



BS 5400: Part 4: 1984

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British Standard

Steel, concrete and composite bridges

Part 4. Code of practice for design of concrete bridges

Ponts en acier, ponts en béton, ponts mixtes
Partie 4. Règles pour le calcul des ponts en béton

Brücken aus Stahl, Beton und Verbundbau
Teil 4. Richtlinie für den Entwurf und die Bemessung von Stahlbetonbrücken

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Foreword

This Part of BS 5400 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. It supersedes BS 5400 : Part 4 : 1978 which is withdrawn.

BS 5400 is a document combining codes of practice to cover the design and construction of steel, concrete and composite bridges and specifications for loads, materials and workmanship. It comprises the following Parts and Sections:

- Part 1 General statement
- Part 2 Specification for loads
- Part 3 Code of practice for design of steel bridges
- Part 4 Code of practice for design of concrete bridges
- Part 5 Code of practice for design of composite bridges
- Part 6 Specification for materials and workmanship, steel

Part 7 Specification for materials and workmanship, concrete, reinforcement and prestressing tendons

Part 8 Recommendations for materials and workmanship, concrete reinforcement and prestressing tendons

Part 9 Bridge bearings
Section 9.1 Code of practice for design of bridge bearings
Section 9.2 Specification for materials, manufacture and installation of bridge bearings

Part 10 Code of practice for fatigue

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

British Standard

Steel, concrete and composite bridges

Part 4. Code of practice for design of concrete bridges

1. Scope

This Part of BS 5400 gives recommendations for the design of concrete bridges. It contains much in common with CP 110 which deals with the structural use of concrete.

After stating the objectives and requirements of design, particular recommendations are given for reinforced concrete, prestressed concrete and composite concrete construction.

Structural elements included are beams, slabs, columns and walls, bases, tension members and connections between precast concrete members.

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

2. Definitions and symbols

2.1 Definitions

2.1.1 General. For the purposes of this Part of BS 5400 the definitions given in Part 1 apply.

All formulae are based on SI units in newtons and millimetres unless otherwise stated.

2.1.2 Partial load factors. For the sake of clarity the factors that together comprise the partial safety factor for loads are restated as follows.

Design loads, Q^* , are the loads obtained by multiplying the nominal loads, Q_k , by γ_{fL} , the partial safety factor for loads. γ_{fL} is a function of two individual factors, γ_{f1} and γ_{f2} , which take account of the following:

- γ_{f1} possible unfavourable deviations of the loads from their nominal values;
- γ_{f2} reduced probability that various loadings acting together will all attain their nominal values simultaneously.

The relevant values of the function γ_{fL} ($= \gamma_{f1} \cdot \gamma_{f2}$) are given in Part 2.

The design load effects, S^* , should be obtained from the design loads by the relation:

$$S^* = \gamma_{f3} \text{ (effects of } Q^*)$$

where

γ_{f3} is a factor that takes account of inaccurate assessment of the effects of loading, unforeseen stress distribution in the structure and variations in dimensional accuracy achieved in construction.

The values of γ_{f3} are given in clause 4.

2.1.3 Materials

2.1.3.1 characteristic strength. Unless otherwise stated, that value of the cube strength of concrete, f_{cu} , the yield

or proof strength of reinforcement, f_y , or the breaking load of a prestressing tendon, f_{pu} , below which not more than 5 % of all possible test results may be expected to fall.

2.1.3.2 characteristic stress. That value of stress at the assumed limit of linearity on the stress-strain curve for the material.

2.2 Symbols. The symbols used in this Part of BS 5400 are as follows.

A_c	area of concrete
A_{cf}	area of effective concrete flange
A_{con}	contact area
A_{cor}	area of core of the concrete section
A_e	area of fully anchored reinforcement per unit length crossing the shear plane
A_o	area enclosed by the median wall line
A_{ps}	area of prestressing tendons in the tension zone
A_s	area of tension reinforcement
A'_s	area of compression reinforcement
A'_{s1}	area of compression reinforcement in the more highly compressed face
A_{s2}	area of reinforcement in other face
A_{sc}	area of longitudinal reinforcement (for columns)
A_{sL}	cross-sectional area of one bar of longitudinal reinforcement provided for torsion
A_{st}	cross-sectional area of one leg of a closed link
A_{sup}	supporting area
A_{sv}	cross-sectional area of the legs of a link
A_t	area of transverse reinforcement
a	deflection
a'	distance from compression face to point at which the crack width is being calculated
a_b	centre-to-centre distance between bars
a_{cent}	distance of the centroid of the concrete flange from the centroid of the composite section
a_{cr}	distance from the point (crack) considered to the surface of nearest longitudinal bar
a_e	modular ratio at first introduction of stress
a_s	distance of the centroid of the steel from the centroid of the net concrete section
a_v	distance between the line of action or point of application of the load and the critical section or supporting member

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b	width or breadth of section	f_{cu}	characteristic concrete cube strength
b_a	average breadth of section excluding the compression flange	f_{pb}	tensile stress in tendons at (beam) failure
b_c	breadth of compression face	f_{pe}	effective prestress (in tendon)
b_{col}	width of column	f_{pt}	stress due to prestress
b_s	width of section containing effective reinforcement for punching shear	f_{pu}	characteristic strength of prestressing tendons
b_t	breadth of section at level of tension reinforcement	f_{s2}	stress in reinforcement in other face
b_w	breadth of web or rib of a member	f_t	maximum principal tensile stress
c_{nom}	nominal cover	f_y	characteristic strength of reinforcement
D_c	density of lightweight aggregate concrete at time of test	f_{yc}	design strength of longitudinal steel in compression
d	effective depth to tension reinforcement	f_{yL}	characteristic strength of longitudinal reinforcement
d'	depth of compression reinforcement	f_{yv}	characteristic strength of link reinforcement
d_c	depth of concrete in compression	h	overall depth (thickness) of section (in plane of bending)
d_e	effective depth for a solid slab or rectangular beam; otherwise the overall depth of the compression flange	h_{agg}	maximum size of aggregate
d_o	depth to additional reinforcement to resist horizontal loading	h_e	effective thickness
d_t	effective depth from the extreme compression fibre to either the longitudinal bars around which the stirrups pass or the centroid of the tendons, whichever is the greater	h_f	thickness of flange
d_2	depth from the surface to the reinforcement in the other face	h_{max}	larger dimension of section
E_c	static secant modulus of elasticity of concrete	h_{min}	smaller dimension of section
E_{cf}	modulus of elasticity of flange concrete	h_{wo}	wall thickness where the stress is determined
E_{cq}	dynamic tangent modulus of elasticity of concrete	h_x	overall depth of the cross section in the plane of bending M_{ly}
E_s	modulus of elasticity of steel	h_y	overall depth of the cross section in the plane of bending M_{lx}
$(EI)_c$	flexural rigidity of the column cross section	I	second moment of area
E_{28}	secant modulus of elasticity of the concrete at the age of 28 days	i	radius of gyration
e	base of Napierian logarithms	j	age at application of increment of stress
e	eccentricity	j_1	age of first loading
e_x	resultant eccentricity of load at right-angles to plane of wall	j_m	number of days during which hardening takes place at $T^\circ C$
F_{bst}	tensile bursting force	j_∞	age at the end of the life of the structure
F_{bt}	tensile force due to ultimate loads in a bar or group of bars	K	a factor depending on the type of duct or sheath used
F_h	maximum horizontal ultimate load	K_1	depends on the shape of the bending moment diagram
F_v	maximum vertical ultimate load	k	a constant (with appropriate subscripts)
f	stress	k_c	depends on the composition of the concrete
f_{bs}	local bond stress	k_e	depends on the effective thickness of the member
f_c	a constant stress in the concrete	k_j	covers the development of the deferred deformation with time
f_{cav}	average compressive stress in the flexural compressive zone	k_L	depends on environmental conditions
f_{ci}	concrete strength at (initial) transfer	k_m	depends on the hardening (maturity) of the concrete at the age of loading
f_{cj}	stress in concrete at application of an increment of stress at time j	k_{m1}	coefficient k_m for age at first loading
f_{co}	stress in concrete at the level of the tendon due to initial prestress and dead load	k_r	depends on grade of reinforcement
f_{cp}	compressive stress at the centroidal axis due to prestress	k_t	depends on the type of tendon
		k_1	depends on the concrete bond across the shear plane
		k_2	depends on the geometric ratio of longitudinal reinforcement
		L_s	length of shear plane
		l	distance from face of support at the end of a cantilever, or effective span of a member

l_e	effective height of a column or wall	r	internal radius of bend
l_{ex}	effective height for bending about the major axis	r_{ps}	radius of curvature of a tendon
l_{ey}	effective height for bending about the minor axis	r_b	radius of curvature of a beam at mid span or, for cantilevers, at the support section
l_o	clear height of column between end restraints	r_{cs}	shrinkage radius of curvature
l_{sb}	length of straight reinforcement beyond the intersection with the link	r_x	radius of curvature of a member at point x
l_t	transmission length	S^*	design load effects
M	bending moment due to ultimate loads	s_L	spacing of longitudinal reinforcement
M_a	increased moment in column	s_v	spacing of links along the member
M_{cr}	cracking moment at the section considered	T	torsional moment due to ultimate loads
M_{cs}	hogging restraint moment at an internal support of a continuous composite beam and slab section due to differential shrinkage	T	temperature (in degrees Celsius)
M_g	moment due to permanent loads	t	time
M_i	maximum initial moment in a column due to ultimate loads	u	perimeter
M_{ix}	initial moment about the major axis of a slender column due to ultimate loads	u_s	effective perimeter of tension reinforcement
M_{iy}	initial moment about the minor axis of a slender column due to ultimate loads	V	shear force due to ultimate loads
M_t	moment due to live loads	V_c	ultimate shear resistance of concrete
M_{tx}	total moment about the major axis of a slender column due to ultimate loads	V_{co}	ultimate shear resistance of a section uncracked in flexure
M_{ty}	total moment about the minor axis of a slender column due to ultimate loads	V_{cr}	ultimate shear resistance of a section cracked in flexure
M_u	ultimate resistance moment	V_l	longitudinal shear force due to ultimate load
M_{ux}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the major axis only	V_{ux}	ultimate shear capacity of a section for the x-x axis
M_{uy}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the minor axis only	V_{uy}	ultimate shear capacity of a section for the y-y axis
M_x, M_y	moments about the major and minor axes of a short column due to ultimate loads	V_x	applied shear due to ultimate loads for the x-x axis
M_0	moment necessary to produce zero stress in the concrete at the depth d	V_y	applied shear due to ultimate loads for the y-y axis
M_1	smaller initial end moment due to ultimate loads (assumed negative if the column is bent in double curvature)	v	shear stress
M_2	larger initial end moment due to ultimate loads (assumed positive)	v_c	ultimate shear stress in concrete
N	ultimate axial load at section considered	v_l	ultimate longitudinal shear stress per unit area of contact surface
N_u	ultimate resistance axial load	v_t	torsional shear stress
N_{uz}	axial loading capacity of a column ignoring all bending	v_{tmin}	minimum ultimate torsional shear stress for which reinforcement is required
n_w	ultimate axial load per unit length of wall	v_{tu}	ultimate torsional shear stress
P_f	effective prestressing force after all losses	x	neutral axis depth
P_h	horizontal component of the prestressing force after all losses	x_1	smaller centre line dimension of a link
P_k	basic load in tendon	y	distance of the fibre considered in the plane of bending from the centroid of the concrete section
P_o	prestressing force in the tendon at the jacking end (or at tangent point near jacking end)	y_o	half the side of end block
P_x	prestressing force at distance x from jack	y_{po}	half the side of loaded area
Q^*	design load	y_1	larger centre line dimension of a link
Q_k	nominal load	z	lever arm
		α	orientation of the reinforcement in the transverse direction
		α_e	modular ratio
		α_n	coefficient as a function of column axial load
		α_1	angle between the axis of the design moment and the direction of the tensile reinforcement
		α_2	angle of friction at the joint
		β_{cc}	ratio of total creep to elastic deformation
		$\gamma_{f1}, \gamma_{f2}, \gamma_{f3}$	partial load factors

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γ_{fL}	product of γ_{f1} , γ_{f2}
γ_m	partial safety factor for strength
Δ_{cc}	concrete creep deformation
Δ_{cct}	creep strain
Δ_{cs}	concrete shrinkage deformation
Δ_{cst}	shrinkage at time t
$\Delta_{f_{\infty}}$	change in stress in concrete due to creep at time t_{∞}
Δ_p	loss of prestress due to shrinkage and creep
δ_m	degree of hardening at moment of loading
ϵ	strain
ϵ_{c1}	creep strain in concrete at the level of the tendon at time of loading
ϵ_{c2}	creep strain in concrete at the centroid of the section at time of loading
ϵ_{cs}	free shrinkage strain
ϵ_{diff}	differential shrinkage strain
ϵ_m	average strain
ϵ_s	strain in tension reinforcement
ϵ_1	strain at level considered
η	relaxation coefficient
θ_s	angle between the compression face and the tension reinforcement
λ_w	coefficient for walls dependent upon concrete used
μ	coefficient of friction
ξ_s	depth factor
ρ	geometrical ratio of reinforcement, is equal to A_s/bd
ρ_o	coefficient which depends upon the percentage of tension and compression steel in the section
ΣA_{sv}	area of shear reinforcement
Σbd	area of the critical section
Σu_s	sum of the effective perimeters of the tension reinforcement
Φ	size (nominal diameter) of bar or tendon
ϕ	creep coefficient
ϕ_{ti}	creep coefficient at time t for load applied at time i
ϕ_1	creep coefficient for prestressed construction
ϕ_2	creep reduction coefficient for axially loaded symmetrically reinforced concrete members
ϕ_3	creep reduction coefficient for axially loaded singly reinforced concrete members
ϕ_4	creep reduction coefficient for symmetrically reinforced concrete members subjected to bending moment
ϕ_5	creep reduction coefficient for singly reinforced concrete members subjected to bending moment

3. Limit state philosophy

3.1 General. The limit states of 4.1.1 and 4.1.2 should be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual approach

will be to design on the basis of the limit state expected to be most critical and then to check that the remaining limit states will not be reached and that all other requirements will be met.

Consideration of other factors, such as deflection, fatigue and durability, might need to be made as referred to in 4.1.3.

3.2 Serviceability limit state. The design should be such that the structure will not suffer local damage which would shorten its intended life or incur excessive maintenance costs. In particular, calculated crack widths should not exceed those permitted in 4.1.1.1.

3.3 Ultimate limit state. The strength of the structure should be sufficient to withstand the design loads, taking due account of the possibility of overturning or buckling. The assessment should ensure that collapse will not occur as a result of rupture of one or more critical sections, by overturning or by buckling caused by elastic or plastic instability, having due regard to the effects of sway when appropriate.

4. Design: general

4.1 Limit state requirements

4.1.1 Serviceability limit states

4.1.1.1 Cracking. Cracking of concrete should not adversely affect the appearance or durability of the structure. The Engineer should satisfy himself that any cracking will not be excessive, having regard to the requirements of the particular structure and the conditions of exposure. In the absence of special investigations, the following limits should be adopted.

(a) *Reinforced concrete.* Design crack widths, as calculated in accordance with 5.8.8.2, should not exceed the values given in table 1 under the loading given in 4.2.2.

(b) *Prestressed concrete structures or elements.* These are classified by reference to flexural tensile limitations. The categories are as follows:

- class 1: no tensile stress permitted;
- class 2: tensile stresses permitted, in accordance with table 24, but no visible cracking;
- class 3: tensile stress permitted, in accordance with table 25, but with design crack widths limited to the values of table 1.

A structure may not have a unique classification and reference should be made to the appropriate bridge authority for its requirements.

Recommendations for load combinations appropriate to particular categories are given in 4.2.2.

4.1.1.2 Vibration. The dynamic effects of standard highway or railway loading on commonly occurring types of bridges are deemed to be covered in the allowance for impact which is included in the nominal live load; the requirements of Part 2 need only be applied to foot-bridges or to the structural components of bridges whose prime function is to carry footway loading.

4.1.1.3 Stress limitations. To prevent unacceptable deformations occurring, compressive stresses in concrete and stresses in steel should be calculated by linear elastic analysis for load combinations 1 to 5 in any of the

following applications:

- (a) for all prestressed concrete construction;
- (b) for all composite concrete construction (see 7.4);
- (c) where the effects of differential settlement, temperature difference, and creep and shrinkage of concrete are not considered at the ultimate limit state;
- (d) where a plastic method (see 4.4.3.1) or redistribution of moments (see 5.2.2) are used for the analysis of the structure at the ultimate limit state;
- (e) under global and local direct and bending effects where these are considered separately at the ultimate limit state.

For reinforced and prestressed concrete, the compressive and tensile stress limitations are summarized in table 2.

4.1.2 Ultimate limit states

4.1.2.1 Rupture or instability. The assessment of the structure under the design loads appropriate to this limit state should ensure that prior collapse does not occur as a result of rupture of one or more critical sections, buckling caused by elastic or plastic instability, or overturning.

The effects of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state provided that these effects have been included in the appropriate load combinations to check the stress limitations given in 4.1.1.3 for the serviceability limit state.

4.1.2.2 Vibration. The criterion for vibration at the serviceability limit state is deemed to satisfy this clause.

Table 1. Design crack widths

Environment	Examples	Design crack width
Extreme Concrete surfaces exposed to: abrasive action by sea water or water with a pH ≤ 4.5	Marine structures Parts of structure in contact with moorland water	mm 0.10
Very severe Concrete surfaces directly affected by: de-icing salts or sea water spray	Walls and structure supports adjacent to the carriageway Parapet edge beams Concrete adjacent to the sea	0.15
Severe Concrete surfaces exposed to: driving rain or alternate wetting and drying	Wall and structure supports remote from the carriageway Bridge deck soffits Buried parts of structures	0.25
Moderate Concrete surfaces above ground level and fully sheltered against all of the following: rain, de-icing salts, sea water spray Concrete surfaces permanently saturated by water with a pH > 4.5	Surface protected by bridge deck water-proofing or by permanent formwork Interior surface of pedestrian subways, voided superstructures or cellular abutments Concrete permanently under water	0.25

Table 2. Stress limitations for the serviceability limit state

Material	Type of stress under design loading	Type of construction	
		Reinforced concrete	Prestressed concrete
Concrete	Triangular or near triangular compressive stress distribution (e.g. due to bending)	$0.50f_{cu}$	$0.40f_{cu}$
	Uniform or near uniform compressive stress distribution (e.g. due to axial loading)	$0.38f_{cu}$	$0.30f_{cu}$
Reinforcement	Compression } Tension }	$0.75f_y$	Not applicable
Prestressing tendons	Tension	Not applicable	Deemed to be satisfied by 6.8.1

NOTE. See 7.3.4 for limiting flexural stresses in joints for post-tensioned segmental construction.

4.1.3 Other considerations

4.1.3.1 Deflections. The deflection of the structure, or any part of it, should not be such as to affect adversely the appearance or efficiency of the structure.

The appearance and function of concrete superstructures are normally unaffected although calculations may be required in the following circumstances:

- (a) where minimum specified clearances may be violated;
- (b) where drainage difficulties might ensue;
- (c) where the method of construction may require careful control of profile, e.g. at discontinuities in serial construction, and where decks comprise abutting prestressed concrete beams.

4.1.3.2 Fatigue. The fatigue life of all bridges should comply with the recommendations of Part 10 (but see also 4.7).

4.1.3.3 Durability. The recommendations in this code regarding concrete cover to the reinforcement and acceptable crack widths (see 4.1.1.1) in association with the cement contents given in Part 8 are intended to meet the durability requirements of almost all structures. Where more severe environments are encountered, however, additional precautions may be necessary, and specialist literature should be consulted.

4.2 Loads, load combinations and partial factors γ_{fL} and γ_{f3}

4.2.1 Loads. The nominal values of loads (and imposed deformations) are given in Part 2.

Creep and shrinkage of concrete and prestress (including secondary effects in statically indeterminate structures) are load effects associated with the nature of the structural material being used; where they occur, they should be regarded as permanent loads.

4.2.2 Serviceability limit state. For the limitations given in 4.1.1.1(a) for reinforced concrete, load combination 1 only should be considered. Where type HB loading is to be taken into account, only 25 units should be considered. The type HA wheel load need not be considered except for cantilever slabs and the top flanges in beam-and-slab, voided slab and box-beam construction.

For the limitations given in 4.1.1.1(b) for prestressed concrete, the load combinations considered for the appropriate category should be as follows:

- (a) class 1 under load combination 1 (as modified in the preceding paragraph), except that live loading may be ignored for lightly trafficked highway bridges and railway bridges where the live loading is controlled; and
- (b) class 2 or class 3 for load combinations 2 to 5.

For the stress limitations given in 4.1.1.3, load combinations 1 to 5 should be considered.

The values of the partial safety factor, γ_{fL} , for the individual loadings comprising combinations 1 to 5 are given in Part 2. The value of γ_{fL} for creep and shrinkage of concrete and for prestress (including secondary effects in statically indeterminate structures) should be taken as 1.0.

The value of γ_{f3} should be taken as 1.0.

4.2.3 Ultimate limit state. To check the recommendations of 4.1.2, load combinations 1 to 5 should be considered.

The values of the partial safety factor, γ_{fL} , for the individual loadings comprising combinations 1 to 5 are given in Part 2. The values of γ_{fL} for the effects of shrinkage and, where relevant, of creep should be taken as 1.2. In calculating the resistance of members to vertical shear and torsion, γ_{fL} for the prestressing force should be taken as 1.15 where it adversely affects the resistance and 0.87 in other cases. In calculating secondary effects in statically indeterminate structures, γ_{fL} for the prestressing force may be taken as 1.0.

The value of γ_{f3} should be taken as 1.10, except that where plastic methods (see 4.4.3.1) are used for the analysis of the structure, γ_{f3} should be taken as 1.15.

4.2.4 Deflection. Minimum specified clearances should be maintained under the action of load combination 1.

The appearance and drainage characteristics of the structure should be considered under the action of permanent loads only.

The values of γ_{fL} for the individual loads and the value of γ_{f3} should be those appropriate to the serviceability limit state.

4.3 Properties of materials

4.3.1 General. The characteristic strengths of materials are given in 5.1.4 and 6.1.4. In general, in analysing a structure to determine load effects, the material properties appropriate to the characteristic strength should be used, irrespective of the limit state being considered.

For the analysis of sections, the material properties to be used for the individual limit states are as follows.

(a) *Serviceability limit state.* The characteristic stresses, which should be taken as $0.75f_y$ for reinforcement, $0.5f_{cu}$ for concrete in compression and $0.56\sqrt{f_{cu}}$ for tension in concrete for prestressed concrete.

(b) *Ultimate limit state.* The characteristic strengths given in 4.3.2.

The appropriate γ_m values are given in 4.3.3.

4.3.2 Material properties

4.3.2.1 Concrete. In assessing the strength of sections at the ultimate limit state, the design stress-strain curve for normal weight concrete may be taken from figure 1, using the value of γ_m for concrete given in 4.3.3.3.

The modulus of elasticity to be used for elastic analysis should be appropriate to the cube strength of the concrete at the age considered and in the absence of special investigations should be obtained as follows.

(a) *Analysis of structure.* To determine the effects of permanent and short term loading: the appropriate value given in table 3.

To determine the effects of imposed deformations and for the calculation of deflections: an appropriate intermediate value between that given in table 3 and half that value.

(b) *Analysis of section.* To determine crack widths and stresses due to the effects of permanent and short term loading and imposed deformations: an appropriate intermediate value between that given in table 3 and half that value.

The appropriate intermediate value of the modulus of elasticity mentioned in (a) and (b) should reflect the proportion of permanent and short term effects in the combination.

Where a more precise elastic analysis for permanent loads is appropriate, the effects of creep and shrinkage should be assessed more accurately. Information on creep and shrinkage, based on CEB/FIP International Recommendations, is given in appendix C.

Further information on the elastic deformation of concrete is given in appendix B.

Table 3. Modulus of elasticity of concrete under short term loading

Characteristic strength (or cube strength of concrete at the age considered)	Modulus of elasticity of concrete, E_c
N/mm ²	kN/mm ²
20	25
25	26
30	28
40	31
50	34
60	36

The effect of creep under long term loading is normally allowed for by using half the values in table 3 for the modulus of elasticity.

For lightweight concrete having an air dry density between 1400 kg/m^3 and 2300 kg/m^3 , the values given in table 3 should be multiplied by $(D_c/2300)^2$ where D_c is the density of the lightweight aggregate concrete in kg/m^3 . Alternatively, the value of the elastic modulus for a specific strength may be obtained from tests.

For elastic analysis, Poisson's ratio may be taken as 0.2. The coefficient of thermal expansion may be taken as $12 \times 10^{-6} / ^\circ\text{C}$ for normal weight concrete. For lightweight concrete and limestone aggregate concrete, the values can be as low as $7 \times 10^{-6} / ^\circ\text{C}$ and $9 \times 10^{-6} / ^\circ\text{C}$ respectively.

4.3.2.2 Reinforcement and prestressing steel. The design stress-strain curves may be taken as follows:

(a) for reinforcement, from figure 2, using the value of γ_m given in 4.3.3;

(b) for prestressing steel, from figure 3 or figure 4, using the value of γ_m given in 4.3.3.

For reinforcement, the modulus of elasticity may be taken as 200 kN/mm^2 for both short and long term loading. For prestressing steel, the short and long term modulus may be taken from figure 3 or figure 4 as the appropriate tangent modulus at zero load.

4.3.3 Values of γ_m

4.3.3.1 General. For the analysis of sections, the values of γ_m are summarized in 4.3.3.2 to 4.3.3.4.

4.3.3.2 Serviceability limit state. The values of γ_m applied to the characteristic stresses defined in 4.3.1 are given in table 4 and have been allowed for in deriving the compressive and tensile stresses given in tables 2 and 24.

The higher values for prestressed concrete arise because the whole concrete cross section is normally in compression and therefore creep will be greater than in reinforced concrete. Similarly in reinforced concrete creep will be greater where the compressive stress distribution is uniform over the whole cross section.

4.3.3.3 Ultimate limit state. For both reinforced concrete and prestressed concrete, the values of γ_m applied to the characteristic strengths (see 4.3.1) are 1.50 for concrete and 1.15 for reinforcement and prestressing tendons.

4.3.3.4 Fatigue. For reinforced concrete, the value of γ_m applied to the stress range limitations given in 4.7 for reinforcement is 1.00.

4.4 Analysis of structure

4.4.1 General. The requirements of methods of analysis appropriate to the determination of the distribution of forces and deformations which are to be used in ascertaining that the limit state criteria are satisfied are described in Part 1. The effects of shear lag on the analysis of the structure may be neglected, except for cable-stayed superstructures.

4.4.2 Analysis for serviceability limit state

4.4.2.1 General. Elastic methods of analysis should be used to determine internal forces and deformations. The flexural stiffness constants (second moment of area) for sections of discrete members or unit widths of slab elements may be based on any of the following.

(a) *Concrete section.* The entire member cross section, ignoring the presence of reinforcement.

Table 4. Values of γ_m for the serviceability stress limitations

Material	Type of stress	Type of construction	
		Reinforced concrete	Prestressed concrete
Concrete	Triangular or near-triangular compressive stress distribution (e.g. due to bending)	1.00	1.25
	Uniform or near-uniform compressive stress distribution (e.g. due to axial loading)	1.33	1.67
	Tension	Not applicable	1.25 pre-tensioned 1.55 post-tensioned
Reinforcement	Compression Tension	1.00	Not applicable
Prestressing tendons	Tension	Not applicable	Not required

(b) *Gross transformed section.* The entire member cross section including the reinforcement, transformed on the basis of modular ratio.

(c) *Net transformed section.* The area of the cross section which is in compression together with the tensile reinforcement, transformed on the basis of modular ratio.

A consistent approach should be used which reflects the different behaviour of various parts of the structure.

Axial, torsional and shearing stiffness constants, when required by the method of analysis, should be based on the concrete section and used with (a) or (b).

Moduli of elasticity and shear moduli values should be appropriate to the characteristic strength of the concrete.

4.4.2.2 Methods of analysis and their requirements.

The method of analysis should ideally take account of all the significant aspects of behaviour of a structure governing its response to loads and imposed deformations.

4.4.3 Analysis for ultimate limit state

4.4.3.1 General. Elastic methods may be used to determine the distribution of forces and deformations throughout the structure. Stiffness constants should be based on the section properties as used for the analysis of the structure at the serviceability limit state (see 4.4.2.1). However, plastic methods of analysis (e.g. plastic hinge methods for beams, or yield line methods for slabs) may be used where such methods can be shown adequately to model the combined local and global effects of all loads due to combinations 1 to 5 as given in Part 2. The use of such methods should be agreed with the relevant bridge authority.

4.4.3.2 Methods of analysis and their requirements.

The application of elastic methods of analysis in association with the design loads for the ultimate limit state in general leads to safe lower bound solutions; these may be refined, and made less conservative, by reference to test results or specialist literature. For longitudinal members load effects due to restraint of torsional and distortional warping may be neglected.

When treating local effects, elastic methods may be applied to derive the in-plane forces and moments due to

out-of-plane loading; alternatively, yield line methods may be used for the local wheel loads. The section should then be detailed according to clauses 5, 6 and 7, as appropriate.

4.5 Analysis of section

4.5.1 Serviceability limit state. At any section, an elastic analysis should be carried out to satisfy recommendations of 4.1.1. In-plane shear flexibility in concrete flanges (shear lag effects) should be allowed for. This may be done by taking an effective width of flange (see 5.3.1.2).

4.5.2 Ultimate limit state. The strength of critical sections should be assessed in accordance with clause 5, 6 or 7 to satisfy the recommendations of 4.1.2. In-plane shear flexibility in concrete flanges (shear lag effects) may be ignored.

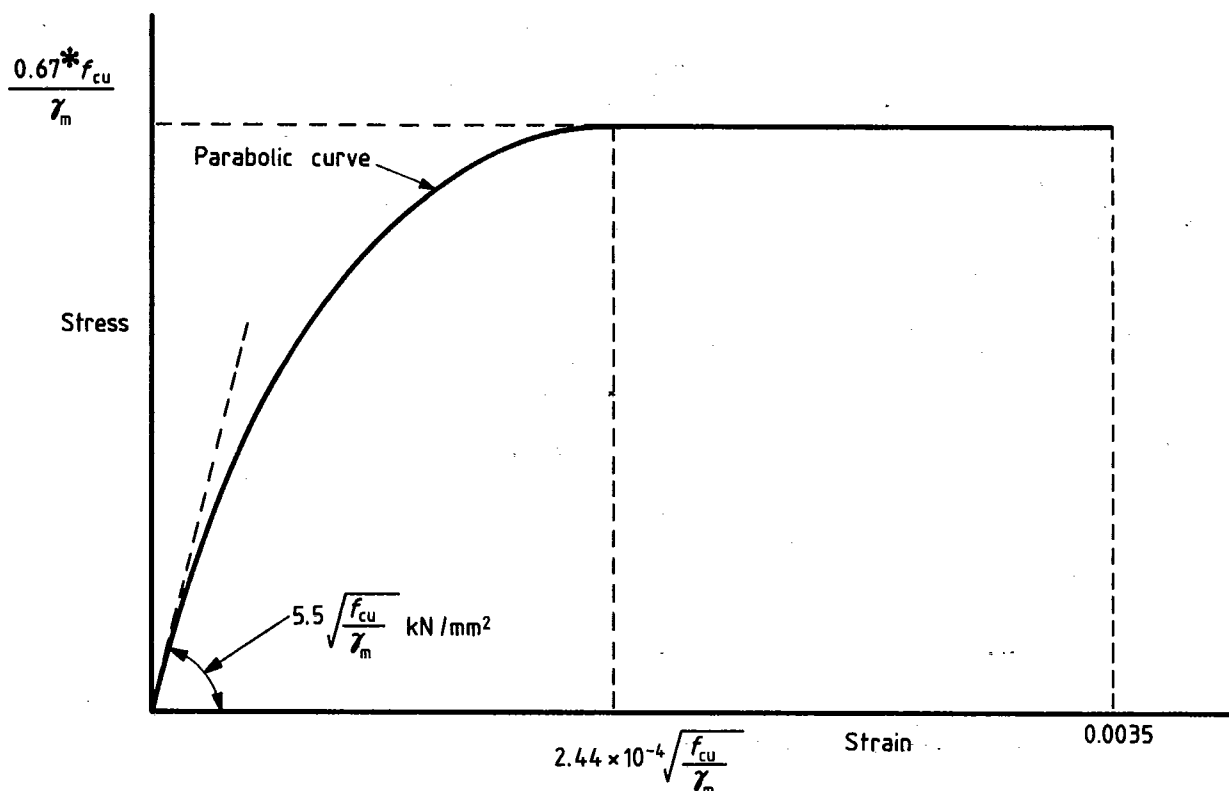
4.6 Deflection. Deflection should be calculated for the most unfavourable distributions of loading for the member (or strip of slab) and may be derived from an elastic analysis of the structure. The material properties, stiffness constants and calculation of deflections may be based on the information given in 4.3.2.1 or in appendix A.

4.7 Fatigue. The effect of repeated live loading on the fatigue strength of a bridge should be considered in respect of reinforcing bars that have been subjected to welding; details of compliance criteria are given in Part 10.

Welding may be used to connect bars subjected to fatigue loading provided that:

- (a) the connection is made to standard workmanship levels as given in Part 7;
- (b) the welded bar is not part of a deck slab spanning between longitudinal and/or transverse members and subjected to the effect of concentrated wheel loads in a traffic lane;
- (c) the detail has an acceptable fatigue life determined in accordance with Part 10;
- (d) lap welding is not used.

For unwelded reinforcing bars, the stress range under load combinations 1 to 5 for the serviceability limit state should be limited to 325 N/mm² for grade 460 bars and to 265 N/mm² for grade 250 bars.



*0.67 takes account of the ratio between the characteristic cube strength and the bending strength in a flexural member.

NOTE 1. f_{cu} is in N/mm^2 .

NOTE 2. The equation for the parabolic curve between $\epsilon = 0$ and $2.44 \times 10^{-4} \sqrt{f_{cu}/\gamma_m}$ may be taken as:

$$f = \left(5500 \sqrt{\frac{f_{cu}}{\gamma_m}} \right) \epsilon - \left(\frac{5500^2}{2.68} \right) \epsilon^2$$

where

f is the stress;

ϵ is the strain.

Figure 1. Short term design stress-strain curve for normal weight concrete

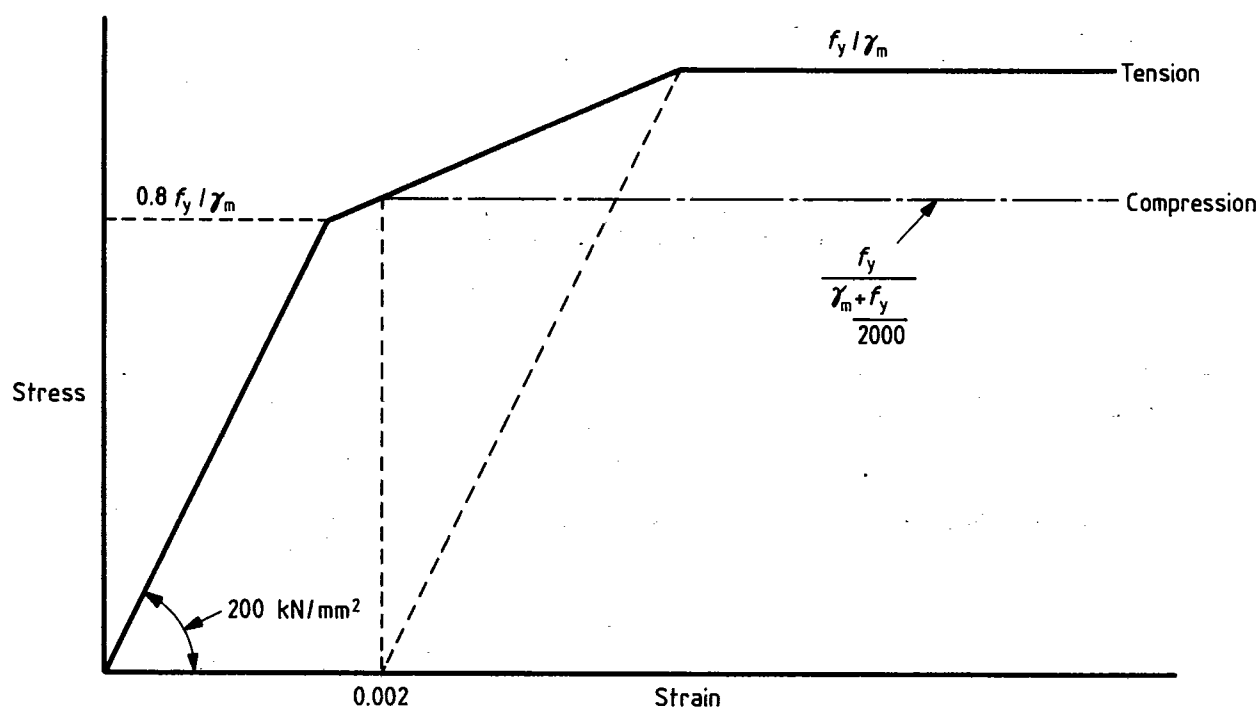


Figure 2. Short term design stress-strain curve for reinforcement

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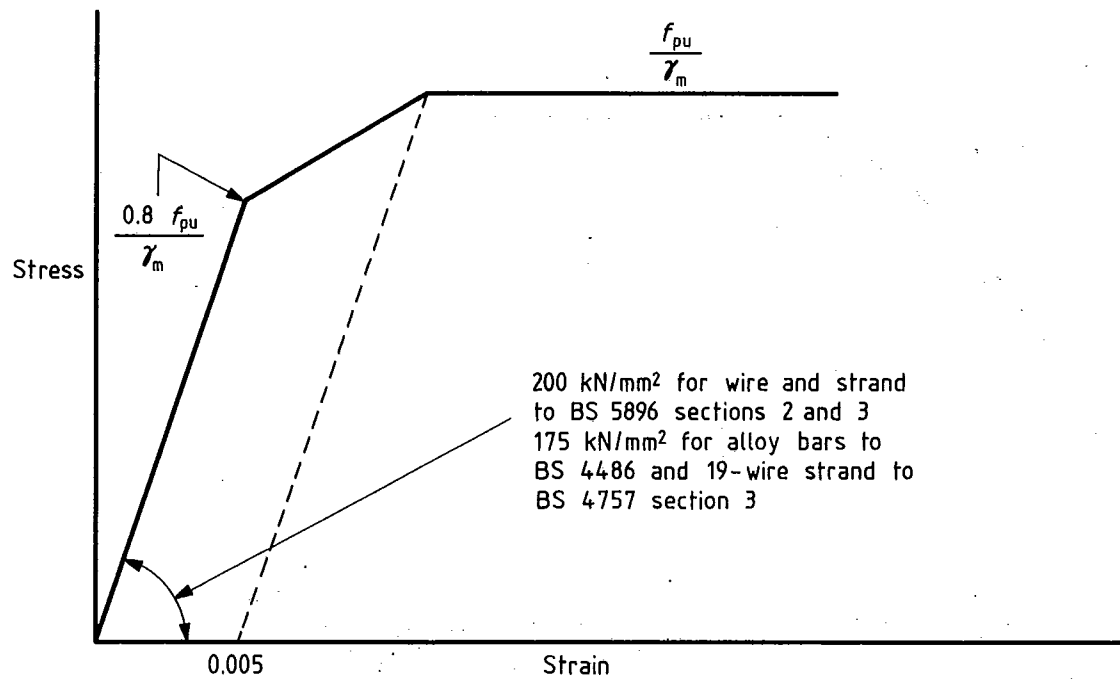


Figure 3. Short term design stress-strain curve for normal and for low relaxation products

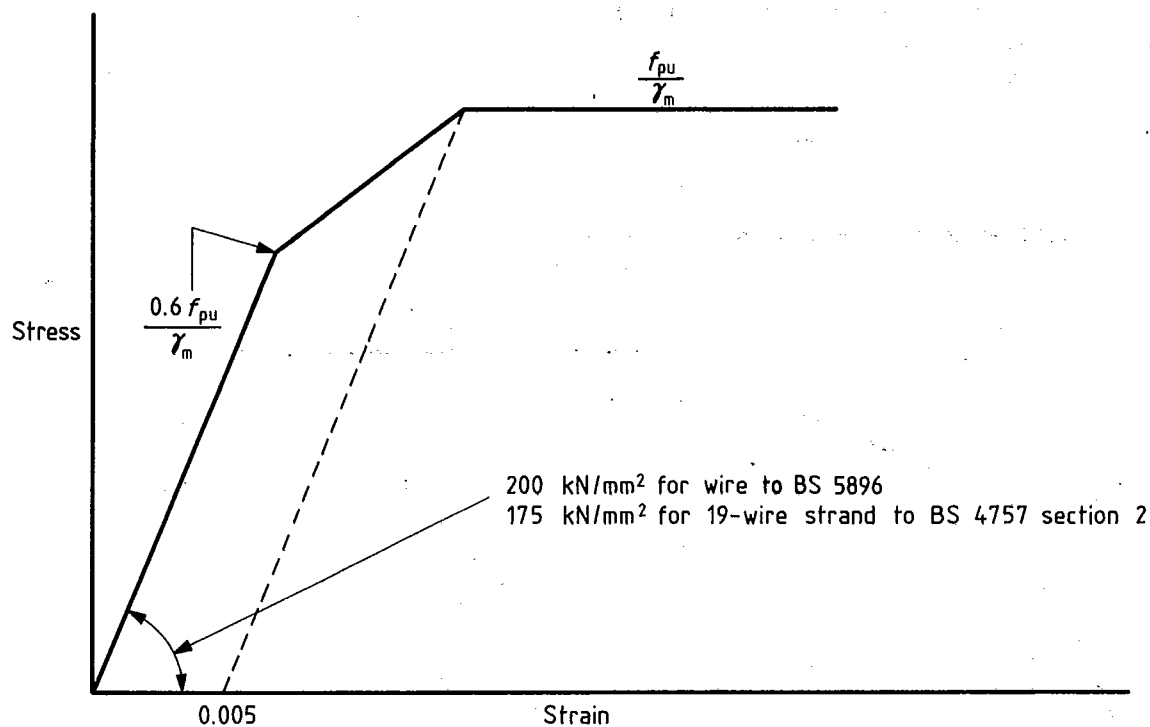


Figure 4. Short term design stress-strain curve for 'as drawn' wire and 'as spun' strand

4.8 Combined global and local effects

4.8.1 General. In addition to the design of individual primary and secondary elements to resist loading applied directly to them, it is also necessary to consider the loading combination that produces the most adverse effects due to global and local loading where these coexist in an element.

4.8.2 Analysis of structure. Analysis of the structure may be accomplished either by one overall analysis (e.g. using finite elements) or by separate analyses for global and local effects. In the latter case the forces and moments acting on the element from global and local effects should be combined as appropriate.

4.8.3 Analysis of section. Section analysis for the combined global and local effects should be carried out in accordance with 4.5 to satisfy the recommendations of 4.1.

(a) Serviceability limit state

(1) For reinforced concrete elements, the total crack width due to combined global and local effects should be determined in accordance with 5.8.8.2.

(2) For prestressed concrete elements, coexistent stresses, acting in the direction of prestress, may be added algebraically in checking the stress limitations.

(b) *Ultimate limit state.* The resistance of the section to direct and flexural effects should be derived from the direct strain due to global effects combined with the flexural strain due to local effects. However, in the case of a deck slab the resistance to combined global and local effects is deemed to be satisfactory if each of these effects is considered separately.

5. Design and detailing: reinforced concrete

5.1 General

5.1.1 Introduction. This clause gives methods of analysis and design which will in general ensure that, for reinforced concrete structures, the recommendations set out in 4.1.1 and 4.1.2 are met. Other methods may be used provided they can be shown to be satisfactory for the type of structure or member considered. In certain cases the assumptions made in this clause may be inappropriate and the Engineer should adopt a more suitable method having regard to the nature of the structure in question.

5.1.2 Limit state design of reinforced concrete

5.1.2.1 Basis of design. Clause 5 follows the limit state philosophy set out in clauses 3 and 4 but, as it is not possible to assume that a particular limit will always be the critical one, design methods are given for both the ultimate and the serviceability limit states.

In general, the design of reinforced concrete members is controlled by the ultimate limit state, but the limitations on crack width and, where applicable, stresses at the serviceability limit state given in 4.1.1.3 should also be met.

Where a plastic method or redistribution of moments is used for the analysis of the structure at the ultimate limit state, or where critical parts of the structure are subjected to the 'extreme' or 'very severe' category of exposure, the design is likely to be controlled by the serviceability limit state of cracking.

5.1.2.2 Durability. In 5.8.2 guidance is given on the nominal cover to reinforcement that should be provided to ensure durability.

5.1.2.3 Other limit states and considerations. Clause 5 does not make recommendations concerning 'vibration' or 'other limit states' and, for these and other considerations, reference should be made to 4.1.1.2 and 4.1.3.

5.1.3 Loads. In clause 5 the design load effects (see 2.1) for the ultimate and serviceability limit states are referred to as 'ultimate loads' and 'service loads' respectively.

The values of the 'ultimate loads' and 'service loads' to be used in design are derived from Part 2 and 4.2.

In clause 5, when analysing sections, the terms 'strength', 'resistance' and 'capacity' are used to describe the design strength of the section.

5.1.4 Strength of materials

5.1.4.1 Definition of strengths. In clause 5 the design strengths of materials for the ultimate limit state are expressed in all the tables and equations in terms of the 'characteristic strength' of the material. Unless specifically stated otherwise, all equations, figures and tables include allowances for γ_m , the partial safety factor for material strength.

5.1.4.2 Characteristic strength of concrete. The characteristic cube strengths of concrete are given in Part 7, and those which may be specified for reinforced concrete are quoted in table 5, together with their related cube strengths at other ages. The values given in table 5 do not include any allowance for γ_m .

Design should be based on the characteristic strength.

5.1.4.3 Characteristic strength of reinforcement.

The characteristic strengths of reinforcement are given in the appropriate British Standards and are quoted in table 6.

The values given in table 6 do not include any allowance for γ_m .

5.2 Structures and structural frames

5.2.1 Analysis of structures. Structures should be analysed in accordance with the recommendations of 4.4.

5.2.2 Redistribution of moments. Redistribution of moments obtained by rigorous elastic analysis under the ultimate limit state may be carried out provided the following conditions are met.

(a) Checks are made to ensure that adequate rotation capacity exists at sections where moments are reduced, making reference to appropriate test data.

In the absence of a special investigation, the plastic rotation capacity may be taken as the lesser of:

$$(1) 0.008 + 0.035 \left(0.5 - \frac{d_c}{d_e} \right)$$

or

$$(2) \frac{0.6\Phi}{d - d_c}$$

but not less than 0 or more than 0.015

where

d_c is the calculated depth of concrete in compression at the ultimate limit state;

d_e is the effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange;

Φ is the diameter of the smallest tensile reinforcing bar;

d is the effective depth to tension reinforcement.

Table 5. 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/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²
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

* Increased strengths at these ages should be used only if the Engineer is satisfied that the materials used are capable of producing these higher strengths.

Table 6. Strength of reinforcement

Designation	Nominal sizes	Characteristic strength, f_y
Hot rolled grade 250 (BS 4449)	mm All sizes	N/mm ² 250
Hot rolled grade 460 (BS 4449)	All sizes	460
Cold worked (BS 4461)		
Hard drawn steel wire (BS 4482)	Up to and including 12	485

(b) Proper account is taken of changes in transverse moments, transverse deflections and transverse shears consequent on redistribution of longitudinal moments by means of a special investigation based on a non-linear analysis.

(c) Shears and reactions used in design are taken as those calculated either prior to redistribution or after redistribution, whichever is greater.

(d) The depth of the members of elements considered is less than 1.2 m.

5.3 Beams

5.3.1 General

5.3.1.1 Effective span. The effective span of a simply supported member should be taken as the smaller of:

- (a) the distance between the centres of bearings or other supports; or
- (b) the clear distance between supports plus the effective depth.

The effective span of a member framing into supporting members should be taken as the distance between the shear centres of the supporting member.

The effective span of a continuous member should be taken as the distance between centres of supports except where, in the case of beams on wide columns, the effect of column width is included in the analysis.

The effective length of a cantilever should be taken as its length from the face of the support plus half its effective depth except where it is an extension of a continuous beam when the length to the centre of the support should be used.

5.3.1.2 Effective width of flanged beams. In analysing structures, the full width of flanges may be taken as effective.

In analysing sections at the serviceability limit state, and in the absence of any more accurate determination (such as that given in Part 3), the effective flange width should be taken as the width of the web plus one-tenth of the distance between the points of zero moment (or the actual width of the outstand if this is less) on each side of the web. For a continuous beam the points of zero moment may be taken to be at a distance of 0.15 times the effective span from the support.

In analysing sections at the ultimate limit state the full width of the flanges may be taken as effective.

5.3.1.3 Slenderness limits for beams. To ensure lateral stability, a simply supported or continuous beam should be so proportioned that the clear distance between lateral restraints does not exceed $60b_c$ or $250b_c^2/d$, whichever is the lesser,

where

d is the effective depth to tension reinforcement;

b_c is the breadth of the compression face of the beam midway between restraints.

For cantilevers with lateral restraint provided only at the support, the clear distance from the end of the cantilever to the face of the support should not exceed $25b_c$ or $100b_c^2/d$, whichever is the lesser.

5.3.2 Resistance moment of beams

5.3.2.1 Analysis of sections. When analysing a cross section to determine its ultimate moment of resistance

the following assumptions should be made.

(a) The strain distribution in the concrete in compression and the strains in the reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane.

(b) The stresses in the concrete in compression are either derived from the stress-strain curve in figure 1 with $\gamma_m = 1.5$ or, in the case of rectangular sections and in flanged, ribbed and voided sections where the neutral axis lies within the flange, the compressive stress may be taken as equal to $0.4f_{cu}$ over the whole compression zone. In both cases the strain at the outermost compression fibre at failure is taken as 0.0035.

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

(d) The stresses in the reinforcement are derived from the stress-strain curves in figure 2 with $\gamma_m = 1.15$.

In addition, if the ultimate moment of resistance, calculated in accordance with this clause, is less than 1.15 times the required value, the section should be proportioned such that the strain at the centroid of the tensile reinforcement is not less than:

$$0.002 + \frac{f_y}{E_s \gamma_m}$$

where E_s is the modulus of elasticity of the steel.

As an alternative, the strains in the concrete and the reinforcement, due to the application of ultimate loads, may be calculated using the following assumptions.

(e) The strain distribution in the concrete in compression and the strains in the reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane.

(f) The stresses in the concrete in compression are derived from the stress-strain curve given in figure 1, with $\gamma_m = 1.5$.

(g) The tensile strength of the concrete is ignored.

(h) The stresses in the reinforcement are derived from the stress-strain curves in figure 2 with $\gamma_m = 1.15$.

In using the alternative method of analysis, the calculated strain due to the application of ultimate loads at the outermost compression fibre of the concrete should not exceed 0.0035 and the strain at the centroid of the tensile reinforcement should be not less than $0.002 + f_y/(E_s \gamma_m)$ except where the requirement for the calculated strain in the concrete, due to the application of 1.15 times the ultimate loads, can be satisfied.

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

5.3.2.2 Design charts. The design charts that form Parts 2 and 3 of CP 110 include charts, based on figure 1, figure 2 and the assumptions in 5.3.2.1, which may be used for the design of beams reinforced in tension only or in tension and compression.

5.3.2.3 Design formulae. Provided that the amount of redistribution of the elastic ultimate moments has been less than 10 %, the following formulae may be used to calculate the ultimate moment of resistance of a solid slab or rectangular beam, or of a flanged beam, ribbed slab or voided slab when the neutral axis lies within the flange.

For sections without compression reinforcement the ultimate moment of resistance may be taken as the lesser of the values obtained from equations 1 and 2. Equations 3 and 4 may be used for sections with compression reinforcement.

A rectangular stress block of maximum depth $0.5d$ and a uniform compression stress of $0.4f_{cu}$ has been assumed.

$$M_u = (0.87f_y)A_s z \quad \text{equation 1}$$

$$M_u = 0.15f_{cu}bd^2 \quad \text{equation 2}$$

$$M_u = 0.15f_{cu}bd^2 + 0.72f_yA'_s(d - d') \quad \text{equation 3}$$

$$(0.87f_y)A_s = 0.2f_{cu}bd + 0.72f_yA'_s \quad \text{equation 4}$$

where

M_u is the ultimate resistance moment;

A_s is the area of tension reinforcement;

A'_s is the area of compression reinforcement;

b is the width of the section;

d is the effective depth to the tension reinforcement;

d' is the depth to the compression reinforcement;

f_y is the characteristic strength of the reinforcement;

z is the lever arm;

f_{cu} is the characteristic strength of the concrete.

When d'/d is greater than 0.2, equation 3 should not be used and the resistance moment should be calculated with the aid of 5.3.2.1.

The term $0.72f_y$ in equations 3 and 4 is a simplification of the equation:

$$\frac{f_y}{\gamma_m + f_y/2000} \text{ shown on figure 2.}$$

The lever arm, z , in equation 1 may be calculated from the equation:

$$z = \left(1 - \frac{1.1f_yA_s}{f_{cu}bd}\right) d \quad \text{equation 5}$$

The value z should not be taken as greater than $0.95d$.

The ultimate resistance moment of a flanged beam may be taken as the lesser of the values given by equations 6 and 7, where h_f is the thickness of the flange.

$$M_u = (0.87f_y)A_s \left(d - \frac{h_f}{2}\right) \quad \text{equation 6}$$

$$M_u = 0.4f_{cu}bh_f \left(d - \frac{h_f}{2}\right) \quad \text{equation 7}$$

Where it is necessary for the resistance moment to exceed the value given by equation 7, the section should be analysed in accordance with 5.3.2.1.

5.3.3 Shear resistance of beams

5.3.3.1 Shear stress. The shear stress, v , at any cross section should be calculated from:

$$v = \frac{V}{bd} \quad \text{equation 8}$$

where

V is the shear force due to ultimate loads;

b is the breadth of the section which, for a flanged beam, should be taken as the rib width;

d is the effective depth to tension reinforcement.

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In no case should v exceed $0.75\sqrt{f_{cu}}$ or 4.75 N/mm^2 , whichever is the lesser, whatever shear reinforcement is provided.

5.3.3.2 Shear reinforcement. Shear reinforcement should be provided as given in table 7.

Table 7. Form and area of shear reinforcement in beams

Value of v (N/mm^2)	Area of vertical shear reinforcement to be provided (mm^2)
$v \leq \xi_s v_c$	$A_{sv} \geq 0.4bs_v/0.87f_{yv}$
$v > \xi_s v_c$	$A_{sv} \geq bs_v(v + 0.4 - \xi_s v_c)/0.87f_{yv}$

In table 7:

- v is the shear stress;
- ξ_s is the depth factor (see table 9);
- v_c is the ultimate shear stress in concrete (see table 8);
- A_{sv} is the cross-sectional area of all the legs of the links at a particular cross section;
- s_v is the spacing of the links along the member;
- f_{yv} is the characteristic strength of link reinforcement but not greater than 460 N/mm^2 .

Where links combined with bent-up bars are used for shear reinforcement, not more than 50 % of the shear force $(v + 0.4 - \xi_s v_c)bd$ should be resisted by bent-up bars. These bars should be assumed to form the tension members of one or more single systems of lattice girders in which the concrete forms the compression members. The maximum stress in any bar should be taken as $0.87f_y$. The shear resistance at any vertical section should be taken as the sum of the vertical components of the tension and compression forces cut by the section. Bars should be checked for anchorage (see 5.8.6.3) and bearing (see 5.8.6.8).

Table 8. Ultimate shear stress in concrete, v_c

$\frac{100A_s}{bd}$	Concrete grade			
	20	25	30	40 or more
N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
≤ 0.15	0.31	0.34	0.36	0.39
0.25	0.37	0.40	0.42	0.47
0.50	0.47	0.50	0.53	0.59
1.00	0.59	0.63	0.67	0.74
2.00	0.74	0.80	0.85	0.93
≥ 3.00	0.85	0.91	0.97	1.06

NOTE. $b = b_s$ for punching shear cases (see figure 5).

Table 8 is derived from the following relationship:

$$v_c = \frac{0.27}{\gamma_m} \left(\frac{100A_s}{b_w d} \right)^{1/3} (f_{cu})^{1/3}$$

where γ_m is taken as 1.25 and f_{cu} should not exceed 40.

The term A_s in table 8 is that area of longitudinal reinforcement which continues at least a distance equal

to the effective depth beyond the section being considered, except at supports where the full area of tension reinforcement may be used provided the recommendations of 5.8.7 are met.

Where both top and bottom reinforcement is provided the area of A_s used should be that which is in tension under the loading which produces the shear force being considered.

Table 9. Values of ξ_s

Effective depth, d	ξ_s
2000 or more	0.70
1500	0.75
1000	0.85
500	1.00
400	1.05
300	1.15
250	1.20
200	1.25
150	1.35
≤ 100	1.50

Table 9 is derived from the following relationship:

$$\xi_s = (500/d)^{1/4} \text{ or } 0.70, \text{ whichever is the greater.}$$

The spacing of the legs of links, in the direction of the span and at right-angles to it, should not exceed $0.75d$.

The area of longitudinal reinforcement in the tensile zone should be such that:

$$A_s \geq \frac{V}{2(0.87f_y)}$$

where

A_s is the area of effectively anchored longitudinal tensile reinforcement (see 5.8.7);

f_y is the characteristic strength of the reinforcement;

V is the shear force due to ultimate loads at the point considered.

5.3.3.3 Enhanced shear strength of sections close to supports. An enhancement of shear strength may be allowed for sections within a distance $a_v < 2d$ from the face of a support, front edge of a rigid bearing or centre line of a flexible bearing.

This enhancement should take the form of an increase in the allowable shear stress, $\xi_s v_c$ to $\xi_s v_c \times 2d/a_v$, but should not exceed $0.75\sqrt{f_{cu}}$ or 4.75 N/mm^2 , whichever is the lesser.

Where this enhancement is used the main reinforcement at the section considered should continue to the support and be provided with an anchorage equivalent to 20 times the bar size.

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

5.3.4 Torsion

5.3.4.1 General. Torsion does not usually decide the dimension of members, therefore torsion design should be carried out as a check, after the flexural design. This is