CODE OF PRACTICE FOR THE STRUCTURAL USE OF CONCRETE—1987

(REPRINTED 1992)

BUILDING AUTHORITY
HONG KONG

FOREWORD

This Code of Practice was originated on the basis of being 'deemed-to-satisfy' the Building (Construction) Regulations as far as concrete design is concerned, and as such replaces the reinforced concrete design rules from the Building (Construction) Regulations issued in 1976. However, this Code includes a wider range of concrete construction than was contained in the Building (Construction) Regulations issued in 1976. Prestressed and precast concrete are included with the reinforced concrete design rules to form a unified Code of Practice for concrete design.

British Standard Codes of Practice BS CP 114, BS CP 115, BS CP 116, BS CP 110 and BS 8110 have been used as the basis for drafting this document, although, only those rules necessary for design and load testing are included. Technical provisions for material quality and workmanship are still covered in the Building (Construction) Regulations.

A draft of this Code of Practice was circulated to selected practising engineers, members of the construction industry and various Government Departments. All comments and views expressed have been taken into consideration in the preparation of this code now published.

It is acknowledged, that preparation of this Code of Practice owes a great deal to the time and effort freely given by Dr. H. C. Chan, Mr. K. M. Cheung, Dr. H. W. Chung, Mr. H. C. Ho, Mr. S. L. Hsu, Mr. C. K. Leung, Mr. Y. W. Liu, Mr. R. L. C. Tsao and Mr. P. C. N. Yim under the direction of Mr. P. L. Wong.

CONTENTS

1. GENERAL

1.1 *SCOPE*

This Code of Practice deals with the structural use of concrete in buildings. In the case of service reservoirs and tanks used for the storage of aqueous liquids the recommendations of this code should be modified by the specific requirements given in BS 5337. This code covers the structural use of reinforced concrete, prestressed concrete and precast concrete, the component materials of which are explicitly specified in the Building (Construction) Regulations. Two design options, namely the working stress method as specified in this Code or the limit state method as given in the alternative recommendations of Clause 7. However, recommendations for the limit state method should not be used with recommendations for working stress design in the same building unless compatibility of the two designs can be demonstrated.

It has been assumed in the drafting of this Code that the design of reinforced, prestressed and precast concrete is entrusted to registered structural engineers, for whose guidance it has been prepared, and that the execution of the work is carried out under proper supervision.

1.2 *BRITISH STANDARDS AND CODES OF PRACTICE*

Any reference to a British Standards Institution publication should be construed as follows:—

(1) where a date is included in the reference, the reference is to the edition of that date, together with any amendments, supplements and addenda published at 30th June, 1986;

(2) where no date is included in the reference, the reference is to the edition current at 30th June, 1986 together with any amendments, supplements and addenda published at that date;

(3) any reference to any publication is a reference to so much only as is relevant in the context in which such a publication is quoted.

2. DESIGN: OBJECTIVES AND GENERAL RECOMMENDATIONS

The purpose of design is to ensure an adequate factor of safety against the structure that is being designed becoming unfit for the use for which it is being designed.

2.1 *BASIC REQUIREMENTS*

2.1.1 STABILITY

The strength of the structure should be sufficient to withstand the design loads taking due account of the possibility of overturning or buckling caused by elastic or plastic instability, having due regard to the effects of sway when appropriate.

The structure should be designed to support loads caused by normal function, but there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. The layout of the structure on plan, and the interaction between the structural members, should be such as to ensure a robust and stable design. No structure can be expected to be resistant to excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause.

In addition, due to the nature of a particular occupancy or use of a structure it may be necessary in the design concept or a design reappraisal to consider the effect of a particular hazard and to ensure that, in the event of an accident, there is an acceptable probability of the structure remaining after the event, even if in a damaged condition.

2.1.2 STIFFNESS

Structural members should possess adequate stiffness to prevent such deflection or deformation as might impair the strength or efficiency of the structure, or produce cracks in finishes or in partitions. The structure as a whole should possess adequate stiffness such that the maximum lateral deflection due solely to wind forces does not exceed 1/500 of the building height.

2.1.3 FIRE RESISTANCE

Structural members should possess the following properties when subjected to fire: retention of structural strength, resistance to penetration of flames, and resistance to heat transmission.

2.1.4 DURABILITY

The concrete cover to the reinforcement and the cement content of the concrete should meet the durability requirements of the structures. Where exceptionally severe environments are encountered, however, additional precautions may be necessary, and specialist literature should be consulted with respect to each particular environment.

2.1.5 VIBRATION

Where there is a likelihood of a structure being subjected to vibration from causes such as wind forces or machinery, measures should be taken to prevent discomfort or alarm, damage to the structure or interference with its proper function. Limits to the level of vibration that may be acceptable are described in specialised literature. In certain circumstances, it may be necessary to isolate the source of vibration or, alternatively, to isolate a part or the whole of the structure. Special consideration may be necessary for flexible elements of the structure.

2.1.6 EFFECTS OF TEMPERATURE, CREEP, SHRINKAGE AND DIFFERENTIAL MOVEMENT

Where the environment or the material of a structure so demands, due consideration should be given to the effects of temperature, creep, shrinkage and differential movement.

2.1.7 FATIGUE

When the imposed load on a structure is predominantly cyclic in character, it may be necessary to consider the effects of fatigue.

2.1.8 OTHER REQUIREMENTS

Structures designed for unusual or special functions should comply with any additional requirements pertaining to the proper functioning of the structures.

2.1.9 BASIS OF DESIGN

The method of design should accord with the laws of mechanics and the general principles relating to the design of reinforced or prestressed concrete. Due account should be taken of the worst combination of loads, stresses and deformations at different construction stages.

2.2 *LOADS*

The design dead, imposed and wind loads should be in accordance with the Building (Construction) Regulations. Other loads such as floatation and earth pressure should be adequately designed for.

For ordinary construction the density of reinforced or prestressed concrete may be taken as 2 400 kg/m^3, but where the amount of steel exceeds 2% some greater weight may be more appropriate.

2.3 *MATERIALS*

The material properties used for the purpose of design should be obtained from Fig. 2.1 to Fig. 2.3 and Table 2.1. Idealised properties adopted in subsequent clauses may also be used.

Table 2.1 Short term elastic modulus of concrete

Strength of Concrete at the Appropriate Age or Stage Considered MPa	Modulus of Elasticity E_c MPa
20	18 900
25	20 200
30	21 700
35	22 900
40	24 000
45	26 000

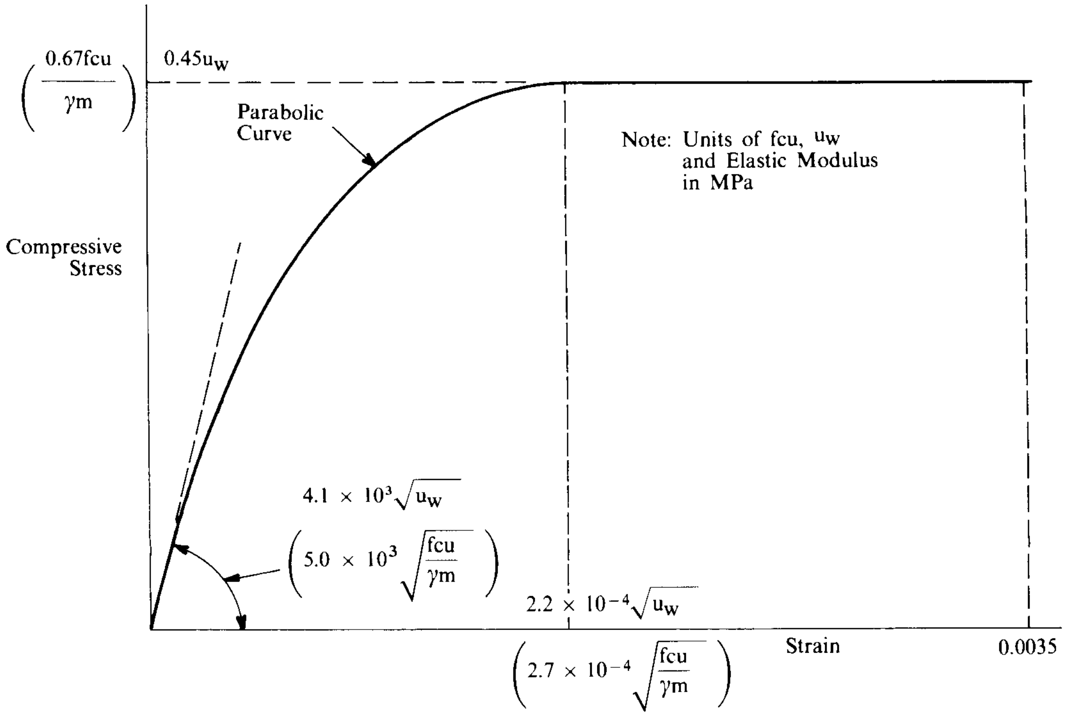


Fig. 2.1 Short term stress-strain curve for normal weight concrete (bracketed values apply to Clause 7)

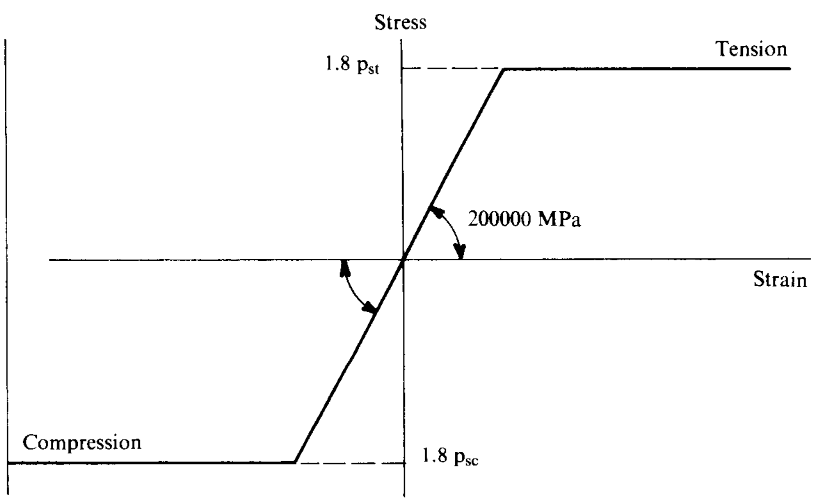


Fig. 2.2 Short term design stress-strain curve for reinforcement

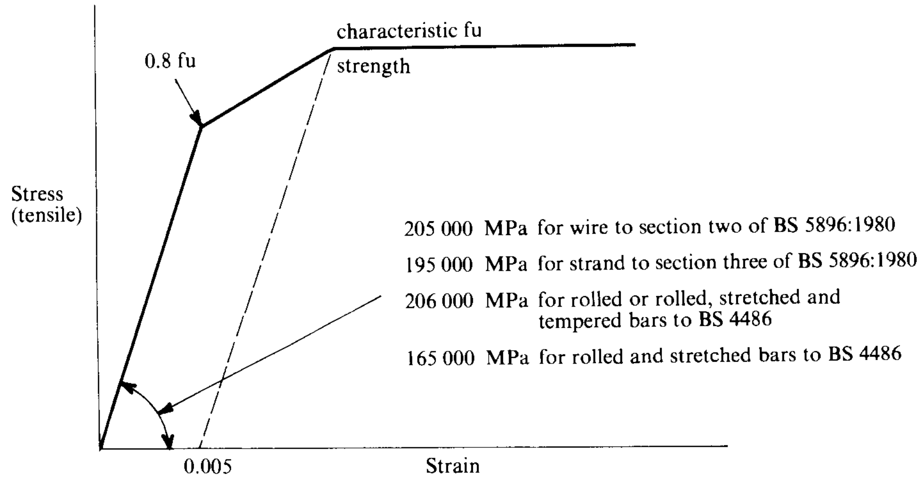


Fig. 2.3 Short term design stress-strain curve for prestressing tendons

3. DESIGN AND DETAILING: REINFORCED CONCRETE

3.1 *GENERAL*

3.1.1 SCOPE

This section gives methods of analysis and design which will in general ensure that, for reinforced concrete structures, the objectives set out in Section 2 are met. Other methods may be used provided that they can be shown to be satisfactory for the type of structure or member considered. In certain cases, the assumptions made in this Section may be inappropriate and a more suitable method should be adopted having regard to the nature of the structure in question.

3.1.2 NOTATION

The notations used in this Code have the following meanings unless otherwise defined:

b is the breadth of the compression flange,

d is the overall depth of the section,

E_c is the modulus of elasticity of concrete,

E_s is the modulus of elasticity of steel,

m is the modular ratio $= E_s/E_c$,

M_r is the moment of resistance of the section,

u_w is the specified grade strength of concrete,

u_t is the specified minimum cube strength at transfer for concrete cubes made in accordance with the requirements of BS1881, but cured under similar conditions to the concrete in the works.

3.1.3 ELASTIC METHOD AND LOAD FACTOR METHOD OF DESIGN OF MEMBERS SUBJECT TO BENDING AND DIRECT FORCE

The elastic theory is concerned with the equilibrium at working stresses of the forces and moments due to the actual loads, the working stresses being the ultimate stresses reduced by a factor of safety.

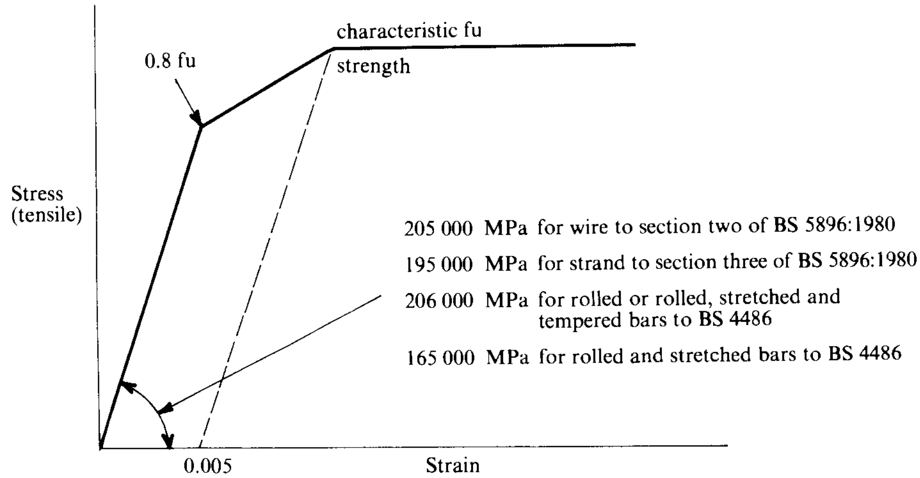
The load factor method is concerned with the equilibrium at ultimate stresses of the forces and moments due to the actual loads multiplied by a load factor. In order to avoid the confusion of having loads and stresses different from the elastic method, the load factor method was modified and introduced in terms of working stresses specified for the elastic theory with the difference that the plastic stress-strain relations for ultimate conditions were to be assumed in place of the elastic relation of Hooke's law appropriate to working loads in the elastic theory.

The method of design should accord with the laws of mechanics and the general principles relating to the design of reinforced concrete.

It may be assumed that:

(1) at any cross-section plane sections remain plane, and

(2) all tensile stresses are taken by the reinforcement except that the concrete may be assumed to resist diagonal tension within the limits of shear stress specified for concrete in Clause 3.1.4.



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The method of design should accord with the laws of mechanics and the general principles relating to the design of reinforced concrete.

It may be assumed that:

(1) at any cross-section plane sections remain plane, and

(2) all tensile stresses are taken by the reinforcement except that the concrete may be assumed to resist diagonal tension within the limits of shear stress specified for concrete in Clause 3.1.4.

The strength of members may be assessed by the commonly employed elastic theory which makes the further assumption that steel and concrete are elastic within the range of the permissible stresses given in Clauses 3.1.4 and 3.1.5, and that the modular ratio m is equal to 15.

Alternatively an inelastic analysis based on the short-term stress-strain curves derived from the design strengths of materials given in Clause 2.3 or the load-factor method described in Clause 3.2.6 and Sub-clause 3.3.2(4) may be adopted, in which the basic requirement is that there should be a suitable load factor (i.e. the ratio of the ultimate strength of the member to its design load). The design recommendations given in this Code may be taken as applicable whichever method is used except where indicated otherwise.

3.1.4 PERMISSIBLE STRESSES IN CONCRETE

The compressive, shear and bond stresses in reinforced concrete should not exceed those shown as appropriate for each grade of concrete in Table 3.1.

Table 3.1 Permissible stresses in concrete

Grade of Concrete	Compressive		Shear		Bond	
	Direct p_{cc}	Due to bending p_{cb}	Flexural p_v	Torsional p_t	Average p_{ba}	Local p_{bl}
	MPa	MPa	MPa	MPa	MPa	MPa
20	5.0	6.7	0.67	0.16	0.80	1.20
25	6.3	8.3	0.77	0.17	0.90	1.34
30	7.5	10.0	0.87	0.18	1.00	1.47
35	8.8	11.7	0.90	0.19	1.00	1.50
40	10.0	13.3	0.90	0.20	1.00	1.50
45	11.3	15.0	0.90	0.21	1.00	1.50

For modifications to the permissible stresses given in this Table see:
Clause 3.1.6 for wind forces
Sub-clause 3.1.10(10) for bond stresses where deformed bars are used
Sub-clause 3.2.1(2) for slender beams

3.1.5 PERMISSIBLE STRESSES IN REINFORCEMENT

(1) General

The tensile and compressive stresses in steel reinforcement should not exceed those shown as appropriate for each designation of stress in Table 3.2.

(2) Tensile stress

In determining the permissible tensile stress, regard should be given to the need for avoiding undesirable cracking. Lower limits than those given in Table 3.2 may have to be adopted in circumstances of exposure to corrosive influences.

(3) Compressive stresses

Compressive stresses in reinforcement in beams or slabs may be calculated as follows:

(*a*) As giving assistance to the concrete, using the elastic theory or, alternatively, on a load-factor basis. In the former case the steel stress should be 15 times the stress in the concrete at the same distance from the neutral axis provided the steel stress does not exceed that set out in Table 3.2. For steel stresses when the load-factor basis of calculation is used, see Clause 3.2.6.

(*b*) As taking the whole compression, when the stresses given in Table 3.2 should be used.

3.1.6 INCREASE OF PERMISSIBLE STRESSES DUE SOLELY TO WIND FORCES

The permissible stresses in concrete and in the reinforcement may exceed those given in Clauses 3.1.3 and 3.1.4 respectively by not more than 25% provided that:

(1) such excess is solely due to stresses induced by wind loading, and

(2) in no case does the stress in the reinforcement exceed 250 MPa.

Where the stress-strain curves in Figs. 2.1 and 2.2 are used to assess the strength of the section, u_w, p_{sc} and p_{st} may be replaced by 1.25 u_w, 1.25 p_{sc} and 1.25 p_{st} respectively provided that 1.25 p_{st} will not exceed 250 MPa.

Table 3.2 Permissible stresses in steel reinforcement

Type of Stress	Grade 250 steel to BS 4449	Grade 460/425 steel to BS 4449 and all steel complying with BS 4461 or BS 4482
	MPa	MPa
Tensile stress other than in shear reinforcement (p_{st})	140	230
Tensile stress in shear reinforcement (p_{sv})	140	175
Compressive stress (p_{sc})	125	175

For modifications to the permissible tensile stresses given above, see:
Clause 3.1.6 for wind forces.

For modifications to the permissible compressive stresses given above, see:
Sub-clause 3.1.5(3) for beams or slabs designed on elastic theory,
Clause 3.1.6 for wind forces,
Clause 3.2.6 for beam or slab sections designed on a load-factor basis.

3.1.7 COVER

Subject to fire resistance requirements or other statutory requirements, reinforcement should have concrete cover and the thickness of such cover (exclusive of plaster or other decorative finish) should be:

(1) for each end of a reinforcing bar, not less than 25 mm nor less than 2 times the diameter of such bar;

(2) for a longitudinal reinforcing bar in a column, not less than 40 mm nor less than the diameter of such bar. In the case of columns with a minimum dimension of 200 mm or under, whose bars do not exceed 12 mm diameter, 25 mm cover may be used;

(3) for a longitudinal reinforcing bar in a beam, not less than 25 mm nor less than the diameter of such bar;

(4) for tensile, compressive, shear or other reinforcement in a slab, not less than 15 mm nor less than the diameter of such reinforcement;

(5) for any other reinforcement not less than 15 mm nor less than the diameter of such reinforcement;

3.1.8 DISTANCE BETWEEN BARS

The horizontal distance between two parallel steel reinforcements in reinforced concrete should usually, except at splices, be not less than the greatest of the three following distances:

(1) the diameter of either bar if their diameters be equal;

(2) the diameter of the larger bar if their diameters be unequal;

(3) 5 mm more than the nominal maximum size of the coarse aggregate used in the concrete.

A greater distance should be provided where convenient. Where immersion vibrators are intended to be used, however, the horizontal distance between bars of a group may be reduced to 2/3 of the nominal maximum size of the coarse aggregate provided that a sufficient space is left between groups of bars to enable the vibrator to be inserted; this would normally be a space of 75 mm.

The vertical distance between two horizontal main steel reinforcements, or the corresponding distance at right angles to two inclined main steel reinforcements, should be not less than 15 mm or the nominal maximum size of aggregate, whichever is the greater, except at splices or where one of such reinforcements is transverse to the other.

When the overall depth of a beam exceeds 750 mm, longitudinal bars should be provided over a distance of 2/3 of the overall depth from the tension face. This reinforcement should be positioned near the side faces and be spaced at not more than 250 mm. The area of these side bars at each face should not be less than 0.2% for Grade 250 steel and 0.12% for Grade 460/425 steel, of the cross-sectional area of the beam. The cross-sectional area of the beam may be taken as the overall depth times the breadth of the rib.

The pitch of the main bars in a reinforced concrete solid slab should be not more than 3 times the effective depth of such slab.

The pitch of distribution bars in a reinforced concrete solid slab should be not more than 5 times the effective depth of such slab.

3.1.9 STIFFNESS OF MEMBERS

(1) General

Reinforced concrete should possess adequate stiffness to prevent such deflection or deformation as might impair the strength or efficiency of the structure, or produce cracks in finishes or in partitions.

For all normal cases it may be assumed that the stiffness will be satisfactory if, for members with steel stresses not more than 140 MPa and concrete stresses not more than 10 MPa, the ratio of span to overall depth does not exceed the values given in Table 3.3; if, for members with steel stresses greater than 140 MPa or concrete stresses greater than 10 MPa, the ratio of span to overall depth does not exceed 90% of the values given in Table 3.3; and if, for members with steel stresses greater than 140 MPa and concrete stresses greater than 10 MPa, the ratio of span to overall depth does not exceed 85% of the values given in Table 3.3.

Table 3.3 Permissible values of span/depth ratio of beams and slabs

	Ratio of span to overall depth
Beams	
Simply supported beams	20
Continuous beams	25
Cantilever beams	10
Slabs	
Slabs spanning in one direction, simply supported	30
Slabs spanning in one direction, continuous	35
Slabs spanning in two direction, simply supported	35
Slabs spanning in two direction, continuous	40
Cantilever slabs	12

(2) Moment of inertia

For the purposes of calculating bending moments in continuous structures, the moment of inertia may be estimated by considering:

(*a*) the entire concrete section, ignoring the reinforcement; or

(*b*) the entire concrete section, including the reinforcement, on the basis of the modular ratio, or

(*c*) the compression area of the concrete section, combined with the reinforcement on the basis of the modular ratio.

Whichever method is adopted for the beams the same method should be used for the columns.

3.1.10 BOND AND ANCHORAGE

(1) Bars in tension

A bar in tension should extend from any section for a distance to the end of the bar such that the average bond stress does not exceed the permissible bond stress given in Clause 3.1.3. This condition will be satisfied if the length measured from such section is not less than:

$$\text{the bar diameter} \times \frac{\text{the tensile stress in the bar}}{\text{4 times the permissible average bond stress}}$$

The bar should extend at least 12 bar diameters beyond the point at which it is no longer required to resist stress.

For the purpose of this clause, the length of the bar so determined may have deducted from it a length equivalent to the value of the hook as given in Sub-clause 3.1.10(5) but no deduction should then be made for the length of the bar contained in the hook.

(2) Local bond stress

The local bond stress may exceed the permissible average bond stress given in Clause 3.1.4, but should not at any point exceed the permissible local bond stress given in that clause.

$$\text{Local bond stress } f_{bl} = \frac{V}{d_l o}$$

Where V is the total shear across the section,
d_l is the effective depth to the tensile reinforcement;
o is the sum of the perimeters of the bars in the tensile reinforcements.

In members of variable depth the effect of the change in depth should be taken into account in calculating the bond stress.

(3) Hooks and other anchorages

Hooks and other anchorages of reinforcement should be of such form, dimensions and arrangement as will ensure their adequacy without over-stressing the concrete or other anchorage material.

(4) Dimensions of hooks

Where hooks are formed in Grade 250 steel bars; the internal radius of the bend should be at least 2 times the diameter of the bar except where the hook fits over a main reinforcing or other adequate anchor bar, when the radius of the bend may be reduced to that of such bar. The length of straight bar beyond the end of the curve should be at least 4 times the diameter of the bar.

Where hooks are formed in Grade 460/425 bars, the internal radius of the bend should be at least 3 times the diameter for bars less than 25 mm in diameter and 4 times the diameter for bars of 25mm or more in diameter. The length of straight bar beyond the end of the curve should be at least 4 times the diameter of the bar.

(5) Anchorage value of bends

A bend in a reinforcing bar may be assumed to have an anchorage value equivalent to a length of bar equal to 4 times the diameter of the bar for each 45 degrees through which the bar is bent; provided that:

(*a*) the radius of the bend be not less than 2 times the diameter of the bar;

(*b*) the length of the straight part of the bar beyond the end of the curve be at least 4 times the diameter of the bar;

(*c*) whatever be the angle through which the bar is bent, the assumed anchorage value should not be taken as more than equivalent to a length of bar equal to 16 times the diameter of the bar.

Thus, a U-hook may be credited with a resistance equivalent to that of a straight bar of a length of 16 bar diameters, and an L-hook with 8 diameters.

(6) Bearing stresses in bends

In bends in reinforcing bars, the local stress on the concrete may be increased to 3 times the value permitted in Clause 3.1.4 for the concrete in direct compression.

(7) Links in beams and transverse ties in columns

Notwithstanding any of the provisions of this Code, in the case of links and transverse ties, complete bond length and anchorage may be deemed to have been provided when the bar is bent through an angle of at least 90° round a bar of at least its own diameter; and the link or tie is continued beyond the end of the curve for a length of at least 8 diameters or, alternatively, through an angle of 180° with the link or tie continued beyond the end of the curve for a length of at least 4 bar diameters.

(8) Bars in compression

A bar in compression should extend from any section for a distance such that the average bond stress does not exceed the permissible bond stress given in Clause 3.1.4 by more than 25%. This condition will be satisfied if the length measured from such section is not less than:

$$\text{the bar diameter} \times \frac{\text{the compressive stress in the bar}}{\text{5 times the permissible average bond stress}}$$

The bar should extend at least 12 bar diameters beyond the point at which it is no longer required to resist stress.

(9) Laps in bars

(*a*) *General.* Laps in bars in any member should be staggered.

(*b*) *Bars in tension.* The length of laps in bars in tension should be not less than:

$$\text{the bar diameter} \times \frac{\text{the tensile stress in the bar}}{\text{4 times the permissible average bond stress}}$$

or 30 bar diameters, whichever is the greater.

(*c*) *Bars in compression.* The length of lap in bars in compression should be not less than:

$$\text{the bar diameter} \times \frac{\text{the compressive stress in the bar}}{\text{5 times the permissible average bond stress}}$$

or 24 bar diameters, whichever is the greater.

(10) Deformed bars

For deformed bars, the bond stresses given in Clause 3.1.4 may be increassed by 25%.

(11) Shear reinforcement

All bent up bars acting as shear reinforcement should be fully anchored in both flanges of the beam, the anchorage length being measured from the end of the sloping portion of the bar.

3.1.11 CONTINUITY JOINTS IN REINFORCEMENT

(1) General

Reinforcement may be jointed by welding provided that the types of steel (including 'weldable' and 'readily weldable' reinforcement at defined in BS 4449 and BS 4461) have the required welding properties, or with a mechanical device. The joints should occur, if possible, away from points of high stress or at bends in reinforcement, and should preferably be staggered so that not more than 50% of the joints occur at any one point. For joints to be considered as staggered, the distance between them must not be less than the end anchorage length of the bar. Welded joints should not be used where the imposed load is predominantly cyclic in nature. Where the stress in the bar at the joint is entirely compressive, the load may be transferred by end bearing of square sawn-cut ends held in concentric contact by suitable sleeves or mechanical devices.

(2) Welded joints

Where the strength of the weld had been proved by tests to be at least as strong as the parent bar, the permissible tensile and compressive forces of joints may be taken respectively as 80% of the permissible tensile force and 100% of the permissible compressive force of the bar provided that the welding operations are carried out under strict supervision or special welding technique are employed. In both cases not more than 20% of the tensile reinforcement at any cross-section should be welded. Special consideration should be given to the design of welded lapped joints.

(3) Mechanical devices

The detailed design of the sleeve and the method of manufacture and assembly should be such as to ensure that the ends of the two bars can be accurately aligned into the sleeve. The strength and deformation characteristics should be determined by tests the results of which should be used as basis for assessment of permissible loads. Where there is a risk of threaded connection working loose, e.g. during vibration of in-situ concrete, a locking device should be provided. The concrete cover provided for the sleeve should not be less than that specified for normal reinforcement.

3.2 *BEAMS AND SLABS*

3.2.1 GENERAL

(1) Effective span

The effective span, L, of a beam or slab should be taken as the lesser of the two following:

(*a*) the distance between the centres of bearings; or

(*b*) the clear distance between supports plus the effective depth of the beam or slab, the effective depth being the distance between the centre of tension and the edge of the compression section.

(2) Slender beams

Where the length L of a beam between lateral restraints exceeds 30 times the breadth b of its compression flange, the maximum depth of beam which may be considered in design should not exceed 8 times this breadth, and the maximum compressive stress in the concrete should not exceed the product of the permissible compressive stress due to bending given in Clause 3.1.4 and the appropriate coefficient given in Table 3.4. Intermediate values of the coefficient may be obtained by linear interpolation.

Table 3.4 Stress reduction coefficient for slender beams

Slenderness ratio L/b	30	40	50	60
Coefficient	1.00	0.75	0.50	0.25

The permissible stresses in compression reinforcement should be reduced in the same ratio.

Where a beam is subjected to load in the direction of its length, the reduction coefficient should be modified as follows. If the ratio of the bending moment to the load is less than 5d (where d is the overall depth) the coefficient should be that given for columns in Table 3.9; if this ratio is greater or equal to 5d the coefficient should be that for beams given in Table 3.4;

In slender beams, the shear resistance of the concrete should be ignored, the whole shearing resistance being provided by shear reinforcement.

(3) Minimum reinforcement in slabs

In solid reinforced concrete slabs the area of tensile reinforcement, expressed as a percentage of the gross cross-sectional area of the concrete, should not be less than:

0.25 where plain bars are used; or
0.15 where deformed bars, or
high-yield wire-mesh, are used.

The amount of reinforcement provided at right angles to the main reinforcement, expressed as a percentage of the gross cross-sectional area of the concrete, should not be less than:

0.15 where plain bars are used; or
0.12 where deformed bars, or
high-yield wire-mesh, are used.

(4) Compression reinforcement in beams

The compression reinforcement should be effectively anchored in two directions at right angles over the distance where it is required to act in compression, at points not further apart, centre to centre, than 12 times the diameter of the anchored bar. The subsidiary reinforcement used for this purpose should pass round, or be hooked over, both the compressive and tensile reinforcement.

The amount of steel in compression should preferably not exceed 4% but, if it does, only 4% should be allowed for in the calculation of the resistance moment of the beam. This percentage should be calculated as follows:

(*a*) in rectangular beams, on the total cross-sectional area;

(*b*) in T-beams or L-beams, on an area equal to the total depth multiplied by width of the rib.

(5) T-Beams

In T-beams the breadth of the flange assumed as taking compression should not exceed the least of the following:

(*a*) 1/3 of the effective span of the T-beam;

(*b*) the distance between the centres of the ribs of the T-beams;

(*c*) the breadth of the rib plus 12 times the thickness of the slab.

(6) L-beams

In L-beams, the breadth of the flange assumed as taking compression should not exceed the least of the following:

(*a*) 1/6 of the effective span of the L-beams;

(*b*) the breadth of the rib plus 1/2 of the clear distance between ribs;

(*c*) the breadth of the rib plus 4 times the thickness of the slab.

When a part of a slab is considered as the flange of a T-beam or L-beam, the reinforcement in the slab transverse to the beam should cross the full breadth of the flange. Where the slab is assumed to be spanning independently in the same direction as the beam, such transverse reinforcement should be near the top surface of the slab.

The quantity of such reinforcement should be related to the shear stress in the slab produced by its acting as the compression member of the T-beam or L-beam and should not be less than 0.3% of the longitudinal cross-sectional area of the flange.

(7) Effect of wear

If the surface of a concrete slab is not adequately protected by a suitable finish against the effect of wear, an appropriate addition should be made to the structural thickness required.

3.2.2 BENDING MOMENTS

Bending moments in beams and slabs should be calculated for the effective span and all loading thereon.

The bending moments to be provided for at a cross-section of a continuous beam or slab should be the maximum positive and negative moments at such cross-section, allowing, in both cases, if so desired, for the reduced moments due to the width of the supports, for the following arrangements of imposed loadings:

(1) alternate spans loaded and all other spans unloaded;

(2) any two adjacent spans loaded and all other spans unloaded;

Nevertheless, except where the approximate values for bending moments given in assumption (3) of Clause 3.2.3 are used, the negative moments at the supports for any assumed arrangement of loading may each be increased or decreased by not more than 10 per cent, provided that these modified negative moments are used for the calculation of the corresponding moments in the spans.

The computation of bending moments in beams and slabs is dealt with in Clauses 3.2.3 and 3.2.4.

3.2.3 BENDING MOMENTS IN BEAMS AND SLABS SPANNING IN ONE DIRECTION

The bending moments in beams and slabs spanning in one direction may be calculated on one of the following assumptions:

(1) Beams may be designed as members of a continuous framework, with monolithic connection between the beams and columns, and the bending moments calculated taking into account the resistance of the columns to bending. Where beams are framed into external columns they should be designed to resist bending moments in combination with the columns in conformity with Sub-clause 3.3.2(2).

(2) Beams and slabs may be designed as continuous over supports and capable of free rotation about them. Nevertheless, where the supports to beams or slabs are monolithic with them and stiff in relation to them, it is preferable to design the beams or slabs with due regard to such stiffness.

(3) Unless more exact estimates are made, the bending moments in uniformly loaded beams and slabs continuous over three or more approximately equal spans may be assumed to have the values given in Table 3.5.

Two spans may be considered as approximately equal when they do not differ by more than 15% of the longer span.

For the purpose of calculating moments in beams or slabs in a monolithic structure, it will usually be sufficiently accurate to assume that members connected to the ends of such beams or slabs are fixed in position and direction at the ends of such members remote from their connections with the beam or slab.

Table 3.5 Approximate values of bending moments in uniformly loaded beams and slabs continuous over three or more approximately equal spans

	Near middle of end span	At support next to end support	At middle of interior spans	At other interior supports
Moment due to dead load	$+\frac{W_dL}{12}$	$-\frac{W_dL}{10}$	$+\frac{W_dL}{24}$	$-\frac{W_dL}{12}$
Moment due to imposed load	$+\frac{W_sL}{10}$	$-\frac{W_sL}{9}$	$+\frac{W_sL}{12}$	$-\frac{W_sL}{9}$

Where W_d is the total dead load per span;
W_s is the total imposed load per span.

3.2.4 BENDING MOMENTS IN SLABS SPANNING IN TWO DIRECTIONS AT RIGHT ANGLES WITH UNIFORMLY DISTRIBUTED LOADS

The design of solid slabs spanning in two directions at right angles, and of their supporting beams, should be based on one of the three methods given below.

(1) Method 1

A purely theoretical analysis based on elastic theory may be made.

The bending moments in the slabs and beams may be calculated on the assumption that the slabs act as perfectly elastic thin plates, Poisson's ratio being assumed equal to zero. The resistance moment of the slabs and beam sections should also be calculated by the commonly employed elastic theory with $m = 15$.

(2) Method 2

In this method the assessment of bending moments in slabs and beams is based on theoretical analysis amplified and adjusted in the light of experimental data, the resistance moments of the slab and beam sections being calculated by the commonly employed elastic theory, with $m = 15$. The recommendations given in (*a*), (*b*) and (*c*) below may be adopted.

(*a*) *Slabs simply supported on four sides.* Where in the case of a simply supported slab, adequate provision is not made to resist torsion at the corners of the slab and to prevent the corners from lifting, the bending moments at mid-span should be assumed to have the values given by the following equations:

$$M_x = \alpha_x wL_x^2$$
$$M_y = \alpha_y wL_x^2$$

Where M_x and M_y are the bending moments at mid-span on strips of unit width and spans L_x and L_y respectively,
w is the total load per unit area,
L_y is the length of the longer side,
L_x is the length of the shorter side,
α_x and α_y are coefficients shown in Table 3.6.

Table 3.6 Bending moment coefficients for slabs spanning in two directions at right angles simply supported on four sides

L_y/L_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

(*b*) *Slabs restrained on four sides.*

(i) Where the corners of a slab are prevented from lifting and adequate provision for torsion in accordance with (v) below is made, the bending moments may be assumed to have the values given in (iii) below.

(ii) Slabs are considered as being divided in each direction into middle strips and edge strips, the middle strip having a width of 3/4 of the width of the slab and each edge strip having a width of 1/8 of the width of the slab, except that, for slabs for which the ratio of the sides L_y/L_x exceeds 4.0, the middle strip in the short direction should be taken to have a width of L_y–L_x and each edge strip a width of $L_x/2$.

(iii) The maximum bending moments per unit width in the middle strip of a slab are given by the following equations:

$$M_x = \beta_x wL_x^2$$
$$M_y = \beta_y wL_x^2$$

Where M_x and M_y are the maximum bending moments on strips of unit width in the direction of spans L_x and L_y respectively,
w is the total load per unit area,
L_y is the length of the longer side,
L_x is the length of the shorter side;
β_x and β_y are coefficients given in Table 3.7.

(iv) No reinforcement parallel to the adjacent edges of the slab need be inserted in the edge strips above that required to comply with Clause 3.1.8, Sub-clauses 3.2.1(3) and (v) below.

(v) Torsion reinforcement should be provided at the corners of a slab except at corners contained by edges over both of which the slab is continuous.

Table 3.7 Bending moment coefficients for rectangular panels supported on four sides with provision for storsion at corners

Type of panel and moments considered	Short span coefficients β_x								Long span co-efficients β_y for all values of $\frac{L_y}{L_x}$
	Values of $\frac{L_y}{L_x}$								
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0 or more	
Case 1. Interior panels.									
Negative moment at continuous edge	0.033	0.040	0.045	0.050	0.054	0.059	0.071	0.083	0.033
Positive moment at mid-span	0.025	0.030	0.034	0.038	0.041	0.045	0.053	0.062	0.025
Case 2. One short or long edge discontinuous.									
Negative moment at continuous edge	0.041	0.047	0.053	0.057	0.061	0.065	0.075	0.085	0.041
Positive moment at mid-span	0.031	0.035	0.040	0.043	0.046	0.049	0.056	0.064	0.031
Case 3. Two adjacent edges discontinuous.									
Negative moment at continuous edge	0.049	0.056	0.062	0.066	0.070	0.073	0.082	0.090	0.049
Positive moment at mid-span	0.037	0.042	0.047	0.050	0.053	0.055	0.062	0.068	0.037
Case 4. Two short edges discontinuous.									
Negative moment at continuous edge	0.056	0.061	0.065	0.069	0.071	0.073	0.077	0.080	
Positive moment at mid-span	0.044	0.046	0.049	0.051	0.053	0.055	0.058	0.060	0.044
Case 5. Two long edges discontinuous.									
Negative moment at continuous edge									0.056
Positive moment at mid-span	0.044	0.053	0.060	0.065	0.068	0.071	0.077	0.080	0.044
Case 6. Three edges discontinuous (one short or long edge continuous).									
Negative moment at continuous edge	0.058	0.065	0.071	0.077	0.081	0.085	0.092	0.098	0.058
Positive moment at mid-span	0.044	0.049	0.054	0.058	0.061	0.064	0.069	0.074	0.044
Case 7. Four edges discontinuous.									
Positive moment at mid-span	0.050	0.057	0.062	0.067	0.071	0.075	0.081	0.083	0.050

At corners contained by edges over neither of which the slab is continuous, top and bottom reinforcement should be provided for torsion at the corners of the slabs. Both top and bottom reinforcement should consist of two layers of bars placed parallel to the sides of the slab and extending in these directions for a distance of 1/5 of the shorter span. The area of the bars in each of the 4 layers, per unit width of the slab, should be 3/4 of the area required for the maximum positive moment in the slab.

At corners contained by edges over only one of which the slab is continuous, the torsional reinforcement may be reduced to 1/2 of that required by the preceeding paragraph.

Any reinforcement provided for the purpose of complying with other clauses of this Code may be included as part of the reinforcement required to comply with this clause.

(vi) Where a slab ends and there is monolithic connection between the slab and the supporting beam or wall, provision should be made for the negative moments that may occur in the slab at such support. The negative moment to be assumed in these cases depends on the degree of fixity afforded to the edge of the slab, but for general purposes it may be taken as 2/3 of the moment given in Table 3.7 for the mid-span of the slab.

(*c*) *Loads on supporting beams.* The loads on the supporting beams may be assumed to be in accordance with Fig. 3.1

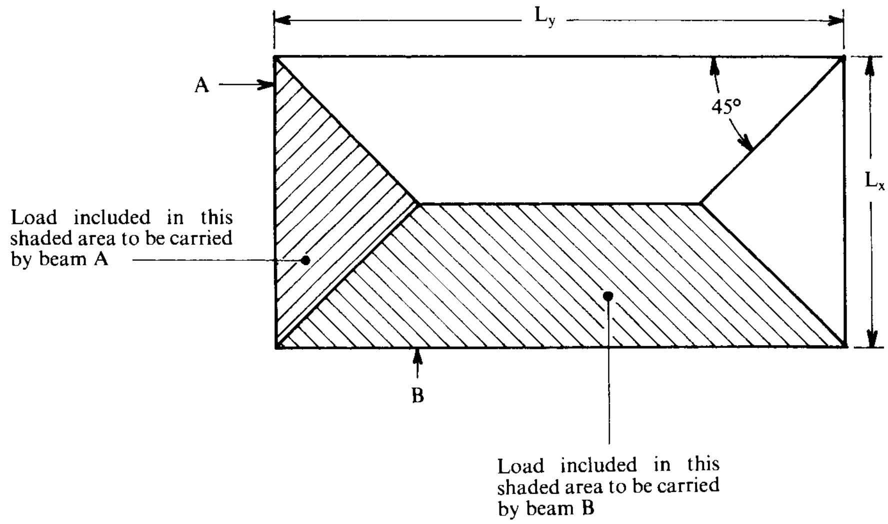


Fig. 3.1 Diagram showing the load carried by supporting beams

(3) Method 3

This method is based on the load-factor method of design. The slabs and beams may be designed to have a load factor generally of 1.8; in the calculations of the ultimate strength, however, the cube strength of the concrete should be taken as only 3/5 of the actual cube strength. This requirement should be complied with in the following way. The ultimate bending moments to be allowed for should be deduced from analysis in which the load is 1.8 times the working (dead and imposed) load and due regard is given to redistribution of moments that would occur before failure of the slab or beam, by the use of Johansen's yield-line theory or other acceptable method; the resistance moments of the slab and beam sections should be calculated in accordance with the recommendations of Clause 3.2.6; and these resistance moments should be equal to at least 55% of the ultimate bending moments at failure.

3.2.5 TRIMMING TO 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 impaired by the opening. Due regard should be paid to the possibility of diagonal cracks developing at the corners of openings.

3.2.6 RESISTANCE TO BENDING

The strength of a section should be assessed by an elastic analysis as described in Clause 3.1.3, or by an inelastic analysis based on the short-term stress-strain curves derived from the design strengths of materials given in Clause 2.3, or by the load factor method described below. Where an inelastic analysis is employed, the depth of the compression zone should not be assumed to exceed 1/2 of the effective depth of the beam or slab, and the load factor (i.e. the ratio of the ultimate strength of the beam or slab to its working load) should not be less than 1.8.

(1) Basis of load factor method

The basic requirement of this method is a suitable load factor. This method does not involve a knowledge of or use of the modular ratio and does not assume a linear relationship between the stress and strain in the concrete. It assumes instead that, as failure is approached, the compressive stresses will adjust themselves to give a total compression greater than that deduced from the elastic theory, the extent of this adjustment having been determined from tests to destruction. These tests have shown that the stress distribution in the concrete at failure may be assumed to be equivalent to a constant compressive stress of 2/3 of the cube strength of the concrete acting over a depth of the beam or slab sufficient to provide a total compression which, if no compressive reinforcement is provided, is equal to the total tension afforded by the tensile reinforcement acting at its yield stress provided that this depth is not assumed to exceed 1/2 of the effective depth of the beam or slab.

The resistance moments of beam and slab sections may be calculated to have a load factor generally of 1.8; in the calculations of the ultimate strength, however, the cube strength of the concrete should be taken as only 3/5 of the actual cube strength. It is necessary also to ensure that the stresses at working loads are not such as to cause excessive cracking. These requirements should be complied with by calculating the resistance moment (corresponding to the working loads) at any cross-section on the following assumptions:

(*a*) The stress in the tensile reinforcement does not exceed the permissible stress appropriate to the particular steel, given in Clause 3.1.5.

(*b*) The compressive stress in the concrete is 2/3 of the permissible compressive stress in the concrete in bending considered to be uniform over the whole part of the area of the concrete section which is in compression. The depth of concrete in compression should not, however, be considered to exceed 1/2 of the effective depth.

(*c*) The stress in the compressive reinforcement does not exceed the permissible stress, appropriate to the particular steel, given in Clause 3.1.5 nor does it exceed:

$$390\left(1 - \frac{d_2}{d_n}\right)\text{ MPa},$$

where d_2 denotes the depth to the compressive reinforcement and d_n denotes the depth of the concrete in compression.*

* At a failure of a beam of slab, the maximum compressive strain in the concrete has been shown by tests to be such that a reinforcing bar at the surface of the beam would develop a stress of 700 MPa with steel of the requisite yield stress. With a load factor of 1.8, the limiting stress to be used in design is thus 390 MPa at the compressive surface. Since tests show that the strain is roughly linear across the section, the limiting compressive stress at the depth d_2 is therefore

$$390\left(1 - \frac{d_2}{d_n}\right)\text{ MPa}.$$

(2) Simplified formulae for rectangular beam and slab sections

For beams and solid slabs of rectangular cross-section without compressive reinforcement and for qualities of concrete and steel within the range permitted by this Code, these requirements may be deemed to be satisfied if the resistance moment M_r (corresponding to the working loads) is assumed to be the lesser of the two values calculated from the following equations:

Based on the tensile reinforcement:

$$M_r = A_{st} p_{st} l_a$$

Based on the strength of the concrete in compression:

$$M_r = \frac{p_{cb}}{4} b d_l^{\,2}$$

Where l_a is the lever arm which may by taken as:

$$d_l - \frac{3A_{st}p_{st}}{4bp_{cb}}$$

Where A_{st} is the area of tensile reinforcement,
p_{st} is the permissible tensile stress in the reinforcement,
p_{cb} is the permissible compressive stress in the concrete in bending,
b is the breadth of the section;
d_l is the effective depth to the tensile reinforcement.

Where it is necessary for the resistance moment to exceed $\frac{p_{cb}}{4} bd_1{}^2$, compressive reinforcement should be provided so that:

$$M_r = \frac{p_{cb}}{4} bd_1{}^2 + A_{sc}p_{sc}(d_1 - d_2)^{\dagger}$$

Where A_{sc} is the area of compressive reinforcement,
p_{sc} is the permissible compressive stress in the reinforcement as given in assumption (*c*) of Sub-clause (1),

and the area of tensile reinforcement should be such that the stress in this steel does not exceed the permissible stress.

† No allowance has been made in this formula for the small reduction in concrete area by the amount desplaced by the compressive reinforcement, having regard to the fact that the coefficient to $p_{cb}bd_1{}^2$ is deduced from experiments, the results of which are always somewhat variable.

(3) Simplified formulae for T-beams or L-beams

For T-beams or L-beams with a breadth of flange b, a rib width b_r and a depth of slab forming the flange d_s, the resistance moment when compressive reinforcement is not provided may by assumed to be the lesser of the two values given by the following equations:

Based on the tensile reinforcement:

$$M_r = A_{st}p_{st}(d_1 - \frac{d_s}{2})$$

Based on the strength of the concrete in compression:

$$M_r = M_F p_{cb} bd_1{}^2$$

Where the factor M_F has the values given in Table 3.8.

Table 3.8 Values of M_F* for computing moment of resistance based on the strength of the concrete in compression

b/b_r	Values of M_F for d_1/d_s					
	2 or less	3	4	5	6	∞
1	0.25	0.25	0.25	0.25	0.25	0.25
2	0.25	0.22	0.20	0.185	0.175	0.125
4	0.25	0.20	0.17	0.15	0.14	0.062
6	0.25	0.195	0.165	0.14	0.125	0.042
8	0.25	0.19	0.16	0.135	0.12	0.031
∞	0.25	0.185	0.145	0.12	0.10	0

Where it is necessary for the resistance moment to exceed $M_F p_{cb} bd_1{}^2$, compressive reinforcement should be provided so that:

$$M_r = M_F p_{cb} bd_1{}^2 + A_{sc}p_{sc}(d_1 - d_2)^{\dagger}$$

and the area of tensile reinforcement should be such that the stress in this steel does not exceed the permissible stress.

* For intermediate values of b/b_r and d_1/d_s, the value of M_F can be calculated from the following formula:

$$M_F = \frac{b_r}{4b} + \frac{1}{3}[1 - \frac{b_r}{b}][2\frac{d_s}{d_1} - (\frac{d_s}{d_1})^2]$$

† No allowance has been made in this formula for the small reduction in concrete area by the amount displaced by the compressive reinforcement, having regard to the fact that the coefficient to $p_{cb}bd_1{}^2$ is deduced from experiments, the results of which are always somewhat variable.

(4) Deflection of beams and slabs

The use of the method of design permitted by this clause can lead to reduced depths of beam and slab sections as compared with those determined from the method based on the elastic theory. It is therefore particularly important to check that the members possess adequate stiffness as provided for in Sub-clause 3.1.9(1).

3.2.7 RESISTANCE TO SHEAR

(1) General

(*a*) The shear stress, v, at any cross-section in a reinforced concrete beam or slab is given by equation:

$$v = \frac{V}{bd_l}$$

Where V is the total shearing force across the section,
b is the breadth of a rectangular beam which for a T-beam or L-beam should be replaced by the breadth of the rib b_r;
d_l is the effective depth to the tensile reinforcement.

(*b*) Where at any cross-section the shear stress, v, exceeds the permissible shear stress for the concrete, the whole shearing force at that cross-section should be provided for by the tensile resistance of the shear reinforcement acting in proper combination with the compression in the concrete. Moreover, even with the whole shearing force so provided for, the shear stress v should not exceed 3.5 times the permissible shear stress for the concrete.

(*c*) Where at any cross-section the shear stress v is less than the permissible shear stress p_v, for the concrete but is greater than $0.5p_v$, reinforcement should be provided at that cross-section to resist 1/2 of the shear force V.

(*d*) Where at any cross-section the shear stress v is less than $0.5p_v$, nominal shear reinforcement should be provided at that cross-section. Neither the longitudinal spacing nor the lateral spacing of the vertical legs of the shear reinforcement should exceed a distance equal to $0.75d_l$. The cross-sectional area of the nominal shear reinforcement at any particular section of a member should be not less than 0.2% (0.12% for Grade 460/425 steel) of the horizontal area of the concrete at that section.

(*e*) Nominal shear reinforcement need not be provided in slabs, footings, bases, pile-caps and members of minor importance provided that, at every cross-section, the shear stress, v, is less than $0.5p_v$. Shear reinforcement should be provided for other values of shear stress as given above.

(2) Shear reinforcement

(*a*) A link in reinforced concrete should pass round, or be otherwise adequately secured to, the appropriate tensile reinforcement, and such link should be anchored adequately in the compression zone.

(*b*) Tensile reinforcement which is inclined and carried through a depth of beam equal to the arm of the resistance moment will also act as shear reinforcement provided it is anchored sufficiently.

(*c*) Where two or more types of shear reinforcement are used in conjunction, the total shearing resistance of the beam may be assumed to be the sum of the shearing resistances computed for each type separately.

(*d*) The spacing of links when required to resist shear should not be less than 8 times the diameter of the link, or 75 mm whichever is the greater. The resistance to shear V is given by equation:

$$V = \frac{p_{sv}A_{sv}d_l}{s}$$

Where p_{sv} is the permissible tensile stress in the shear reinforcement,
A_{sv} is the area of cross-section of the links per unit spacing,
d_l is the effective depth of the section;
s is the spacing of the links.

(*e*) The resistance to shear at any section of a beam, reinforced with inclined bars, may be calculated on the assumption that the inclined bars form the tension members of one or more single systems of lattice girders in which the concrete forms the compression members. The shear resistance at any vertical section should then be taken as the sum of the vertical components of the tension and compression forces cut by the section. Care must be taken that such assumptions do not involve greater stresses in the horizontal bars than the permissible stresses.

3.2.8 RESISTANCE TO TORSION

(1) General

(*a*) The torsional shear stress v_t at any section should be calculated assuming a plastic stress distribution. For rectangular sections v_t is given by equation:

$$v_t = \frac{2T}{b^2(d - \frac{b}{3})}$$

Where T is the torsional moment,
b is the smaller dimension of the section;
d is the larger dimension of the section.

T-, L- or I- sections may be treated by dividing them into their component rectangles. This should normally be done so as to maximise the function $\Sigma(b^3d)$ which will generally be achieved if the widest rectangle is made as long as possible. The torsional shear stress carried by each component rectangle may be calculated by treating them as rectangular sections subjected to a torsional moment of:

$$T\left(\frac{b^3d}{\Sigma b^3d}\right)$$

Box sections in which wall thickness exceed 1/4 of the overall thickness of the member in the direction of measurement may be treated as solid rectangular sections. For other sections specialist literature should be consulted.

(*b*) Where the torsional shear stress v_t exceeds the value p_t from Table 3.1, reinforcement should be provided. In no case should the sum of the shear stresses resulting from shear force and torsion $(v+v_t)$ exceed 3.5 times the permissible shear stress p_v for the concrete nor, in the case of small sections ($B <$ 550 mm) should the torsional shear stress, v_t, exceed $3.5p_vB/550$ mm, where B is the larger dimension of a link.

(2) Torsion reinforcement

(*a*) Torsion reinforcement should consist of rectangular closed links together with longitudinal reinforcement. This reinforcement is additional to any requirement for shear or bending and should be such that:

$$\frac{A_{sv}}{s} \geqslant \frac{T}{0.8\,AB\,p_{sv}}$$

$$A_{tl} \geqslant \frac{A_{sv}}{s}\left(\frac{p_{sv}}{p_{st}}\right)(A + B)$$

Where A_{sv} is the area of the legs of closed links at a section,
A_{tl} is the area of longitudinal reinforcement,
p_{sv} is the permissible tensile stress in shear reinforcement (see Table 3.2),
p_{st} is the permissible tensile stress other than in shear reinforcement (see Table 3.2),
s is the spacing of the links,
A is the smaller dimension of the links;
B is the larger dimension of the links.

(*b*) The spacing of links should not exceed the least of A, B/2 or 200 mm and the links should be of a closed type as shown in Fig. 3.2.

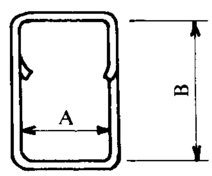


Fig. 3.2 Closed type link for torsional reinforcement

(*c*) The extra longitudinal reinforcement should be distributed evenly round the inside perimeter of the links. The clear distance between these bars should not exceed 300 mm and at least 4 bars, one in each corner of the links, should be used. Additional longitudinal reinforcement required at the lever of the tension or compression reinforcement may be provided by using larger bars than those required for bending alone. The torsion reinforcement should extend a distance at least equal to the largest dimension of the section beyond where it ceases to be required.

(*d*) In the component rectangles of T-, L-, or I- sections, the reinforcement cages should be detailed so that they interlock and tie the component rectangles of the section together. Where the torsional shear stress in a minor component rectangle is less than the permissible torsional shear stress, no torsion reinforcement need be provided in that rectangle.

3.2.9 DISTRIBUTION OF CONCENTRATED LOADS ON SLABS

Allowance should be made for the bending moments due to concentrated loads, using methods based on the elastic theory, such as those of Pigeaud or Westergaard, or other acceptable method. Alternatively, allowance should be based on the load-factor method of design on the same basis as given for slabs with uniform loading in Sub-clause 3.2.4(3).

If a slab is simply supported on two opposite edges and carries one or more concentrated loads in a line in the direction of the span, it should be designed to resist the maximum bending moment caused by the loading system. Such bending moment may be assumed to be resisted by an effective width of slab (measured parallel to the supports) as follows:

(1) For solid slabs, the effective width may be taken as the sum of the load width and $2.4x(1 - x/L)$ where x is the distance from the nearer support to the section under consideration and L is the span.

(2) For other slabs, except where specially provided for, the effective width will depend on the ratio of the transverse and longitudinal flexural rigidities of the slab. Where these are approximately equal, the value for the effective width as given for solid slabs may be used, but as the ratio decreases a smaller value should be taken. The minimum value which need be taken, however, is the load width plus $4(x/L)(1 - x/L)$ metres where x and L have the same meanings as in (1); so that, for a section at midspan, the effective width is equal to 1 m plus the load width.

(3) Where the concentrated load is near an unsupported edge of a slab the effective width should not exceed the value in (1) or (2) above as appropriate, nor 1/2 that value plus the distance of the centre of the load from the unsupported edge as shown in Fig. 3.3.

The critical sections for shear should be taken to be at a distance from the edges of the concentrated load equal to the effective depth of the slab. Due allowance should be made for openings or unsupported edges close to the critical section.

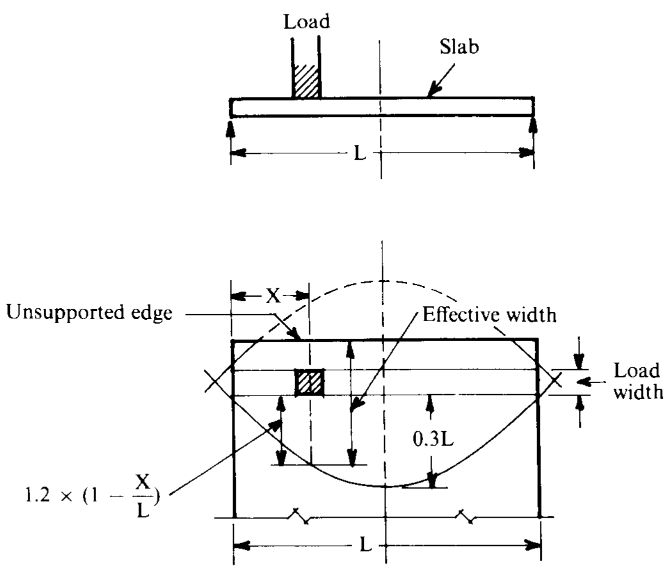


Fig. 3.3 Effective width of solid slab carrying a concentrated load near an unsupported edge

3.2.10 FLOORS AND ROOFS OF RIBBED AND HOLLOW BLOCK CONSTRUCTION

(1) General

This type of construction consists of a series of reinforced concrete ribs cast in-situ between blocks which remain part of the completed floor or on forms which may be removed after the concrete has set.

(2) Blocks and forms

Blocks and forms may be of any suitable material which will retain its shape and dimensions and is strong enough to support the concrete when placed.

Blocks which are required to remain as part of the slab and to contribute to its structural strength should be of concrete or burnt clay and should have a crushing strength of at least 17.5 MPa measured on the net section when axially loaded in a direction corresponding with that in which they will function in the floor slab. Burnt clay blocks should also comply with BS 3921, 'Clay bricks and blocks'.

(3) Topping

The tops of the ribs may be connected by a topping of concrete cast in-situ over the blocks or forms. The concrete used for the topping should be of the same quality as that used for the ribs.

(4) Calculation of resistance moments

In determining the bending resistance of hollow block construction, the blocks may be neglected. Alternatively, they may be assumed to act in structural combination with the ribs and topping (when used), provided that the blocks are properly jointed with a 1:3 cement-sand mortar or that a topping of at least 30 mm is used.

Where the thickness of the top of hollow blocks composed of material other than concrete is regarded as contributing to the structural strength of the floor slab, the permissible working stress on the blocks should not exceed 1/5 of their crushing strength.

For the purpose of calculation, the elastic modulus of the material forming the block may be assumed to be the same as for concrete.

(5) Resistance to shear

Where the blocks are considered as adding to the strength of the floor, the thickness of one wall of the block may be added to the thickness of the rib. Alternatively, the walls of both the adjacent blocks may be taken into account, using a shear stress appropriate to the material.

(6) Thickness of topping

When topping is used the thickness, after allowance has been made for the effect of wear if necessary, should not be less than the thicknesses given in (*a*) to (*c*) below for various conditions:

(*a*) In floors with permanent blocks regarded as contributing to the strength of the construction, and with a clear distance between the ribs not exceeding 450 mm, 30 mm.

When the blocks are properly jointed the minimum thickness may be reduced to 25 mm.

(*b*) In floors with permanent blocks not regarded as contributing to the strength of the construction, 40 mm or 1/12 the clear distance between the ribs, whichever is the greater.

(*c*) In all other cases, 50 mm.

(7) Size and spacing of rib

The width of the rib should be not less than 65 mm. The depth, excluding any topping, should not be more than four times the width. The spacing should be not more than 1 m centre to centre.

(8) Reinforcement of in ribs

(*a*) *General.* At least 50% of the total main tensile reinforcement should be carried through at the bottom on to the bearing and effectively anchored.

In floors continuous over supports, it may sometimes be impracticable to provide sufficient reinforcement to develop the full support moment on the basis of continuity. Such floors may be treated as simply supported and the reinforcement in the span determined accordingly. If so treated, it is desirable to provide reinforcement over the support to prevent cracking; it is recommended that such reinforcement should have a cross-sectional area of not less than 1/4 of that in the middle of the adjoining bays and should extend at least 1/10 of the clear spans into the adjoining bays.

(*b*) *Spacing.* Provided that both the permissible bond and compressive stresses in the rib (below the topping where used) are reduced by 40%, the lateral spacing of bars running parallel in the concrete ribs may be reduced to 12 mm or the diameter of the bar, whichever is greater.

(*c*) *Cover to reinforcement.* For hollow tile slabs having slip tiles not less than 12 mm thick under reinforced ribs, a cover of 12 mm should be given to the bars above the tiles.

(9) Supports parallel to ribs

Where a slab reinforced in one direction only is built into a wall, or rests on a beam, parallel to the ribs, a rib should be placed along the wall or beam, the minimum width of such rib being that of the bearing. Consideration should be given to the necessity for some reinforcement at right angles to the ribs.

Where a slab butts against a wall parallel with the ribs, there should be a rib against the wall at least 50 mm wide.

3.3 *COLUMNS*

3.3.1 REINFORCEMENT IN COLUMNS

(1) Longitudinal reinforcement

A reinforced concrete column should have longitudinal steel reinforcement, and the cross-sectional area of such reinforcement should not be less than 1% nor more than 8% of the gross cross-sectional area of the column required to transmit all the loading in accordance with this Code.

It should be noted that the use of 8% of steel may involve serious practical difficulties in the placing and compacting of concrete and a lower percentage would be recommended. Where bars from the column below have to be lapped with those in the column, the percentage of steel should usually not exceed 4%.

A reinforced concrete column having helical reinforcement should have at least six bars of longitudinal reinforcement within this helical reinforcement. The longitudinal bars should be in contact with the helical reinforcement and equidistant around its inner circumference. Circular columns should have similar arrangement of reinforcement.

For laps in spliced longitudinal bars see Sub-clause 3.1.10(9).

The bars should be not less than 12 mm in diameter.

(2) Transverse reinforcement

(*a*) *General.* A reinforced concrete column should have transverse reinforcement so disposed as to provide restraint against the buckling of each of the longitudinal reinforcements. Links should be so arranged that every corner and alternate bar in an outer layer of reinforcement is supported by a link passing round the bar and having an included angle of not more than 135°. All other bars within a compression zone should be within 150 mm of a restrained bar. For circular columns, where the longitudinal reinforcement is located round the periphery of a circle, adequate lateral support is provided by a circular tie passing round the bars.

(*b*) *Pitch.* The pitch of transverse reinforcement should be not more than the least of the 3 following distances:

1. the least lateral dimension of column;
2. 12 times the diameter of the smallest longitudinal reinforcement in the column;
3. 300 mm.

(*c*) *Helical reinforcement.* Helical reinforcement should be of regular formation, with the turns of the helix spaced evenly, and its ends should be anchored properly. Where an increased load on the column on account of the helical reinforcement is allowed for under Sub-clause 3.3.2(1)(*c*), the pitch of the helical turns should be not more than 75 mm or more than 1/6 of the core diameter of the column, nor less than 25 mm nor less than 3 times the diameter of the steel bar forming the helix. In other cases the requirements of (*b*) above should be complied with.

(*d*) *Diameter.* The diameter of the transverse reinforcements should be not less than 1/4 the diameter of the largest main bar, and in no case less than 5 mm.

3.3.2 PERMISSIBLE LOADS ON COLUMNS

(1) Axially loaded columns

(*a*) *Short columns defined.* Columns may be treated as short columns where the ratio of the effective column length to least lateral dimension does not exceed 15. The maximum permissible stresses for these should be as specified in Clauses 3.1.4 and 3.1.5.

(*b*) *Short columns with lateral ties.* The axial load P_o permissible on a short column reinforced with longitudinal bars and lateral ties should not exceed that given by equation (1):

$P_o = p_{cc} A_c + p_{sc} A_{sc}$ .... (1)

Where p_{cc} is the permissible stress for the concrete in direct compression,
A_c is the cross-sectional area of concrete excluding any finishing material and reinforcing steel,
p_{sc} is the permissible compression stress for column bars;
A_{sc} is the cross-sectional area of the longitudinal steel.

(*c*) *Short columns with helical reinforcement.* Where helical reinforcement is used, the axial load permissible on a short column should not exceed that given by equations (1) or (2) whichever is the greater:

$P_o = p_{cc} A_k + p_{sc} A_{sc} + 185A_b$ .... (2)

Where A_k is the cross-sectional area of concrete in the core excluding the area of longitudinal reinforcement,
A_b is the equivalent area of helical reinforcement (volume of helix per unit length of the column);
185 is the stress in MPa.

The sum of the terms $p_{cc}A_k + 185A_b$ should not exceed $0.5\ u_w A_c$ where u_w is the specified grade strength of the concrete.

(*d*) *Long columns.* The axial load permissible on a reinforced concrete column or part thereof, having a ratio of effective length to least lateral dimension between 15 and 57, should not exceed that which results from the multiplication of the appropriate permissible load specified for a short column in (*b*) or (*c*) above by the coefficient shown as appropriate for each ratio of effective column length to least lateral dimension in Table 3.9. Intermediate values of the coefficient may be obtained by linear interpolation.

When, in a column having helical reinforcement, the permissible load is based on the core area; the least lateral dimension should be taken as the diameter of the core.

Table 3.9 Reduction coefficient for loads on long columns

Ratio of effective length to least lateral dimension of column	Coefficient
15	1.0
18	0.9
21	0.8
24	0.7
27	0.6
30	0.5
33	0.4
36	0.35
39	0.3
42	0.25
45	0.2
48	0.15
51	0.1
54	0.05
57	0.0

Table 3.10 Effective column length

Type of column	Effective column length
Properly restrained at both ends in position and direction	0.75L.
Properly restrained at both ends in position and imperfectly restrained in direction at one end or both ends	A value intermediate between 0.75 and L depending upon the efficiency of the directional restraint.
Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end	A value intermediate between L and 2L depending upon the efficiency of the imperfect restraint.

(e) *Determination of ratio of effective length to least lateral dimension of a column.* For the purpose of this clause, the effective column length given in Table 3.10 should be used, where L is the length of the column from floor to floor, or between adequately restrained supports. The effective column length values given in thes table are in respect of typical cases only and embody the general principles which should be employed in assessing the appropriate value for any particular column.

(2) Bending moments in columns

Bending moments in internal columns supporting an approximately symmetrical arrangement of beams and loading need not be calculated.

Bending moments in external columns and in internal columns supporting an arrangement of beams and loading not approximately symmetrical should be calculated and provided for.

The expressions given in Table 3.11 below may be used for estimating the moments:

Table 3.11 Moments in columns

	Moments for frames of one bay	Moments for frames of two or more bays
External (and similarly loaded) columns Moment at foot of upper column	$M_e \dfrac{K_u}{K_L + K_u + 0.5K_b}$	$M_e \dfrac{K_u}{K_L + K_u + K_b}$
Moment at head of lower column	$M_e \dfrac{K_L}{K_L + K_u + 0.5K_b}$	$M_e \dfrac{K_L}{K_L + K_u + K_b}$
Internal columns Moment at foot of upper column		$M_{es} \dfrac{K_u}{K_L + K_u + K_{b1} + K_{b2}}$
Moment at head of lower column		$M_{es} \dfrac{K_L}{K_L + K_u + K_{b1} + K_{b2}}$

Where M_e is the bending moment at the end of the beam framing into the column, assuming fixity at both ends of the beam,
M_{es} is the maximum difference between the moments at the ends of the two beams framing into opposite sides of the column, each calculated on the assumption that the ends of the beams are fixed and assuming one of the beams unloaded,
K_b is the stiffness of the beam,
K_{b1} is the stiffness of the beam on one side of the column,
K_{b2} is the stiffness of the beam on the other side of the column,
K_L is the stiffness of the lower column;
K_u is the stiffness of the upper column.

For the purposes of this table, the stiffness of a member may be obtained by dividing the moment of inertia of a cross-section by the length of the member, provided that the member is of constant cross-section throughout its length.

The equations for the moment at the head of the lower column may be used for columns in a topmost storey by taking K_u as zero.

Where the bending moment is calculated in the internal columns it is permissible to take into account the reduction in load resulting from the beam on one side of the column being fully loaded and the beam on the other side being loaded with dead load only.

(3) Columns subject to both direct load and bending

The permissible combinations of direct load and bending moment to which a short column may be subjected should be determined on the basis of either the commonly employed elastic theory, with a modular ratio of 15, and the permissible stresses in bending given in Clauses 3.1.4 and 3.1.5; or by an inelastic analysis based on the short-term stress-strain curves derived from the design strengths of materials given in Clause 2.3 with a load factor not less than 1.8 or the load-factor method described in d below.

The permissible combination of direct load and bending moment to which a long column may be subjected should not exceed the values appropriate to a short column multiplied by the appropriate reduction coefficient from Table 3.9.

Where, due to bending, the maximum stresses in the column occur at the ends of the column, the permissible load may be determined without reference to the reduction coefficient for sections within 1/8 of the column length L from the centre line of the beams.

When the elastic theory is used the load on a column subject to both direct load and bending moment should not exceed that permissible for an axially loaded column.

(4) Load-factor method of design for short columns subject to both direct load and bending

In the load-factor method the column is designed to have a load factor generally of 1.8; in the strength calculations however, the cube strength of the concrete should be taken as only 0.67 times the actual cube strength. It should be assumed that the maximum concrete strain in compression does not exceed 0.0035 at failure; that the compressive stress distribution in the concrete at failure is rectangular, parabolic or such other shape as is shown by tests to be reasonable; that the maximum stress in the concrete at failure does not exceed 2/3 of the cube strength of the concrete; and that the maximum stresses in the reinforcement at failure do not exceed 1.8 times the premissible stresses. It is necessary also to ensure that the stresses at working loads are not such as to cause excessive cracking.

These requirements will be satisfied, for columns of rectangular section with symmetrical reinforcement and placed only in the faces parallel to the axis of bending, if the following rules are adopted:

The section should be assumed to be controlled by compression when the load exceeds P_b given by equation (3):

$$P_b = p_{cc} b d_1 X - A_{sc}(p_{st} - p_{sc}) \quad \text{.......} \quad (3)$$

Where p_{cc} is the permissible stress for the concrete in direct compression given in Clause 3.1.4,
b is the breadth of the column,
d_1 is the effective depth to the tensile reinforcement,
A_{sc} is the area of the compressive reinforcement, which for the conditions of bending is equal to 1/2 of the total area of reinforcement in the column,
P_{sc} and p_{st} are the permissible stresses in the reinforcement given in Clause 3.1.5 for compression and tension, respectively;

$$X = \frac{595}{700 + 1.8p_{st}}$$

At the load P_b as defined by equation (3) the corresponding eccentricity of load e_b relative to the centre of the section is given by equation (4):

$$P_b\left(e_b + \frac{d_1 - d_2}{2}\right) = p_{cc} b d_1{}^2 X\left(1 - \frac{1}{2}X\right) + A_{sc} p_{sc}(d_1 - d_2) \quad \text{.......} \quad (4)$$

Where d_2 is the depth to the compressive reinforcement.

When the section is controlled by compression, the permissible load P on the column is related to the permissible load P_o for an axially loaded column, as given by equation (1) and the eccentricity e of the load P relative to the centre of the section, given by equation (5):

$$P = \frac{P_o}{1 + \left(\frac{P_o}{P_b} - 1\right)\frac{e}{e_b}} \quad \text{.......} \quad (5)$$

When the applied load is less than P_b the section is controlled by tension and the permissible load is given by equation:

$$P = p_{cc} b d\left[\left(0.5 - \frac{e}{d} - Y\right) + \sqrt{\left(0.5 - \frac{e}{d} - Y\right)^2 + r\frac{p_{sc}}{p_{cc}}\left(\frac{d_1 - d_2}{d}\right) + Y\left(2\frac{d_1}{d} - Y\right)}\right]$$

in which $Y = \frac{r}{2}\left(\frac{p_{st} - p_{sc}}{p_{cc}}\right)$

Where $r = \frac{\text{total area of reinforcement}}{bd}$

and d is the overall depth of the column section.

The above rules may be modified as follows to give greater premissible loads when the section is controlled by compression. The relationship between permissible loads and bending moments may be assumed to be linear between the load P_o for an axially loaded column and a load $P_{bl} = 0.59p_{cc}bd_l$; and linear between this latter load and the load P_b given by equation (3). The eccentricity c_{bl} corresponding to the load P_{bl} may be obtained from equation (4) by replacing P_b by P_{bl}, e_b by e_{bl} and X by 0.59 (and the same substitutions in equation (5)) may be used to obtain the permissible loads for the range $P_o > P > P_{bl}$). For loads between P_{bl} and P_b the permissible load is given by equation:

$$P = \frac{e_{bl} - e_b}{(e_{bl} - e)\frac{1}{P_b} - (e_b - e)\frac{1}{P_{bl}}}$$

For columns of rectangular section with symmetrical reinforcement subject to axial compression and biaxial bending, the permissible axial compression is given by equation:

$$\frac{1}{P} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o}$$

Where P is the permissible axial compression under biaxial bending,
P_x is the permissible axial compression when only eccentricity e_x is present,
P_y is the permissible axial compression when only eccentricity e_y is present;
P_o is the permissible axial compression.

This relation is adequate provided that P is not less than 10% of P_o. If it is less, sufficient accuracy is obtained by designing the member for flexure only.

3.4 *WALLS*

3.4.1 GENERAL

A reinforced wall is a vertical load bearing concrete member whose greatest lateral dimension is more than 4 times its least lateral dimension. Reinforced concrete walls should be designed generally in accordance with the recommendations given for columns. The cross-sectional area of the vertical reinforcement should not be less than 0.4% of the gross cross-sectional area of the wall. The bars should not be less than 10 mm in diameter and the distance between two vertical bars should not exceed 300 mm. The cross-sectional area of the lateral reinforcement parallel to the wall face should not be less than 0.25% in the case of Grade 460/426 or 0.3% in the case of Grade 250 steel of the gross cross-sectional area of the wall. The bars should not be less than 1/4 of the size of the vertical bars nor 6 mm, in diameter. The vertical distance between two horizontal bars should not exceed 300 mm. The provisions of Sub-clause 3.3.1(2) with regard to transverse reinforcement to restrain the vertical bars against buckling need not be taken to apply to walls in which the vertical bars are not assumed to assist in resisting compression. Where transverse links are required, the vertical and horizontal spacing of links should not exceed 400 mm or 2 times the wall thickness, whichever is the lesser. Any vertical compression bar not enclosed by a link should be within 200 mm of a restrained bar. The vertical spacing should not exceed 16 times the diameter of the smallest vertical reinforcement. The links should not be less than 1/4 of the size of the largest compression bar nor 6 mm in diameter. The wall thickness should not be less than 125 mm.

3.4.2 PERMISSIBLE LOADS

The permissible load on any storey height should be calculated in the general manner specified for columns (omitting, however, the contribution of the vertical reinforcement if transverse reinforcement satisfying Sub-clause 3.3.1(2) is not provided).

When the effective height of the wall exceeds 15 times the wall thickness, the permisible load should be reduced to allow for the effect of slenderness of the wall. The reduction coefficients given in Table 3.9 for columns should be used for this purpose, the value of the slenderness ratio to be adopted in the first column of this table being the ratio between the effective height of the wall and the wall thickness.

The effective height of the wall should be determined as for columns in accordance with Sub-clause 3.3.2(2)(*e*). Where, as may occasionally happen, the wall is stiffened by closely spaced cross walls such that the length of wall between adjacent cross walls is les than the effective height, the slenderness ratio may be assumed to be the ratio of this length to the wall thickness.

The effective width of a reinforced concrete wall subjected to concentrated loads, should not exceed the distance measured from centre to centre between the concentrated loads or the width of bearing plus 4 times the wall thickness on each side of the concentrated load, whichever is the smaller dimension.

A wall should be effectively supported laterally to prevent buckling by horizontal reinforced concrete slabs of adequate thickness; or beams spaced with a clear spacing of not more than a distance of 8 times the thickness of the wall; or reinforced concrete cross walls. Where the wall is stiffened by closely spaced cross walls such that the length of wall between adjacent cross walls is less than the effective height, the slenderness ratio should be assumed to be the ratio of this length of the wall thickness.

3.5 *BASES AND PILE CAPS*

3.5.1 BENDING MOMENTS

The bending moments at any section of a base or pile cap for a reinforced concrete column or wall should be taken to be the moment of the forces over the entire area on one side of the section. The critical section for bending in the base or pile cap should be taken at the face of the column or wall.

3.5.2 REINFORCEMENT

The reinforcement provided to resist the bending moments specified in Clause 3.5.1 should be distributed uniformly across the full width of the section; except that, in rectangular bases or pile caps for columns, the reinforcement parallel to the short edge should be more closely spaced near the column.

3.5.3 SHEAR

The critical sections for shear in bases should be taken to be at a distance from the column faces equal to the effective depth of the base. The shear resistance of pile caps should be checked for corbel action in accordance with Clause 5.2.1.

3.5.4 BOND

The critical section for local bond stress should be taken to be the same section as the critical section for bending moment, i.e. at the face of the column or wall.

3.6 *STAIRCASES*

3.6.1 DISTRIBUTION OF LOADING

In general the load should be assumed to be uniformly distributed over the plan area of a staircase. When, however, staircases surrounding open wells include two spans which intersect at right angles, the load on the areas common to both spans may be assumed to be divided equally between the two spans.

When staircases or landings, which span in the direction of the flight, are built at least 110 mm into walls along part or all of their length, a 150 mm strip adjacent to the wall may be deducted from the loaded area.

3.6.2 EFFECTIVE BREADTH OF STAIRCASES

The effective breadth of a staircase without stringer beams should normally be taken as the actual breadth of the staircase. When a staircase is built into a wall along part or all of its span, 2/3 of the embedded breadth up to a maximum of 80 mm should be included in the effective breath.

3.6.3 EFFECTIVE SPAN OF STAIRCASE

When a staircase without stringer beams is built monolithically at its ends into structural members spanning at right angles to the span of the staircase, the effective span should be taken as the sum of the clear horizontal distance between the supporting members and 1/2 the breadths of the supporting members subject to maximum additions of 900 mm at both ends.

When a staircase without stringer beams is simply supported the effective span should be taken as the horizontal distance between the centre lines of the supports.

For the purpose of this sub-clause a staircase may be taken to include a section of landing spanning in the same direction and continuous with the stair flight.

3.7 *FLAT SLAB*

3.7.1 GENERAL

The term flat slab means a reinforced concrete slab with or without drops, supported, generally without beams, by columns with or without flared column heads (see Fig. 3.4). A flat slab may be a solid slab or may have recesses formed on the soffit so that the soffit comprises a series of ribs two directions. The recesses may be formed by removable or permanent filler blocks.

3.7.2 METHODS OF DESIGN

Flat slabs may be designed:

(1) as continuous frames using the method described in Clause 3.7.10 or by any other method satisfying the principles of statics and continuity; or

(2) by the empirical method described in Clauses 3.7.11 to 3.7.16 which is applicable only to the more common forms of this construction described in Clause 3.7.11.

In both methods Clauses 3.7.3 to 3.7.9 apply.

3.7.3 DIVISION OF PANELS *(See Fig. 3.5)*.

Flat slab panels should be assumed to be divided into strips as follows:—

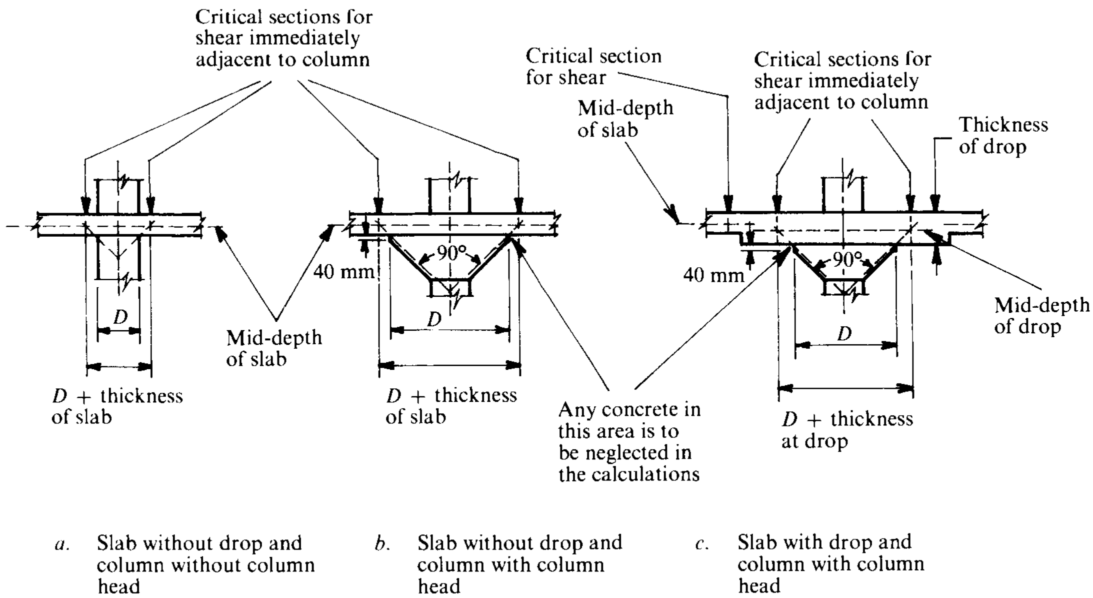


Fig. 3.4 Critical sections for shearing stresses in flat slabs

(1) Column strip. The width of the column strip should be taken as 1/2 of the width of the panel, except that, where drops are used, it may be taken as the width of the drop;

(2) Middle strip. The width of the middle strip should be taken as 1/2 of the width of the panel, except that, where drops are used, and the column strip is taken as the width of the drop, the width of the middle strip should be taken as the difference between the width of the panel and that of the drop.

3.7.4 NOTATION FOR FLAT SLAB CONSTRUCTION

In the following clauses and formulae relating to flat slabs,

L_1 is the length of the panel in the direction of the span,
L_2 is the width of the panel at right angles,
L is the average of L_1 and L_2,
D is the diameter of the column head (see Fig. 3.4 and Clause 3.7.9),
w is the total load per unit area on the panel,
b_s is the length of a shear perimeter,
d_1 is the effective depth of the slab at the shear perimeter,
c is the length of the side of the perimeter parallel to the axis of bending,
V is the total shear force on a column,
M is the moment being transferred through the column/slab junction.

3.7.5 THICKNESS OF FLAT SLAB

The total thickness of the slab should in no case be less than the greatest of the following values:—

1. 125 mm;

2. L/32 for end panels without drops;

3. L/36 for interior panels, fully continuous, without drops, and for end panels with drops;

4. L/40 for interior panels, fully continuous, with drops.

3.7.6 SHEARING STRESSES IN FLAT SLABS

At any section on a perimeter at a distance farther away than 1/2 the slab thickenss from the periphery of the column, column head or drop panel, see Fig. 3.4 the nominal shear stress v should be calculated by the appropriate equations:

Internal columns:

$$v = \frac{V}{b_s d_l}\left(1 + \frac{1.5M}{V_c}\right)$$

Edge or corner columns:

$$v = \frac{V}{b_s d_l}\left(1.25 + \frac{1.5M}{V_c}\right)$$

Where V is the total shear force,
M is the total moment being transferred to the column(s),
b_s is the length of the shear perimeter,
d_l is the effective depth of the slab at the shear perimeter;
c is the length of the side of the perimeter parallel to the axis of bending. (For the purpose of calculating c, non-rectangular columns may be taken as square columns of equivalent area).

For sections without shearing resistance offered by reinforcement, v should not exceed 0.8 times the permissible shear stress p_v given in Clause 3.1.4.

For slabs less than 250 mm thick reinforcement should be assumed not to contribute to the shearing resistance. For slabs of thickness 250 mm and over, reinforcement should be provided to resist $0.5\ v\ b_s\ d_l$ for $0.8\ p_v \leqslant v < 1.0\ p_v$ and reinforcement should be provided to resist $v\ b_s\ d_l$ for $1.0\ p_v \leqslant v \leqslant 3.5\ p_v$. In no case should v exceed $3.5\ p_v$.

In the calculation of shear stress or design of shear reinforcement:—

The factor $\left(1 + \frac{1.5M}{V_c}\right)$ should be taken as not less than 1.15.

Where bending about an axis parallel to the free edge is being considered, the factor $\left(1.25 + \frac{1.5M}{V_c}\right)$ may be taken as 1.25.

Alternatively, this factor may be taken as 1.4 for approximately equal spans.

Design of shear reinforcement may be in accordance with Clause 3.2.7.

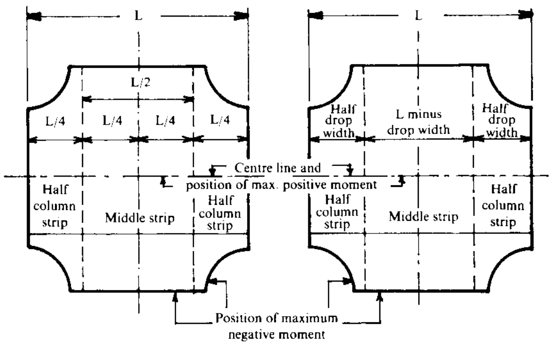
3.7.7 OPENINGS IN PANELS

Except for openings complying with (1), (2) or (3) below, openings should be completely framed on all sides with beams to carry the loads to the columns, and an opening should not encroach upon a column head or drop.

(1) Openings of a size such that the greatest dimension in a direction parallel to a centre-line of the panel does not exceed 0.4L may be formed in the area common to two intersecting middle strips, provided that the total positive and negative moments specified in Sub-clause 3.7.10(5) or Clause 3.7.13 are redistributed between the remaining principal design sections to meet the changed conditions.

(2) Openings of aggregate length or width not exceeding 1/10 of the width of the column strip may be made in the area common to two column strips, provided that the reduced sections are capable of carrying the appropriate moments specified in Sub-clause 3.7.10(5) or Clause 3.7.13 and provided that the perimeter for calculating shear stress is reduced if appropriate.

(3) Openings of aggregate length or width not exceeding 1/4 of the width of the strip may be made in any area common to one column strip and one middle strip, provided that the reduced sections are capable of carrying the appropriate moments specified in Sub-clause 3.7.10(5) of Clause 3.7.13.



a. Where drops are not used *b.* Where drops are used

Fig. 3.5 Division of flat slab panels into column and middle strips

3.7.8 BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS

Where the slab is supported by a marginal beam with a depth greater than 1.5 times the thickness of the slab, or by a wall then:

(1) the total load to be carried by the beam or wall should comprise those loads directly on the wall or beam plus a uniformly distributed load equal to 1/4 of the total load on the slab; and

(2) the bending moments on the half-column strip adjacent to the beam or wall should be 1/4 of the bending moments specified in Sub-clause 3.7.10(5) or Clause 3.7.13.

3.7.9 COLUMN HEADS

Where column heads are provided, the heads of interior columns and such portions of the heads of exterior columns as will lie within the building should satisfy the following requirements:—

(1) The angle of greatest slope of the head should not exceed 45° from the vertical;

(2) The diameter of the column head, D, should be taken as its diameter measured at a distance of 40 mm below the underside of the slab or the underside of the drop where provided, as shown in Fig. 3.4(*b*) and (*c*);

(3) The diameter D should be not more than 0.25L;

(4) Where the column and column head are not of circular cross-section the term diameter used in this clause should be deemed to mean the diameter of the largest circle which can be drawn within the section.

3.7.10 DESIGN OF FLAT SLABS AS CONTINUOUS FRAMES

(1) General

Flat slabs may be designed as continuous frames on the assumptions given in (2), (3), (4), (5) and (6), below. Clauses 3.7.3 to 3.7.9 are also applicable to this method of design.

(2) Bending moments and shearing forces

The bending moments and shearing forces may be determined by an analysis of the structure as a continuous frame and the following assumptions may be made:

(*a*) The structure may be considered to be divided longitudinally and transversely into frames consisting of a row of columns and strips of slab with a width equal to the distance between the centre-lines of the panels on each side of the row of columns;

(*b*) each frame may be analysed by Hardy Cross, or other suitable methods, in its entirety; or each strip of floor and roof may be analysed as a separate frame with the columns above and below assumed fixed at their extremities. The spans used in the analyses should be the distances between the centres of the supports except where the slab is supported by a wall, when the span should be the distance to the face of the wall plus 1/2 the depth of the slab.

(3) Stiffness of members

For the purpose of determining the relative stiffnesses of the members, the moment of inertia of any section of a slab or column may be assumed to be that of the gross cross-section of the concrete alone. Variations of the moment of inertia along the axes of the slabs and columns should be taken into account. The joints between the columns and slabs may be assumed to have an infinite moment of inertia

(4) Maximum bending moments on slabs

The maximum bending moments near the mid-span of a slab and at the centre-line of the supports should be calculated for the following arrangements of the imposed loads:—

(*a*) alternate spans loaded and all other spans unloaded;

(*b*) any two adjacent spans loaded and all other spans unloaded.

(5) Design moments for flat slabs

The slab should be designed for the bending moments so calculated at any section, except that provision need not be made for greater negative moments than those at the critical sections for shear immediately adjacent to a column, as shown in Fig. 3.4. In all cases the sum of the maximum positive bending moment and the average of the negative bending moments, used in the design of any one span of the slab should, for the whole panel with, be not less than:

$$\frac{wL_2}{8}\left(L_1 - \frac{2D}{3}\right)^2$$

where w is the total load per unit area on the panel and D is the diameter of the column heads supporting the slab concerned see Clause 3.7.9. Where the diameters of the column heads supporting the slab are not equal, D should be assumed to be the average of the two diameters.

The bending moments for which provision is made should be divided between the column and the middle strips in the proportion given in Table 3.12.

Table 3.12 Distribution of bending moments in panels of flat slabs designed as continuous frames

	Apportionment between column and middle strip expressed as percentages of the total negative or positive moment*	
	Column strip	Middle strip
Negative moments	75	25
Positive moments	55	45

* Where the column strip is taken as equal to the width of the drop, and the middle strip is thereby increased in width to a value greater than 1/2 of the width of the panel, the moments to be resisted by the middle strip should be increased in proportion to its increased width. The moments to be resisted by the column strip may then be decreased by an amount such that there is no reduction in either the total positive or the total negative moments resisted by the column strip and middle strip together.

(6) Design moments in columns

The maximum bending moments in the columns may be assumed to occur when the imposed load is applied to alternate panels. The columns should be designed to resist that combination of bending moment and direct load consistent therewith which produces the greatest stresses in a column.

3.7.11 EMPIRICAL DESIGN OF FLAT SLABS

(1) General

This empirical method is described in Clauses 3.7.12 to 3.7.16. Clause 3.7.3 to 3.7.9 are also applicable.

(2) Applicability of method

The bending moments given in Clause 3.7.13 apply only when conditions (*a*) and (*b*) below are satisfied.

(*a*) *Limitations regarding numbers and shape of a series of panels.* The slabs should comprise a series of rectangular panels of approximately constant thickness, arranged in at least 3 rows in two directions at right angles, and the ratio of the length of a panel to its width should not exceed 4:3. The lengths and/or widths of any 2 adjacent panels in a series should not differ by more than 15% of the greater length or width. End spans may be shorter, but not longer, than interior spans. Where adjacent spans differ, the length should always be taken as that of the longer span in calculating the bending moments. Stability of the structure is provided by bracing or shear walls designed to resist all the lateral force;

(*b*) *Limitations regarding drops.* The drops should be rectangular on plan, and have a length in each direction not less than 1/3 of the panel length in that direction. For exterior panels the width of drop at right angles to the non-continuous edge and measured from the centre line of the columns should be equal to 1/2 of the width of drop for interior panels.

3.7.12 CRITICAL SECTIONS FOR BENDING MOMENTS IN FLAT SLABS

For interior panels, fully continuous, the critical sections for the bending moments given in Clause 3.7.13 are as follows, see Fig. 3.5:

(1) Positive moment along the centre lines of the panel;

(2) Negative moment along the edges of the panel on lines joining the centres of the columns and around the perimeter of the column heads.

3.7.13 BENDING MOMENTS IN FLAT SLAB PANELS

The bending moments for which provision is made should be divided between the column and middle strips as shown in Table 3.13, where:

$$M_o = \frac{wL_2}{8}(L_l - \frac{2D}{3})^2$$

3.7.14 WIDTHS OF REINFORCING BANDS

In slabs reinforced in two directions only, the reinforcement should be so disposed that each strip is reinforced over its full width.

Table 3.13 Distribution of bending moments in panels of flat slabs designed by the empirical method

	Apportionment of moments between the column and middle strip expressed as percentages of M_o			
	Column strip		Middle strip	
Interior panels				
With drops				
Negative moments	50		15	
Positive moments	20		15	
Without drops				
Negative moments	46		16	
Positive moments	22		16	
	Column Supports	Wall Supports	Column Supports	Wall Supports
Exterior panels				
With drops				
Exterior negative moments	45	6	10	6
Positive moments	25	36	19	26
Interior negative moments	50	72	15	22
Without drops				
Exterior negative moments	41	6	10	6
Positive moments	28	40	20	28
Interior negative moments	46	66	16	24

Note 1. Where the column strip is taken as equal to the width of the drop, and the middle strip is thereby increased in width to a value greater than 1/2 of the width of the panel, the moments to be resisted by the middle strip should be increased in proportion to its increased width. The moments to be resisted by the column strip may then be decreased by an amount such that there is no reduction in either the total positive or the total negative moments resisted by the column strip and middle strip together.

Note 2. Where end spans are shorter than interior spans, the moments given in this table may be suitably modified.

3.7.15 ARRANGEMENT OF REINFORCEMENT IN FLAT SLABS

(1) Slabs reinforced in two directions

(*a*) In each strip or band 40% of the positive reinforcement should extend in the lower part of the slab to within a distance of 0.125L, measured from the line joining the centres of the columns;

(*b*) The negative reinforcement in the top of the slab should extend into adjacent panels for an average distance, measured from the line joining the centres of the columns, of not less than 0.25L, and no bar should extend less than 0.2L from this line;

(*c*) The full area of negative reinforcement should be provided for a distance of not less than 0.2L, measured from the line joining the centres of the columns. The full area of positive reinforcement should be provided for a distance of not less than 0.25L measured from the centre line of the panel;

(*d*) In flat slabs supported on columns without heads, or when the diameter of the head is less than 2 times the average width of the top of the column, 2/3 of the amount of reinforcement required to resist the negative moment in the column strip should be placed in a width equal to 1/2 that of the column strip and central with the column;

(2) Slabs with discontinuous edges

At all discontinuous edges the positive and negative reinforcement should extend to within 75 mm of the edge of the panel, and should be provided with U-hooks.

3.7.16 BENDING MOMENTS IN COLUMNS

(1) Internal and external columns should be designed to resist bending moments equal to 50 and 90% respectively of the negative moment in the column strip specified in Clause 3.7.13. These moments should be apportioned between the upper and lower columns in proportion to their stiffness. In internal columns, the direct load acting with the moment may be reduced to allow for the panel on one side being free of imposed load;

(2) In the case of external columns carrying portions of the floors and walls as a cantilevered load, the specified column moments may be reduced by the moment due to the dead load on the cantilevered portion.

3.8 *STABILITY REQUIREMENTS*

3.8.1 GENERAL

The overall stability of the building including the stability during the period of construction should be considered in the design. The recommendations given in this Clause on tying the structure together, and on the plan form of the building, aim at enabling the structure to accommodate a limited amount of accidental loading which may occur as a result of causes such as construction loading, differential settlement of the supports, thermal movements, explosions, accidental impact etc., which are not defined as normal loading. These accidental loadings may produce local damage, but the recommendations have as their objective the limitation of the extent of such damage.

3.8.2 PLAN FORM

The choice of plan form is a most important consideration for ensuring stability and, as far as practicable, the various elements of a building should be arranged in such a way as to reduce the effect of any local accident.

3.8.3 VEHICLE IMPACT

The provision of bollards, walls retaining earth banks, etc., should be considered to obviate the possibility of vehicles running into and damaging or removing vital load bearing members of the structure in the ground floor.

3.8.4 TIES

(1) General

The recommendations of this Clause may be considered to satisfy the general stability requirement that in addition to safely supporting all appropriate dead, imposed and wind loads, buildings should be designed and constructed so that if any one structural member (other than one purposely designed to resist initial damage) were considered to have been removed, the consequent structural failure would affect only a small part of the building.

(2) Interaction of members in the horizontal plane

To ensure a degree of interaction between members in the horizontal plane, every building should be provided with ties in accordance with (3), (4) and (5) below. In general, these ties can be provided by effectively connecting together part of the reinforcement provided in the members to support loads caused by normal function. It may be assumed, when calculating the area of the tie required, that the tie has only to resist the stated horizontal force and that the steel is otherwise unstressed.

The ties should be so placed as to provide the best assistance in resisting by cantilever, catenary or other action, the possible results of accidental damage to a part of the building. At re-entrant corners, or at substantial changes in construction, care should be taken to ensure that the ties are adequately anchored into the adjacent floor or otherwise made effective.

(3) Peripheral tie

At each floor and roof level an effectively uninterrupted peripheral tie should be provided, located within 1.2 m of the edge of the building or perimeter wall. This tie should be capable of resisting a horizontal tensile force of 40 kN without exceeding the permissible stress in the steel.

(4) Internal ties

In addition to the peripheral tie, internal ties should be provided at each floor and roof level in two directions approximately at right angles. The internal ties should be effectively uninterrupted throughout their length and should, unless they continue as column or wall ties (5) below, be anchored to the peripheral tie at both ends. Provided that the beam or slab spans do not exceed 5 m and the gross weight of the construction and imposed loads does not exceed 7.5 kPa, the ties in each direction should be capable of resisting a horizontal tensile force of 25 kN per metre width without exceeding the permissible stress in the steel. For construction involving greater beam or slab spans, and/or supporting greater gross weights, the ties should be proportionately larger; the span to be considered for this purpose may, however, be limited to 5 times the clear storey height under the beam or slab.

Part or all of the internal ties may be spread evenly over the width of the structure or may be grouped at beams, walls or other appropriate intervals. The ties may be in the slabs, beams or walls; where they are in walls they should be located within 0.5 m of the top or bottom of the floor slab.

(5) Horizontal column and wall ties

All external load bearing vertical members should be anchored or tied horizontally into the structure at each floor and roof level with a tie capable of resisting, without exceeding the permissible stress in the steel, a horizontal force equal to the greater of:

(*a*) 25 kN for each column, or 25 kN per metre length of load bearing wall; or

(*b*) 3% of the total vertical load in the column or wall at floor level.

The figures given in (*a*) relate to a floor to ceiling height not exceeding 2.5 m and should be increased in proportion for greater heights; the increase may, however, be limited to 100%.

Corner columns should be tied into the structure at each floor and roof level in each of two directions approximately at right angles, with ties capable of resisting a force equal to the greater of (*a*) or (*b*) above. Column and wall ties may be partly or, wholly the same reinforcement as that provided for the peripheral or internal ties. Column and wall ties should not rely solely on the bond of a plain straight bar for their anchorage at either end. Plain bar should be bent or hooked so as to provide the required anchorage in bearing on sound concrete unless welded or mechanically anchored to the main reinforcement.

(6) Vertical column and wall ties

Vertical ties should be provided in all columns and walls. The area of these ties should be at least equal to the minima given in Clauses 3.3.1 and 3.4 for their main reinforcement as reinforced concrete members. Each tie should be effectively uninterrupted from foundation to roof level.

4. DESIGN AND DETAILING: PRESTRESSED CONCRETE

4.1 *GENERAL*

4.1.1 SCOPE

This section gives methods of analysis and design which will in general ensure that, for prestressed concrete structures, the objectives set out in Section 2 are met. Other methods may be used provided that they can be shown to be satisfactory for the type of structure or member considered. In certain cases, the assumptions made in this Section may be inappropriate and a more suitable method should be adopted having regard to the nature of the structure in question.

4.1.2 DEFINITIONS

PRESTRESSED CONCRETE — Concrete in which effective internal stresses are induced artificially, usually by means of tensioned steel, prior to loading the structure.

TENDON — A stretched element used in a concrete member or structure to impart prestress to the concrete. For the purpose of this Code tendons are assumed to be of steel.

(3) Peripheral tie

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Part or all of the internal ties may be spread evenly over the width of the structure or may be grouped at beams, walls or other appropriate intervals. The ties may be in the slabs, beams or walls; where they are in walls they should be located within 0.5 m of the top or bottom of the floor slab.

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Corner columns should be tied into the structure at each floor and roof level in each of two directions approximately at right angles, with ties capable of resisting a force equal to the greater of (*a*) or (*b*) above. Column and wall ties may be partly or, wholly the same reinforcement as that provided for the peripheral or internal ties. Column and wall ties should not rely solely on the bond of a plain straight bar for their anchorage at either end. Plain bar should be bent or hooked so as to provide the required anchorage in bearing on sound concrete unless welded or mechanically anchored to the main reinforcement.

(6) Vertical column and wall ties

Vertical ties should be provided in all columns and walls. The area of these ties should be at least equal to the minima given in Clauses 3.3.1 and 3.4 for their main reinforcement as reinforced concrete members. Each tie should be effectively uninterrupted from foundation to roof level.

4. DESIGN AND DETAILING: PRESTRESSED CONCRETE

4.1 *GENERAL*

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This section gives methods of analysis and design which will in general ensure that, for prestressed concrete structures, the objectives set out in Section 2 are met. Other methods may be used provided that they can be shown to be satisfactory for the type of structure or member considered. In certain cases, the assumptions made in this Section may be inappropriate and a more suitable method should be adopted having regard to the nature of the structure in question.

4.1.2 DEFINITIONS

PRESTRESSED CONCRETE — Concrete in which effective internal stresses are induced artificially, usually by means of tensioned steel, prior to loading the structure.

TENDON — A stretched element used in a concrete member or structure to impart prestress to the concrete. For the purpose of this Code tendons are assumed to be of steel.

TRANSFER	The action of transferring load in the prestressing tendons to the concrete.
PRE-TENSIONING	A method of prestressing in which tendons are tensioned before the concrete is placed.
POST-TENSIONING	A method of prestressing in which tendons are tensioned after the concrete has hardened.

4.1.3 BASIC ASSUMPTIONS

Computation of stresses should accord with the laws of mechanics and the recognized general principles relating to the design of prestressed concrete.

4.1.4 HANDLING STRESSES

The effects of handling and construction on the stresses in members should be considered at the design stage. Consideration should also be given to the secondary effects due to prestress.

4.2 *PERMISSIBLE STRESSES*

4.2.1 GENERAL

The permissible stresses given in Clause 4.2.2 and 4.2.3 are for the purpose of the design of structures generally. In beams, permissible stresses may require further limitation to comply with the requirement of Clause 4.9.1 as regards ultimate strength, which is an over-riding consideration, or with the requirements of Clause 4.9.11 as to deflections.

4.2.2 PERMISSIBLE STRESS IN CONCRETE

(1) Compressive stress

The compressive stresses in concrete should not exceed those given in Table 4.1.

Table 4.1 Permissible compressive stresses in concrete

Nature of loading	Permissible compressive stress
Maximum working load	
In bending	$0.33\ u_w$ In continuous beams and other statically indeterminate structures this may be increased to $0.4\ u_w$ within the range of support moments.
In direct compression	$0.25\ u_w$
Wind loading	As for maximum working load plus 25%, provided the excess is solely due to wind forces.
Transfer*	$0.5\ u_t$ for a triangular or roughly triangular distribution of prestress, but not greater than $0.33\ u_w$ or $0.4\ u_w$ as described above $0.4\ u_t$ for a uniform or approximately uniform distribution of prestress, but not greater than $0.25\ u_w$

* Where the stress at transfer approaches the limit given in Table 4.1, the allowances for losses must be carefully considered and working stresses restricted to proper values.

(2) Tensile stresses due to bending

The tensile stresses due to bending under maximum working load should not exceed those given in Table 4.2.

These stress are appropriate to a member or structure which acts as if monolithic, but no tension is permissible at mortar or concrete joints of members made up of precast units.

The stresses given in Table 4.2 may be increased by up to 1.75 MPa provided that it is shown by tests that such enhanced stress does not exceed 75% of the tensile stress calculated from the loading in the performance test corresponding to the appearance of the first crack. Where such increase is permitted, prestress in the concrete should be at least 10 MPa.

Pre-tensioned steel should be well distributed throughout the tensile zone of the section; and post-tensioned steel should be supplemented, if necessary, by additional steel which need not be prestressed, located near the tension face of the member.

When the maximum working load to be considered is of a temporary nature and is exceptionally high in comparison with the load normally carried, a higher calculated tensile stress than that given in Table 4.2 is permissible provided that under normal conditions the stress is compressive to ensure that any cracks which might have occurred close up.

The tensile stresses at transfer should not in general exceed the permissible stresses given in Table 4.3.

Table 4.2 Permissible stresses in concrete in tension due to bending

Occurance and duration of loading	Pre-tensioning	Post-tensioning with adequate grouting and bonding	
	Specified grade strength of concrete MPa	Specified grade strength of concrete MPa	
	40	30	40
Maximum working load occuring often and/or for long durations	2.0	1.2	1.4
Maximum working load occuring rarely and/or short durations e.g. wind loading	3.0	1.9	2.0

Table 4.3 Permissible stresses in concrete in tension at transfer

Cube strength of concrete at transfer MPa	Permissible tensile stress MPa
20	1.0
30	1.2
40	1.4
45	1.45

(3) Direct tensile stress

The permissible stress in direct tension should not exceed 50% of the permissible bending tensile stress given in Table 4.2 for the appropriate conditions of loading and specified grade strength of the concrete.

4.2.3 MAXIMUM INITIAL PRESTRESS IN PRESTRESSING TENDON

The jacking force shall not normally exceed 70% of the characteristic strength of the tendon.

4.2.4 SEQUENCE OF PRESTRESSING

The order in which prestressing tendons in a structure are to be stressed should be such that the stresses permitted by Clauses 4.2.2(1), 4.2.2(2) and 4.2.2(3) are not at any stage exceeded. The order should be specified on the working drawings.

4.3 *LOSS OF PRESTRESS OTHER THAN FRICTION LOSSES*

4.3.1 GENERAL

Allowance should be made when calculating the forces in tendons at the various stages considered in design for the appropriate losses of prestress resulting from:

(1) relaxation of the steel comprising the tendons;

(2) the elastic deformation and subsequent shrinkage and creep of the concrete;

(3) slip or movement of tendons at anchorage during anchoring, and

(4) other causes in special circumstances as for example when steam curing is used with pre-tensioning.

If experimental evidence on performance is not available, account should be taken of the properties of the steel and of the concrete when calculating the losses of prestress from these causes.

4.3.2 LOSS OF PRESTRESS DUE TO RELAXATION OF STEEL

The loss of force in tendon allowed for in the design should be the maximum relaxation after 1 000 hour duration, for a jacking force equal to that imposed at transfer, as given by the British Standard appropriate for that type of tendon.

The loss of prestress given by the above British Standard will be for a jacking force of 70%, but when the jacking force in a tendon is less than 70% of its characteristic strength and there is no experimental evidence available, the relaxation loss may be assumed to decrease linearly from 8% for an initial prestress of 70%, to zero for an initial prestress of 50% of the characteristic strength of the tendon.

No reduction in the value of the relaxation loss should be made for a tendon when a load equal to or greater than the relevant jacking force has been applied for a short time prior to the anchoring of the tendon.

In special cases, such as tendons at high temperatures or subjected to large lateral loads, greater relaxation losses will occur. Due consideration should be given to the relaxation losses in these cases.

4.3.3 LOSS OF PRESTRESS DUE TO ELASTIC DEFORMATION OF THE CONCRETE

Calculation of the immediate loss of force in the tendon due to elastic deformation of the concrete at transfer may be based on the values for the modulus of elasticity of the concrete given in Table 2.1. The modulus of elasticity of the tendons may be taken as:

$Es = 200\,000$ MPa for wire and strands to BS 5896
$Es = 175\,000$ MPa for alloy bars to BS 4486 and nineteen-wire strand to BS 4757.

For pre-tensioning, the loss of prestress in the tendons at transfer should be calculated on a modular ratio basis using the stress in the adjacent concrete.

For members with post-tensioning tendons which are not stressed simultaneously, there is a progressive loss of prestress during transfer due to the gradual application of the prestressing force. The resulting loss of prestress in the tendons should be calculated on the basis of 50% the product of the modular ratio and the stress in the concrete adjacent to the tendons averaged along their length; alternatively, the loss of prestress may be exactly computed based on the sequence of tensioning.

In making these calculations, it may usually be assumed that the tendons are located at their centroid.

4.3.4 LOSS OF PRESTRESS DUE TO SHRINKAGE OF CONCRETE

The loss of prestress in the tendons due to shrinkage of the concrete should be calculated from the modulus of elasticity for the tendons given in Clause 4.3.3, assuming that the shrinkage strain of concrete Δ_{cs} at any instant is given by the product of five partial coefficients:

$$\Delta_{cs} = C_s\, K_L\, K_c\, K_e\, K_j$$

Where $C_s = 4.0$,
K_L depends on the environment (Fig. 4.1a),
K_c depends on the composition of the concrete (Fig. 4.1b),
K_e depends on the effective thickness of the member (Fig. 4.1c);
K_j defines the development of shrinkage as a function of time (Fig. 4.1d).

The shrinkage to be expected over an interval of time should be taken as the difference between the shrinkage calculated for the beginning and the end of the interval. The values of shrinkage which are for plain concrete, should be multiplied by the reinforcement coefficient K_s to obtain the corresponding values for reinforced concrete:

$$K_s = \frac{1}{1 + pm}$$

Where p is the steel ratio $= \frac{A_s}{A_c}$,
A_s is the area of longitudinal reinforcement,
A_c is the area of concrete;
m is the modular ratio $= \frac{E_s}{E_c}$.

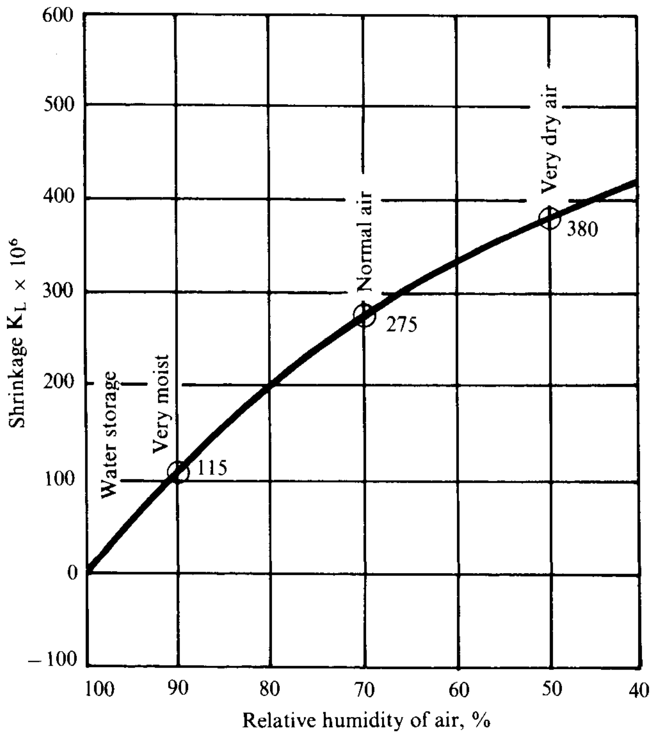

Fig. 4.1.a. Coefficient K_L (environment) for calculating shrinkage

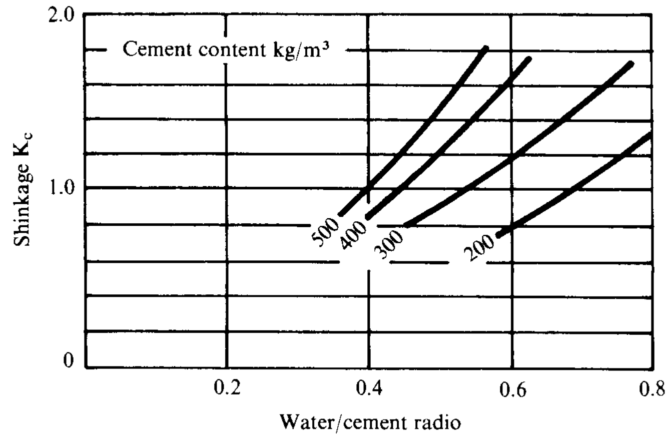

Fig. 4.1.b. Coefficient K_c (composition of the concrete) for calculating shrinkage

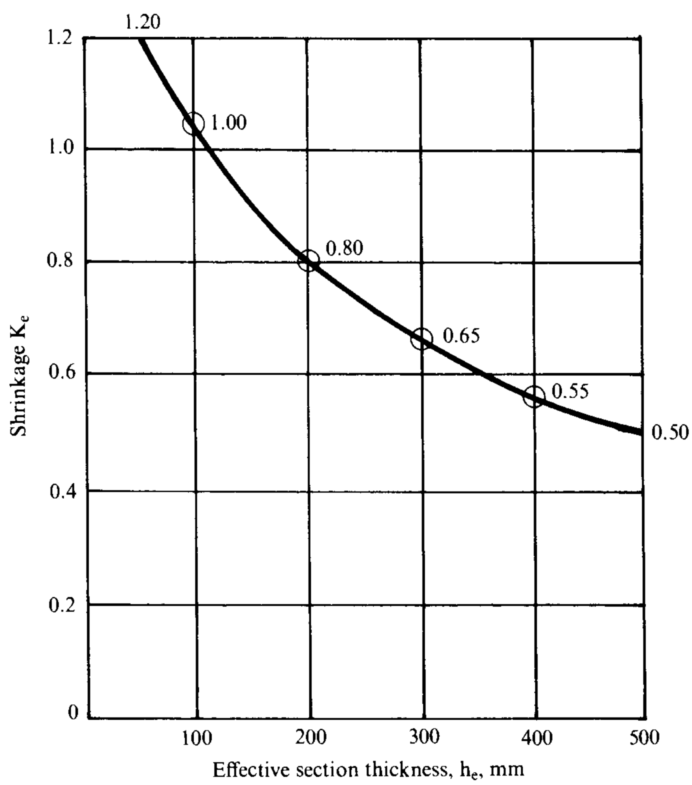

Fig. 4.1.c. Coefficient K_e (effective thickness) for calculating shrinkage

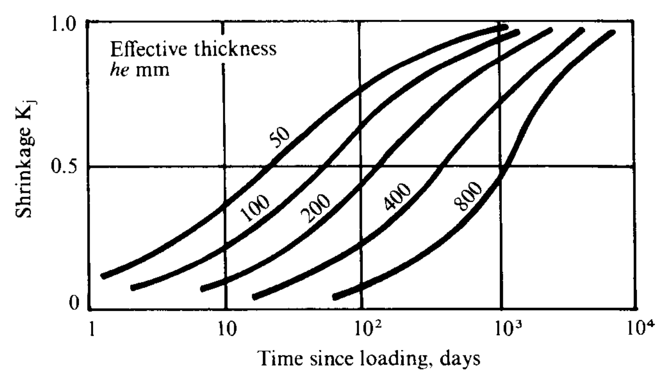

Fig. 4.1.d. Coefficient K_j (variation as a function of time) for calculating shrinkage

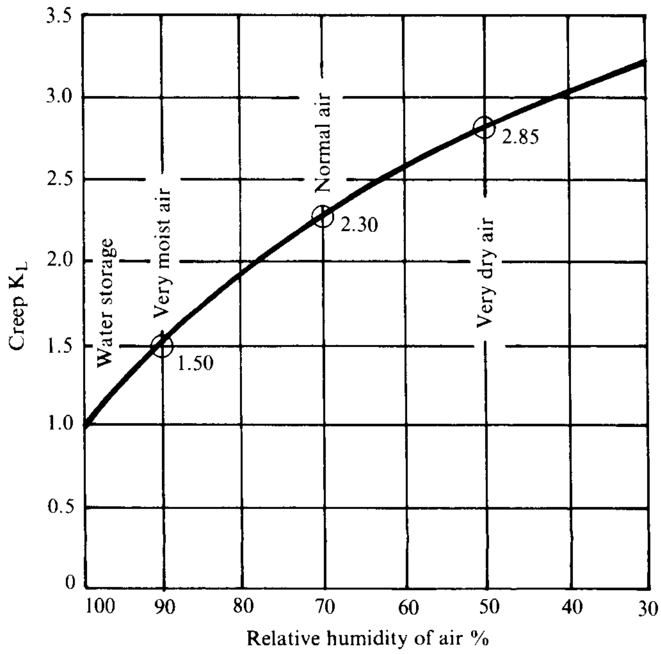

Fig. 4.1.e. Coefficient K_L (environmental conditions) for calculating creep

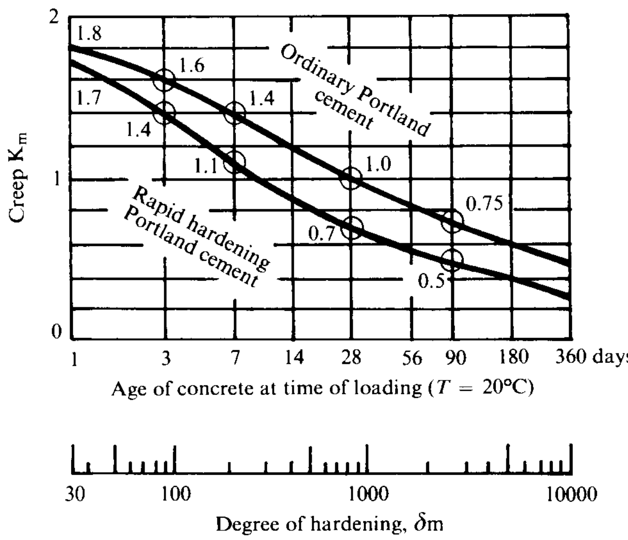

Fig. 4.1.f. Coefficient K_m (hardening (maturity) at the age of loading) for calculating creep

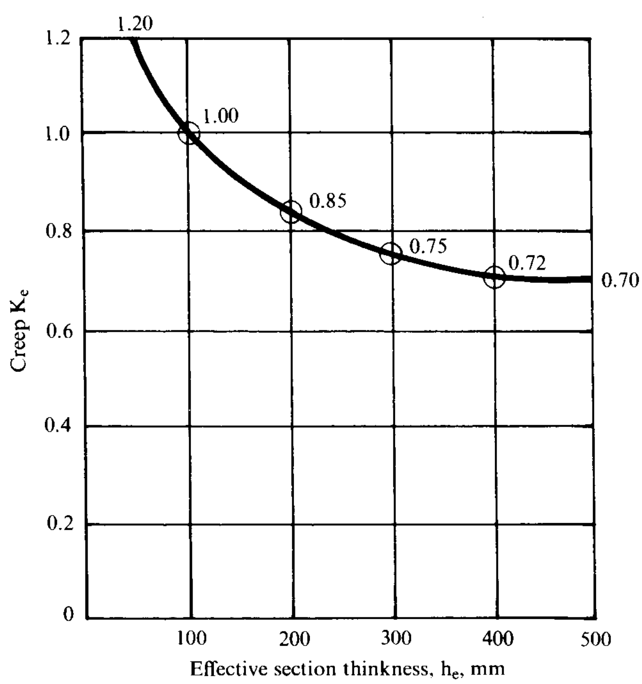

Fig. 4.1.g. Coefficient K_e (effective thickness) for calculating creep

4.3.5 LOSS OF PRESTRESS DUE TO CREEP OF CONCRETE

The loss of prestress in the tendons due to creep of the concrete should be calculated on the assumption that creep is proportional to the stress in the concrete for stress up to 1/3 of the cube strength at transfer. The loss of prestress is obtained from the product of the modulus of elasticity of the tendon see Clause 4.3.3 and the creep of the concrete adjacent to the tendon. Usually it is sufficient to assume, in calculating this loss, that the tendons are located at their centroid.

The ultimate creep strain of concrete is given by equation:

$$\Delta cc = \frac{f_c}{E28} \times ø$$

Where Δcc is the ultimate creep strain,
f_c is the stress in concrete adjacent to tendons,
E28 is the 28-day value of concrete secant modulus which may be taken from Table 2.1;
ø is the creep factor.

The value of the creep factor is given by equation:

$$ø = K_L\, K_m\, K_c\, K_e\, K_j$$

Where K_L depends on the environmental conditions (Fig. 4.1e),
K_m depends on the hardening (maturity) of the concrete (Fig. 4.1f),
K_c depends on the composition of the concrete (Fig. 4.1b),
K_e depends on the effective section thickness which is defined for uniform sections as twice the cross-sectional area divided by the exposed perimeter (Fig. 4.1g);
K_j defines the development of creep with time (Fig. 4.1d).

The values derived for Δcc should be multiplied by the reinforcement coefficient K_s from Clause 4.3.4 to obtain the corresponding values for reinforced concrete.

Where the maximum stress anywhere in the section at transfer exceeds 1/3 of the cube strength of the concrete the value for the creep per unit length used in calculations should be increased. When the maximum stress at transfer is 1/2 the cube strength, the values for creep are 1.25 times the values given above; at intermediate stresses, the values should be interpolated linearly.

4.3.6 LOSS OF PRESTRESS DURING ANCHORING

In post-tensioning systems allowance shall be made for any movement of the tendon at the anchorage when the prestressing force is transferred from the tensioning equipment to the anchorage.

4.3.7 LOSSES OF PRESTRESS DUE TO STEAM CURING

Where steam curing is employed in the manufacture of prestressed concrete units, changes in the behaviour of the material at higher than normal temperatures will need to be considered. In addition, where the long-line method of pre-tensioning is used there may be additional losses as a result of bond developed between the tendon and the concrete when the tendon is hot and relaxed.

4.4 *LOSSES DUE TO FRICTION*

4.4.1 GENERAL

In post-tensioning systems there will be movement of the greater part of the tendon relative to the surrounding duct during the tensioning operation, and if the tendon is in contact with either the duct or any spacers provided, friction will cause a reduction in the prestressing force as the distance from the jack increases. In addition, a certain amount of friction will be developed in the jack itself and in the anchorage through which the tendon passes.

In the absence of established evidence, the stress variation likely to be expected along the design profile should be assessed in accordance with Clauses 4.4.2 to 4.4.5 in order to obtain the prestressing force at the critical sections considered in design. The extension of the tendon should be calculated allowing for the variation in tension along its length.

4.4.2 FRICTION IN THE JACK AND ANCHORAGE

This is directly proportional to the jack pressure, but it will vary considerably between systems and should be ascertained for the type of jack and the anchorage system to be used.

4.4.3 FRICTION IN THE DUCT DUE TO UNINTENTIONAL VARIATION FROM THE SPECIFIED PROFILE

Whether the desired duct profile is straight or curved or a combination of both, there will be slight variations in the actual line of the duct, which may cause additional points of contact between the tendon and the sides of the duct, and so produce friction. The prestressing force P_x in the tendon during tensioning at any distance x from the jack is given by equation:

$$P_x = P_{oj}e^{-Kx}$$

and where $Kx \leqslant 0.2$, e^{-Kx} may be taken as $1-Kx$

Where P_{oj} is the prestressing force in the tendon at the jacking end,
e is the base of Napierian logarithms (2.718);
K is the constant depending on the type of duct, or sheath employed, the nature of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete.

The value of K per metre length in the above formula should generally be taken as not less than 33×10^{-4} but where strong rigid sheaths or duct formers are used closely supported so that they are not displaced during the concreting operation, the value of K may be taken as 17×10^{-4}. Other values may be used provided they have been established by tests.

4.4.4 FRICTION IN THE DUCT DUE TO CURVATURE OF THE TENDON

When a tendon is curved, the loss of tension due to friction is dependent on the angle turned through and the coefficient of friction μ between the tendon and its supports.

The prestressing force in the tendon during tensioning at any distance x along the curve from the tangent point is given by equation:

$$P_x = P_{ot}e^{-\mu x/R}$$

Where P_{ot} is the prestressing force in the tendon at the tangent point near the jacking end;
R is the radius of curvature of the tendon.

Where $\mu x/R \leqslant 0.2$, $e^{-\mu x/R}$ may be taken as $1-\mu x/R$.

Where $(Kx + \mu x/R) \leqslant 0.2$, $e^{-(Kx + \mu x/R)}$ may be taken as $1-(Kx + \mu x/R)$.

Values of μ may be taken as:
0.55 for steel moving on concrete,
0.30 for steel moving on steel,
0.25 for steel moving on lead.

The value of μ may be reduced where special precautions are taken and where results are available to justify the values assumed.

4.4.5 FRICTION IN CIRCULAR CONSTRUCTION

Where circumferential tendons are tensioned by means of jacks the losses due to friction may be calculated from the formula in Clause 4.4.4, but the values of μ may be taken as:

0.45 for steel moving on smooth concrete
0.25 for steel moving on steel bearers fixed to the concrete
0.10 for steel moving on steel rollers.

4.5 *TRANSMISSION LENGTH IN PRE-TENSIONED MEMBERS*

The transmission length is defined as being the length of member required to transmit the initial prestressing force in a tendon to the concrete.

The transmission length depends on a number of variables, the most important being: the degree of compaction of the concrete, the size and type of tendon, the strength of the concrete, and the deformation, e.g. crimp of the tendon.

The transmission length can vary a great deal for different factory or site conditions; for example, it has been shown that the transmission length for wire can vary between 50 and 160 diameters. As far as possible, therefore, the transmission length should be based on experimental evidence for known site or factory conditions.

The following general recommendations, based on research, should be considered in relation to the known site or factory conditions.

4.5.1 For factory produced units using plain or indented wire with a small offset crimp (e.g. 0.3 mm offset, 40 mm pitch), a transmission length of 100 diameters may be assumed when the ends of the units are fully compacted and the cube strength of the concrete at transfer is not less than 35 MPa.

4.5.2 For units using wire of a considerable crimp (e.g. 1.0 mm offset, 40 mm pitch), a bond length of 65 diameters may be assumed when the ends of the units are fully compacted and the cube strength of the concrete at transfer is not less than 35 MPa.

4.5.3 The development of stress from the end of the unit to the point of maximum stress is such that it may be assumed that 80% of the maximum stress is developed in a length of 70 diameters for the conditions mentioned in Clause 4.5.1 and in a length of 54 diameters for the conditions mentioned in Clause 4.5.2.

4.5.4 When the cube strength of the concrete at transfer is less than 35 MPa, the transmission lengths are likely to be greater.

4.5.5 The transmission length for tendons near the top of a beam may well be greater than that for identical tendons placed lower in the beam, since the concrete near the top is less likely to be as well compacted.

4.5.6 The sudden release of tendons leads to a great increase in the transmission lengths in the units near the releasing end of the bed.

4.5.7 From the available experimental data, the transmission length for small diameter strand is not proportional to the diameter of the tendon, nor is the scatter of results so great as it is for wire. Table 4.4 gives values for the transmission length for strand; in the absence of more exact data, these values may be used in design.

4.5.8 If the tendons are prevented from bonding to the concrete near the ends of the members by the use of sleeves or tape, the values given in Table 4.4 for transmission length may be used, it being assumed that the transmission zone starts at the point where the de-bonding process has been stopped.

Table 4.4 Transmission lengths for small diameter strand

Diameter of strand	Transmission length (range of results given in brackets)
mm	mm
9.3	200 ($\pm$ 25)
12.5	330 ($\pm$ 25)
18	500 ($\pm$ 50)

Where required the transmission lengths used in designs should be confirmed by tests.

4.6 *END BLOCKS IN POST-TENSIONED MEMBERS*

The bursting tensile forces in the end blocks, or regions of bonded post-tensioned members, should be assessed on the basis of 1.15 times of the tendon jacking load. For unbonded members, the bursting tensile forces should preferably be assessed on the basis of 1.15 times of the tendon jacking load or the load in the tendon at ultimate load condition, calculated from Table 4.9, whichever is the greater.

The bursting tensile force, F_{bst}, existing in an individual square end block loaded by a symmetrically placed square anchorage or bearing plate may be derived from Table 4.5.

Table 4.5 Design bursting tensile forces in end blocks

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_k	0.23	0.23	0.20	0.17	0.14	0.11

Where y_o is half the side of the end block,
y_{po} is half the side of loaded area,
P_k is the load in the tendon assessed as above;
F_{bst} is the bursting tensile force.

This force, F_{bst}, will be distributed in a region extending from $0.2y_o$ to $2y_o$ from the loaded face of the end block, and should be resisted by reinforcement in the form of spirals or closed links, uniformly distributed throughout this region, and acting at a stress not exceeding 200 MPa.

When circular anchorage or bearing plates are used, the side of the equivalent square area should be derived.

Where groups of anchorages or bearing plates occur, the end blocks should be divided into a series of symmetrically loaded prisms and each prism treated in the above manner.

Special attention should be paid to end blocks having a cross-section different in shape from that of the general cross-section of the beam, reference should be made to specialist literature.

Compliance with the above requirements will generally ensure that bursting tensile forces along the load axis are provided for. Alternative methods of design which make allowance for the tensile strength of the concrete may be used, in which case reference should be made to specialist literature.

Consideration should also be given to the spalling tensile stresses that occur in end blocks where the anchorages or bearing plates are highly eccentric; these reach a maximum at the loaded face.

4.7 *SPACING OF TENDONS*

4.7.1 TENDONS IN DUCT

In post-tensioned members where tendons are normally placed in ducts, the clear distance between ducts or between ducts and other tendons should be not less than the following, whichever is the greatest:

(1) h_{agg} + 5 mm, where h_{agg} is the nominal maximum size of the coarse aggregate;

(2) in the vertical direction: the vertical internal dimension of the duct;

(3) in the horizontal direction: the horizontal internal dimension of the duct. Where internal vibrators are used sufficient space should be provided between ducts to enable the vibrator to be inserted.

Where two or more rows of ducts are used the horizontal gaps between the ducts should be vertically in line.

4.7.2 CURVED TENDONS IN DUCTS

In order to prevent crushing of the concrete between ducts for curved tendons in post-tensioned members, the minimum spacing should be as follows:

(1) In the place of curvature: the values given in Table 4.6 or the value required by Clause 4.7.1 whichever is the greater.

(2) Perpendicular to the plane of curvature: the requirement of Clause 4.7.1.

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 Table 4.6 will need to be increased. If necessary, reinforcement should be provided between ducts.

The distance for a given combination of duct internal diameter and radius of curvature shown in Table 4.6 may be reduced pro rata with the tendon force when this is less than the value tabulated, subject to the requirements of Clause 4.7.1.

4.7.3 PRE-TENSIONED MEMBERS

In pre-tensioned members, there should be sufficient gaps between the tendons or groups of tendons to allow the largest size of aggregate used to move, under vibration, to all parts of the mould. In addition, the spacing of the wires or strands in the ends of the members should be such as to allow the transmission lengths to be developed. If the tendons are positioned in two or more widely spaced groups, the possibility of longitudinal splitting of the members should be considered.

Table 4.6 Minimum distance between centre lines of ducts in plane of curvature, in millimetres

Radius of curvature of ducts (m)	Duct internal diameter (mm) 20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
	Tendon force (kN) 296	387	960	1337	1920	2640	3360	4320	5183	6019	7200	8640	9424	10336	11248	13200
2	110	140	350	485	700	960								Radii not		
4	55	70	175	245	350	480	610	785	940					normally		
6	40	60	120	165	235	320	410	525	630	730	870	1045		used		
8			90	125	175	240	305	395	470	545	655	785	855	940		
10			80	100	120	195	245	315	375	440	525	630	685	750	815	
12						160	205	265	315	365	435	525	570	625	680	800
14						140	175	225	270	315	375	450	490	535	585	785
16							160	195	235	275	330	395	430	470	510	600
18								180	210	245	290	350	380	420	455	535
20									200	220	265	315	345	375	410	480
22											240	265	310	340	370	435
24												260	285	315	340	400
26													280	300	320	370
28																345
30																340
32																
34																
36																
38																
40	40	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340

Note 1 The tendon force shown is the maximum normally available for the given size of duct.

Note 2 Values less than 2 × duct internal diameter are not included.

4.8 *COVER TO PRESTRESSING TENDONS*

4.8.1 GENERAL

The concrete cover to the tendons and the cement content of the concrete should meet the statutory durability requirements and fire resistance requirements of the structure, as well as the following requirements.

4.8.2 COVER TO CURVED TENDONS IN DUCTS

In order to prevent bursting of the cover:

(1) perpendicular to the plane of curvature; and

(2) In the plane of curvature, e.g. where the curved tendons run close to and approximately parallel to the surface of a member.

The cover should be in accordance with the values given in Table 4.7. 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 Table 4.7. will need to be increased.

Table 4.7 Minimum cover to ducts perpendicular to plane of curvature, in millimetres

Radius of curvature of ducts (m)	Duct internal diameter (mm)															
	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
	Tendon force (kN)															
	296	387	960	1337	1920	2640	3360	4320	5183	6019	7200	8640	9424	10336	11248	13200
2	50	55	155	220	320	445								Radii not		
4		50	70	100	145	205	265	350	420					normally		
6			50	65	90	125	165	220	265	310	375	460		used		
8				55	75	95	115	150	185	220	270	330	360	395		
10				50	65	85	100	120	140	165	205	250	275	300	330	
12					60	75	90	110	125	145	165	200	215	240	260	315
14					55	70	85	100	115	130	150	170	185	200	215	260
16					55	65	80	95	110	125	140	160	175	190	205	225
18					50	65	75	90	105	115	135	150	165	180	190	215
20						60	70	85	100	110	125	145	155	170	180	205
22						55	70	80	95	105	120	140	150	160	175	195
24						55	65	80	90	100	115	130	145	155	165	185
26						50	65	75	85	100	110	125	135	150	160	180
28							60	75	85	95	105	120	130	145	155	170
30							60	70	80	90	105	120	130	140	150	165
32							55	70	80	90	100	115	125	135	145	160
34							55	65	75	85	100	110	120	130	140	155
36							55	65	75	85	95	110	115	125	140	150
38							50	60	70	80	90	105	115	125	135	150
40	50	50	50	50	50	50	50	60	70	80	90	100	110	120	130	145

Note The tendon force shown is the maximum normally available for the given size of duct.

The cover for a given combination of duct internal diameter and radius of curvature shown in Table 4.9, may be reduced pro rata with the square root of the tendon force when this is less than the value tabulated, subject to a minimum value of 50 mm.

In the case of (2) above, if the tendon develops radial forces perpendicular to the exposed surface of the concrete, the duct should be restrained by stirrup reinforcement anchored into the members.

4.9 *BEAMS AND OTHER MEMBERS IN BENDING*

4.9.1 BASIS OF DESIGN

The design of prestressed concrete beams and other members in bending should conform with the two following requirements:

(1) the computed stresses in the concrete and in the steel should not exceed the permissible stresses given in Clause 4.2 during transfer, handling and construction, and under working loads;

(2) the member should be capable of carrying without collapse the following ultimate loads:

(*a*) Dead and imposed load:
1.5 × dead load + 2.5 × imposed load,
or 2.0 × (dead load + imposed load)
whichever is the lesser

(*b*) Dead and wind load:
1.0 × dead load + 2.0 × wind load.

(*c*) Dead, imposed and wind load:
1.6 × (dead load + imposed load + wind load)

In assessing the effect of these loads on the structure as a whole, or on any part or section of the structure, the arrangement of the loads should be such as to cause the most severe effects.

Under load combination (*a*), a dead load factor of 1.0 should be applied to such parts of the structures as may result in the most unfavourable condition.

The design of the whole, or of any part or section of a structure, may be controlled by any of the load combinations (*a*) to (*c*); each should be considered in design, and the most severe adopted.

In general, for the ultimate load condition, the effects of creep, shrinkage and temperature will be of secondary order and no specific calculations will be necessary.

4.9.2 EFFECTIVE SPAN

For the purpose of calculating bending moments, the effective span L of a simply supported beam should be taken as the lesser of the two following:

(1) the distance between the centres of supports, or

(2) the clear distance between supports, plus the depth of the beam.

4.9.3 DESIGN FOR WORKING LOADS

Computation of stresses for the conditions of loading defined in the first requirement of Clause 4.9.1 above should accord with the laws of mechanics. In making these calculations it may be assumed that:

(1) plane sections remain plane;

(2) after the initial stressing of the tendons which may be subject to relaxation of stress see Clause 4.3.2, the behaviour of the steel is otherwise elastic within the limits of permissible stress given in Clause 4.2.2,

(3) shrinkage of the concrete, for which values are given in Clause 4.3.4, is uniform across the section and affects the distribution of stress in the concrete only by its influence on the stress in the tendons,

(4) concrete, when stressed within the limits of permissible stress given in Clause 4.2.2, exhibits an immediate elastic strain, which may be calculated from the values for the modulus of elasticity for concrete given in Table 2.1; this elastic strain is followed by creep, which at any time is proportional to the applied stress and may be calculated, see Clause 4.3.5,

(5) there is complete adhesion between the tendons and the concrete only where the steel is pre-tensioned or where the steel is post-tensioned and properly grouted and bonded with cement grout, or in the case of external tendons where adequate shear bars and concrete cover are provided.

4.9.4 CALCULATION OF ULTIMATE FLEXURAL STRENGTH OF BEAMS

(1) General

The design of prestressed concrete beams for working load conditions normally leads to the selection of sections in which the proportions of prestressing steel are relatively small. For steel effectively bonded to the concrete, these proportions are such that the tensile strength of the steel is closely approached when the loading is sufficient to cause bending failure. For these sections, the distribution of stress in the concrete of the compression zone and its maximum value, assumed in calculation, have little influence on the computed failing moment. Where greater proportions of prestressing steel are used, or in the unusual circumstances where this steel is not bonded to the concrete, the stress in the tendons at failure is not likely to reach its tensile strength.

(2) Assumptions

When analysing sections under ultimate loads the following assumptions should be made:

(*a*) The strain distribution in the concrete in compression is derived from the assumption that plane sections remain plane.

(*b*) The stresses in the concrete in compression are either derived from the stress—strain curve given in Fig. 2.1, or, where the compression zone is of rectangular section above the neutral axis at failure, may be taken to be $0.4u_w$ over the whole zone with the resultant acting at a depth of 0.4 times the depth of the neutral axis. In both cases the strain at the outermost compression fibre is taken to be 0.0035.

(*c*) The tensile strength of the concrete is ignored.

(*d*) The strains in loaded prestressing tendons and in any additional reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane.

(*e*) The tensile stresses in bonded prestressing tendons, whether initially tensioned or untensioned, and in additional reinforcement, are derived from the appropriate stress-strain curves; the stress-strain curves for prestressing tendons are given in Fig. 2.3 and those for reinforcement in Fig. 2.2. In the case of a rectangular compression zone above the neutral axis at failure stresses in bonded prestressing tendons may be obtained from Clause 4.9.4(3) and Table 4.8.

(*f*) In post-tensioned members where the tendons are unbonded, the stress in the tendons does not exceed the value given in Table 4.9 unless a higher stress can be justified on the basis of tests.

(*g*) The size and number of prestressing tendons should be such that cracking of the concrete would precede failure of the beam.

(3) Design formula

The resistance moment of a rectangular beam, or of a flanged beam in which the neutral axis lies within the flange, is given by equation:

$$M_u = f_m A_{sp} (d_l - 0.4 d_n)$$

Where A_{sp} is the cross-sectional area of the prestressing tendons,
f_m is the tensile stress in the tendon at failure,
d_l is the distance of the centroid of the tendons in the tensile zone from the outer compression surface under ultimate load conditions;
d_n is the distance of the neutral axis from the surface under ultimate load conditions.

Values for f_m and d_n may be derived from Table 4.8 for pre-tensioned members and for post-tensioned members with effective bond between the concrete and tendons. The effective prestress after all losses should be not less than 0.45 f_u where f_u is the characteristic strength of the prestressing tendon. Prestressing tendons and additional reinforcement in the compression zone should be ignored in calculating the ultimate flexural strength by this method.

Table 4.8 Conditions at failure for beams with pre-tensioned steel, or with post-tensioned steel having effective bond

$\frac{f_u A_{sp}}{u_w b d_l}$	Stress in tendons at failure as a proportion of the tensile strength f_m/f_u		Ratios of depth of neutral axis to that of the centroid of tendons in the tensile zone d_n/d_l	
	Pre-tensioning	Post-tensioning with effective bond (Upper limit)	Pre-tensioning	Post-tensioning with effective bond
0.025	1.0	1.0	0.06	0.06
0.05	1.0	1.0	0.125	0.125
0.10	1.0	1.0	0.25	0.25
0.15	1.0	1.0	0.375	0.375
0.20	1.0	0.95	0.50	0.475
0.25	1.0	0.90	0.625	0.56
0.30	1.0	0.85	0.75	0.64
0.40	0.9	0.75	0.90	0.75

Where there is doubt as to the efficiency of the bond likely to be attained with post-tensioned steel, the stress in the tendons at failure should normally be related to the effective prestress Pe (that is, the remaining prestress in the tendons after allowing for all losses) as given in Table 4.9. This relationship corresponds to the condition of no bond, and is based on the following assumptions:

(*a*) The effective prestress Pe does not exceed 0.55 f_u.

(*b*) The tendons are either in ducts or, if they are free as in hollow sections, diaphragms are provided to prevent a reduction of the effective depth.

(*c*) The effective depth is determinded by assuming that the tendons are in contact with the top of the duct or the soffit of the diaphragms.

(*d*) The compression block is rectangular with a uniform stress of $0.4\ u_w$.

In special circumstances a stress for post-tensioned steel which is intermediate between the upper and lower limits corresponding to effective bond and no bond, respectively, may be adopted.

Table 4.9 Conditions at failure for beams with post-tensioned steel without bond

$\dfrac{Pe\ A_{sp}}{u_w\ b\ d_l}$	Stress in tendons at failure as a proportion of the effective prestress f_m/Pe	Ratio of depth of neutral axis to that of the centroid of tendons in the tension zone d_n/d_l
0.025	1.7	0.11
0.05	1.6	0.20
0.10	1.4	0.35
0.15	1.3	0.49
0.20	1.2	0.60

(4) Allowance for steel reinforcement

Steel reinforcement in the tensile zone may be taken into consideration in the calculation of ultimate strength. The cross-sectional area of the tendons may then be increased by the addition of the effective area of the steel reinforcement as given by equation:

$$\text{Effective area} = \frac{A_{su} f_y}{f_u}$$

Where A_{su} is the cross-sectional area of the steel reinforcement in the tensile zone;
f_y is the specified characteristic strength of the reinforcement.

(5) Proportion of prestressing steel in beams

The size and number of prestressing tendons should be such that cracking of the concrete would precede failure of the beam.

This requirement will be satisfied for under-reinforced beams, where failure would be due to fracture of the tendons, if the percentage of steel, calculated on an area equal to the width of the beam soffit multiplied by its overall depth, is not less than 0.15. For over-reinforced beams, where failure would be due to crushing of the concrete, the maximum number and size of tendons will be governed by strain compatibility considerations see Clause 4.9.4(2).

4.9.5 SHEAR RESISTANCE OF BEAMS

Calculations for shear are only required for the ultimate load condition. At any section the ultimate shear resistance of the concrete alone, V_c, should be considered for the section both uncracked, Clause 4.9.5(1) and cracked Clause 4.9.5(2) in flexure, the lesser value taken and, if necessary, shear reinforcement Clause 4.9.5(3) provided.

For a cracked section the conditions of maximum shear with co-existent bending moment and maximum bending moment with co-existent shear should both be considered.

(1) Sections uncracked in flexure

The ultimate shear resistance of a section uncracked in flexure, V_{co}, corresponds to the occurance of a maximum principal tensile stress f_t, at the centroidal axis of the section. The value of V_{co} is given by equation:

$$V_{co} = 0.67\ bd\ \sqrt{f_t^2 + f_{cp} f_t}$$

Where f_t is $0.294\ \sqrt{u_w}$, taken as positive,
f_{cp} is the compressive stress at the centroidal axis due to prestress, taken as positive,
b is the breadth of the member which for T-, I- and L-beams should be replaced by the breadth of the rib b_w, where the position of a duct co-incides with the position of maximum principal tensile stress, e.g. at or near a support, the value of b should be reduced by the full diameter of the duct if ungrouted and by 2/3 of the diameter if grouted;
d is the overall depth of the member.

In flanged members where the centroidal axis occurs in the flange the principal tensile stress should be limited to $0.294\sqrt{u_w}$ at the intersection of the flange and web.

For a section uncracked in flexure and with inclined tendons or vertical prestress the component of prestressing force after all losses, V_p, normal to the longitudinal axis of the member, may be added to V_{co}.

(2) Section cracked in flexure

The ultimate shear resistance of a section cracked in flexure, V_{cr}, is given by equation:

$$V_{cr} = 0.045\, bd_1\sqrt{u_w} + \frac{Mt}{M} V$$

Where d_l is the distance from the extreme compression fibre to the centroid of the tendons at the section considered;

M_t is the cracking moment at the section considered.

$$M_t = (0.45\sqrt{u_w} + f_{pt})\frac{I}{y},$$

f_{pt} is the compressive stress taken as positive, due solely to prestress, at the extreme fibre at which applied loading causes tensile cracks, this fibre being at a distance y from the centroid of the section which has a second moment of area I,

V and M are the shear force and bending moment at the section considered due to ultimate loads;

V_{cr} should be taken as not less than $0.12bd_1\sqrt{u_w}$.

The value of V_{cr} calculated at a particular section may be assumed to be constant for a distance equal to $d_l/2$, measured in the direction of increasing moment, from that particular section. For a section cracked in flexure and with inclined tendons, the component of prestressing force normal to the longitudinal axis of the member should be ignored.

(3) Shear reinforcement

When V, the shear force due to the ultimate load, is less than V_c, the shear force which can be carried by the concrete, shear reinforcement need not be provided under the following circumstances:

(*a*) where V is less than $0.5V_c$; and

(*b*) in members of minor importance;

In all other members minimum shear reinforcement should be provided in the from of links such that:

$$\frac{A_{sv}}{s} \times \frac{f_{yv}}{b} = 0.4 \text{ MPa}$$

Where f_{yv} is the characteristic strength of the reinforcement which should be taken as not greater than 425 MPa;

A_{sv} is the area of cross-section of the links(s) per spacing interval, s.

When the shear force V, due to the ultimate loads, exceeds V_c, the shear reinforcement provided should in addition be such that:

$$\frac{A_{sv}}{s} = \frac{V - V_c}{f_{yv}d_2}$$

In rectangular beams, at both corners on the tensile zone, a link should pass round a longitudinal bar, a tendon, or a group of tendons having a diameter not less than the link diameter. In this clause on shear reinforcement, the depth, d_2, should be taken as the depth from the extreme compression fibre either to these longitudinal bars or to the centroid of the tendons, whichever is greater. A link should extend as close to the tension and compression faces as possible, with due regard to cover. The links provided at a cross section should between them enclose all the tendons and additional reinforcement provided at the cross-section and should be adequately anchored.

The spacing of links along a member should not exceed $0.75d_2$, nor 4 times the web thickness for flanged members. When V exceeds $1.8V_c$, the maximum spacing should be reduced to $0.5d_2$. The lateral spacing of the individual legs of the links provided at a cross-section should not exceed $0.75d_2$.

(4) Maximum shear force

In no circumstances should the shear force V, due to ultimate loads, exceed the appropriate value given by Table 4.10 multiplied by bd_2 where b is as defined in 4.9.5(1) and d_2 is the depth from the extreme compression fibre either to the centroid of the longitudinal bars or tendons enclosed by the shear links or to the centroid of all tendons, whichever is the greater.

The shear force V should include an allowance for prestress only for sections uncracked in flexure Clause 4.9.5(1).

Table 4.10 Maximum shear stress

	Grade of concrete	
	30	40
	MPa	MPa
Maximum torsional shear stress	0.45	0.52
Maximum shear stress	5.0	5.8

4.9.6 TORSIONAL RESISTANCE OF BEAMS

In general where the torsional resistance or stiffness of members has not been taken into account in the analysis of the structure, no specific calculations for torsion will be necessary, adequate control of any torsional cracking being provided by the required nominal shear reinforcement.

When it is considered that torsional resistance or stiffness of members at the ultimate limit state should be taken into account in the analysis and design of a structure the method adopted for reinforced concrete beams in Clause 3.2.8 may generally be used.

In the application of Clause 3.2.8, the following sub-clauses should be modified as below:—

(1) (*a*) T should be taken as the torsional moment due to ultimate loads.

(*b*) Where v_t exceeds the maximum torsional shear stress in Table 4.10, torsional reinforcement should be provided. In no case should the sum of the shear stresses resulting from shear force and torsion ($v + v_t$) exceed the maximum shear stress in Table 4.10, nor in the case of small sections ($B < 550$ mm) should the torsional shear stress, v_t, exceed $B/550$ mm times the maximum shear stress in Table 4.10, where B is the larger dimension of a link. The shear stress v should be taken as V/bd_2 where V is the shear force due to ultimate loads and bd_2 is the area defined in Clause 4.9.5(4).

(2) (*a*) p_{sv} and p_{st} should be taken as the characteristic strength of the links and longitudinal reinforcement respectively, where the numerical value of p_{sv} should not be taken to exceed 350 MPa.

4.9.7 REINFORCEMENT IN BEAMS

Reinforcement may, in certain circumstances, be desirable in prestressed concrete beams.

It should be noted that steel reinforcement lying parallel to the axis of prestress in a prestressed concrete member may at some time act as longitudinal reinforcement in compression. Transverse binding may be required to prevent buckling of this reinforcement particularly if its diameter is large.

Reinforcement will often be required at the ends of members to take the tensile stresses that may be induced near the ends by the prestressing force, and particularly stresses caused during transfer.

Reinforcement may be necessary, particularly where post-tensioning systems are used, to control any cracking resulting from restraint to longitudinal shrinkage of members provided by the formwork during the time before the prestress is applied.

The amount and disposition of links in rectangular beams and in the webs of flanged beams will normally be governed by considerations of shear Clause 4.9.5.

Links to resist the bursting tensile forces in the end zones of post-tensioned members should be provided in accordance with Clause 4.6.

Links should be provided in the transmission length of pre-tensioned members in accordance with the requirements of Clause 4.9.5 and using the information given in Clause 4.5.

When a prestressed concrete beam may be required to resist shock loading, it should be reinforced with closed links and longitudinal reinforcement preferably of Grade 250. Other methods of design and detailing may be used, provided it can be shown that the beam can develop the required ductility.

4.9.8 SLENDER BEAMS

The problems associated with slender prestressed concrete beams are different from those encountered in reinforced concrete beams. Two sets of conditions are likely to be experienced:

(1) During erection

When a prestressed member is being erected, the stresses at the top surface are usually small while the compressive stresses at the bottom surface may be large. When the span/breadth ratio does not exceed 60, it is normally safe to erect such beams without any limitation of stress, provided reasonable care is taken during erection. (For effective span see Clause 4.9.2). Greater span/breadth ratios may be used provided adequate temporary lateral support is given during hoisting and erection and until the permanent lateral support becomes effective. If the depth is large compared with the breadth (say 4 times), particular care should be taken, even when the span/breadth ratio is less than 60. In these cases little restraint is exercised by the stressed portion of the beam on the relatively unstressed portion, and a small lateral movement could create conditions which might lead to torsional failure.

(2) After erection and under design load

When a prestressed beam is in position and carrying its design load, the compressive stresses at the top surface of the beam may be large while the stresses at the bottom surface are usually small.

If no transverse forces occur and the depth of the beam does not exceed 2.5 times its breadth b, no decrease need be made in the permissible stresses given in Clause 4.2, provided that the ratio of the length between effective lateral supports to the breadth of the section does not exceed 30.

Where this ratio exceeds 30, it may be necessary to reduce the working stresses unless some special precautions are taken to provide the beam with adequate lateral support or lateral stiffeners.

4.9.9 BEAM AND SLAB CONSTRUCTION

In beam and slab construction, the effective width of the slab for internal beams should be taken as the least of the following:

(1) the spacing of the beams;

(2) 1/3 of the effective span;

(3) the width of the beam plus 12 times the thickness of the slab;

For edge beams beyond which the slab does not extend, the effective width of the slab should be taken as the least of the following:

(4) 1/2 the spacing of the beams;

(5) 1/6 of the effective span;

(6) the width of the beam plus 6 times the thickness of the slab;

The stresses at transfer should not exceed the permissible values, either when the width of the slab is assumed to be the effective width, or when the width of the slab is assumed to be the spacing of the beams for internal beams or 1/2 the spacing of the beams for edge beams, if these latter widths are greater than the effective width.

4.9.10 CONTINUOUS FLOORING SYSTEMS

Where a continuous floor system consists of precast prestressed concrete units together with in-situ reinforced concrete over the supports, any compression in the ends of the prestressed units may be ignored, provided that the width of the unit does not exceed 1/3 of the distance between the centres of the units, and the intervening spaces between the units at the ends are filled with structural concrete. Where, with pretensioning, it exceeds 1/3, account should be taken of the fact that it builds up to a maximum in the transmission length, see Clause 4.5.

4.9.11 DEFLECTION OF BEAMS

(1) General

The deflection of prestressed concrete beams under normal combinations of working loads, taking into consideration creep and shrinkage of the concrete, should not be sufficient to impair the strength or efficiency of the structure, or to produce cracks in finishes or superstructures, etc.

(2) Upward deflection of beams

The upward deflection of beams caused by a prestressing force may be calculated by using the normal laws of mechanics, assuming that the force creates a moment which, however, may be variable because the tendons have a variable eccentricity. When permanently applied loads are small, the upward deflection may tend to increase due to the effects of creep of the concrete, and appropriate allowance should be made in the calculation. This is of particular importance for roof members, where additional upward deflection may result from thermal effects.

For a beam of uniform cross-section with a constant eccentricity of the tendons and for which the stresses due to dead load are negligible, the upward deflection immediately after transfer is given by equation:

$$\frac{P_1 e_s L^2}{8 E c_1 I}$$

Due to the effects of creep of the concrete this deflection may be expected to increase over a period of time to a value given by equation:

$$\frac{P_2 e_s L^2}{8 E c_2 I}(1 + \o)$$

Where P_1 is the prestressing force after transfer,

P_2 is the prestressing force after the period considered,

e_s is the eccentricity of the tendons relative to the centroid of the concrete section,

E_{c1} is the modulus of elasticity of the concrete corresponding to the cube strength at transfer (See Table 2.1),

E_{c2} is the modulus of elasticity of the concrete corresponding to the cube strength after the period of time considered (See Table 2.1),

$\o$ is the creep factor (see Clause 4.3.5);

I is the second moment of area of the whole concrete section.

If finishes are to to be applied to the prestressed concrete beams, the total upward deflection should not, in general, exceed $\frac{L}{300}$.

Where uniformity of camber of adjacent units can be ensured however, a greater upward deflection may be permissible.

4.9.12 COMPOSITE BEAMS

(1) General

Prestressed concrete units may be deemed to act in conjunction with added concrete where provision is made for the horizontal shear at the surface of contact by the use of adequate shear connectors or suitable roughening or irregularities of the surface of the concrete.

In general, the analysis and design of composite concrete structures and members comprising precast prestressed concrete units should be in accordance with this Section. Particular attention should be given in the design of both the component parts and the composite section to the effect, on stresses and deflections, of the method of construction and whether or not props are used.

When precast prestressed units, having pre-tensioned tendons, are designed as continuous members and continuity is obtained with reinforced concrete cast in situ over the supports the compressive stresses due to prestress in the ends of the units may be ignored over the transmission length for the tendons in assessing the strength of sections.

Where an in-situ slab is cast on a prestressed concrete beam, the effective width of the slab should be in accordance wth Clause 4.9.9.

(2) Compression in the concrete

The compressive stress in the prestressed concrete unit at the interface with the added concrete may be increased above the values given in Clause 4.2.2 by not more than 50% provided the ultimate failure of the composite beam would be due to excessive elongation of the steel.

(3) Tension in the concrete

When the composite member considered in design comprises precast prestressed concrete units and in-situ concrete and flexural tensile stresses are induced in the in-situ concrete by imposed service loading, the tensile stresses in the in-situ concrete at the contact surface should be limited to the values given in Table 4.11. These values may, however, be increased by 50% provided the permissible tensile stress in the prestressed concrete unit see Clause 4.2.2 is reduced by the same numerical amount. When the in-situ concrete is not in direct contact with a precast prestressed unit, the flexural tensile stresses in the in-situ concrete should be limited by cracking considerations.

Where continuity is obtained with reinforced concrete cast in-situ over the supports, the flexural tensile stresses and the hypothetical tensile stresses in the precast prestressed units at the supports should normally be limited in accordance with Clause 4.2.2.

Table 4.11 Permissible bending tensile stresses in added concrete

Specified grade strength of added concrete	25	30	40
Maximum tensile stress MPa	3.2	3.6	4.4

(4) Shear

The analysis of composite sections resisting vertical shear due to ultimate loads should be carried out in accordance with Clause 4.9.5. However, when in-situ concrete is placed between precast prestressed units and the composite concrete section is used in design, the principal tensile stress should not exceed $0.294\sqrt{u_w}$ anywhere in the prestressed units; this stress should be calculated by making due allowance for the construction sequence.

The horizontal shear stress due to the design service loads at the contract surface of the precast and in-situ components at any point along the length of the member is given by equation:

$$v_h = \frac{V_d S_c}{I b_e}$$

Where v_h is the horizontal shear stress at the contact surface,
V_d is the total vertical shear at the point considered due to the design service load,
S_c is the first moment of the concrete to one side of the contact surface, about the neutral axis of the transformed composite section,
I is the second moment of area of the composite section;
b_e is the width of the contact surface.

The shear stresses along the contact surface under maximum shear conditions calculated as above should not exceed the values given in Table 4.12 for beam and slab construction and for the three types of surface considered. The types of surface are defined as follows:

(*a*) *Type 1*. Where links are not provided and the contact surface has been prepared in the following manner. When the concrete in the precast member has set but not hardened, the surface which will subsequently receive the in-situ concrete should be sprayed with a fine spray of water or brushed with a stiff brush, just sufficient to remove the outer mortar skin and expose the larger aggregate without disturbing it.

(*b*) *Type 2*. Where the contact surface is not as described in (*a*) and where links are provided having a minimum cross-sectional area 0.15% of the contact area. The spacing of links should not be excessive; for composite T-beams with an in-situ flange, the spacing should not exceed 4 times the minimum thickness of the in-situ concrete nor 600 mm.

(*c*) *Type 3*. Where the contact surface has been prepared as described in (*a*) (or where this treatment proved impracticable and the surface skin and laitence has been removed by sand blasting or the use of a needle gun and not by hacking) and minimum links as described in (*b*) are provided. When links are provided in excess of the minimum, the allowable shear stresses given in Table 4.12 for this type of surface may be increased by 0.5 MPa for each additional area of links equal to 1% of the contact area.

Table 4.12 Horizontal shear stresses for composite beam and slab sections

	Specified grade strength of concrete MPa		
	25	30	40
	MPa	MPa	MPa
Surface Type 1	0.38	0.45	0.54
Surface Type 2	0.36	0.38	0.42
Surface Type 3	1.22	1.25	1.32

Shear reinforcement provided in the composite section to resist vertical shear should extend into the in-situ slab and can be deemed to resist horizontal shear at the precast/in-situ interface.

In composite slabs when links are not provided, the horizontal shear stresses may be limited as follows:

(i) for surface Type 1 the stress should not exceed 1.2 times the appropriate value given in Table 4.12.

(ii) where the top surface of the precast concrete unit has not been so treated, the stress should be limited to 0.8 times the appropriate value given in Table 4.12 for surface Type 2.

(5) Differential shrinkage

Where there is an appreciable difference between the age and quality of the concrete in the components, differential shrinkage may lead to increased stresses in the composite section and these should be investigated.

In computing the tensile stresses due to differential shrinkage, a value will be required for the differential shrinkage coefficient (the difference in total free strain between the two components of the composite member), the magnitude of which can be obtained from Clause 4.3.4.

The effects of differential shrinkage will be reduced by creep and the reduction coefficient is given by equation:

$$\frac{1 - e^{-\phi}}{\phi},$$

Where ϕ is the creep factor (see Clause 4.3.5.);
e is the base of Napierian logarithms.

(6) Continuity in composite construction

When continuity is obtained in composite construction by providing reinforcement over the supports, consideration should be given to the secondary effects of differential shrinkage and creep on the moments in continuous beams and on the reactions at the supports. The hogging restraint moment, M_{cs}, at an internal support of a continuous composite beam and slab section due to differential shrinkage is given by equation:

$$M_{cs} = \varepsilon_{diff} E_{cf} A_{cf} a_{cent} \frac{(1 - e^{-\phi})}{\phi}$$

Where ε_{diff} is the differential shrinkage strain,
E_{cf} is the modulus of elasticity of the flange concrete,
A_{cf} is the area of the effective concrete flange,
a_{cent} is the distance of the centroid of the concrete flange from the centroid of the composite section,
ϕ is the creep factor (see Clause 4.3.5);
e is the base of Napierian logarithms.

The restraint moment, M_{cs}, will be modified with time by creep due to dead load and creep due to the prestress in the precast units. The restraint moment due to prestres may be taken as the restraint moment which would have been set up if the composite section as a whole had been prestressed, multiplied by a reduction coefficient equal to $(1 - e^{-\phi})$.

The information given in Clause 4.9.12(5) on differential shrinkage should be used in assessing a value for the differential shrinkage strain.

4.10 *COMPRESSION MEMBERS*

4.10.1 GENERAL

A concrete member acting in compression is sometimes given a prestress to resist bending arising either from continuity, eccentricity of load or other cause.

Provided that the tendons are fixed relative to the compression member at a sufficient number of points, the compression member will not buckle under the prestressing forces since the compression member and the line of action of the forces deflect together. However, a prestressed concrete member requires examination for stability under external loading in a similar manner to other forms of construction.

For the purpose of this Code, a compression member is only considered to be prestressed when the mean prestress in the concrete section imposed by the tendons exceeds 2.7 MPa.

4.10.2 BASIS OF DESIGN OF COMPRESSION MEMBERS

When the ratio of the effective length to the least lateral dimension is less than 15, the design of prestressed concrete compression members should conform with the requirements that the computed stresses in the concrete and in the steel should not exceed the permissible stresses given in Clause 4.2 for normal combinations of loading during transfer, handling and construction and under working loads.

When a compression member is subjected to a combination of axial loading and bending due to dead and live loads and the stress in the concrete due to axial loading is less than 25% of the maximum stress in the concrete due to bending under this loading, then the permissible compressive stress in the concrete may be taken as that given for bending in Clause 4.2.2(1). When the ratio of stress due to axial loading to maximum stress due to bending is greater than 1/4, the permissible compressive stress in the concrete should be that given for direct compression in Clause 4.2.2(1).

Computation of the stresses in the materials should be based on the assumptions given in Clause 4.9.3.

When the ratio of the effective length of the least lateral dimension is greater than 15, the stability of the compression member should be considered. For guidance, the reduction coefficients for the permissible loads on long columns given in Section 3 may be used.

No recommendations are made for the calculation of the ultimate strength of compression members, but the disposition of the tendons and of any additional steel reinforcement should be favourable to the development of an adequate margin of security against failure.

4.10.3 REINFORCEMENT IN COMPRESSION MEMBERS

The provision of reinforcement in compression members should, in general, conform with the recommendations given for beams in Clause 4.9.7. In addition where a compression member has longitudinal reinforcement, it should also have transverse or helical reinforcement so disposed as to provide all necessary restraint against the buckling of each of the longitudinal reinforcements. Every bar in a column near the face should be properly linked. The ends of such transverse reinforcement should be properly anchored.

The pitch of transverse reinforcement should be not more than the least of the three following distances:

(1) The least lateral dimension of the members;

(2) 12 times the diameter of the smallest longitudinal reinforcement in the member;

(3) 300 mm.

Where, however, the longitudinal reinforcement is provided only for the purpose of holding the prestressing steel in position and the diameter of such reinforcement does not exceed 10 mm nominal links will in general be satisfactory.

4.11 *OTHER STRUCTURES*

4.11.1 STATICALLY INDETERMINATE STRUCTURES

In calculations for working loads of systems which are statically indeterminate account should be taken not only of the applied and dead loads, but also of the strains in the structure caused by the application of the prestress and also of the subsequent creep and shrinkage in the concrete after prestressing: this is particularly important when restraints are added during or after the initial prestressing.

4.11.2 SPECIAL STRUCTURES

Some special structures cannot easily be analysed as regards the stresses in the members during transfer, handling and construction and under working loads, or as regards their ultimate strength. The strength requirements of this Code may be deemed to be satisfied for such structures if it can be established by test that they behave satisfactorily as working loads and have an adequate ultimate strength. For beams and other members in bending, the ultimate strength should be shown to satisfy the second requirement of Clause 4.9.1.

5. DESIGN AND DETAILING: PRECAST CONCRETE

5.1 *GENERAL*

5.1.1 SCOPE

The Section is concerned with the additional considerations which arise in design and detailing when precast concrete members or precast concrete components are incorporated into a structure. It does not cover the use of plain concrete for walls, large panels, composite construction or when a structure in its entirety is of precast concrete construction.

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When a compression member is subjected to a combination of axial loading and bending due to dead and live loads and the stress in the concrete due to axial loading is less than 25% of the maximum stress in the concrete due to bending under this loading, then the permissible compressive stress in the concrete may be taken as that given for bending in Clause 4.2.2(1). When the ratio of stress due to axial loading to maximum stress due to bending is greater than 1/4, the permissible compressive stress in the concrete should be that given for direct compression in Clause 4.2.2(1).

Computation of the stresses in the materials should be based on the assumptions given in Clause 4.9.3.

When the ratio of the effective length of the least lateral dimension is greater than 15, the stability of the compression member should be considered. For guidance, the reduction coefficients for the permissible loads on long columns given in Section 3 may be used.

No recommendations are made for the calculation of the ultimate strength of compression members, but the disposition of the tendons and of any additional steel reinforcement should be favourable to the development of an adequate margin of security against failure.

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The pitch of transverse reinforcement should be not more than the least of the three following distances:

(1) The least lateral dimension of the members;

(2) 12 times the diameter of the smallest longitudinal reinforcement in the member;

(3) 300 mm.

Where, however, the longitudinal reinforcement is provided only for the purpose of holding the prestressing steel in position and the diameter of such reinforcement does not exceed 10 mm nominal links will in general be satisfactory.

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5.1.2 BASIS OF DESIGN

The design philosophy set out in Section 2 applies equally to precast and in-situ construction and therefore, in general, the recommended methods of design and detailing given in Section 3 for reinforced concrete and those for prestressed concrete given in Section 4 apply also to precast concrete construction. Clauses in Section 3 and 4 which do not apply are either specifically worded for in-situ construction or are modified by this section.

5.1.3 HANDLING STRESSES

Precast units should be designed to resist without permanent damage all stresses induced by handling, storage, transport and erection. When necessary the positions of lifting and supporting points should be specified. Consultation at the design stage with those responsible for handling is an advantage. The design should take account of the effect of snatch-lifting and placing on to supports.

5.1.4 CONNECTIONS AND JOINTS

The design of connections is of fundamental importance in precast concrete construction and must be carefully considered. The overall stability of the structure as well as the compatibility of the design and details of parts and components should be ascertained. Joints to allow for movements due to shrinkage, thermal effects and possible differential settlement of foundations are of as great importance in precast as in-situ construction. The number and spacing of such joints should be determined at an early stage in the design. In the design of beam and slab ends on corbels and nibs particular care should be taken to provide overlap and anchorage of all reinforcement adjacent ot the contact faces, full regard being paid to constructional tolerances.

5.2 *BEARINGS FOR PRECAST CONCRETE MEMBERS*

5.2.1 CONCRETE CORBELS

A corbel is a short cantilever beam in which the principal load is applied such that the distance between the line of action of the load and the face of the supporting member is less than 0.6d and the depth at the outer edge of the bearing is not less than 1/2 of the depth at the face of the supporting member.

The depth at the face of the supporting member should be determined from shear conditions in accordance with Clause 3.2.7(1).

The main tension reinforcement in a corbel should be designed and the strength of the corbel checked, on the assumption that it behaves as a simple strut and tie system. The reinforcement so obtained should be not less than 0.4% of the section at the face of the supporting member and should be adequately anchored. At the front face of the corbel, the reinforcement should be anchored either by welding to a transverse bar of equal strength or by bending back the bars to form a loop; in the latter case, the bearing area of the load should not project beyond the straight portion of the bars forming the main tension reinforcement.

When the corbel is designed to resist a stated horizontal force, additional reinforcement should be provided to transmit this force in its entirety; the reinforcement should be welded to the bearing plate and should be adequately anchored within the supporting member.

Shear reinforcement should be provided in the form of horizontal links distributed in the upper 2/3 of the effective depth of the corbel at the column face; this reinforcement should not be less than 1/2 of the area of the main tension reinforcement and should be adequately anchored.

5.2.2 CONTINUOUS CONCRETE NIBS

Where continuous nib less than 300 mm deep provides a bearing as on a boot lintel, the nib should normally be designed as a short cantilever slab in accordance with the following provisions and definitions:

(1) the projection of the nib should be sufficient to provide an adequate width of bearing for the type of member to be supported see Clause 5.2.3. The reinforcement in the nib and any reinforcement in the supported member should have a minimum nominal overlap on plan of 60 mm;

(2) The line of action of the load should be assumed to occur at the outer edge of the loaded area, i.e. at the front edge of the nib, at the beginning of the chambered edge, or at the outer edge of the bearing pad as appropriate;

(3) The maximum bending moment in the nib should be taken as the product of the load supported and the distance from its line of action to the nearest vertical leg of the links (see (6)) in the beam. The tension reinforcement in the nib should not be less than that required by Clause 3.2.1(3) and should be adequately anchored;

(4) the tension reinforcement should extend as near to the front face of the nib as considerations of adequate cover will allow and be anchored there, either by welding to a transverse bar of equal strength or by bending the bars through 180° to form loops in the horizontal or vertical plane. Vertical loops should be of bar size not greater than 12 mm;

(5) the shear resistance of the nib should be checked in accordance with Clause 3.2.7.

(6) links capable of transmitting, in addition to any other forces which they resist, the load from the nib to the compression zone of the main beam should be provided in the main beam.

5.2.3 WIDTH OF BEARINGS FOR PRECAST CONCRETE UNITS

The width of bearing of precast units should be sufficient to ensure proper anchorage of tension reinforcement see Clause 3.1.10. Precast concrete units should have a bearing of at least 100 mm on masonry or brickwork supports and of at least 75 mm on steel or concrete; this bearing may be reduced by taking into account relevant factors such as tolerances, loading, span, height of support and the provision of continuity reinforcement. Nevertheless, when reduced bearings are used, the minimum anchorage lengths of reinforcement required by 3.1.10 must be provided and precautions must be taken to ensure that collapse of the unit cannot occur due to accidental displacement during erection.

5.2.4 BEARING STRESSES

The contact surfaces should not contain excessive irregularities and when adequate intermediate padding is not provided, the compressive stress in the contact area should not normally exceed 0.25 u_w under the working loads. When the members are made of concretes of different strength, the lower concrete strength is applicable. Higher bearing stresses may be used where suitable measures are taken to prevent splitting or spalling of the concrete at the interface, such as the provision of well-defined bearing areas and additional binding reinforcement in the ends of the members. Stresses in excess of 0.5 u_w due to working loads, should only be used where justified by tests. Direct bearing connections should not be used for column/column or wall/wall connections either with or without flexible padding.

5.2.5 HORIZONTAL FORCES OR ROTATING IN BEARING

The presence of horizontal forces at a bearing can reduce the load-carrying capacity of the supporting member considerably by causing pre-mature splitting or shearing. These forces may be due to creep, shrinkage and temperature effects or result from misalignment, lack of plumb or other causes. When they are likely to be significant, these forces should be allowed for in designing and detailing the connection by providing either:

(1) sliding bearings; or

(2) suitable lateral reinforcement in the top of the supporting member; and

(3) continuity reinforcement to tie together the ends of the supported members.

Where owing to large spans or other reasons, large rotations are likely to occur at the end supports of flexural members, suitable bearings capable of accommodating these rotations should be used.

5.3 *JOINTS BETWEEN PRECAST CONCRETE MEMBERS*

5.3.1 GENERAL

The critical sections of members close to joints should be designed to resist the worst combinations of shear, axial force and bending caused by the vertical and horizontal forces. When the design of the precast members is based on the assumption that the joint between them is not capable of transmitting moment, suitable precautions should be taken to ensure that if any cracking develops it will not be unsightly and will not excessively reduce the member's resistance to shear or axial force.

Where a space is left between two or more precast concrete units, which is to be filled later with in-situ concrete or mortar, the space should be large enough for the filling material to be placed easily and compacted sufficiently to completely fill the gap, without the need for abnormally high standards of workmanship or supervision. The assembly instructions should contain definite information as to the stage during construction when the gap should be filled.

The majority of joints will incorporate a structural connection and this should be considered in the design of the joint.

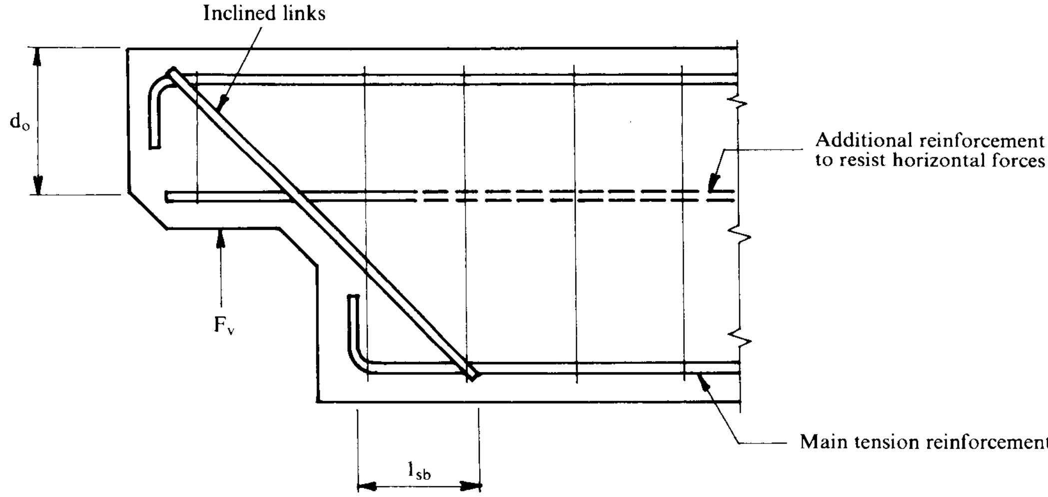
5.3.2 HALVING JOINT

For the type of joint shown in Fig. 5.1, the maximum vertical working load, F_v, should not exceed that given by equation:

$$F_v = 2p_v b d_o$$

Where b is the breadth of the beam,
p_V is the permissible shear stress (see Table 3.1);
d_o is the depth to additional reinforcement to resist horizontal forces.

When determining the value of F_v consideration should be given to the method of erection and the forces involved.



Note. This figure is diagrammatic.

Fig. 5.1 Halving joint

The joint should be reinforced by inclined links so that the vertical component of force in the link is equal to F_V, as given in equation:

$$F_v = A_{sv}p_{sv} \cos 45° \text{ for link at } 45°$$

Where A_{sv} is the cross-sectional area of the inclined links;
p_{sv} is the permissible stress of the inclined links.

The links must intersect the line of action of Fv.

In the compression face of the beam the links should be anchored in accordance with Clause 3.1.10. In the tension face of the beam, the horizontal component, F_h, which for 45° links is equal to F_V, should be transferred to the main reinforcement. If the main reinforcement is continued straight on without hooks or bends the links may be considered anchored if:

$$\frac{F_h}{2\,\Sigma o\, l_{sb}} < \text{Pba (see Table 3.1)}$$

Where Σo is the perimeter of the main reinforcement;
l_{sb} is the length of the straight reinforcement beyond the intersection with the link.

If the main reinforcement is hooked or bent vertically, the inclined links should be anchored by bending them parallel to the main reinforcement; in this case, or if inclined links are replaced by bent-up bars, the bearing stress inside the bends should not exceed the value given in Clause 3.1.10(6).

If there is a possibility of a horizontal load being applied to the joint, horizontal links should be provided to carry the load as shown in Fig. 5.1; such links should also be provided if there is a possibility of the inclined links being displaced so that they do not intersect the line of action of F_V. The joint may alternatively be reinforced with vertical links, designed in accordance with Clause 3.2.7 provided the links are adequately anchored.

5.3.3 SIDE JOINT

When a secondary member is supported within the span of a primary beam, attention should be paid, in designing the connection between them, to the conditions during construction as well as those due to the design working loads. The possibility of torsion should be considered. When the joint is formed when the secondary beam penetrating into a pocket formed in the side of the primary beam, the reduced section of the latter should be used in design for conditions before the grout in the pocket has reached its characteristic strength, unless the main beam is propped during construction.

The recommendations of Clause 5.2.1 and 5.2.2 apply where the joint is formed outside the limits of the normal cross-section of the main beam.

6. LOAD TESTING

Load testing of components and structures or parts of structures should be carried out in accordance with Sections 9.5 and 9.6 of the British Standard Code of Practice BS 8110: Part 2.

7. ALTERNATIVE RECOMMENDATIONS

The recommendations of BS 8110, or BS 5400 as modified by the Civil Engineering Manual, Volume 5, Chapter 4, as appropriate to the type of structure, may be used as an alternative to recommendations of this Code. In such cases the following specific requirements shall apply—

(1) The characteristic dead load, imposed load and wind load should be taken as the dead load, imposed load and wind load calculated in accordance with the provisions of the Building (Construction) Regulations;

(2) The characteristic strength of concrete should be taken as the specified grade strength given in the Building (Construction) Regulations;

(3) The characteristic strength of concrete used for design should not exceed 45 MPa;

(4) The characteristic strength of concrete used for design should not be increased in respect of age at loading;

(5) The short-term modulus of elasticity, creep, shrinkage and other properties of concrete should be taken from this Code instead of from the British Standards;

(6) In ultimate strength design each of the combinations of loading given in Table 7.1 should be considered, instead of the load combinations given in BS 8110, or in addition to the load combinations given in BS 5400, as appropriate to the type of structure.

(7) Specified characteristic strength of grade 460/425 reinforcement, f_y, should not to exceed—

460 MPa—6 mm dia up to and including 16 mm dia

425 MPa—Over 16 mm dia

(8) Concrete material specification and construction Clauses 6.1, 6.2, 6.3, 6.4, 6.6, 6.7, 6.8, 6.9, 6.10, 6.11 of BS 8110 are not suitable and should not be used.

Table 7.1 Load combinations

Load combination	Load type					
	Dead		Imposed		Earth and Water Pressure	Wind
	Adverse	Beneficial	Adverse	Beneficial		
1. Dead and imposed (and earth and water pressure)	1.5	1.0	1.7	0	1.5	—
2. Dead and wind (and earth and water pressure)	1.5	1.0	—	—	1.5	1.5
3. Dead and wind and imposed (and earth and water pressure)	1.3	1.0	1.3	0	1.3	1.3

(9) Immediately after compaction and finishing concrete should be protected against harmful effects of weather, running water and drying out. The protection should be applied by using one of the following methods—

(*a*) For non-liquid retaining structures

(i) Except for surfaces against which concrete or applied finishes have subsequently to be placed, the concrete should be cured by application of an approved liquid curing membrane. Application should be by a low-pressure spray at the rate recommended by the manufacturer. On horizontal surfaces the membrane should be applied immediately after finishing the concrete, and on vertical surfaces immediately after removing the formwork.

(ii) After thoroughly wetting, the concrete should be covered with a layer of approved water-proof paper or plastic membrane until the concrete has reached the age of 4 days.

(iii) The concrete should be completely covered with a layer of fine aggregate at least 25 mm thick, hessian, sacking, canvas or similar absorbent material. Such covering layer should be kept constantly wet until the concrete has reached the age of 4 days.

(*b*) For liquid-retaining structures

(i) After completion of the finishing process, all exposed surfaces shall be covered with a layer of approved plastic sheeting until such time as the concrete has hardened sufficiently to permit water curing. Water curing shall be effected whenever possible by the continuous spraying of cool water for a period of 2 days. Particular care shall be taken to avoid thermal shock at the surface of the concrete caused by the intermittent application of large quantities of cold water.

(ii) Curing method as detailed in sub-clause (*b*)(i) should continue or be substituted by the method detailed in sub-clause (*a*)(ii) until the concrete has reached the age of 7 days.

(iii) Formwork to concrete walls and columns should be kept cool by water sprays as soon as the concrete has hardened sufficiently and until the formwork is removed, then one of the curing methods as detailed in sub-clauses (*a*)(ii) and (*a*)(iii) should be adopted until the concrete has reached the age of 7 days.