British Standard

BS 8110-2:
1985

Reprinted, incorporating Amendments Nos. 1 and 2

Structural use of concrete —

Part 2: Code of practice for special circumstances

ICS 91.080.40
UDC 624.012.3/.4+691.3

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Civil Engineering and Building Structures Standards Committee (CSB/-) to Technical Committee CSB/39, upon which the following bodies were represented:

Association of Consulting Engineers
British Aggregate Construction Materials Industries
British Precast Concrete Federation Ltd.
British Railways Board
British Ready Mixed Concrete Association
British Reinforcement Manufacturers' Association
British Steel Industry
Building Employers' Confederation
Cement Admixtures Association
Cement and Concrete Association
Cement Makers' Federation
Concrete Society
Department of the Environment (Building Research Establishment)
Department of the Environment (Housing and Construction Industries)
Department of the Environment (Property Services Agency)
District Surveyors' Association
Federation of Civil Engineering Contractors
Greater London Council
Incorporated Association of Architects and Surveyors
Institute of Clerks of Works of Great Britain Incorporated
Institution of Civil Engineers
Institution of Municipal Engineers
Institution of Structural Engineers
Precast Flooring Federation
Royal Institute of British Architects
Sand and Gravel Association Limited
Coopted Member

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Committee, was published under the authority of the Board of BSI and comes into effect on 30 August 1985

The following BSI references relate to the work on this standard:
Committee reference CSB/39
Draft for comment 81/15604 DC

ISBN 0 580 14490 9

Amendments issued since publication

Amd. No.	Date of issue	Comments
5914	May 1989	
12061	July 2001	Indicated by a sideline

Contents

Foreword

This part of BS 8110 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. Together with BS 8110-1 it supersedes CP 110-1:1972, which is withdrawn.

BS 8110-1 gives recommendations for design and construction. These recommendations relate particularly to routine building construction which makes up the majority of structural applications; they are in the form of a statement of design objectives and limit state requirements followed by methods to ensure that these are met.

Generally, these methods will involve calculations for one limit state and simple deemed-to-satisfy provisions for the others; for example with reinforced concrete, initial design will normally be for the ultimate limit state, with span/depth ratios and bar spacing rules used to check the limit states of deflection and cracking respectively. This approach is considered the most appropriate for the vast majority of cases.

However, circumstances may arise that would justify a further assessment of actual behaviour, in addition to simply satisfying limit state requirements. This part of BS 8110 gives recommendations to cover the more commonly occurring cases that require additional information or alternative procedures to those given in BS 8110-1; thus this part is complementary to BS 8110:Part 1.

NOTE The numbers in square brackets used throughout the text of this standard relate to the bibliographic references given in Appendix A.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 60, an inside back cover 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 design and construction of structural concrete that arise in special circumstances and are not covered in BS 8110-1.

This part gives guidance on ultimate limit state calculations and the derivation of partial factors of safety, serviceability calculations with emphasis on deflections under loading and on cracking. Further information for greater accuracy in predictions of the different strain components is presented. The need for movement joints is considered and recommendations are made for the provision and design of such joints. General guidance and broad principles relevant to the appraisal and testing of structures and components during construction are included.

NOTE The titles of the publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purposes of this part of BS 8110, the definitions given in BS 8110-1 apply, together with the following.

autoclaving curing with high-pressure steam at not less than 1.0 N/mm^2

1.3 Symbols

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

γ_f	partial safety factor for load
γ_m	partial safety factor for the strength of materials
f_y	characteristic strength of reinforcement
f_{cu}	characteristic strength of concrete

blank

Section 2. Methods of analysis for the ultimate limit state

2.1 General

BS 8110-1 provides methods by which the requirements of the ultimate limit state (ULS) may be satisfied for most normal situations in a reasonably economical manner, from the point of view both of design effort and of material usage. Situations do, however, occasionally arise where the methods given in BS 8110-1 are not directly applicable or where the use of a more rigorous method could give significant advantages. In many cases it would be unreasonable to attempt to draft detailed provisions which could be relied upon to cope with all eventualities. Much of this section is therefore concerned with developing rather more general treatments of the various methods covered than has been considered appropriate in BS 8110-1. The section also gives specific recommendations for certain less common design procedures, such as design for torsion.

2.2 Design loads and strengths

2.2.1 General

2.2.1.1 *Choice of values.* Design loads and strengths are chosen so that, taken together, they will ensure that the probability of failure is acceptably small. The values chosen for each should take account of the uncertainties inherent in that part of the design process where they are of most importance. Design may be considered to be broken down into two basic phases and the uncertainties apportioned to each phase are given in **2.2.1.2** and **2.2.1.3**.

2.2.1.2 *Analysis phase.* This phase is the assessment of the distribution of moments, shear, torsion and axial forces within the structure.

Uncertainties to be considered within this phase are as follows:

a) the magnitude and arrangement of the loads;

b) the accuracy of the method of analysis employed;

c) variations in the geometry of the structures as these affect the assessment of force distributions.

Allowances for these uncertainties are made in the values chosen for γ_f.

2.2.1.3 *Element design phase.* This phase is the design of elements capable of resisting the applied forces calculated in the analysis phase.

Uncertainties to be considered within this phase are as follows:

a) the strength of the material in the structure;

b) the accuracy of the methods used to predict member behaviour;

c) variations in geometry in so far as these affect the assessment of strength.

Allowances for these uncertainties are made in the values chosen for γ_m.

2.2.2 Selection of alternative partial factors

NOTE *Basis of factors in BS 8110-1.* The partial factors given in section 2 of BS 8110-1:1997 have been derived by calibration with pre-existing practice together with a subjective assessment of the relative uncertainties inherent in the various aspects of loading and strength. From experience, they define an acceptable level of safety for normal structures.

2.2.2.1 *General.* There may be cases where, due to the particular nature of the loading or the materials, other factors would be more appropriate. The choice of such factors should take account of the uncertainties listed in **2.2.1.2** and **2.2.1.3** and lead to probabilities of failure similar to those implicit in the use of the factors given in BS 8110-1. Two possible approaches may be adopted; these are given in **2.2.2.2** and **2.2.2.3**.

2.2.2.2 *Statistical methods.* When statistical information on the variability of the parameters considered can be obtained, statistical methods may be employed to define partial factors. The recommendation of specific statistical methods is beyond the scope of this standard and specialist literature should be consulted (for example, CIRIA Report 63[1)] [1]).

2.2.2.3 *Assessment of worst credible values.* Where, by the nature of the parameter considered, clear limits can be placed on its possible value, such limiting values may be used directly in the assessment of a reduced γ factor. The approach is to define, from experience and knowledge of the particular parameter, a "worst credible" value. This is the worst value that the designer realistically believes could occur (it should be noted that, in the case of loading, this could be either a maximum or a minimum load, depending upon whether the effect of the load is adverse or beneficial). This value takes into account some, but not generally all, of the uncertainties given in **2.2.1.2** and **2.2.1.3**. It is therefore still necessary to employ a partial factor but the value can be considerably reduced from that given in BS 8110-1. Absolute minimum values of partial safety factors are given in Table 2.1.

Table 2.1 — Minimum values of partial safety factors to be applied to worst credible values

Parameter	Minimum factor
Adverse loads:	
a) dead load	1.2
b) combined with dead load only	1.2
c) combined with other loads	1.1
Beneficial loads	1.0
Material strengths	1.05

2.2.2.4 *Worst credible values for earth and water pressures.* The use of worst credible values is considered appropriate for many geotechnical problems where statistical methods are of limited value.

Worst credible values of earth and water load should be based on a careful assessment of the range of values that might be encountered in the field. This assessment should take account of geological and other background information, and the results of laboratory and field measurements. In soil deposits the effects of layering and discontinuities have to be taken into account explicitly.

The parameters to be considered when assessing worst credible values include:

a) soil strength in terms of cohesion and/or angle of shearing resistance where appropriate;

b) ground water tables and associated pore water pressures;

c) geometric values, for example excavation depths, soil boundaries, slope angles and berm widths;

NOTE Because of the often considerable effect of these parameters it is essential that explicit allowance is made for them by the designer.

d) surcharge loadings.

NOTE Methods of deriving earth pressures from these parameters can be found in the relevant code of practice.

When several independent parameters may affect the earth loading, a conservative approach is to use worst credible values for all parameters simultaneously when deriving the earth loading.

2.2.3 Implications for serviceability

The simplified rules given in BS 8110-1 for dealing with the serviceability limit state (SLS) are derived on the assumption that the partial factors given in section 2 of BS 8110-1:1997 have been used for both steel and concrete. If significantly different values have been adopted, a more rigorous treatment of the SLS may be necessary (see section 3).

1) Available from the Construction Industry Research and Information Association, 6 Storey's Gate, Westminster, SW1P 3AU.

2.3 Non-linear methods

2.3.1 General

The load-deformation characteristics of reinforced and prestressed concrete members are markedly influenced by the quantity and arrangement of the reinforcement, particularly after cracking has occurred. Analysis can only lead to superior results to the methods suggested in BS 8110-1 where the influence of the reinforcement is taken into account. It follows that more rigorous or non-linear methods are only useful for checking designs or for use in an iterative procedure where the analysis is used as a step in the refinement of a design carried out initially by simpler methods.

2.3.2 Basic assumptions

2.3.2.1 *Design strengths.* It is to be assumed that the material strengths at critical sections within the structure (i.e. sections where failure occurs or where hinges develop) are at their design strength for the ultimate limit state while the materials in all other parts of the structure are at their characteristic strength. If this is difficult to implement within the particular analytical method chosen, it will be acceptable, but conservative, to assume that the whole structure is at its design strength.

2.3.2.2 *Material properties.* Characteristic stress-strain curves may be obtained from appropriate tests on the steel and concrete, taking due account of the nature of the loading. For critical sections, these curves will require modification by the appropriate value of γ_m. In the absence of test data, the following may be used.

a) For critical sections, the design stress-strain curves given in Figures 2.1, 2.2 and 2.3 of BS 8110-1:1997 for both steel and concrete. Concrete is assumed to have zero tensile strength.

b) For non-critical sections, the characteristic stress-strain curves given in Figures 2.2 and 2.3 of BS 8110-1:1997 may be used for reinforcement or prestressing tendons. For concrete, Figure 2.1 of this part of BS 8110 may be adopted. The tensile strength of the concrete may be taken into account up to the cracking load. Above the cracking load, the contribution of the concrete in tension may be taken into account using the assumptions given in item 4) of **3.6**a).

NOTE Information on creep and shrinkage is given in Section 7.

2.3.2.3 *Loading.* The load combinations given in section 2 of BS 8110-1:1997 should be considered. The partial safety factors may be taken from section 2 of BS 8110-1:1997 or derived in accordance with **2.2**. Where the effects of creep, shrinkage or temperature are likely to affect adversely the behaviour (for example where second order effects are important), it will be necessary to consider what part of the loading should be assumed to be long-term. It is acceptable, but conservative in such cases, to consider the full design load as permanent.

2.3.3 Analysis methods

The rapidity of developments in computing methods makes it inappropriate to define specific methods. Any method may be adopted that can be demonstrated to be appropriate for the particular problem being considered (e.g. see [2] and [3]).

2.4 Torsional resistance of beams

2.4.1 General

In normal slab-and-beam or framed construction specific calculations are not usually necessary, torsional cracking being adequately controlled by shear reinforcement. However, when the design relies on the torsional resistance of a member, the recommendations given in **2.4.3**, **2.4.4**, **2.4.5**, **2.4.6**, **2.4.7**, **2.4.8**, **2.4.9** and **2.4.10** should be taken into account.

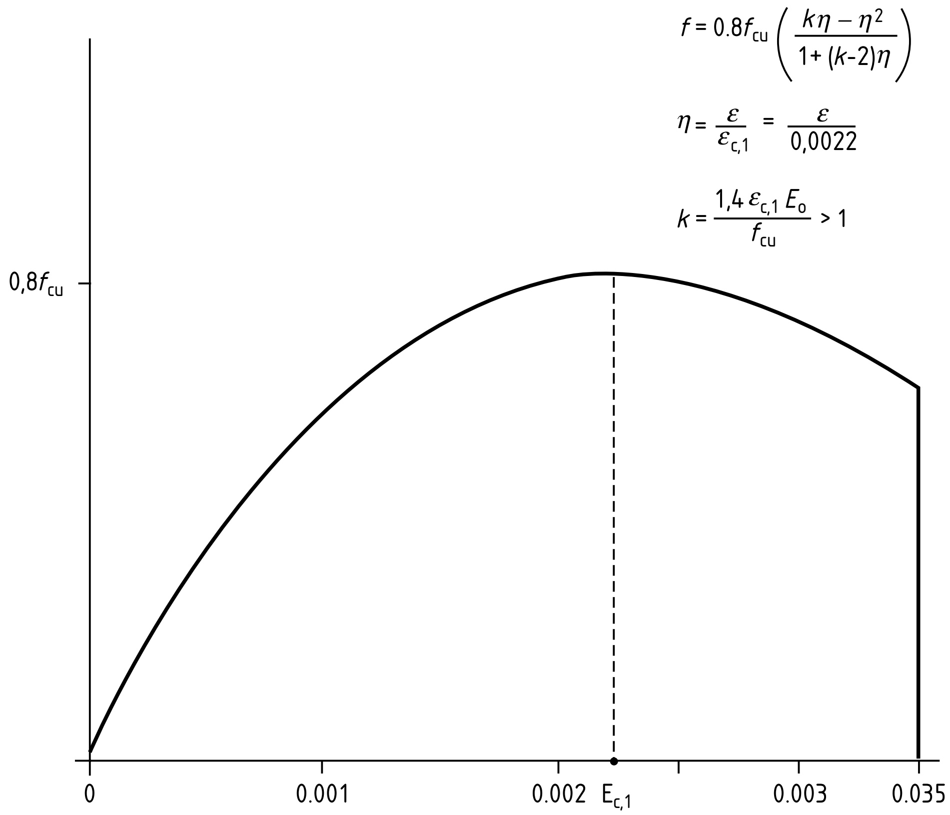


Figure 2.1 — Stress strain curve for rigorous analysis of non-critical sections

2.4.2 Symbols

For the purposes of **2.4** the following symbols apply.

A_s	area of longitudinal reinforcement
A_{sv}	area of two legs of closed links at a section[a]
C	torsional constant (equals half the St. Venant value for the plain concrete section)
f_{yv}	characteristic strength of the links
G	shear modulus
h_{max}	larger dimension of a rectangular section
h_{min}	smaller dimension of a rectangular section
s_v	spacing of the links
T	torsional moment due to ultimate loads
v_t	torsional shear stress
$v_{t,min}$	minimum torsional shear stress, above which reinforcement is required (see Table 2.3)
v_{tu}	maximum combined shear stress (shear plus torsion)
x_1	smaller centre-to-centre dimension of a rectangular link
y_1	larger centre-to-centre dimension of a rectangular link

[a] In a section reinforced with multiple links, only the area of the legs lying closest to the outside of the section should be used.

2.4.3 Calculation of torsional rigidity *(G × C)*

If required in structural analysis or design, the torsional rigidity may be calculated by assuming the shear modulus G equal to 0.42 times the modulus of elasticity of the concrete and assuming the torsional constant C equal to half the St. Venant value calculated for the plain concrete section.

The St. Venant torsional stiffness of a rectangular section may be calculated from equation 1:

$$C = \beta \, \eta^3_{\mu\iota\nu} \, \eta_{\mu\alpha\xi} \qquad \text{equation 1}$$

where

β is a coefficient depending on the ratio h/b (overall depth of member divided by the breadth).

NOTE Values of β are given in Table 2.2.

Table 2.2 — Values of coefficient β

h_{max}/h_{min}	1	1.5	2	3	5	> 5
β	0.14	0.20	0.23	0.26	0.29	0.33

The St. Venant torsional stiffness of a non-rectangular section may be obtained by dividing the section into a series of rectangles and summing the torsional stiffness of these rectangles. The division of the section should be arranged so as to maximize the calculated stiffness. This will generally be achieved if the widest rectangle is made as long as possible.

2.4.4 Torsional shear stress

2.4.4.1 *Rectangular sections.* The torsional shear stress v_t at any section should be calculated assuming a plastic stress distribution and may be calculated from equation 2:

$$v_t = \frac{2T}{h_{min}^2 \left(h_{max} - \frac{h_{min}}{3}\right)} \qquad \text{equation 2}$$

2.4.4.2 *T-, L- or I- sections.* T-, L- or I- sections are divided into their component rectangles; these are chosen in such a way as to maximize $h^3_{min}\ h_{max}$ in the following expression.

The torsional shear stress v_t carried by each of these component rectangles may be calculated by treating them as rectangular sections subjected to a torsional moment of:

$$T \left(\frac{h^3_{min}\ h_{max}}{\Sigma\ (h^3_{min}\ h_{max})} \right)$$

2.4.4.3 *Hollow sections.* Box and other hollow sections in which wall thicknesses exceed one-quarter of the overall thickness of the member in the direction of measurement may be treated as solid rectangular sections.

NOTE For other sections, specialist literature should be consulted.

2.4.5 Limit to shear stress

In no case should the sum of the shear stresses resulting from shear force and torsion ($v + v_t$) exceed v_{tu} in Table 2.3 nor, in the case of small sections where $y_1 < 550$ mm, should the torsional shear stress v_t exceed $v_{tu}\ y_1/550$.

2.4.6 Reinforcement for torsion

Where the torsion shear stress v_t exceeds $v_{t,min}$ in Table 2.3, reinforcement should be provided. Recommendations for reinforcement for combinations of shear and torsion are given in Table 2.4.

Table 2.3 — Values of $v_{t,min}$ and v_{tu}

Concrete grade	$v_{t,min}$	v_{tu}
	N/mm^2	N/mm^2
25	0.33	4.00
30	0.37	4.38
40 or above	0.40	5.00

NOTE 1 Allowance is made for γ_m.

NOTE 2 Values of $v_{t,min}$ and v_{tu} (in N/mm^2) are derived from the equations:

$v_{t,min}$ = 0.067 $\sqrt{f_{cu}}$ but not more than 0.4 N/mm^2

v_{tu} = 0.8 $\sqrt{f_{cu}}$ but not more than 5 N/mm^2

Table 2.4 — Reinforcement for shear and torsion

	$v_t < v_{t,min}$	$v_t > v_{t,min}$
$v \leq v_c + 0.4$	Minimum shear reinforcement; no torsion reinforcement	Designed torsion reinforcement but not less than the minimum shear reinforcement
$v > v_c + 0.4$	Designed shear reinforcement; no torsion reinforcement	Designed shear and torsion reinforcement

2.4.7 Area of torsional reinforcement

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

$$\frac{A_{sv}}{s_v} > \frac{T}{0.8\,x_1\,y_1\,(0.95 f_{yv})}$$

$$A_s > \frac{A_{sv}\,f_{yv}\,(x_1 + y_1)}{s_v\,f_y}$$

NOTE f_y and f_{yv} should not be taken as greater than 460 N/mm².

2.4.8 Spacing and type of links

The value s_v should not exceed the least of x_1, $y_1/2$ or 200 mm. The links should be a closed shaped

with dimensions x_1 and y, as above.

2.4.9 Arrangement of longitudinal torsion reinforcement

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

2.4.10 Arrangement of links in T-, L- or I-sections

In the component rectangles, the reinforcement cages should be detailed so that they interlock and tie the component rectangles of the section together. Where the torsional shear stress in a minor component rectangle does not exceed $v_{t,min}$, no torsion reinforcement need be provided in that rectangle.

2.5 Effective column height

2.5.1 General

Simplified recommendations are given in BS 8110-1 for the assessment of effective column heights for common situations. Where a more accurate assessment is desired, the equations given in **2.5.5** and **2.5.6** may be used.

2.5.2 Symbols

For the purposes of **2.5** the following symbols apply.

I	second moment of area of the section
l_e	effective height of a column in the plane of bending considered
l_o	clear height between end restraints
$\alpha_{c,1}$	ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the lower end of a column
$\alpha_{c,2}$	ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the upper end of a column
$\alpha_{c,min}$	lesser of $\alpha_{c,1}$ and $\alpha_{c,2}$

2.5.3 Stiffness of members

In the calculation of α_c, only members properly framed into the end of the column in the appropriate plane of bending should be considered. The stiffness of each member equals I/l_0.

2.5.4 Relative stiffness

In specific cases of relative stiffness the following simplifying assumptions may be used:

a) *flat slab construction:* the beam stiffness is based on an equivalent beam of the width and thickness of the slab forming the column strip;

b) *simply-supported beams framing into a column:* α_c to be taken as 10;

c) *connection between column and base designed to resist only nominal moment:* α_c to be taken as 5;

d) *connection between column and base designed to resist column moment:* α_c to be taken as 1.0.

2.5.5 Braced columns: effective height for framed structures

The effective height for framed structures may be taken as the lesser of:

$l_e = l_0 [0.7 + 0.05 (\alpha_{c,1} + \alpha_{c,2})] < l_0$ equation 3

$l_e = l_0 (0.85 + 0.05 \alpha_{c,min}) < l_0$ equation 4

2.5.6 Unbraced columns: effective height for framed structures

The effective height for framed structures may be taken as the lesser of:

$l_e = l_0 [1.0 + 0.15 (\alpha_{c,1} + \alpha_{c,2})]$ equation 5

$l_e = l_0 (2.0 + 0.3 \alpha_{c,min})$ equation 6

2.6 Robustness

2.6.1 General

Section 3 of BS 8110-1:1997 gives details of the normal method of ensuring robustness by the provision of vertical and horizontal ties. There may, however, be cases where there are key elements as defined in **2.2.2.2**c) of BS 8110-1:1997 or where it is impossible to provide effective ties in accordance with **3.12.3** of BS 8110-1:1997. Details of such cases are given in **2.6.2** and **2.6.3**.

2.6.2 Key elements

2.6.2.1 *Design of key elements (where required in buildings of five or more storeys).* Whether incorporated as the only reasonable means available to ensuring a structure's integrity in normal use or capability of surviving accidents, key elements should be designed, constructed and protected as necessary to prevent removal by accident.

2.6.2.2 *Loads on key elements.* Appropriate design loads should be chosen having regard to the importance of the key element and the likely consequences of its failure, but in all cases an element and its connections should be capable of withstanding a design ultimate load of 34 kN/m^2, to which no partial safety factor should be applied, from any direction. A horizontal member, or part of a horizontal member that provides lateral support vital to the stability of a vertical key element, should also be considered a key element. For the purposes of **2.6.2**, the area to which these loads are applied will be the projected area of the member (i.e. the area of the face presented to the loads).

2.6.2.3 *Key elements supporting attached building components.* Key elements supporting attached building components should also be capable of supporting the reactions from any attached building components also assumed to be subject to a design ultimate loading of 34 kN/m^2. The reaction should be the maximum that might reasonably be transmitted having regard to the strength of the attached component and the strength of its connection.

2.6.3 Design of bridging elements (where required in buildings of five or more storeys)

2.6.3.1 *General.* At each storey in turn, each vertical load-bearing element, other than a key element, is considered lost in turn. (The design should be such that collapse of a significant part of the structure does not result.) If catenary action is assumed, allowance should be made for the horizontal reactions necessary for equilibrium.

2.6.3.2 ***Walls***

2.6.3.2.1 *Length considered lost.* The length of wall considered to be a single load-bearing element should be taken as the length between adjacent lateral supports or between a lateral support and a free edge (see **2.6.3.2.2**).

2.6.3.2.2 *Lateral support.* For the purposes of this subclause, a lateral support may be considered to occur at:

a) a stiffened section of the wall (not exceeding 1.0 m in length) capable of resisting a horizontal force (in kN per metre height of the wall) of 1.5 F_t; or

b) a partition of mass not less than 100 kg/m^2 at right angles to the wall and so tied to it as to be able to resist a horizontal force (in kN per metre height of wall) of 0.5 F_t;

where

F_t is the lesser of $(20 + 4\ n_0)$ or 60, where n_0 is the number of storeys in the structure.

blank

Section 3. Serviceability calculations

3.1 General

3.1.1 Introduction

In BS 8110-1 design requirements for the serviceability limit state are stated and two alternative approaches are suggested namely:

a) by analysis whereby the calculated values of effects of loads, e.g. deflections and crackwidths, are compared with acceptable values;

b) by deemed-to-satisfy provisions, such as limiting span/depth ratios and detailing rules.

The purpose of this section is to provide further guidance when the first of these approaches is adopted. In addition this information will be of use when it is required not just to comply with a particular limit state requirement but to obtain a best estimate of how a particular structure will behave, for example when comparing predicted deflections with on-site measurements.

3.1.2 Assumptions

When carrying out serviceability calculations it is necessary to make sure that the assumptions made regarding loads and material properties are compatible with the way the results will be used.

If a best estimate of the expected behaviour is required, then the expected or most likely values should be used.

In contrast, in order to satisfy a serviceability limit state, it may be necessary to take a more conservative value depending on the severity of the particular serviceability limit state under consideration, i.e. the consequences of failure. (Failure here means failure to meet the requirements of a limit state rather than collapse of the structure.) It is clear that serviceability limit states vary in severity and furthermore what may be critical in one situation may not be important in another.

In **3.2** the various limit states are examined in greater detail. Guidance on the assumptions regarding loads and material values are given in **3.3** and **3.4** respectively and **3.5** gives further guidance on methods of calculation.

3.2 Serviceability limit states

3.2.1 Excessive deflections due to vertical loads

3.2.1.1 *Appearance.* For structural members that are visible, the sag in a member will usually become noticeable if the deflection exceeds $l/250$, where l is either the span or, in the case of a cantilever, its length.

This shortcoming can in many cases be at least partially overcome by providing an initial camber. If this is done, due attention should be paid to the effects on construction tolerances, particularly with regard to thicknesses of finishes.

This shortcoming is naturally not critical if the element is not visible.

3.2.1.2 *Damage to non-structural elements.* Unless partitions, cladding and finishes, have been specifically detailed to allow for the anticipated deflections, some damage can be expected if the deflection after the installation of such finishes and partitions exceeds the following values:

a) $L/500$ or 20 mm, whichever is the lesser, for brittle materials;

b) $L/350$ or 20 mm, whichever is the lesser, for non-brittle partitions or finishes;

where L is the span or, in the case of a cantilever, its length.

NOTE These values are indicative only.

These values also apply, in the case of prestressed construction, to upward deflections.

3.2.1.3 *Construction lack of fit.* All elements should be detailed so that they will fit together on site allowing for the expected deflections, together with the tolerances allowed by the specification.

3.2.1.4 *Loss of performance.* Loss of performance includes effects such as excessive slope and ponding.

Where there are any such specific limits to the deflection that can be accepted, these should be taken account of explicitly in the design.

3.2.2 Excessive response to wind loads

3.2.2.1 *Discomfort or alarm to occupants.* Excessive accelerations under wind loads that may cause discomfort or alarm to occupants should be avoided.

NOTE For guidance on acceptable limits, reference should be made to specialist literature.

3.2.2.2 *Damage to non-structural elements.* Unless partitions, cladding and finishes, etc. have been specifically detailed to allow for the anticipated deflections, relative lateral deflection in any one storey under the characteristic wind load should not exceed $H/500$, where H is the storey height.

3.2.3 Excessive vibration

Excessive vibration due to fluctuating loads that may cause discomfort or alarm to occupants, either from people or machinery, should be avoided.

NOTE For further guidance reference should be made to specialist literature.

3.2.4 Excessive cracking

3.2.4.1 *Appearance.* For members that are visible, cracking should be kept within reasonable bounds by attention to detail. As a guide the calculated maximum crack width should not exceed 0.3 mm.

3.2.4.2 *Corrosion.* For members in aggressive environments, the calculated maximum crack widths should not exceed 0.3 mm.

3.2.4.3 *Loss of performance.* Where cracking may impair the performance of the structure, e.g. watertightness, limits other than those given in **3.2.4.1** and **3.2.4.2** may be appropriate.

For prestressed members, limiting crack widths are specified in section 2 of BS 8110-1:1997.

3.3 Loads

3.3.1 General

The loading assumed in serviceability calculations will depend on whether the aim is to produce a best estimate of the likely behaviour of the structure or to comply with a serviceability limit state requirement and, if the latter, the severity of that limit state (see **3.1.2**).

In assessing the loads, a distinction should be made between "characteristic" and "expected" values. Generally, for best estimate calculations, expected values should be used. For calculations to satisfy a particular limit state, generally lower or upper bound values should be used depending on whether or not the effect is beneficial. The actual values assumed however should be a matter for engineering judgement.

For loads that vary with time, e.g. live and wind loads, it is necessary to choose values that are compatible with the response time of the structure and the assumptions made regarding material and section properties (see **3.5**).

3.3.2 Dead loads

For dead loads, the expected and characteristic values are the same. Generally, in serviceability calculations (both best estimate and limit state) it will be sufficient to take the characteristic value.

3.3.3 Live loads

Generally, the characteristic value should be used in limit state calculations and the expected value in best estimate calculations.

When calculating deflections, it is necessary to assess how much of the load is permanent and how much is transitory. The proportion of the live load that should be considered as permanent will, however, depend on the type of structure. It is suggested that for normal domestic or office occupancy, 25 % of the live load should be considered as permanent and for structures used for storage, at least 75 % should be considered permanent when the upper limit to the deflection is being assessed.

3.4 Analysis of structure for serviceability limit states

In general, it will be sufficiently accurate to assess the moments and forces in members subjected to their appropriate loadings for the serviceability limit states using an elastic analysis. Where a single value of stiffness is used to characterize a member, the member stiffness may be based on the concrete section. In this circumstance it is likely to provide a more accurate picture of the moment and force fields than will the use of a cracked transformed section, even though calculation shows the members to be cracked. Where more sophisticated methods of analysis are used in which variations in properties over the length of members can be taken into account, it will frequently be more appropriate to calculate the stiffness of highly stressed parts of members on the basis of a cracked transformed section.

3.5 Material properties for the calculation of curvature and stresses

For checking serviceability limit states, the modulus of elasticity of the concrete should be taken as the mean value given in Table 7.2 appropriate to the characteristic strength of the concrete. The modulus of elasticity may be corrected for the age of loading where this is known. Where a "best estimate" of the curvature is required, an elastic modulus appropriate to the expected concrete strength may be used. Attention is, however, drawn to the large range of values for the modulus of elasticity that can be obtained for the same cube strength. It may therefore be appropriate to consider either calculating the behaviour using moduli at the ends of the ranges given in Table 7.2 to obtain an idea of the reliability of the calculation or to have tests done on the actual concrete to be used. Reference may be made to Section 7 for appropriate values for creep and shrinkage in the absence of more direct information.

3.6 Calculation of curvatures

The curvature of any section may be calculated by employing whichever of the following sets of assumptions a) or b) gives the larger value. Item a) corresponds to the case where the section is cracked under the loading considered, item b) applies to an uncracked section.

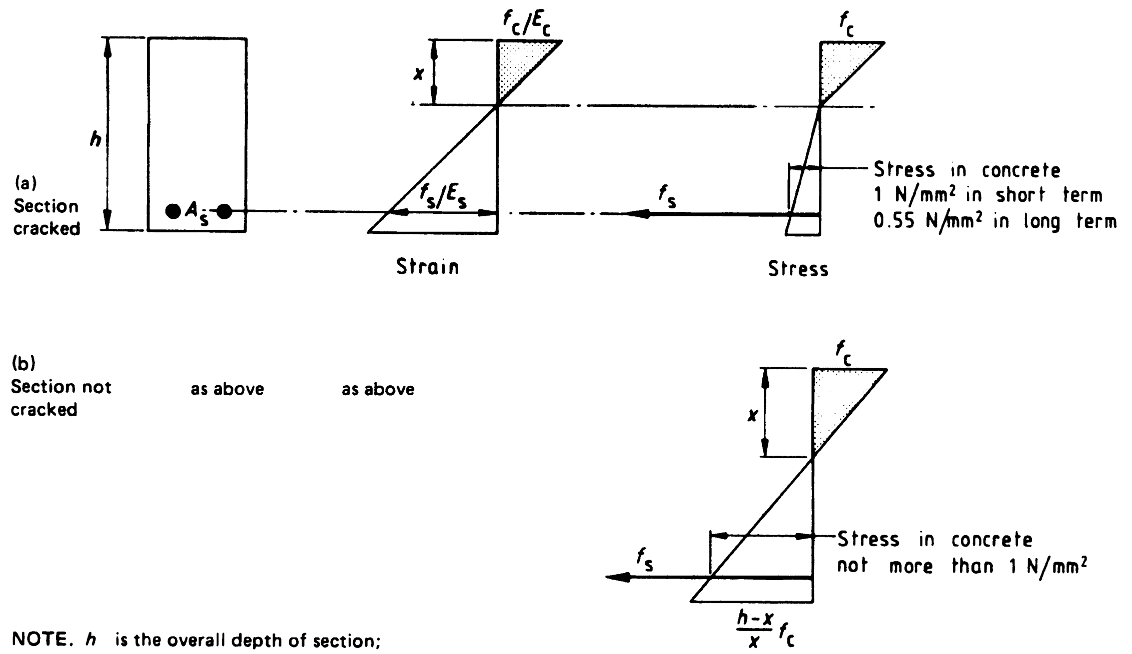
a) 1) Strains are calculated on the assumption that plane sections remain plane.
   2) The reinforcement, whether in tension or in compression, is assumed to be elastic. Its modulus of elasticity may be taken as 200 kN/mm^2.
   3) The concrete in compression is assumed to be elastic. Under short-term loading the modulus of elasticity may be taken as that obtained from **3.5**. Under long-term loading, an effective modulus may be taken having a value of $1/(1 + \phi)$ times the short-term modulus where ϕ is the appropriate creep coefficient (see **7.3**).
   4) Stresses in the concrete in tension may be calculated on the assumption the stress distribution is triangular, having a value of zero at the neutral axis and a value at the centroid of the tension steel of 1 N/mm^2 instantaneously, reducing to 0.55 N/mm^2 in the long term.

b) The concrete and the steel are both considered to be fully elastic in tension and in compression. The elastic modulus of the steel may be taken as 200 kN/mm^2 and the elastic modulus of the concrete is as derived from a) 3) both in compression and in tension.

These assumptions are illustrated in Figure 3.1

In each case, the curvature can be obtained from the following equation:

$$\frac{1}{r_b} = \frac{f_c}{xE_c} = \frac{f_s}{(d-x)\,E_s} \qquad \text{equation 7}$$



NOTE. h is the overall depth of section;
x is the depth from the compression face to the neutral axis;
f_c is the maximum compressive stress in the concrete;
f_s is the tensile stress in the reinforcement;
E_s is the modulus of elasticity of the reinforcement.

Figure 3.1 — Assumptions made in calculating curvatures

where

$\frac{1}{r_b}$ is the curvature at mid-span or, for cantilevers, at the support section;

f_c is the design service stress in the concrete;

E_c is the short-term modulus of the concrete;

f_s is the estimated design service stress in tension reinforcement;

d is the effective depth of the section;

x is the depth to the neutral axis;

E_s is the modulus of elasticity of the reinforcement.

For b) the following alternative may be more convenient:

$$\frac{1}{r_b} = \frac{M}{E_c I}$$ equation 8

where

M is the moment at the section considered;
I is the second moment of area.

Assessment of the stresses by using a) requires a trial-and-error approach. Calculation by means of a computer or programmable calculator is straightforward.

In assessing the total long-term curvature of a section, the following procedure may be adopted.

i) Calculate the instantaneous curvatures under the total load and under the permanent load.

ii) Calculate the long-term curvature under the permanent load.

iii) Add to the long-term curvature under the permanent load the difference between the instantaneous curvature under the total and permanent load.

iv) Add to this curvature the shrinkage curvature calculated from the following equation:

$$\frac{1}{r_{cs}} = \frac{\epsilon_{cs}\,\alpha_e\,S_s}{I}$$ equation 9

where

$\frac{1}{r_{cs}}$	is the shrinkage curvature;
α_e	is the modular ratio = $\frac{E_s}{E_{eff}}$;
ϵ_{cs}	is the free shrinkage strain (see **7.4**);
E_{eff}	is the effective modulus of elasticity of the concrete which can be taken as $E_c/(1 + \phi)$;
E_c	is the short-term modulus of the concrete;
E_s	is the modulus of elasticity of the reinforcement;
ϕ	is the creep coefficient;
I	is the second moment of area of either the cracked or the gross section, depending on whether the curvature due to loading is derived from assumptions a) or b) respectively. NOTE In assessing the transformed steel area, the modular ratio should be as defined above.
S_s	is the first moment of area of the reinforcement about the centroid of the cracked or gross section, whichever is appropriate.

3.7 Calculation of deflection

3.7.1 General

When the deflections of reinforced concrete members are calculated, it should be realized that there are a number of factors that may be difficult to allow for in the calculation which can have a considerable effect on the reliability of the result. These are as follows.

a) Estimates of the restraints provided by supports are based on simplified and often inaccurate assumptions.

b) The precise loading, or that part which is of long duration, is unknown.

The dead load is the major factor determining the deflection, as this largely governs the long-term effects. Because the dead load is known to within quite close limits, lack of knowledge of the precise imposed load is not likely to be a major cause of error in deflection calculations. Imposed loading is highly uncertain in most cases; in particular, the proportion of this load which may be considered to be permanent and will influence the long-term behaviour (see **3.3.3**).

c) Lightly reinforced members may well have a working load that is close to the cracking load for the members. Considerable differences will occur in the deflections depending on whether the member has or has not cracked.

d) The effects on the deflection of finishes and partitions are difficult to assess and are often neglected.

Finishes and rigid partitions added after the member is carrying its self-weight will help to reduce the long-term deflection of a member. As the structure creeps, any screed will be put into compression, thus causing some reduction in the creep deflection. The screed will generally be laid after the propping has been removed from the member, and so a considerable proportion of the long-term deflection will have taken place before the screed has gained enough stiffness to make a significant contribution. It is suggested that only 50 % of the long-term deflection should be considered as reduceable by the action of the screed. If partitions of blockwork are built up to the underside of a member and no gap is left between the partition and the member, creep can cause the member to bear on the partition which, since it is likely to be very stiff, will effectively stop any further deflection along the line of the wall. If a partition is built on top of a member where there is no wall built up to the underside of the member, the long-term deflection will cause the member to creep away from the partition. The partition may be left spanning as a self-supporting deep beam that will apply significant loads to the supporting member only at its ends. Thus, if a partition wall is built over the whole span of a member with no major openings near its centre, its mass may be ignored in calculating long-term deflections.

A suitable approach for assessing the magnitude of these effects is to calculate a likely maximum and minimum to their influence and take the average.

3.7.2 Calculation of deflection from curvatures

The deflected shape of a member is related to the curvatures by the equation:

$$\frac{1}{r_x} = \frac{d^2a}{dx^2} \qquad \text{equation 10}$$

where

$\frac{1}{r_x}$ is the curvature at x;

a is the deflection at x.

Deflections may be calculated directly from this equation by calculating the curvatures at successive sections along the member and using a numerical integration technique. Alternatively, the following simplified approach may be used:

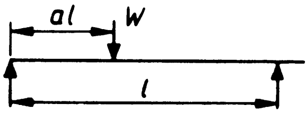
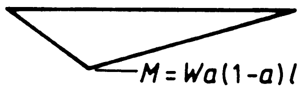
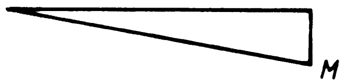
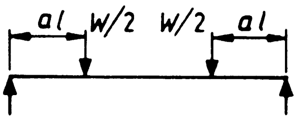
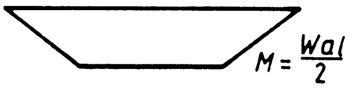
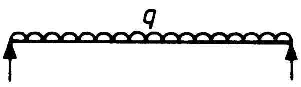
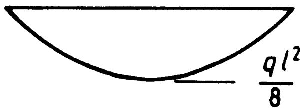
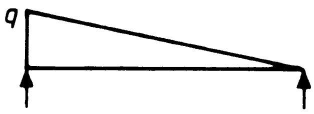
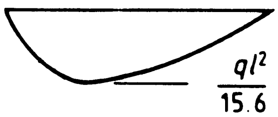
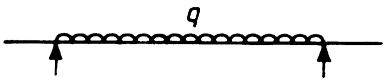
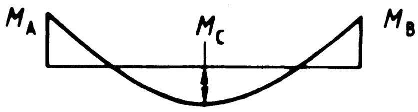
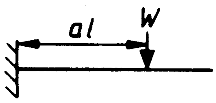
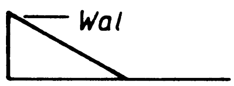
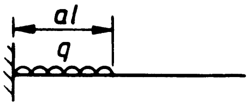
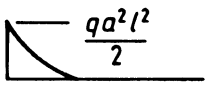
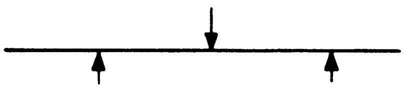
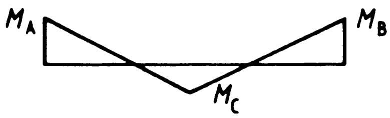
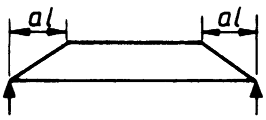
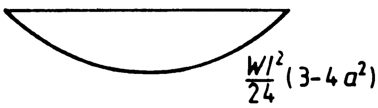
$$a = Kl^2 \frac{1}{r_b} \qquad \text{equation 11}$$

where

l	is the effective span of the member;
$\frac{1}{r_b}$	is the curvature at mid-span or, for cantilevers, at the support section;
K	is a constant that depends on the shape of the bending moment diagram.

Table 3.1 gives values of the coefficient K for various common shapes of bending moment diagram. As the calculation method does not describe an elastic relationship between moment and curvature, deflections under complex loads cannot be obtained by summing the deflections obtained by separate calculation for the constituent simpler loads. A value of K appropriate to the complete load should be used.

Table 3.1 — Values of K for various bending moment diagrams

Loading	Bending moment diagram	K
		0.125
		$\frac{3-4a^2}{48(1-a)}$ if $a = \frac{1}{2}$ $K = \frac{1}{12}$
		0.0625
		$0.125 - \frac{a^2}{6}$
		0.104
		0.102
		$K = 0.104\left(1 - \frac{\beta}{10}\right)$ $\beta = \frac{M_A + M_B}{M_C}$
		end deflection $= \frac{a(3-a)}{6}$ load at end $K = 0.333$
		$\frac{a(4-a)}{12}$ if $a = l$ $K = 0.25$
		$K = 0.083\left(1 - \frac{\beta}{4}\right)$ $\beta = \frac{M_A + M_B}{M_C}$
		$\frac{1}{80}\ \frac{(5-4a^2)^2}{3-4a^2}$

The calculation of the deflection of cantilevers requires very careful consideration in some circumstances. The usual formulae for the end deflection of cantilevers assume that the cantilever is rigidly fixed and is therefore horizontal at the root. In practice, this is by no means necessarily so, because the loading on the cantilever itself, or on other members to which the cantilever connects, may cause the root of the cantilever to rotate. If this root rotation is θ, the deflection of the tip of the cantilever will be decreased or increased by an amount $l\theta$. There are two sources of root rotation which may occur. First, rotation of the joint in the frame to which the cantilever connects (see Figure 3.2). This problem will require attention only when the supporting structure is fairly flexible. Secondly, even where the cantilever connects to a substantially rigid structure, some root rotation will occur. This is because the steel stress, which is at a maximum at the root, should be dissipated into the supporting structure over some length of the bar embedded in the support. To allow for this, it is important to use the effective span of the cantilever as defined in **3.4.1.4** of BS 8110-1:1997.

If Table 3.1 is used to assess the value of K by superposition, it may be assumed that the maximum deflection of a beam occurs at mid-span without serious errors being introduced.

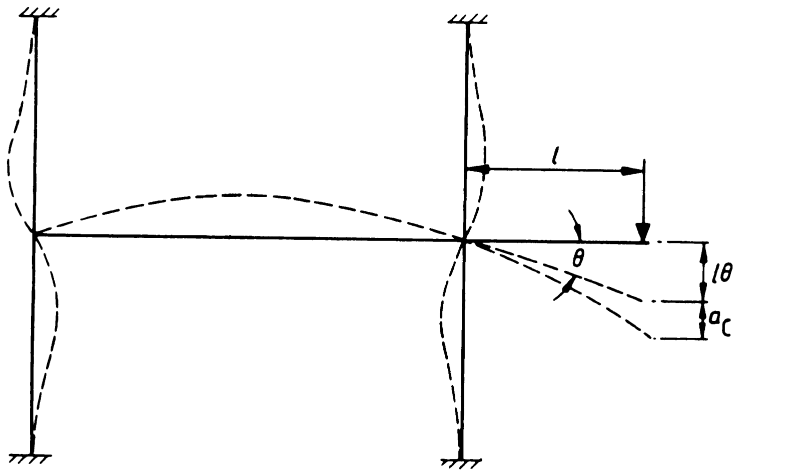
The problem of estimating the deflection of two-way spanning slabs is not simple. Before they crack, slabs will behave substantially as elastic, isotropic slabs. As soon as cracking occurs, the slabs become anisotropic, the amount of this anisotropy varying continuously as the loading varies, and so a reliable determination of the moment surface for the slab under any particular load is not normally practicable. Deflections of slabs are therefore probably best dealt with by using the ratios of span to effective depth. However, if the engineer feels that the calculation of the deflections of a slab is essential, it is suggested that the following procedure be adopted.

A strip of slab of unit width is chosen such that the maximum moment along it is the maximum moment of the slab, i.e. in a rectangular slab, a strip spanning across the shorter dimension of the slab connecting the centres of the longer sides. The bending moments along this strip should preferably be obtained from an elastic analysis of the slab but may be assessed approximately by taking 70 % of the moments used for the collapse design. The deflection of the strip is then calculated as though it were a beam. This method will be slightly conservative.

3.8 Calculation of crack width

3.8.1 General

Since the bar spacing rules given in **3.12.11** of BS 8110-1:1997 have to ensure that cracking is not serious in the worst likely practical situation, it will almost always be found that wider bar spacings can be used if the crack widths are checked explicitly. This will be particularly true for fairly shallow members.



NOTE. a_c is the deflection of the top of a cantilever relative to the tangent at the support.

Figure 3.2 — Deflection of a cantilever forming part of a framed structure

The widths of flexural cracks at a particular point on the surface of a member depend primarily on three factors:

a) the proximity to the point considered of reinforcing bars perpendicular to the cracks;

b) the proximity of the neutral axis to the point considered;

c) the average surface strain at the point considered.

Equation 12 in **3.8.3** gives a relationship between crack width and these three principal variables which gives acceptably accurate results in most normal design circumstances; however, the formula should be used with caution in members subjected dominantly to an axial tension.

It should be remembered that cracking is a semi-random phenomenon and that an absolute maximum crack width cannot be predicted. The formula is designed to give a width with an acceptably small chance of being exceeded, thus an occasional crack slightly larger than the predicted width should not be considered as cause for concern. However, should a significant number of cracks in a structure exceed the calculated width, reasons other than the statistical nature of the phenomenon should be sought to explain their presence.

3.8.2 Symbols

For the purposes of **3.8** the following symbols apply.

a'	distance from the compression face to the point at which the crack width is being calculated
a_{cr}	distance from the point considered to the surface of the nearest longitudinal bar
A_s	area of tension reinforcement
b_t	width of the section at the centroid of the tension steel
c_{min}	minimum cover to the tension steel
d	effective depth
E_s	modulus of elasticity of the reinforcement (N/mm²)
h	overall depth of the member
R	restraint factor (see Table 3.3)
x	depth of the neutral axis
α	coefficient of expansion of the concrete
Δ_t	temperature differential
ϵ_1	strain at the level considered, calculated ignoring the stiffening effect of the concrete in the tension zone
ϵ_m	average strain at the level where the cracking is being considered
ϵ_r	strain accompanied by cracking

3.8.3 Assessment of crack widths

Provided the strain in the tension reinforcement is limited to $0.8f_y/E_s$, the design surface crack width, which should not exceed the appropriate value given in **3.2.4** may be calculated from the following equation:

$$\text{Design surface crack width} = \frac{3a_{cr}\,\epsilon_m}{1 + 2\left(\dfrac{a_{cr} - c_{min}}{h - x}\right)} \qquad \text{equation 12}$$

The average strain ϵ_m may be calculated on the basis of the assumptions given in **3.6**. Alternatively, as an approximation, it will normally be satisfactory to calculate the steel stress on the basis of a cracked section and then reduce this by an amount equal to the tensile force generated by the stress distribution defined in **3.6** a) 4) acting over the tension zone divided by the steel area. For a rectangular tension zone, this gives:

$$\epsilon_m = \epsilon_1 - \frac{b_t (h - x)(a' - x)}{3E_s A_s (d - x)}$$ equation 13

In equation 13 for cases where the whole section is in tension, an effective value of $(h - x)$ can be estimated by interpolation between the following limiting conditions:

a) where the neutral axis is at the most compressed face, $(h - x) = h$ (i.e. $x = 0$);

b) for axial tension, $(h - x) = 2h$.

A negative value for ϵ_m indicates that the section is uncracked.

In assessing the strains, the modulus of elasticity of the concrete should be taken as half the instantaneous values.

Where it is expected that the concrete may be subject to abnormally high shrinkage (> 0.0006), ϵ_m should be increased by adding 50 % of the expected shrinkage strain; otherwise, shrinkage may be ignored.

NOTE This approach makes a notional allowance for long-term effects.

Table 3.2 — Estimated limiting temperature changes to avoid cracking

Aggregate type	Thermal expansion coefficient	Tensile strain capacity (10^{-6})	Limiting temperature drop for varying restraint factor (R)				Limiting temperature differential when $R = 0.36$)
			1.00	0.75	0.50	0.25	
	(10^{-6}/ °C)		°C	°C	°C	°C	°C
Gravel	12.0	70	7.3	9.7	14.6	29.2	20.0
Granite	10.0	80	10.0	13.3	20.0	40.0	27.7
Limestone	8.0	90	14.1	18.8	28.2	56.3	39.0
Sintered p.f.a.	7.0	110	19.6	26.2	39.2	78.4	54.6

3.8.4 Early thermal cracking

3.8.4.1 *General.* In pours that are subjected to either internal or external restraint, thermal stresses may develop which can cause cracking. Cracking can occur through two different mechanisms.

a) *Internal temperature gradients.* Cracking due to differential temperature changes is most common in massive pours. Since the low thermal conductivity of concrete prevents rapid heat dissipation, the temperature in the mass of concrete increases. The concrete surface, in direct contact with the environment, loses heat more quickly and therefore undergoes a much lower rise in temperature. The resulting expansion of the hot core, if excessive, can stretch the cooler surface zone to the extent that cracking occurs. During subsequent cooling, the opposite effect may occur causing internal cracking of the central zone.

b) *External restraint during cooling*. Cracking resulting from restraint to thermal movement most commonly occurs in walls cast into rigid bases as described in BS 5337. During the temperature rise period, the concrete has a relatively low elastic modulus and the compressive stresses due to restrained expansion are easily relieved by creep. During cooling, the concrete matures and, when the thermal contraction is restrained, the tensile stresses generated are less easily relieved. These can be of sufficient magnitude to cause cracking which commonly occurs at the half or one-third points along a bay. In the extreme case of a fully restrained element, a change in temperature of the order of only 10 °C can result in cracking (see Table 3.2). Therefore, the high temperature rises which can result in long-term strength reductions are not essential to the promotion of cracks. However, if there was no restraint, the concrete would contract without cracking.

Typical values of restraint recorded for a range of pour configurations have been given in Table 3.3. For most situations there is always some degree of restraint but complete restraint is very rare. Even when a wall is cast on to a nominally rigid foundation, the restraint is unlikely to exceed a value of R equal to 0.70. To minimize restraint, infill bays should be avoided wherever possible and the pour provided with a free end to accommodate thermal movement.

The maximum acceptable temperature reductions given in Table 3.2 apply to pours that are subjected to a well defined form of thermal restraint. In practice, however, restraints result in differential thermal strains which depend on the nature of the temperature distribution and the ratio of the "hot" and "cold" areas. Experience has shown that by limiting temperature differentials to 20 °C in gravel aggregate concrete, cracking can be avoided. This represents an equivalent restraint factor R of 0.36 and the corresponding values for concrete with other aggregate types are given in Table 3.2.

3.8.4.2 *Estimating early thermal crack widths.* The restrained component of the thermal strain ϵ_r which will be accommodated by cracks is given by the following equation:

$$\epsilon_r = 0.8\, \Delta_t \alpha R \qquad \text{equation 14}$$

Crack widths may be estimated by substituting ϵ_r for ϵ_m in equation 12 (see **3.8.3**).

Table 3.3 — Values of external restraint recorded in various structures

Pour configuration	Restraint factor (R)
Thin wall cast on to massive concrete base	0.6 to 0.8 at base
	0.1 to 0.2 at top
Massive pour cast into blinding	0.1 to 0.2
Massive pour cast on to existing mass concrete	0.3 to 0.4 at base
	0.1 to 0.2 at top
Suspended slabs	0.2 to 0.4
Infill bays, i.e. rigid restraint	0.8 to 1.0

blank

Section 4. Fire resistance

4.1 General

4.1.1 Methods

Throughout this section the fire resistance of an element of structure or combination of elements is to be determined from one of the following three methods.

a) *Method 1. Tabulated data:* information and tables as approved for general use by the Building Research Establishment and published in "Guidelines for the construction of fire resisting structural elements"[2].

b) *Method 2. Fire test:* direct application of the results of a fire resistance test on an element of structure.

c) *Method 3. Fire engineering calculations:* a basis for calculating the fire resistance of a structural element.

NOTE This method is not applicable to columns or walls.

4.1.2 Elements

The fire resistance of a structural element is expressed in terms of time as determined in accordance with BS 476-8:1972, in which the element is exposed to heating which is controlled to follow a standard temperature/time curve.

NOTE The relationship between the effects of a real fire and of a standard fire on the element is outside the scope of this standard.

4.1.3 Whole structures

The fire resistance of a whole concrete structure would not necessarily be that ascribed to its individual elements. Better fire behaviour could arise from such factors as robustness, adequate continuity of reinforcement, reduced level of loading, composite constructions and availability of alternative paths for load support. With precast structures or in-situ structures of slender proportions, therefore, it is necessary to pay particular attention to the detailing.

4.1.4 Surfaces exposed to fire

The surfaces exposed to fire in the standard test of the element are as follows:

walls:	one side
floor:	soffit
beams:	sides and soffit
columns:	all sides (fully exposed) or one or more sides (protected by adjacent walls)

There are circumstances in practice where a wall may be heated on both sides when there is a fire spread from room to room, or for external walls, flame projection from windows. This effect is likely to be important only where the wall is load-bearing and is not designed as a barrier to fire spread. Similar considerations may apply to floors.

4.1.5 Factors affecting fire resistance

In each of the three methods the factors that influence the fire resistance of concrete elements are as follows:

a) size and shape of elements;

b) disposition and properties of reinforcement or tendon;

c) the load supported;

d) the type of concrete and aggregate;

e) protective concrete cover provided to reinforcement or tendons;

f) conditions of end support.

Method 3 allows interaction between these factors to be taken into account.

[2] Available from The Building Research Station, Garston, Watford, Herts WD2 7JR.

4.1.6 Spalling of concrete at elevated temperatures

Rapid rates of heating, large compressive stresses or high moisture contents (over 5 % by volume or 2 % to 3 % by mass of dense concrete) can lead to spalling of concrete cover at elevated temperatures, particularly for thicknesses exceeding 40 mm to 50 mm. Such spalling may impair performance by exposing the reinforcement or tendons to the fire or by reducing the cross-sectional area of concrete. Concretes made from limestone aggregates are less susceptible to spalling than concretes made from aggregates containing a higher proportion of silica, e.g. flint, quartzites and granites. Concrete made from manufactured lightweight aggregates rarely spalls.

It may be possible to show that a particular form of construction has given the required performance in a fire resistance test without any measures to avoid spalling. Alternatively, the designer may be able to demonstrate by fire engineering principles that the particular performance can be provided, even with spalling of concrete cover to the main tensile reinforcement.

4.1.7 Protection against spalling

In any method of determining fire resistance where loss of cover can endanger the structural element, measures should be taken to avoid its occurrence. Acceptable measures are:

a) an applied finish by hand or spray of plaster, vermiculite, etc.;

b) the provision of a false ceiling as a fire barrier;

c) the use of lightweight aggregates;

d) the use of sacrificial tensile steel.

NOTE An applied finish or false ceiling may increase the fire resistance of an element as described in **4.2.4**.

Welded steel fabric as supplementary reinforcement is sometimes used to prevent spalling; it is then placed within the cover at 20 mm from the concrete face. There are practical difficulties in keeping the fabric in place and in compacting the concrete; in certain circumstances there would also be a conflict with the durability recommendations of this standard.

4.1.8 Detailing

The detailing of the structure for any of the three methods of design should be such as to implement the design assumptions for the changes during a fire in the distribution of load and the characteristic strengths of the materials. In particular, the reinforcement detailing should reflect the changing pattern of the structural action and ensure that both individual elements and the structure as a whole contain adequate supports, ties, bonds and anchorages for the required fire resistance.

4.2 Factors to be considered in determining fire resistance

4.2.1 General

The factors given in **4.2.2**, **4.2.3**, **4.2.4**, **4.2.5**, **4.2.6**, **4.2.7**, **4.2.8**, **4.2.9** and **4.2.10** should be considered for the determination of the fire resistance of any element by any method.

4.2.2 Aggregates

Table 4.1, Table 4.2, Table 4.3, Table 4.4, Table 4.5 and Table 4.6 in method 1 refer to two types of concrete:

a)	dense concrete:	calcareous aggregates and aggregates siliceous in character, e.g. flints, quartzites and granites;
b)	lightweight concrete: ($\leqslant$ 2 000 kg/m^3)	aggregates made from sintered p.f.a., expanded clays and shales, etc.

In general, calcareous aggregates, i.e. limestone, give superior performance in fire compared with siliceous aggregates. However, insufficient data are available to provide comprehensive tables, except for columns. Therefore, where calcareous aggregates are used in method 1, the dimensions used should be those for dense concrete.

4.2.3 Cover to main reinforcement

Cover has to provide lasting protection to the reinforcement from both fire and environmental attack. Choice of thickness should be on the basis of the more onerous. In this section "cover" is the distance between the nearest heated face of the concrete and the surface of the main reinforcement or an average value determined as shown below.

NOTE 1 This definition differs from that of "nominal cover" used in BS 8110-1; for practical purposes cover is stated as nominal cover to all steel reinforcement.

a) *Floor slabs.* Cover is the average distance from the soffit or the heated face. With one-way spanning single layer reinforcement the actual distance is used, i.e. C_1. With two-way spanning floor slabs the average distance is calculated taking into account reinforcement in both directions as multi-layer reinforcement. With one-way spanning floor slabs only multi-layer reinforcement in the same direction should be used to determine the average distance. The average distance C_{ave} is calculated as follows:

$$C_{ave} = \frac{A_1C_1 + A_2C_2 + A_3C_3 + \dots A_nC_n}{A_1 + A_2 + A_3 + \dots A_n} = \frac{\Sigma AC}{\Sigma A} \qquad \text{equation 15}$$

where

A is the area of tensile reinforcement/tendons;

C is the distance between the nearest exposed surface and the main reinforcement.

b) *Rectangular beams.* The effective cover C_{ave} for the assembly of main reinforcement is determined as in a). Examples of calculation of average cover are given in Figure 4.1.

NOTE 2 *Method 3.* Where C_1 (floor slabs) or C_1 or C_3 to individual corner bars (rectangular beams) is less than half C_{ave} then that reinforcement should be disregarded in the calculation of the ultimate resistance at high temperature.

c) *I-section beams.* The effective cover $C_{ave,}$ after determination as in b) is adjusted by multiplying it by 0.6 to allow for the additional heat transfer through the upper flange face.

4.2.4 Additional protection

Where plaster, except Gypsum, or sprayed fibre is used as an applied finish to other elements, it may be assumed that the thermal insulation provided is at least equivalent to the same thickness of concrete. Such finishes can therefore be used to remedy deficiencies in cover thickness. For selected materials and, subject to riders existent in BRE Guidelines, the following guidance can be given with respect to the allowance of the use of additional protection not exceeding 25 mm in thickness as a means of providing effective cover to steel reinforcing or prestressing elements. In each case the equivalent thickness of concrete may be replaced by the named protection.

Mortar Gypsum plaster	≡ 0.6 × concrete thickness
Lightweight plaster Sprayed lightweight insulation	≡ 1.0 × concrete thickness up to 2 h 2.0 × concrete thickness > 2 h
Vermiculite slabs	≡ 1.0 × concrete thickness up to 2 h ≡ 1.5 × concrete thickness > 2 h

4.2.5 Floor thickness

For all methods the thickness of floors is governed by the dimensions of slabs. In the case of solid slabs the thickness to consider is the actual thickness of the slab plus any non-combustible finish on top. With hollow slabs (or beams with filler blocks) the effective thickness t_e should be obtained by considering the total solid per unit width as follows:

$$t_e = h \times \sqrt{\xi} + t_f \qquad \text{equation 16}$$

where

h is the actual thickness of slab;
ξ is the proportion of solid material per unit width of slab;
t_f is the thickness of non-combustible finish.

For ribbed slabs the thickness may include any non-combustible finish on top.

4.2.6 Width of beams

For all beams, the width for the purpose of satisfying tabular data is the width determined at the level of the lowest reinforcement. For I-section beams the web thickness b_w of fully exposed I-section beams should be not less than 0.5 of the minimum width stated in the table for beams for various fire resistance periods.

4.2.7 Distinction between ribs and beams

Where failure of a rib does not critically affect the stability and integrity of a floor, the rib spacing is at the choice of the designer; otherwise ribs should be spaced at a maximum of 1.5 m centres or be treated as beams.

4.2.8 Beams and floors

Table 4.3 to Table 4.5 relating to beams and floors give minimum dimensions for widths, thicknesses and covers. Examples of such constructions are shown in Figure 4.2.

4.2.9 Columns

Table 4.2 relating to reinforced concrete columns gives minimum dimensions for width and actual cover (i.e. not C_{ave}). Examples are shown in Figure 4.3.

4.3 Tabulated data (method 1)

4.3.1 Method by design from BRE guidelines

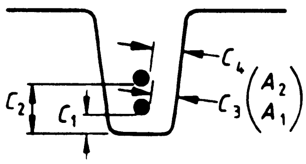
This method employs information and tabular data contained in a Building Research Establishment Report published by the Department of the Environment [4] and also takes into account international test data given in Table 4.2, Table 4.3, Table 4.4, Table 4.5 and Table 4.6 reproduce BRE tabular data but are updated by information received between the publication dates of the BRE report 1980 and this code. The method may be used when no relevant test result is available from a laboratory that has carried out a test in accordance with BS 476-8:1972.

4.3.2 Support conditions: simply supported and continuous

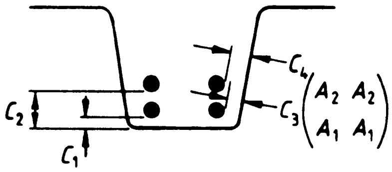
The data set out in the following tables distinguishes between simply supported and continuous constructions for flexural members, i.e. beams and slabs for both reinforced concrete and prestressed concrete. In practice the majority of constructions will be continuous and benefits can be derived from the permissible reductions in cover and other dimensions, where the designer has made provision for fixity in the resistance to normal loads by the provision of reinforcement properly detailed and adequately tied to adjacent members. In the case of precast construction or a mixture of precast and in situ construction, it will be necessary for adequate provision to be made for continuity and restraint to end rotation.

4.3.3 Use of tabular data

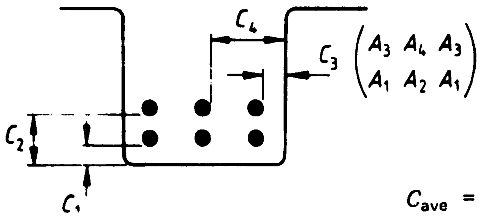
All tabular data should be read in conjunction with **4.2**. The tables are based on the assumption that the elements considered are supporting the full design load.



$$C_{ave} = \frac{A_1 \times \begin{pmatrix}\text{lesser of}\\ C_1 \text{ or } C_3\end{pmatrix} + A_2 \times \begin{pmatrix}\text{lesser of}\\ C_2 \text{ or } C_4\end{pmatrix}}{A_1 + A_2}$$



$$C_{ave} = \frac{2A_1 \times \begin{pmatrix}\text{lesser of}\\ C_1 \text{ or } C_3\end{pmatrix} + 2A_2 \times \begin{pmatrix}\text{lesser of}\\ C_2 \text{ or } C_4\end{pmatrix}}{2A_1 + 2A_2}$$



$$C_{ave} = \frac{A_2 C_1 + 2A_1 \begin{pmatrix}\text{lesser of}\\ C_1 \text{ or } C_3\end{pmatrix} + 2A_3 \begin{pmatrix}\text{lesser of}\\ C_2 \text{ or } C_3\end{pmatrix} + A_4 \begin{pmatrix}\text{lesser of}\\ C_2 \text{ or } C_4\end{pmatrix}}{2\,(A_1 + A_3) + A_2 + A_4}$$

Figure 4.1 — Calculation of average cover

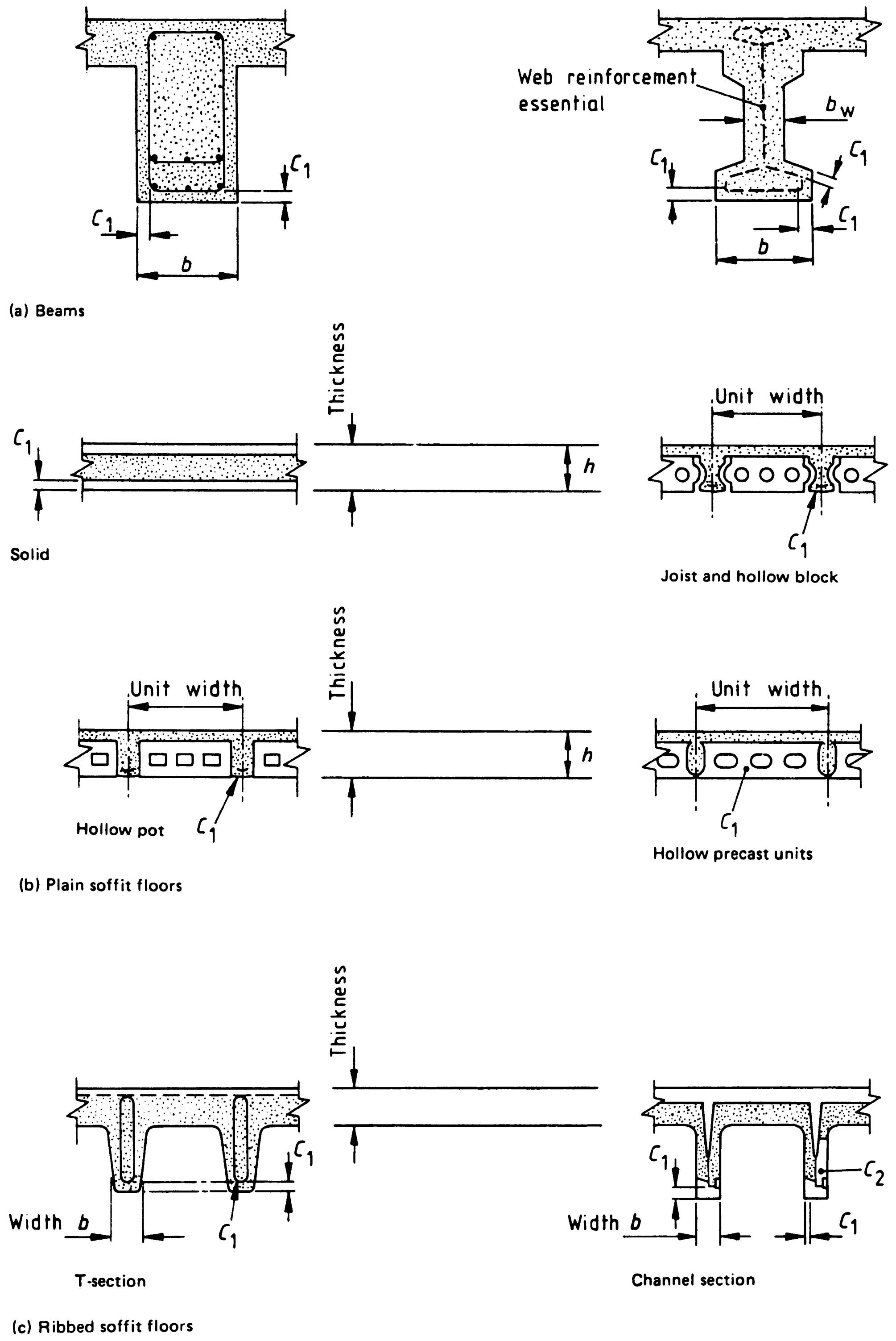
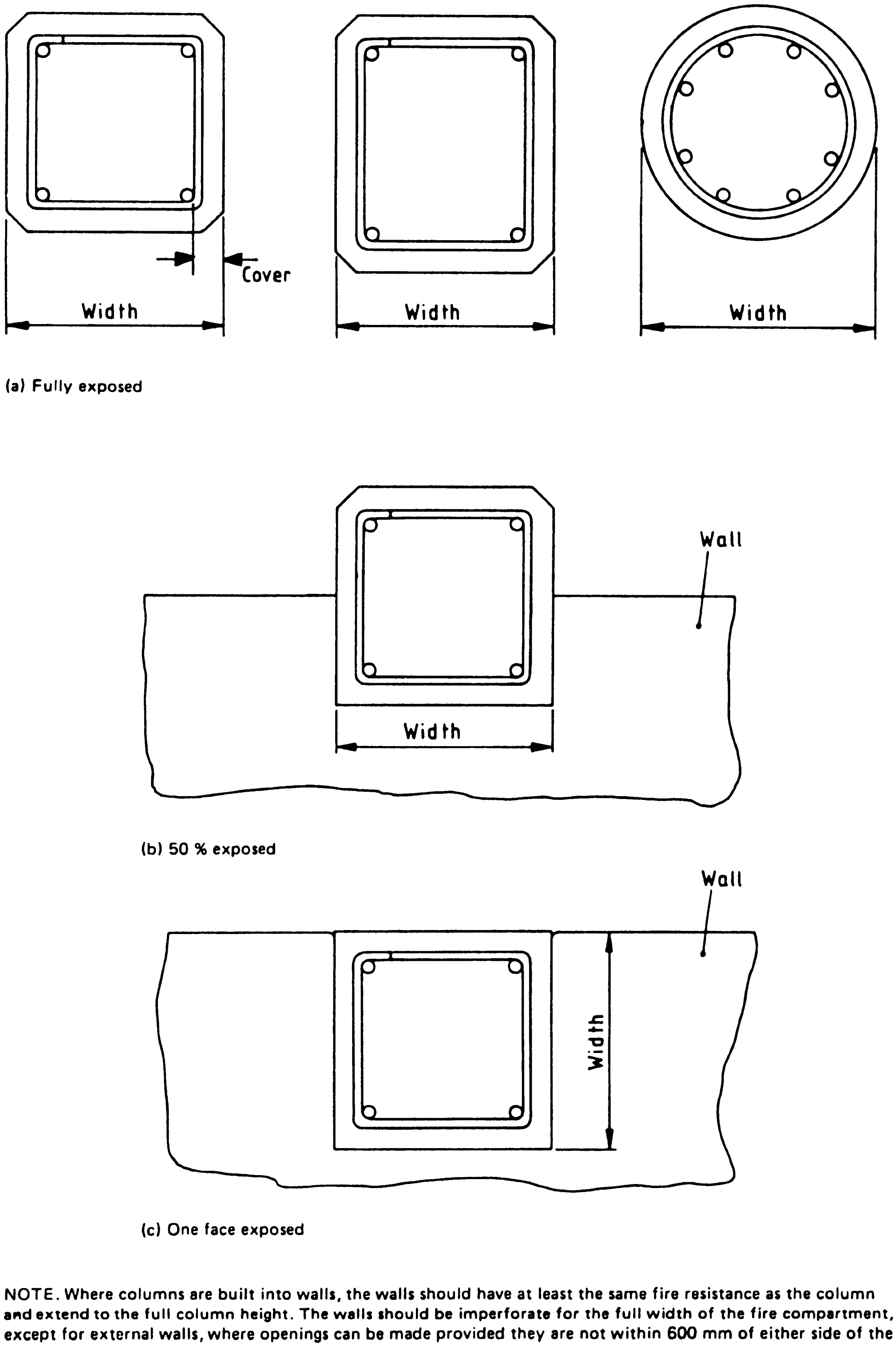


Figure 4.2 — Typical examples of beams, plain soffit floors and ribbed soffit floors



NOTE. Where columns are built into walls, the walls should have at least the same fire resistance as the column and extend to the full column height. The walls should be imperforate for the full width of the fire compartment, except for external walls, where openings can be made provided they are not within 600 mm of either side of the columns.

Figure 4.3 — Typical examples of reinforced concrete columns

4.3.4 Spalling of nominal cover

If the nominal cover, i.e. the cover to the outermost steel exceeds 40 mm for dense or 50 mm for lightweight aggregate concrete, there is a danger of concrete spalling. Where spalling could endanger the structural element, measures should be taken to avoid its occurrence (see **4.1.7**).

4.3.5 Variation of cover to main reinforcement with member width

The values in Table 4.2, Table 4.3, Table 4.4, Table 4.5 and Table 4.6 relate cover to main reinforcement to minimum member sizes. These minima may be adjusted by applying the corrections given in Table 4.1; however, cover should in no case be less than required in the case of plain soffit floors of the same fire resistance.

Table 4.1 — Variation of cover to main reinforcement with member width

Minimum increase in width	Decrease in cover	
	Dense concrete	Lightweight concrete
mm	mm	mm
25	5	5
50	10	10
100	15	15
150	15	20
NOTE This table has been used to produce Table 3.4 of BS 8110-1:1997.		

Where a member is wider than the tabulated minimum, some decrease in the cover to main reinforcement may be appropriate. Decreases should be made with caution in the light of the principles of fire safety design and should not exceed the values in Table 4.1. In no case should the resulting cover be less than the values required for plain soffit floors of the same fire resistance.

4.3.6 Reinforcement

For method 1, the tabulated data for simply-supported elements are based on the steel reinforcement retaining a proportion of its strength at high temperatures; the data are based on the reinforcing bars and prestressing tendons retaining 50 % of their ambient strength at 550 °C and 450 °C, respectively. For steels with other strength characteristics, an appropriate adjustment in cover will be needed (see also Figure 4.5).

4.4 Fire test (method 2)

Any form of concrete element covered by a valid fire test report may be deemed to have the fire resistance ascribed to it by such a test provided that the element has similar details of constructions, stress level and support as the test specimen.

4.5 Fire engineering calculations (method 3)

NOTE This is not applicable to columns and walls.

4.5.1 General

This method is a calculation method based on design from first principles for structural elements acting in flexure, i.e. beams and slabs. The calculations are based on the preceding methods and authenticated published data incorporating one or more of the following:

a) direct extrapolation or interpolation of the test data;

b) estimation of reinforcement temperature with different arrangements of cover/aggregate;

c) effect of alteration in test load, boundary and support conditions or material properties.

When further research information is available, it should be possible to determine the fire behaviour of whole concrete structures by analytical methods.

4.5.2 Principles of design

The principles employed in the calculation of the fire resistance of structural concrete elements as opposed to acceptance of tabulated data in method 1 is based on the work of recent international research on the insulating properties of concrete, strengths of concrete and steel reinforcement/tendons at high temperatures and consideration of such effects as spalling, disposition of reinforcement and the nature of load redistribution with consequent alterations in forces during a fire.

The use of calculations to determine fire resistance will normally permit a reduction in concrete volume and reinforcement cover, compared to those values given in Table 4.1, Table 4.2, Table 4.3, Table 4.4, Table 4.5 and Table 4.6. This is achieved, in safety, by the better disposition of reinforcement to suit conditions caused by a fire. Guidance on the design of structural concrete members by calculation methods is given in specialist reports [5].

4.5.3 Application to structural elements

The design approach in this method relates to structural elements in flexure, e.g. beams and floors, where failure of the element in a fire is governed by the yielding of the main tensile reinforcement.

It is not yet possible to formulate recommendations for columns and walls. Consequently design of these compression elements should be based on tabulated data (see method 1) or experience from fire tests (see method 2) with emphasis on good detailing.

4.5.4 Material properties for design

The behaviour of flexural elements in fire is largely determined by the strength at elevated temperatures of the concrete in compression and the reinforcement in tension.

4.5.5 Design curve for concrete

Figure 4.4 provides design curves for the reduction in strength of concrete at elevated temperatures.

Table 4.2 — Reinforced concrete columns

Nature of construction and materials		Minimum dimensions excluding any combustible finish for a fire resistance of:					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
		mm	mm	mm	mm	mm	mm
Fully exposed:							
dense concrete	Width	150	200	250	300	400	450
	Cover[a]	20	25	30	35	35	35
lightweight concrete	Width	150	160	200	240	320	360
	Cover[a]	20	20	25	35	35	35
50 % exposed:							
dense concrete	Width	125	160	200	240	300	350
	Cover[a]	20	25	25	25	30	35
lightweight concrete	Width	125	130	160	185	250	275
	Cover[a]	20	20	25	25	30	30
One face exposed:							
dense concrete	Thickness	100	120	140	160	200	240
	Cover[a]	20	25	25	25	25	25
lightweight concrete	Thickness	100	100	115	130	160	190
	Cover[a]	10	20	20	25	25	25

[a] Cover is expressed here as cover to main reinforcement (see **4.2.3**). For practical purposes cover is expressed as nominal cover to all reinforcement and these tabulated values need to be decreased accordingly.

Table 4.3 — Concrete beams

Nature of construction and materials		Minimum dimensions excluding any combustible finish for a fire resistance of:					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
		mm	mm	mm	mm	mm	mm
Reinforced concrete (simply supported):							
dense concrete	Width	80	120	150	200	240	280
	Cover[a]	20	30	40	50	70	80
lightweight concrete	Width	80	100	130	160	200	250
	Cover[a]	15	20	35	45	55	65
Reinforced concrete (continuous):							
dense concrete	Width	80	80	120	150	200	240
	Cover[a]	20	20	35	50	60	70
lightweight concrete	Width	60	80	90	110	150	200
	Cover[a]	15	20	25	35	45	55
Prestressed concrete (simply supported):							
dense concrete	Width	100	120	150	200	240	280
	Cover[a]	25	40	55	70	80	90
lightweight concrete	Width	80	110	130	160	200	250
	Cover[a]	25	30	45	55	65	75
Prestressed concrete (continuous):							
dense concrete	Width	80	100	120	150	200	240
	Cover[a]	20	30	40	55	70	80
lightweight concrete	Width	80	90	100	125	150	200
	Cover[a]	20	25	35	45	55	65

[a] Cover is expressed here as cover to main reinforcement (see **4.2.3**). For practical purposes cover is expressed as nominal cover to all reinforcement and these tabulated values need to be decreased accordingly.

Table 4.4 — Plain soffit concrete floors

Nature of construction and materials		Minimum dimensions excluding any combustible finish for a fire resistance of:					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
		mm	mm	mm	mm	mm	mm
Reinforced concrete (simply supported):							
dense concrete	Thickness	75	95	110	125	150	170
	Cover[a]	15	20	25	35	45	55
lightweight concrete	Thickness	70	90	105	115	135	150
	Cover[a]	15	15	20	25	35	45
Reinforced concrete (continuous):							
dense concrete	Thickness	75	95	110	125	150	170
	Cover[a]	15	20	20	25	35	45
lightweight concrete	Thickness	70	90	105	115	135	150
	Cover[a]	15	15	20	20	25	35
Prestressed concrete (simply supported):							
dense concrete	Thickness	75	95	110	125	150	170
	Cover[a]	20	25	30	40	55	65
lightweight concrete	Thickness	70	90	105	115	135	150
	Cover[a]	20	20	30	35	45	60
Prestressed concrete (continuous):							
dense concrete	Thickness	75	95	110	125	150	170
	Cover[a]	20	20	25	35	45	55
lightweight concrete	Thickness	70	90	105	115	135	150
	Cover[a]	20	20	25	30	35	45

[a] Cover is expressed here as cover to main reinforcement (see **4.2.3**). For practical purposes cover is expressed as nominal cover to all reinforcement and these tabulated values need to be decreased accordingly.

No allowance is made for the beneficial effect of applied load which can inhibit cracking and thus reduce the loss in strength on heating.

4.5.6 Design curve for steel

Figure 4.5 provides design curves for the reduction in the strength of reinforcement and prestressing steels. These curves are a simplification of experimental results and are based on 50 % of the strength of the steel at 20 °C being retained by:

a) reinforcement steels at 550 °C;

b) prestressing tendons at 400 °C.

Separate consideration should be given to extra high tensile steels and steels not in accordance with British Standards.

4.5.7 Design

Design may be carried out by analysing the structure under conditions of fire exposure having due regard to the reduction in strength of the structural materials. The following partial safety factors are recommended and any recognized methods of analysis may be adopted.

For materials (γ_m):	concrete	1.3
	steel	1.0
For loads (γ_f):	dead loads	1.05
	imposed loads	1.00

Methods of design for fire resistance in cases where failure is governed simply by the limit of flexural strength are given in **4.5**. For other methods of failure, reference should be made to the Institution of Structural Engineers/Concrete Society report [5].

Table 4.5 — Ribbed open soffit concrete floors

Nature of construction and materials		Minimum dimensions excluding any combustible finish for a fire resistance of:					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
		mm	mm	mm	mm	mm	mm
Reinforced concrete (simply supported):							
dense concrete	Thickness	70	90	105	115	135	150
	Width	75	90	110	125	150	175
	Cover[a]	15	25	35	45	55	65
lightweight concrete	Thickness	70	85	95	100	115	130
	Width	60	75	85	100	125	150
	Cover[a]	15	25	30	35	45	55
Reinforced concrete (continuous):							
dense concrete	Thickness	70	90	105	115	135	150
	Width	75	80	90	110	125	150
	Cover[a]	15	20	25	35	45	55
lightweight concrete	Thickness	70	85	95	100	115	130
	Width	70	75	80	90	100	125
	Cover[a]	15	20	25	30	35	45
Prestressed concrete (simply supported):							
dense concrete	Thickness	70	90	105	115	135	150
	Width	80	110	135	150	175	200
	Cover[a]	25	35	45	55	65	75
lightweight concrete	Thickness	70	85	95	100	115	130
	Width	75	90	110	125	150	175
	Cover[a]	20	30	35	45	55	65
Prestressed concrete (continuous):							
dense concrete	Thickness	70	90	105	115	135	150
	Width	70	75	110	125	150	175
	Cover[a]	20	25	35	45	55	65
lightweight concrete	Thickness	70	85	95	100	115	130
	Width	70	75	90	110	125	150
	Cover[a]	20	25	30	35	45	55

[a] Cover is expressed here as cover to main reinforcement (see **4.2.3**). For practical purposes cover is expressed as nominal cover to all reinforcement and these tabulated values need to be decreased accordingly.

Table 4.6 — Concrete walls with vertical reinforcement

Nature of construction and materials		Minimum dimensions excluding any combustible finish for a fire resistance of:					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
		mm	mm	mm	mm	mm	mm
Walls with less than 0.4 % reinforcement made from dense aggregate	Thickness	150	150	175	—	—	—
Walls with 0.4 % to 1.0 % reinforcement made from dense aggregate (concrete density up to 2.4 t/m^3)	Thickness	100	120	140	160	200	240
	Cover[a]	25	25	25	25	25	25
Walls made from lightweight aggregate (concrete density 1.2 t/m^3)[b]	Thickness	100	100	115	130	160	190
	Cover[a]	10	20	20	25	25	25
Walls with over 1.0 % reinforcement made from dense aggregate	Thickness	(See note)	(See note)	100	100	150	180
	Cover[a]	15	15	25	25	25	25

[a] Cover is expressed here as cover to main reinforcement (see **4.2.3**). For practical purposes cover is expressed as nominal cover to all reinforcement and these tabulated values need to be decreased accordingly.
[b] For concrete of densities between 1.2 t/m^3 and 2.4 t/m^3 the value of wall thickness may be interpolated.
NOTE Use the minimum practical dimension but not less than 75 mm.

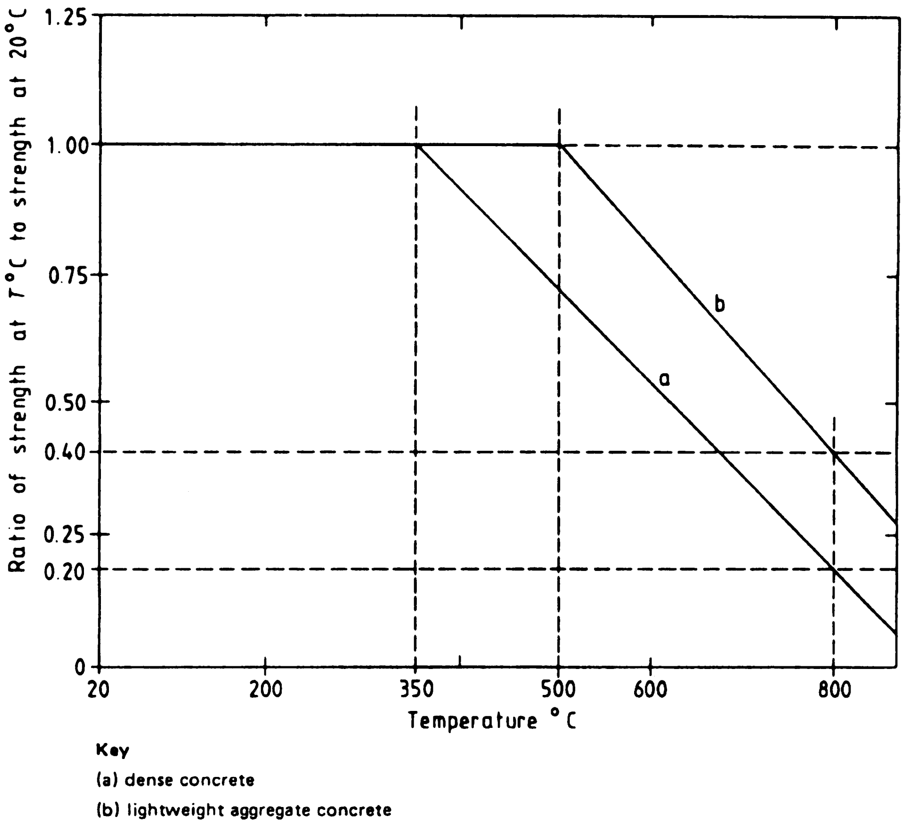
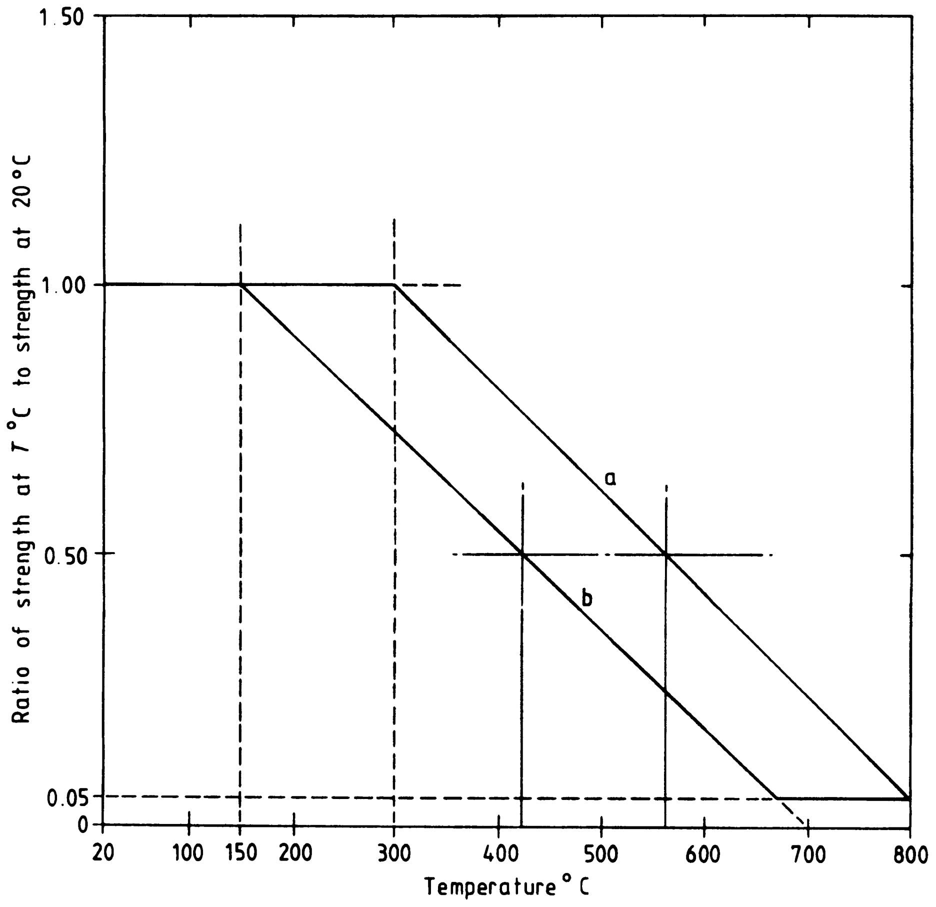


Figure 4.4 — Design curves for variation of concrete strength with temperature



Key

(a) grade 460 and grade 250 reinforcement and extra high-strength steel bars

(b) prestressing wires or strands

Figure 4.5 — Design curves for variation of steel strength or yield stress with temperature

Section 5. Additional considerations in the use of lightweight aggregate concrete

5.1 General

5.1.1 Introduction

Lightweight aggregate concrete may generally be designed in accordance with section 2 and **3.1** of BS 8110-1:1997. These clauses relate specifically to reinforced lightweight aggregate concrete of grade 15 or above. The structural use of concretes below grade 20 should be limited to plain walls (see **3.9.4** of BS 8110-1:1997).

In considering lightweight aggregate concrete, the properties for any particular type of aggregate can be established far more accurately than for most naturally occurring materials and the engineer should therefore obtain specific data direct from the aggregate producer in preference to using tabulated values taken from British Standard codes of practice or specifications.

NOTE Further guidance on the use of lightweight aggregate concrete is given in [11].

5.1.2 Symbols

For the purposes of section 5 the following symbols apply.

a_b	for a given bar (or group of bars in contact) is the centre-to-centre distance between bars (or groups of bars) perpendicular to the plane of the bend; for a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus ϕ
A_s	area of reinforcement
b	width of section
d	effective depth
e_a	additional eccentricity due to deflection
h	depth of the cross section measured in the plane under consideration (or, more particularly, in respect of the major axis if the sense makes this apparent)
l_e	effective height of a column in the plane of bending considered
l_{ex}	effective height in respect of the major axis
l_{ey}	effective height in respect of the minor axis
v	design shear stress at a cross section
v_c	design concrete shear stress
v_t	torsional shear stress
$v_{t,min}$	minimum torsional shear stress above which reinforcement is required
ϕ	size of the bar (or, for a group, the size of a bar of equivalent area)

5.2 Cover for durability and fire resistance

In general, the rules given in **3.3** of BS 8110-1:1997 apply except that Table 5.1 and Table 5.2 of this standard replace Table 3.4 and Table 3.5 respectively.

The estimated free water/cement ratio is based on the calculated free water in excess of that estimated to bring the aggregate to a nominally saturated surface-dry condition at the time of mixing.

Table 5.1 — Nominal cover to all reinforcement (including links) to meet durability requirements

Conditions of exposure (see **3.3.4** of BS 8110-1:1997)	**Nominal cover**				
	mm	mm	mm	mm	mm
Mild	25	20	20[a]	20[a]	20[a]
Moderate	—	45	40	35	30
Severe	—	—	50	40	35
Very severe	—	—	60[b]	50[b]	40
Extreme	—	—	—	70[b]	60
Maximum estimated free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content, kg/m^3	300	325	350	375	425
Minimum grade	20	25	30	35	40

[a] These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm.
[b] Where concrete is subject to freezing whilst wet, air entrainment should be used.

Table 5.2 — Nominal cover to all steel to meet specified periods of fire resistance (lightweight aggregate concrete)

Fire resistance	**Nominal cover**						
	Beams[a]		**Floors**		**Floors**		**Columns**[a]
			Plain soffit		**Ribbed soffit**		
	Simply supported	**Continuous**	**Simply supported**	**Continuous**	**Simply supported**	**Continuous**	
h	mm	mm	mm	mm	mm	mm	mm
0.5	15	15	15	15	15	15	15
1	15	15	15	15	15	15	15
1.5	15	15	15	15	15	15	15
2	30	15	25	20	30	25	25
3	45	25	35	25	45	30	25
4	55	35	45	35	55	45	25

[a] 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 have 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** of BS 8110-1:1997).

NOTE 1 The nominal covers given relate specifically to the minimum member dimensions given in Figure 3.2 of BS 8110-1:1997. Guidance on increased covers necessary if smaller members are used is given in section 4.

NOTE 2 Cases that lie below the bold line require attention to the additional measures necessary to reduce the risk of spalling (see section 4).

5.3 Characteristic strength of concrete

The characteristic strength of lightweight aggregate concrete should be selected from the preferred grades in BS 5328. Grades below 20 should not be used for reinforced concrete.

5.4 Shear resistance

The shear resistance and shear reinforcement requirements for lightweight aggregate concrete members should be established in accordance with BS 8110-1 except that for concrete grades of 25 or more, the design concrete shear stress v_c should be taken as 0.8 times the values given in Table 3.8 of BS 8110-1:1997. For grade 20 concrete, the values given in Table 5.3 should be used. In no case should the shear stress v exceed the lesser of 0.63 $\sqrt{f_{cu}}$ or 4 N/mm².

5.5 Torsional resistance of beams

The torsional resistance and reinforcement for lightweight aggregate concrete beams should be calculated in accordance with **2.4** except that the values given in Table 2.3 for $v_{t,min}$ and v_{tu} should be multiplied by 0.8.

Table 5.3 — Values of v_c, design shear stress for grade 20 lightweight concrete

$\frac{100 A_s}{bd}$	v_c
	N/mm²
0.15	0.25
0.25	0.30
0.50	0.37
0.75	0.43
1.00	0.47
1.50	0.53
2.00	0.59
$\geqslant$ 3.00	0.68

5.6 Deflections

The deflection of lightweight aggregate members may be checked by direct calculation using the methods given in section 3. Where this is done, appropriate values of modulus elasticity, free shrinkage and creep coefficient should be obtained for concrete made with the aggregate in question. Alternatively, for normal structures, members may be checked using the span/effective depth ratios given in **3.4.6.3** of BS 8110-1:1997 except that for all beams and for slabs where the characteristic imposed load exceeds 4 kN/m², the limiting span/effective depth ratio should be multiplied by 0.85.

5.7 Columns

5.7.1 General

The recommendations of **3.8** of BS 8110-1:1997 apply to lightweight aggregate concrete columns subject to the provisions of **5.7.2** and **5.7.3**.

5.7.2 Short and slender columns

A column in reinforced lightweight aggregate concrete may be considered as short when the ratios l_{ex}/h and l_{ey}/b are less than 10; all others are slender.

5.7.3 Slender columns

In **3.8.3.1** of BS 8110-1:1997 the divisor 2000 in equation 34 should be replaced by the divisor 1200.

5.8 Walls

5.8.1 General

The recommendations of **3.9** of BS 8110-1:1997 apply to lightweight aggregate concrete walls, subject to the provisions of **5.8.2** and **5.8.3**.

5.8.2 Stocky and slender walls

A wall in lightweight aggregate concrete may be considered as stocky when l_e/h does not exceed 10; all others are slender.

5.8.3 Slender walls

Slender reinforced walls, when considered as slender columns, should be designed in accordance with **3.9.3.7** of BS 8110-1:1997, but the divisor 2000 in equation 34 should be replaced by the divisor 1200 (see **3.8.3.1** of BS 8110-1:1997). For plain slender walls, in the use of **3.9.4.16** and **3.9.4.17** of BS 8110-1:1997, the additional eccentricity due to deflection e_a used in equation 44, should be taken as $l_e/1700h$.

5.9 Anchorage bond and laps

Anchorage bond stress and lap lengths in reinforcement for lightweight aggregate concrete members should be established in accordance with **3.12.8** of BS 8110-1:1997 except that the bond stresses should not exceed 80 % of those calculated for normal-weight concrete.

For foamed slag or similar aggregates it may be necessary to ensure that bond stresses are kept well below the above maximum values for reinforcement which is in a horizontal position during casting, and the advice of the manufacturer should be obtained.

5.10 Bearing stress inside bends

The recommendations of **3.12.8.25** of BS 8110-1:1997 apply to lightweight aggregate concrete, except that the bearing stress should not exceed

$$\frac{4f_{cu}}{3\left(1+\frac{2\phi}{a_b}\right)}$$

Section 6. (*deleted*)

Section 7. Elastic deformation, creep, drying shrinkage and thermal strains of concrete

7.1 General

Reliable prediction of the deformation of structural concrete requires the assessment of elastic, creep, shrinkage and thermal strains. Creep, shrinkage and thermal strains are affected by time-dependent factors, such as stress, relative humidity and temperatures, thus a knowledge of environmental history is required for accurate predictions of deformation.

This section gives general guidance on the predictions of the different strain components. It should be recognized that, if an accurate prediction of any or all of these strains is considered to be an essential part of the design, this can only be obtained from tests carried out on concrete made with materials to be used in the structure. However, the information that follows gives a level of accuracy greater than that in BS 8110-1. It is intended for regular use in a design office and is considered satisfactory for the majority of structures, when movements and deformations are being considered.

7.2 Elastic deformation

The most important factor influencing the elastic modulus of concrete is the aggregate used. With a given aggregate, the elastic modulus increases with the characteristic strength of the concrete. The elastic modulus is also affected by the aggregate/cement ratio and the age of the concrete. However, variations due to any of these factors need not be taken into account in the design of normal structures in accordance with BS 8110-1. The mean values given in Table 7.2 are deemed to be sufficiently accurate in analysing the structure to determine force distributions where better data are unavailable. They may also be used for estimating loss of prestress (see **4.8** of BS 8110-1:1997).

Where, in special circumstances, an as-accurate-as-possible assessment of actual behaviour is required, it will be necessary to consider possible variations in the value for modulus of elasticity. Guidance on this follows but it is emphasized that the value chosen in any particular case should depend on the importance of the estimate and why it is needed.

The mean values for normal-weight concrete in Table 7.2 are derived from the following equation:

$$E_{c,28} = K_o + 0.2\, f_{cu,28} \qquad \text{equation 17}$$

where

$E_{c,28}$	is the static modulus of elasticity at 28 days;
$f_{cu,28}$	is the characteristic cube strength at 28 days (in N/mm²);
K_o	is a constant closely related to the modulus of elasticity of the aggregate (taken as 20 kN/mm² for normal-weight concrete).

The modulus of elasticity of concrete at an age t may be derived from the following equation:

$$E_{c,t} = E_{c,28}\,(0.4 + 0.6\, f_{cu,t}/f_{cu,28}) \qquad \text{equation 18}$$

where

$t \geqslant 3$ days.

Values of $f_{cu,t}/f_{cu,28}$ for use in equation 18 can be obtained from Table 7.1. This table shows that on average there is likely to be a gain of strength beyond 28 days, since this will lead to a more realistic assessment of the modulus of elasticity. It should be noted that there is a difference here from BS 8110-1 where no increase in strength beyond 28 days is permitted in satisfying limit state requirements. A smaller increase in strength will occur with small structural members that are exposed to a dry environment after initial curing.

Where calculations of deflection or deformation are to be made, the reliability of the estimate of the static modulus of elasticity will depend on the precision required from the calculation. Where deflections are of great importance, tests should be carried out on concrete made with the aggregate to be used in the structure. In other cases, experience with a particular aggregate, backed by general test data, will often provide a reliable value for Ko, and hence for $E_{c,28}$, but with unknown aggregates, it would be advisable at the design stage to consider a range of values for $E_{c,28}$, based on K_o = 14 kN/mm² to 26 kN/mm², as given in Table 7.2.

For lightweight aggregate concrete, the values of the static modulus in Table 7.2 should be multiplied by $(w/2400)^2$ where w is the density of lightweight aggregate concrete (in kg/m³).

Table 7.1 — Strength of concrete

Grade	Characteristic strength f_{cu}	Cube strength at an age of:				
		7 days	2 months	3 months	6 months	1 year
	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
20	20.0	13.5	22	23	24	25
25	25.0	16.5	27.5	29	30	31
30	30.0	20	33	35	36	37
40	40.0	28	44	45.5	47.5	50
50	50.0	36	54	55.5	57.5	60

Table 7.2 — Typical range for the static modulus of elasticity at 28 days of normal-weight concrete

$f_{cu,28}$	$E_{c,28}$	
	Mean value	Typical range
N/mm²	kN/mm²	kN/mm²
20	24	18 to 30
25	25	19 to 31
30	26	20 to 32
40	28	22 to 34
50	30	24 to 36
60	32	26 to 38

Where it is more convenient to use the dynamic modulus method of test to obtain an estimated value for the static modulus of elasticity of natural aggregate concrete, the following equation may be used:

$$E_c = 1.25\, E_{cq} - 19 \qquad \text{equation 19}$$

where

E_{cq} is the dynamic modulus of elasticity.

Such an estimate will generally be correct within ± 4 kN/mm².

7.3 Creep

The final (30 year) creep strain in concrete ϵ_{cc} can be predicted from

$$\epsilon_{cc} = \frac{\text{stress}}{E_t} \times \phi \qquad \text{equation 20}$$

where

E_t is the modulus of elasticity of the concrete at the age of loading t.

ϕ is the creep coefficient.

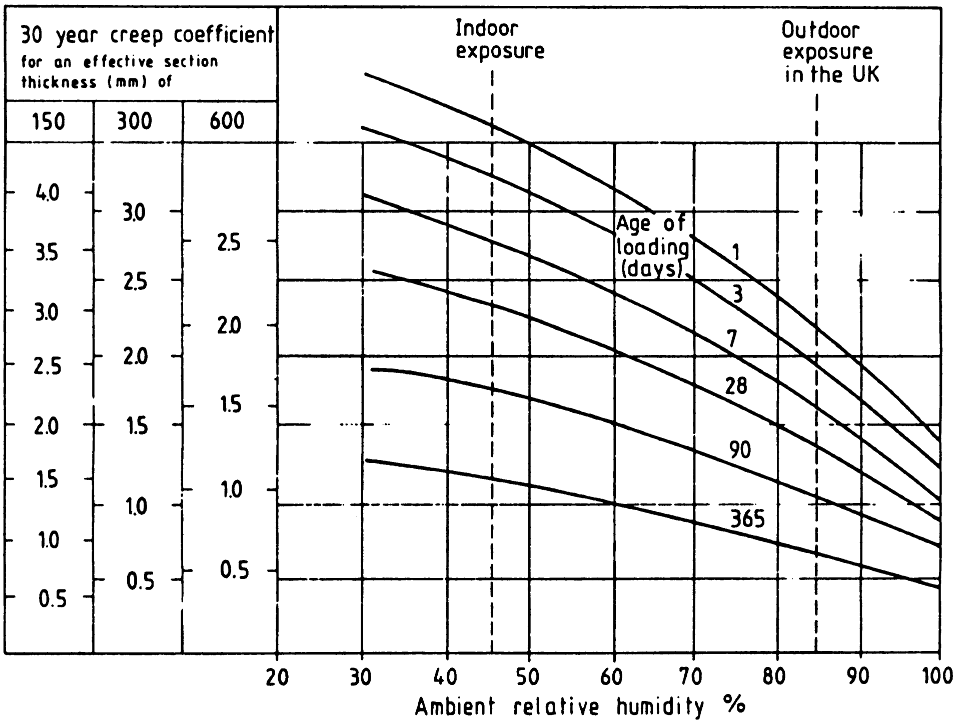


Figure 7.1 — Effects of relative humidity, age of loading and section thickness upon creep factor

The creep coefficient may be estimated from Figure 7.1. In this Figure, the effective section thickness is defined, for uniform sections, as twice the cross-sectional area divided by the exposed perimeter. If drying is prevented by immersion in water or by sealing, the effective section thickness should be taken as 600 mm. Suitable values of relative humidity for indoor and outdoor exposure in the UK are 45 % and 85 %, when using Figure 7.1 for general design purposes.

It can be assumed that about 40 %, 60 % and 80 % of the final creep develops during the first month, 6 months and 30 months under load respectively, when concrete is exposed to conditions of constant relative humidity.

Creep is partly recoverable with a reduction in stress. The final creep recovery after 1 year is approximately 0.3 × stress reduction/E_u, where E_u is the modulus of elasticity of the concrete at the age of unloading.

It is stressed that these statements provide only general guidance and are based primarily on laboratory data. Judgement, based on experience, is essential in interpreting these data in individual cases; as with elastic deformation, this will depend on the importance of the estimate and why it is needed. It may be advisable at the design stage to consider a range of values to bracket the problem, since an overestimate may be just as bad as an underestimate. In particular, it should be noted that where detailed calculations are being made, stresses and relative humidities may vary considerably during the lifetime of the structure and appropriate judgements should be made.

7.4 Drying shrinkage

An estimate of the drying shrinkage of plain concrete may be obtained from Figure 7.2. Recommendations for effective section thickness and relative humidity are given in **7.3**.

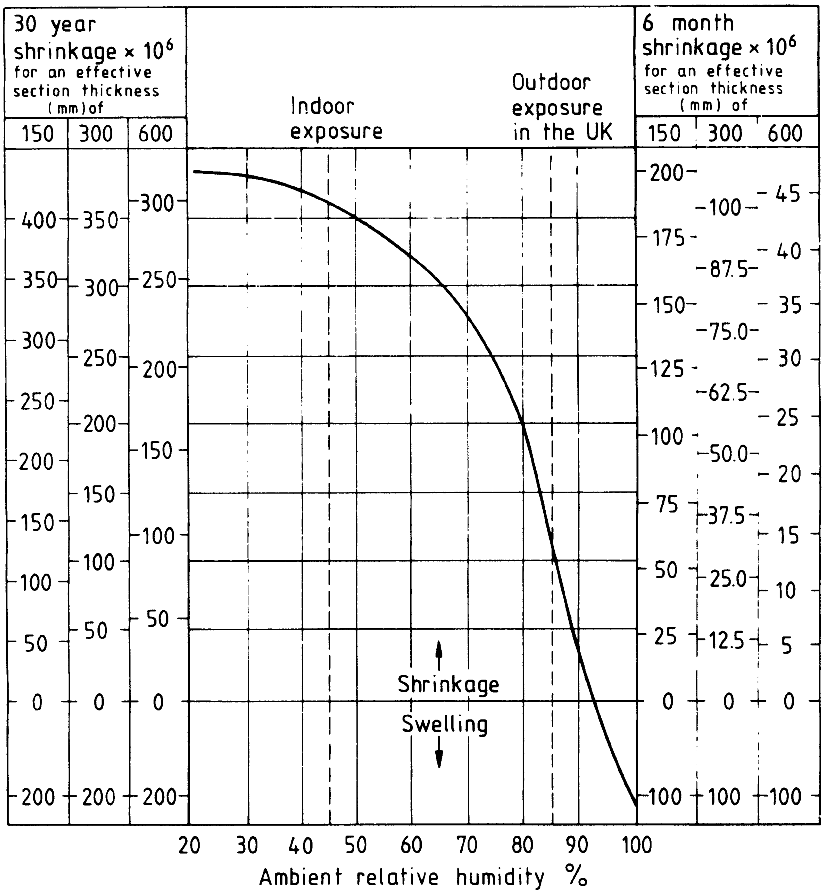


Figure 7.2 — Drying shrinkage of normal-weight concrete

Figure 7.2 relates to concrete of normal workability made without water reducing admixtures; such concretes will have an original water content of about 190 L/m^3. Where concrete is known to have a different water content, shrinkage may be regarded as proportional to water content within the range 150 L/m^3 to 230 L/m^3.

The shrinkage of plain concrete is primarily dependent on the relative humidity of the air surrounding the concrete, the surface area from which moisture can be lost relative to the volume of concrete and on the mix proportions; it is increased slightly by carbonation and self-desiccation and reduced by prolonged curing.

Aggregates having a high moisture movement, such as some Scottish dolerites and whinstones, and gravels containing these rocks, produce concrete having a higher initial drying shrinkage than that normally expected. Further information on these is given in reference [7], for consideration in using the data given in Figure 7.2. Aggregates with a low modulus may also lead to higher than normal concrete shrinkage, and this should also be borne in mind when using Figure 7.2 for estimating drying shrinkage for design purposes.

Concrete exposed to the outdoor climate in the UK will exhibit seasonal cyclic strains of ± 0.4 times the 30 year shrinkage superimposed on the average shrinkage strain; the maximum shrinkage will occur at the end of each summer.

An estimate of the shrinkage of symmetrically reinforced concrete sections may be obtained from:

$$\frac{\epsilon_{sh}}{1 + K\rho}$$

where

ϵ_{sh}	is the shrinkage of the plain concrete;
ρ	is the area of steel relative to that of the concrete;
K	is a coefficient, taken as 25 for internal exposure and as 15 for external exposure.

For non-symmetrically reinforced sections, the influence of the reinforcement on shrinkage, and hence on curvature and deflection is more complex. The procedures outlined in **3.4.6** of BS 8110-1:1997 take account of this for most normal cases. Where calculations of deflection are deemed necessary, reference should be made to section 3 of this Part.

The general remarks in **7.3** on creep apply equally to shrinkage. Such estimates may be required in allowing for movement (see section 5 of BS 8110-1:1997), in estimating loss of prestress (see **4.8** of BS 8110-1:1997) and in the assessment of differential shrinkage effects (see **5.4.6.4** of BS 8110-1:1997). In all cases judgement, based on experience, is essential.

7.5 Thermal strains

The information given in this clause is intended only for the estimation of movements and of deformation.

Thermal strains are calculated from the product of a suitable coefficient of thermal expansion and a temperature change. The temperature change can be determined from the expected service conditions and climatic data. Externally exposed concrete does not respond immediately to air temperature change, and climatic temperature ranges may require adjustment before use in movement calculations.

The coefficient of thermal expansion of concrete is dependent mainly on the expansion coefficients for the aggregate and the cement paste, and the degree of saturation of the concrete. The thermal expansion of aggregate is related to mineralogical composition (see Table 7.3).

Table 7.3 — Thermal expansion of rock group and related concrete

Aggregate type (see BS 812)	**Typical coefficient of expansion** (1×10^{-6}/ °C)	
	Aggregate	**Concrete**
Flint, quartzite	11	12
Granite, basalt	7	10
Limestone	6	8

As with all the other factors dealt with in this section, the information given provides only general guidance. These coefficients can vary, this variation being least for flints and quartzites and greatest for limestone. However, the above coefficients will be adequate for design purposes; it is only if the estimate of deformation is exceptionally important that it will be necessary to examine the aggregate actually to be used.

Cement paste has a coefficient of thermal expansion that is a function of moisture content, and this affects the concrete expansion as shown in Figure 7.3. It may be seen that partially dry concrete has a coefficient of thermal expansion that is approximately 2×10^{-6} /°C greater than the coefficient for saturated concrete.

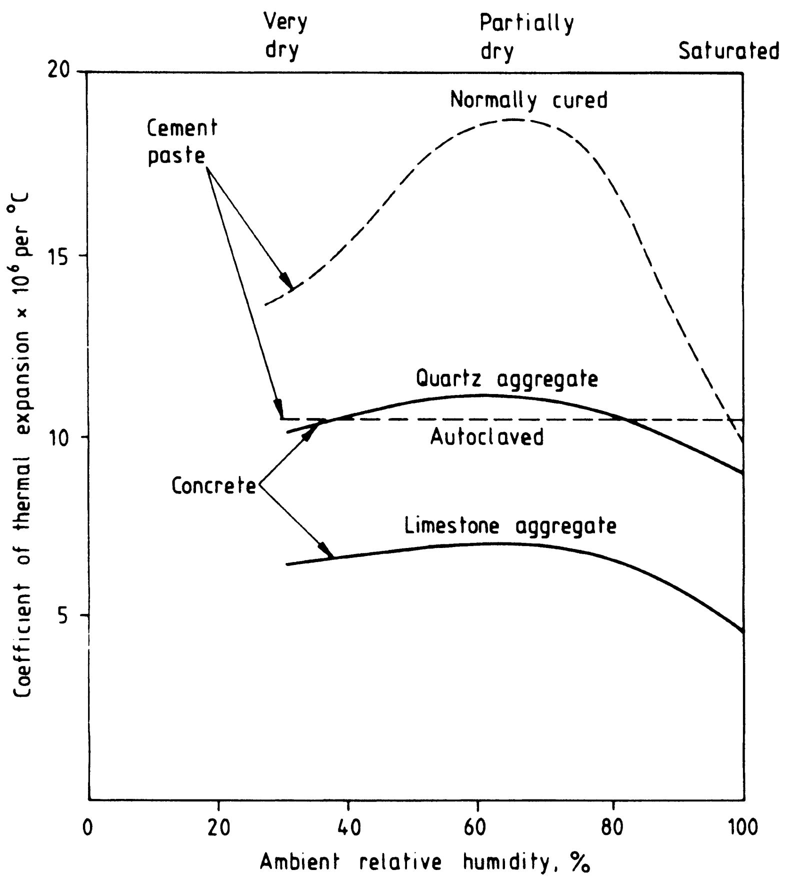


Figure 7.3 — Effect of dryness upon the coefficient of thermal expansion of hardened cement and concrete

Section 8. Movement joints

8.1 General

Many factors influence the tendency for concrete to crack and the limitation of such cracking is also influenced by many factors, probably the most important of which is the proper provision of adequate reinforcement. However, there are cases where the most appropriate or only control measure is a movement joint.

Movement joints are those specifically designed and provided to permit relative movement of adjacent parts of a member or structure to occur without impairing the functional integrity of the member or structure. Their general function is to permit controlled movement to occur so as to prevent the build-up of harmful stresses. They may also be the connection joint between the several parts of a member or structure or they may be provided solely to permit translation or rotation or both.

8.2 Need for movement joints

In common with all other structural materials, concrete expands when heated and contracts when cooled; it also expands when wetted and shrinks when dried. It also undergoes other strains due to the hydration of the cement and other properties of the material itself and of its constituent parts. If these expansions and contractions are restrained, stresses will occur which can be of sufficient magnitude to cause immediate cracking of the concrete or cause cracking to occur later owing to fatigue failure due to long-term repetition of the stresses. Creep of the concrete over a long period can in some cases reduce stresses due to restraint, but generally this should not be relied upon. Differential settlements of foundations due, for example, to mining subsidence should also be taken into consideration. As these factors may cause unsightly cracking, damage to finishes and even structural failure, the possibilities and effects of such cracking should be properly investigated in relation to the design, reinforcement and form of the member or structure concerned and in the light of published information, and if then found necessary to prevent or limit the effects of such potential cracking, movement joints should be provided at predetermined locations.

Some indication of the possible magnitude of the movements to be dealt with in a concrete structure may be gained from the following examples (but see also section 7).

a) The average coefficient of thermal expansion of normal-weight concrete is of the order of 10×10^{-6}/°C and 8×10^{-6}/°C for lightweight aggregate concrete (see Table 7.3); thus the difference in length of a concrete member 30 m long due to a 33 °C change in temperature could be approximately 10 mm. If this change in length were prevented by complete restraint of the member, it would cause a stress of the order of 7 N/mm^2 in an unreinforced concrete member made of concrete having a modulus of elasticity of 20 kN/mm^2. If such stress were tensile and superimposed upon other already existing tensile stresses, cracking would occur. If, however, the concrete were to be reinforced, the distribution of the cracking would be controlled by the amount, form and distribution of the reinforcement which might even reduce the crack width and spacing to an extent such as to cause no harmful consequence.

b) Drying shrinkage strains may be of the order of 500×10^{-6}. In thin unreinforced sections this represents an unrestrained shrinkage of the order of 1.5 mm per 3 m length of a concrete member. If this change in length were prevented, a tensile stress of about 10 N/mm^2 would occur. Since shrinkage develops over a period of months, this value would be reduced considerably in practice to about 2 N/mm^2 to 3 N/mm^2.

c) Creep of concrete under stress tends to reduce the maximum stresses arising from the restraint of movements of the types referred to in a) and b), the degree of reduction depending on, amongst other factors, the rate of change of the stresses. Creep is a long-term process and if the stresses change rapidly, e.g. because the cross section of the member is small enough to permit its temperature change or shrinkage to occur in a relatively short time, it has negligible effect in reducing the stresses.

However, creep of the concrete can itself create strains that might lead to harmful and unsightly effects if no movement joints are provided. For example, creep of the concrete can cause deflections of beams to increase over a long period under sustained loading. Unless suitable movement joints are provided between floors or roofs and partitions, these deflections can lead to heavy loads being imposed upon the partitions, which, if of a non-loadbearing type, may then suffer severe cracking.

d) Increased provision for expansion forces arising from the abnormal load of fire may need to be provided where the structure or its individual elements are deemed by the designer to be sensitive to these effects.

NOTE Details of fire sensitive structures are given in specialist reports.

8.3 Types of movement joint

Movement joints may be of the following types.

a) *Contraction joint.* A contraction joint is a joint with a deliberate discontinuity but no initial gap between the concrete on both sides of the joint, the joint being intended to permit contraction of the concrete.

A distinction should be made between a complete contraction joint, in which both the concrete and reinforcement are interrupted, and a partial contraction joint, in which only the concrete is interrupted, the reinforcement running through

b) *Expansion joint.* An expansion joint is a joint with complete discontinuity in both reinforcement and concrete and intended to accommodate either expansion or contraction of the structure.

In general, such a joint requires the provision of a sufficiently wide gap between the adjoining parts of a structure to permit the amount of expansion expected to occur. Design of the joint so as to incorporate sliding surfaces is not, however, precluded and may sometimes be advantageous

c) *Sliding joint.* A sliding joint is a joint with complete discontinuity in both reinforcement and concrete at which special provision is made to facilitate relative movement in the plane of the joint.

d) Hinged joint. A hinged joint is a joint specially designed and constructed to permit relative rotation of the members at the joint. This type of joint is usually required to prevent the occurrence of reverse moments or of undesirable restraint, for example in a three-hinged portal.

e) Settlement joint. A settlement joint is a joint permitting adjacent members or structures to settle or deflect relative to each other in cases, for example, where movements of the foundations of a building are likely due to mining subsidence. The relative movements may be large

It may be necessary to design a joint to fulfil more than one of these items.

Joints in fire resistant walls or floors should be fire stopped to an equivalent degree of fire resistance.

8.4 Provision of joints

The risk of cracking due to thermal movement and shrinkage may be minimized by limiting the changes in temperature and moisture content to which the concrete of the structure is subjected. The extent to which this can be done in the completed structure will depend very largely on its type and environment, ranging from the underground basement which is in conditions of relatively constant temperature and humidity, to the uninsulated elevated structure which might follow closely the atmospheric temperature and humidity. Furthermore, in buildings the effects of central heating on both the temperature and moisture content of the structure, combined with the relatively low thermal storage capacity of buildings clad with lightweight curtain walls, may give rise to more onerous thermal and humidity conditions than in the older, heavier, relatively unheated buildings. Thus, the investigation of the necessity to provide movement joints is becoming more important.

Cracking can be minimized by reducing the restraints on the free movement of the structure, and the control of cracking normally requires the subdivision of the structure into suitable lengths separated by the appropriate movement joints.

The effectiveness of movement joints in controlling cracking in a structure will also depend upon their precise location; this latter is frequently a matter of experience and may be characterized as the place where cracks would otherwise most probably develop, e.g. at abrupt changes of cross section.

The location of all movement joints should be clearly indicated on the drawings, both for the individual members and for the structure as a whole. In general, movement joints in the structure should pass through the whole structure in one plane.

8.5 Design of joints

A movement joint should fulfil all necessary functions. It should possess the merits of simplicity and freedom of movement, yet still retain the other appropriate characteristics necessary, e.g. weatherproofness, fire resistance, resistance to corrosion, durability and sound insulation. The design should also take into consideration the degree of control and workmanship and the tolerances likely to occur in the actual structure of the type being considered.

Where joints are of a filled type, they may in appropriate cases be filled with a building mastic. There are at present no standard specifications for such materials, but attention is drawn to BS 3712, BS 6093 and BS 6213.

Section 9. Appraisal and testing of structures and components during construction

9.1 General

This section refers to methods for appraisal and, where necessary, for testing whole structures, finished parts of a structure or structural components during the construction phase. It is assumed that the structure and components have been designed in accordance with this standard.

The section gives only general guidelines and broad principles; detailed recommendations for particular cases may be obtained from more comprehensive documents which are referenced.

The recommendations of this section may not generally be suitable for:

a) model testing when used as a basis for design;

b) development testing of prototype structures as a basis for design;

c) checking the adequacy of existing structures (related to change of use or loading, or to deterioration or accidental damage), unless their performance is calibrated against a design to this standard (see **9.3**).

9.2 Purpose of testing

Within **9.1**, the methods given in **9.3**, **9.4**, **9.5** and **9.6** may be appropriate in any of the following circumstances:

a) where the compliance procedures in sections 6, 7 and 8 of BS 8110-1:1997 indicate that the materials used may be sub-standard or defective;

b) where supervision and inspection procedures indicate poor workmanship on site, producing construction outside the specification and design;

c) where there are visible defects, particularly at critical sections or in sensitive structural members;

d) where a check is required on the quality of the construction, or manufacture of precast units.

9.3 Basis of approach

The basic objective of appraisal under the circumstances described in **9.2** is to assess the structure as built and to decide whether or not it meets the requirements of the original design.

The type and extent of any tests used in a particular case should be chosen to achieve this objective, and should be agreed in advance by all the parties concerned, both in principle and in detail. The tests should be relevant, and the results used in recalculation procedures to assess and justify the structure as appropriate. Based on the information so obtained and on an examination of all other relevant factors, a judgement can be made on the acceptability of the structure.

In general, these procedures should be systematic and progressive, i.e. the methods given in **9.4** should be used first, and only if there is still doubt should those in **9.5** be considered.

9.4 Check tests on structural concrete

9.4.1 General

This clause covers tests used to determine the quality of the materials used in the structure as built; values for the material parameters so obtained may then be used in calculations to appraise the structure. The prime concern is with the measurement of strength in situ, either directly or indirectly (see **9.4.2**) but tests may also be required to determine concrete cover and integrity, material composition, the presence of defects or contaminants, etc. Available test techniques are listed in [8] together with an assessment of their applicability, advantages and limitations.

9.4.2 Concrete strength in structures

The routine sampling of concrete and the testing of standard concrete control specimens to ensure compliance with strength criteria is covered in section 6 of BS 8110-1:1997.

This subclause covers special testing, carried out for the reasons given in **9.2**, and following the basic approach given in **9.3**. The test methods to be used are given in BS 6089, which also describes test procedures and methods for evaluating results; these test methods are deemed satisfactory for all structural concrete in common use.

In particular circumstances, where there is still some doubt about the acceptability of the structure, a loading test may be required; this should be carried out in accordance with **9.5**. However, in most cases, the procedures given in this subclause will provide sufficient relevant information to permit a proper appraisal and justification of a structure.

9.5 Load tests of structures or parts of structures

9.5.1 General

If a load test is deemed necessary, it may be to check on either strength or serviceability. It should be recognized that loading a structure to its design ultimate loads may impair its subsequent performance in service, without necessarily giving a true measure of load-carrying capacity. While such overload tests may sometimes be justified (see **9.6**), it is generally recommended that the structure be loaded to a level appropriate to the serviceability limit states. If sufficient measurements of deformations are taken, then these, together with the results from the test described in **9.5.2**, can be used to calibrate the original design in predicting the ultimate strength and long-term performance of the structure.

Detailed recommendations on test procedures are given in [8] with background information being provided in [9] and [10]. Some general principles are given in **9.5.2**, **9.5.3**, **9.5.4** and **9.5.5**.

9.5.2 Test loads

The total load to be carried (W) should be not less than 1.0 times the characteristic dead load plus 1.0 times the characteristic live load, and should normally be the greater of a) the sum of the characteristic dead load and 1.25 times the characteristic imposed load or b) 1.125 times the sum of the characteristic dead and imposed loads. In deciding on suitable figures for this, and on how to apply the test load to the structure, due allowance should be made for finishes, partitions, etc. and for any load sharing that could occur in the completed structure, i.e. the level of loading should be representative and capable of reproducing the proper internal force system reasonably closely.

Test loads should be applied and removed incrementally, while observing all proper safety precautions. The test loading should be applied at least twice, with a minimum of 1 h between tests, and allowing 5 min after a load increment is applied before recording deformation measurements. Consideration may also be given to a third application of load, which is left in position for 24 h.

9.5.3 Assessment of results

In determining deformation measurements, due allowance should be made for changes in environmental conditions that have occurred during the test.

The main objective in assessing the results is to compare the measured performance with that expected on the basis of the design calculations. This means that due allowance should be made for any differences in material strength, or stress, or other characteristic, in the as-built structure, compared with that assumed in the design. Steps should be taken to determine these material parameters as accurately as possible, using the methods referred to in **9.6**, standard control test results, tensioning records (for prestressed concrete), etc.

9.5.4 Test criteria

In assessing test data and in recalculation procedures, the following criteria should be considered:

a) the initial deflection and cracking should be in accordance with the design requirements;

b) where significant deflections have occurred under the normal loads given in **9.5.2**, the percentage recovery after the second loading should at least equal that for the first loading cycle, and should be at least 75 % for reinforced concrete and class 3 prestressed concrete, and 85 % for classes 1 and 2 prestressed concrete;[3)]

c) the structure should be examined for unexpected defects, which should then be evaluated in the recalculation procedures.

9.5.5 Special tests

In certain cases, it may be necessary to devise special tests to reproduce the internal force system expected in the completed structure. This need can arise in the testing of the precast parts of composite members, or where the final boundary conditions have not yet been achieved in the construction. Such tests should be relevant, and agreed in advance by all the parties concerned.

9.6 Load tests on individual precast units

Load tests on precast units may be necessary for reasons a) to c) of **9.2**. In these circumstances, the procedures should be in accordance with **9.3**, **9.4** and **9.5**.

If load testing is also required as a check on the quality of the units for the acceptance of new units, the procedures may again be in accordance with **9.3**, **9.4** and **9.5**, or as determined by the technical schedule in a satisfactory quality assurance system. Sampling rates should be as given in the technical schedule, or as in the specification. The basis of the overall approach should be as outlined in **9.3** for the assessment of both serviceability and strength, in which case overload tests will not normally be required. Should doubt exist about the ultimate strength of a series of units, then tests to failure may be necessary, at a rate to be agreed by all the parties concerned. In such tests, the performance should be in accordance with that expected from the design calculations. In general, the ultimate strength should exceed the design ultimate load by a margin of at least 5 %; moreover, the deflection, up to the design ultimate load, should not exceed 1/40 of the span.

3) Where the measured deflections are very small (e.g. < span/1 000), estimates of recovery become meaningless.

blank

Appendix A
Bibliography

1. *Rationalisation of safety and serviceability factors in Structural Codes.* CIRIA Report 63. Construction Industry Research and Information Association, London 1976.

2. Cranston, W.B. *Analysis and design of reinforced concrete columns.* Research Report 20. Cement and Concrete Association 1972.

3. Meyer, C. and Bathe, K.J. Nonlinear analysis of RC Structures in practice. *Proceedings of ASCE, Structures Division*, July 1982 **108**. No. ST7, pp 1605-1622.

4. Read, R.E.H., Adams, F.C. and Cooke, G.M.E. *Guidelines for the construction of fire resisting structural elements.* Building Research Establishment Report. HMSO 1982.

5. *Design and Detailing of concrete structures for fire resistance.* Joint Committee of the Institution of Structural Engineers and the Concrete Society. April 1978.

6. Teychenne, D.C., Parrott, L.J. and Pomeroy, C.D. *The estimation of the elastic modulus of concrete for the design of structures.* Report CP 23/78 Building Research Establishment.

7. Shrinkage of natural aggregates in concrete. *Building Research Establishment Digest 35 (second series).* Building Research Establishment.

8. *Appraisal of existing structures.* Report of the Institution of Structural Engineers, July 1980.

9. Menzies, J.B. Loading testing of concrete building structures. *The Structural Engineer*. December 1978 **56A**, No. 12, pp 347-353.

10. Jones, D.S. and Oliver, C.W. The practical aspects of load testing. *The Structural Engineer.* December 1978 **56A**, No. 12, pp 353-356.

11. *Guide to Lightweight Aggregate Concrete.* The Institution of Structural Engineers. 1985.

Index

blank

Publications referred to

BS 12, *Specification for ordinary and rapid-hardening Portland cement.*
BS 146, *Portland-blastfurnace cement.*
BS 476, *Fire tests on building materials and structures.*
BS 476-8, *Test methods and criteria for the fire resistance of elements of building construction.*
BS 812, *Methods for sampling and testing of mineral aggregates, sands and fillers.*
BS 890, *Building limes.*
BS 3148, *Methods of tests for water for making concrete (including notes on the suitability of the water).*
BS 3712, *Methods of test for building sealants.*
BS 4466, *Specification for bending dimensions and scheduling of reinforcement, for concrete.*
BS 5328, *Methods for specifying concrete, including ready-mixed concrete.*
BS 5337, *Code of practice for the structural use of concrete for retaining aqueous liquids.*
BS 6089, *Guide to the assessment of concrete strength in existing structures.*
BS 6093, *Code of practice for the design of joints and jointing in building construction.*
BS 6213, *Guide to the selection of constructional sealants.*
BS 8110, *Structural use of concrete.*
BS 8110-1, *Code of practice for design and construction.*

BSI — British Standards Institution

BSI is the independent national body responsible for preparing British Standards. It presents the UK view on standards in Europe and at the international level. It is incorporated by Royal Charter.

Revisions

British Standards are updated by amendment or revision. Users of British Standards should make sure that they possess the latest amendments or editions.

It is the constant aim of BSI to improve the quality of our products and services. We would be grateful if anyone finding an inaccuracy or ambiguity while using this British Standard would inform the Secretary of the technical committee responsible, the identity of which can be found on the inside front cover.
Tel: 020 8996 9000. Fax: 020 8996 7400.

BSI offers members an individual updating service called PLUS which ensures that subscribers automatically receive the latest editions of standards.

Buying standards

Orders for all BSI, international and foreign standards publications should be addressed to Customer Services. Tel: 020 8996 9001. Fax: 020 8996 7001.
Standards are also available from the BSI website at http://www.bsi-global.com.

In response to orders for international standards, it is BSI policy to supply the BSI implementation of those that have been published as British Standards, unless otherwise requested.

Information on standards

BSI provides a wide range of information on national, European and international standards through its Library and its Technical Help to Exporters Service. Various BSI electronic information services are also available which give details on all its products and services. Contact the Information Centre.
Tel: 020 8996 7111. Fax: 020 8996 7048.

Subscribing members of BSI are kept up to date with standards developments and receive substantial discounts on the purchase price of standards. For details of these and other benefits contact Membership Administration.
Tel: 020 8996 7002. Fax: 020 8996 7001. Further information about BSI is available on the BSI website at http://www.bsi-global.com.

BSI
389 Chiswick High Road
London
W4 4AL