

Steel, concrete and composite bridges —

Part 4: Code of practice for design of concrete bridges

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Contents

	Page
Committees responsible	Inside front cover
Foreword	x
1 Scope	1
2 Definitions and symbols	1
2.1.1 General	1
2.1.2 Partial load factors	1
2.1.3 Materials	1
2.2 Symbols	1
3 Limit state philosophy	5
3.1 General	5
3.2 Serviceability limit state	5
3.3 Ultimate limit state	5
4 Design: general	5
4.1 Limit state requirements	5
4.1.1 Serviceability limit states	5
4.1.1.1 Cracking	5
4.1.1.2 Vibration	6
4.1.1.3 Stress limitations	6
4.1.2 Ultimate limit states	6
4.1.2.1 Rupture or instability	6
4.1.2.2 Vibration	6
4.1.3 Other considerations	8
4.1.3.1 Deflections	8
4.1.3.2 Fatigue	8
4.1.3.3 Durability	8
4.2 Loads, load combinations and partial factors γ_{fL} and γ_{f3}	8
4.2.1 Loads	8
4.2.2 Serviceability limit state	8
4.2.3 Ultimate limit state	9
4.2.4 Deflection	9
4.3 Properties of materials	9
4.3.1 General	9
4.3.2 Material properties	9
4.3.2.1 Concrete	9
4.3.2.2 Reinforcement and prestressing steel	10
4.3.3 Values of γ_m	10
4.3.3.1 General	10
4.3.3.2 Serviceability limit state	10
4.3.3.3 Ultimate limit state	10
4.3.3.4 Fatigue	10
4.4 Analysis of structure	10
4.4.1 General	10
4.4.2 Analysis for serviceability limit state	10
4.4.2.1 General	10
4.4.2.2 Methods of analysis and their requirements	11
4.4.3 Analysis for ultimate limit state	11

	Page
4.4.3.1 General	11
4.4.3.2 Methods of analysis and their requirements	11
4.5 Analysis of section	11
4.5.1 Serviceability limit state	11
4.5.2 Ultimate limit state	11
4.6 Deflection	11
4.7 Fatigue	11
4.8 Combined global and local effects	14
4.8.1 General	14
4.8.2 Analysis of structure	14
4.8.3 Analysis of section	14
5 Design and detailing: reinforced concrete	14
5.1 General	14
5.1.1 Introduction	14
5.1.2 Limit state design of reinforced concrete	14
5.1.2.1 Basis of design	14
5.1.2.2 Durability	14
5.1.2.3 Other limit states and considerations	14
5.1.3 Loads	14
5.1.4 Strength of materials	14
5.1.4.1 Definition of strengths	14
5.1.4.2 Characteristic strength of concrete	14
5.1.4.3 Characteristic strength of reinforcement	15
5.2 Structures and structural frames	15
5.2.1 Analysis of structures	15
5.2.2 Redistribution of moments	15
5.3 Beams	16
5.3.1 General	16
5.3.1.1 Effective span	16
5.3.1.2 Effective width of flanged beams	16
5.3.1.3 Slenderness limits for beams	16
5.3.2 Resistance moment of beams	16
5.3.2.1 Analysis of sections	16
5.3.2.2 Design charts	17
5.3.2.3 Design formulae	17
5.3.3 Shear resistance of beams	18
5.3.3.1 Shear stress	18
5.3.3.2 Shear reinforcement	18
5.3.3.3 Enhanced shear strength of sections close to supports	19
5.3.3.4 Bottom loaded beams	19
5.3.4 Torsion	19
5.3.4.1 General	19
5.3.4.2 Torsionless systems	19
5.3.4.3 Stresses and reinforcement	19
5.3.4.4 Treatment of various cross sections	19
5.3.4.5 Detailing	20
5.3.5 Longitudinal shear	21
5.3.6 Deflection in beams	21
5.3.7 Crack control in beams	21

		Page
5.4	Slabs	21
5.4.1	Moments and shear forces in slabs	21
5.4.2	Resistance moments of slabs	21
5.4.3	Resistance to in-plane forces	21
5.4.4	Shear resistance of slabs	21
5.4.4.1	Shear stress in solid slabs: general	21
5.4.4.2	Shear stresses in solid slabs under concentrated loads (including wheel loads)	2
5.4.4.3	Shear in voided slabs	24
5.4.5	Deflection of slabs	24
5.4.6	Crack control in slabs	24
5.5	Columns	24
5.5.1	General	24
5.5.1.1	Definitions	24
5.5.1.2	Effective height of a column	24
5.5.1.3	Slenderness limits for columns	24
5.5.1.4	Assessment of strength	25
5.5.2	Moments and forces in columns	25
5.5.3	Short columns subject to axial load and bending about the minor axis	25
5.5.3.1	General	25
5.5.3.2	Analysis of sections	25
5.5.3.3	Design charts for rectangular and circular columns	25
5.5.3.4	Design formulae for rectangular columns	27
5.5.3.5	Simplified design formulae for rectangular columns	27
5.5.4	Short columns subject to axial load and either bending about the major axis or biaxial bending	27
5.5.5	Slender columns	28
5.5.5.1	General	28
5.5.5.2	Slender columns bent about a minor axis	28
5.5.5.3	Slender columns bent about a major axis	29
5.5.5.4	Slender columns bent about both axes	29
5.5.6	Shear resistance of columns	29
5.5.7	Crack control in columns	29
5.6	Reinforced concrete walls	29
5.6.1	General	29
5.6.1.1	Definition	29
5.6.1.2	Limits to slenderness	30
5.6.2	Forces and moments in reinforced concrete walls	30
5.6.3	Short reinforced walls resisting moments and axial forces	30
5.6.4	Slender reinforced walls	30
5.6.5	Shear resistance of reinforced walls	30
5.6.6	Deflection of reinforced walls	30
5.6.7	Crack control in reinforced walls	30
5.7	Bases	30
5.7.1	General	30
5.7.2	Moments and forces in bases	31
5.7.3	Design of bases	31
5.7.3.1	Resistance to bending	31

	Page
5.7.3.2 Shear	31
5.7.3.3 Bond and anchorage	31
5.7.4 Deflection of bases	31
5.7.5 Crack control in bases	31
5.8 Considerations affecting design details	32
5.8.1 Constructional details	32
5.8.1.1 Sizes of members	32
5.8.1.2 Accuracy of position of reinforcement	32
5.8.1.3 Construction joints	32
5.8.1.4 Movement joints	32
5.8.2 Concrete cover to reinforcement	32
5.8.3 Reinforcement: general considerations	32
5.8.3.1 Groups of bars	32
5.8.3.2 Bar schedule dimensions	32
5.8.4 Minimum areas of reinforcement in members	33
5.8.4.1 Minimum area of main reinforcement	33
5.8.4.2 Minimum area of secondary reinforcement	33
5.8.4.3 Minimum area of links	33
5.8.5 Maximum areas of reinforcement in members	35
5.8.6 Bond anchorage and bearing	35
5.8.6.1 Geometrical classification of deformed bars	35
5.8.6.2 Local bond	35
5.8.6.3 Anchorage bond	35
5.8.6.4 Effective perimeter of a bar or group of bars	35
5.8.6.5 Anchorage of links	36
5.8.6.6 Laps and joints	36
5.8.6.7 Lap lengths	36
5.8.6.8 Hooks and bends	36
5.8.6.9 Bearing stress inside bends	36
5.8.7 Curtailment and anchorage of reinforcement	37
5.8.8 Spacing of reinforcement	37
5.8.8.1 Minimum distance between bars	37
5.8.8.2 Maximum distance between bars in tension	38
5.8.9 Shrinkage and temperature reinforcement	39
5.8.10 Arrangement of reinforcement in skew slabs	39
5.8.10.1 General	39
5.8.10.2 Solid slabs	39
5.8.10.3 Voided slabs	39
5.8.10.4 Solid composite slabs	39
5.9 Additional considerations in the use of lightweight aggregate concrete	39
5.9.1 General	39
5.9.2 Durability	40
5.9.3 Characteristic strength	40
5.9.4 Shear resistance of beams	40
5.9.5 Torsional resistance of slabs	40
5.9.6 Deflection of beams	40
5.9.7 Shear resistance of slabs	40
5.9.8 Deflection of slabs	40

	Page
5.9.9 Columns	40
5.9.9.1 General	40
5.9.9.2 Short columns	40
5.9.9.3 Slender columns	40
5.9.10 Local bond, anchorage bond and laps	40
5.9.11 Bearing stress inside bends	41
6 Design and detailing: prestressed concrete	41
6.1 General	41
6.1.1 Introduction	41
6.1.2 Limit state design of prestressed concrete	41
6.1.2.1 Basis of design	41
6.1.2.2 Durability	41
6.1.2.3 Other limit states and considerations	41
6.1.3 Loads	41
6.1.4 Strength of materials	41
6.1.4.1 Definition of strengths	41
6.1.4.2 Characteristic strength of concrete	41
6.1.4.3 Characteristic strength of prestressing tendons	41
6.2 Structures and structural frames	42
6.2.1 Analysis of structures	42
6.2.2 Redistribution of moments	42
6.3 Beams	42
6.3.1 General	42
6.3.1.1 Definitions	42
6.3.1.2 Slender beams	42
6.3.2 Serviceability limit state: flexure	42
6.3.2.1 Section analysis	42
6.3.2.2 Concrete compressive stress limitations	43
6.3.2.3 Steel stress limitations	43
6.3.2.4 Cracking	43
6.3.3 Ultimate limit state: flexure	45
6.3.3.1 Section analysis	45
6.3.3.2 Design charts	46
6.3.3.3 Design formula	46
6.3.3.4 Non-rectangular sections	46
6.3.4 Shear resistance of beams	46
6.3.4.1 General	46
6.3.4.2 Sections uncracked in flexure	46
6.3.4.3 Sections cracked in flexure	47
6.3.4.4 Shear reinforcement	48
6.3.4.5 Maximum shear force	49
6.3.4.6 Segmental construction	49
6.3.5 Torsional resistance of beams	49
6.3.5.1 General	49
6.3.5.2 Stresses and reinforcement	49
6.3.5.3 Segmental construction	50
6.3.5.4 Other design methods	50
6.3.6 Longitudinal shear	50
6.3.7 Deflection of beams	50

	Page
6.3.7.1 Class 1 and class 2 members	50
6.3.7.2 Class 3 members	50
6.4 Slabs	50
6.5 Columns	50
6.6 Tension members	50
6.7 Prestressing requirements	50
6.7.1 Maximum initial prestress	50
6.7.2 Loss of prestress, other than friction losses	51
6.7.2.1 General	51
6.7.2.2 Loss of prestress due to relaxation of steel	51
6.7.2.3 Loss of prestress due to elastic deformation of the concrete	51
6.7.2.4 Loss of prestress due to shrinkage of the concrete	51
6.7.2.5 Loss of prestress due to creep of the concrete	51
6.7.2.6 Loss of prestress during anchorage	52
6.7.2.7 Losses of prestress due to steam curing	52
6.7.3 Loss of prestress due to friction	52
6.7.3.1 General	52
6.7.3.2 Friction in the jack and anchorage	52
6.7.3.3 Friction in the duct due to unintentional variation from the specified profile	52
6.7.3.4 Friction in the duct due to curvature of the tendon	53
6.7.3.5 Friction in circular construction	53
6.7.3.6 Lubricants	53
6.7.4 Transmission length in pre-tensioned members	53
6.7.5 End blocks	54
6.8 Considerations affecting design details	55
6.8.1 General	55
6.8.2 Cover to prestressing tendons	55
6.8.2.1 General	55
6.8.2.2 Pre-tensioned tendons	55
6.8.2.3 Tendons in ducts	55
6.8.3 Spacing of prestressing tendons	55
6.8.3.1 General	55
6.8.3.2 Pre-tensioned tendons	55
6.8.3.3 Tendons in ducts	55
6.8.4 Longitudinal reinforcement in prestressed concrete beams	55
6.8.5 Links in prestressed concrete beams	55
6.8.6 Shock loading	56
6.8.7 Deflected tendons	56
6.8.7.1 Pre-tensioned tendons	56
6.8.7.2 Post-tensioned tendons	56
7 Design and detailing: precast, composite and plain concrete construction	56
7.1 General	56
7.1.1 Introduction	56
7.1.2 Limit state design	56
7.1.2.1 Basis of design	56
7.1.2.2 Handling stresses	56
7.1.2.3 Connections and joints	56

		Page
7.2	Precast concrete construction	56
7.2.1	Framed structures and continuous beams	56
7.2.2	Other precast members	56
7.2.3	Supports for precast members	56
7.2.3.1	Concrete corbels	56
7.2.3.2	Width of supports for precast units	57
7.2.3.3	Bearing stresses	57
7.2.3.4	Horizontal forces or rotations at bearings	57
7.2.4	Joints between precast members	58
7.2.4.1	General	58
7.2.4.2	Halving joint	58
7.3	Structural connections between units	58
7.3.1	General	58
7.3.1.1	Structural requirements of connections	58
7.3.1.2	Design method	58
7.3.1.3	Considerations affecting design details	59
7.3.1.4	Factors affecting design and construction	59
7.3.2	Continuity of reinforcement	60
7.3.2.1	General	60
7.3.2.2	Sleeving	60
7.3.2.3	Threading	60
7.3.2.4	Welding of bars	60
7.3.3	Other types of connection	60
7.4	Composite concrete construction	60
7.4.1	General	60
7.4.2	Ultimate limit state	61
7.4.2.1	General	61
7.4.2.2	Vertical shear	61
7.4.2.3	Longitudinal shear	61
7.4.3	Serviceability limit state	62
7.4.3.1	General	62
7.4.3.2	Compression in the concrete	62
7.4.3.3	Tension in the concrete	62
7.4.3.4	Differential shrinkage	63
7.4.3.5	Continuity in composite construction	63
7.5	Plain concrete walls and abutments	64
7.5.1	General	64
7.5.2	Moments and forces in walls and abutments	64
7.5.3	Eccentricity in the plane of the wall or abutment	64
7.5.4	Eccentricity at right-angles to walls of abutments	64
7.5.5	Analysis of section	64
7.5.6	Shear	65
7.5.7	Bearing	65
7.5.8	Deflection of plain concrete walls or abutments	65
7.5.9	Shrinkage and temperature reinforcement	65
7.5.10	Stress limitations for serviceability limit state	65
Appendix A Methods of compliance with serviceability criteria by direct calculation		66
Appendix B Elastic deformation of concrete		69

	Page
Appendix C Shrinkage and creep	69
Appendix D Cover and spacing of curved tendons in ducts for prestressed concrete	75
Figure 1 — Short term design stress-strain curve for normal weight concrete	12
Figure 2 — Short term design stress-strain curve for reinforcement	13
Figure 3 — Short term design stress-strain curve for normal and for low relaxation products	13
Figure 4 — <i>Figure deleted</i>	13
Figure 5 — Parameters for shear stress in solid slabs under concentrated load	23
Figure 6 — Openings in slabs	24
Figure 6a — Bearing stress	57
Figure 7 — Halving joint (diagrammatic only)	59
Figure 8 — Potential shear planes	62
Figure 9 — Coefficient k_L (environmental conditions)	70
Figure 10 — Coefficient k_m [hardening (maturity) at the age of loading]	70
Figure 11 — Coefficient k_c (composition of the concrete)	70
Figure 12 — Coefficient k_e (effective thickness)	71
Figure 13 — Coefficient k_j (variation as a function of time)	71
Figure 14 — Coefficient k_L (environment)	72
Figure 15 — Coefficient k_e (effective thickness)	72
Figure 16 — Relaxation coefficient	73
Figure 17 — Coefficient ϕ_2	74
Figure 18 — Coefficient ϕ_3	75
Figure 19 — Coefficient ϕ_4	75
Table 1 — Design crack widths	7
Table 2 — Stress limitations for the serviceability limit state	8
Table 3 — Modulus of elasticity of concrete under short term loading	9
Table 4 — Values of γ_m for the serviceability stress limitations	10
Table 5 — Strength of concrete	15
Table 6 — Strength of reinforcement	15
Table 7 — Form and area of shear reinforcement in beams	18
Table 8 — Ultimate shear stress in concrete, v_c	18
Table 9 — Values of ξ_s	18
Table 10 — Ultimate torsion shear stress	21
Table 11 — Effective height, l_e , for columns	26
Table 12 — Relationship of N/N_{uz} to α_n	28
Table 13 — Nominal cover to reinforcement under particular conditions of exposure	34
Table 14 — Ultimate local bond stresses	35
Table 15 — Ultimate anchorage bond stresses	37
Table 16 — Reduction factor for effective perimeter of a group of bars	37
Table 17 — Ultimate shear stress, v_c , in lightweight aggregate concrete beams without shear reinforcement	40
Table 18 — Maximum value of shear stress in lightweight aggregate concrete beams	40
Table 19 — Ultimate torsion shear stress in lightweight aggregate concrete beams	40
Table 20 — Strength of concrete	43

	Page
Table 22 — Compressive stresses in concrete for serviceability limit states	43
Table 23 — Allowable compressive stresses at transfer	43
Table 24 — Flexural tensile stresses for class 2 members: serviceability limit state: cracking	44
Table 25 — Hypothetical flexural tensile stresses for class 3 members	45
Table 26 — Depth factor for tensile stresses for class 3 members	45
Table 27 — Conditions at the ultimate limit state for rectangular beams with pre-tensioned tendons, or with post-tensioned tendons having effective bond	47
Table 28 — Maximum shear stress	49
Table 29 — Shrinkage of concrete	51
Table 30 — Design bursting tensile forces in end blocks	54
Table 31 — Ultimate longitudinal shear stress, v_1 , and values of k_1 for composite members	63
Table 32 — Flexural tensile stresses in in situ concrete	63
Table 33 — Values of ρ_0 for calculation of shrinkage curvatures ⁶⁷	67
Table 34 — Values of K_1 for various bending moment diagrams	68
Table 35 — Modulus of elasticity	69
Table 36 — Minimum cover to ducts perpendicular to plane of curvature	76
Table 37 — Minimum distance between centre lines of ducts in plane of curvature	77
Publications referred to	Inside back cover

Foreword

This Part of BS 5400 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. It supersedes BS 5400-4:1984 which is withdrawn.

This edition introduces technical changes but does not constitute a full review or revision of the standard, which will be undertaken in due course. Changes from the 1984 edition are indicated by a sideline in the margin.

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.*

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to x, pages 1 to 78, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

1 Scope

This Part of BS 5400 gives recommendations for the design of concrete bridges. It contains much in common with BS 8110 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_{sa}	area of longitudinal tensile reinforcement
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

α_{cent}	distance of the centroid of the concrete flange from the centroid of the composite section	E_{28}	secant modulus of elasticity of the concrete at the age of 28 days
α_{cr}	distance from the point (crack) considered to the surface of nearest longitudinal bar	e	base of Napierian logarithms
α_e	modular ratio at first introduction of stress	e	eccentricity
α_s	distance of the centroid of the steel from the centroid of the net concrete section	e_x	resultant eccentricity of load at right-angles to plane of wall
α_v	distance between the line of action or point of application of the load and the critical section or supporting member	F_{bst}	tensile bursting force
b	width or breadth of section	F_{bt}	tensile force due to ultimate loads in a bar or group of bars
b_a	average breadth of section excluding the compression flange	F_h	maximum horizontal ultimate load
b_c	breadth of compression face	F_v	maximum vertical ultimate load
b_{col}	width of column	f	stress
b_s	width of section containing effective reinforcement for punching shear	f_{bs}	local bond stress
b_t	breadth of section at level of tension reinforcement	f_c	a constant stress in the concrete
b_w	breadth of web or rib of a member	f_{cav}	average compressive stress in the flexural compressive zone
c_{nom}	nominal cover	f_{ci}	concrete strength at (initial) transfer
D_c	density of lightweight aggregate concrete at time of test	f_{cj}	stress in concrete at application of an increment of stress at time j
d	effective depth to tension reinforcement	f_{co}	stress in concrete at the level of the tendon due to initial prestress and dead load
d'	depth of compression reinforcement	f_{cp}	compressive stress at the centroidal axis due to prestress
d_c	depth of concrete in compression	f_{cu}	characteristic concrete cube strength
d_e	effective depth for a solid slab or rectangular beam; otherwise the overall depth of the compression flange	f_{pb}	tensile stress in tendons at (beam) failure
d_o	depth to additional reinforcement to resist horizontal loading	f_{pe}	effective prestress (in tendon)
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	f_{pt}	stress due to prestress
d_2	depth from the surface to the reinforcement in the other face	f_{pu}	characteristic strength of prestressing tendons
E_c	static secant modulus of elasticity of concrete	f_{S2}	stress in reinforcement in other face
E_{cf}	modulus of elasticity of flange concrete	f_t	maximum principal tensile stress
E_{cq}	dynamic tangent modulus of elasticity of concrete	f_y	characteristic strength of reinforcement
E_s	modulus of elasticity of steel	f_{yc}	design strength of longitudinal steel in compression
$(EI)_c$	flexural rigidity of the column cross section	f_{yL}	characteristic strength of longitudinal reinforcement
		f_{yv}	characteristic strength of link reinforcement
		h	overall depth (thickness) of section (in plane of bending)
		h_{agg}	maximum size of aggregate
		h_e	effective thickness
		h_f	thickness of flange

$h_{\max}$	larger dimension of section	l_0	clear height of column between end restraints
$h_{\min}$	smaller dimension of section	l_{sb}	length of straight reinforcement beyond the intersection with the link
h_{wo}	wall thickness where the stress is determined	l_t	transmission length
h_x	overall depth of the cross section in the plane of bending M_{iy}	M	bending moment due to ultimate loads
h_y	overall depth of the cross section in the plane of bending M_{ix}	M_a	increased moment in column
I	second moment of area	M_{cr}	cracking moment at the section considered
i	radius of gyration	M_{cs}	hogging restraint moment at an internal support of a continuous composite beam and slab section due to differential shrinkage
j	age at application of increment of stress	M_g	moment due to permanent loads
j_i	age of first loading	M_i	maximum initial moment in a column due to ultimate loads
j_m	number of days during which hardening takes place at $T^\circ\text{C}$	M_{ix}	initial moment about the major axis of a slender column due to ultimate loads
j_∞	age at the end of the life of the structure	M_{iy}	initial moment about the minor axis of a slender column due to ultimate loads
K	a factor depending on the type of duct or sheath used	M_q	moment due to live loads
K_1	depends on the shape of the bending moment diagram	M_{tx}	total moment about the major axis of a slender column due to ultimate loads
k	a constant (with appropriate subscripts)	M_{ty}	total moment about the minor axis of a slender column due to ultimate loads
k_c	depends on the composition of the concrete	M_u	ultimate resistance moment
k_e	depends on the effective thickness of the member	M_{ux}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the major axis only
k_j	covers the development of the deferred deformation with time	M_{uy}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the minor axis only
k_L	depends on environmental conditions	M_x, M_y	moments about the major and minor axes of a short column due to ultimate loads
k_m	depends on the hardening (maturity) of the concrete at the age of loading	M_0	moment necessary to produce zero stress in the concrete at the depth d
k_{mi}	coefficient k_m for age at first loading	M_1	smaller initial end moment due to ultimate loads (assumed negative if the column is bent in double curvature)
k_r	depends on grade of reinforcement	M_2	larger initial end moment due to ultimate loads (assumed positive)
k_t	depends on the type of tendon	N	ultimate axial load at section considered
k_1	depends on the concrete bond across the shear plane	N_u	ultimate resistance axial load
k_2	depends on the geometric ratio of longitudinal reinforcement	N_{uz}	axial loading capacity of a column ignoring all bending
L_s	length of shear plane	n_w	ultimate axial load per unit length of wall
l	distance from face of support at the end of a cantilever, or effective span of a member	P_f	effective prestressing force after all losses
l_e	effective height of a column or wall		
l_{ex}	effective height for bending about the major axis		
l_{ey}	effective height for bending about the minor axis		

P_h	horizontal component of the prestressing force after all losses	v_t	torsional shear stress
P_k	basic load in tendon	v_{tmin}	minimum ultimate torsional shear stress for which reinforcement is required
P_o	prestressing force in the tendon at the jacking end (or at tangent point near jacking end)	v_{tu}	ultimate torsional shear stress
P_x	prestressing force at distance x from jack	x	neutral axis depth
Q^*	design load	x_1	smaller centre line dimension of a link
Q_k	nominal load	y	distance of the fibre considered in the plane of bending from the centroid of the concrete section
r	internal radius of bend	y_o	half the side of end block
r_{ps}	radius of curvature of a tendon	y_{po}	half the side of loaded area
r_b	radius of curvature of a beam at mid span or, for cantilevers, at the support section	y_1	larger centre line dimension of a link
r_{cs}	shrinkage radius of curvature	z	lever arm
r_x	radius of curvature of a member at point x	α	orientation of the reinforcement in the transverse direction
S^*	design load effects	α_e	modular ratio
s_L	spacing of longitudinal reinforcement	α_n	coefficient as a function of column axial load
s_v	spacing of links along the member	α_1	angle between the axis of the design moment and the direction of the tensile reinforcement
T	torsional moment due to ultimate loads	α_2	angle of friction at the joint
T	temperature (in degrees Celsius)	β_{cc}	ratio of total creep to elastic deformation
t	time	$\gamma_{f1}, \gamma_{f2}, \gamma_{f3}$	partial load factors
u	perimeter	γ_{fL}	product of γ_{f1}, γ_{f2}
u_s	effective perimeter of tension reinforcement	γ_m	partial safety factor for strength
V	shear force due to ultimate loads	Δ_{cc}	concrete creep deformation
V_c	ultimate shear resistance of concrete	Δ_{cct}	creep strain
V_{co}	ultimate shear resistance of a section uncracked in flexure	Δ_{cs}	concrete shrinkage deformation
V_{cr}	ultimate shear resistance of a section cracked in flexure	Δ_{cst}	shrinkage at time t
V_l	longitudinal shear force due to ultimate load	$\Delta_{f\infty}$	change in stress in concrete due to creep at time t_∞
V_{ux}	ultimate shear capacity of a section for the x-x axis	Δ_p	loss of prestress due to shrinkage and creep
V_{uy}	ultimate shear capacity of a section for the y-y axis	δ_m	degree of hardening at moment of loading
V_x	applied shear due to ultimate loads for the x-x axis	ϵ	strain
V_y	applied shear due to ultimate loads for the y-y axis	ϵ_{c1}	creep strain in concrete at the level of the tendon at time of loading
v	shear stress	ϵ_{c2}	creep strain in concrete at the centroid of the section at time of loading
v_c	ultimate shear stress in concrete	ϵ_{cs}	free shrinkage strain
v_l	ultimate longitudinal shear stress per unit area of contact surface	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	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 mm
<i>Extreme</i> Concrete surfaces exposed to: abrasive action by sea water or water with a $\text{pH} \leq 4.5$	Marine structures Parts of structure in contact with moorland water	0.10
<i>Very severe</i> 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
<i>Severe</i> 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
<i>Moderate</i> Concrete surfaces above ground level <i>and</i> fully sheltered against all of the following: rain, de-icing salts, sea water spray Concrete surfaces permanently saturated by water with a $\text{pH} > 4.5$	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 6.7.1
NOTE See 7.3.3 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 loading 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 1 400 kg/m³ and 2 300 kg/m³, the values given in Table 3 should be multiplied by $(D_c/2\ 300)^2$ where D_c is the density of the lightweight aggregate concrete in kg/m³. 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:

- for reinforcement, from Figure 2, using the value of γ_m given in 4.3.3;
- for prestressing steel, from Figure 2, 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 Table 2 and Table 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.

- Concrete section.** The entire member cross section, ignoring the presence of reinforcement.
- Gross transformed section.** The entire member cross section including the reinforcement, transformed on the basis of modular ratio.
- 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.

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

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.

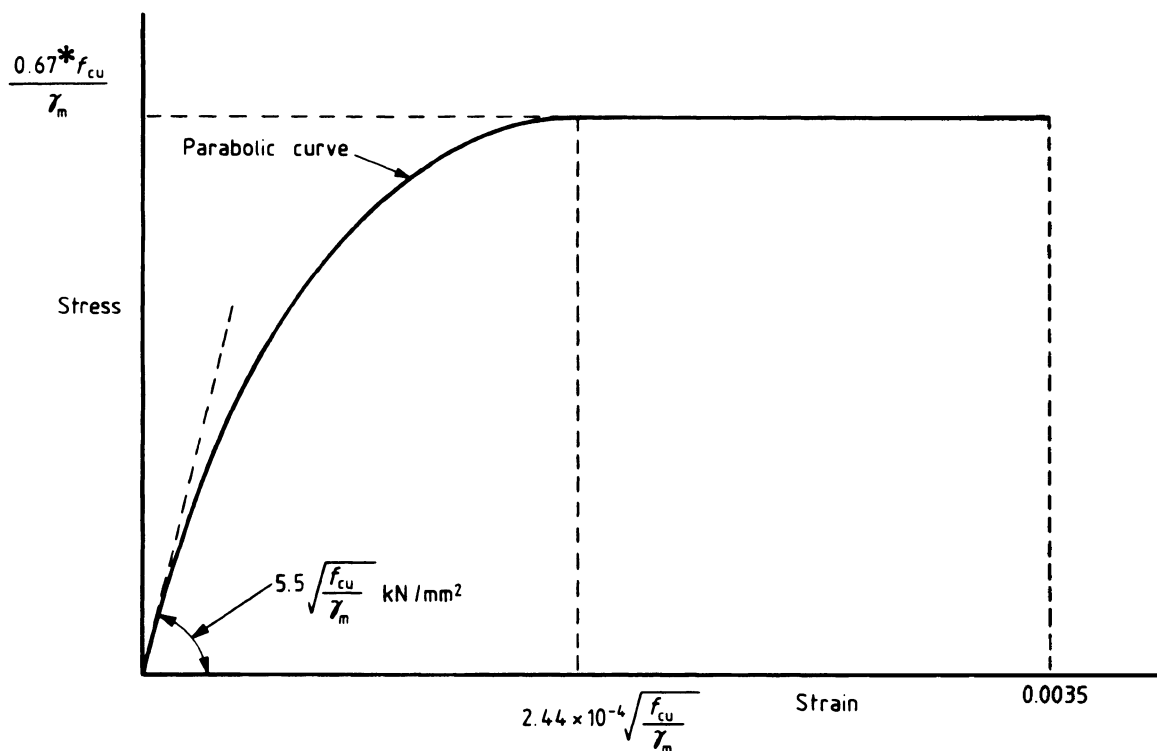
4.6 Deflection. Deflection should be calculated for the most unfavourable distributions of loading for the member (or strip of slab) and may be derived from an elastic analysis of the structure. The material properties, stiffness constants and calculation of deflections may be based on the information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

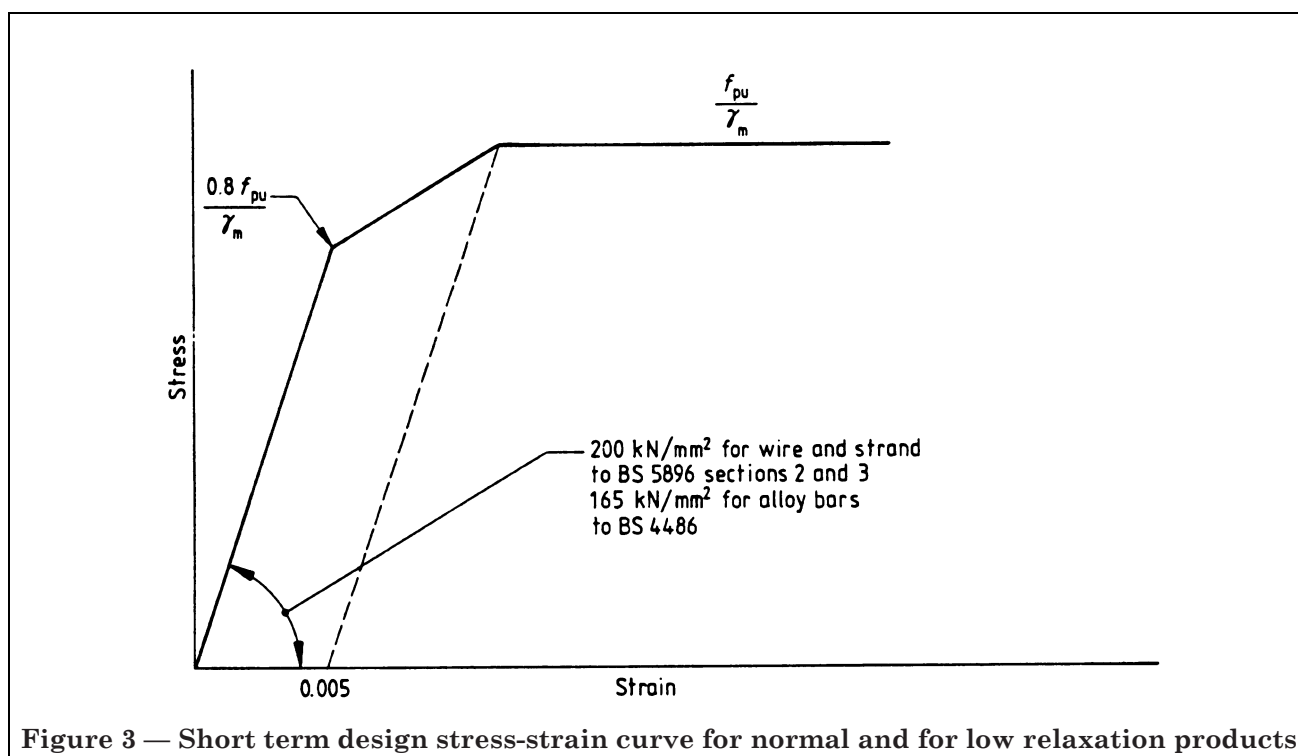
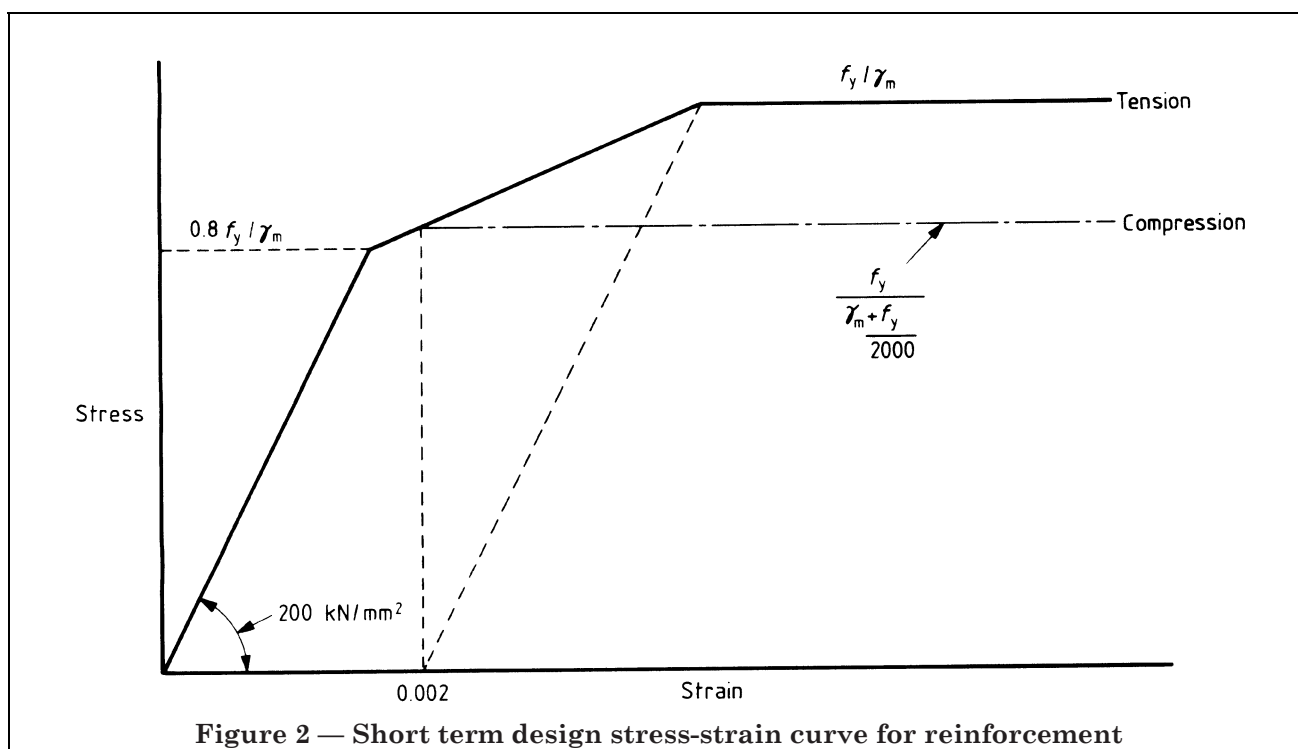


Figure 4 — *Figure deleted*

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 in appropriate 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_c 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 ^a	3 months ^a	6 months ^a	1 Year ^a
	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

^a 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
	mm	N/mm ²
Grade 250 (BS 4449)	8, 10, 12, and 16	250
Grade 460 (BS 4449)	All sizes	460
Cold reduced 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 1 200 mm.

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.