type which will concentrate the radial force, the values given in the Table will need to be increased.
- d The cover for a given combination of duct internal diameter and radius of curvature shown in the Table, may be reduced in proportion to the square root of the tendon force when this is less than the value tabulated.

## 7.6 Stiffness

### 7.6.1 Slabs

To provide adequate stiffness, the effective depths of slabs and the waist of stairs should not be less than those derived from Table 7.4. It should be noted that Table 7.4 is applicable for multi-span floors only. For single-span floors the depth should be increased by approximately 15%.

**Table 7.4 Typical span/depth ratios for a variety of section types for multi-span floors**

Section type	Total imposed load (kN/m)	Span/depth ratios $6m \leq L \leq 13m$ (kN/m)		Additional requirements for vibration
1 Solid flat slab	2.5 5.0 10.0	40 36 30		A
2 Solid flat slab with drop panel	2.5 5.0 10.0	44 40 36		A
3 Banded flat slab	2.5 5.0 10.0	Slab 45 40 35	Beam 25 22 18	A
4 Coffered flat slab	2.5 5.0 10.0	25 23 20		B
5 Coffered flat slab with solid panels	2.5 5.0 10.0	28 26 23		B

**Table 7.4 Typical span/depth ratios for a variety of section types for multi-span floors (cont.)**

Section type	Total imposed load (kN/m)	Span/depth ratios $6m \leq L \leq 13m$ (kN/m)	Additional requirements for vibration	
6 Coffered slab with band beam <sup>d</sup>	2.5 5.0 10.0	28 26 23	B	
7 Ribbed slab <sup>e</sup>	2.5 5.0 10.0	30 27 24	B	
8 One-way slab with narrow beam	2.5 5.0 10.0	Slab 42 38 34	Beam 18 16 13	A

Notes

- a Vibration. The following additional check should be made for normal office conditions if no further vibration checks are carried out:
  - A either the floor has at least four panels and is at least 250mm thick or the floor has at least eight panels and is at least 200mm thick.
  - B either the floor has at least four panels and is at least 400mm thick or the floor has at least eight panels and is at least 300mm thick.
- b All panels assumed to be square.
- c Span/depth ratios not affected by column head.
- d It may be possible that prestressed tendons will not be required in the banded sections and that untensioned reinforcement will suffice in the ribs, or vice versa.
- e The values of span/depth ratio can vary according to the width of the beam.

## 7.6.2 Isolated beams

For initial sizing the effective depth of isolated beams should be determined from Table 7.5. For further refinement reference should be made to a specialist contractor or EC2<sup>1</sup>.

**Table 7.5 Span/effective depth ratios for initial sizing of isolated beams**

Cantilever	8
Simply supported	18
Continuous	22

## 7.7 Sizing

### 7.7.1 Introduction

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

- width of beams and ribs
- column sizes and reinforcement (see Section 4.8.4)
- shear in flat slabs at columns
- practicality of tendon and reinforcement arrangements in beams, slabs and at beam-column junctions.

### 7.7.2 Loading

Serviceability loads should be used throughout for initial design (see Section 7.2). For prestressed concrete members it is necessary to calculate the maximum shear force and both the maximum and minimum bending moments at the critical sections. A critical condition may occur at transfer (when the prestressing force is initially applied to the concrete member) as the prestressing force is at its highest value and the applied loading may be significantly less than the maximum load which the member will eventually have to carry. Secondary effects of prestress are taken into account using equivalent loads (see Section 6.3.1).

The loading arrangements to be considered for continuous members may follow either of the following methods:

- i) Alternate spans carrying the design variable and permanent load, other spans carrying the design permanent load and  
Any two adjacent spans carrying the design variable and permanent load, other spans carrying design permanent load.
- ii) All spans carrying the design variable and permanent load and  
Alternate spans carrying the design variable and permanent load, other spans carrying design permanent load.

### 7.7.3 Width of beams and ribs

The width may be determined by limiting the shear stress in beams to 2.0MPa and in ribs to 0.8MPa.

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

$$\text{width of rib (in mm)} = \frac{1000V_{\text{SLS}}}{0.8d}$$

Where:  $V_{\text{SLS}}$  is the maximum shear force at the serviceability limit state (in kN) on the beam or rib, considered as simply supported, and

$d$  is the effective depth in mm.

### 7.7.4 Shear

#### 7.7.4.1 General

Shear checks for prestressed concrete members are carried out at the ultimate limit state.

The shear capacity of prestressed elements is made up from three components:

- i) The concrete shear component (or in the case of shear reinforced concrete the combined concrete and shear steel component):  $V_{\text{Rd},c}$ ,  $V_{\text{Rd},cs}$ ,  $V_{\text{Rd},s}$ .
- ii) The part of the shear that is carried by arch (vault for flat slabs) action not dependent on bonded reinforcement.
- iii) The part of the load which does not act on the failure surface as it is carried to the columns by the vertical component of the tendons:  $V_p$ .

When calculating the contribution of the prestressing force at ULS, both the direct stress,  $\sigma_{cp}$ , and the beneficial effects due to the vertical component of the prestress force, the mean value of prestress calculated should be multiplied by an appropriate safety factor  $\gamma_p$ . The value of  $\gamma_p$ , given in UK National Annex, is 0.9 when the prestress effect is favourable, and 1.1 when it is unfavourable.

In the case of linear elements requiring shear reinforcement the contribution of the concrete to the shear strength is ignored and calculation is based on the variable strut method.

The effective depth,  $d$ , used in checking the shear capacity should be calculated ignoring any inclined tendons.

#### 7.7.4.2 Beams and slabs (single and two way)

For slabs not requiring shear reinforcement reference should be made to Section 5.2.4.2.

Where beams (or slabs) require shear reinforcement the contribution of the concrete to the shear strength is ignored and calculation is based on the variable strut method (see Section 5.4.4.3).

The effect of levels of prestress up to 25% of the design compressive strength is to increase the value of  $V_{\text{Rd,max}}$  (the maximum strut force) that can be used. This allows higher values of  $\text{Cot}\theta$  (the strut angle) to be used which in turn reduces the number of links required to take a given shear force. The result of this approach is that to obtain benefits from the axial compression

the designer will need to maximise the value of  $\text{Cot}\theta$ . The contribution of the tendon inclination can be included with the term  $V_{\text{td}}$  which is the design value of the shear component in an inclined tensile chord.

#### 7.7.4.3 Flat slabs

For the preliminary design it is considered reasonable to neglect any prestressing effects and treat the slab as a reinforced concrete slab (see Section 4.8). For more information reference should be made to the Concrete Society Technical Report No. 43<sup>27</sup>.

#### 7.7.5 Adequacy of chosen sections to accommodate the tendons and reinforcement

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

##### 7.7.5.1 Bending moments and shear forces

###### Beams and one-way solid slabs

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

###### Two-way solid slabs on linear supports

If the longer span  $l_y$  does not exceed 1.5 times the shorter span  $l_x$ , the average moment per metre width may be taken as that shown in Table 7.6.

**Table 7.6 Moment coefficients for two-way solid slabs on linear supports**

	<b>Short span (kNm per metre)</b>	<b>Long span (kNm per metre)</b>
at midspan	$\frac{w_{\text{SLS}} l_x l_y}{20}$	$\frac{w_{\text{SLS}} l_x^2}{20}$
at a continuous support	$-\frac{w_{\text{SLS}} l_x l_y}{17}$	$-\frac{w_{\text{SLS}} l_x^2}{17}$

###### Notes

- a  $w_{\text{SLS}}$  is the design load at the serviceability limit state in  $\text{kN/m}^2$ , and  $l_x$  and  $l_y$  are in metres.
- b If  $l_y > 1.5 l_x$  the slab should be treated as acting one-way.

###### Solid flat slabs

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

###### Ribbed slabs

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

### 7.7.5.2 Provision of tendons and reinforcement

#### Determine the number of tendons

An estimate of the size and number of tendons required can be made using ‘equivalent loads’ to represent the effects of prestressing (see Section 6.3.1). In this approach the concrete section is considered loaded by the applied dead and imposed loads which are partially supported (or balanced) by the forces from the prestressing tendons. Any out-of-balance forces have to be resisted by the concrete section and ordinary reinforcement. The exact degree of load balancing required is a matter of experience, judgement and the degree of cracking that is considered acceptable. When draped tendons are used and cracking is acceptable an economical design will generally be obtained when the prestressing tendons balance approximately 50% of the total dead and imposed loading.

The bending moments, axial forces and corresponding stresses in the concrete member under dead, imposed and the equivalent loads are calculated as described in Section 7.7.5.1, assuming a homogeneous section. The prestressing force and tendon profile are adjusted until the stresses obtained comply with the limits given in Table 7.7. When checking the stresses at transfer, the prestressing force required for the in-service condition should be increased by 60%, as the long-term (time-dependent) losses will not have occurred, and no tension should be allowed in the concrete.

**Table 7.7 Allowable stresses for initial design**

Maximum compressive stress	$0.6 f_{ck}$
Maximum tensile stress: no tension allowed otherwise	0 5MPa
<i>Note</i>	
$f_{ck}$ is the concrete cylinder strength.	

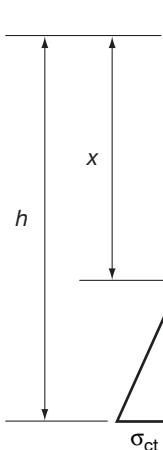
When the prestressing force has been determined, calculate the size and number of tendons using Table 7.1.

When tension in concrete has been taken into account, ordinary reinforcement should be provided in order to control crack widths and to give adequate ultimate capacity. The area of reinforcement provided should be sufficient to resist the total net tensile force on the section when acting at a stress of  $0.63 f_y k$  (see Figure 7.3) when bonded tendons are used. When unbonded tendons are employed, increase this area by 20%.

#### Tendon and reinforcement arrangements

When the number and size of the prestressing tendons, ordinary reinforcement and the areas of the main reinforcement in any non-prestressed elements (see Section 4.8.7) have been determined, check that the tendons and bars can be arranged with the required cover in a practicable manner avoiding congested areas.

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



Note

$$\text{Tensile force} = \frac{b(h-x)\sigma_{ct}}{2}$$

where  $b$  = width of tension zone

$h$  = depth of tension zone

$\sigma_{ct}$  = max. tensile stress in concrete

**Fig 7.3** Total net tensile force

## 7.8 Initial design

### 7.8.1 Introduction

The following clauses are intended for initial design only.

#### 7.8.1.1 Tendon profile

A tendon profile is chosen which satisfies the cover requirements and reflects the bending moment diagram for the applied loads (i.e. high over the supports and low at midspan). For slabs it is common practice to choose the points of contraflexure of the tendon profile so that 90% of the drape occurs over 90% of the half span (see Figure 7.4). The tendon profile should be sketched at this stage and the leading dimensions indicated.

#### 7.8.1.2 Tendon force profile - Initial force ( $P_0$ )

The force in the tendon reduces with distance from the jacking point because of friction which arises due to both intentional and unintentional deviations of the tendon. For most building structures with draped tendons it is sufficiently accurate to assume that the unintentional angular deviation is uniform along the member so that the force in the tendon,  $P_x$ , becomes

$$P_x = P_0 [1 - (\mu\alpha + K)x]$$

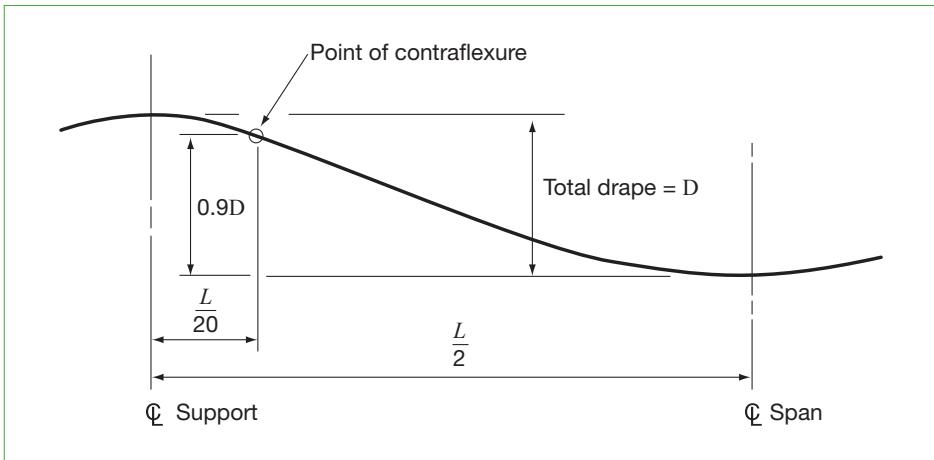
Where:  $P_0$  is the initial jacking force

$\mu$  is the coefficient of friction of the duct

$\alpha$  is the angular deviation per unit length in rad/m

$K$  is the wobble factor (which allows for unintentional deviation) in  $m^{-1}$ , and

$x$  is the distance from the point at which the tendon is jacked in metres.



**Fig 7.4** Geometry of tendon profile

Values of  $\mu$  and  $K$  depend on the surface characteristics of the tendon and duct, on the presence of rust, on the elongation of the tendon and on the tendon profile. Information on appropriate values are provided by the manufacturers of prestressing systems, but in the absence of more exact data values of  $\mu = 0.25$  and  $K = 0.003 \text{ m}^{-1}$  may be used.

With wedge-anchored tendon systems the tendon force profile must be adjusted to allow for the wedge draw-in or anchorage set. This depends on the wedge set movement which can be obtained from the prestressing system manufacturer and is typically 6mm.

The loss due to anchorage draw-in,  $\Delta P$  is given by:

$$\Delta P = \frac{E_p A_{ps} \delta}{L} + m L$$

Where:  $L = \sqrt{\frac{\delta A_{ps} E_p}{m}}$   $\leq$  length of tendon

$\delta$  is the wedge draw-in

$A_{ps}$  is the area of a tendon

$E_p$  is the elastic modulus of the tendon which may be taken as  $190 \text{ kN/mm}^2$

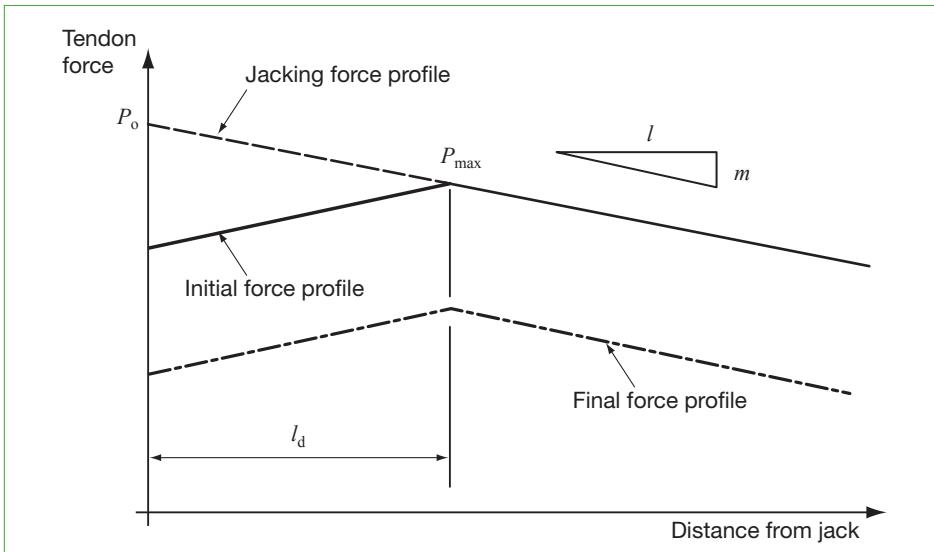
$m$  is the rate of change of prestressing force due to friction, i.e.  $P_0 \mu(\alpha + k)$

$\mu$  is the coefficient of friction

$\alpha$  is the angular deviation per unit length

$k$  is the unintentional angular deviation per unit length.

The tendon force profile is shown in Figure 7.5. The maximum force in the tendon after lock-off is  $P_{\max} = P_0 - m l_d$ . The initial jacking force should be chosen so that  $P_0$  and  $P_{\max}$  do not exceed the values given in Table 7.8, which gives values for commonly used strands.



**Fig 7.5** Tendon force profile

**Table 7.8 Maximum jacking loads per strand**

Strand type	Diameter (mm)	Area (mm <sup>2</sup> )	$f_{pk}$ (MPa)	$f_{p0,1k}$ (MPa)	$P_0$ (kN)	$P_{max}$ (kN)
Y1770S7	15.2	140.0	1770	1520	192	181
	16.0	150.0			205	194
Y1860S7	12.5	93.0	1860	1600	134	126
	13.0	100.0			144	136
	15.2	140.0			202	190
	16.0	150.0			216	204
Y1860S7G	12.7	112.0	1860	1610	162	153
Y1820S7G	15.2	165.0	1820	1560	232	219
Y1700S7G	18.0	223.0	1700	1470	294	278

When the force from pretensioned tendons is transferred to the concrete, the concrete and tendons shorten, reducing the prestress force. The tendon force should be calculated prior to transfer of prestress.

In a post-tensioned member the tendons are normally stressed sequentially and the average loss due to elastic shortening is only 50% of the value for a pretensioned member.

Calculate the initial prestressing force,  $P_{m,0}$ , by multiplying the force profile for one tendon (see Figure 7.5) by the number of tendons and subtracting the elastic loss.

### 7.8.1.3 Tendon force profile – Final force ( $P_{m,\infty}$ )

The tendon force will reduce with time due to relaxation of the tendon and creep and shrinkage of the concrete. The loss due to relaxation depends on the tendon type and the initial stress in the tendon. For low relaxation strands this loss may be taken as 5% of the jacking force.

The loss of force in the tendon due to creep and shrinkage is calculated by multiplying together the strain due to these effects, the elastic modulus of the tendon and the total area of the tendons. Creep and shrinkage strains depend on the type of concrete and its environment. In the absence of more specific data, the shrinkage strain may be obtained from Tables 7.9 and 7.10. The strain due to creep is proportional to the stress in the concrete and is calculated by dividing a creep coefficient by the 28-day elastic modulus of the concrete, i.e.

Creep strain = (Creep coefficient /  $E_c$ ) x (stress in the concrete at transfer at the level of the tendons).

**Table 7.9 Elastic modulus of concrete**

Strength class	C30/37	C35/45	C40/50	C50/60
$f_{ck}$ (MPa)	30.0	35.0	40.0	50.0
$E_c$ (GPa)	32.0	33.5	35.0	37.0

**Table 7.10 Shrinkage strains for strength class C35/45**

Location of the member	Relative humidity (%)	Notional size $2A_c/u$ (mm)	
		$\leq 150$	$> 600$
Indoors	50	$465 \times 10^{-6}$	$360 \times 10^{-6}$
Outdoors	80	$285 \times 10^{-6}$	$220 \times 10^{-6}$

Notes

- a  $A_c$  is the cross-sectional area of concrete.
- b  $u$  is the perimeter of the cross-sectional area of concrete.

In the absence of more specific data, the creep coefficient can be obtained from Table 7.11. For unbonded tendons the creep strain should be the mean value averaged over the length of the tendon.

Add together the losses due to relaxation, shrinkage and creep to give the long-term (time-dependent) losses and subtract them from the initial force,  $P_{m,0}$ , to give the final prestressing force,  $P_{m,\infty}$ .

**Table 7.11 Creep coefficients**

Age at loading (days)	Notional size $2A_c/u$ (mm)					
	Indoors (RH = 50%)			Outdoors (RH = 80%)		
	50	150	600	50	150	600
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2.0
28	3.0	2.5	2.0	1.9	1.7	1.5
90	2.4	2.0	1.6	1.5	1.4	1.2

Notes

- a  $A_c$  is the cross-sectional area of concrete.
- b  $u$  is the perimeter of the cross-sectional area of concrete.

#### 7.8.1.4 Tendon spacing

The minimum clear horizontal distance between pretensioned tendons should not be less than twice the diameter of the tendon or the maximum size of the aggregate plus 5mm, nor less than 20mm. In post-tensioned construction, the minimum clear horizontal distance between ducts should not be less than the outside diameter of the sheath or 50mm. Where there are two or more rows the gaps between corresponding tendons or ducts in each row should be vertically in line. In members where internal vibration is being used the horizontal gaps should be wide enough to allow the passage of a poker vibrator.

In pretensioned members, the clear vertical distance between tendons should not be less than twice the diameter of the tendon or the maximum size of the aggregate 10mm.

In post-tensioned construction, the clear vertical distance should not be less than the outside diameter of the sheath or 40mm, nor less than the maximum size of the aggregate plus 5mm.

When tendons are curved, the minimum clear spacing between tendons in the plane of curvature should be increased to prevent local failure of the concrete in accordance with Table 7.12.

**Table 7.12 Minimum distance between centre-lines of ducts in plane of curvature (mm)**

Radius of curvature of duct (m)	Duct internal diameter (mm)													
	25	30	40	45	50	55	60	75	80	100				
	Tendon force (kN)													
(m)	159	229	477	636	916	1113	1603	1908	2748	4351				
2	80	95	180	240	Radii not normally used									
4		85	100	125	175	215	300	355						
6			95	100	125	145	205	245	345	535				
8					110	125	160	190	265	420				
10						120	140	170	215	330				
12							130	160	185	280				
14									180	245				
16									170	230				
18										220				
20	80	85	95	100	110	120	130	160	170	215				

#### Notes

- a The Table is based on reference 32.
- b The tendon force shown is the maximum normally available for the given size of duct.
- c Where tendon profilers or spacers are provided in the ducts, and these are of a type which will concentrate the radial force, the values given in the Table will need to be increased. If necessary reinforcement should be provided between the ducts.
- d The distance for a given combination of duct internal diameter and radius of curvature shown in the Table may be reduced in proportion to the tendon force when this is less than the value tabulated.

## 7.8.2 Post-tensioned anchorages

### 7.8.2.1 Anchorage zones

In post-tensioned members the prestress force is transferred from the tendons to the concrete through anchorage assemblies. The anchorages are supplied as part of the prestressing system and will have been designed by the manufacturer to limit the bearing stress on the concrete to acceptable values, providing the requirements on concrete strength, spacing and the length of straight tendon adjacent to the anchorage specified by the manufacturer are satisfied, see Section 6.4.

The concentrated loads from the anchorages can induce the following tensile forces in the concrete member:

- i) Bursting forces behind the anchorages. These forces act normally to the line of the prestress force in all lateral planes.
- ii) Forces required to maintain overall equilibrium of the anchorage zone.
- iii) Spalling forces on the end face of the anchorage zone.

Reinforcement to resist these forces should be designed at the ultimate limit state as described below. At each point in the anchorage zone the area of reinforcement provided should be that required for the most critical effect, i.e. i) or ii) or iii). The design force from each tendon,  $P_d$ , should be taken as equal to 1.2 x the jacking force.

The resulting tensile forces should be resisted by reinforcement working at a stress  $\leq 300\text{MPa}$  in order to obviate the need to check crack widths.

As well as tensile forces, significant compressive forces occur in anchorage zones and it is essential that the reinforcement is detailed to allow sufficient room for the concrete to be placed and properly compacted.

### 7.8.2.2 Bursting

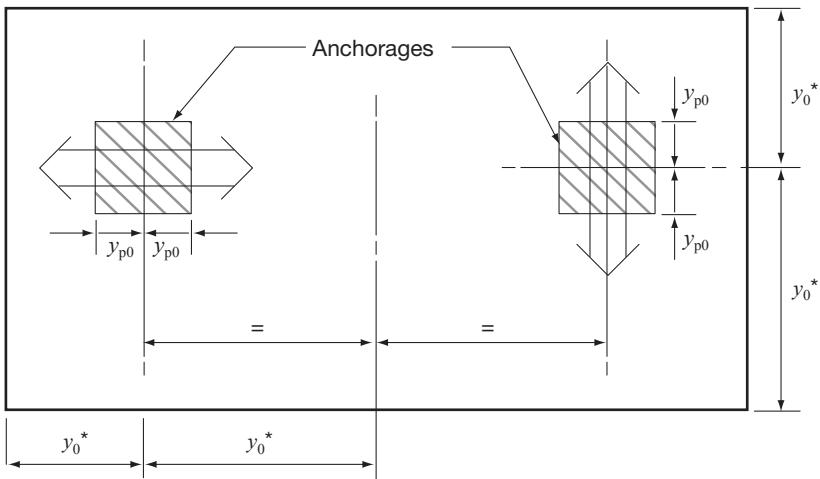
The design bursting tensile force,  $F_{bst}$ , can be derived from Table 7.13. For rectangular anchorages and/or rectangular end blocks, the bursting force should be calculated in each of the two principal directions using the appropriate value of  $y_{p0}/y_0$  for each direction (see Figure 7.6), where  $y_0$  is the half the side of the end block,  $y_{p0}$  is the half the side of the loaded area, and  $P_d$  is the design force in the tendon.

Circular bearing plates should be treated as square plates of equivalent area.

**Table 7.13 Design bursting tensile forces in anchorage zones**

$y_{p0}/y_0$	0.2	0.3	0.4	0.5	0.6	0.7
$F_{bst}/P_d$	0.20	0.18	0.15	0.13	0.10	0.08

Reinforcement to resist this force should be distributed in a region extending from  $0.2y_0$  to  $2y_0$  from the loaded face. The reinforcement should be provided in the form of closed hoops or spirals and be positioned as near as possible to the outer edge of the largest prism whose cross section is similar to and concentric with that of the anchor plate, having regard to the direction in which the load is spreading, and at least 50mm outside the edge of the anchor plate. Where spirals are provided as part of the anchorage system additional reinforcement may still be required to resist the bursting forces.



**Note**

\*Use the smaller of the two values for  $y_0$ .

**Fig 7.6** Definition of  $y_{p0}$  and  $y_0$  for end blocks

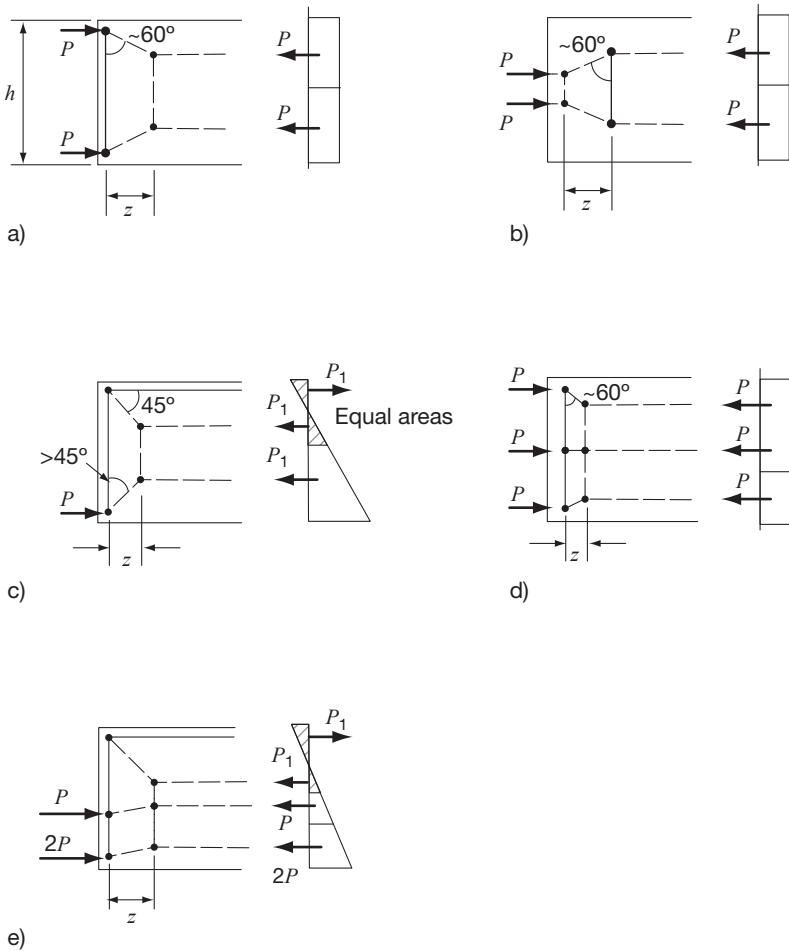
#### 7.8.2.3 Overall equilibrium

Determine the tensile forces required to maintain overall equilibrium of the anchorage zone using strut-and-tie models. Separate models can be used when considering equilibrium in the horizontal and vertical directions, or a three dimensional model may be used. Construct the models by sketching the flow of forces within the anchorage zone and providing notional concrete struts to carry the compressive forces and ties to carry the tensile forces. Calculate the forces in the resulting truss by considering equilibrium.

The model should be chosen to minimise the length of the ties and with the angles between the struts and ties not less than  $45^\circ$  and preferably approximately equal to  $60^\circ$ . Models for some common anchorage zone arrangements are shown in Figure 7.7. For more detailed information the engineer should refer to specialist texts.

Reinforcement to resist the tensile forces should be distributed over a length of  $z/2$  where it is adjacent to a free edge, see Figure 7.7(a), (c), (d) and (e). Where the tensile force occurs within the body of the member, as in Figure 7.7(b), the reinforcement should be distributed over a length equal to  $z$ , where  $z$  is defined in Figure 7.7.

When tendons are to be stressed sequentially, overall equilibrium should be checked at each stage as the tendons are stressed.



#### Notes

- a** Generally  $z = \frac{3}{8} \times \text{depth of section } (h)$ .
- b** Similar principles can be applied to anchorage zones with inclined tendons.

**Fig 7.7** Strut and tie models for the design of anchorage zones

### 7.8.2.4 Spalling<sup>33</sup>

Effectively bonded reinforcement should be placed as close to the loaded face of the anchorage zone as cover requirements allow to resist a force of  $0.04P_d$  in either direction. The reinforcement should be distributed as uniformly as possible over the end face.

When the configuration of the anchorages is such that the prestressing force acts on an unsymmetrical prism see Figure 7.8 and  $d_1 > 2d_2$ , additional spalling stresses are set up and additional reinforcement should be provided close to the loaded face to carry a force equal to:

$$0.2 \left( \frac{d_1 - d_2}{d_1 + d_2} \right)^3 P_d$$

This reinforcement need only be provided in the plane of the unsymmetrical prism and not at right angles to it.

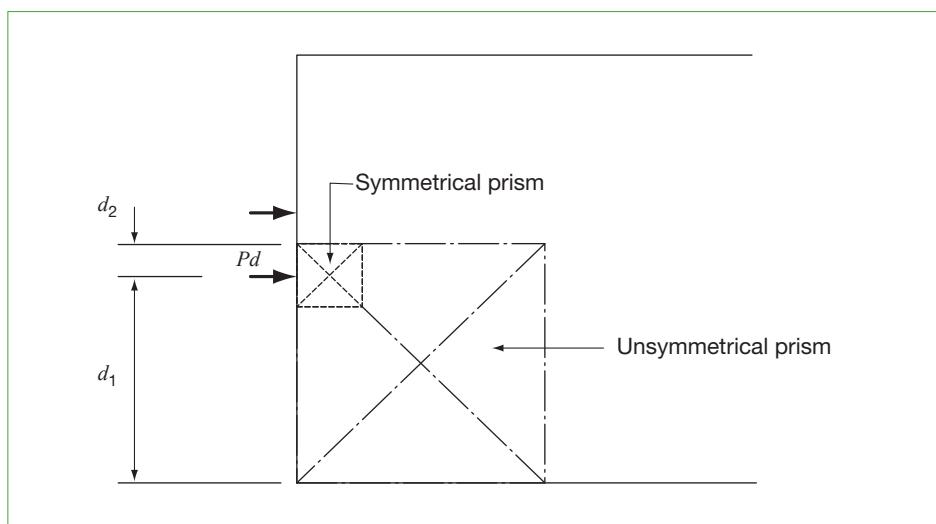
### 7.8.3 Post-tensioned Couplers

A coupler is an anchorage assembly which allows a tendon to be extended. It consists of two parts – an anchorage and the coupling assembly.

The spacing and alignment of tendons at coupler locations should be in accordance with the prestressing system manufacturer's recommendations.

The zone behind the anchorage-side of the coupler should be designed in accordance with Section 7.8.2.

The placing of couplers on more than 50% of the tendons at any cross-section should be avoided. Tendons which are not coupled at a section should not be coupled within 1.5m of that section.



**Fig 7.8** Dimensions used in calculating spalling forces

## 7.9 The next steps

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

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

## 7.10 Reinforcement estimates

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

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

### Slabs – Post-tensioned

The area calculated in section 7.7.5.2, but not less than 0.15% of cross-sectional area longitudinally, and

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

Where:  $h$  is the depth of the end block in metres

$b$  is the width of the end block in metres

$A_p$  is the total area of prestressing tendons anchored within the end block in  $\text{mm}^2$ , and

$f_{pk}$  is the characteristic strength of the tendons in MPa.

### Beams – Shear links

Treat as unprestressed beam (refer to Section 4.10).

### Beams – Post-tensioned

The area calculated in Section 7.7.5.2, but not less than 0.15% (high yield) of cross-sectional area longitudinally, and

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

Where:  $h$  is the depth of the end block in metres

$b$  is the width of the end block in metres

$A_p$  is the total area of prestressing tendons anchored within the end block in  $\text{mm}^2$ , and

$f_{pk}$  is the characteristic strength of the tendons in MPa.

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

- Laps and starter bars
  - A reasonable allowance for normal laps has been made in the previous paragraphs.  
It should however be checked if special lapping arrangements are used.
- Architectural features
  - The drawings should be looked at and sufficient allowance made for the reinforcement required for such ‘non-structural’ features.
- Contingency
  - A contingency of between 10% and 15% should be added to cater for some changes and for possible omissions.

## Appendix A Design data

Design data should include:

- General description, intended use, unusual environmental conditions
- Site constraints
- Design assumptions
- Stability provisions
- Movement joints
- Loading
- Fire resistance
- Durability
- Soil conditions and foundation design
- Performance criteria
- Materials
- Ground slab construction
- Method statement assumed in design
- Quality plan
- Method of analysis
- Computer programs used
- Responsible designer
- Responsible approver
- Health and safety plan
- Risk analyses
- Other data.

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