

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
- Co-opted members

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

© BSI 30 November 2005

Amendments issued since publication

Amd. No.	Date of issue	Comments
5914	May 1989	
12061	July 2001	
16017	30 November 2005	Indicated by a sideline

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

Contents

	Page
Committees responsible	Inside front cover
Foreword	iv
Section 1. General	
1.1 Scope	1
1.2 Definitions	1
1.3 Symbols	1
Section 2. Methods of analysis for the ultimate limit state	
2.1 General	3
2.2 Design loads and strengths	3
2.3 Non-linear methods	5
2.4 Torsional resistance of beams	5
2.5 Effective column height	9
2.6 Robustness	10
Section 3. Serviceability calculations	
3.1 General	13
3.2 Serviceability limit states	13
3.3 Loads	14
3.4 Analysis of structure for serviceability limit states	15
3.5 Material properties for the calculation of curvature and stresses	15
3.6 Calculation of curvatures	15
3.7 Calculation of deflection	17
3.8 Calculation of crack width	20
Section 4. Fire resistance	
4.1 General	25
4.2 Factors to be considered in determining fire resistance	26
4.3 Tabulated data (method 1)	28
4.4 Fire test (method 2)	32
4.5 Fire engineering calculations (method 3)	32
Section 5. Additional considerations in the use of lightweight aggregate concrete	
5.1 General	39
5.2 Cover for durability and fire resistance	39
5.3 Characteristic strength of concrete	40
5.4 Shear resistance	40
5.5 Torsional resistance of beams	40
5.6 Deflections	41
5.7 Columns	41
5.8 Walls	41
5.9 Anchorage bond and laps	41
5.10 Bearing stress inside bends	41

Section 6. (deleted)

Section 7. Elastic deformation, creep, drying shrinkage and thermal strains of concrete	
7.1 General	45
7.2 Elastic deformation	45
7.3 Creep	46
7.4 Drying shrinkage	48
7.5 Thermal strains	49
Section 8. Movement joints	
8.1 General	51
8.2 Need for movement joints	51
8.3 Types of movement joint	52
8.4 Provision of joints	52
8.5 Design of joints	52
Section 9. Appraisal and testing of structures and components during construction	
9.1 General	53
9.2 Purpose of testing	53
9.3 Basis of approach	53
9.4 Check tests on structural concrete	53
9.5 Load tests of structures or parts of structures	54
9.6 Load tests on individual precast units	55
Appendix A Bibliography	57
Index	58
Figure 2.1 — Stress strain curve for rigorous analysis of non-critical sections	6
Figure 3.1 — Assumptions made in calculating curvatures	16
Figure 3.2 — Deflection of a cantilever forming part of a framed structure	20
Figure 4.1 — Calculation of average cover	29
Figure 4.2 — Typical examples of beams, plain soffit floors and ribbed soffit floors	30
Figure 4.3 — Typical examples of reinforced concrete columns	31
Figure 4.4 — Design curves for variation of concrete strength with temperature	37
Figure 4.5 — Design curves for variation of steel strength or yield stress with temperature	38
Figure 7.1 — Effects of relative humidity, age of loading and section thickness upon creep factor	47
Figure 7.2 — Drying shrinkage of normal-weight concrete	48
Figure 7.3 — Effect of dryness upon the coefficient of thermal expansion of hardened cement and concrete	50

	Page
Table 2.1 — Minimum values of partial safety factors to be applied to worst credible values	4
Table 2.2 — Values of coefficient β	7
Table 2.3 — Values of $v_{t,min}$ and v_{tu}	8
Table 2.4 — Reinforcement for shear and torsion	8
Table 3.1 — Values of K for various bending moment diagrams	19
Table 3.2 — Estimated limiting temperature changes to avoid cracking	22
Table 3.3 — Values of external restraint recorded in various structures	23
Table 4.1 — Variation of cover to main reinforcement with member width	32
Table 4.2 — Reinforced concrete columns	33
Table 4.3 — Concrete beams	34
Table 4.4 — Plain soffit concrete floors	34
Table 4.5 — Ribbed open soffit concrete floors	35
Table 4.6 — Concrete walls with vertical reinforcement	36
Table 5.1 — <i>deleted</i>	39
Table 5.2 — Nominal cover to all steel to meet specified periods of fire resistance (lightweight aggregate concrete)	40
Table 5.3 — Values of v_c , design shear stress for strength class LC20/22 lightweight concrete	40
Table 7.1 — Strength of concrete	46
Table 7.2 — Typical range for the static modulus of elasticity at 28 days of normal-weight concrete	46
Table 7.3 — Thermal expansion of rock group and related concrete	49
Publications referred to	Inside back cover

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 8110-1.

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

Amendment No. 3 to BS 8110-2:1985 has been necessitated through the adoption of BS 8500, *Concrete — Complementary British Standard to BS EN 206-1*, the withdrawal of BS 5328, *Concrete* and the adoption of 500 Grade high yield steel for use in the UK.

Amendment No. 3 aligns the recommendations for exposure classification and durability in BS 8110-2:1985 with those in BS 8500. Recommendations for durability have been removed from BS 8110-2:1985 and replaced with references to the appropriate guidance in BS 8500. Essential changes have been made to account for the terminology of BS 8500 (e.g. the dual cylinder/cube strength notation, and “strength classes” instead of “grades”).

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

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

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

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

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

This publication does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for its 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 61 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

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 63¹⁾ [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 3.6a).

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.

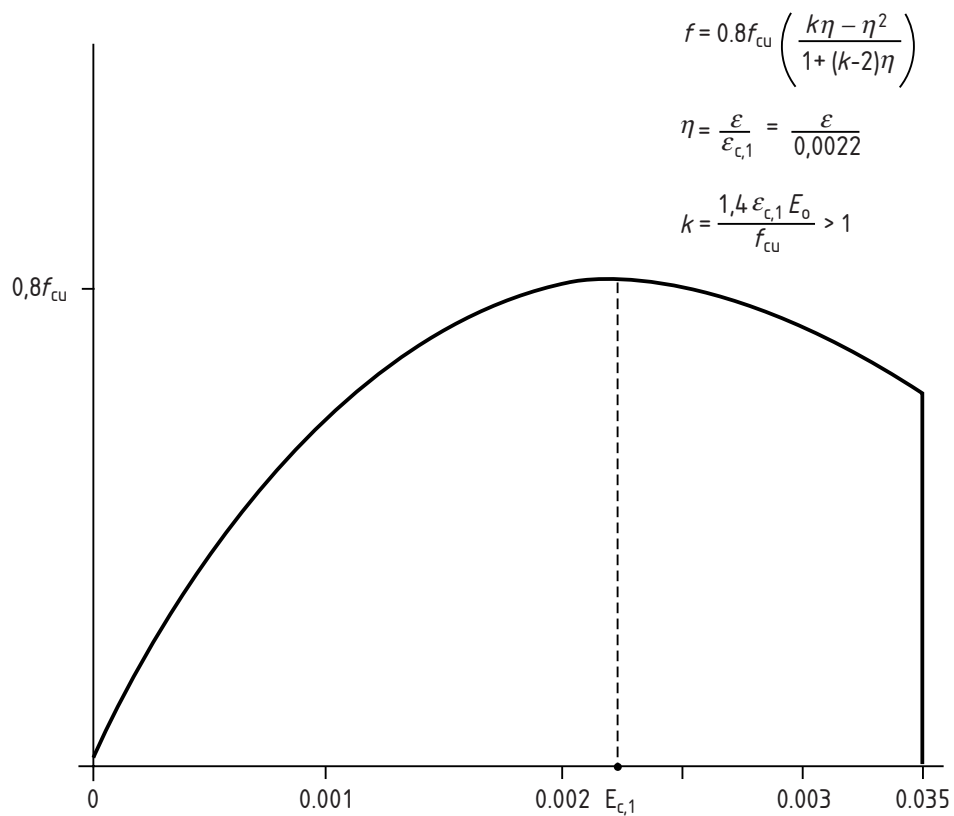


Figure 2.1 — Stress strain curve for rigorous analysis of non-critical sections

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 ($G \times C$)

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:

$$C = \beta \times h_{min}^3 \times h_{max} \quad \text{equation 1}$$

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:

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

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 $h_{\min}^3 h_{\max}$ 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:

$$\tau \left(\frac{h_{\min}^3 h_{\max}}{\sum (h_{\min}^3 h_{\max})} \right)$$

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 strength class	$v_{t,\min}$	v_{tu}
	N/mm ²	N/mm ²
20/25	0.33	4.00
25/30	0.37	4.38
32/40 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:

$$\frac{A_{sv}}{s_v} > \frac{T}{0.8 x_1 y_1 (0.87 f_{yv})}$$

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

NOTE f_y and f_{yv} should not be taken as greater than 500 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_0	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:

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

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

2.5.6 Unbraced columns: effective height for framed structures

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

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

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

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 2.2.2.2c) 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 Class 2B or Class 3 of Section 5 of The Building Regulations 2000 — Approved Document A: Structure). 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/m², 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/m². 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 Class 2B or Class 3 of Section 5 of The Building Regulations 2000 — Approved Document A: Structure)

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**). The length considered need not exceed 2.25 times the storey height.

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.

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.