

# Structural use of concrete —

## Part 1: Code of practice for design and construction

ICS 91.080.40

## Committees responsible for this British Standard

The preparation of this British Standard was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/2, Structural use of concrete, upon which the following bodies were represented:

Association of Consulting Engineers

British Cement Association

British Precast Concrete Federation Ltd.

Concrete Society

Department of the Environment (Building Research Establishment)

Department of the Environment (Property and Buildings Directorate)

Department of Transport (Highways Agency)

Federation of Civil Engineering Contractors

Institution of Civil Engineers

Institution of Structural Engineers

Steel Reinforcement Commission

This British Standard, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 March 1997

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First published August 1985  
Second edition March 1997

The following BSI references relate to the work on this standard:

Committee reference B/525/2  
Draft for comment 95/105430 DC

ISBN 978 0 580 59893 7

### Amendments issued since publication

| Amd. No. | Date             | Comments                |
|----------|------------------|-------------------------|
| 9882     | September 1998   |                         |
| 13468    | 27 May 2002      |                         |
| 16016    | 30 November 2005 | Indicated by a sideline |
| 17307    | 31 August 2007   | See national foreword   |

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## Foreword

This part of BS 8110 has been prepared by Subcommittee B/525/2. It is a revision of BS 8110-1:1985 which is withdrawn.

BS 8110-1:1997 incorporates all published amendments made to BS 8110-1:1985.

Amendment No. 1 (AMD 5917) published on 31 May 1989;

Amendment No. 2 (AMD 6276) published on 22 December 1989;

Amendment No. 3 (AMD 7583) published on 15 March 1993;

Amendment No. 4 (AMD 7973) published on 15 September 1993.

It also includes changes made by incorporating Draft Amendments Nos. 5 and 6 to BS 8110-1:1985 issued for public comment during 1994 and 1995.

Amendment No. 1 to BS 8110-1:1997 detailed the insertion of various references to different cements used in concrete construction, covered by BS 5328 and the recommendations of BS 5328 for concrete as a material, up to the point of placing, curing and finishing in the works.

Amendment No. 2 to BS 8110-1:1997 dealt with the change of the partial safety factor for reinforcement  $\gamma_m$ , from 1.15 to 1.05.

Amendment No. 3 to BS 8110-1:1997 has been necessitated through the adoption of BS 8500, *Concrete — Complementary British Standard to BS EN 206-1*, the withdrawal of BS 5328, *Concrete*, and the adoption of 500 Grade high yield steel for use in the UK

Amendment No. 3 aligns the recommendations for exposure classification and durability in BS 8110-1:1997 with those in BS 8500. Recommendations for durability have been removed from BS 8110-1:1997 and replaced with references to the appropriate guidance in BS 8500. Essential changes have been made to account for the terminology of BS 8500 (e.g. the dual cylinder/cube strength notation, and “strength classes” instead of “grades”).

The industry has agreed to adopt Grade 500 high yield steel for reinforcement in line with the common grade available in Europe. For some types of process in the manufacture of reinforcement this means that the required properties will be closer to the limits than for Grade 460. For this reason it is considered wise to increase the material factor to 1.15 until such time that there is sufficient data available to demonstrate a return to the value of 1.05 introduced by Amendment No. 2 to BS 8110-1:1997.

Where Grade 460 deformed bars are used in design, the changes to the partial safety factor for reinforcement, and the effects of these changes, referred to in Amendment No. 3 to BS 8110-1:1997 may be disregarded and a partial factor of 1.05 for such reinforcement be used.

NOTE Grade 460 deformed bars were specified and/or referred to in BS 4449:1997, BS 4482:1985, BS 4483:1998 and BS 8666:2000 which have been withdrawn and superseded by 2005 editions.

The start and finish of text introduced by Amendment No. 4 is indicated in the text by tags  $\mathbb{A}_4$   $\mathbb{A}_4$ .

Users of this British Standard need to note the ongoing introduction and publication of European Standards and supersession of British Standards.

It has been assumed in the drafting of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

**Compliance with a British Standard cannot confer immunity from legal obligations.**

### Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 159 and a back cover.

The BSI copyright notice displayed in this document indicates when the document was last issued.

Sidelining in this document indicates the most recent changes by amendment.



# Section 1. General

## 1.1 Scope

This part of BS 8110 gives recommendations for the structural use of concrete in buildings and structures, excluding bridges and structural concrete made with high alumina cement.

The recommendations for robustness have been prepared on the assumption that all load-bearing elements, e.g. slabs, columns and walls are of concrete. In a structure where concrete elements such as floor slabs are used in conjunction with load-bearing elements of other materials, similar principles are appropriate but, when adequate robustness is provided by other means, the ties recommended by this code may not be required.

NOTE 1 Where appropriate British Standards are available for precast concrete products, e.g. kerbs and pipes, it is not intended that this code should replace their more specific requirements.

## 1.2 References

### 1.2.1 Normative references

This part of BS 8110 incorporates, by reference, provisions from specific editions of other publications. These normative references are cited at the appropriate points in the text and the publications are listed on page 155. Subsequent amendments to, or revisions of, any of these publications apply to this part of BS 8110 only when incorporated in it by updating or revision.

### 1.2.2 Informative references

This Part of BS 8110 refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on the inside back cover, but reference should be made to the latest editions.

## 1.3 Definitions

For the purposes of this part of BS 8110, the following definitions apply.

### 1.3.1 General

#### 1.3.1.1

#### **design ultimate load<sup>1)</sup>**

the design load for the ultimate limit state

#### 1.3.1.2

#### **design service load<sup>1)</sup>**

the design load for the serviceability limit state

### 1.3.2 Terms specific to flat slabs (see 3.7)

#### 1.3.2.1

#### **flat slab**

a slab with or without drops and supported, generally without beams, by columns with or without column heads. It may be solid or may have recesses formed on the soffit so that the soffit comprises a series of ribs in two directions (waffle or coffered slab)

#### 1.3.2.2

#### **column head**

local enlargement of the top of a column providing support to the slab over a larger area than the column section alone

#### 1.3.2.3

#### **drop**

thickening of a slab in the region of a column

<sup>1)</sup> Design loads are obtained by multiplying the characteristic loads by the appropriate partial safety factors for loading ( $\gamma_f$ ).

**1.3.3 Terms specific to perimeters (see 3.7.7)****1.3.3.1****perimeter**

a boundary of the smallest rectangle that can be drawn round a loaded area which nowhere comes closer to the edges of the loaded area than some specified distance  $l_p$  (a multiple of  $0.75d$ )

NOTE See 3.7.7.8 for loading close to a free edge, and Figure 3.16 for typical cases.

**1.3.3.2****failure zone**

an area of slab bounded by two perimeters  $1.5d$  apart

NOTE See 3.7.7.8 for loading close to a free edge.

**1.3.3.3****effective length of a perimeter**

the length of the perimeter reduced, where appropriate, for the effects of holes or external edges

**1.3.3.4****effective depth ( $d$ )**

the average effective depth for all effective reinforcement passing through a perimeter

**1.3.3.5****effective steel area**

the total area of all tension reinforcement that passes through a zone and that extends at least one effective depth (see 1.3.3.4) or 12 times the bar size beyond the zone on either side

NOTE The reinforcement percentage used to calculate the design ultimate shear stress  $v_c$  is given by:

$$v_c = \frac{100 \times \text{effective reinforcement area}}{ud}$$

where

$u$  is the outer perimeter of the zone considered;

$d$  is as defined in 1.3.3.4.

**1.3.4 Terms specific to walls (see 3.9)****1.3.4.1****wall**

a vertical load-bearing member whose length exceeds four times its thickness

**1.3.4.2****unbraced wall**

a wall providing its own lateral stability

**1.3.4.3****braced wall**

a wall where the reactions to lateral forces are provided by lateral supports

**1.3.4.4****lateral supports**

an element (which may be a prop, a buttress, a floor, crosswall or other horizontal or vertical element) able to transmit lateral forces from a braced wall to the principal structural bracing or to the foundations

**1.3.4.5****principal structural bracing**

strong points, shear walls or other suitable bracing providing lateral stability to the structure as a whole

**1.3.4.6****reinforced wall**

a concrete wall containing at least the minimum quantities of reinforcement given in 3.12.5

**1.3.4.7****plain wall**

a wall containing either no reinforcement or insufficient to satisfy the criteria in **3.12.5**

NOTE For a “plain wall”, any reinforcement is ignored when considering the strength of the wall.

**1.3.4.8****stocky wall**

a wall where the effective height divided by the thickness ( $l_e/h$ ) does not exceed 15 (braced) or 10 (unbraced)

**1.3.4.9****slender wall**

a wall other than a stocky wall

**1.3.5**

**Terms relating to bearings for precast members** (see **5.2.3**)

**1.3.5.1****simple bearing**

a supported member bearing directly on a support, the effect of projecting steel or added concrete being discounted

**1.3.5.2****dry bearing**

a bearing with no immediate padding material

**1.3.5.3****bedded bearing**

a bearing with contact surfaces having an immediate padding of cementitious material

**1.3.5.4****non-isolated member**

a supported member which, in the event of loss of an assumed support, would be capable of carrying its load by transverse distribution to adjacent members

**1.3.5.5****bearing length**

the length of support, supported member or intermediate padding material (whichever is the least) measured along the line of support

**1.3.5.6****net bearing width** (of a simple bearing)

the bearing width (of a simple bearing) after allowance for ineffective bearing and for constructional inaccuracies (see Figure 5.4)

**1.4 Symbols**

For the purposes of this part of BS 8110, the following symbols apply.

- $\gamma_f$  partial safety factor for load.
- $\gamma_m$  partial safety factor for strength of materials.
- $E_n$  nominal earth load.
- $G_k$  characteristic dead load.
- $Q_k$  characteristic imposed load.
- $W_k$  characteristic wind load.
- $f_{cu}$  characteristic strength of concrete.
- $f_y$  characteristic strength of reinforcement.
- $f_{pu}$  characteristic strength of a prestressing tendon.

Other symbols are defined in the text where they occur.



## Section 2. Design objectives and general recommendations

### 2.1 Basis of design

#### 2.1.1 Aim of design

The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and resistance to the effects of misuse and fire.

#### 2.1.2 Design method

The method recommended in this code is that of limit state design. Account should be taken of accepted theory, experiment and experience and the need to design for durability. Calculations alone do not produce safe, serviceable and durable structures. Suitable materials, quality control and good supervision are equally important.

#### 2.1.3 Durability, workmanship and materials

It is assumed that the quality of the concrete, steel and other materials and of the workmanship, as verified by inspections, is adequate for safety, serviceability and durability (see Section 6, Section 7 and Section 8).

#### 2.1.4 Design process

Design, including design for durability, construction and use in service should be considered as a whole. The realization of design objectives requires conformity to clearly defined criteria for materials, production, workmanship and also maintenance and use of the structure in service.

### 2.2 Structural design

NOTE See 1.3.1 for definitions of design ultimate load and design service load.

#### 2.2.1 General

Well-detailed and properly-erected structures designed by the limit state method will have acceptable probabilities that they will not reach a limit state, i.e. will not become unfit for their purpose by collapse, overturning, buckling (ultimate limit states), deformation, cracking, vibration, etc. (serviceability limit states) and that the structure will not deteriorate unduly under the action of the environment over the design life, i.e. will be durable. The usual approach is to design on the most critical limit state and then to check that the remaining limit states will not be reached.

#### 2.2.2 Ultimate limit state (ULS)

##### 2.2.2.1 Structural stability

The structure should be so designed that adequate means exist to transmit the design ultimate dead, wind and imposed loads safely from the highest supported level to the foundations. The layout of the structure and the interaction between the structural members should be such as to ensure a robust and stable design. The engineer responsible for the overall stability of the structure should ensure the compatibility of the design and details of parts and components, even where some or all of the design and details of those parts and components are not made by this engineer.

The design strengths of materials and the design loads should be those given in 2.4, as appropriate for the ULS. The design should satisfy the requirement that no ULS is reached by rupture of any section, by overturning or by buckling under the worst combination of ultimate loads. Account should be taken of elastic or plastic instability, or sway when appropriate.

##### 2.2.2.2 Robustness

Structures should be planned and designed so that they are not unreasonably susceptible to the effects of accidents. In particular, situations should be avoided where damage to small areas of a structure or failure of single elements may lead to collapse of major parts of the structure.

Unreasonable susceptibility to the effects of accidents may generally be prevented if the following precautions are taken.

- a) All buildings are capable of safely resisting the notional horizontal design ultimate load as given in **3.1.4.2** applied at each floor or roof level simultaneously.
- b) All buildings are provided with effective horizontal ties (see **3.12.3**):
  - 1) around the periphery;
  - 2) internally;
  - 3) to columns and walls.
- c) The layout of building is checked to identify any key elements the failure of which would cause the collapse of more than a limited portion close to the element in question. Where such elements are identified and the layout cannot be revised to avoid them, the design should take their importance into account. Recommendations for the design of key elements are given in **2.6** of BS 8110-2:1985.
- d) Buildings are detailed so that any vertical load-bearing element other than a key element can be removed without causing the collapse of more than a limited portion close to the element in question. This is generally achieved by the provision of vertical ties in accordance with **3.12.3** in addition to satisfying a), b) and c) above. There may, however, be cases where it is inappropriate or impossible to provide effective vertical ties in all or some of the vertical load-bearing elements. Where this occurs, each such element should be considered to be removed in turn and elements normally supported by the element in question designed to “bridge” the gap in accordance with the provisions of **2.6** of BS 8110-2:1985.
- e) A connection is provided between the horizontal ties and the vertical elements.

### **2.2.2.3 Special hazards**

The design for a particular occupancy, location or use, e.g. flour mills or chemical plant, may need to allow for the effects of particular hazards or for any unusually high probability of the structure's surviving an accident even though damaged. In such cases, partial safety factors greater than those given in **2.4** may be required.

## **2.2.3 Serviceability limit states (SLS)**

### **2.2.3.1 General**

The design properties of materials and the design loads should be those given in Section 3 of BS 8110-2:1985 as appropriate for SLS. Account should be taken of such effects as temperature, creep, shrinkage, sway, settlement and cyclic loading as appropriate.

### **2.2.3.2 Deflection due to vertical loading**

The deformation of the structure or any part of it should not adversely affect its efficiency or appearance. Deflections should be compatible with the degree of movement acceptable by other elements including finishes, services, partitions, glazing and cladding; in some cases a degree of minor repair work or fixing adjustment to such elements may be acceptable. Where specific attention is required to limit deflections to particular values, reference should be made to **3.2** of BS 8110-2:1985; otherwise it will generally be satisfactory to use the span/effective depth ratios given in Section 3 for reinforced concrete.

### **2.2.3.3 Response to wind loads**

The effect of lateral deflection should be considered, particularly for a tall, slender structure. However the accelerations associated with the deflection may be more critical than the deflection itself (see **3.2.2** of BS 8110-2:1985).

### **2.2.3.4 Cracking**

#### **2.2.3.4.1 Reinforced concrete**

Cracking should be kept within reasonable bounds by attention to detail. It will normally be controlled by adherence to the detailing rules given in **3.12.11**. Where specific attention is required to limit the design crack width to particular values, reference should be made to **3.2.4** of BS 8110-2:1985.

#### 2.2.3.4.2 Prestressed concrete

In the assessment of the likely behaviour of a prestressed concrete structure or element the amount of flexural tensile stress determines its class, as follows:

- class 1: no flexural tensile stresses;
- class 2: flexural tensile stresses but no visible cracking;
- class 3: flexural tensile stresses but surface width of cracks not exceeding 0.1 mm for members in exposure classes XS2, XS3, XD2, XD3, XF3 and XF4 (see Table A.1 of BS 8500-1) and not exceeding 0.2 mm for all other members.

#### 2.2.3.5 Vibration

Discomfort or alarm to occupants, structural damage, and interference with proper function should be avoided. Isolation of the source of vibration or of part or all of the structure may be needed. Flexible structural elements may require special consideration.

NOTE Acceptable vibration limits are described in specialist literature.

#### 2.2.4 Durability

To produce a durable structure requires the integration of all aspects of design, materials and construction. The environmental conditions to which the concrete will be exposed should be defined at the design stage. The design should take account of the shape and bulk of the structure, and the need to ensure that surfaces exposed to water are freely draining (see 3.1.5). Adequate cover to steel has to be provided for protection (see 3.3 and 4.1.5). Consideration may also be given to the use of protective coatings to either the steel or the concrete, or both, to enhance the durability of vulnerable parts of construction.

Concrete should be of the relevant quality; this depends on both its constituent materials and mix proportions. There is a need to avoid some constituent materials which may cause durability problems and, in other instances, to specify particular types of concrete to meet special durability requirements (see 3.1.5 and BS 8500-1).

Good workmanship, particularly curing, is essential and dimensional tolerances and the levels of control and inspection of construction should be specified. Use should be made of suitable quality assurance schemes where they exist (see 2.3, 6.1, 7.1 and 8.1 of this standard and BS 8500-1 and BS 8500-2).

NOTE For exceptionally severe environments additional precautions may be necessary and specialist literature should be consulted.

#### 2.2.5 Fatigue

When the imposed load on a structure is predominantly cyclical it may be necessary to consider the effects of fatigue.

#### 2.2.6 Fire resistance

A structure or structural element required to have fire resistance should be designed to possess an appropriate degree of resistance to flame penetration, heat transmission and collapse. Recommendations are given in Section 4 of BS 8110-2:1985.

#### 2.2.7 Lightning

Reinforcement may be used as part of a lightning protection system in accordance with BS 6651.

### 2.3 Inspection of construction

To ensure that the construction is in accordance with the design, an inspection procedure should be set up covering materials, records, workmanship and construction.

Tests should be made on reinforcement and the constituent materials of concrete in accordance with the relevant standards; the production and testing of concrete should conform to BS 8500-1 and BS 8500-2. Where applicable, use should be made of suitable quality assurance schemes.

Care should be taken to ensure that:

- a) design and detail are capable of being executed to a suitable standard, with due allowance for dimensional tolerances;
- b) there are clear instructions on inspection standards;
- c) there are clear instructions on permissible deviations;
- d) elements critical to workmanship, structural performance, durability and appearance are identified; and
- e) there is a system to verify that the quality is satisfactory in individual parts of the structure, especially the critical ones.

## 2.4 Loads and material properties

### 2.4.1 Loads

#### 2.4.1.1 *Characteristic values of loads*

The following loads should be used in design:

- a) characteristic dead load  $G_k$  i.e. the weight of the structure complete with finishes, fixtures and partitions;
- b) characteristic imposed load,  $Q_k$ ; and
- c) characteristic wind load,  $W_k$ .

The characteristic load in each case should be the appropriate load as defined in and calculated in accordance with BS 6399-1, BS 6399-2 and BS 6399-3.

#### 2.4.1.2 *Nominal earth loads $E_n$*

Nominal earth loads should be obtained in accordance with normal practice (see, for example, BS 8004).

#### 2.4.1.3 *Partial safety factors for load $\gamma_f$*

The design load for a given type of loading and limit state is obtained from:

$$G_k\gamma_f \text{ or } Q_k\gamma_f \text{ or } W_k\gamma_f \text{ or } E_n\gamma_f$$

where

- $\gamma_f$  is the appropriate partial safety factor. It is introduced to take account of unconsidered possible increases in load, inaccurate assessment of load effects, unforeseen stress redistribution, variation in dimensional accuracy and the importance of the limit state being considered. The value of  $\gamma_f$  chosen also ensures that the serviceability requirements can generally be met by simple rules.

#### 2.4.1.4 *Loads during construction*

The loading conditions during erection and construction should be considered in design and should be such that the structure's subsequent conformity to the limit state requirements is not impaired.

### 2.4.2 Material properties

#### 2.4.2.1 *Characteristic strengths of materials*

Unless otherwise stated in this code the term characteristic strength means that value of the cube strength of concrete  $f_{cu}$ , the yield or proof strength of reinforcement  $f_y$  or the ultimate strength of a prestressing tendon  $f_{pu}$  below which 5 % of all possible test results would be expected to fall.

When specifying concrete in accordance with BS 8500-1 and BS 8500-2 characteristic strength is expressed as a dual classification comprising the characteristic strength of cylinder test specimens followed by the characteristic strength of cube test specimens, e.g. C28/35 ~~Text deleted~~.

#### 2.4.2.2 *Partial safety factors for strength of materials $\gamma_m$*

For the analysis of sections, the design strength for a given material and limit state is derived from the characteristic strength divided by  $\gamma_m$ , where  $\gamma_m$  is the appropriate partial safety factor given in 2.4.4.1 and 2.4.6.2.  $\gamma_m$  takes account of differences between actual and laboratory values, local weaknesses and inaccuracies in assessment of the resistance of sections. It also takes account of the importance of the limit state being considered.

### 2.4.2.3 Stress-strain relationships

The short-term stress-strain relationships may be taken as follows:

- a) for normal-weight concrete, from Figure 2.1 with  $\gamma_m$  having the relevant value given in 2.4.4 or 2.4.6;
- b) for reinforcement, from Figure 2.2 with  $\gamma_m$  having the relevant value;
- c) for prestressing tendons, from Figure 2.3 with  $\gamma_m$  having the relevant value.

When sustained loading is being considered, for reinforcement the short-term stress-strain curves should be taken to apply; for prestressing tendons, appropriate allowance for relaxation should be made. For concrete, information on creep and shrinkage is given in Section 7 of BS 8110-2:1985.

### 2.4.2.4 Poisson's ratio for concrete

Where linear elastic analysis is appropriate, Poisson's ratio may be taken as 0.2.

## 2.4.3 Values of loads for ultimate limit state (ULS)

### 2.4.3.1 Design loads

#### 2.4.3.1.1 General

In ULS design of the whole or any part of a structure each of the combinations of loading given in Table 2.1 should be considered and the design of cross-sections based on the most severe stresses produced.

**Table 2.1 — Load combinations and values of  $\gamma_f$  for the ultimate limit state**

| Load combination                                            | Load type |            |         |            |                                                    | Wind |
|-------------------------------------------------------------|-----------|------------|---------|------------|----------------------------------------------------|------|
|                                                             | Dead      |            | Imposed |            | Earth <sup>a</sup> and water <sup>b</sup> pressure |      |
|                                                             | Adverse   | Beneficial | Adverse | Beneficial |                                                    |      |
| 1. Dead and imposed (and earth and water pressure)          | 1.4       | 1.0        | 1.6     | 0          | 1.2 <sup>c</sup><br>1.0 <sup>d</sup>               | —    |
| 2. Dead and wind (and earth and water pressure)             | 1.4       | 1.0        | —       | —          | 1.2 <sup>c</sup><br>1.0 <sup>d</sup>               | 1.4  |
| 3. Dead and imposed and wind (and earth and water pressure) | 1.2       | 1.2        | 1.2     | 1.2        | 1.2 <sup>c</sup><br>1.0 <sup>d</sup>               | 1.2  |

<sup>a</sup> The earth pressure is that obtained from BS 8002 including an appropriate mobilisation factor. The more onerous of the two factored conditions should be taken.

<sup>b</sup> The value of 1.2 may be used where the maximum credible level of the water can be clearly defined. If this is not feasible, a factor of 1.4 should be used.

<sup>c</sup> Unplanned excavation in accordance with BS 8002, **3.2.2.2** not included in the calculation.

<sup>d</sup> Unplanned excavation in accordance with BS 8002, **3.2.2.2** included in the calculation.

For load combinations 1 and 2 in Table 2.1, the “adverse” partial factor is applied to any loads that tend to produce a more critical design condition while the “beneficial” factor is applied to any loads that tend to produce a less critical design condition at the section considered. For load combinations 2 and 3, see 3.1.4.2 for minimum horizontal load.

#### 2.4.3.1.2 Partial factors for earth pressures

The overall dimensions and stability of earth retaining and foundation structures, e.g. the area of pad footings, should be determined by appropriate geotechnical procedures which are not considered in this code. However, in order to establish section sizes and reinforcement areas which will give adequate safety and serviceability without undue calculation, it is appropriate in normal design situations to apply values of  $\gamma_f$  comparable to those applied to other forms of loading.

The factor  $\gamma_f$  should be applied to all earth and water pressures unless they derive directly from loads that have already been factored, in which case the pressures should be derived from equilibrium with other design ultimate loads. When applying the factor, no distinction is made between adverse and beneficial loads.

Where a detailed investigation of the soil conditions has been undertaken and account has been taken of possible structure-soil interaction in the assessment of the earth pressure, it may be appropriate to derive design ultimate values for earth and water pressure by different procedures. In this case, additional consideration should be given to conditions in the structure under serviceability loads. This approach is also recommended for all design situations which involve uncommon features. Further guidance is given in Section 2 of BS 8110-2:1985.

#### 2.4.3.2 Effects of exceptional loads or localized damage

If in the design it is necessary to consider the probable effects of excessive loads caused by misuse or accident,  $\gamma_f$  should be taken as 1.05 on the defined loads, and only those loads likely to be acting simultaneously need be considered. Again, when considering the continued stability of a structure after it has sustained localized damage,  $\gamma_f$  should be taken as 1.05. The loads considered should be those likely to occur before temporary or permanent measures are taken to repair or offset the effect of the damage.

For these exceptional cases all the following should be taken into account:

- a) dead-load;
- b) one-third of the wind load;
- c) for buildings used predominantly for storage or industrial purposes or where the imposed loads are permanent, 100 % of the imposed load or, for other buildings, one-third of the imposed load.

#### 2.4.3.3 Creep, shrinkage and temperature effects

For the ULS, these effects will usually be minor and no specific calculations will be necessary.

### 2.4.4 Strengths of materials for the ultimate limit state

#### 2.4.4.1 Design strengths

In the assessment of the strength of a structure or any of its parts or cross-sections, appropriate  $\gamma_m$  values should be taken from Table 2.2.

**Table 2.2 — Values of  $\gamma_m$  for the ultimate limit state**

|                                             |            |
|---------------------------------------------|------------|
| Reinforcement (prestressing steel included) | 1.15       |
| Concrete in flexure or axial load           | 1.50       |
| Shear strength without shear reinforcement  | 1.25       |
| Bond strength                               | 1.4        |
| Others (e.g. bearing stress)                | $\geq 1.5$ |

A more detailed method for the assessment of  $\gamma_m$  is given in Section 2 of BS 8110-2:1985. In Section 3, Section 4 and Section 5 of this standard these values have been used in the preparation of the various tables associated with the ULS.

#### 2.4.4.2 Effects of exceptional loads or localized damage

In the consideration of these effects  $\gamma_m$  may be taken as 1.3 for concrete in flexure and 1.0 for steel.

### 2.4.5 Design loads for serviceability limit states

For SLS calculations the design loads should be those appropriate to the SLS under consideration as discussed in 3.3 of BS 8110-2:1985.

### 2.4.6 Material properties for serviceability limit states

#### 2.4.6.1 General

For SLS calculations, the material properties assumed (modules of elasticity, creep, shrinkage, etc.) should be taken as those appropriate to the SLS under consideration as discussed in 3.2 of BS 8110-2:1985.

#### 2.4.6.2 Tensile stress criteria for prestressed concrete

In assessing the cracking strength for a class 2 member,  $\gamma_m$  should be taken as 1.3 for concrete in tension due to flexure. Allowable design stresses are given in 4.3.4.3.

### 2.4.7 Material properties for durability

Some durability problems are associated with the characteristics of the constituent materials whilst others require particular characteristics of the concrete to overcome them. Guidance on these is given in the following sections and subclauses of this standard and BS 8500-1 and BS 8500-2:

- a) durability and constituent materials:
  - 1) chlorides and corrosion of steel (see **A4** A.4.2 of BS 8500-1:2006 **A4**);
  - 2) disruption due to excess sulfates (see **A4** A.7.4 of BS 8500-1:2006 **A4**);
  - 3) disruption due to alkali-silica reaction (see **A4** A.8.1 of BS 8500-1:2006 **A4**);
  - 4) aggregates with high drying shrinkage (see **A4** A.7.3 of BS 8500-1:2006 **A4**);
  - 5) aggregates and fire resistance (see Section 4 of BS 8110-2:1985 and **A4** A.7.1 of BS 8500-1:2006 **A4**);
- b) durability and concrete characteristics:
  - 1) concrete quality and cover to reinforcement (see 3.1.5, 3.3 and 4.12.3 of this standard and Annex A of BS 8500-1:2002);
  - 2) air-entrained concrete for freeze/thaw resistance (see **A4** A.4.3 and Table A.8 of BS 8500-1:2006 **A4**);
  - 3) concrete subject to exposure to aggressive chemicals (see **A4** A.4.4 of BS 8500-1:2006 **A4**);
  - 4) concrete properties and durability (see Annex A of BS 8500-1:2002);
  - 5) fire resistance (see Section 4 of BS 8110-2:1985 and **A4** A.7.1 of BS 8500-1:2006 **A4**);
  - 6) lightweight aggregate concrete (see Section 5 of BS 8110-2:1985).

## 2.5 Analysis

### 2.5.1 General

The analysis that is carried out to justify a design can be broken into two stages as follows:

- a) analysis of the structure;
- b) analysis of sections.

In the analysis of the structure, or part of the structure, to determine force distributions within the structure, the properties of materials may be assumed to be those associated with their characteristic strengths, irrespective of which limit state is being considered. In the analysis of any cross-section within the structure, the properties of materials should be assumed to be those associated with their design strengths appropriate to the limit state being considered.

The methods of analysis used should be based on as accurate a representation of the behaviour of the structure as is reasonably practicable. The methods and assumptions given in this clause are generally adequate but, in certain cases, more fundamental approaches in assessing the behaviour of the structure under load may be more appropriate.

### 2.5.2 Analysis of structure

The primary objective of structural analysis is to obtain a set of internal forces and moments throughout the structure that are in equilibrium with the design loads for the required loading combinations.

Under design ultimate loads, any implied redistribution of forces and moments should be compatible with the ductility of the members concerned. Generally it will be satisfactory to determine envelopes of forces and moments by linear elastic analysis of all or parts of the structure and allow for redistribution and possible buckling effects using the methods described in Section 3 and Section 4. Alternatively plastic methods, e.g. yield line analysis, may be used.

For design service loads, the analysis by linear elastic methods will normally give a satisfactory set of moments and forces.

When linear elastic analysis is used, the relative stiffnesses of members may be based on any of the following.

- a) *The concrete section*: the entire concrete cross-section, ignoring the reinforcement.
- b) *The gross section*: the entire concrete cross-section, including the reinforcement on the basis of modular ratio.
- c) *The transformed section*: the compression area of the concrete cross-section combined with the reinforcement on the basis of modular ratio.

In b) and c) a modular ratio of 15 may be assumed in the absence of better information.

A consistent approach should be used for all elements of the structure.

### 2.5.3 Analysis of sections for the ultimate limit state

The strength of a cross-section at the ULS under both short and long term loading may be assessed assuming the short term stress/strain curves derived from the design strengths of the materials as given in 2.4.4.1 and Figure 2.1, Figure 2.2 and Figure 2.3 as appropriate. In the case of prestressing tendons the moduli of elasticity in Figure 2.3 are those given for information in BS 4486 and BS 5896.

### 2.5.4 Analysis of sections for serviceability limit states

The behaviour of a section at a SLS may be assessed assuming plane sections remain plain and linear stress/strain relationships for both steel and concrete.

Allowance should be made where appropriate for the effects of creep, shrinkage, cracking and prestress losses.

The elastic modulus for steel should be taken as 200 kN/mm<sup>2</sup>. Information on the selection of elastic moduli for concrete may be found in Section 7 of BS 8110-2:1985.

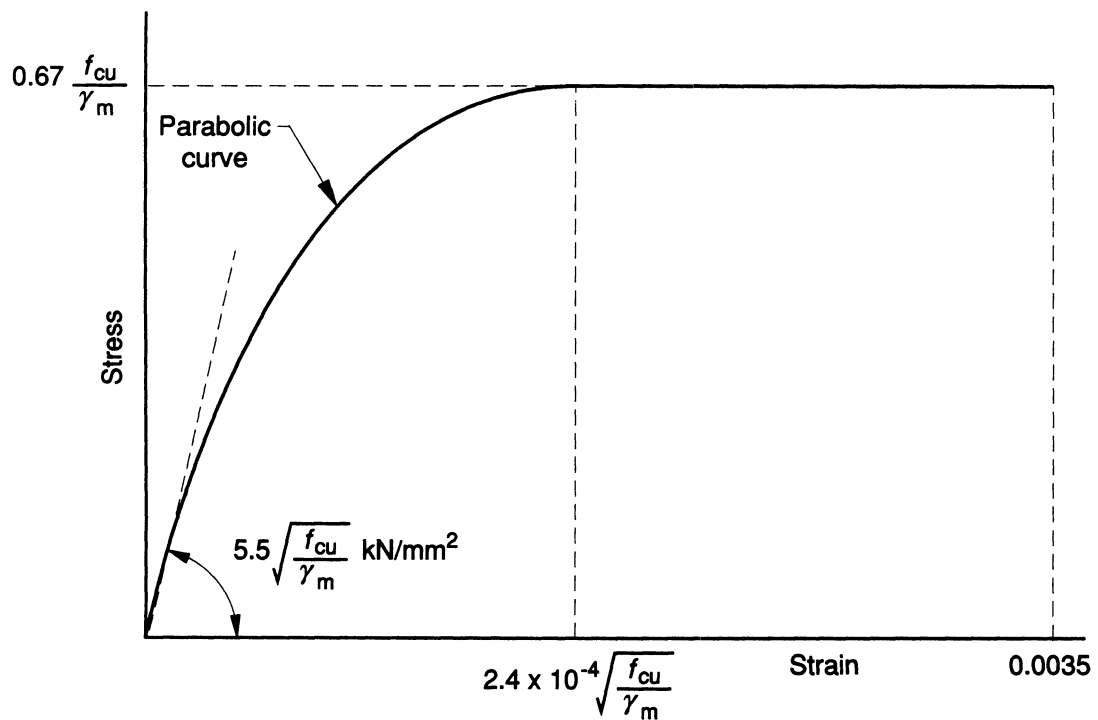
## 2.6 Design based on tests

### 2.6.1 Model tests

A design may be deemed satisfactory on the basis of results from an appropriate model test coupled with the use of model analysis to predict the behaviour of the actual structure, provided the work is carried out by engineers with relevant experience using suitable equipment.

### 2.6.2 Prototype tests

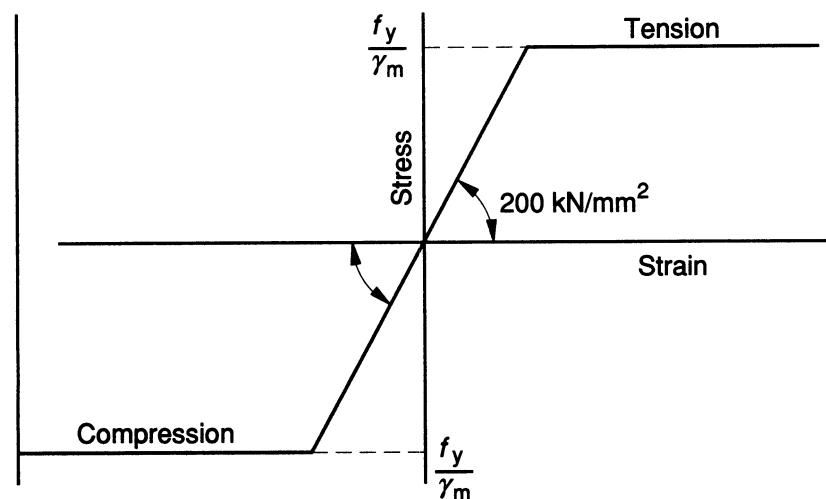
A design may be deemed satisfactory if the analytical or empirical basis of the design has been justified by development testing of prototype units and structures relevant to the particular design under consideration.



NOTE 1 0.67 takes account of the relation between the cube strength and the bending strength in a flexural member. It is simply a coefficient and not a partial safety factor.

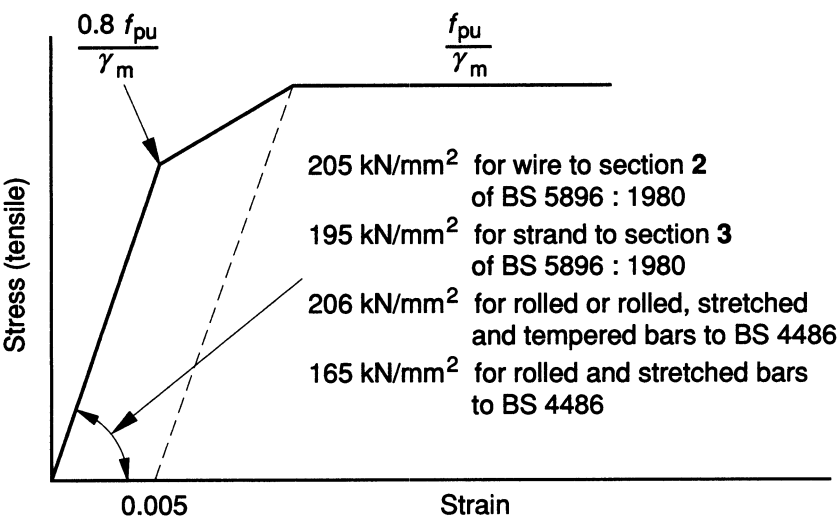
NOTE 2  $f_{cu}$  is in  $\text{N/mm}^2$ .

**Figure 2.1 — Short term design stress-strain curve for normal-weight concrete**



NOTE  $f_y$  is in  $\text{N/mm}^2$ .

**Figure 2.2 — Short term design stress-strain curve for reinforcement**



NOTE  $f_{pu}$  is in N/mm<sup>2</sup>.

Figure 2.3 — Short term design stress-strain curve for prestressing tendons

## Section 3. Design and detailing: reinforced concrete

NOTE Bridges, water-retaining structures, chimneys and some other structures are more appropriately covered by other codes. For deep beams and other uncommon elements, other relevant specialist literature may be used providing the resulting designs satisfy Section 2.

### 3.1 Design basis and strength of materials

#### 3.1.1 General

This section gives methods of analysis and design that will in general ensure that for reinforced concrete structures, the objectives set out in Section 2 are met. Other methods may be used provided they can be shown to be satisfactory for the type of structure or member considered. The design recommendations assume the use of normal-weight aggregate. Where lightweight aggregate is to be used, see Section 5 of BS 8110-2:1985. In certain cases, the assumptions made in this section may be inappropriate and the engineer should adopt a more suitable method having regard to the nature of the structure in question.

#### 3.1.2 Basis of design for reinforced concrete

Here the ULS is assumed to be the critical limit state; the SLS of deflection and cracking will not then normally be reached if the recommendations given for span/effective depth ratios and reinforcement spacings are followed.

#### 3.1.3 Alternative methods (serviceability limit state)

As an alternative to 3.1.2 (deflection and crack width may be calculated; suitable methods are given in Section 3 of BS 8110-2:1985).

#### 3.1.4 Robustness

##### 3.1.4.1 *General check of structural integrity*

A careful check should be made and appropriate action taken to ensure that there is no inherent weakness of structural layout and that adequate means exist to transmit the dead, imposed and wind loads safely from the highest supported level to the foundations.

##### 3.1.4.2 *Notional horizontal load*

All buildings should be capable of resisting a notional design ultimate horizontal load applied at each floor or roof level simultaneously equal to 1.5 % of the characteristic dead weight of the structure between mid-height of the storey below and either mid-height of the storey above or the roof surface [i.e. the design ultimate wind load should not be taken as less than this value when considering load combinations 2 or 3 (see 2.4.3.1)].

##### 3.1.4.3 *Provision of ties*

In structures where all load-bearing elements are concrete, horizontal and vertical ties should be provided in accordance with 3.12.3.

##### 3.1.4.4 *Key elements and bridging structures*

Where key elements and bridging structures are necessary, they should be designed in accordance with 2.6 of BS 8110-2:1985.

##### 3.1.4.5 *Safeguarding against vehicular impact*

Where vertical elements are particularly at risk from vehicle impact, consideration should be given to the provision of additional protection, such as bollards, earth banks or other devices.

##### 3.1.4.6 *Flow chart of design procedure*

Figure 3.1 summarizes the design procedure envisaged by the code for ensuring robustness.

### 3.1.5 Durability of structural concrete

#### 3.1.5.1 General

A durable concrete element is one that is designed and constructed to protect embedded metal from corrosion and to perform satisfactorily in the working environment for the life-time of the structure.

To achieve this it is necessary to consider many interrelated factors at various stages in the design and construction process. Thus the structural form and cover to steel are considered at the design stage and this involves consideration of the environmental conditions (see A.2 of BS 8500-1:2002). If these are particularly aggressive, it may be necessary to consider the type of cement at the design stage.

The main characteristics influencing the durability of concrete are the rates at which oxygen, carbon dioxide, chloride ions and other potentially deleterious substances can penetrate the concrete, and the concrete's ability to bind these substances. These characteristics are governed by the constituents and procedures used in making the concrete (see Annex A of BS 8500-1:2002 and 2.4.7 of this standard).

The factors influencing durability include:

- a) the design and detailing of the structure (see 3.1.5.2.1);
- b) the cover to embedded steel (see 3.3, and 4.12.3);
- c) the exposure conditions (see 3.3.4);
- d) the type of cement (see A.4 of BS 8500-1:2006 A.4);
- e) the type of aggregate (see A.7 of BS 8500-1:2006 A.7);
- f) the cement content and water/cement ratio of the concrete (see 3.3.5 of this standard and A.4 of BS 8500-1:2006 A.4);
- g) the type and dosage of admixture (see A.8 of BS 8500-1:2006 A.8);
- h) workmanship, to obtain a specified cover, full compaction and efficient curing (see 6.2);
- i) joints and connections (see 6.2.9 and 6.2.10).

The degree of exposure anticipated for the concrete during its service life together with other relevant factors relating to mix composition, workmanship and design should be considered. To provide adequate durability under these conditions, the concrete should be chosen and specified in accordance with BS 8500-1 and BS 8500-2.

#### 3.1.5.2 Design for durability

##### 3.1.5.2.1 Design and detailing of the structure

Since many processes of deterioration of concrete only occur in the presence of free water, the structure should be designed, wherever possible, to minimize uptake of water or exposure to moisture. The shape and design details of exposed structures should be such as to promote good drainage of water and to avoid standing pools and rundown of water.

Care should also be taken to minimize any cracks that may collect or transmit water.

Concrete is more vulnerable to deterioration due to chemical or climatic attack when it is in thin sections, in sections under hydrostatic pressure from one side only, in partly immersed sections and at corners and edges of elements. The life of the structure can be lengthened by providing extra cover to steel at the corners, by chamfering the corners or by using circular cross-sections or by using surface treatments which prevent or reduce the ingress of water, carbon dioxide or aggressive chemicals.

Good curing (see 6.2.3) is essential to avoid the harmful effects of early loss of moisture.

Where the minimum dimension of the concrete to be placed at a single time is greater than 600 mm, and especially where the cement content is 400 kg/m<sup>3</sup> or more, measures to reduce the temperature rise and/or peak temperature such as using material with a slower release of heat of hydration should be considered.

The amount and the rate of heat evolution is related to the cement content and the chemistry of the Portland cement, and the chemistry and amount of the ground granulated blastfurnace slag (g.g.b.f.s.) or pulverized fuel ash (p.f.a.) in blended cements, or combined in the concrete mixer. These factors may also affect the rate of strength development, ultimate strength and other properties.

#### 3.1.5.2.2 Depth of concrete cover and concrete quality

The protection of the steel in concrete against corrosion depends upon the alkaline environment provided by an adequate thickness of good quality concrete.

Table A.3, Table A.4 and Table A.9 of BS 8500-1:2006 give the limiting values of the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links, and to prestressing tendons depending on the condition of exposure described in A.2 of BS 8500-1:2002 and on the characteristics of the concrete.

#### 3.1.5.2.3 Other properties

Where it is anticipated that any aggregate is likely to have an unusual effect on the physical and mechanical properties of concrete, or its interaction with steel reinforcement, these factors should be taken into account in structural design and in the workmanship. For example, the elastic modulus depends mainly on the aggregate used (see Section 3 of BS 8110-2:1985).

#### 3.1.5.2.4 Unreinforced concrete

Table A.8, Table A.11 and Table A.12 of BS 8500-1:2006 give recommended values for the maximum free water cement/ratio, minimum cement content and minimum strength class of concrete to ensure long service life under appropriate conditions of exposure.

For concrete made with normal-weight aggregate and used in foundations and slabs for low rise structures in non-aggressive soil conditions (see aggressive chemical environment for concrete (ACEC) class AC-1 of Table A.2 of BS 8500-1:2006), a minimum strength class of C8/10 may be used provided the minimum cement content is not less than 180 kg/m<sup>3</sup> for other types of concrete.

Where a member is designed as unreinforced but contains reinforcing bars, the member may be treated as unreinforced for the purposes of this sub-clause provided that any damage to the cover concrete or unsightliness that may result from corrosion of the bars is acceptable.

### 3.1.6 Loads

The values of the design ultimate loads to be used in design are those given in 2.4.3 and the values of the design service loads in 2.4.5.

### 3.1.7 Strength of materials

#### 3.1.7.1 General

The design strengths of materials for ULS are expressed in the tables and the equations in terms of the characteristic strengths of the materials and partial safety factors.

#### 3.1.7.2 Selection of compressive strength class of concrete

The compressive strength class of concrete appropriate for use should be selected from the preferred compressive strength classes in Table 7 and Table 8 of BS EN 206-1:2000 and the Note to A.4.2 of BS 8500-1:2006 taking account of the following factors:

- adequate strength for the limit state requirements of Section 2;
- durability (see 3.1.5 and 3.3 of this standard, A.4, Table A.4 and Table A.8 of BS 8500-1:2006);
- any other special overriding characteristic.

For reinforced concrete, the minimum strength class that should be used is LC16/18 for concrete made with lightweight aggregates, and C20/25 for concrete made with normal-weight aggregates.

#### 3.1.7.3 Age allowance for concrete

Design should be based on the 28 day characteristic strength unless there is evidence to justify a higher strength for a particular structure.

#### 3.1.7.4 Characteristic strengths of reinforcement

Characteristic strengths of reinforcement are given in BS 4449, BS 4482 and BS 4483 and are as shown in Table 3.1. Design may be based on the appropriate characteristic strength or a lower value if necessary to reduce deflection or control cracking.

Table 3.1 — Strength of reinforcement

| Designation                                  | Specified characteristic strength, $f_y$ |
|----------------------------------------------|------------------------------------------|
|                                              | N/mm <sup>2</sup>                        |
| Hot rolled mild steel                        | 250                                      |
| High yield steel (hot rolled or cold worked) | 500                                      |

NOTE 1 Where Grade 460 deformed bars are used in design, a partial factor of 1.05 for such reinforcement may be used.  
 NOTE 2 This code envisages the use of reinforcement grades 500 A and 500 B. Where reinforcement grade 500 C is used, it may be deemed to satisfy the requirements within this code for grade 500 B.

## 3.2 Structures and structural frames

### 3.2.1 Analysis of structures

#### 3.2.1.1 Complete structures and complete structural frames

Analysis may follow the recommendations of 2.5 but the methods of 3.2.1.2 or 3.2.1.3 may be adopted if appropriate.

#### 3.2.1.2 Monolithic frames not providing lateral stability

##### 3.2.1.2.1 Simplification into sub-frames

The moments, loads and shear forces to be used in the design of individual columns and beams of a frame supporting vertical loads only may be derived from an elastic analysis of a series of sub-frames (but see 3.2.2 concerning redistribution of moments). Each sub-frame may be taken to consist of the beams at one level together with the columns above and below. The ends of the columns remote from the beams may generally be assumed to be fixed unless the assumption of a pinned end is clearly more reasonable (for example, where a foundation detail is considered unable to develop moment restraint).

##### 3.2.1.2.2 Choice of critical loading arrangements

It will normally be sufficient to consider the following arrangements of vertical load:

- all spans loaded with the maximum design ultimate load ( $1.4G_k + 1.6Q_k$ );
- alternate spans loaded with the maximum design ultimate load ( $1.4G_k + 1.6Q_k$ ) and all other spans loaded with the minimum design ultimate load ( $1.0G_k$ ).

##### 3.2.1.2.3 Alternative simplification for individual beams (and associated columns)

As an alternative to 3.2.1.2.1 the moments and forces in each individual beam may be found by considering a simplified sub-frame consisting only of that beam, the columns attached to the ends of the beam and the beams on either side, if any. The column and beam ends remote from the beam under consideration may generally be assumed to be fixed unless the assumption of pinned ends is clearly more reasonable. The stiffness of the beams on either side of the beam considered should be taken as half their actual values if they are taken to be fixed at their outer ends. The critical loading arrangements should be in accordance with 3.2.1.2.2.

The moments in an individual column may also be found from this simplified sub-frame provided that the sub-frame has as its central beam the longer of the two spans framing into the column under consideration.

##### 3.2.1.2.4 "Continuous beam" simplification

As a more conservative alternative to the preceding sub-frame arrangements the moments and shear forces in the beams at one level may also be obtained by considering the beams as a continuous beam over supports providing no restraint to rotation. The critical loading arrangements should be in accordance with 3.2.1.2.2.

##### 3.2.1.2.5 Asymmetrically-loaded columns where a beam has been analysed in accordance with 3.2.1.2.4

In these columns the ultimate moments may be calculated by simple moment distribution procedures, on the assumption that the column and beam ends remote from the junction under consideration are fixed and that the beams possess half their actual stiffness. The arrangement of the design ultimate imposed load should be such as to cause the maximum moment in the column.

### 3.2.1.3 *Frames providing lateral stability*

#### 3.2.1.3.1 *General*

Where the frame provides lateral stability to the structure as a whole, sway should be considered. In addition, if the columns are slender, additional moments (e.g. from eccentricity) may be imposed on beams at beam-column junctions (see 3.8.3). The load combinations 2 and 3 (see 2.4.3.1) should be considered in addition to load combination 1.

#### 3.2.1.3.2 *Sway-frame of three or more approximately equal bays*

The design of individual beams and columns may be based on either the moments, loads and shear obtained by considering vertical loads only, as in 3.2.1.2.2 or, if more severe, on the sum of those obtained from a) and b) as follows.

- a) An elastic analysis of a series of sub-frames each consisting of the beams at one level together with the columns above and below assumed to be fixed at their ends remote from those beams (or pinned if this is more realistic). Lateral loads should be ignored and all beams should be considered to be loaded with their full design load ( $1.2G_k + 1.2Q_k$ ).
- b) An elastic analysis of the complete frame, assuming points of contraflexure at the centres of all beams and columns, ignoring dead and imposed loads and considering only the design wind load ( $1.2W_k$ ) on the structure. If more realistic, instead of assuming points of contraflexure at the centres of ground floor columns the feet should be considered pinned.

It will also be necessary to consider the effects of load combination 2 (see 2.4.3.1) i.e.  $1.0G_k + 1.4W_k$ .

### 3.2.2 *Redistribution of moments*

#### 3.2.2.1 *General*

Redistribution of the moments obtained by means of a rigorous elastic analysis or by the simplified methods of 3.2.1.2 and 3.2.1.3 may be carried out provided the following conditions are satisfied.

- a) *Condition 1.* Equilibrium between internal and external forces is maintained under all appropriate combinations of design ultimate load.
- b) *Condition 2.* Where the design ultimate resistance moment of the cross-section subjected to the largest moment within each region of hogging or sagging is reduced, the neutral axis depth  $x$  should be checked to see that it is not greater than  $(\beta_b - 0.4)d$  where  $d$  is the effective depth and  $\beta_b$  is the ratio:

$$\frac{(\text{moment at the section after redistribution})}{(\text{moment at the section before redistribution})} \leq 1$$

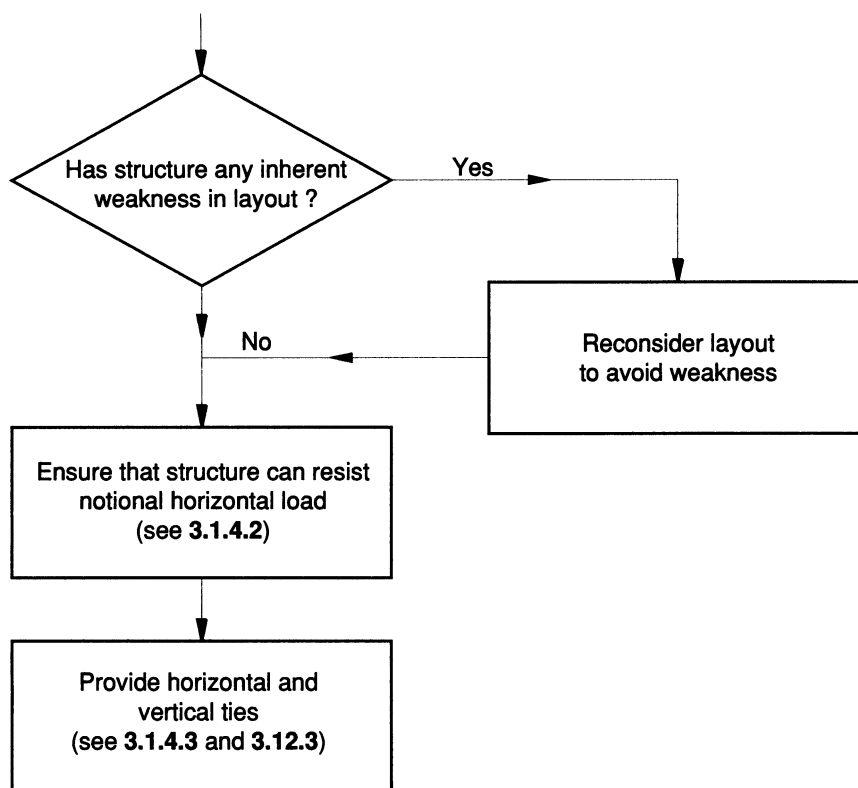
from the respective maximum moments diagram.

- c) *Condition 3.* Resistance moment at any section should be at least 70 % of moment at that section obtained from an elastic maximum moments diagram covering all appropriate combinations of design ultimate load (but see 3.2.2.2 for tall structures).

NOTE Unless the column axial load is small, condition 2 will generally rule out reduction in column moment.

#### 3.2.2.2 *Restriction to redistribution of moments in structures over four storeys where the structural frame provides lateral stability*

The provisions of 3.2.2.1 apply except that redistribution is limited to 10 % and the value given in condition 3 should read 90 %.



Where for any reason it is not feasible to introduce ties the following procedure should be adopted :

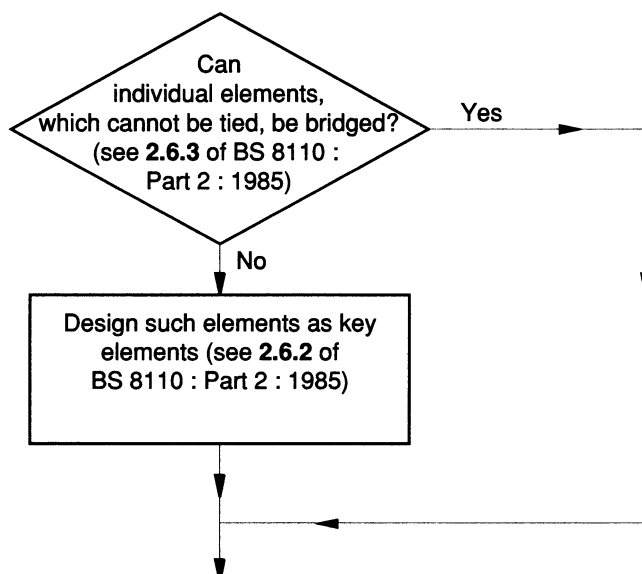


Figure 3.1 — Flow chart of design procedure

### 3.3 Concrete cover to reinforcement

#### 3.3.1 Nominal cover

##### 3.3.1.1 General

Nominal cover is the design depth of concrete cover to all steel reinforcement, including links. It is the dimension used in design and indicated on the drawings. The nominal cover should:

- a) be in accordance with the recommendations for bar size and aggregate size for concrete cast against uneven surfaces (see 3.3.1.2, 3.3.1.3 and 3.3.1.4);
- b) protect the steel against corrosion (see 3.3.3);
- c) protect the steel against fire (see 3.3.6); and
- d) allow for surface treatments such as bush hammering.

The sum of the minimum cover and design fixing tolerance should be calculated in accordance with BS 8110-1, BS 8110-2, BS 8500-1 and BS 8500-2 to obtain nominal cover values for design and specification using the specified construction fixing tolerances described in 7.3. The actual cover to all reinforcement should never be less than the nominal cover minus 5 mm.

NOTE The durability guidance given in BS 8500-1 is based on the assumption that the minimum cover for durability is achieved. A variable design fixing tolerance ( $\Delta c$ ) is added to the minimum cover to obtain the nominal cover. It is recommended that  $\Delta c$  is taken as 10 mm, unless the fabrication is subjected to a quality assurance system which includes the measurement of cover, where it is permitted to reduce  $\Delta c$  to 5 mm.

##### 3.3.1.2 Bar size

The nominal cover to all steel should be such that the resulting cover to a main bar should not be less than the size of the main bar or, where bars are in pairs or bundles, the size of a single bar of cross-sectional area equal to the sum of their cross-sectional areas. At the same time the nominal cover to any links should be preserved.

##### 3.3.1.3 Nominal maximum size of aggregate

Nominal covers should be not less than the nominal maximum size of the aggregate. The nominal maximum size of coarse aggregate should not normally be greater than one-quarter of the minimum thickness of the concrete section or element.

For most work, 20 mm aggregate is suitable. Larger sizes should be permitted where there are no restrictions to the flow of concrete into sections. In thin sections or elements with closely spaced reinforcement, consideration should be given to the use of 14 mm or 10 mm nominal maximum size.

##### 3.3.1.4 Concrete cast against uneven surfaces

In such cases the specified nominal cover should generally be increased beyond the values given in [Table A.4](#) of BS 8500-1:2006 [A4](#) to ensure that an adequate minimum cover will be obtained. For this reason, the nominal cover specified where concrete is cast directly against the earth should generally be not less than 75 mm. Where concrete is cast against an adequate blinding, a nominal cover of less than [50 mm](#) [A4](#) (excluding blinding) should not generally be specified.

#### 3.3.2 Ends of straight bars

Cover is not required to the end of a straight bar in a floor or roof unit where its end is not exposed to the weather or to condensation.

#### 3.3.3 Cover against corrosion

The cover required to protect the reinforcement against corrosion depends on the exposure conditions and the quality of the concrete as placed and cured immediately surrounding the reinforcement. [Table A.3](#), [Table A.4](#) and [Table A.9](#) of BS 8500-1:2006 [A4](#) give limiting values for the nominal cover of concrete made with normal-weight aggregates as a function of these factors. There may be cases where extra precautions are needed beyond those given in 3.3.4 in order to ensure protection of the reinforcement. Further information is given in 3.1.5.

#### 3.3.4 Exposure conditions

The exposure conditions in service are defined in [A.2](#) of BS 8500-1:2002.

### 3.3.5 Method of specifying concrete for durability

Concrete to meet the requirements of **Table A.4**, **Table A.8** and **Table A.9** of BS 8500-1:2006 **Table A.4** should be specified as designated concrete, designed concrete or standardized prescribed concrete in accordance with BS 8500-1 and BS 8500-2.

**3.3.5.1** — *deleted*

**3.3.5.2** — *deleted*

**3.3.5.3** — *deleted*

**Table 3.2** — *deleted*

### 3.3.6 Cover as fire protection

Cover for protection against corrosion may not suffice as fire protection. The values given in **Table 3.4** and **Figure 3.2** will ensure that fire resistance requirements are satisfied. The tables are based on recommendations given in Section 4 of BS 8110-2:1985; however, in columns and beams the covers included in the tables have been adjusted to permit nominal covers to be specified to all steel (including links). Minimum dimensions of members for fire resistance are also included in **Figure 3.2**. In some circumstances a more detailed treatment of the design for fire may give significant economies. Section 4 of BS 8110-2:1985 gives further information on design for fire, including information on surface treatments for improving fire resistance.

### 3.3.7 Control of cover

Good workmanship is required to ensure that the reinforcement is properly placed and that the specified cover is obtained. Recommendations for this are given in **7.3**.

NOTE Further information on cover is given in the following:

- a) durability in general (see **3.1.5** of this standard and Annex A of BS 8500-1:2002);
- b) prestressed concrete (see **4.12.3**);
- c) control of cover (see **7.3**);
- d) fire resistance (see Section 4 of BS 8110-2:1985);
- e) lightweight aggregate concrete (see Section 5 of BS 8110-2:1985).

**Table 3.3** — *deleted*

**Table 3.4 — Nominal cover to all reinforcement (including links) to meet specified periods of fire resistance (see NOTE 1 and NOTE 2)**

| Fire resistance<br><br><i>h</i> | Nominal cover<br>mm |                 |                  |                 |                  |                 |                      |
|---------------------------------|---------------------|-----------------|------------------|-----------------|------------------|-----------------|----------------------|
|                                 | Beams <sup>a</sup>  |                 | Floors           |                 | Ribs             |                 | Columns <sup>a</sup> |
|                                 | Simply supported    | Continuous      | Simply supported | Continuous      | Simply supported | Continuous      |                      |
| 0.5                             | 20 <sup>b</sup>     | 20 <sup>b</sup> | 20 <sup>b</sup>  | 20 <sup>b</sup> | 20 <sup>b</sup>  | 20 <sup>b</sup> | 20 <sup>b</sup>      |
| 1                               | 20 <sup>b</sup>     | 20 <sup>b</sup> | 20               | 20              | 20               | 20 <sup>b</sup> | 20 <sup>b</sup>      |
| 1.5                             | 20                  | 20 <sup>b</sup> | 25               | 20              | 35               | 20              | 20                   |
| 2                               | 40                  | 30              | 35               | 25              | 45               | 35              | 25                   |
| 3                               | 60                  | 40              | 45               | 35              | 55               | 45              | 25                   |
| 4                               | 70                  | 50              | 55               | 45              | 65               | 55              | 25                   |

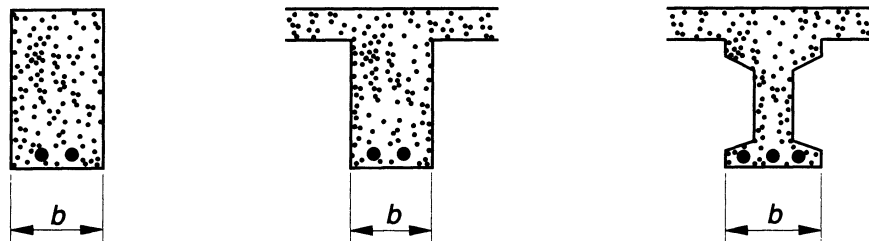
NOTE 1 The nominal covers given relate specifically to the minimum member dimensions given in **Figure 3.2**. Guidance on increased covers necessary if smaller members are used is given in Section 4 of BS 8110-2:1985.

NOTE 2 Cases that lie below the bold line require attention to the additional measures necessary to reduce the risks of spalling (see Section 4 of BS 8110-2:1985).

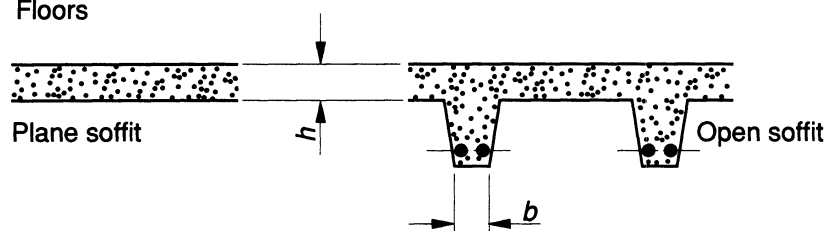
<sup>a</sup> For the purposes of assessing a nominal cover for beams and columns, the cover to main bars which would have been obtained from **Table 4.2** and **Table 4.3** of BS 8110-2:1985 has been reduced by a notional allowance for stirrups of 10 mm to cover the range 8 mm to 12 mm (see also **3.3.6**).

<sup>b</sup> These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm (see **3.3.1.3**).

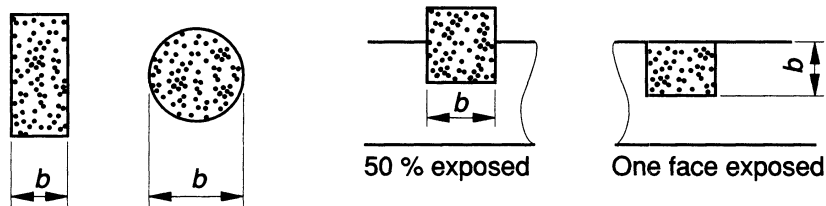
## Beams



## Floors



## Columns



## Fully exposed

| Fire resistance<br><br>$h$ | Minimum beam width ( $b$ )<br><br>mm | Rib width ( $b$ )<br><br>mm | Minimum thickness of floors ( $h$ )<br><br>mm | Column width ( $b$ ) |              |                  | Minimum wall thickness |                     |            |
|----------------------------|--------------------------------------|-----------------------------|-----------------------------------------------|----------------------|--------------|------------------|------------------------|---------------------|------------|
|                            |                                      |                             |                                               | Fully exposed        | 50 % exposed | One face exposed | $p < 0.4 \%$           | $0.4 \% < p < 1 \%$ | $p > 1 \%$ |
|                            |                                      |                             |                                               | mm                   | mm           | mm               | mm                     | mm                  | mm         |
| 0.5                        | 200                                  | 125                         | 75                                            | 150                  | 125          | 100              | 150                    | 100                 | 75         |
| 1                          | 200                                  | 125                         | 95                                            | 200                  | 160          | 120              | 150                    | 120                 | 75         |
| 1.5                        | 200                                  | 125                         | 110                                           | 250                  | 200          | 140              | 175                    | 140                 | 100        |
| 2                          | 200                                  | 125                         | 125                                           | 300                  | 200          | 160              | —                      | 160                 | 100        |
| 3                          | 240                                  | 150                         | 150                                           | 400                  | 300          | 200              | —                      | 200                 | 150        |
| 4                          | 280                                  | 175                         | 170                                           | 450                  | 350          | 240              | —                      | 240                 | 180        |

NOTE 1 These minimum dimensions relate specifically to the covers given in Table 3.4 and Table 4.9.

NOTE 2  $p$  is the area of steel relative to that of concrete.

Figure 3.2 — Minimum dimensions of reinforced concrete members for fire resistance

## 3.4 Beams

### 3.4.1 General

#### 3.4.1.1 *Design limitations*

This sub-clause deals with the design of beams of normal proportions. Deep beams of clear span less than twice the effective depth are not considered.

NOTE For the design of deep beams, reference should be made to specialist literature.

#### 3.4.1.2 *Effective span of simply-supported beams*

The effective span of a simply-supported beam may be taken as the smaller of the distance between the centres of bearings, or the clear distance between supports plus the effective depth.

#### 3.4.1.3 *Effective span of a continuous member*

The effective span of a continuous member should be taken as the distance between centres of supports. The centre of action of support at an encastred end should be taken to be at half the effective depth from the face of the support.

#### 3.4.1.4 *Effective length of a cantilever*

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

#### 3.4.1.5 *Effective width of flanged beam*

In the absence of any more accurate determination this should be taken as:

- a) for *T-beams*: web width  $+l_z/5$  or actual flange width if less;
- b) for *L-beams*: web width  $+l_z/10$  or actual flange width if less;

where

- $l_z$  is the distance between points of zero moment (which, for a continuous beam, may be taken as 0.7 times the effective span).

#### 3.4.1.6 *Slenderness limits for beams for lateral stability*

The clear distance between restraints should not exceed:

- a) for *simply-supported or continuous beams*:  $60b_c$  or  $250b_c^2/d$  if less;
- b) for *cantilevers with lateral restraint only at support*:  $25b_c$  or  $100b_c^2/d$  if less;

where

- $b_c$  is the breadth of the compression face of the beam, measured mid-way between restraints (or the breadth of the compression face of a cantilever);
- $d$  is the effective depth (which need not be greater than whatever effective depth would be necessary to withstand the design ultimate load with no compression reinforcement).

### 3.4.2 Continuous beams

Continuous beams may be analysed in accordance with Section 2 or designed and detailed to resist the moments and shear forces given by 3.2.1.2 or 3.4.3, as appropriate.

### 3.4.3 Uniformly-loaded continuous beams with approximately equal spans: moments and shears

Table 3.5 may be used to calculate the design ultimate bending moments and shear forces, subject to the following provisions:

- a) characteristic imposed load  $Q_k$  may not exceed characteristic dead load  $G_k$ ;
- b) loads should be substantially uniformly distributed over three or more spans;
- c) variations in span length should not exceed 15 % of longest.

**Table 3.5 — Design ultimate bending moments and shear forces**

|                                                                                                                                                                                | At outer support | Near middle of end span | At first interior support | At middle of interior spans | At interior supports |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------|-------------------------|---------------------------|-----------------------------|----------------------|
| Moment                                                                                                                                                                         | 0                | $0.09Fl$                | $-0.11Fl$                 | $0.07Fl$                    | $-0.08Fl$            |
| Shear                                                                                                                                                                          | $0.45F$          | —                       | $0.6F$                    | —                           | $0.55F$              |
| NOTE $l$ is the effective span;<br>$F$ is the total design ultimate load ( $1.4G_k + 1.6Q_k$ ).<br>No redistribution of the moments calculated from this table should be made. |                  |                         |                           |                             |                      |

### 3.4.4 Design resistance moment of beams

#### 3.4.4.1 Analysis of sections

In the analysis of a cross-section to determine its ultimate moment of resistance the following assumptions should be made.

- a) The strain distribution in the concrete in compression and the strains in the reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane.
- b) The stresses in the concrete in compression may be derived from the stress-strain curve in Figure 2.1 with  $\gamma_m = 1.5$ . Alternatively, the simplified stress block illustrated in Figure 3.3 may be used.
- c) The tensile strength of the concrete is ignored.
- d) The stresses in the reinforcement are derived from the stress-strain curve in Figure 2.2 with  $\gamma_m = 1.15$ .
- e) Where a section is designed to resist only flexure, the lever arm should not be assumed to be greater than 0.95 times the effective depth.

In the analysis of a cross-section of a beam that has to resist a small axial thrust, the effect of the design ultimate axial force may be ignored if it does not exceed  $0.1f_{cu}$  times the cross-sectional area.

#### 3.4.4.2 Design charts

The design charts which form BS 8110-3 include charts based on Figure 2.1, Figure 2.2 and the assumptions<sup>2)</sup> of 3.4.4.1, which may be used for the design of beams reinforced in tension only or in tension and compression.

<sup>2)</sup> BS 8110-3 uses reinforcement Grade 460 which is conservative.

### 3.4.4.3 Symbols

For the purposes of 3.4.4 the following symbols apply.

- $A_s$  area of tension reinforcement.  
 $A_s'$  area of compression reinforcement.  
 $b$  width or effective width of the section or flange in the compression zone.  
 $b_w$  average web width of a flanged beam.  
 $d$  effective depth of the tension reinforcement.  
 $d'$  depth to the compression reinforcement.  
 $h_f$  thickness of the flange.  
 $M$  design ultimate moment.  
 $x$  depth to the neutral axis.  
 $z$  lever arm.  
 $\beta_b$  the ratio:  

$$\frac{(\text{moment at the section after redistribution})}{(\text{moment at the section before redistribution})}$$
 from the respective maximum moments diagram.

### 3.4.4.4 Design formulae for rectangular beams

The following equations, which are based on the simplified stress block of Figure 3.3, are also applicable to flanged beams where the neutral axis lies within the flange:

$K' = 0.156$  where redistribution does not exceed 10 % (this implies a limitation of the neutral axis depth to  $d/2$ ); or

$K' = 0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2$  where redistribution exceeds 10 %;

and  $K = M/bd^2f_{cu}$ .

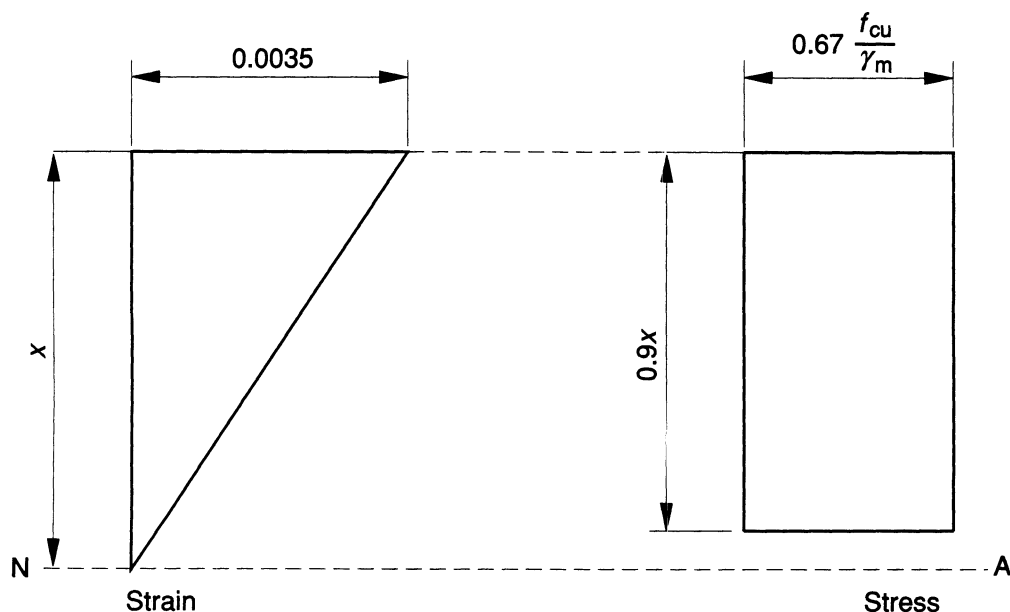


Figure 3.3 — Simplified stress block for concrete at ultimate limit state

Table 3.6 — Values of the factor  $\beta_f$ 

| $b/b_w$  | $d/h_f$  |      |      |      |      |          |
|----------|----------|------|------|------|------|----------|
|          | $\leq 2$ | 3    | 4    | 5    | 6    | $\infty$ |
| 1        | 0.15     | 0.15 | 0.15 | 0.15 | 0.15 | 0.15     |
| 2        | 0.15     | 0.14 | 0.12 | 0.12 | 0.11 | 0.08     |
| 4        | 0.15     | 0.13 | 0.11 | 0.10 | 0.09 | 0.04     |
| 6        | 0.15     | 0.13 | 0.11 | 0.09 | 0.08 | 0.03     |
| 8        | 0.15     | 0.13 | 0.10 | 0.09 | 0.08 | 0.02     |
| $\infty$ | 0.15     | 0.13 | 0.10 | 0.08 | 0.07 | 0        |

If  $K \leq K'$ , compression reinforcement is not required and:

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\}$$

but not greater than  $0.95d$ .

$$x = (d - z)/0.45$$

$$A_s = M/0.87f_y z$$

If  $K > K'$ , compression reinforcement is required and:

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K'}{0.9}} \right\}$$

$$x = (d - z)/0.45$$

$$A_{s'} = (K - K')f_{cu}bd^2/0.87f_y(d - d')$$

$$A_s = (K'f_{cu}bd^2/0.87f_y z + A_{s'})$$

If  $d'/x$  exceeds 0.37 (for  $f_y = 500 \text{ N/mm}^2$ ), the compression stress will be less than  $0.87f_y$  and should be obtained from Figure 2.2.

#### 3.4.4.5 Design formulae for flanged beams where the neutral axis falls below the flange

Provided that the design ultimate moment is less than  $\beta_f f_{cu} b d^2$  and that not more than 10 % of redistribution has been carried out, the required area of tension steel may be calculated using the following equation:

$$A_s = \frac{M + 0.1f_{cu}b_w d(0.45d - h_f)}{0.87f_y(d - 0.5h_f)} \quad \text{equation 1}$$

If the design ultimate moment exceeds  $\beta_f f_{cu} b d^2$  or more than 10 % redistribution has been carried out, the section may be designed by direct application of the assumptions given in 3.4.4.1.  $\beta_f$  in this expression is a factor given in Table 3.6.

Equation 1 is only applicable when  $h_f < 0.45d$ .

The values in Table 3.6 are calculated from the following equation:

$$\beta_f = 0.45 \frac{h_f}{d} \left( 1 - \frac{b_w}{b} \right) \left( 1 - \frac{h_f}{2d} \right) + 0.15 \frac{b_w}{b} \quad \text{equation 2}$$

### 3.4.5 Design shear resistance of beams

#### 3.4.5.1 Symbols

For the purposes of 3.4.5 the following symbols apply.

- $a_v$  length of that part of a member traversed by a shear failure plane.
- $A_c$  area of concrete section.
- $A_{sv}$  total cross-section of links at the neutral axis, at a section.
- $A_{sb}$  cross-sectional area of bent-up bars.
- $b_v$  breadth of section (for a flanged beam this should be taken as the average width of the rib below the flange.)
- $d$  effective depth.
- $f_{yv}$  characteristic strength of links (not to be taken as more than 500 N/mm<sup>2</sup>).
- $M$  design ultimate moment at the section considered.
- $N$  design axial force.
- $s_v$  spacing of links along the member.
- $s_b$  spacing of bent-up bars.
- $V$  design shear force due to ultimate loads.
- $V_b$  design shear resistance of bent-up bars.
- $v$  design shear stress at a cross-section.
- $v_c$  design concrete shear stress (see Table 3.8).
- $v_c'$  design concrete shear stress corrected to allow for axial forces.
- $\alpha$  angle between a bent-up bar and the axis of a beam.
- $\beta$  angle between the "compression strut" of a system of bent-up bars and the axis of the beam.

#### 3.4.5.2 Shear stress in beams

The design shear stress  $v$  at any cross-section should be calculated from:

$$v = \frac{V}{b_v d} \quad \text{equation 3}$$

In no case should  $v$  exceed:

$0.8\sqrt{f_{cu}}$  or 5 N/mm<sup>2</sup>, whichever is the lesser, whatever shear reinforcement is provided. (This limit includes an allowance for  $\gamma_m$  of 1.25).

#### 3.4.5.3 Shear reinforcement: form, area and stress

- | Shear reinforcement should be as given in Table 3.7. Stress in any bar should not exceed  $0.87f_{yv}$ .

#### 3.4.5.4 Concrete shear stresses

Values for the design concrete shear stress  $v_c$  (in N/mm<sup>2</sup>) are given in Table 3.8.

The term  $A_s$  is that area of longitudinal tension reinforcement which continues for a distance at least equal to  $d$  beyond the section being considered. At supports, the full area of tension reinforcement at the section may be applied in the table provided the requirements for curtailment and anchorage of reinforcement are met (see 3.12.9 where general recommendations and simplified rules are given).

At a monolithic beam-column junction where the beam has been designed on the assumption that the column provides a simple support but where some nominal top steel has been provided to control cracking,  $v_c$  may be calculated on the basis of the area of the bottom steel at the support provided that this has been anchored in accordance with the rules for detailing simply-supported ends given in 3.12.9.4 and 3.12.10.3.2. If this anchorage has not been provided then  $v_c$  should be calculated on the basis of the top steel. This steel should extend into the span for a distance of at least three times the effective depth from the face of the support.

**3.4.5.5 Spacing of links** (see Table 3.7)

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

**3.4.5.6 Shear resistance of bent-up bars**

The design shear resistance of a system of bent-up bars may be calculated by assuming that the bent-up bars form the tension members of one or more single systems of trusses in which the concrete forms the compression members (see Figure 3.4). The resistance of a system of bent-up bars is given by the following equation:

$$V_b = A_{sb}(0.87f_{yv})(\cos \alpha + \sin \alpha \cot \beta) \frac{d - d'}{s_b} \quad \text{equation 4}$$

**Table 3.7 — Form and area of shear reinforcement in beams**

| Value of $v$<br>N/mm <sup>2</sup>                                                                                                                                                                                                                      | Form of shear reinforcement to be provided                                                                                                                  | Area of shear reinforcement to be provided                                                                                              |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------|
| Less than $0.5v_c$ throughout the beam                                                                                                                                                                                                                 | See NOTE 1                                                                                                                                                  | —                                                                                                                                       |
| $0.5v_c < v < (v_c + 0.4)$                                                                                                                                                                                                                             | Minimum links for whole length of beam                                                                                                                      | $A_{gv} \geq 0.4b_v s_v / 0.87f_{yv}$<br>(see NOTE 2)                                                                                   |
| $(v_c + 0.4) < v < 0.8 \sqrt{f_{cu}}$ or 5 N/mm <sup>2</sup>                                                                                                                                                                                           | Links or links combined with bent-up bars. Not more than 50 % of the shear resistance provided by the steel may be in the form of bent-up bars (see NOTE 3) | Where links only provided:<br>$A_{gv} \geq b_v s_v (v - v_c) / 0.87f_{yv}$<br>Where links and bent-up bars provided: see <b>3.4.5.6</b> |
| NOTE 1 While minimum links should be provided in all beams of structural importance, it will be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half $v_c$ . |                                                                                                                                                             |                                                                                                                                         |
| NOTE 2 Minimum links provide a design shear resistance of 0.4 N/mm <sup>2</sup> .                                                                                                                                                                      |                                                                                                                                                             |                                                                                                                                         |
| NOTE 3 See <b>3.4.5.5</b> for guidance on spacing of links and bent-up bars.                                                                                                                                                                           |                                                                                                                                                             |                                                                                                                                         |

Table 3.8 — Values of  $v_c$  design concrete shear stress

| $\frac{100A_s}{b_v d}$ | Effective depth<br>mm |                   |                   |                   |                   |                   |                   |                   |
|------------------------|-----------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|
|                        | 125                   | 150               | 175               | 200               | 225               | 250               | 300               | 400               |
|                        | N/mm <sup>2</sup>     | N/mm <sup>2</sup> | N/mm <sup>2</sup> | N/mm <sup>2</sup> | N/mm <sup>2</sup> | N/mm <sup>2</sup> | N/mm <sup>2</sup> | N/mm <sup>2</sup> |
| ≤ 0.15                 | 0.45                  | 0.43              | 0.41              | 0.40              | 0.39              | 0.38              | 0.36              | 0.34              |
| 0.25                   | 0.53                  | 0.51              | 0.49              | 0.47              | 0.46              | 0.45              | 0.43              | 0.40              |
| 0.50                   | 0.67                  | 0.64              | 0.62              | 0.60              | 0.58              | 0.56              | 0.54              | 0.50              |
| 0.75                   | 0.77                  | 0.73              | 0.71              | 0.68              | 0.66              | 0.65              | 0.62              | 0.57              |
| 1.00                   | 0.84                  | 0.81              | 0.78              | 0.75              | 0.73              | 0.71              | 0.68              | 0.63              |
| 1.50                   | 0.97                  | 0.92              | 0.89              | 0.86              | 0.83              | 0.81              | 0.78              | 0.72              |
| 2.00                   | 1.06                  | 1.02              | 0.98              | 0.95              | 0.92              | 0.89              | 0.86              | 0.80              |
| ≥ 3.00                 | 1.22                  | 1.16              | 1.12              | 1.08              | 1.05              | 1.02              | 0.98              | 0.91              |

NOTE 1 Allowance has been made in these figures for a  $\gamma_m$  of 1.25.

NOTE 2 The values in the table are derived from the expression:

$$0.79\{100A_s/(b_v d)\}^{1/3} (400/d)^{1/4} / \gamma_m$$

where

$$\frac{100A_s}{b_v d} \text{ should not be taken as greater than } 3;$$

$$\left(\frac{400}{d}\right)^{1/4} \text{ should not be taken as less than } 0.67 \text{ for members without shear reinforcement;}$$

$$\left(\frac{400}{d}\right)^{1/4} \text{ should not be taken as less than } 1 \text{ for members with shear reinforcement providing a design shear resistance of } \geq 0.4 \text{ N/mm}^2.$$

For characteristic concrete strengths greater than 25 N/mm<sup>2</sup>, the values in this table may be multiplied by  $(f_{cu}/25)^{1/3}$ . The value of  $f_{cu}$  should not be taken as greater than 40.

The truss should be arranged so that  $\alpha$  and  $\beta$  are both greater than or equal to 45° giving a maximum value of  $s_t$  of  $1.5d$ . At least 50 % of the shear resistance provided by the steel should be in the form of links.

### 3.4.5.7 Anchorage and bearing of bent-up bars

Bars should be checked for anchorage (see 3.12.8.2 and 3.12.8.3) and bearing (see 3.12.8.25).

### 3.4.5.8 Enhanced shear strength of sections close to supports

Shear failure at sections of beams and cantilevers without shear reinforcement will normally occur on a plane inclined at an angle of about 30° to the horizontal. If the angle of failure plane is forced to be inclined more steeply than this (because the section considered (X – X) in Figure 3.5 is close to a support or for other reasons) the shear force required to produce failure is increased.

The enhancement of shear strength may be taken into account in the design of sections near a support by increasing the design concrete shear stress  $v_c$  to  $2dv_c/a_v$  provided that  $v$  at the face of the support remains less than the lesser of  $0.8\sqrt{f_{cu}}$  or 5 N/mm<sup>2</sup> (this limit includes a  $\gamma_m$  of 1.25).

Account may be taken of the enhancement in any situation where the section considered is closer to the face of a support or concentrated load than twice the effective depth,  $d$ . This enhancement is particularly useful for corbels (see 5.2.7) or pile-caps (see 3.11.4.4) or where concentrated loads are applied close to the support of a beam.

To be effective, tension reinforcement should extend on each side of the point where it is intersected by a possible failure plane for a distance at least equal to the effective depth, or be provided with an equivalent anchorage.

### 3.4.5.9 Shear reinforcement for sections close to supports

If shear reinforcement is required, the total area of this is given by:

$$\Sigma A_{sv} = a_v b_v (v - 2dv_c/a_v)/0.87f_{yv} \geq 0.4b_v a_v/0.87f_{yv}$$

equation 5 |

This reinforcement should be provided within the middle three-quarters of  $a_v$ . Where  $a_v$  is less than  $d$  horizontal shear reinforcement will be more effective than vertical.

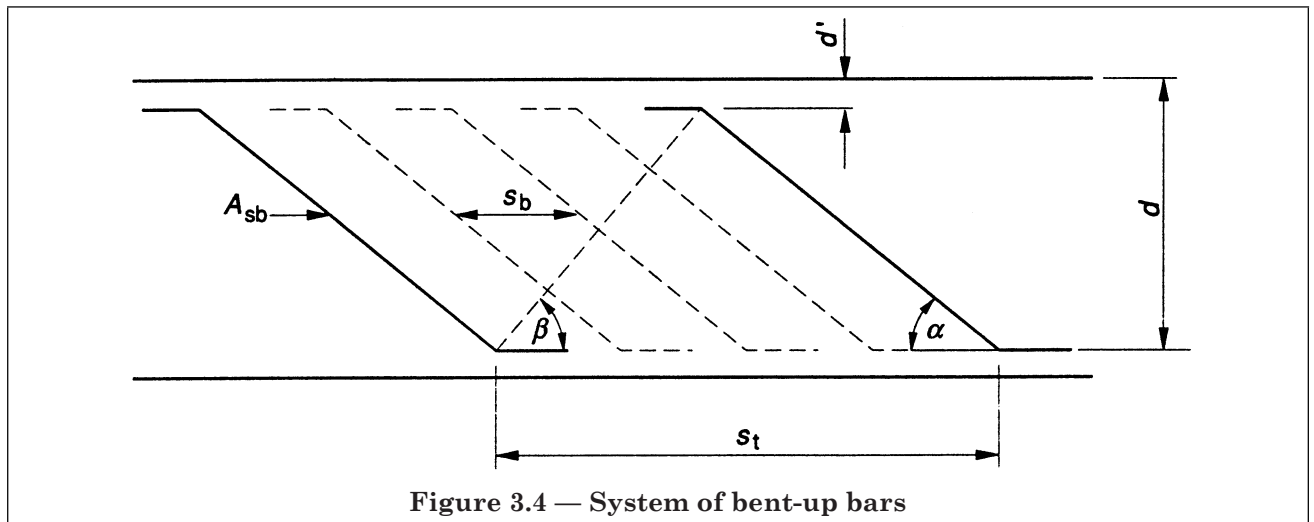


Figure 3.4 — System of bent-up bars

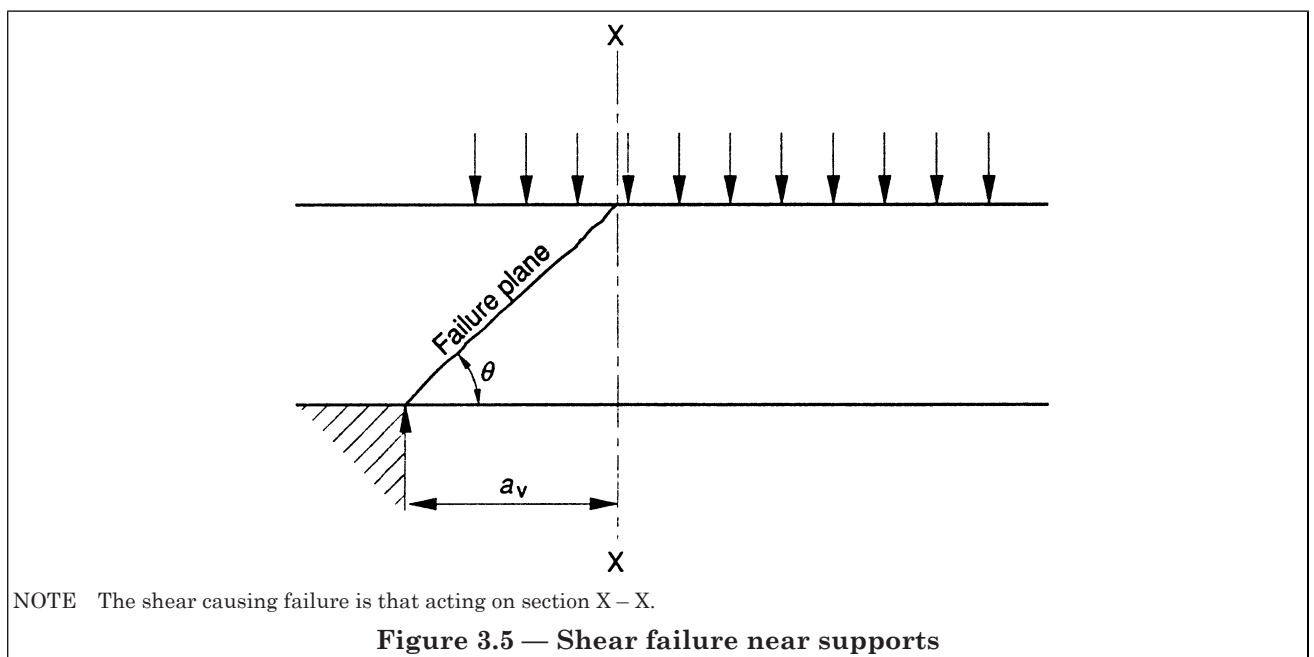


Figure 3.5 — Shear failure near supports

#### 3.4.5.10 Enhanced shear strength near supports (simplified approach)

The procedures given in 3.4.5.8 and 3.4.5.9 may be used for all beams. However, for beams carrying generally uniform load or where the principal load is located further than  $2d$  from the face of the support, the shear stress may be calculated at a section a distance  $d$  from the face of the support. The value of  $v_c$  is calculated in accordance with 3.4.5.4 and appropriate shear reinforcement assessed in accordance with Table 3.7. If this amount of shear reinforcement is provided at sections closer to the support, no further check for shear at such sections is required.

#### 3.4.5.11 Bottom loaded beams

Where load is applied near the bottom of a section, sufficient vertical reinforcement to carry the load should be provided in addition to any reinforcement required to resist shear.

### 3.4.5.12 Shear and axial load

The design shear stress  $v_c'$  that can be supported by a section subjected to shear and axial compression without shear reinforcement can be calculated from equation 6a). Both adverse and beneficial load combinations should be considered (see Table 2.1).

$$v_c' = v_c + 0.6 \frac{NVh}{A_c M} \quad \text{equation 6a)}$$

where

- $v_c$  is the shear stress obtained from Table 3.8. This should not be adjusted in accordance with 3.4.5.8;
- $N$  is the applied axial load;
- $V$  is the shear;
- $M$  is the moment;
- $A_c$  is the gross area of the concrete section.

NOTE 1  $N/A_c$  is intended to be the average stress in the concrete acting at the centroid of the section.

NOTE 2 The value of  $Vh/M$  should be taken as not greater than 1.

Where it is considered necessary to avoid shear cracking prior to the ultimate limit state, the shear stress should be limited to the value given by equation 6b).

$$v_c' = v_c \sqrt{1 + N/(A_c v_c)} \quad \text{equation 6b)}$$

If  $v$  exceeds  $v_c'$  shear reinforcement should be provided in accordance with Table 3.7 where  $v_c$  is replaced by  $v_c'$ .

$v$  should not exceed  $0.8\sqrt{f_{cu}}$  or  $5 \text{ N/mm}^2$ .

For situations where the axial load is tensile, equations 6a) and 6b) may be used with  $N$  taken as negative.

### 3.4.5.13 Torsion

In normal slab-and-beam or framed construction specific calculations are not usually necessary, torsional cracking being adequately controlled by shear reinforcement but when the design relies on the torsional resistance of a member, explicit design for torsion will be necessary. Recommendations are given in 2.4 of BS 8110-2:1985.

## 3.4.6 Deflection of beams

### 3.4.6.1 General

Deflections may be calculated and then compared with the serviceability requirements given in Section 3 of BS 8110-2:1985 but in all normal cases the deflections of a beam will not be excessive if the ratio of its span to its effective depth is not greater than the appropriate ratio obtained from 3.4.6.3 or 3.4.6.4. Where appropriate, Table 3.9 should be modified by Table 3.10 and Table 3.11.

### 3.4.6.2 Symbols

For the purposes of 3.4.6 the following symbols apply.

|                       |                                                                                                                                                                               |
|-----------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $A_{s \text{ prov}}$  | area of tension reinforcement provided at mid-span (at support for cantilever).                                                                                               |
| $A'_{s \text{ prov}}$ | area of compression reinforcement.                                                                                                                                            |
| $A_{s \text{ req}}$   | area of tension reinforcement required at mid-span to resist the moment due to design ultimate loads (at support for cantilever).                                             |
| $b$                   | effective width of a rectangular beam, the effective flange width of a flanged beam or the total average width of the flanges.                                                |
| $b_w$                 | average web width of a beam.                                                                                                                                                  |
| $d$                   | effective depth.                                                                                                                                                              |
| $f_s$                 | estimated design service stress in the tension reinforcement.                                                                                                                 |
| $M$                   | design ultimate moment at the centre of the span or, for a cantilever, at the support.                                                                                        |
| $\beta_b$             | the ratio:<br>$\frac{\text{(moment at the section after redistribution)}}{\text{(moment at the section before redistribution)}}$ from the respective maximum moments diagram. |

### 3.4.6.3 Span/effective depth ratio for a rectangular or flanged beam

The basic span/effective depth ratios for beams are given in Table 3.9. These are based on limiting the total deflection to span/250 and this should normally ensure that the part of the deflection occurring after construction of finishes and partitions will be limited to span/500 or 20 mm, whichever is the lesser, for spans up to 10 m. For values of  $b_w/b$  greater than 0.3, linear interpolation between the values given in Table 3.9 for rectangular sections and for flanged beams with  $b_w/b$  of 0.3 may be used.

**Table 3.9 — Basic span/effective depth ratio for rectangular or flanged beams**

| Support conditions | Rectangular section | Flanged beams with<br>$\frac{b_w}{b} \leq 0.3$ |
|--------------------|---------------------|------------------------------------------------|
| Cantilever         | 7                   | 5.6                                            |
| Simply supported   | 20                  | 16.0                                           |
| Continuous         | 26                  | 20.8                                           |

### 3.4.6.4 Long spans

For spans exceeding 10 m, Table 3.9 should be used only if it is not necessary to limit the increase in deflection after the construction of partitions and finishes. Where limitation is necessary, the values in Table 3.9 should be multiplied by 10/span except for cantilevers where the design should be justified by calculation.

### 3.4.6.5 Modification of span/depth ratios for tension reinforcement

Deflection is influenced by the amount of tension reinforcement and its stress. The span/effective depth ratio should therefore be modified according to the ultimate design moment and the service stress at the centre of the span (or at the support in the case of a cantilever). Values of span/effective depth ratio obtained from Table 3.9 should be multiplied by the appropriate factor obtained from Table 3.10.

### 3.4.6.6 Modification of span/depth ratios for compression reinforcement

Compression reinforcement also influences deflection and the value of the span/effective depth ratio obtained from Table 3.9 modified by the factor obtained from Table 3.10 may be multiplied by a further factor obtained from Table 3.11.

### 3.4.6.7 Deflection due to creep and shrinkage

Permissible span/effective depth ratios obtained from Table 3.9, Table 3.10 and Table 3.11 take account of normal creep and shrinkage deflection. If it is expected that creep or shrinkage of the concrete may be particularly high (e.g. if the free shrinkage strain is expected to be greater than 0.00075 or the creep coefficient greater than 3) or if other abnormally adverse conditions are expected, the permissible span/effective depth ratio should be suitably reduced. A reduction of more than 15 % is unlikely to be required (see also Section 7 of BS 8110-2:1985 for creep coefficient).

### 3.4.7 Crack control in beams

Flexural cracking may be controlled by use of the rules of 3.12.11.2 (maximum distance between bars in tension). If greater spacings are required (e.g. between groups of bars), the expected crack widths should be checked by calculation (see Section 3 of BS 8110-2:1985).

**Table 3.10 — Modification factor for tension reinforcement**

| Service stress | $M/bd^2$ |      |      |      |      |      |      |      |      |
|----------------|----------|------|------|------|------|------|------|------|------|
|                | 0.50     | 0.75 | 1.00 | 1.50 | 2.00 | 3.00 | 4.00 | 5.00 | 6.00 |
| $(f_y = 250)$  | 100      | 2.00 | 2.00 | 2.00 | 1.86 | 1.63 | 1.36 | 1.19 | 1.01 |
|                | 150      | 2.00 | 2.00 | 1.98 | 1.69 | 1.49 | 1.25 | 1.11 | 0.94 |
|                | 167      | 2.00 | 2.00 | 1.91 | 1.63 | 1.44 | 1.21 | 1.08 | 0.92 |
|                | 200      | 2.00 | 1.95 | 1.76 | 1.51 | 1.35 | 1.14 | 1.02 | 0.88 |
|                | 250      | 1.90 | 1.70 | 1.55 | 1.34 | 1.20 | 1.04 | 0.94 | 0.82 |
|                | 300      | 1.60 | 1.44 | 1.33 | 1.16 | 1.06 | 0.93 | 0.85 | 0.76 |
| $(f_y = 500)$  | 333      | 1.41 | 1.28 | 1.18 | 1.05 | 0.96 | 0.86 | 0.79 | 0.72 |

NOTE 1 The values in the table derive from the equation:

$$\text{Modification factor} = 0.55 + \frac{(477 - f_s)}{120 \left( 0.9 + \frac{M}{bd^2} \right)} \leq 2.0 \quad \text{equation 7}$$

where

$M$  is the design ultimate moment at the centre of the span or, for a cantilever, at the support.

NOTE 2 The design service stress in the tension reinforcement in a member may be estimated from the equation:

$$f_s = \frac{2f_{yA_{s \text{ req}}}}{3A_{s \text{ prov}}} \times \frac{1}{\beta_b} \quad \text{equation 8}$$

NOTE 3 For a continuous beam, if the percentage of redistribution is not known but the design ultimate moment at mid-span is obviously the same as or greater than the elastic ultimate moment, the stress  $f_s$  in this table may be taken as  $2/3f_y$ .

**Table 3.11 — Modification factor for compression reinforcement**

| $100 \frac{A_s'_{\text{prov}}}{bd}$ | Factor |
|-------------------------------------|--------|
| 0.00                                | 1.00   |
| 0.15                                | 1.05   |
| 0.25                                | 1.08   |
| 0.35                                | 1.10   |
| 0.50                                | 1.14   |
| 0.75                                | 1.20   |
| 1.0                                 | 1.25   |
| 1.5                                 | 1.33   |
| 2.0                                 | 1.40   |
| 2.5                                 | 1.45   |
| $\geq 3.0$                          | 1.50   |

NOTE 1 The values in this table are derived from the following equation:

$$\text{Modification factor for compression reinforcement} = 1 + \frac{100A_s'_{\text{prov}}}{bd} / \left( s + \frac{100A_s'_{\text{prov}}}{bd} \right) \leq 1.5 \quad \text{equation 9}$$

NOTE 2 The area of compression reinforcement  $A$  used in this table may include all bars in the compression zone, even those not effectively tied with links.

## 3.5 Solid slabs supported by beams or walls

### 3.5.1 Design

In general, the recommendations given in 3.4 for beams will apply also to solid slabs but 3.5.2, 3.5.3, 3.5.4, 3.5.5, 3.5.6, 3.5.7 and 3.5.8 should be taken into account.

### 3.5.2 Moments and forces

#### 3.5.2.1 General

In addition to the methods used for beams, the moments and shear forces resulting from both distributed and concentrated loads may be determined by appropriate elastic analyses. Alternatively, Johansen's yieldline method or Hillerborg's strip method may be used provided the ratio between support and span moments are similar to those obtained by the use of the elastic theory.

#### 3.5.2.2 Distribution of concentrated loads on slabs

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.

- 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.
- 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. When 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  $4x/l(1 - x/l)$  metres where  $x$  and  $l$  are as defined in a) so that, for a section at mid-span, the effective width is equal to 1 m plus the load width.
- Where the concentrated load is near an unsupported edge of a slab the effective width should not exceed the value in a) or b) above as appropriate, nor half that value plus the distance of the centre of the load from the unsupported edge (see Figure 3.6).

### 3.5.2.3 Simplification of load arrangements

In principle a slab should be designed to withstand the most unfavourable arrangements of design loads; however, slabs will normally be able to satisfy this requirement if they are designed to resist the moments and forces arising from the single-load case of maximum design load on all spans or panels provided the following conditions are met.

- a) In a one-way spanning slab the area of each bay exceeds  $30 \text{ m}^2$ .

In this context, a bay means a strip across the full width of a structure bounded on the other two sides by lines of support (see Figure 3.7).

- b) The ratio of the characteristic imposed load to the characteristic dead load does not exceed 1.25.  
c) The characteristic imposed load does not exceed  $5 \text{ kN/m}^2$  excluding partitions.

Where analysis is carried out for the single load case of all spans loaded, the resulting support moments except those at the supports of cantilevers should be reduced by 20 %, with a consequential increase in the span moments.

The resulting bending moment envelope should satisfy the provisions of 3.2.2.1. No further redistribution should be carried out.

Where a span or panel is adjacent to a cantilever of significant length, the possibility should be considered of the case of slab unloaded/cantilever loaded.

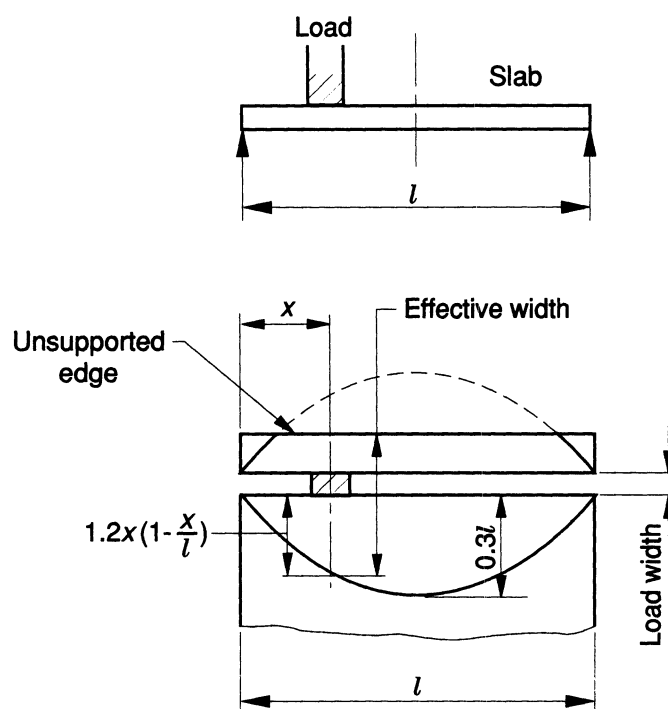
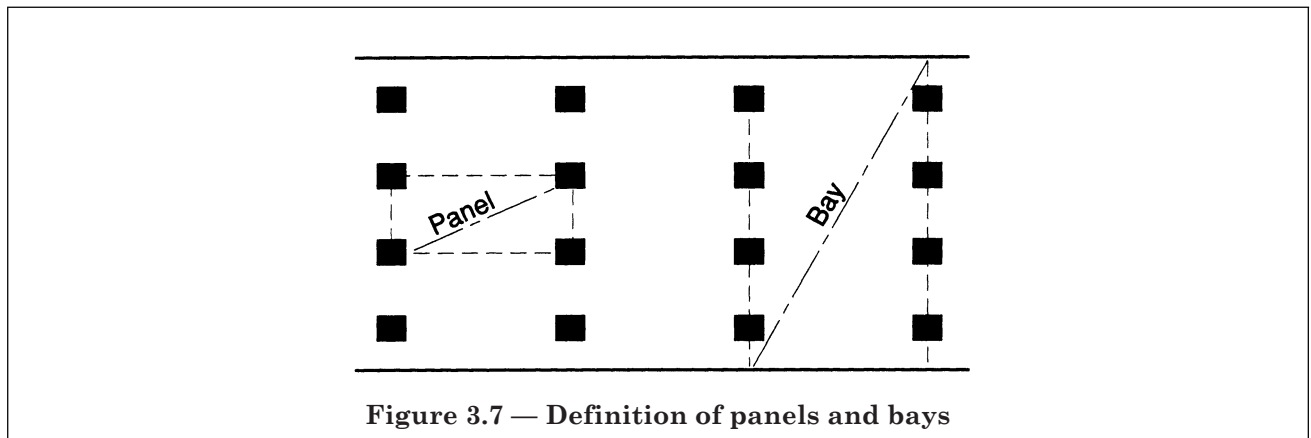


Figure 3.6 — Effective width of solid slab carrying a concentrated load near an unsupported edge



#### 3.5.2.4 One-way spanning slabs of approximately equal span: uniformly distributed loads

Where the conditions of 3.5.2.3 are met, the moments and shears in continuous one-way spanning slabs may be calculated using the coefficients given in Table 3.12. Allowance has been made in these coefficients for the 20 % redistribution mentioned above.

The curtailment of reinforcement designed in accordance with Table 3.12 may be carried out in accordance with the provisions of 3.12.10.

**Table 3.12 — Ultimate bending moment and shear forces in one-way spanning slabs**

|                                                                                                 | End support/slab connection |                         |                  |                         | At first interior support | Middle interior spans | Interior supports |
|-------------------------------------------------------------------------------------------------|-----------------------------|-------------------------|------------------|-------------------------|---------------------------|-----------------------|-------------------|
|                                                                                                 | Simple                      |                         | Continuous       |                         |                           |                       |                   |
|                                                                                                 | At outer support            | Near middle of end span | At outer support | Near middle of end span |                           |                       |                   |
| Moment                                                                                          | 0                           | $0.086Fl$               | $-0.04Fl$        | $0.075Fl$               | $-0.086Fl$                | $0.063Fl$             | $-0.063Fl$        |
| Shear                                                                                           | $0.4F$                      | —                       | $0.46F$          | —                       | $0.6F$                    | —                     | $0.5F$            |
| NOTE $F$ is the total design ultimate load ( $1.4G_k + 1.6Q_k$ );<br>$l$ is the effective span. |                             |                         |                  |                         |                           |                       |                   |

#### 3.5.3 Solid slabs spanning in two directions at right angles: uniformly distributed loads

##### 3.5.3.1 General

Subclauses 3.5.3.3, 3.5.3.4, 3.5.3.5, 3.5.3.6 and 3.5.3.7 may be used for the design of slabs spanning in two directions at right angles and supporting uniformly distributed loads.

### 3.5.3.2 Symbols

For the purposes of 3.5.3, the following symbols apply.

|                                 |                                                                                                                           |
|---------------------------------|---------------------------------------------------------------------------------------------------------------------------|
| $l_x$                           | length of shorter side.                                                                                                   |
| $l_y$                           | length of longer side.                                                                                                    |
| $m_{sx}$                        | maximum design ultimate moments either over supports or at mid-span on strips of unit width and span $l_x$ .              |
| $m_{sy}$                        | maximum design ultimate moments either over supports or at mid-span on strips of unit width and span $l_y$ .              |
| $n$                             | total design ultimate load per unit area ( $1.4G_k + 1.6Q_k$ ).                                                           |
| $N_d$                           | number of discontinuous edges ( $0 \leq N \leq 4$ ).                                                                      |
| $v_{sx}$                        | design end shear on strips of unit width and span $l_x$ and considered to act over the middle three-quarters of the edge. |
| $v_{sy}$                        | design end shear on strips of unit width and span $l_y$ and considered to act over the middle three-quarters of the edge. |
| $\beta_x$                       | sagging moment in the span, per unit width, in the direction of the shorter span, $l_x$ , divided by $nl_x^2$ .           |
| $\beta_y$                       | sagging moment in the span, per unit width, in the direction of the longer span, $l_y$ divided by $nl_x^2$ .              |
| $\beta_1$ and $\beta_2$         | hogging moments, per unit width, over the shorter edges divided by $nl_x^2$ .                                             |
| $\beta_3$ and $\beta_4$         | hogging moments, per unit width, over the longer edges divided by $nl_x^2$ .                                              |
| $\alpha_{sx}$ and $\alpha_{sy}$ | moment coefficients shown in Table 3.13.                                                                                  |
| $\beta_{sx}$ and $\beta_{sy}$   | moment coefficients shown in Table 3.14.                                                                                  |
| $\beta_{vx}$ and $\beta_{vy}$   | shear force coefficients shown in Table 3.15.                                                                             |

### 3.5.3.3 Simply-supported slabs

When simply-supported slabs do not have adequate provision to resist torsion at the corners, and to prevent the corners from lifting, the maximum moments per unit width are given by the following equations:

$$m_{sx} = \alpha_{sx}nl_x^2 \quad \text{equation 10}$$

$$m_{sy} = \alpha_{sy}nl_x^2 \quad \text{equation 11}$$

NOTE Values for  $\alpha_{sx}$  and  $\alpha_{sy}$  are given in Table 3.13.

The values in Table 3.13 are derived from the following equations:

$$\alpha_{sx} = \frac{(l_y/l_x)^4}{8\{1 + (l_y/l_x)^4\}} \quad \text{equation 12}$$

$$\alpha_{sy} = \frac{(l_y/l_x)^2}{8\{1 + (l_y/l_x)^4\}} \quad \text{equation 13}$$

### 3.5.3.4 Restrained slabs

In slabs where the corners are prevented from lifting, and provision for torsion is made, the maximum design moments per unit width are given by equations 14 and 15:

$$m_{sx} = \beta_{sx}nl_x^2 \quad \text{equation 14}$$

$$m_{sy} = \beta_{sy}nl_x^2 \quad \text{equation 15}$$

Where these equations are used, the conditions and rules of 3.5.3.5 should be applied.

NOTE Values of  $\beta_{sx}$  and  $\beta_{sy}$  are given in Table 3.14.

Equations 14 and 15 and the coefficients in Table 3.14 may be derived from the following equations:

$$\beta_y = (24 + 2N_d + 1.5N_d^2)/1\,000 \quad \text{equation 16}$$

$$\gamma = \frac{2}{9} [3 - \sqrt{(18) \frac{l_x}{l_y} \{ \sqrt{(\beta_y + \beta_1)} + \sqrt{(\beta_y + \beta_2)} \}}] \quad \text{equation 17}$$

$$\sqrt{\gamma} = \sqrt{(\beta_x + \beta_3)} + \sqrt{(\beta_x + \beta_4)} \quad \text{equation 18}$$

NOTE  $\beta_1$  and  $\beta_2$  take values of  $4/3\beta_y$  for continuous edges or zero for discontinuous edges.

$\beta_3$  and  $\beta_4$  take values of  $4/3\beta_x$  for continuous edges or zero for discontinuous edges.

### **3.5.3.5 Restrained slabs where the corners are prevented from lifting and adequate provision is made for torsion: conditions and rules for the use of equations 14 and 15**

The conditions in which the equations may be used for continuous slabs only are as follows.

- a) The characteristic dead and imposed loads on adjacent panels are approximately the same as on the panel being considered.
- b) The span of adjacent panels in the direction perpendicular to the line of the common support is approximately the same as the span of the panel considered in that direction.

The rules to be observed when the equations are applied to restrained slabs (continuous or discontinuous) are as follows.

- 1) Slabs are considered as divided in each direction into middle strips and edge strips as shown in Figure 3.9, the middle strip being three-quarters of the width and each edge strip one-eighth of the width.
- 2) The maximum design moments calculated as above apply only to the middle strips and no redistribution should be made.
- 3) Reinforcement in the middle strips should be detailed in accordance with **3.12.10** (simplified rules for curtailment of bars).
- 4) Reinforcement in an edge strip, parallel to the edge, need not exceed the minimum given in **3.12.5** (minimum areas of tension reinforcement), together with the recommendations for torsion given in 5), 6) and 7).
- 5) Torsion reinforcement should be provided at any corner where the slab is simply supported on both edges meeting at that corner. It should consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers should be three-quarters of the area required for the maximum mid-span design moment in the slab.
- 6) Torsion reinforcement equal to half that described in the preceding paragraph should be provided at a corner contained by edges over only one of which the slab is continuous.
- 7) Torsion reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.

### 3.5.3.6 Restrained slab with unequal conditions at adjacent panels

In some cases the support moments calculated from Table 3.14, for adjacent panels, may differ significantly. To adjust them the following procedures may be used.

- Calculate the sum of the midspan moment and the average of the support moments (neglecting signs) for each panel.
- Treat the values from Table 3.14 as fixed end moments (FEMs).
- Distribute the FEMs across the supports according to the relative stiffness of adjacent spans, giving new support moments.
- Adjust midspan moment for each panel: this should be such that when it is added to the average of the support moments from c) (neglecting signs) the total equals that from a).

If, for a given panel, the resulting support moments are now significantly greater than the value from Table 3.14, the tension steel over the supports will need to be extended beyond the provisions of 3.12.10.3. The procedure should be as follows.

- The span moment is taken as parabolic between supports; its maximum value is as found from d).
- The points of contraflexure of the new support moments [from c)] with the span moment [from e)] are determined.
- At each end half the support tension steel is extended to at least an effective depth or 12 bar diameters beyond the nearest point of contraflexure.
- At each end the full area of the support tension steel is extended to half the distance from g).

### 3.5.3.7 Loads on supporting beams

The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations:

$$v_{sy} = \beta_{vy} n l_x \quad \text{equation 19}$$

$$v_{sx} = \beta_{vx} n l_x \quad \text{equation 20}$$

Where design ultimate support moments are used which differ substantially from those that would be assessed from Table 3.14, adjustment of the values given in Table 3.15 may be necessary. The assumed distribution of the load on a supporting beam is shown in Figure 3.10.

**Table 3.13 — 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   |
|---------------|-------|-------|-------|-------|-------|-------|-------|-------|
| $\alpha_{sx}$ | 0.062 | 0.074 | 0.084 | 0.093 | 0.099 | 0.104 | 0.113 | 0.118 |
| $\alpha_{sy}$ | 0.062 | 0.061 | 0.059 | 0.055 | 0.051 | 0.046 | 0.037 | 0.029 |

**Table 3.14 — Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners**

| Type of panel and moments considered                         | Short span coefficients, $\beta_{sx}$ |       |       |       |       |       |       |       | Long span coefficients, $\beta_{sy}$ for all values of $l_y/l_x$ |
|--------------------------------------------------------------|---------------------------------------|-------|-------|-------|-------|-------|-------|-------|------------------------------------------------------------------|
|                                                              | Values of $l_y/l_x$                   |       |       |       |       |       |       |       |                                                                  |
|                                                              | 1.0                                   | 1.1   | 1.2   | 1.3   | 1.4   | 1.5   | 1.75  | 2.0   |                                                                  |
| <b>Interior panels</b>                                       |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.031                                 | 0.037 | 0.042 | 0.046 | 0.050 | 0.053 | 0.059 | 0.063 | 0.032                                                            |
| Positive moment at mid-span                                  | 0.024                                 | 0.028 | 0.032 | 0.035 | 0.037 | 0.040 | 0.044 | 0.048 | 0.024                                                            |
| <b>One short edge discontinuous</b>                          |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.039                                 | 0.044 | 0.048 | 0.052 | 0.055 | 0.058 | 0.063 | 0.067 | 0.037                                                            |
| Positive moment at mid-span                                  | 0.029                                 | 0.033 | 0.036 | 0.039 | 0.041 | 0.043 | 0.047 | 0.050 | 0.028                                                            |
| <b>One long edge discontinuous</b>                           |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.039                                 | 0.049 | 0.056 | 0.062 | 0.068 | 0.073 | 0.082 | 0.089 | 0.037                                                            |
| Positive moment at mid-span                                  | 0.030                                 | 0.036 | 0.042 | 0.047 | 0.051 | 0.055 | 0.062 | 0.067 | 0.028                                                            |
| <b>Two adjacent edges discontinuous</b>                      |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.047                                 | 0.056 | 0.063 | 0.069 | 0.074 | 0.078 | 0.087 | 0.093 | 0.045                                                            |
| Positive moment at mid-span                                  | 0.036                                 | 0.042 | 0.047 | 0.051 | 0.055 | 0.059 | 0.065 | 0.070 | 0.034                                                            |
| <b>Two short edges discontinuous</b>                         |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.046                                 | 0.050 | 0.054 | 0.057 | 0.060 | 0.062 | 0.067 | 0.070 | —                                                                |
| Positive moment at mid-span                                  | 0.034                                 | 0.038 | 0.040 | 0.043 | 0.045 | 0.047 | 0.050 | 0.053 | 0.034                                                            |
| <b>Two long edges discontinuous</b>                          |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | —                                     | —     | —     | —     | —     | —     | —     | —     | 0.045                                                            |
| Positive moment at mid-span                                  | 0.034                                 | 0.046 | 0.056 | 0.065 | 0.072 | 0.078 | 0.091 | 0.100 | 0.034                                                            |
| <b>Three edges discontinuous (one long edge continuous)</b>  |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | 0.057                                 | 0.065 | 0.071 | 0.076 | 0.081 | 0.084 | 0.092 | 0.098 | —                                                                |
| Positive moment at mid-span                                  | 0.043                                 | 0.048 | 0.053 | 0.057 | 0.060 | 0.063 | 0.069 | 0.074 | 0.044                                                            |
| <b>Three edges discontinuous (one short edge continuous)</b> |                                       |       |       |       |       |       |       |       |                                                                  |
| Negative moment at continuous edge                           | —                                     | —     | —     | —     | —     | —     | —     | —     | 0.058                                                            |
| Positive moment at mid-span                                  | 0.042                                 | 0.054 | 0.063 | 0.071 | 0.078 | 0.084 | 0.096 | 0.105 | 0.044                                                            |
| <b>Four edges discontinuous</b>                              |                                       |       |       |       |       |       |       |       |                                                                  |
| Positive moment at mid-span                                  | 0.055                                 | 0.065 | 0.074 | 0.081 | 0.087 | 0.092 | 0.103 | 0.111 | 0.056                                                            |

