

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

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

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

1.3 Scope of the Manual

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

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

- The *Manual* has been drafted to comply with BS EN 1992-1-1¹ (EC2 Part 1-1) and BS EN 1992-1-2² (EC2 Part 1-2) together with the UK National Annexes.
- The assumed design working life of the structure is 50 years (see BS 8500³).
- The structures are braced and non-sway.
- The concrete is of normal weight concrete (see Appendix D for properties).
- The structure is predominantly *in-situ*. For precast concrete, reference should be made to the EC2 manual for precast concrete⁴.
- Normal structure/cladding and finishes interfaces are assumed. For sensitive cladding or finishes reference should be made to the deflection assessment methods in EC2¹.
- Only initial design information is given with regard to prestressed concrete.
- Prestressed concrete members have bonded or unbonded internal tendons.
- Only tabular methods of fire design are covered.
- The use of mild steel reinforcement is not included. Refer to other standards if its use is required.
- Structures requiring seismic resistant design are not covered. Refer to BS EN 1998⁵ (Eurocode 8).
- For elements of foundation and substructure the *Manual* assumes that appropriate section sizes and loads have been obtained from BS EN 1997⁶.
- The *Manual* can be used in conjunction with all commonly used materials in construction; however the data given assumes the following:
 - concrete up to characteristic cylinder strength of 50MPa (cube strength 60MPa)
 - high-tensile reinforcement with characteristic strength of 500MPa with Class B ductility
 - ribbed wire fabric reinforcement with characteristic strength of 500MPa with Class A ductility. Moment redistribution is limited to 20% and yield line design is excluded, except where noted
 - prestressing tendons with 7-wire low-relaxation (Class 2) strands.

For structures or elements outside this scope EC2^{1,2,7} should be used.

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

4.5 Fire resistance

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

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

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

Notes

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

4.8.7.2 Provision of reinforcement

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

Tension reinforcement

Reinforcement can now be calculated by the following formula:

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

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

Compression reinforcement

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

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

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

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

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

Shear reinforcement

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

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

Bar arrangements

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

In beams, this area should generally be provided by not less than 2 nor more than 8 bars. In slabs, the bar spacing should not be less than 150mm nor more than 300mm; the bars should not be less than 10mm nor normally more than 20mm in diameter.

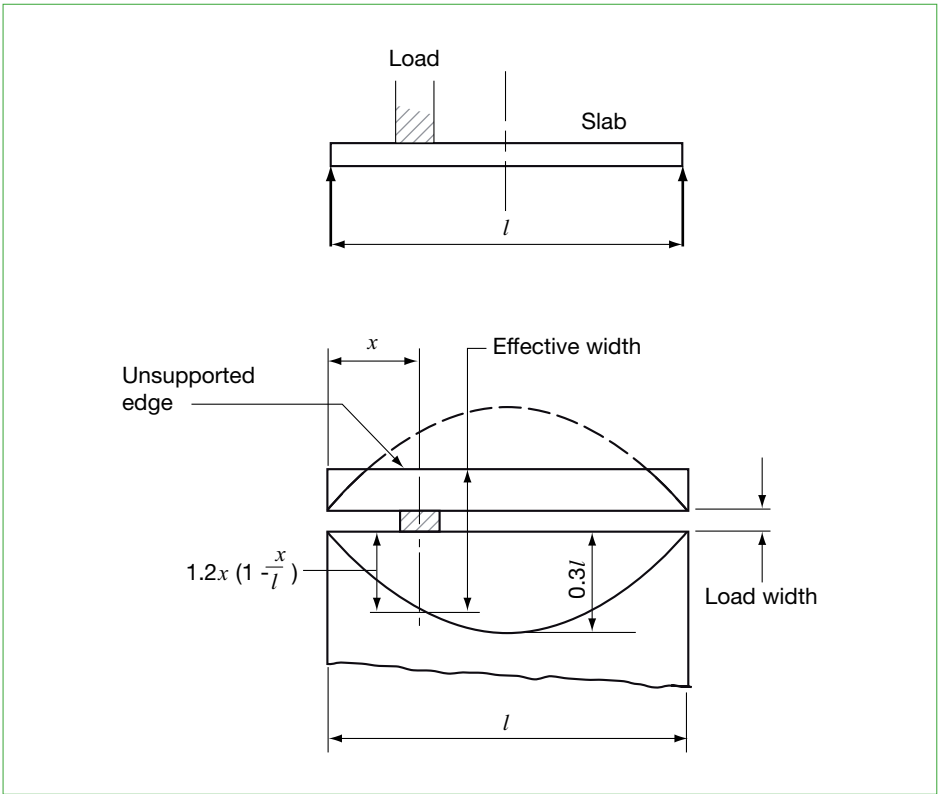


Fig 5.1 Effective width of solid slab carrying a concentrated load near an unsupported edge

5.2.3.3 Two-way spanning slabs on linear supports

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

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

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

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