CORRECTIONS WITHIN DESIGN MANUAL FOR ROADS AND BRIDGES AUGUST 2008

SUMMARY OF CORRECTION – BD 44/95 Volume 3, Section 4, Part 14 THE ASSESSMENT OF CONCRETE HIGHWAY BRIDGES AND STRUCTURES

Please replace pages A/67 and A/68.

The equation in clause 7.2.4.2 has been amended.

We apologise for the inconvenience caused.

Highways Agency
August 2008

London: The Stationery Office

THE HIGHWAYS AGENCY BD 44/95

THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF THE ENVIRONMENT FOR NORTHERN IRELAND

Your attention is drawn to Interim Advice Note 4, which has been issued by the Highways Agency for use on trunk roads and motorways in England. Click here to view this Interim Advice Note.

The Assessment of Concrete Highway Bridges and Structures

Summary: This Standard gives the requirements for the assessment of existing concrete highway bridges and structures on motorways and trunk roads. This Standard supersedes BD 44/90.

REGISTRATION OF AMENDMENTS

Amend No	Page No	Signature & Date of incorporation of amendments	Amend No	Page No	Signature & Date of incorporation of amendments
1	A46	Equation in clause 5.8.6.3 corrected in electronic versions only, paper version not affected. Square root correctly truncated – August 2005			

REGISTRATION OF AMENDMENTS

Amend No	Page No	Signature & Date of incorporation of amendments	Amend No	Page No	Signature & Date of incorporation of amendments

VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 14

BD 44/95

THE ASSESSMENT OF CONCRETE HIGHWAY BRIDGES AND STRUCTURES

Contents

Chapter

1. INTRODUCTION

1.1 This Standard, which for assessment purposes replaces BD 24, gives requirements for the assessment of existing concrete structures and structural elements, and shall be used in conjunction with Standard BD 21 "The Assessment of Highway Bridges and Structures" (DMRB 3.4.3).

1.2 Appendix A of this standard contains the relevant assessment clauses which have been presented in the same format as the design clauses in BS 5400: Part 4 to assist the engineer carrying out the assessment. These clauses have been specifically developed to suit assessment conditions and therefore shall not be used in new design or construction.

1.3 An Advice Note BA 44 "The Assessment of Concrete Highway Bridges and Structures" will accompany this Standard, giving the necessary background information and also guidance on the application of this Standard. It is recommended that the Advice Note should be used in conjunction with the Standard.

Scope

1.4 This Standard should be used in the assessment of all concrete elements in highway bridges and structures on trunk roads and motorways. For use in Northern Ireland, this Standard will be applicable to those roads designated by the Overseeing Organisation.

Implementation

1.5 This standard should be used forthwith on all bridge maintenance schemes including those currently in progress, provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay. Design Organisations should confirm its application to particular schemes with the Overseeing Organisation.

2. ASSESSMENT OF STRENGTH

General

2.1 The objective of this Standard is to produce a more realistic assessment of the strength of a concrete element than has previously been possible using the requirements of the existing design code. This is in part achieved by taking advantage of the information available to an assessing engineer which can only be predicted at the design stage.

2.2 Many of the criteria given in the design code are based on experimental evidence which in some cases have been either conservatively interpreted for use in design or updated by later evidence allowing a less conservative interpretation. For assessment purposes such criteria have been reviewed and amended where appropriate.

2.3 An important feature of the design code is the application of the partial safety factor for material strength γ_m to the characteristic values. This approach is retained in Appendix A but the concept of worst credible strength with a reduced value of γ_m has been introduced as an alternative.

Worst Credible Strength

2.4 The term worst credible strength has been introduced in this standard to allow for the actual material strengths of the structures and structural elements being used for assessment. Worst credible strength can be defined as the worst value of that strength which the engineer, based on his experience and knowledge of the material, realistically believes could be obtained in the structure of element under consideration. This value may be greater or less than the characteristic strength of the material assumed at the design stage. Since this value eliminates some of the uncertainties associated with the use of characteristic strengths, reductions may be made in the partial safety factor for material γ_m.

2.5 Worst credible strengths should be used in the following circumstances:

(i) When an initial assessment using characteristic values has shown an element of structure to be incapable of carrying the full assessment loading of BD 21.

(ii) If a structure has suffered damage or deterioration in such a way that the actual strengths are or are thought to be, less than the assumed characteristic values.

(iii) Where no information exists on the characteristic values used in design.

2.6 The worst credible value shall generally be taken as the lower bound value of the estimated insitu strength for the element under consideration. Further guidance on the assessment of worst credible strength is given in the Appendix A of BA 44.

2.7 Worst credible strength for concrete should generally be derived from tests carried out on cores. Cores are destructive and can not normally be taken at the critical (most highly stressed) locations of an element; hence interpolation or extrapolation is necessary, to arrive at worst credible strengths in these locations.

2.8 To assist in interpolating or extrapolating the results of core tests, an integrated programme of testing which may include destructive, semi-destructive (eg near surface tests) and non-destructive tests will be necessary for each element. The assessing engineer should use his judgement in selecting the locations and numbers of samples for such tests.

2.9 For reinforcement or prestressing tendons and bars, a worst credible value should be obtained by testing samples taken from the element being assessed. Removal of prestressing steel for sampling purposes will alter the stress distribution in the concrete section, and this must be allowed for in the assessment calculations.

Partial Safety Factor for Materials, γ_m

2.10 The values of γ_m for concrete and reinforcing or prestressing steel given in of BD 21 shall be replaced by the appropriate γ_m values given in Appendix A of this standard. The values for use with the characteristic strength are different from those for use with worst credible strengths. To enable the correct value of γ_m to be used, all limiting criteria have been expressed as formulae with γ_m stated explicitly rather than as tabulated values.

Limit State

2.11 Although this Standard, in common with BD 21, specifies that assessments shall be carried out at the ultimate limit state, there are some references to serviceability requirements. These should only be applied if required by the Overseeing Organisation.

Condition Factor in BD 21

2.12 While the application of the condition factor F_c in section 5.4.3 of BD 21 is not affected in principle by the requirements of this Standard, care should be taken to ensure that the estimated values of F_c do not allow for any deficiencies of the materials in a structure which are separately allowed for by using worst credible strengths.

USE OF APPENDIX A

2.13 Appendix A is based on BS 5400: Part 4 and retains the terminology and clause numbering of that document. In cases where the BS 5400: Part 4 clauses are not required for assessment, the numbers and headings of those clauses have been included to retain the structure of the document but the words "Not applicable to assessment" have been added in italics.

2.14 Wherever possible, tabulated values in BS 5400: Part 4 have been replaced by formulae in which γ_m is stated explicitly.

2.15 Those equations from BS 5400: Part 4 which have been amended, other than simply to incorporate γ_m, have their equation number followed by the letter `A'. Amended tables and figures retain their BS 5400: Part 4 numbers but the letter `A' is added to the number.

3. REFERENCES

1. The Design Manual for Roads and Bridges (DMRB)

BD 9. Implementation of BS5400: Part 10: 1980 (DMRB 1.3)

BD 24. Design of Concrete Bridges.
Use of BS 5400: Part 4: 1990. (DMRB 1.3.1)

BD 37. Loads for Highway Bridges (DMRB 1.3)

BD 21. The assessment of Highway Bridges
and Structures. (DMRB 3.4.3)

BA 38. Assessment of the Fatigue Life
of Corroded or Damaged Reinforcing Bars. (DMRB 3.4.5)

BA 39. The Assessment of Reinforced
Concrete Half-Joints. (DMRB 3.4.6)

BA 44. The Assessment of Concrete Highway Bridges
and Structures (DMRB 3.4)

2. British Standards

BS 5400 Part 4: 1990. Steel, Concrete and
Composite Bridges. Code of Practice for Design
of Concrete Bridges. BSI.

BS 5400: Part 10: 1980. Steel, Concrete and
Composite Bridges. Code of Practice for Fatigue.

British Standard Code of Practice,
CP110: Parts 2&3: 1972 Use of Concrete. BSI.

3. Manual of Contract Documents for Highway Works (MCHW)

Specification for Highway Works (MCHW 1)

4. ENQUIRIES

All technical enquiries or comments on this Standard should be sent in writing as appropriate to:

The Chief Highway Engineer The Highways Agency St Christopher House Southwark Street London SE1 OTE	T A ROCHESTER Chief Highway Engineer

The Deputy Chief Engineer The Scottish Office Industry Department Roads Directorate New St Andrew's House Edinburgh EH1 3TG	N B MACKENZIE Deputy Chief Engineer

The Director of Highways Welsh Office Y Swyddfa Gymreig Government Buildings Ty Glas Road Llanishen Cardiff CF4 5PL	K THOMAS Director of Highways

Director of Roads Service Department of the Environment for Northern Ireland Roads Service Headquarters Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	W J McCOUBREY Director of Roads Service

THE ASSESSMENT OF CONCRETE HIGHWAY BRIDGES AND STRUCTURES

1. SCOPE

This Standard is intended to be used for the assessment of concrete highway bridges and structural elements.

Where appropriate, the design requirements of BS 5400 Part 4 have been modified to suit the assessment of reinforced concrete, prestressed concrete and composite concrete structures. Structural elements included are beams, slabs, columns and walls, bases, tension members and connections between precast concrete members.

2. DEFINITIONS AND SYMBOLS

2.1 Definitions

2.1.1 General For the purposes of this Standard the definitions given in BD 21 shall 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.

Assessment loads, Q_A, are obtained by multiplying the nominal loads, Q_k, by γ_{fL}, the partial safety factory for loads. γ_{fL} is a functions of two individual factors, γ_{fl} and γ_{f2}, which take account of the following:

γ_{fl} 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} ($=\gamma_{fl}\gamma_{f2}$) are given in BD 21.

The assessment load effects, S_A, should be obtained from the assessment loads by the relation:

$$S_A = \gamma_{f3} \text{ (effects of } Q_A)$$

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 Strength Material strengths are expressed in terms of the cube strength of concrete, f_{cu}, the yield or proof strength of reinforcement, f_y or the breaking stress of prestressing tendon, f_{pu}.

The material strengths used may be either

(a) Characteristic strength, which is the strength below which not more than 5% of all possible test results may be expected to fall or

(b) Worst credible strength, which is the worst value of the strength which the Engineer based on his experience and knowledge of the material realistically believes could occur. The method of determining the worst credible strength shall be agreed with the relevant Operating Unit.

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 are as follows:

A_c area of concrete
A_{con} contact area
A_e area of fully anchored reinforcement per unit length crossing the shear plane
A_i cross-section area of in-situ concrete
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_{sl} 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' distance from compression face to a point at which the crack width is being calculated
a_b centre-to-centre distance between bars
a_{cr} distance from the point (crack) considered to the surface of nearest longitudinal bar
a_v distance from the section under consideration to the supporting member
b width or breadth of section
b_a average breadth of section excluding the compression flange
b_c breadth of compression face
b_{col} width of column
b_s width of section containing effective reinforcement for punching shear
b_t breadth of section at level of tension reinforcement
b_w breadth of web or rib of a member
c cover
c_{nom} nominal cover
D_c density of lightweight aggregate concrete at time of test
d effective depth to tension reinforcement
d' depth of compression reinforcement
d_c depth of concrete in compression
d_e effective depth for a solid slab or rectangular beam; otherwise the overall depth of the compression flange
d_s effective depth to tension steel in prestressed member
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
d_2 depth from the surface to the reinforcement in the other face
E_c static secant modulus of elasticity of concrete
$(El)_c$ flexural rigidity of the column cross section
e eccentricity
e_c eccentricity to compression face
e_x resultant eccentricity of load at right-angles to plane of wall
F_{bst} tensile bursting force
F_{bt} tensile force due to ultimate loads in a bar or group of bars
f stress
f_{cav} average compressive stress in the flexural compressive zone
f_{ci} concrete strength at (initial) transfer
f_{cp} compressive stress at the centroidal axis due to prestress
f_{cu} characteristic or worst credible concrete cube strength
f_{pb} tensile stress in tendons at (beam) failure

f_{pe}	effective prestress (in tendon)
f_{pt}	stress due to prestress
f_{pu}	characteristic or worst credible strength of prestressing tendons
f_s	reinforcement stress
f_{s2}	stress in reinforcement in other face
f_t	maximum principal tensile stress; tensile strength of reinforcement
f_y	characteristic or worst credible strength of reinforcement
f_{yc}	design strength of longitudinal steel in compression
f_{yL}	characteristic strength of longitudinal reinforcement
f_{yv}	characteristic of worst credible strength of shear reinforcement
h	overall depth (thickness) of section (in plane of bending)
h_f	thickness of flange
h_{max}	larger dimension of section
h_{min}	smaller dimension of section
h_w	wall thickness
h_x	overall depth of the cross section in the plane of bending M_{iy}
h_y	overall depth of the cross section in the plane of bending M_{ix}
I	second moment of area
K	a factor depending on the type of duct or sheath used
k	a constant (with appropriate subscripts)
k_t	depends on the type of tendon
k_l	depends on the concrete bond across the shear plane
L_s	length of shear plane
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
l_o	clear height of column between end restraints
l_t	transmission length
M	bending moment due to ultimate loads
M_{cr}	cracking moment at the section considered
M_g	moment due to permanent loads
M_i	maximum initial moment in a column due to ultimate loads
M_{ix}	initial moment about the major axis of a slender column due to ultimate loads
M_{iy}	initial moment about the minor axis of a slender column due to ultimate loads
M_{nt}	twisting moment per unit length in the slab adjacent to the edge zone
M_q	moment due to live loads
M_{tx}	total moment about the major axis of a slender column due to ultimate loads
M_{ty}	total moment about the minor axis of a slender column due to ultimate loads
M_u	ultimate resistance moment
M_{ux}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the major axis only
M_{uy}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the minor axis only
M_x,M_y	moments about the major and minor axes of a short column due to ultimate loads
M_o	moment necessary to produce zero stress in the concrete at the depth d
M_1	smaller initial end moment due to ultimate loads (assumed negative if the column is bent in double curvature)
M_2	larger initial end moment due to ultimate loads (assumed positive)
N	ultimate axial load at section considered; number of bars in a group
N_u	ultimate resistance axial load
N_{uz}	axial loading capacity of a column ignoring all bending
n_w	ultimate axial load per unit length of wall
P_f	effective prestressing force after all losses
P_h	horizontal component of the prestressing force after all losses
P_k	basic load in tendon
P_o	prestressing force in the tendon at the jacking end (or at tangent point near jacking end)
P_x	prestressing force at distance x from jack
Q_A	assessment load
Q_k	nominal load

r	internal radius of bend
r_{ps}	radius of curvature of a tendon
S_A	assessment load effects
s_L	spacing of longitudinal reinforcement
s_v	spacing of links along the member
T	torque due to ultimate loads
T_u	ultimate torsional strength
V	shear force due to ultimate loads
V_c	ultimate shear resistance of concrete
V_{co}	ultimate shear resistance of a section uncracked in flexure
V_{cr}	ultimate shear resistance of a section cracked in flexure
V_i	shear capacity of infill concrete
V_1	longitudinal shear force due to ultimate load
V_s	shear resistance of shear reinforcement
V_t	flexural shear force per unit width acting perpendicular to a slab edge
V_u	ultimate shear resistance of section
V_{ux}	ultimate shear capacity of a section for the x-x axis
V_{uy}	ultimate shear capacity of a section for the y-y axis
V_x	applied shear due to ultimate loads for the x-x axis
V_y	applied shear due to ultimate loads for the y-y axis
v	shear stress
v_u	ultimate shear stress in concrete (Halving joint)
v_c	ultimate shear stress in concrete
v_1	ultimate longitudinal shear stress per unit area of contact surface
v_t	torsional shear stress
v_{tmin}	minimum ultimate torsional shear stress for which reinforcement is required
v_{tu}	ultimate torsional shear stress
x	neutral axis depth
x_1	smaller centre line dimension of a link
y	distance of the fibre considered in the plane of bending from the centroid of the concrete section
y_o	half the side of end block
y_{po}	half the side of loaded area
y_1	larger centre line dimension of a link
z	lever arm
α	inclination of shear reinforcement; factor to determine f_{pb}
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	coefficient dependent on bar type
$\gamma_{f1},\gamma_{f2},\gamma_{f3}$	partial load factors
γ_{fL}	product of γ_{f1},γ_{f2}
γ_m	partial safety factor for strength
γ_{mb}	partial safety factor for bond
γ_{mc}	partial safety for concrete
γ_{mcw}	partial safety for plain concrete wall
γ_{ms}	partial safety factor for steel
γ_{mv}	partial safety factor applied to v_c
ϵ	strain
ϵ_m	average strain
ϵ_s	strain in tension reinforcement
ϵ_1	strain at level considered
μ	coefficient of friction
ξ_s	depth factor
ρ	geometrical ratio of reinforcement, is equal to A_s/bd
ρ_{net}	ρ for a flange
$\sum A_{sv}$	area of shear reinforcement

$\sum bd$ area of the critical section
ϕ size (nominal diameter) of bar or tendon)

3. LIMIT STATE PHILOSOPHY

3.1 General It is always necessary to assess the degree of safety in accordance with the limit state of 4.1.2.

In some situations the Overseeing Department may also require the degree of serviceability to be assessed in accordance with the limit state of 4.1.1 considering factors such as deflection, fatigue and durability.

3.2 Serviceability limit state Under serviceability loads the structure should not suffer local damage which would shorten its intended life or incur excessive maintenance costs.

3.3 Ultimate limit state The strength of the structure shall be sufficient to withstand the assessment loads taking due account of the possibility of overturning or buckling. The assessment shall 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. ASSESSMENT: GENERAL

4.1 Limit state requirements

4.1.1 Serviceability limit states When a serviceability limit state assessment is required, the necessary criteria shall be agreed with the Overseeing Department.

4.1.2 Ultimate limit states The assessment of the structure under the assessment loads appropriate to this limit state shall 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.

4.1.3 Other considerations When the Overseeing Department requires other effects (such as vibration, deflections, fatigue and durability) to be considered, appropriate criteria would be specified.

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

4.2.1 Loads The nominal values of loads and load combinations are given in BD 21/93.

4.2.2 Serviceability limit state When serviceability limit state checks are required, the loading shall be agreed with the Overseeing Department.

4.2.3 Ultimate limit state The values of the partial safety factor, γ_{fL}, are given in BD 21/93. In calculating the resistance of members to vertical shear and torsion, γ_{fL} for the prestressing force shall 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} shall be taken as 1.10, except that where plastic methods are used for the analysis of the structure, γ_{f3} shall be taken as 1.15.

4.2.4 Deflection When required, the loading shall be agreed with the Overseeing Department.

4.3 Properties of materials

4.3.1 General Either the characteristic strength, or the worst credible strength (see 2.1.3.1) may be used for a material strength. In general, in analysing a structure to determine load effects, the material properties appropriate to the characteristic, or worst credible, strength shall 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 shall be as follows:

(a) Serviceability limit state. The characteristic stresses are to be agreed with the Overseeing Department.

(b) Ultimate limit state. The values given in 4.3.2.

The appropriate γ_m values are given in 4.3.3: γ_{mc} for concrete, and γ_{ms} for steel.

4.3.2 Material properties

4.3.2.1 Concrete In assessing the strength of sections at the ultimate limit state, the assessment stress-strain curve for normal weight concrete may be taken from figure 1, using the value of γ_{mc} 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, and, in the absence of test data, the short term value shall be taken as $(20 + 0.27\ f_{cu})$ kN/mm² with f_{cu} in N/mm^2 units. The effect of creep under long term loading may be allowed for by using half of the short term modulus of elasticity.

For lightweight concrete having an air dry density between 1400 kg/m³ and 2300 kg/m³, the values given in the previous paragraph shall be multiplied by $(D_c/2300)^2$ where D_c is the density of the lightweight aggregate concrete in kg/m³.

Poisson's ratio may be taken as 0.2. The value for the coefficient of thermal expansion may be taken from Table 4/3 of BD 21.

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

(a) for reinforcement, from Figure 2, using the value of γ_{ms} given in 4.3.3;

(b) for prestressing steel; from Figure 3 or Figure 4, using the value of γ_{ms} given in 4.3.3.

Alternatively where the reinforcement or tendon type is known, manufacturers' stress-strain curves may be used with the values of γ_{ms} given in 4.3.3.

For reinforcement, the modulus of elasticity may be taken as 200 kN/mm². For prestressing steel, the modulus of elasticity 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 summarised in 4.3.3.2 to 4.3.3.4.

4.3.3.2 Serviceability limit state Values of γ_{mc} and γ_{ms} shall be agreed with the Overseeing Department.

4.3.3.3 Ultimate limit state For both reinforced concrete and prestressed concrete, the values of γ_m applied to either the characteristic strengths or worst credible strengths are summarised in Table 4A.

Application	Symbol	Value for use with	
		Characteristic strength	Worst credible strength
Reinforcement and prestressing tendons	γ_{ms}	1.15	1.10*
Concrete	γ_{mc}	1.50	1.20
Shear in concrete	γ_{mv}	1.25	1.15
Bond	γ_{mb}	1.4	1.25
Plain concrete wall	γ_{mcw}	2.25	1.80

* May be reduced to 1.05 if measured steel depths are used in addition to the worst credible steel strength.

Table 4A. Values of γ_m at the ultimate limit state.

4.3.3.4 Fatigue When applying 4.7, the values of γ_{ms} applied to a reinforcement stress range 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 BS 5400:Part 1.

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.

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

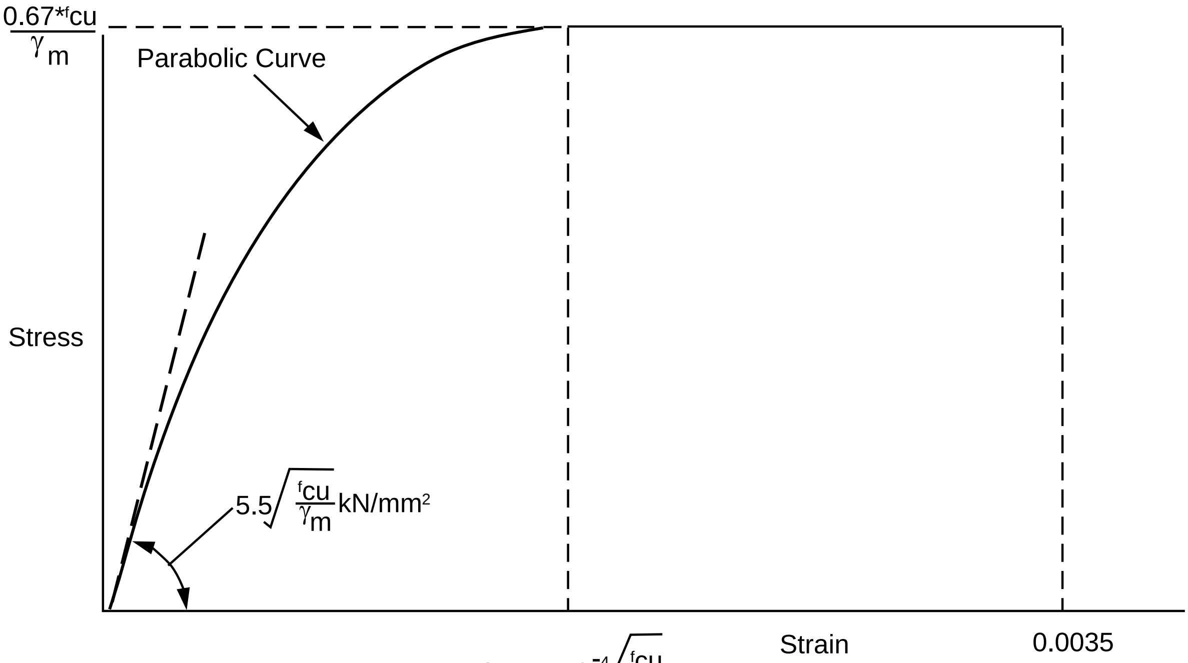
(c) Net transformed section The area of the cross section which is in compression together with the tensile reinforcement, transformed on the basis of modular ratio.

A consistent approach shall 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, shall be based on the concrete section and used with (a) or (b).

Moduli of elasticity and shear moduli values shall be appropriate to the characteristic, or worst credible 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.



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

NOTE 1 f_m is in N/mm²
NOTE 2 The equation for the parabolic curve between = 0 and $2.44 \times 10^{-4}\sqrt{\frac{f_{cu}}{\gamma_m}}$ 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 assessment stress-strain curve for normal weight concrete

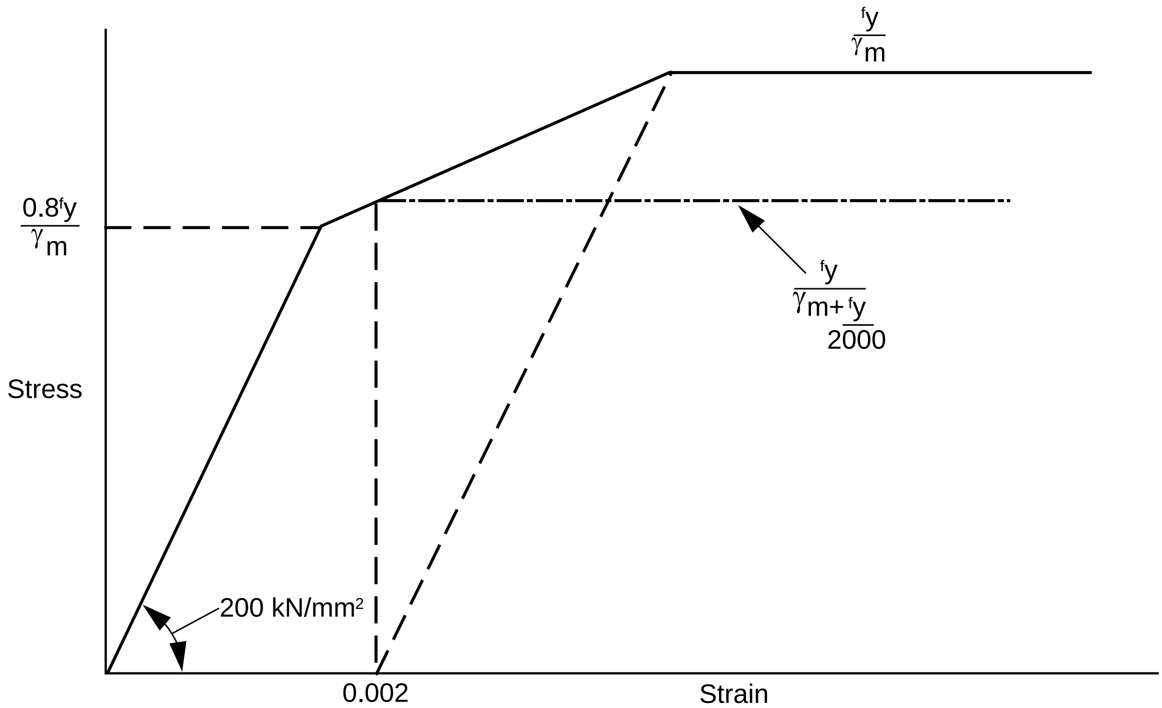


Figure 2 Short term assessment stress-strain curve for reinforcement

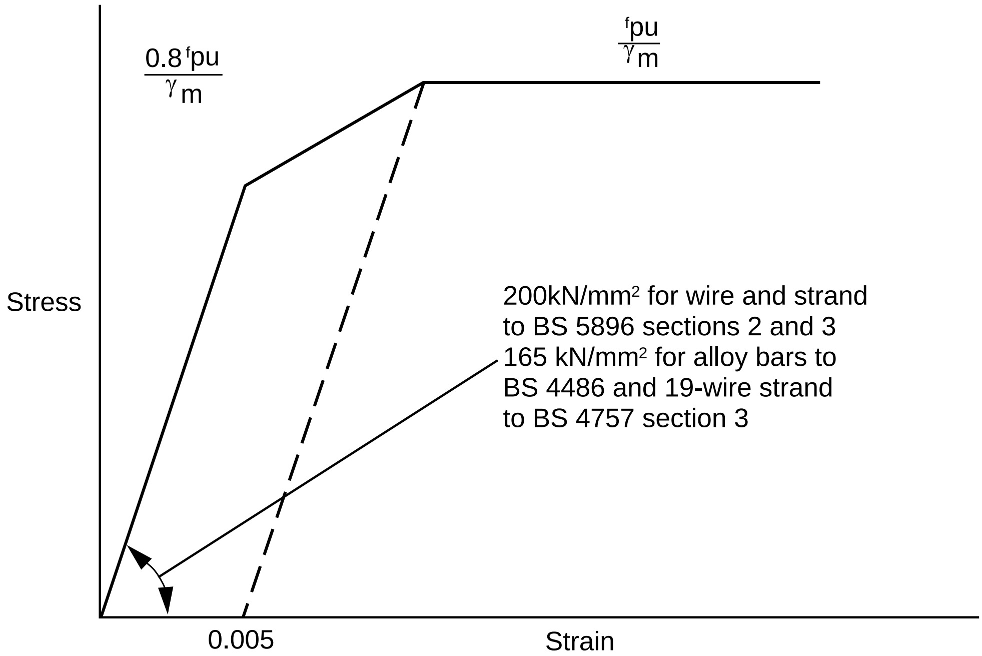


Figure 3 Short term assessment stress-strain curve for normal and low relaxation products

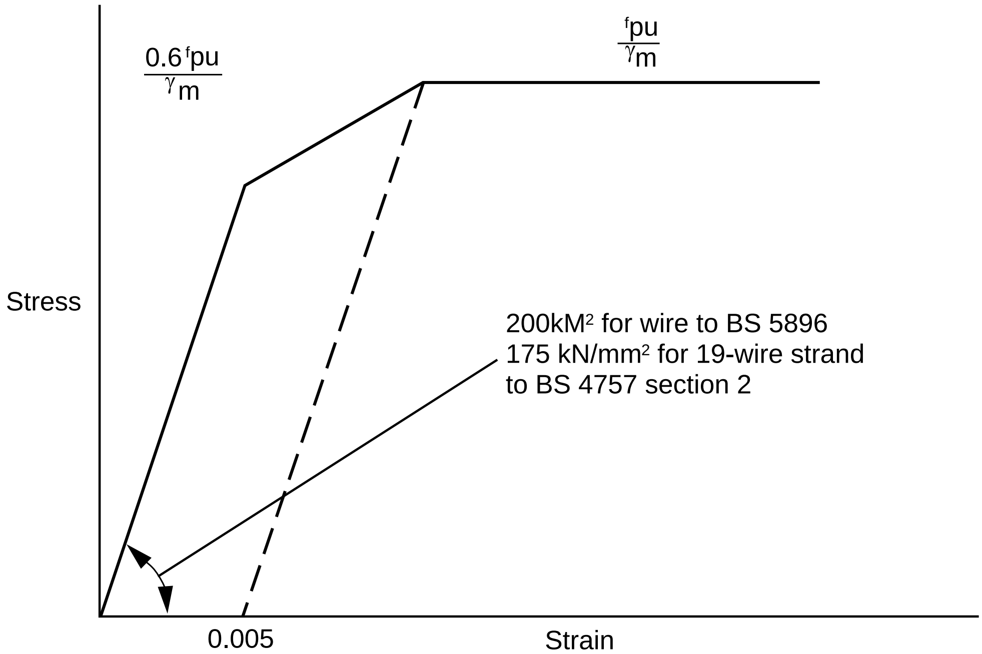


Figure 4 Short term assessment stress-strain curve for 'as drawn' wire and 'as spun' strand

4.4.3 Analysis for ultimate limit state Elastic methods may be used to determine the distribution of forces and deformations throughout the structure. Stiffness constants may be based on any of those listed in 4.4.2.1. However non-linear and plastic methods of analysis (e.g. plastic hinge methods for beams, or yield line methods for slabs) may be used with the agreement of the Overseeing Department.

4.5 Analysis of section

4.5.1 Serviceability limit state An elastic analysis shall be carried out. In-plane shear flexibility in concrete flanges (shear lag effects) shall 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 shall 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 When required by the Overseeing Department, deflection shall 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 calculations of deflections may be based on the information given in 4.3.2.1 or in Appendix A of BS 5400:Part 4.

4.7 Fatigue When required by the Overseeing Department, the effect of repeated live loading on the fatigue strength of a bridge shall be considered. For reinforcing bars that have been subjected to welding; details of compliance criteria are given in BS 5400 Part 10 as implemented by BD 9.

For unwelded non-corroded reinforcement the fatigue life shall be determined in accordance with BS 5400 Part 10 as implemented by BD 9, using the following parameters for the σ_r - N relationship.

bars $\leq$ 16mm diameter; $m = 9$ $k_2 = 0.75 \times 10^{27}$
bars > 16mm diameter; $m = 9$ $k_2 = 0.07 \times 10^{27}$

However, where the stress range under load combination 1 HA loading for the serviceability limit state is within the following values, no further fatigue check is required.

spans	bars ≤ 16mm dia	bars > 16mm dia
less than 3.5m	280 N/mm²	220 N/mm²
3.5m -5m	250 N/mm²	190 N/mm²
5m - 10m	195 N/mm²	150 N/mm²
10m - 200m	155 N/mm²	120 N/mm²
200m and greater	250 N/mm²	190 N/mm²

The fatigue life of corroded reinforcement shall be determined in accordance with BA 38.

4.8 Combined global and local effects

4.8.1 General In addition to the assessment 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 shall be combined as appropriate.

4.8.3 <u>Analysis of section</u> Section analysis for combined global and local effects shall be carried out in accordance with 4.5 to satisfy the requirements of 4.1.

(a) <u>Serviceability limit state</u>

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

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

(b) <u>Ultimate limit state</u> The resistance of the section to direct and flexural effects shall be derived from the direct strain due to global effects combined with the flexural strain due to local effects.

5. ASSESSMENT: REINFORCED CONCRETE

5.1 General

5.1.1 Introduction This clause gives methods of assessment which will in general ensure that, for reinforced concrete structures, the requirements set out in 4.1.1 and 4.1.2 are met. Other methods may be used with the approval of the Overseeing Department. 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 assessment of reinforced concrete

5.1.2.1 Basis of assessment Clause 5 follows the limit state philosophy set out in clauses 3 and 4. Usually a structure will need to be assessed only at the ultimate limit state. A serviceability limit state assessment is required only when specifically requested by the Overseeing Department.

5.1.2.2 Durability In 5.8.2 guidance is given on the nominal cover to reinforcement that is necessary 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 4.1.3.

5.1.3 Loads In Clause 5 the assessment 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 loads to be used in assessment are derived from BD 21 and 4.2.

In Clause 5, when analysing sections, the terms `strength', `resistance' and `capacity' are used to describe the assessment resistance of the section (see BD 21).

5.1.4 Strength of materials

5.1.4.1 Definition of strengths Throughout Clause 5 the symbol f_{cu} represents either the characteristic or the worst credible cube strength of concrete; and the symbol f_y represents either the characteristic or the worst credible reinforcement strength.

The assessment strengths of concrete and reinforcement are given by f_{cu} and γ_{mc}, respectively where γ_{mc} and γ_{ms} are the appropriate material partial safety factors given in 4.3.3.3. The appropriate value of γ_{mc} or γ_{ms} has to be substituted in all equations in Clause 5.

5.1.4.2 Strength of concrete Assessment may be based on either the specified characteristic cube strength, or the worst credible cube strength assessed as the lower bound to the estimated in-situ cube strength determined in accordance with BS 6089.

5.1.4.3 Strength of reinforcement Assessment may be based on either the specified characteristic yield or proof stress, or the worst credible yield or proof stress assessed from tests on reinforcement samples extracted from the structure.

5.2 Structures and structural frames

5.2.1 Analysis of structures Structures shall be analysed in accordance with the requirements 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 rotational 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) $0.008 + 0.035\ (0.5 - d_c/d_e)$

or

(2) $\dfrac{0.6\phi}{d-d_c}$ *but not less than* 0

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.

(b) Proper account is taken of changes in transverse moments and transverse shears consequent on redistribution of longitudinal moments.

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

5.3 Beams

5.3.1 General

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

(a) the distance between the centres of bearings or other supports

(b) the clear distance between supports plus the effective depth.

(c) for members resting directly on masonry, concrete or brick, the distance between the centroids of the bearing pressure diagrams. In this case, the bearing pressure diagrams shall be determined by assuming that the reaction is distributed linearly from a maximum at the front edge of the support to zero at the back of the bearing area. The length of the bearing area shall not be taken as greater than the depth of the beam where the support is of soft brick, or one-quarter of the depth of the beam where the support is of hard material such as granite or concrete.

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

The effective span of a continuous member shall 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 shall 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 shall 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 BS 5400: Part 3 as implemented by BD 13), the effective flange width shall 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 adequate lateral stability, a simply supported or continuous beam shall be so proportioned that the clear distance between lateral restraints should not exceed 300 b_c^2/d, where d is the effective depth to tension reinforcement and 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 shall not exceed 150 b_c^2/d.

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 shall 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 the appropriate value of γ_{mc} given in 4.3.3.3 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 0.6 f_{cu}/γ_{mc} 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 concrete is ignored.

(d) The stresses in the reinforcement are derived from either the stress-strain curves in Figure 2 or, when available, manufacturers' stress-strain curves. The values of γ_{ms} are given in 4.3.3.3.

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 0.1 f_{cu} times the cross-sectional area.

5.3.2.2 Design charts The design charts that form Parts 2 and 3 of CP 110, based on Figure 1, Figure 2 and the assumptions of 5.3.2.1, which, with appropriate modifications for the value of γ_m, may be used for the analysis of beams reinforced in tension only or in tension and compression.

5.3.2.3 Assessment 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 0.6 f_{cu}/γ_{mc} has been assumed.

$$M_u = (f_y/\gamma_{ms})A_s z \qquad \text{equation 1}$$

$$M_u = (0.225\ f_{cu}/\gamma_{mc})bd^2 \qquad \text{equation 2}$$

$$M_u = (0.66\ f_{cu}/\gamma_{mc})bx(d-0.5x) + f'_s A'_s(d-d') \qquad \text{equation 3}$$

$$(f_y/\gamma_{ms})A_s = (0.6 f_{cu}/\gamma_{mc})bx + f'_s A'_s \qquad \text{equation 4}$$

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, or worst credible strength, of the reinforcement;

$$f'_s = \frac{f_y}{\gamma_{ms} + f_y/2000}$$

x	is the depth of the neutral axis;
z	is the lever arm;
f_{cu}	is the characteristic, or worst credible strength of the concrete;
γ_{mc}	and γ_{ms} are the material partial safety factors given in 4.3.3.3.

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

$$z = [1 - \frac{0.84(f_y/\gamma_{ms})A_s}{(f_{cu}/\gamma_{mc})bd}]d \qquad \text{equation 5}$$

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

When using equations 3 and 4 for sections with compression reinforcement the neutral axis depth x should first be calculated from equation 4.

If $d' \leq 0.429x$ then the ultimate resistance moment should be determined from equation 3.

If $d' > 0.429x$ then either the compression steel should be ignored and the section treated as singly reinforced or the ultimate resistance determined using 5.3.2.1 or 5.3.2.2 as appropriate.

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.

$$M_u = (f_y/\gamma_{ms})A_s(d - \frac{h_f}{2})$$ equation 6

$$M_u = (0.6f_{cu}/\gamma_{mc})bh_f(d - \frac{h_f}{2})$$ equation 7

5.3.3 Shear resistance of beams

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

$$v = \frac{V}{b_w d}$$ equation 8

where

V is the shear force due to ultimate loads;
b_w is the breadth of the section which, for a flanged beam, shall be taken as the rib width;
d is the effective depth to tension reinforcement.

In no case shall v exceed $0.92\sqrt{f_{cu}/\gamma_{mc}}$ *or* $7/\sqrt{\gamma_{mc}}$ N/mm^2 (where γ_{mc} is the partial safety factor for concrete given in 4.3.3.3), whichever is the lesser, whatever shear reinforcement is provided.

5.3.3.2 Shear capacity Shear reinforcement may take the form of vertical links, inclined links or bent-up bars, and shall only be considered effective in resisting shear if the spacing of the legs of links in the direction of the span and at the right angles to it, does not exceed the effective depth, d, and if

$$A_{sv}(\sin\alpha + \cos\alpha)(f_{yv}/\gamma_{ms}) \geq 0.2\, b_w s_v \text{ and } \alpha \geq 30°$$

where

A_{sv} is the cross-sectional area of shear reinforcement at a particular cross-section;
s_v is the spacing of the shear reinforcement along the member;
α is the inclination of the shear reinforcement to the axis of the member;
f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 480 N/mm²;
γ_{ms} is the material partial safety factor for steel given in 4.3.3.3.

In the absence of effective shear reinforcement, the ultimate shear resistance, V_u, of a section is given by

$$V_u = \xi_s v_c b_w d.$$

Where effective vertical links are present the ultimate shear resistance V_u of a section shall be taken as

$$V_u = \xi_s v_c b_w d + (f_{yv}/\gamma_{ms})\frac{d}{s_v}A_{sv}$$

For vertical links to be effective, the tensile capacity of the longitudinal reinforcement at a section, $A_s f_y/\gamma_{ms}$, must begreater than

$$\frac{M}{z} + \frac{(V - \xi_s v_c b_w d)}{2}$$

where M and V are the co-existent ultimate bending moment and shear force at the section under consideration and z is the lever arm which may be taken as 0.9d. However, within an individual sagging or hogging region, the tensile capacity need not exceed M_{max}/z, where M_{max} is the maximum ultimate moment within that region.

In the above expressions

$$\xi_s = \left(\frac{550}{d}\right)^{1/4} \quad but \not< \ 0.7;$$

v_c is the ultimate shear stress in concrete and shall be calculated as follows:

$$v_c = \frac{0.24}{\gamma_{mv}} \left(\frac{100A_s}{b_w d}\right)^{1/3} (f_{cu})^{1/3}$$

In the formula for v_c the term $\left(100\frac{A_s}{b_w d}\right)$ should not be taken less than 0.15 or greater than 3.0.

γ_{mv} is a partial safety factor which is taken as 1.25 if the characteristic concrete strength is used, and 1.15 if the worst credible concrete strength is used.

The term A_s is that area of longitudinal tension reinforcement which continues at least a distance equal to the effective depth, d, beyond the section being considered. However, at supports the full area of tensile reinforcement may be used provided, the requirements of 5.8.7 are met. Where both top and bottom reinforcement are provided, the area of A_s used shall be that which is in tension under the loading which produces the shear force being considered.

Sections within a distance d from the support need not be assessed for shear providing the shear reinforcement calculated for the section at distance d is continued up to the support and the anchorage requirements of 5.8.7 are met.

Inclined links or bent up bars shall 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 link or bar shall be taken as

$$\frac{f_{yv}}{\gamma_{ms}} \ .$$

Bent-up bars shall be checked for anchorage (see 5.8.6.3) and bearing (see 5.8.6.8).

5.3.3.3 **Enhanced shear strength of sections close to supports**

If the main reinforcement continues to the support and is provided with an effective anchorage equivalent to 20 times one bar size (see Cl. 5.8.7) then an enhancement of shear strength may be allowed for sections within a distance a_v < 3d from the face of a support, front edge of a rigid bearing or centre line of a flexible bearing.

This enhancement shall take the form of an increase in the allowable shear stress,

$\xi_s v_c$ *to* $\xi_s v_c \ x \ 3d/a_v$ but should not exceed $0.92 \sqrt{f_{cu}/\gamma_{mc}}$ *or* $7/\sqrt{\gamma_{mc}}$ N/mm^2 whichever is the lesser.

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 shall be present in addition to any reinforcement required to resist shear.

5.3.4 Torsion

5.3.4.1 General In some members. the maximum torsional moment does not occur under the same loading as the maximum flexural moment. In such circumstances reinforcement in excess of that required for flexure and other forces may be used in torsion.

5.3.4.2 Torsionless systems In general, where the torsional resistance or stiffness of members has not been taken into account in the analysis of the structure, no specific calculations for torsion will be necessary. However, in applying this clause it is essential that sound engineering judgement has shown that torsion plays only a minor role in the behaviour of the structure, otherwise torsional stiffness shall be used in analysis.

5.3.4.3 Stresses and reinforcement Where torsion in a section substantially increases the shear stresses, the torsional shear stress shall be calculated assuming a plastic stress distribution.

Where the torsional shear stress, v_t, exceeds the value v_{tmin}, (given below), torsion reinforcement should be present.

$$\text{where} \quad v_{tmin} = 0.082\sqrt{f_{cu}/\gamma_{mc}}$$

In no case shall the sum of the shear stresses resulting from shear force and torsion ($v + v_t$) exceed the value of the ultimate shear stress, v_{tu}, nor, in the case of small sections ($y_1 < 550$mm), shall the torsional shear stress, v_t, exceed v_{tu} $y_1/550$, where y_1 is the larger centre line dimension of a link and,

$$v_{tu} = 0.92\sqrt{f_{cu}/\gamma_{mc}} \quad but \not> 7/\sqrt{\gamma_{mc}}$$

Torsion reinforcement shall consist of rectangular closed links in accordance with 5.8.6.5 together with longitudinal reinforcement. Only reinforcement in excess of that required to resist shear or bending shall be considered as torsion reinforcement.

Torsional capacity shall be calculated assuming that the closed links form a thin-walled tube, the shear stresses in which are balanced by longitudinal and transverse forces provided by the resistance of the reinforcement.

5.3.4.4 Treatment of various cross sections

(a) Box sections The ultimate torsional strength (T_u) shall be taken as the greater of:

$$T_u = 2A_o\sqrt{\left\{\frac{A_{sL}(f_{yL}/\gamma_{ms})}{2(x_1 + y_1)}\right\}\left\{\frac{A_{st}(f_{yv}/\gamma_{ms})}{s_v}\right\}} \qquad \text{equation 10/11A}$$

and

$$T_u = 2h_w A_o v_{tmin} \qquad \text{equation 9A}$$

where

h_w is the thickness of the thinnest wall;
A_o is the area enclosed by the median wall line;
A_{st} is the area of one leg of a closed link of a section;
A_{sL} is the area of one bar of longitudinal reinforcement;
f_{yv} is the characteristic, or worst credible, strength of the links;
f_{yL} is the characteristic, or worst credible, strength of the longitudinal reinforcement;
s_v is the spacing of the links along the member;
s_L is the average spacing of the longitudinal reinforcement.

In equation 10/11A, f_{yv} and f_{yL} shall not be taken as greater than 480 N/mm².

In addition, the torsional shear stress calculated from:

$$v_t = \frac{T}{2h_w A_o} \qquad \text{equation 9}$$

shall satisfy the requirements of 5.3.4.3. T is the torque due to ultimate loads.

(b) Rectangular sections The ultimate torsional resistance shall be taken as the greater of the value calculated from equation 10/11A (with A_o taken as 0.8 x_1y_1, where x_1 and y_1 are the smaller and larger centre line dimensions, respectively, of the links), and

$$T_u = \frac{h^2_{min}}{2}\left(h_{max} - \frac{h_{min}}{3}\right) v_{tmin} \qquad \text{equation 9(a)A}$$

where h_{min} and h_{max} are, respectively, the smaller and larger dimensions of the section.

In addition, the torsional shear stress calculated from:

$$v_t = \frac{2T}{h^2_{min}\left(h_{max} - \frac{h_{min}}{3}\right)} \qquad \text{equation 9(a)}$$

shall satisfy the requirements of 5.3.4.3.

(c) T,L and I sections Such sections shall be divided into component rectangles for purposes of torsional

assessment. Any division into component rectangles may be chosen which is compatible with the torsional reinforcement present. The ultimate torsional resistance of each component rectangle shall then be determined using 5.3.4.4(b), and the sectional torsional resistance taken as the sum of the torsional resistances of the component rectangles.

In addition, the torsional shear stress in each component rectangle shall be calculated from equation 9(a) and shall satisfy the requirements of 5.3.4.3.

A component rectangle shall be treated as reinforced for torsion only if its link reinforcement ties it to its adjacent rectangles.

5.3.4.5 Detailing A section shall be treated as reinforced for torsion only if the pitch of the closed links is less than the smaller of $(x_1 + y_1)/4$ or 16 longitudinal corner bar diameters. The diameter of the longitudinal corner bars shall not be less than the diameter of the links.

In areas subjected to simultaneous flexural compressive stress, the value of $\sum A_{sL}$ of reinforcement in the compressive zone used in equation 10/11A may be notionally increased by:

f_{cav} (area of section subject to flexural compression)/(f_{yL} /γ_{ms})

where f_{cav} is the average compressive stress in the flexural compressive zone.

In the case of beams, the depth of the compression zone used to calculate the area of section subject to flexural compression shall be taken as twice the cover to the closed links.

5.3.5 Longitudinal shear For flanged beams, the longitudinal shear resistance at the horizontal flange/web junction and across vertical sections of the flange which may be critical shall be checked in accordance with 7.4.2.3.

5.3.6 Deflection in beams If required by the Overseeing Department, deflections may be calculated in accordance with clause 4.6.

5.3.7 Crack control in beams If required by the Overseeing Department, flexural crack widths in beams shall be calculated in accordance with 5.8.8.2.

5.4 Slabs

5.4.1 Moments and shear forces in slabs Moments and shear forces in slab bridges and in the top slabs of beam and slab, voided slab and box beam bridges may be obtained from a general elastic analysis, or such particular elastic analyses as those due to Westergaard or Pucher. Non-linear methods may also be used. Alternatively, Johansen's yield line method may be used to obtain the slab strength directly. When using non-elastic methods, the agreement of the Overseeing Department shall be obtained.

The effective spans shall be in accordance with 5.3.1.1.

5.4.2 Resistance moments of slabs The ultimate resistance moment in a reinforcement direction may be determined by the methods given in 5.3.2. In assessing whether the reinforcement can resist a combination of two bending moments and a twisting moment at a point in a slab, allowance shall be made for the fact that the principal moment and reinforcement directions do not generally coincide. This should be done by checking the strength in all directions.

In voided slabs, the transverse flexural strength shall be calculated allowing for the effects of transverse shear by an appropriate analysis (e.g. an analysis based on the assumption that the transverse section acts as a Vierendeel frame).

5.4.3 Resistance to in-plane forces When checking whether reinforcement can resist a combination of two in-plane direct forces and an in-plane shear force, allowance shall be made for the fact that the principal stress and reinforcement directions do not generally coincide. This should be done by checking the strength in all directions.

5.4.4 Shear resistance of slabs

5.4.4.1 Shear stress in solid slabs: general The shear stress, v, at any cross section in a solid slab, shall be calculated from:

$$v = \frac{V}{bd}$$ equation 12

where

V is the shear force due to ultimate loads;
b is the width of slab under consideration;
d is the effective depth to tension reinforcement.

In no case shall v exceed the appropriate maximum value given in 5.3.3.1 for beams.

The shear capacity shall be assessed in accordance with 5.3.3.2 and 5.3.3.3, with the following amendments:

(a) b_w shall be replaced with b in all equations;
(b) shear reinforcement shall not be considered as effective in slabs less than 200mm thick.

5.4.4.2 Shear stresses in solid slabs under concentrated loads (including wheel loads) When considering this clause the dispersal of wheel loads allowed in BD 37 shall be taken to the top surface of the concrete slab only and not down to the neutral axis.

The critical section for calculating shear shall be taken on a perimeter 1.5d from the boundary of the loaded area, as shown in figure 5A(a) where d is the effective depth to the flexural tension reinforcement. Where concentrated loads occur on a cantilever slab or near unsupported edges, the relevant portions of the critical section shall be taken as the worst case from (a), (b) or (c) of figure 5A. For a group of concentrated loads, adjacent loaded areas shall be considered singly and in combination using the preceding requirements.

The ultimate punching shear capacity, V_u, is given by:

$$V_u = V_c + \sum A_{sv} \sin\alpha\ (f_{yv}/\gamma_{ms})$$ equation 13A

where

f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 480 N/mm²;
γ_{ms} is the material partial safety factor for steel given in 4.3.3;
$\sum A_{sv}$ is the area of shear reinforcement within the area between the loaded area and the critical perimeter, except for case c(ii) of figure 5A when it is the area of shear reinforcement within a distance from the load equal to the effective depth. However, shear reinforcement shall be considered to be effective only if:

$$\sum A_{sv} \sin\alpha\ (f_{yv}/\gamma_{ms}) \geq 0.2 \sum bd$$

$\sum bd$ is the area of the critical section;
α is the inclination of the shear reinforcement to the plane of the slab;
V_c is the shear resistance of the concrete.

V_c shall be taken as the sum of the shear resistances of each portion of the critical perimeter (see figure 5A). The value of $100\ A_s/(bd)$ to be used to calculate v_c from 5.3.3.2 shall be derived by considering the effectively anchored flexural tensile reinforcement associated with each portion as shown in figure 5A.

The ultimate punching shear capacity shall also be checked on perimeters progressively 0.75d from the critical perimeter. The value of A_{sv} to be used in equation 13A is the area of shear reinforcement between the perimeter under consideration and a perimeter 1.5d within the perimeter under consideration.

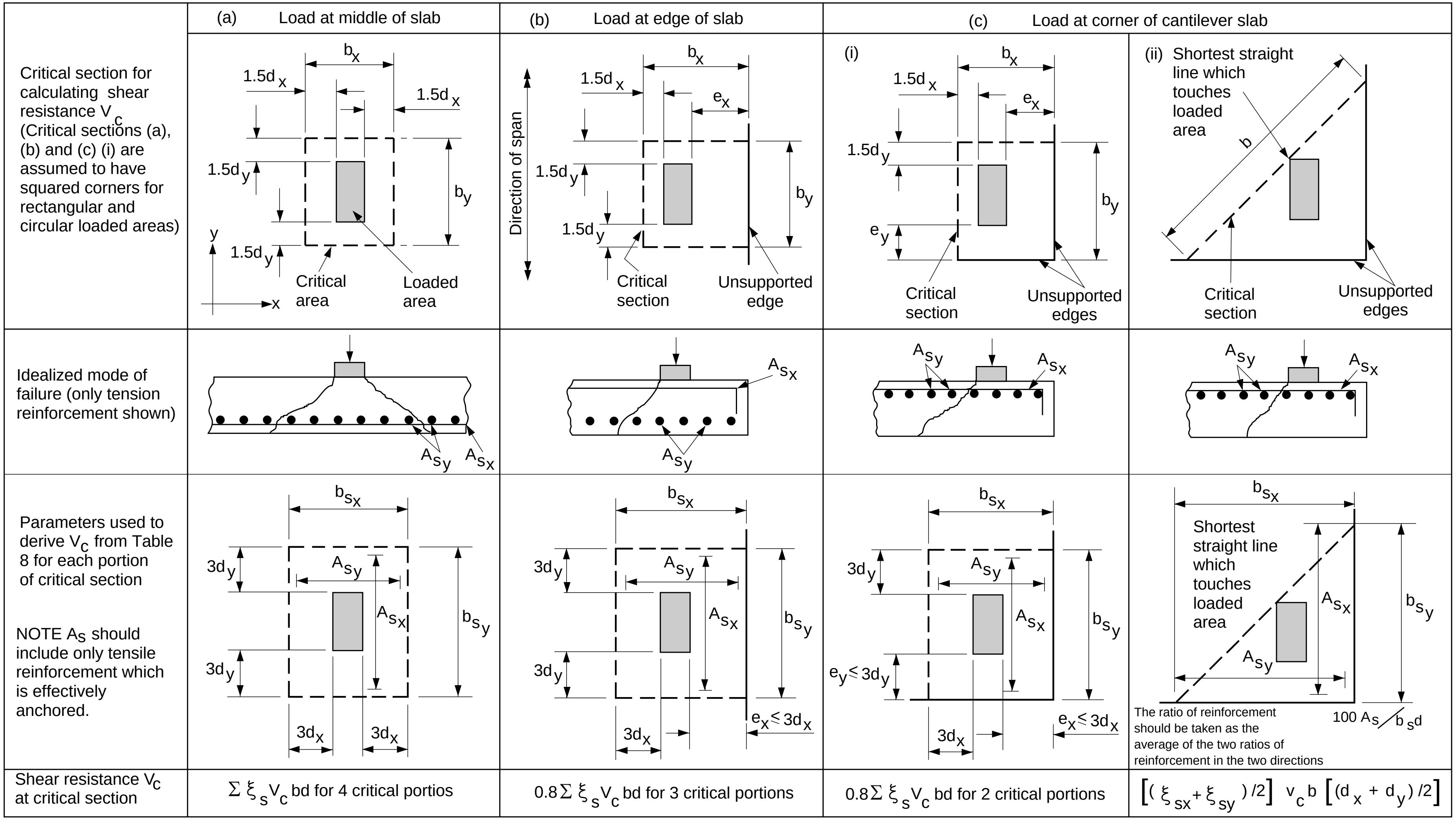


Figure 5A Parameters for shear in solid slabs under concentrared loads

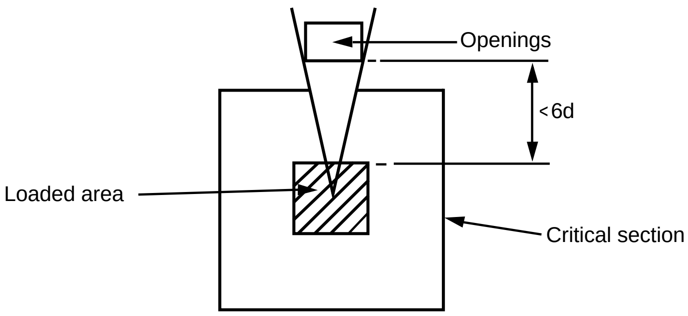


Figure 6 Openings in slabs

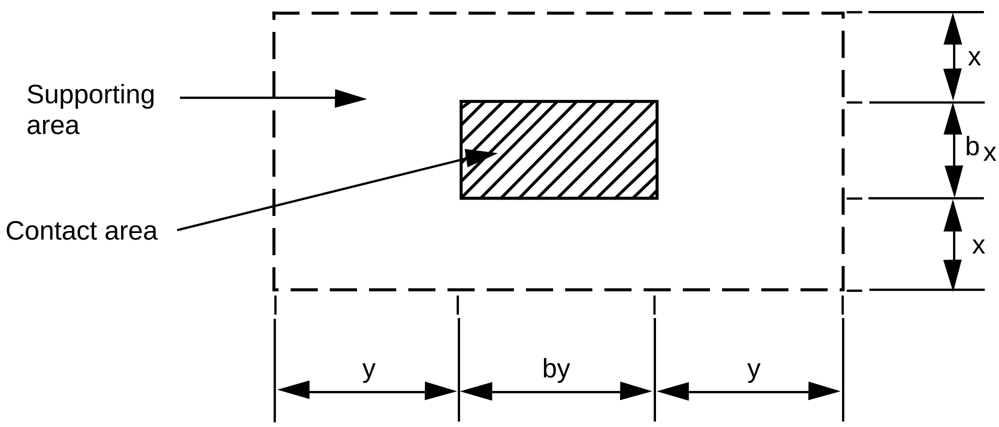


Figure 6A Bearing areas

If a part of a perimeter cannot, physically, extend 1.5d from the boundary of the loaded area, then the part perimeter shall be taken as far from the loaded areas as is physically possible and the value of v_c for that part may be increased by a factor $1.5d/a_v$, where a_v is the distance from the boundary of the loaded area to the perimeter actually considered.

When openings in slabs and footings (see figure 6) are located at a distance less than 6d from the edge of a concentrated load or reaction, then that part of the periphery of the critical section which is enclosed by radial projections of the openings to the centroid of the loaded area shall be considered as ineffective.

Where one hole is adjacent to the loaded area and its greatest width is less than one-quarter of the side of the loaded area or one-half of the slab depth, whichever is the lesser, its presence may be ignored.

5.4.4.3 Shear in voided slabs The longitudinal ribs between the voids shall be assessed as beams (see 5.3.3) to resist the shear forces in the longitudinal direction including any shear due to torsional effects.

The top and bottom flanges, acting as solid slabs, shall each be capable of resisting a part of the global transverse shear force proportional to the flange thickness. The top flange of a rectangular voided slab shall be capable of resisting the punching effect due to wheel loads (see 5.4.4.2). Where wheel loads may punch through the slab as a whole, this should also be checked.

5.4.5 Deflection of slabs If required by the Overseeing Department, deflections may be calculated in accordance with clause 4.6.

5.4.6 Crack control in slabs If required by the Overseeing Department, flexural crack widths in slabs shall be calculated in accordance with 5.8.8.2.

5.4.7 Torsion in slabs

5.4.7.1 Slab interior The assessment of interior regions of slabs to resist twisting moments shall be in accordance with 5.4.2.

5.4.7.2 Slab edges This clause is concerned with slab edge zones of width equal to the overall depth of the slab.

An edge zone shall be capable of resisting a total shear force of $(V_t b_e + M_{nt})$ when assessed in accordance with 5.3.3, with b_w taken as the width of the edge zone (b_e) which may be assumed to be equal to the slab overall depth (h). V_t is the flexural shear force per unit width at the edge acting on a vertical plane perpendicular to the edge, and M_{nt} is the twisting moment per unit length in the slab adjacent to the edge zone referred to axes perpendicular (n) and parallel (t) to the edge.

5.5 Columns

5.5.1 General

5.5.1.1 Definitions A reinforced concrete column is a compression member whose greater lateral dimension is less than or equal to four times its lesser lateral dimension, and in which the reinforcement is taken into account when considering its strength.

A column shall be considered as short if the ratio l_e/h in each plane of buckling is less than 12, where:

l_e is the effective height in the plane of buckling under consideration;
h is the depth of the cross section in the plane of buckling under consideration.

It shall otherwise be considered as slender.

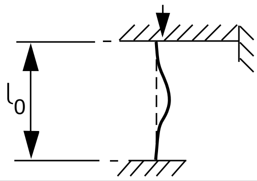
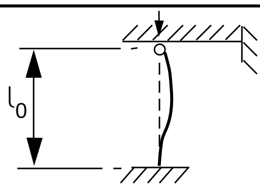
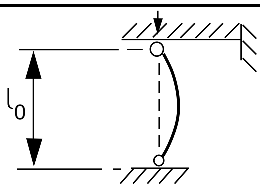
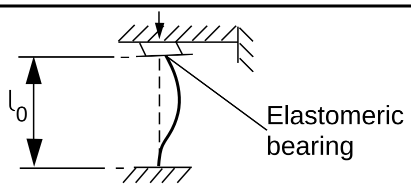
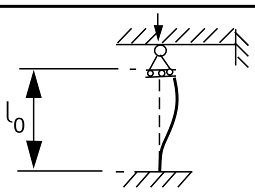
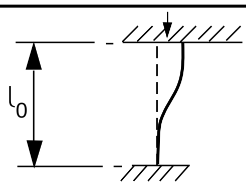
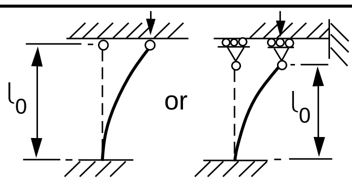
Case	Idealised column and buckling mode	Restraints: Location	Restraints: Position	Restraints: Rotation	Effective Height, l_e
1		Top	Full	Full	0.70 l_o
		Bottom	Full	Full	
2		Top	Full	None	0.85 l_o
		Bottom	Full	Full	
3		Top	Full	None	1.0 l_o
		Bottom	Full	None	
4		Top	None	None	1.3 l_o
		Bottom	Full	Full	
5		Top	None	None	1.4 l_o
		Bottom	Full	Full	
6		Top	None	Full	1.5 l_o
		Bottom	Full	Full	
7		Top	None	None	2.3 l_o
		Bottom	Full	Full	

Table 11 Effective height, l_e, for columns

5.5.1.2 Effective height of column The effective height, l_e, in a given plane may be obtained from Table 11 where l_o is the clear height between end restraints.

The values given in Table 11 are based on the following assumptions:

(a) rotational restraint is at least $4(EI)_c/l_o$ for cases 1,2 and 4 to 6 and $8(EI)_c/l_o$ for case 7, $(EI)_c$ being the flexural rigidity of the column cross section;
(b) lateral and rotational rigidity of elastomeric bearings are zero.

Where a more accurate evaluation of the effective height is required or where the end stiffness values are less than those values given in (a), the effective heights shall be derived from first principles.

The accommodation of movements and the method of articulation will influence the degree of restraint developed for columns. These factors shall be assessed as accurately as possible using engineering principles based on elastic theory and taking into account all relevant factors such as foundation flexibility, type of bearings, articulation system, etc.

5.5.1.3 Slenderness limits for columns In each plane of buckling, the ratio l_o/h shall not exceed 60.

5.5.1.4 Assessment of strength Subclauses 5.5.2 to 5.5.7 give methods, for assessing the strength of columns at the ultimate limit state, which are based on a number of assumptions. These methods may be used provided the assumptions are realized for the case being considered and the effective height is determined accurately. In addition, for columns subject to applied bending moments the Overseeing Department may require crack widths to be calculated at the serviceability limit state (see 4.1.1).

5.5.2 Moments and forces in columns The moments, shear forces and axial forces in a column should be determined in accordance with 4.4, except that if the column is slender the moments induced by deflection should be considered. An allowance for these additional moments is made in the assessment requirements for slender columns which follow, and the bases or other members connected to the ends of such columns shall also be capable of resisting these additional moments.

In columns with end moments it is generally necessary to consider the maximum and minimum ratios of moment to axial load.

5.5.3 Short columns subject to axial load and bending about the minor axis

5.5.3.1 General A short column shall be assessed at the ultimate limit state in accordance with the following requirements provided that the moment at any cross section has been increased by that moment caused by the actual eccentricity of the (assumed) axial load arising from construction tolerances. If the actual eccentricity has not been determined, the construction tolerance eccentricity shall be taken as equal to 0.05 times the overall depth of the cross section in the plane of bending, but not more than 20mm.

5.5.3.2 Analysis of sections When analysing a column cross section to determine its ultimate resistance to moment and axial load, the following assumptions shall be made.

(a) The strain distribution in the concrete in compression and the compressive and tensile strains in the reinforcement 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 the appropriate value of γ_{mc} from 4.3.3.3, or taken as equal to 0.6 f_{cu}/γ_{mc} over the whole compression zone where this is rectangular or circular. In both cases, the concrete strain at the outermost 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 the appropriate value of γ_{ms} from 4.3.3.3.

For rectangular and circular columns the following assessment methods, based on the preceding assumptions, may be used. For other column shapes, assessment methods shall be derived from first principles using the preceding assumptions.

5.5.3.3 Design charts for rectangular columns The design charts that form Parts 2 and 3 of CP 110 include charts (based on Figure 1, Figure 2 and the assumptions in 5.5.3.2) which, with appropriate modifications for the value of γ_m, may be used for the analysis of rectangular and circular column sections having a symmetrical arrangement of reinforcement.

5.5.3.4 Assessment formulae for rectangular columns The following formulae (based on a concrete stress of 0.6 f_{cu}/γ_{mc} over the whole compression zone and the assumptions in 5.5.3.2) may be used for the analysis of a rectangular column having longitudinal reinforcement in the two faces parallel to the axis of bending, whether that reinforcement is symmetrical or not. Both the ultimate axial load, N, and the ultimate moment, M, shall not exceed the values of N_u and M_u given by equation 14 and 15 for the appropriate value of d_c.

$$N_u = (0.6\ f_{cu}/\gamma_{mc})bd_c + f_{yc}A'_{sl} + f_{s2}A_{s2} \qquad \text{equation 14}$$

$$M_u = (0.3\ f_{cu}/\gamma_{mc})bd_c(h-d_c) + f_{yc}\ A'_{sl}(\frac{h}{2} - d') - f_{s2}\ A_{s2}\ (\frac{h}{2} - d_2) \qquad \text{equation 15}$$

where
N is the ultimate axial load applied on the section considered;
M is the moment applied about the axis considered due to ultimate loads including the allowance for construction tolerance (see 5.5.3);
N_u, M_u are the ultimate axial load and bending capacities of the section for the particular value of d_c assumed;
f_{cu} is the characteristic, or worst credible, cube strength of the concrete;
b is the breadth of the section;
d_c is the depth of concrete in compression assumed subject to a minimum value of 2d';
f_{yc} is the assessment compressive strength of the reinforcement (in N/mm^2) taken as:

$$\frac{f_y}{\gamma_{ms} + f_y/2000}$$

A'_{sl} is the area of compression reinforcement in the more highly compressed face;
f_{s2} is the stress in the reinforcement in the other face, derived from Figure 2 and taken as negative if tensile;
A_{s2} is the area of reinforcement in the other face which may be considered as being:
(1) in compression,
(2) inactive, or
(3) in tension,

as the resultant eccentricity of load increases and d_c decreases from h to 2d´;

h is the overall depth of the section in the plane of bending;
d´ is the depth from the surface to the reinforcement in the more highly compressed face;
d_2 is the depth from the surface to the reinforcement in the other face;
f_y is the characteristic or worst credible strength of reinforcement.

5.5.3.5 Simplified design formulae for rectangular columns *"Not applicable to assessment"*

5.5.4 Short columns subject to axial load and either bending about the major axis or biaxial bending The moment about each axis due to ultimate loads shall be increased by that moment caused by the actual eccentricity arising from construction tolerances of the (assumed) axial load. If the actual eccentricity has not been determined, the construction tolerance eccentricity shall be taken as equal to 0.03 times the overall depth of the cross section in the appropriate plane of bending, but not more than 20mm.

For square and rectangular columns having a symmetrical arrangement of reinforcement about each axis, the section may be analysed for axial load and bending about each axis in accordance with any one of the methods of assessment given in 5.5.3.2, 5.5.3.3 or 5.5.3.4. The following relationship shall be satisfied:

$$\left[\frac{M_x}{M_{ux}}\right]^{\alpha_n} + \left[\frac{M_y}{M_{uy}}\right]^{\alpha_n} \leq 1.0 \qquad \text{equation 16}$$

where

M_x and M_y are the moments about the major x-x axis and minor y-y axis respectively due to ultimate loads including the allowance for construction tolerances (see preceding paragraph);
M_{ux} is the ultimate moment capacity about the major x-x axis assuming an ultimate axial load capacity, N_u, not less than the value of the ultimate axial load, N;
M_{uy} is the ultimate moment capacity about the minor y-y axis assuming an ultimate axial load capacity, N_u, not less than the value of the ultimate axial load, N;
α_n is given by $\alpha_n = 0.67 + 1.66\, N_u/N_{uz}$ but $\nless$ 1.0 and $\ngtr$ 2.0 where N_{uz} is the axial loading capacity of a column ignoring all bending, taken as:

$$N_{uz} = (0.675\, f_{cu}/\gamma_{mc})\, A_c + f_{yc}\, A_{sc} \qquad \text{equation 17}$$

where f_{cu} and f_{yc} are as defined in 5.5.3.4
A_c is the area of concrete
A_{sc} is the area of longitudinal reinforcement.

For other column sections, assessment shall be in accordance with 5.5.3.2.

5.5.5 Slender columns

5.5.5.1 General A cross section of a slender column may be assessed by the methods given for a short column (see 5.5.3 and 5.5.4) but, in the assessment, account shall be taken of the additional moments induced in the column by its deflection. For slender columns of constant rectangular or circular cross section having a symmetrical arrangement of reinforcement, the column shall be able to resist the ultimate axial load, N, together with the moments M_{tx} and M_{ty} derived in accordance with 5.5.5.4. Alternatively, the simplified formulae given in 5.5.5.2 and 5.5.5.3 may be used where appropriate; in this case the moment due to ultimate loads need not be increased by the allowance for construction tolerances given in 5.5.3. It will be sufficient to limit the minimum value of moment to not less than the allowance given in 5.5.3.

5.5.5.2 Slender columns bent about a minor axis A slender column of constant cross section bent about the minor y-y axis should be assessed for its ultimate axial load, N, together with the moment M_{ty} given by:

$$M_{ty} = M_{iy} + \frac{Nh_x}{1750}\left(\frac{l_e}{h_x}\right)^2\left(1-\frac{0.0035 l_e}{h_x}\right)$$ equation 18

where

M_{iy} is the initial moment due to ultimate loads, but not less than that corresponding to the allowance for construction tolerances as given in 5.5.3;

h_x is the overall depth of the cross section in the plane of bending M_{ty} ;

l_e is the effective height either in the plane of bending or in the plane at right-angles, whichever is greater.

For a column fixed in position at both ends where no transverse loads occur in its height the value of M_{iy} may be reduced to:

$$M_{iy} = 0.4\ M_1 + 0.6\ M_2$$ equation 19

where

M_1 is the smaller initial end moment due to ultimate loads (assumed negative if the column is bent in double curvature):

M_2 is the larger initial end moment due to ultimate loads (assumed positive).
In no case, however, shall M_{iy} be taken as less than 0.4 M_2 or such that M_{ty} is less than M_2 .

5.5.5.3 Slender columns bent about a major axis When the overall depth of its cross section, h_y, is less than three times the width, h($_{hx}$), a slender column bent about the major x-x axis shall be assessed for its ultimate axial load, N, together with the moment M_{tx} given by:

$$M_{tx} = M_{ix} + \frac{Nh_y}{1750}\left(\frac{l_e}{h_x}\right)^2\left(1-\frac{0.0035 l_e}{h_x}\right)$$ equation 20

where l_e and h_x are as defined in 5.5.5.2;

M_{ix} is the initial moment due to ultimate loads, but not less than that corresponding to the allowance for construction tolerances as given in 5.5.3;

h_y is the overall depth of the cross section in the plane of bending M_{tx} .

Where h_y is equal to or greater than three times h_x , the column shall be considered as biaxially loaded with the moment about the minor axis equal to that due to construction tolerances (see 5.5.3).

5.5.5.4 Slender columns bent about both axes A slender column bent about both axes shall be assessed for its ultimate axial load, N, together with the moments M_{tx} about its major axis and M_{ty} about its minor axis, given by:

$$M_{tx} = M_{ix} + \frac{Nh_y}{1750}\left(\frac{l_{ex}}{h_y}\right)^2\left(1-\frac{0.0035 l_{ex}}{h_y}\right)$$ equation 21

$$M_{ty} = M_{iy} + \frac{Nh_x}{1750}\left(\frac{l_{ey}}{h_x}\right)^2\left(1-\frac{0.0035 l_{ey}}{h_x}\right)$$ equation 22

where

h_x and h_y are as defined in 5.5.5.2 and 5.5.5.3 respectively;

M_{ix} is the initial moment due to ultimate loads about the x-x axis, including the allowance for construction tolerances (see 5.5.4);

M_{iy} is the initial moment due to ultimate loads about the y-y axis, including the allowance for construction tolerances (see 5.5.4);

l_{ex} is the effective height in respect of bending about the major axis;

ℓ_{ey} is the effective height in respect of bending about the minor axis.

5.5.6 Shear resistance of columns A column subject to uniaxial shear due to ultimate loads shall be assessed in accordance with 5.5.3 except that the ultimate shear stress, $\xi_s v_c$, may be multiplied by:

$$1 + \frac{0.07N}{A_c}$$

where

N is the ultimate axial load (in Newtons)

A_c is the area of the entire concrete section (in mm²)

A column subjected to biaxial shear due to ultimate loads shall satisfy the expression:

$$\frac{V_x}{V_{ux}} + \frac{V_y}{V_{uy}} \leq 1.0$$

where

V_x and V_y are the applied shears due to ultimate loads for the x-x axis and y-y axis respectively,

V_{ux} and V_{uy} are the corresponding ultimate shear capacities of the concrete and link reinforcement for the x-x and y-y axis respectively derived allowing for the enhancement factor given in this clause.

In calculating the ultimate shear capacity of a circular column, the area of longitudinal reinforcement A_s to be used to calculate v_c shall be taken as the area of reinforcement which is in the half of the column opposite the extreme compression fibre. The effective depth shall be taken as the distance from the extreme fibre with maximum compression to the centroid of this reinforcement. The web width shall be taken as the column diameter.

5.5.7 Crack control in columns If required by the Overseeing Department, a column subjected to bending shall be considered as a beam for the purpose of calculating flexural crack widths (see 5.8.8.2).

5.6 Reinforced concrete walls

5.6.1 General

5.6.1.1 Definition A reinforced wall is a vertical load-bearing concrete member whose greater lateral dimension is more than four times its lesser lateral dimension, and in which the reinforcement is taken into account when considering its strength.

Retaining walls, wing walls, abutments, piers and other similar elements subjected principally to bending moments, and where the ultimate axial load is less than 0.1 $f_{cu} A_c$, may be treated as cantilever slabs and assessed in accordance with 5.4. In other cases, this clause applies.

A reinforced wall shall be considered as either short or slender. In a similar manner to columns, a wall shall be considered as short where the ratio of its effective height to its thickness does not exceed 12. It shall otherwise be considered as slender.

5.6.1.2 Limits to slenderness The slenderness ratio is the ratio of the effective height of the wall to its thickness. The effective height shall be obtained from table 11. When the wall is restrained in position at both ends and the reinforcement complies with the requirements of 5.8.4, the slenderness ratio shall not exceed 40 unless more than 1% of vertical reinforcement is provided, when the slenderness ratio may be up to 45.

When the wall is not restrained in position at one end the slenderness ratio should not exceed 30.

5.6.2 Forces and moments in reinforced concrete walls Forces and moments shall be calculated in accordance with 4.4 except that, if the wall is slender, the moments induced by deflection shall also be considered. The distribution of axial and horizontal forces along a wall from the loads on the superstructure shall be determined by analysis and their points of application decided by the nature and location of the bearings. For walls fixed to the deck, the moments shall similarly be determined by elastic analysis.

Unless the actual eccentricity of load is determined, the moment per unit length in the direction at right-angles to a wall shall be taken as not less than 0.05 $n_w h$, where n_w is the ultimate axial load per unit length and h is the thickness of the wall. Moments in the plane of a wall can be calculated from statics for the most severe positioning of the relevant loads.

Where the axial load is non-uniform, consideration shall be given to deep beam effects and the distribution of axial loads per unit length of wall.

It will generally be necessary to consider the maximum and minimum ratios of moment to axial load in assessing a wall.

5.6.3 Short reinforced walls resisting moments and axial forces Each cross section of the wall shall be capable of resisting the appropriate ultimate axial load and the transverse moment per unit length calculated in accordance with 5.6.2. The assumptions made when analysing beam sections (see 5.3.2.1) apply and also when the wall is subject to significant bending only in the plane of the wall.

When the wall is subjected to significant bending both in the plane of the wall and at right-angles to it, consideration shall be given first to bending in the plane of the wall in order to establish a distribution of tension and compression along the length of the wall. The resulting tension and compression shall then be combined with the compression due to the ultimate axial load to determine the combined axial load per unit length of wall. This may be done by an elastic analysis assuming a linear distribution along the wall.

The bending moment at right-angles to the wall shall then be considered and the section checked for this moment and the resulting compression or tension per unit length at various points along the wall length, using the assumptions of 5.3.2.1.

5.6.4 Slender reinforced walls The distribution of axial load along a slender reinforced wall shall be determined as for a short wall. The critical portion of wall shall then be considered as a slender column of unit width and assessed as such in accordance with 5.5.5.

5.6.5 Shear resistance of reinforced walls A wall subject to uniaxial shear due to ultimate loads shall be assessed in accordance with 5.4.4.1 except that the ultimate shear stress, $\xi_s v_c$, may be multiplied by:

$$1 + \frac{0.07N}{A_c}$$

where

N is the ultimate load (in Newtons);

A_c is the area of entire concrete section (in mm²);

A wall subject to biaxial shear due to ultimate loads shall satisfy the expression:

$$\frac{V_x}{V_{ux}} + \frac{V_y}{V_{uy}} \leq 1.0$$

where

V_x and V_y are the applied shears due to ultimate loads for the x-x axis and y-y axis respectively;

V_{ux} and V_{uy} are the corresponding ultimate shear capacities of the concrete and link reinforcement for the x-x axis and y-y axis respectively, derived allowing for the enhancement factor given in this clause.

5.6.6 Deflection of reinforced walls Deflections of walls need not be calculated.

5.6.7 Crack control in reinforced walls If required by the Technical Approval Authority, flexural crack widths in walls subject to bending shall be calculated in accordance with 5.8.8.2.

5.7 Bases

5.7.1 General Where pockets have been left for precast members allowance shall be made, when calculating the flexural and shear strength of base sections, for the effects of these pockets unless they have been grouted up using a cement mortar of compressive strength not less than that of the concrete in the base.

5.7.2 Moments and forces in bases Except where the reactions to the applied loads and moments are derived by more accurate methods, e.g. an elastic analysis of a pile group or the application of established principles of soil mechanics, the following assumptions shall be made.

(a) Where the base is axially loaded, the reactions to ultimate loads are uniformly distributed per unit area or per pile.

(b) Where the base is eccentrically loaded, the reactions vary linearly across the base. For columns and walls restrained in direction at the base, the moment transferred to the base shall be obtained from 5.5.

The critical section in the assessment of an isolated base may be taken as the face of the column or wall.

The moment at any vertical section passing completely across a base shall be taken as that due to all external ultimate loads and reactions on one side of that section. No redistribution of moments shall be made.

5.7.3 Assessment of bases

5.7.3.1 Resistance to bending Bases shall be assessed in accordance with 5.4, and shall be capable of resisting the total moments and shears at the sections considered.
Where the width of the section considered is less than or equal to 1.5 (b_{col} + 3d), where b_{col} is the width of the column and d is the effective depth to the tension reinforcement of the base, all reinforcement crossing the section may be considered to be effective in resisting bending. For greater widths, all reinforcement within a band of width (b_{col} + 3d) centred on the column may be considered to be effective and the area of effective reinforcement outside this band should be taken as the lesser of:
(a) the actual area of reinforcement outside the band, and
(b) 50 % of the area of reinforcement within the band.

Pile caps may be assessed either by bending theory or by truss analogy taking the apex of the truss at the centre of the loaded area and the corners of the base of the truss at the intersections of the centre lines of the piles with the tensile reinforcement.

Pile caps may only be assessed as beams if the reinforcement is uniformly distributed across the section under consideration.

In pile caps assessed by truss analogy, the effective area of reinforcement at a section shall be taken as the lesser of (a) the total area at the section and (b) 1.25 times the area of reinforcement in the strips linking the pile heads.

5.7.3.2 <u>Shear</u> The assessment shear force is the algebraic sum of all ultimate vertical loads acting on one side of or outside the periphery of the critical section. The shear strength of bases in the vicinity of concentrated loads is governed by the more severe of the following two conditions.

(a) Shear along a vertical section extending across the full width of the base, at a distance equal to the effective depth from the face of the loaded area. The requirements of 5.4.4.1 apply.

(b) Punching shear around the loaded area, where the requirements of 5.4.4.2 apply.

The shear strength of pile caps is governed by the more severe of the following two conditions.

(1) Shear along any vertical section extending across the full width of the cap. The requirements of 5.4.4.1 apply, except that the enhancement of the shear resistance for sections close to supports (see 5.3.3.3) shall be applied only to strips of width equal to the pile diameter centred on each pile. Where a_v is taken as the distance between the face of the column or wall and the nearer edge of the piles it shall be increased by 20% of the pile diameter. In applying 5.4.4.1 the allowable ultimate shear stress shall be taken as the average over the whole section.

(2) Punching shear around loaded areas, where the requirements of 5.4.4.2 apply. When considering case (c)(ii) of Figure 5, the allowable ultimate shear stress may be enhanced, in accordance with 5.3.3.3, over a width equal to the pile diameter centred on the corner pile.

5.7.3.3 <u>Bond and anchorage</u> The requirements of 5.8.6 apply to reinforcement in bases.

5.7.4 <u>Deflection of bases</u> The deflection of bases need not be considered.

5.7.5 <u>Crack control in bases</u> If required by the appropriate Technical Approval authority, crack widths may be calculated in accordance with 5.8.8.2 taking into account the type of base and treatment of assessment (see 5.7.3.1).

5.8 <u>Considerations of details</u>

5.8.1 <u>Constructional details</u>

5.8.1.1 Size of members "Not applicable to assessment"

5.8.1.2 Accuracy of position of reinforcement When the reduced material partial safety factor for steel of 1.05, given in 4.3.3.3, is adopted, then covers and effective depths must be measured.

5.8.1.3 Construction joints "*Not applicable to assessment*"

5.8.1.4 Movement joints "*Not applicable to assessment*"

5.8.2 Concrete cover to reinforcement 'Nominal' cover is that dimension used in design and indicated on the drawings. However, the actual cover may be up to 5mm less than the nominal cover.

The nominal cover should be not less than the size of the bar or maximum aggregate size, plus 5mm; in the case of a bundle of bars (see 5.8.8.1), it should be equal to or greater than the size of a single bar of equivalent area plus 5mm.

The nominal cover of dense natural aggregate concrete to all reinforcement, including links, should be not less than the value given in table 13 for particular grades of concrete and conditions of exposure.

Where surface treatment such as bush hammering has cut into the face of the concrete, the depth of treatment shall not be considered as contributing to the cover.

Special care shall be exercised for conditions of extreme exposure or where lightweight or porous aggregates are used (see 5.9.2).

5.8.3 Reinforcement: general considerations

5.8.3.1 Groups of bars Subject to the reductions in bond stress, bars arranged as pairs in contact or in groups of three or four bars bundled in contact shall be considered as effective only if the following conditions are satisfied:

(1) the bundle is restrained by links;

(2) the bars in a bundle terminate at different points spaced at least 40 times the bar size apart except for bundles stopping at a support;

(3) bars in pairs or bundles of three may be lapped one bar at a time, but the laps shall be so staggered that in any cross section there are no more than four bars in a bundle.

5.8.3.2 Bar schedule dimensions "*Not applicable to assessment*"

5.8.4 Minimum area of reinforcement in members

5.8.4.1 Minimum area of main reinforcement *"Not applicable to assessment"*

5.8.4.2 Minimum area of secondary reinforcement *"Not applicable to assessment"*

Environment	Examples	Nominal cover* (mm)			
		Concrete grade			
		25	30	40	50 & over
Extreme		+	+	65†	55
Concrete surfaces exposed to: abrasive action by sea water	Marine Structures				
or water with a pH ≤ 4.5	Parts of structure in contact with moorland water				
Very severe Concrete surfaces directly affected by: de-icing salts	Walls and structure supports adjacent to the carriageway Parapet edge beams	+	§	50†	40
or sea water spray	Concrete adjacent to the sea				
Severe Concrete surfaces exposed to: driving rain	Wall and structure supports remote from the carriageway	+	45†	35	30
or alternative wetting and drying	Bridge deck soffits Buried parts of structure				
Moderate		45	35	30	25
Concrete surfaces above ground level and fully sheltered against all of the following:	Surface protected by bridge deck water-proofing or by permanent formwork				
rain, de-icing salts, sea water spray	Interior surface of pedestrian subways, voided superstructures or cellular abutments				
Concrete surfaces permanently saturated by water with a pH >4.5	Concrete permanently under water				

* Actual cover may be up to 5 mm less than nominal cover
\+ Concrete grade could cause durability problems
† Concrete should be air entrained where the surface is liable to freezing whilst wet
§ For parapet beams only, grade 30 concrete should be entrained and the nominal cover should be 60mm.

Table 13. Nominal cover to reinforcement under particular conditions of exposure

5.8.4.3 Minimum area of links *"Not applicable to assessment"*

5.8.5 Maximum areas of reinforcement in members "*Not applicable to assessment*"

5.8.6 Bond, anchorage and bearing

5.8.6.1 Geometrical classification of deformed bars For the purposes of this Departmental Standard there are two types of deformed bars, as follows:

Type 1 A plain square twisted bar or a plain chamfered square twisted bar, each with a pitch of twist not greater than 18 times the nominal size of the bar.

Type 2 A bar with transverse ribs with a substantial uniform spacing not greater than 0.8 ϕ (and continuous helical ribs where present), having a mean area of ribs (per unit length) above the core of the bar projected on a plane normal to the axis of the bar, of not less than 0.15 mm^2/mm where ϕ is the size (nominal diameter) of the bar.

Other bars may be classified as types 1 or 2 from the results of the performance tests described in Part 5 of the Specification for Highway Works (SHW).

5.8.6.2 Local bond "*Not applicable to assessment*"

5.8.6.3 (08/05) Anchorage bond To prevent bond failure the tension or compression in any bar at any section due to ultimate loads shall be developed on each side of the section by an appropriate embedment length or other end anchorage. The anchorage bond stress, assumed to be constant over the effective anchorage length, taken as the force in the bar divided by the product of the effective anchorage length and the effective perimeter of the bar or group of bars (see 5.8.6.4),

shall not exceed the value $\beta \sqrt{f_{cu}} / \gamma_{mb}$

where:

β is a coefficient dependent on bar type, and given in table 15A;
f_{cu} is the characteristic, or worst credible concrete cube strength;
γ_{mb} is a partial safety factor equal to 1.4, unless the worst credible concrete strength is used, in which case it is equal to 1.25.

5.8.6.5 Anchorage of links A link may be considered to be fully anchored if it passes round another bar through an angle of 90° and continues beyond for a minimum length of eight times its own size, or through 180° and continues for a minimum length of four times its own size. In no case shall the radius of any bend in the link be less than twice the radius of the test bend guaranteed by the manufacturer of the bar. Where full anchorage of links is not achieved, its effective size shall be taken as the equivalent bar diameter that the anchorage provides.

5.8.6.6 Laps and joints Continuity of reinforcement may be achieved by a connection using any of the following jointing methods:

(a) lapping bars;
(b) butt welding (see 4.7);
(c) sleeving (see 7.3.2.2);
(d) threading of bars (see 7.3.2.3).

The strength of joints using the methods given in (c) and (d) and any other method not listed shall be verified by test evidence.

5.8.6.7 Lap lengths When bars are lapped, the length of the lap shall at least equal the anchorage length (derived from 5.8.6.3) required to develop the stress in the smaller of the two bars lapped.

Bar type	β	
	Bars in tension	Bars in compression
Plain bars	0.39	0.49
Type 1:deformed bars	0.56	0.70
Type 2:deformed bars	0.70	0.88
Fabric	0.91	1.13

Table 15A Values of bond coefficient β

The lap length calculated as above shall be increased for bars in tension by a factor of 1.4 if any of the following conditions apply:

(a) the cover to the lapped bars from the top of the section as cast is less than twice the bar size;

(b) the clear distance between the lap and another pair of lapped bars is less than 150 mm;

(c) a corner bar is lapped and the cover to either face is less than twice the bar size.

Where conditions (a) and (b) or conditions (a) and (c) apply the required lap length shall be increased by a factor of 2.0.

The minimum lap length for bar reinforcement under any condition shall not be less than 15 times the size of the smaller of the two bars lapped. Where the minimum lap length is not achieved the effective size of the smaller bar at the section shall be determined as being l/15 where l is the lap length provided.

5.8.6.8 Hooks and bends Hooks, bends and other reinforcement anchorages should be of such form, dimension and arrangement as to avoid overstressing the concrete. Hooks and bends shall be considered fully effective if they are in accordance with BS 4466:1981.

The effective anchorage length of a hook or bend shall be measured from the start of the bend to a point four times the bar size beyond the end of the bend, and may be taken as the lesser of 24 times the bar size or:

(a) for a hook, eight times the internal radius of the hook;

(b) for a 90° bend, four times the internal radius of the bend.

In no case shall the radius of the bend be less than twice the radius of the test bend guaranteed by the manufacturer of the bar. However, it shall be sufficient to ensure that the bearing stress at the mid-point of the curve does not exceed the value given in 5.8.6.9.

For a hooked bar to be effective at a support, the beginning of the hook shall be at least four times the bar size inside the face of the support.

The effective anchorage length of a hook or bend which does not satisfy paragraphs 1,3 and 4 of this clause shall be taken as not greater than the actual length of bar from the start of the bend to a point four times the bar size beyond the end of the bend.

5.8.6.9 Bearing stress inside bends The bearing stress inside a bend, in a bar which does not extend or is not assumed to be stressed beyond a point four times the bar size past the end of the bend, need not be checked.

The bearing stress inside a bend in any other bar shall be calculated from the equation:

Bearing stress = $F_{bt}/(r\phi)$

where

F_{bt} is the tensile force due to ultimate loads in a bar or group of bars;

r is the internal radius of the bend;

ϕ is the size of the bar or, in a bundle, the size of a bar of equivalent area.

The stress shall not exceed

$$\frac{5.63}{\sqrt{\gamma_{mc}}}\sqrt{\left(\frac{l}{l_l}\right)f_{cu}}\left(\frac{a_b}{\phi}\right)^{1/3}$$

where

a_b for a particular bar or group of bars in contact shall be taken as the centre-to-centre distance between bars or groups of bars perpendicular to the plane of the bend; for a bar or group of bars adjacent to the face of the member, a_b shall be taken as the cover plus ϕ. The ratio of a_b/ϕ shall not exceed 8.

Values of the partial safety factor, γ_{mc}, are given in 4.3.3.3.

l_1 is the length of the bar measured inside the bend and bearing on to the concrete.

l is the thickness of concrete member in the plane of the bend, but not greater than $3l_1$.

5.8.7 Curtailment and anchorage of reinforcement Curtailment lengths and anchorages of bars shall be assessed either by rigorous analysis at the curtailment or anchorage point for the worst load case in accordance with 5.8.6.3, or by use of the requirements of paragraphs 2 and 3 of this clause.

Bars shall be considered effective at a distance from their end equal to the effective depth of the member or 12 times the size of the bar, whichever is the greater. In addition, where reinforcement is stopped in a tension zone then one of the following conditions must be satisfied.

(a) the bars extend an anchorage length appropriate to their assessment strength (f_y/γ_{ms}) from the point at which they are no longer required to resist bending; or

(b) the shear capacity at the section where the reinforcement stops is greater than 1.5 times the shear force actually present; or

(c) the continuing bars at the section where the reinforcement stops provide double the area required to resist the moment at that section.

At a simply supported end of a member a tension bar shall only be considered fully effective if anchored by one of the following:

(1) an effective anchorage equivalent to 12 times the bar size beyond the centre line of the support; no bend or hook shall begin before the centre of the support;

(2) an effective anchorage equivalent to 12 times the bar size plus d/2 from the face of the support, where d is the effective depth to tension reinforcement of the member; no bend shall begin before d/2 from the face of the support.

Where these conditions are not met, the effective size of the tension bar at the support may be taken as 1/12 of the effective anchorage present beyond the centre line of the support.

5.8.8 Spacing of reinforcement

5.8.8.1 Minimum distance between bars "*Not applicable to assessment*"

5.8.8.2 Maximum distance between bars in tension When required by the Operating Units, crack widths, under the loads specified by the authority, shall be calculated in accordance with this clause.

(a) For solid rectangular sections, stems of T beams and other solid sections shaped without re-entrant angles, the crack widths at the surface (or, where the cover to the outermost bar is greater than c_{nom}, on a surface at a distance c_{nom} from the outermost bar) shall be calculated from the following equation:

$$\text{Crack width} = \frac{3a_{cr}\epsilon_m}{1 + 2(a_{cr}-c)/(h-d_c)} \qquad \text{equation 24A}$$

where

a_{cr} is the distance from the point (crack) considered to the surface of the nearest bar which controls the crack width;
c_{nom} is the required nominal cover to the outermost reinforcement given in table 13;
c is the effective cover to the reinforcement which controls the width of the cracks under consideration and shall be taken as the lesser of (a) actual cover to this reinforcement and (b) perpendicular distance from this reinforcement to a surface at a distance c_{nom} from the outermost bars;
d_c is the depth of the concrete in compression (if d_c = 0 the crack widths shall be calculated using equation 26);
h is the overall depth of the section;
ϵ_m is the calculated strain at the level where cracking is being considered, allowing for the stiffening effect of the concrete in the tension zone; a negative value of ϵ_m indicates that the section is uncracked. The value of ϵ_m shall be obtained from the equation:

$$\epsilon_m = \epsilon_1 - \left[\frac{3.8b_t\ h(a'-d_c)}{\epsilon_s A_s(h-d_c)}\right] \left[(1-\frac{M_q}{M_g})10^{-9}\right] \qquad \text{equation 25}$$

but not greater than ϵ_1.

where

ϵ_1 is the calculated strain at the level where cracking is being considered, ignoring the stiffening effect of the concrete in the tension zone;
b_t is the width of the section at the level of the centroid of the tension steel;

a´ is the distance from the compression face to the point at which the crack width is being calculated;
M_g is the moment at the section considered due to permanent loads;
M_q is the moment at the section considered due to live loads;
ϵ_s is the calculated strain at the centroid of reinforcement, ignoring the stiffening effect of the concrete in the tension zone;
A_s is the area of tension reinforcement.

Where the axis of the moment and the direction of the tensile reinforcement resisting that moment are not normal to each other (e.g. in a skew slab), A_s shall be taken as:

$$A_s = \sum(A_t \cos^4 \alpha_1)$$

where

A_t is the area of reinforcement in a particular direction;
α_1 is the angle between the normal to the axis of the moment and the direction of the tensile reinforcement, A_t, resisting that moment.

(b) For flanges in overall tension, including tensile zones of box beams, rectangular voided slabs and, when subjected to longitudinal bending, circular voided slabs, the crack width at the surface (or at a distance c_{nom} from the outermost bar) shall be calculated from the following equation:

Crack width = $3a_{cr} \epsilon_m$ equation 26

where ϵ_m is obtained from equation 25.

For flanges of circular voided slabs subjected to transverse bending, the crack width at the surface (or at a distance c_{nom} from the outermost bar) shall be calculated from the following equation:

Crack width = $1.2\epsilon_m(hf/\rho_{net})\sqrt{(c/\phi)}$ equation 26A

where

h_f is the minimum flange thickness;
ρ_{net} is the area of transverse reinforcement in the flange as a percentage of the minimum flange area;
ϕ is the diameter of the outermost transverse bar;

$$\epsilon_m = \epsilon_1 - \left[\frac{3.8\ b_t h_f}{\epsilon_s A_s}\right]\left[(1 - \frac{M_q}{M_g})10^{-9}\right]$$ equation 25A

(c) Where global and local effects are calculated separately (see 4.8.3) the value of ϵ_m may be obtained by algebraic addition of the strains calculated separately. The crack width shall then be calculated in accordance with (b) but may, in the case of deck slab where a global compression is being combined with a local moment, be obtained using (a), calculating d_c on the basis of the local moment only.

5.8.9 Shrinkage and temperature reinforcement "*Not applicable to assessment*"

5.8.10 Arrangement of reinforcement in skew slabs "*Not applicable to assessment*"

5.9 Additional considerations in the use of lightweight aggregate concrete

5.9.1 General Lightweight aggregate concrete may generally be assessed in accordance with the requirements of clause 4 and of 5.1 to 5.8; 5.9.2 to 5.9.11 relate specifically to reinforced lightweight aggregate concrete of strength 25 N/mm² or above. Only the requirements of 7.5 (plain concrete walls) apply to concretes below a strength of 25 N/mm².

For lightweight aggregate concrete, the properties for any particular type of aggregate can be established far more accurately than for most naturally occurring materials and, when the aggregate type can be identified, specific data shall be obtained from the aggregate producer.

All the properties of lightweight aggregate concrete to be used in assessment shall be supported by appropriate test data.

5.9.2 Durability For durability, the cover to reinforcement should be 10mm greater than the values given in table 13.

5.9.3 Strength of concrete See 5.1.4.2.

5.9.4 Shear resistance of beams The shear resistance of lightweight aggregate concrete beams shall be established in accordance with 5.3.3.1 to 5.3.3.3 except that the value of v_c calculated from the expression given in 5.3.3.2 shall be multiplied by 0.9 and the maximum allowable value of v referred to in 5.3.3.1 and 5.3.3.3 shall be multiplied by 0.8.

5.9.5 Torsional resistance of beams The torsional resistance of lightweight aggregate concrete beams shall be established in accordance with 5.3.4 except that the values of v_{tmin} and v_{tu} calculated from the expressions given in 5.3.4.3 shall be multiplied by 0.8.

5.9.6 Deflection of beams Deflection of lightweight aggregate concrete beams may be calculated using a value of E_t as described in 4.3.2.1.

5.9.7 Shear resistance of slabs The shear resistance of lightweight aggregate concrete slabs shall be established in accordance with 5.4.4, except that v_c and the maximum allowable value of v shall be modified in accordance with 5.9.4.

5.9.8 Deflection of slabs Deflection of lightweight aggregate concrete slabs may be calculated using a value of E_t as described in 4.3.2.1.

5.9.9 Columns

5.9.9.1 General The requirements of 5.5 apply to lightweight aggregate concrete columns subject to the conditions in 5.9.9.2 and 5.9.9.3.

5.9.9.2 Short columns In 5.5.1.1, the ratio of effective height, l_e, to thickness, h, for a short column shall not exceed 10.

5.9.9.3 Slender columns In 5.5.5, the divisor 1750 in equations 18, 20, 21 and 22 shall be replaced by the divisor 1200.

5.9.10 Local bond, anchorage bond and laps Anchorage bond stresses and laps lengths in reinforcement for lightweight aggregate concrete members shall be assessed in accordance with 5.8.6 except that the bond stresses shall not exceed 80% of those given in 5.8.6.3.

For foamed slag or similar aggregates it may be necessary to ensure that bond stresses are well below the maximum values in the preceding paragraph for reinforcement that was in a horizontal position during casting, and values shall be obtained from test data.

5.9.11 Bearing stress inside bends The requirements of 5.8.6.9 apply to lightweight aggregate concrete, except that the bearing stress shall not exceed two-thirds of the allowable value given by the expression in 5.8.6.9.

6. ASSESSMENT: PRESTRESSED CONCRETE

6.1 General

6.1.1 Introduction This clause gives methods of assessment which will in general assure that, for prestressed concrete construction, the requirements set out in Clause 4 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.

This clause does not cover prestressed concrete construction using any of the following in the permanent works:

(a) unbonded tendons:
(b) external tendons (a tendon is considered external if, after stressing and incorporating in the permanent works but before protection, it is outside the concrete section):
(c) lightweight aggregate.

6.1.2 Limit state assessment of prestressed concrete

6.1.2.1 Basis of assessment Clause 6 follows the limit state philosophy set out in Clauses 3 and 4. Usually a structure will need to be assessed only at the ultimate limit state. A serviceability limit state assessment is required only when specifically requested by the Overseeing Department.

6.1.2.2 Durability In 6.8.2 guidance is given on the nominal cover to reinforcement and prestressing tendons that is needed to ensure durability.

6.1.2.3 Other limit states and considerations Clause 6 does not specify requirements for vibration or other limit states and for these and other considerations reference should be made to 4.1.3.

6.1.3 Loads In Clause 6, the assessment 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 assessment are derived from BD 21 and Clause 4.2.

In Clause 6, when analysing sections, the terms `strength', `resistance' and `capacity' are used to describe the assessment resistance of the section (see BD 21).

Consideration shall be given to the construction sequence and to the secondary effects due to prestress particularly if a serviceability limit state assessment is being performed.

6.1.4 Strength of materials

6.1.4.1 Definition of strengths In Clause 6 the symbol f_{cu} represents either the characteristic or the worst credible cube strength of the concrete; and the symbol f_{pu} represents either the characteristic or the worst credible tendon strength.

The assessment strengths of concrete and prestressing tendons are given by f_{cu}/γ_{mc} and f_{pu}/γ_{ms}, respectively, where γ_{mc} and γ_{ms} are the appropriate material partial safety factors given in 4.3.3.3. The appropriate value of γ_{mc} or γ_{ms} has to be substituted in all equations in Clause 6.

6.1.4.2 Strength of concrete Assessment may be based on either the specified characteristic cube strength, or the worst credible cube strength assessed as the lower bound to the estimated in-situ cube strength determined in accordance with BS 6089.

6.1.4.3 Strength of prestressing tendons Assessment may be based on either the specified characteristic strength, or the worst credible strength assessed from tests on tendon samples extracted from the structure.

6.2 Structures and structural frames

6.2.1 Analysis of structures Complete structures and complete structural frames may be analysed in accordance with the requirements of 4.4 but when appropriate the methods given in 6.3 may be used for the assessment of individual members.

The relative stiffness of members should generally be based on the concrete section as described in 4.4.2.1.

6.2.2 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) Appropriate checks are made to ensure that adequate rotational 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) $$0.008 + 0.035\left(0.5 - \frac{d_c}{d_e}\right) \quad \text{or}$$

(2) $$\frac{10}{d - d_c}$$

but not less than 0

where

d_c is the calculated depth of concrete in compression at the ultimate limit state (in mm);
d_e is the effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange (in mm);
d is the effective depth to tension reinforcement (in mm).

(b) Proper account is taken of changes in transverse moments and transverse shears consequent on redistribution of longitudinal moments.

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

6.3 Beams

6.3.1 General

6.3.1.1 Definitions The definitions and limitations of the geometric properties for prestressed beams are as given for reinforced concrete beams in 5.3.1.

6.3.1.2 Slender beams "*Not applicable to assessment*"

6.3.2 Serviceability limit state: flexure When a serviceability limit state assessment is required, the necessary criteria shall be agreed with the Operating Units.

6.3.3 Ultimate limit state: flexure

6.3.3.1 Section analysis When analysing a cross section to determine its ultimate strength the following assumptions shall be made.

(a) The strain distribution in the concrete in compression is derived from the assumption that plane sections remain plane.

(b) The stresses in the concrete in compression are derived either from the stress-strain curve given in Figure 1 with the appropriate value of γ_{mc} given in 4.3.3.3 or, in the case of rectangular sections or flanged sections with the neutral axis in the flange, the compressive stress may be taken as equal to 0.6 f_{cu}/γ_{mc} over the whole compression zone; in both cases the strain at the outermost compression fibre is taken as 0.0035.

(c) The tensile strength of concrete is ignored.

(d) The strains in bonded prestressing tendons and in any additional reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane. In addition, the tendon will have an initial strain due to prestress after all losses.

(e) The stresses in bonded prestressing tendons, whether initially tensioned or untensioned, and in additional reinforcement, are derived either from the appropriate stress-strain curve in Figures 2, 3 and 4 or, when available, manufacturers' stress-strain curves. The values of γ_{ms} are given in 4.3.3.3. An empirical approach for obtaining the stress in the tendons at failure is given in 6.3.3.3.

6.3.3.2 Design charts The design charts in CP 110: Part 3 include charts, based on figures 1, 3 and 4, and the assumptions given in 6.3.3.1, which, with appropriate modifications for γ_{ms}, may be used for the assessment of rectangular prestressed beams.

6.3.3.3 Assessment formula In the absence of an analysis based on the assumptions given in 6.3.3.1, the resistance moment of a rectangular beam, or of a flanged beam in which the neutral axis lies within the flange, may be obtained from equation 27

$$M_u = f_{pb} A_{ps} (d - 0.5x) \qquad \text{equation 27}$$

where

M_u is the ultimate moment of resistance of the section;
f_{pb} is the tensile stress in the tendons at failure;
x is the neutral axis depth;
d is the effective depth to tension reinforcement;
A_{ps} is the area of the prestressing tendons in the tension zone.

The tensile stress, f_{pb}, may be calculated from:

$$f_{pb} / [\frac{f_{pu}}{\gamma_{ms}}] = (\alpha - \frac{f_{pu} A_{ps}}{f_{cu} b d}) \text{ but } \not> 1.0$$

where α is 1.3 for pre-tensioning, and
1.15 for post-tensioning with effective bond; and γ_{ms} is the partial safety factor for the tendons given in 4.3.3.3.

The neutral axis depth, x, may be calculated from

$$x = \frac{f_{pb} A_{ps} \gamma_{mc}}{0.6 f_{cu} b}$$

where γ_{mc} is the partial safety factor for concrete given in 4.3.3.3.

Prestressing tendons and additional reinforcement in the compression zone are ignored in strength calculations when using this method.

6.3.3.4 Non-rectangular sections Non-rectangular beams shall be analysed using the assumptions given in 6.3.3.1.

6.3.4 Shear resistance of beams

6.3.4.1 General Calculations for shear are only required for the ultimate limit state.

At any section the ultimate shear resistance is the sum of the resistances of the concrete alone, V_c, (see 6.3.4.2 and 6.3.4.3) of the shear reinforcement, V_s, (see 6.3.4.4).

For vertical links to be effective, the tensile capacity of the longitudinal steel at a section must be greater than

$$\frac{M}{z} + \frac{(V - \xi_s v_c b_w d)}{2}$$

where M and V are the co-existent ultimate bending moment and shear force at the section under consideration, z is the lever arm which may be taken as 0.9d, and ξ_s, v_c, b_w and d are as defined in Clause 5.3.3.2. The tensile capacity of the longitudinal steel is

$$\left[A_{s(t)} f_{pu(t)} + A_{s(u)} f_{yL(u)}\right] / \gamma_{ms}$$

where $A_{s(t)}$, $f_{pu(t)}$, $A_{s(u)}$, and $f_{yL(u)}$ are as defined in Clause 6.3.4.3. However within an individual sagging or hogging region, such tension need not exceed M_{max}/z where M_{max} is the maximum ultimate moment within that region.

At a section at which the applied moment, M, does not exceed the cracking moment, M_{cr}, calculated in accordance with 6.3.4.2, V_c may be taken as equal to the uncracked value, V_{co}, (see 6.3.4.2). In all other cases V_c shall be taken as the lesser of the uncracked value, V_{co}, (see 6.3.4.2) and the cracked value, V_{cr}, (see 6.3.4.3).

For a cracked section the conditions of maximum shear with co-existent bending moment and maximum bending moment with co-existent shear shall both be considered.

Within the transmission length of pre-tensioned members (see 6.7.4), the shear resistance of a section shall be taken as the greater of the values calculated from:

(a) 5.3.3 except that in determining the area A_s, the area of tendons shall be ignored unless the tendons are rigid bars; and

(b) 6.3.4.2 to 6.3.4.4, using the appropriate value of prestress at the section considered, assuming a linear variation of prestress over the transmission length.

6.3.4.2 Sections uncracked in flexure A section may be assumed to be uncracked in flexure if the applied moment does not exceed the cracking moment, M_{cr}:

$$M_{cr} = (0.49\sqrt{f_{cu}/\gamma_{mc}} + f_{pt})I/y$$

where f_{pt} is the stress due to prestress only at the tensile fibre distance y from the centroid of the concrete section which has a second moment of area I; the value of f_{pt} shall be derived from the prestressing force after all losses have occurred, multiplied by the appropriate value of γ_{fL} (see 4.2.3). Values of the partial safety factor γ_{mc} are given in 4.3.3.3.

It may be assumed that the ultimate shear resistance of a section uncracked in flexure, V_{co}, corresponds to the occurrence of a maximum principal tensile stress, at the centroidal axis of the section, of $f_t = 0.32\sqrt{f_{cu}/\gamma_{mc}}$

In the calculation of V_{co}, the value of f_{cp} shall be derived from the prestressing force after all losses have occurred, multiplied by the appropriate value of γ_{fL} (see 4.2.3). The value of V_{co} is given by:

$$V_{co} = 0.67\ bh\ \sqrt{f_t^2 + f_{cp}f_t} \qquad \text{equation 28}$$

where

f_t is $0.32\sqrt{f_{cu}/\gamma_{mc}}$, taken as positive;

f_{cp} is the compressive stress at the centroidal axis due to prestress, taken as positive;

b is the breadth of the member which for T, I and L beams shall be replaced by the breadth of the rib, b_w. (Where the position of a duct coincides with the position of maximum principle tensile stress, e.g. at or near the junction of flange and web near a support, the value of b shall be reduced by the full diameter of the duct if ungrouted and by two-thirds of the diameter if grouted.);

h is the overall depth of the member.

In flanged members where the centroidal axis occurs in the flange, the principal tensile stress shall be limited to f_t at the intersection of the flange and web. For such members, the algebraic sum of the stress due to the bending moment under ultimate loads and the stress due to prestress at this intersection shall be used in calculating V_{co}.

For a section with inclined tendons, the component of prestressing force (multiplied by the appropriate value of γ_{fL}) normal to the longitudinal axis of the member shall be algebraically added to V_{co}. This component shall be taken as positive where the shear resistance of the section is increased.

6.3.4.3 Sections cracked in flexure The ultimate shear resistance of a section cracked in flexure, V_{cr}, shall be calculated using equation 29A when the factored effective prestress, f_{pe}, exceeds 0.6 f_{pu}, and equation 30A when f_{pe} does not exceed 0.6 f_{pu}.

$$V_{cr} = 0.045bd\sqrt{f_{cu}/\gamma_{mc}} + M_{cr}/(M/V - d/2)$$
$$but \not< 0.12\ bd\sqrt{f_{cu}/\gamma_{mc}}$$ equation 29A

$$V_{cr} = (1-0.55\ f_{pe}/f_{pu})v_c bd_s + M_o/(M/V-d_s/2)$$
$$but \not< 0.12\ bd_s\sqrt{f_{cu}/\gamma_{mc}}$$ equation 30A

where

d	is the distance from the extreme compression fibre to the centroid of the tendons at the section considered; but not less than 0.625h
γ_{mc}	is the partial safety factor for concrete given in 4.3.3.3;
M_{cr}	is the cracking moment defined in 6.3.4.2;
V and M	are the shear force and bending moment (both taken as positive) at the section considered due to ultimate loads;
M_o	is the moment necessary to produce zero stress in the concrete at the depth d: $M_o = f_{pt}$ I/y in which f_{pt} is the stress due to prestress only at the depth d, distance y from the centroid of the concrete section which has a second moment of area I; the value of f_{pt} shall be derived from the prestressing forces after all losses have occurred, multiplied by the appropriate value of γ_{fL} (see 4.2.3);
f_{pe}	is the factored effective prestress which is equal to the effective prestress after all losses have occurred, multiplied by the appropriate value of γ_{fL} (see 4.2.3);
v_c	is obtained from 5.3.3.2;
A_s	(required to obtain v_c) shall be taken as the actual area of steel in the tension zone, irrespective of its characteristic strength;
d_s	is the distance from the compression face to the centroid of the steel area, A_s .

For cases where both tensioned and untensioned steel are contained in A_s, f_{pe}/f_{pu} may be taken as:

$$\frac{P_f}{A_{s(t)}f_{pu(t)} + A_{s(u)}f_{yL(u)}}$$

where

P_f	is the effective prestressing force after all losses;
$A_{s(t)}$	is the area of tensioned steel;
$A_{s(u)}$	is the area of untensioned steel;
$f_{pu(t)}$	is the characteristic strength or the worst credible strength of the tensioned steel;
$f_{yL(u)}$	is the characteristic strength or the worst credible strength of the untensioned steel.

For sections cracked in flexure and with inclined tendons, the component of prestressing force normal to the longitudinal axis of the member shall be ignored.

6.3.4.4 Shear reinforcement Types of shear reinforcement and the criterion for the amount of shear reinforcement required to be present for it to be considered effective are defined in 5.3.3.2. In addition links shall be considered as effective only if their spacing both along a beam and laterally does not exceed d_t, nor four times the web thickness for flanged beams.

When the above criteria are met, the shear resistance of vertical links is:

$V_s = A_{sv} (f_{yv}/\gamma_{ms}) d_t/s_v$

where d_t is the depth from the extreme compression fibre either to the centroid of the tendons or to the longitudinal bars, tendons, or groups of tendons in the tension zone around which the links are anchored in accordance with 5.8.6.5, whichever is greater.

All other terms in the equation for V_s are defined in 5.3.3.2.

Sections within a distance d from the support need not be assessed for shear providing the shear reinforcement calculated for the section at a distance d is continued up to the support.

Inclined links or bent up bars shall be assumed to form the tension members of lattice girders as described in 5.3.3.2.

6.3.4.5 Maximum shear force In no circumstance shall the shear force, V, due to ultimate loads, exceed the stress $0.92\sqrt{f_{cu}/\gamma_{mc}}$ or $7/\sqrt{\gamma_{mc}}\ N/mm^2$, whichever is the lesser, multiplied by bd_s, where b is as defined in 6.3.4.2 and d_s is as defined in 6.3.4.3. γ_{mc} is the partial safety factor for concrete given in 4.3.3.3.

6.3.4.6 Segmental construction In post-tensioned segmental construction, the shear force due to ultimate loads shall not be greater than:

$$0.7\gamma_{fL} P_h \tan \alpha_2$$

where

γ_{fL} is the partial safety factor for the prestressing force, to be taken as 0.87;
P_h is the horizontal component of the prestressing force after all losses;
α_2 is the angle of friction at the joint. Tan α_2 can be taken as 0.7 for a smooth interface and 1.4 for a roughened or castellated interface. If there is any doubt regarding the type of interface, tan α_2 shall be taken as 0.7.

The method of assessment of match cast joints with shear keys shall be agreed with the Overseeing Department.

6.3.5 Torsional resistance of beams

6.3.5.1 General In some members, the maximum torsional moment does not occur under the same loading as the maximum flexural moment. In such circumstances reinforcement and prestress in excess of that required for flexure and shear may be used in torsion.

6.3.5.2 Stresses and reinforcement Calculations of torsion are only required for the ultimate limit state and the torsional shear stresses shall be calculated assuming a plastic shear stress distribution.

Calculations for torsion shall be in accordance with 5.3.4 with the following modifications. When prestressing steel is used as transverse torsional steel, or as longitudinal torsional steel, the stress assumed in assessment shall be the lesser of $(f_{pe} + 460/\gamma_{ms})$ or f_{pu}/γ_{ms}, where γ_{ms} is the partial safety factor for steel given in 4.3.3.3.

The compressive stress in the concrete due to prestress shall be taken into account separately in accordance with 5.3.4.5.

In calculating $(v+v_t)$ for comparison with v_{tu} (see 5.3.4.3), v shall be calculated from equation 8 regardless of whether 6.3.4.2 or 6.3.4.3 is critical in shear.

6.3.5.3 Segmental construction "Not applicable to assessment."

6.3.5.4 Other assessment methods Alternative methods of assessing members subjected to combined bending, shear and torsion may be used provided that it can be shown that they satisfy the ultimate limit state requirements.

6.3.6 Longitudinal shear For flanged beams the longitudinal shear resistance across vertical sections of the flange which may be critical shall be checked in accordance with 7.4.2.3.

6.3.7 Deflection of beam If required by the Overseeing Department, deflections may be calculated by a method appropriate to the level of prestress in the member and the level of loading.

6.4 Slabs The analysis of prestressed slabs shall be in accordance with 5.4.1 provided that due allowance is made for moments due to prestress. The assessment shall be in accordance with 6.3.

The assessment of shear shall be in accordance with 6.3.4.

In the treatment of shear stresses under concentrated loads, the ultimate shear resistance of a section uncracked in flexure, V_{co}, may be taken as corresponding to the occurrence of a maximum principal tensile stress of $f_t = 0.32\sqrt{f_{cu}/\gamma_{mc}}$ at the centroidal axis around the critical section which is assumed as a perimeter h/2 from the loaded area. The shear resistance of any shear reinforcement, V_s, shall be assessed in accordance with 6.3.4.4.

6.5 Columns Prestressed concrete columns, where the mean stress in the concrete section imposed by the tendons is less than 2.5 N/mm^2, may be assessed as reinforced columns in accordance with 5.5, otherwise the full effects of the prestress shall be considered.

6.6 Tension members The tensile strength of tension members shall be based on the assessment strength (f_{pu}/γ_{ms}) of the prestressing tendons and the strength developed by any additional reinforcement. The additional reinforcement may usually be assumed to be acting at its assessment stress (f_y/γ_{ms}); in special cases it may be necessary to check the stress in the reinforcement using strain compatibility.

6.7 Prestressing requirements

6.7.1 Maximum initial prestress The initial prestress shall be assessed from contract record drawings, available site data or original design calculations. In the absence of such information, the likely nominal value of the initial prestress shall be assessed from the standards current at the time of the design.

6.7.2 Loss of prestress, other than friction losses

6.7.2.1 General Allowance shall be made when calculating the forces in tendons for the appropriate losses of prestress resulting from:

(a) relaxation of the steel comprising the tendons;
(b) the elastic deformation and subsequent shrinkage and creep of the concrete;
(c) slip or movement of tendons at anchorages during anchoring;
(d) other causes in special circumstances, eg. when steam curing has been used with pre-tensioning.

If experimental evidence on performance is not available, account shall be taken of the properties of the steel and of the concrete when calculating the losses of prestress from these causes. For a wide range of structures, the simple requirements given in this clause shall be used; it should be recognised, however, that these requirements are necessarily general and approximate.

6.7.2.2 Loss of prestress due to relaxation of steel The loss of force in the tendon allowed for in the assessment shall be the maximum relaxation after 1000h duration, for a jacking force equal to that which is estimated was imposed at transfer, as given by the appropriate British Standard or manufacturer's data.

In special cases, such as tendons at high temperature or subjected to large lateral loads (eg. deflected tendons), greater relaxation losses will occur and specialist literature shall be consulted.

6.7.2.3 Loss of prestress due to elastic deformation of the concrete

Calculation of the immediate loss of force in the tendons due to elastic deformation of the concrete at transfer may be based on the values for the modulus of elasticity of the concrete given in 4.3.2.1. The modulus of elasticity of the tendons may be obtained from 4.3.2.2.

For pre-tensioning, the loss of prestress in the tendons at transfer shall be calculated on a modular ratio basis using the stress in the adjacent concrete.

For members with post-tensioning tendons that were not stressed simultaneously, there would have been a progressive loss of prestress during transfer due to the gradual application of the prestressing force. The resulting loss of prestress in the tendons shall be calculated on the basis of half the product of the modular ratio and the stress in the concrete adjacent to the tendons, averaged along their length; alternatively, the loss of prestress may be computed exactly based on the sequence of tensioning when it is known.

In making these calculations, it may usually be assumed that the tendons are located at their centroid.

6.7.2.4 Loss of prestress due to shrinkage of the concrete The loss of prestress in the tendons due to shrinkage of the concrete may be calculated from the modulus of elasticity for the tendons given in 4.3.2.2, assuming the values for shrinkage per unit length given in Table 29.

System	Shrinkage per unit length	
	Humid exposure (90% r.h.)	Normal exposure (70% r.h.)
Pre-tensioning: transfer at between 3 days and 5 days after concreting	100×10^{-6}	300×10^{-6}
Post-tensioning: transfer at between 7 days and 14 days after concreting	70×10^{-6}	200×10^{-6}

Table 29 Shrinkage of concrete

For other ages of concrete at transfer, for other conditions of exposure, or for massive structures, some adjustment to these figures will be necessary, in which case reference shall be made to Appendix C of BS 5400: Part 4 or specialist literature.

6.7.2.5 Loss of prestress due to creep of the concrete The loss of prestress in the tendons due to creep of the concrete shall be calculated on the assumption that creep is proportional to stress in the concrete for stress of up to one-third of the cube strength at transfer. The loss of prestress is obtained from the product of the modulus of elasticity of the tendons (see 4.3.2.2) and the creep of the concrete adjacent to the tendons. Usually it is sufficient to assume, in calculating this loss, that the tendons are located at their centroid.

For pre-tensioning at between 3 days and 5 days after concreting and for humid or dry conditions of exposure where the cube strength at transfer was greater than 40 N/mm^2, the creep of the concrete per unit length shall be taken as 48 x 10^{-6} per N/mm. For lower values of cube strength at transfer the creep per unit length shall be taken as 48 x 10^{-6} x40/f_{ci} per N/mm^2, where f_{ci} is the concrete strength at transfer, see 6.7.4.

For post-tensioning at between 7 days and 14 days after concreting and for humid or dry conditions of exposure where the cube strength at transfer was greater than 40N/mm^2, the creep of the concrete per unit length shall be taken as 36 x 10^{-6} per N/mm^2. For lower values of cube strength at transfer the creep per unit length shall be taken as 36 x 10^{-6} x 40/f_{ci} per N/mm^2; where f_{ci} is the concrete strength at transfer, see 6.7.4.

Where the maximum stress anywhere in the section at transfer exceeded one-third of the cube strength of the concrete at transfer the value of the creep per unit length used in calculations shall be increased. When the maximum stress at transfer was half the cube strength at transfer, the values for creep are 1.25 times those given in the preceding paragraphs; at intermediate stresses, the values shall be interpolated linearly.

In applying these requirements, which are necessarily general, reference shall be made to Appendix C of BS 5400: Part 4 or specialist literature for more detailed information on the factors affecting creep.

6.7.2.6 Loss of prestress during anchorage In post-tensioning systems allowance shall be made for any movement of the tendon at the anchorage which would have occurred when the prestressing force was transferred from the tensioning equipment to the anchorage.

6.7.2.7 Losses of prestress due to steam curing Where steam curing was employed in the manufacture of prestressed concrete units, changes in the behaviour of the material at higher than normal temperatures will need to be considered. In addition, where the `long-line' method of pre-tensioning was used there may be additional losses as a result of bond developed between the tendon and the concrete when the tendon was hot and relaxed. Since the actual losses of prestress due to steam curing are a function of the techniques used by the various manufacturers, specialist advice shall be sought.

6.7.3 Loss of prestress due to friction

6.7.3.1 General In post-tensioning systems there will have been movement of the greater part of the tendon relative to the surrounding duct during the tensioning operation, and if the tendon were in contact with either the duct or any spacers provided, friction would have caused a reduction in the prestressing force as the distance from the jack increased. In addition, a certain amount of friction would have developed in the jack itself and in the anchorage through which the tendon passed.

In the absence of site data, the stress variation likely to be expected along the tendon profile shall be assessed in accordance with 6.7.3.2 to 6.7.3.5 in order to obtain the prestressing force at the critical sections considered in assessment.

6.7.3.2 Friction in the jack and anchorage "Not applicable to assessment."

6.7.3.3 Friction in the duct due to unintentional variation from the specified profile

Whether the desired duct profile was straight or curved or a combination of both, there will have been slight variations in the actual line of the duct, which may have caused additional points of contact between the tendon and the sides of the duct, and so produced friction. The prestressing force, P_x, at any distance x from the jack may be calculated from:

$$P_x = P_o e^{-Kx} \qquad \text{equation 31}$$

and where $Kx \leq 0.2$, e^{-Kx} may be taken as 1-Kx

where

P_o is the prestressing force in the tendon at the jacking end;

e is the base of Napierian logarithms (2.718);

K is the constant depending on the type of duct, or sheath employed, the nature of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete.

The value of K per metre length in equation 31 shall generally be taken as not less than 33×10^{-4}, but where strong rigid sheaths or duct formers were used closely supported so that they would not have been displaced during the concreting operation, the value of K may be taken as 17×10^{-4}. Other values may be used provided they have been established by tests to the satisfaction of the Engineer.

6.7.3.4 Friction in the duct due to curvature of the tendon

When a tendon is curved, the loss of tension due to friction is dependent on the angle turned through and the coefficient of friction, between the tendon and its supports.

The prestressing force, P_x, at any distance x along the curve from the tangent point may be calculated from:

$$P_x = p_0 \; e^{-\mu x / r_{ps}}$$

where

P_o is the prestressing force in the tendon at the tangent point near the jacking end;

r_{ps} is the radius of curvature

Where $\mu x/r_{ps} \leq 0.2$, $e^{-\mu x/r_{ps}}$ may be taken as $1 - \mu x/r_{ps}$.

Where $(Kx + \mu x/r_{ps}) \leq 0.2$, $e^{-(Kx + \mu x/r_{ps})}$ may be taken as $1-(Kx + \mu x/r_{ps})$.

Values of μ may be taken as:
0.55 for steel moving on concrete;
0.30 for steel moving on steel;
0.25 for steel moving on lead.

The value of μ may be reduced where special precautions have been taken and where results are available to justify the value assumed. For example, a value of $\mu = 0.10$ has been observed for strand moving on rigid steel spacers coated with molybdenum disulphide. Such reduced values shall be agreed with the Overseeing Department.

6.7.3.5 Friction in circular construction Where circumferential tendons have ben tensioned by means of jacks the losses due to friction may be calculated from the formula in 6.7.3.4, but the value of μ may be taken as:

0.45 for steel moving in smooth concrete;
0.25 for steel moving on steel bearers fixed to the concrete;
0.10 for steel moving on steel rollers.

6.7.3.6 Lubricants Where lubricants were specified and lower values of μ than those given in 6.7.3.4 and 6.7.3.5 were obtained by trials and agreed with Overseeing Department, the lower values may be used for assessment.

6.7.4 Transmission length in pre-tensioned members The transmission length is defined as the length over which a tendon is bonded to concrete to transmit the initial prestressing force in a tendon to the concrete.

The transmission length depends on a number of variables, the most important being:

(a) the degree of compaction of the concrete;
(b) the strength of the concrete;
(c) the size and type of tendon;
(d) the deformation (eg. crimp) of the tendon;
(e) the stress in the tendon; and
(f) the surface condition of the tendon.

The transmission lengths of the tendons towards the top of a unit may be greater than those at the bottom.

The sudden release of tendons may also cause a considerable increase in the transmission lengths.

Where the initial prestressing force was not greater than 75% of the characteristic strength of the tendon and where the concrete strength at transfer was not less than 30 N/mm², the transmission length, l_t, may be taken as follows:

$$l_t = k_t \, \phi / \sqrt{f_{ci}}$$

where

f_{ci} is the concrete strength at transfer (in N/mm^2) which shall be assessed from contract record drawings, available site data or original design calculations. In the absence of such information, the likely nominal value shall be assessed from the standards current at the time of the design:

l_t is the transmission length (in mm);
ϕ is the nominal diameter of the tendon (in mm);
k_t is a coefficient dependent on the type of tendon, to be taken as:
600 for plain, indented and crimped wire with a total wave height less than 0.15ϕ;
400 for crimped wire with a total wave height greater than or equal to 0.15ϕ;
240 for 7-wire standard and super strand;
360 for 7-wire drawn or compacted strand.

The development of stress from the end of the unit to the point of maximum stress shall be assumed to be linear over the transmission length.

If the tendons have been prevented from bonding to the concrete near the ends of the unit by the use of sleeves or tape, the transmission lengths shall be taken from the ends of the de-bonded portions.

6.7.5 End blocks The end block (also known as the anchor block or end zone) is defined as the highly stressed zone of concrete around the termination points of a pre or post-tensioned prestressing tendon. It extends from the point of application of prestress (ie the end of the bonded part of the tendon in pre-tensioned construction or the anchorage in post-tensioned construction to that section of the member at which linear distribution of stress is assumed to occur over the whole cross section.

The following aspects shall be considered in assessing the strength of end blocks:

(a) bursting forces around individual anchorages;
(b) overall equilibrium of the end block;
(c) spalling of the concrete from the loaded face around anchorages.

In considering each of these aspects, particular attention shall be given to factors such as the following:

(1) shape, dimensions and position of anchor plates relative to the cross section of the end block;
(2) the magnitude of the prestressing forces and the sequence of prestressing;
(3) shape of the end block relative to the general shape of the member;
(4) layout of anchorages including asymmetry, group effects and edge distances;
(5) influence of the support reaction;
(6) forces due to curved or divergent tendons.

The following requirements are appropriate to a circular, square or rectangular anchor plate, symmetrically positioned on the end face of a square or rectangular post-tensioned member; these requirements are followed by some guidance on other aspects.

The bursting tensile forces in the end blocks, or end regions of post-tensioned members shall be assessed on the basis of the load in the tendon at the ultimate limit state.

The bursting tensile force, F_{bst}, existing in an individual square end block loaded by a symmetrically placed square anchorage or bearing plate, may be derived form:

$$F_{bst} = P_k\,(0.32 - 0.3y_{po}/y_o)$$

where

P_k is the load in the tendon;
y_o is half the side of end block;
y_{po} is half the side of loaded area.

The force, F_{bst}, is distributed in a region extending from $0.2y_o$ to $2y_o$ from the loaded face of the end block. Reinforcement present may be assumed to sustain the bursting tensile force with a stress of f_y/γ_{ms}.

In rectangular end blocks, the bursting tensile forces in the two principal directions shall be assessed from the above expression for F_{bst}.

When circular anchorage or bearing plates are present, the side of the equivalent square area shall be derived.

Where groups of anchorages or bearing plates occur, the end blocks shall be divided into a series of symmetrically loaded prisms and each prism treated in the preceding manner. When assessing the end block as a whole, it is necessary to check that the groups of anchorages are appropriately tied together by reinforcement.

Special attention shall be paid to end blocks having a cross-section different in shape from that of the general cross-section of the beam; reference shall be made to the specialist literature.

Compliance with the preceding requirements will generally ensure that bursting tensile forces along the load axis can be sustained. Alternative methods of assessment which use higher values of F_{bst}/P_k and allow for the tensile strength of concrete may be more appropriate in some cases, particularly where large concentrated tendon forces are involved.

Consideration shall also be given to the spalling tensile stresses that occur in end blocks where the anchorage or bearing plates are highly eccentric; these reach a maximum at the loaded face.

6.8 Considerations of details

6.8.1 General The considerations in 6.8.2 to 6.8.6 are intended to supplement those for reinforced concrete given in 5.8.

6.8.2 Cover to prestressing tendons

6.8.2.1 General The covers given in 6.8.2.2 and 6.8.2.3, other than those for curved ducts, are those which are currently considered to be necessary for durability.

6.8.2.2 Pre-tensioned tendons The requirements of 5.8.2 concerning cover to reinforcement may be taken to be applicable. The ends of individual pre-tensioned tendons do not normally require concrete cover.

6.8.2.3 Tendons in ducts The cover to any duct should be not less than 50mm.

Requirements for the cover to curved tendons in ducts are given in Appendix D of BS 5400:Part 4 as implemented by BD 24.

6.8.3 Spacing of prestressing tendons

6.8.3.1 General *"Not applicable to assessment."*

6.8.3.2 Pre-tensioned tendons *"Not applicable to assessment."*

6.8.3.3 Tendons in ducts Requirements for the spacing of curved tendons in ducts are given in Appendix D of BS 5400:Part 4 as implemented by BD 24.

6.8.4 Longitudinal reinforcement in prestressed concrete beams

Reinforcement in prestressed concrete members may be considered to enhance the strength of the sections.

6.8.5 Links in prestressed concrete beams Links present in a beam may be considered as shear reinforcement (see 6.3.4) or to resist bursting tensile stresses in the end zones of post-tensioned members (see 6.3.4 and 6.7.4).

6.8.6 Shock loading *"Not applicable to assessment."*

7. ASSESSMENT: PRECAST, COMPOSITE AND PLAIN CONCRETE CONSTRUCTION

7.1 General

7.1.1 Introduction This clause is concerned with the additional considerations that arise in assessment when precast members or precast components are incorporated into a structure or when a structure in its entirety is of precast concrete construction. It also covers the assessment of plain concrete walls and abutments.

7.1.2 Limit state assessment

7.1.2.1 Basis of assessment The limit state philosophy set out in clause 4 applies equally to precast and in situ construction and therefore, in general, the relevant methods of assessment for reinforced concrete given in clause 5 and those for prestressed concrete given in clause 6 apply also to precast and composite construction. Subclauses in clause 5 or 6 which do not apply are either specifically worded for in situ construction or are modified by this clause.

7.1.2.2 Handling stresses *"Not applicable to assessment."*

7.1.2.3 Connections and joints The strength of connections is of fundamental importance in precast construction and shall be carefully considered in assessment.

In the assessment of beam and slab ends on corbels and nibs, particular attention shall be given to the detailing of overlaps and anchorages and all reinforcement adjacent to the contact faces. The detailing shall be assessed in accordance with 5.8.7.

7.2 Precast concrete construction

7.2.1 Framed structures and continuous beams When the continuity of reinforcement or tendons through the connections and/or the interaction between members is such that the structure behaves as a frame, or other rigidly interconnected system, the analysis, re-distribution of moments and the assessment of individual members, may all be in accordance with clause 5 or 6, as appropriate.

7.2.2 Other precast members All other precast concrete members shall be assessed in accordance with the appropriate requirements of clauses 5, 6 or 7.5 and connections shall be assessed in accordance with 7.3.

7.2.3 Supports for precast members

7.2.3.1 Concrete corbels A corbel is a short cantilever beam in which the principal load is applied such that the distance a_v between the line of action of the load and the face of the supporting member does not exceed the effective depth and the depth at the outer edge of the bearing is not less than one-half of the depth at the face of the supporting member.

The shear capacity at the face of the supporting member shall be assessed in accordance with 5.3.3.3, but using the modified definition of a_v given in the preceding paragraph.

The adequacy of the main tension reinforcement in a corbel shall be assessed on the assumption that the corbel behaves as a simple strut and tie system; with due allowance made for horizontal forces. The tensile force which the main reinforcement can develop may be limited by any one of the following: the yield of the reinforcement; the anchorage of the reinforcement in the supporting member and the anchorage at the front face of the corbel.

Any part of the area of the bearing which projects beyond the straight portion of the bars forming the main tension reinforcement shall be ignored when proportioning the strut and tie system, and when checking bearing stresses in accordance with 7.2.3.3.

7.2.3.2 Width of supports for precast units The width of supports for precast units shall be sufficient to ensure proper anchorage of tension reinforcement in accordance with 5.8.7.

7.2.3.3 Bearing stresses The compressive stress in the contact area shall not exceed 0.6 f_{cu}/γ_{mc} under the ultimate loads. When the members are made of concretes of different strengths, the lower concrete strength is applicable.

Higher bearing stresses are acceptable where suitable measures have been taken to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas and additional binding reinforcement in the ends of the members. Bearing stresses due to ultimate loads shall then be limited to:

$$\frac{3(f_{cu}/\gamma_{mc})}{1+2\sqrt{A_{con}/A_{sup}}} \quad \textit{but not more than } (1.5\, f_{cu}/\gamma_{mc})$$

where

A_{con} is the contact area;

A_{sup} is the supporting area.

For rectangular bearings (see Figure 6a)

$A_{sup} = (b_x + 2x)(b_y + 2y)$ and $x \leq b_x$, $y \leq b_y$ where

b_x, b_y are the dimensions of the bearing in the x, y directions respectively;

x, y are the dimensions from the boundary of the contact area to the boundary of the support area.

For lightweight aggregate concrete the bearing stresses due to ultimate loads shall be limited to two-thirds of those for normal weight aggregate concrete given by the above formula.

Higher bearing stresses due to ultimate loads shall be used only where justified by tests, eg concrete hinges.

7.2.3.4 Horizontal forces or rotations at bearings The presence of significant horizontal forces at bearing can reduce the load-carrying capacity of the supporting and supported members considerably by causing premature splitting or shearing. These forces may be due to creep, shrinkage and temperature effects or result from misalignment, lack of plumb or other causes. When these forces are likely to be significant, it is necessary to check that either:

(a) sliding bearings are present; or

(b) suitable lateral reinforcement is present in the top of the supporting member; and

(c) continuity reinforcement is present to tie together the ends of the supported members.

Where, owing to large spans or other reasons, large rotations are likely to occur at the end supports of flexural members, suitable bearings capable of accommodating these rotations should be present. In the absence of such bearings, bearing stresses could be increased due to concentration of the reaction towards one edge of a bearing and/or flexure of the supported member could result, depending on the type of bearing actually present.

7.2.4 Joints between precast members

7.2.4.1 General The critical sections of members close to joints shall be assessed under the worst combinations of shear, axial force and bending effects caused by the co-existing ultimate vertical and horizontal forces. The evaluation of the effects shall take due account of any fixities imposed by the joints.

7.2.4.2 Halving joint For the type of joint shown in figure 7A(a), the maximum vertical ultimate load, F_v, shall not exceed $v_u b d_o$ where:

v_u is $0.36\ (0.7 - f_{cu}/250)\ f_{cu}/\gamma_{mc}$
b is the breadth of the beam, and
d_o is the depth to the horizontal reinforcement in the halving joint.

The capacity of halving joint may be determined by considering the two following strut and tie systems and summing the capacities of the two systems, and in accordance with the requirements of BA 39.

The first system, shown in figure 7A(b), involves the inclined reinforcement which intersects the line of action of F_v. The inclined reinforcement may take the form of bent-up bars or links. In the case of bent-up bars the bearing stresses inside the bends (see 5.8.6.9) shall be checked to determine whether the stress in the bars should be limited to less than f_y/γ_{ms}.

In the case of links, their anchorage in the compression face of the beam shall be in accordance with 5.8.6.5, whilst in the tension face the horizontal component, F_h, of the link force is transferred to the main reinforcement. The links may be considered to be fully anchored in the tension face if the anchorage bond stress of the main reinforcement due to the force F_h does not exceed twice the anchorage bond stresses given in 5.8.6.3.

The second strut and tie system shown in figure 7A(c) involves the vertical reinforcement in the full depth section adjacent to the halving joint, and the horizontal reinforcement in the halving joint in excess of that required to resist the horizontal ultimate load.

7.3 Structural connections between units

7.3.1 General

7.3.1.1 Structural requirements of connections When assessing the connections across joints between precast members the overall stability of the structure shall be considered.

7.3.1.2 Assessment method Connections shall, where possible, be assessed in accordance with the generally accepted methods applicable to reinforced concrete (see clause 5), prestressed concrete (see clause 6) or structural steel. Where, by the nature of the construction or material used, such methods are not applicable, the efficiency of the connection shall be proved by appropriate tests.

7.3.1.3 Considerations affecting design details

"Not applicable to assessment."

7.3.1.4 Factors affecting design and construction

"Not applicable to assessment."

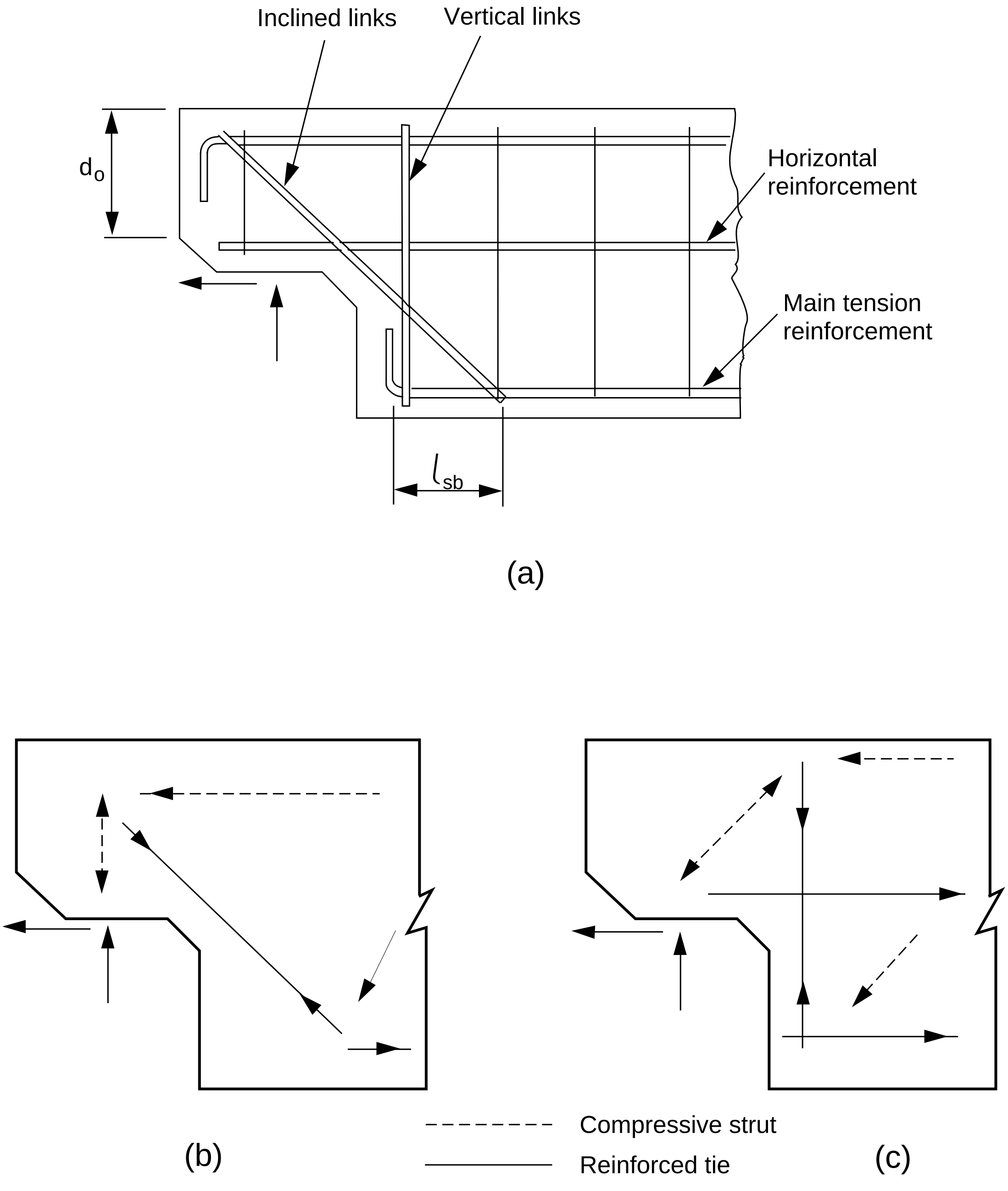


Figure 7A Halving Joint

7.3.2 Continuity of reinforcement

7.3.2.1 General The assumptions made in analysing the structure and assessing critical sections shall reflect the degree of continuity of reinforcement through a connection. The following methods are capable of achieving continuity of reinforcement:

(a) lapping bars;
(b) butt welding;
(c) sleeving;
(d) parallel threading of bars and tapered threads.

The strength of the joints in (c) and (d) and any other method not listed shall be assessed on the basis of test evidence.

7.3.2.2 Sleeving The following three principal types of sleeve jointing are acceptable provided that appropriate test data are available:

(a) grout or resin filled sleeves;

(b) sleeves that mechanically align the square-sawn ends of two bars to allow the transmission of compressive forces only;

(c) sleeves that are mechanically swaged to the bars.

7.3.2.3 Threading The following methods for joining threaded bars are acceptable:

(a) Parallel threaded ends of bars are joined by a coupler having left- and right-hand threads.

(b) One set of bars is welded to a steel plate that is drilled to receive the threaded ends of the second set of bars, which are fixed to the plate by means of nuts.

(c) Threaded anchors cast into a precast unit to receive the threaded ends of reinforcement.

(d) Taper threaded bars joined by the use of internally taper threaded couplers.

The structural assessment of special threaded connections shall be based on tests in accordance with BS 5400:Part 1, including behaviour under fatigue conditions where relevant. Where tests have shown the strength of the threaded connection to be greater than or equal to the characteristic strength of the parent bars, the strength of the joint may be based on the specified characteristic strength of the joined bars divided in each case by the appropriate γ_{ms} factor.

7.3.2.4 Welding of bars See clause 4.7.

7.3.3 Other types of connection The load carrying capacity of any other type of connection shall be justified by test evidence. For resisting shear and flexure suitable are those types which are made by prestressing across the joint.

7.4 Composite concrete construction

7.4.1 General The requirements of 7.4 apply to flexural members consisting of precast concrete units acting in conjunction with added concrete where the contact surface is capable of transmitting longitudinal shear. The precast units may be of either reinforced or prestressed concrete.

In general, the analysis and assessment of composite concrete structures and members shall be in accordance with clause 5 or 6, modified where appropriate by 7.4.2 and 7.4.3. Particular attention shall be given in the assessment to the effect of the method of construction and whether or not props were used. The relative stiffness of members shall be based on the concrete, gross transformed or net transformed section properties as described in 4.4.2.1; if the concrete strengths in the two components of the composite member differ by more than 10 N/mm^2, allowance for this shall be made in assessing stiffness and stresses.

Differential shrinkage of the added concrete and precast concrete members requires consideration in analysing composite members for the serviceability limit states (see 7.4.3); it need not be considered for the ultimate limit state.

When precast prestressed units, having pre-tensioned tendons, are assessed as continuous members with continuity obtained with reinforced concrete cast in situ over the supports, the compressive stresses due to prestress in the ends of the units may be assumed to vary linearly over the transmission length for the tendons in assessing the strength of sections.

7.4.2 Ultimate limit state

7.4.2.1 General Where the cross-section of composite members and the applied loading have increased by stages (eg a precast prestressed unit initially supporting self-weight and the weight of added concrete and subsequently acting compositely for live loading), the entire load may be assumed to act on the final cross-section.

7.4.2.2 Vertical shear The assessment of the resistance of composite sections to vertical shear shall be in accordance with 5.3.3 for reinforced concrete (except that in determining the area A_s, the area of tendons within the transmission length shall be ignored) and 6.3.4 for prestressed concrete, modified where appropriate as follows:

(a) For I, M, T, U and box beam precast prestressed concrete units with an in situ reinforced concrete top slab cast over the precast units (including pseudo box construction), the shear resistance shall be based on either of the following:

(1) the precast unit acting alone assessed in accordance with 6.3.4;

(2) the composite section assessed in accordance with 6.3.4. In this case, section properties shall be based on those of the composite section, with due allowance for the different grades of concrete where appropriate.

(b) For inverted T beam precast prestressed concrete units with transverse reinforcement placed through holes in the bottom of the webs of the units, completely infilled with concrete placed between and over the units to form a solid deck slab, the shear resistance shall be based on either of the following:

(1) as for (a)(1);

(2) the shear resistances of the infill concrete section and the precast prestressed section shall be assessed separately in accordance with 5.3.3 and 6.3.4 respectively. The shear capacity, V, of the composite section shall then be taken as the lesser of:

$V_i(A_i + A_p)/A_i$ and $V_p(A_i + A_p)/A_p$,

where

V_i is the shear capacity of the infill concrete assessed in accordance with 5.3.3. with the breadth taken as the distance between adjacent precast webs and the depth as the mean depth of infill concrete, or the mean effective depth to the longitudinal reinforcement where this is provided in the infill section;

V_p is the shear capacity of the precast prestressed section assessed in accordance with 6.3.4 with the breadth taken as the web thickness and the depth as the depth of the precast unit;

A_i is the cross-sectional area of the infill concrete with due allowance for the concrete grade being different to that of the precast units where appropriate;

A_p is the cross-sectional area of the precast units.

(c) In applying 6.3.4.4, d_t shall be derived for the composite section.

7.4.2.3 Longitudinal shear The longitudinal shear force, V_1, per unit length of a composite member, whether simply supported or continuous, shall be calculated at the interface of the precast unit and the in situ concrete and at any vertical planes which may be critical in longitudinal shear (eg planes 2-2 or 2´-2´ in figure 8A) by an elastic method using properties of the composite concrete section (see 4.4.2.1) with due allowance for different grades of concrete where appropriate.

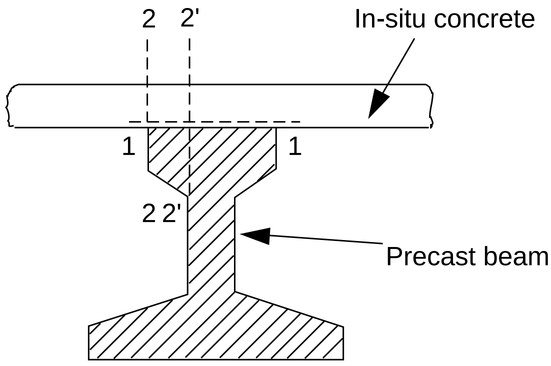


Figure 8A Potential Planes

V_1 shall not exceed the lesser of the following:

(a) $(k_1 f_{cu}/\gamma_{mc})L_s$

(b) $(v_1/\gamma_{mv})L_s + (0.8\ A_e f_s/\gamma_{ms})$, where

k_1 is a constant depending on the concrete bond across the shear plane under consideration and shall be taken as 0.24 for monolithic construction or surface type 1 or 0.14 for surface type 2. These values shall be reduced by 25% for lightweight aggregate concrete construction;

f_{cu} is the characteristic, or worst credible, strength of the weaker of the two concretes each side of the shear plane; but shall not be taken as >45 N/mm²

γ_{mc} is the partial safety factor for concrete given in 4.3.3.3;

L_s is the breadth of the shear plane under consideration;

v_1 is the longitudinal shear stress in the concrete for the plane under consideration, and shall be taken as:

for monolithic construction: 0.05 f_{cu} but not less than 1.13 N/mm^2 and not greater than 1.56 N/mm^2.

for surface type 1: 0.04 f_{cu} but not less than 0.8 N/mm² and not greater than 1.28 N/mm²;

for surface type 2: 0.019 f_{cu} but not less than 0.38 N/mm² and not greater than 0.63 N/mm².

All values shall be reduced by 25% for lightweight aggregate concrete construction;

γ_{mv} is the material partial safety factor for shear given in 5.3.3.2;

A_e is the area of reinforcement per unit length crossing the shear plane under consideration; reinforcement assumed to resist co-existent bending and vertical shear (see 7.4.2.2) may be included;

f_s is the stress at the ultimate limit state in the steel reinforcement of area A_e. The stress may be assumed to be the characteristic, or worst credible, strength, f_y, if the reinforcement A_e is fully anchored (see 5.8.6), otherwise f_s should be such that $A_e f_s / b \ngtr 10$ N/mm² where b is the width of the interface under consideration.

γ_{ms} is the partial safety factor for steel given in 4.3.3.

For composite beam and slab construction, reinforcement crossing the shear plane shall be considered as effective only if its spacing does not exceed the lesser of the following:

(a) six times the minimum thickness of the in situ concrete flange;

(b) 900 mm.

The types of surface are defined as follows.

Type 1. The contact surface of the concrete in the precast members was prepared as described in either (1) or (2) as appropriate.

(1) When the concrete had set but not hardened the surface was sprayed with a fine spray of water or brushed with a stiff brush, just sufficient to remove the outer mortar skin and expose the larger aggregate without disturbing it.

(2) The surface skin and laitance were removed by sand blasting or the use of a needle gun, but not by hacking.

Type 2. The contact surface of the concrete in the precast member was jetted with air and/or water to remove laitance and all loose material. (This type of surface is known as 'rough as cast'.)

The type of surface shall be assessed from contract record drawings, available site data or original design calculations. In the absence of such information, surface type 2 shall be assumed.

For inverted T beams defined in 7.4.2.2(b) no longitudinal shear strength check is required.

7.4.3. Serviceability limit state When a serviceability limit state assessment is required, the necessary criteria shall be agreed with the Overseeing Department.

7.5 Plain concrete walls and abutments

7.5.1 General A plain concrete wall or abutment is a vertical load bearing concrete member whose greatest lateral dimension is more than four times its least lateral dimension and which is assumed to be without reinforcement when considering strength.

The requirements given in 7.5.2 to 7.5.10 refer to the assessment of a plain concrete wall that has a height not exceeding five times its average thickness.

7.5.2 Moments and forces in walls and abutments Moments, shear forces and axial forces in a wall shall be determined in accordance with 4.4.

The axial force may be calculated on the assumption that the beams and slabs transmitting forces into it are simply supported.

The resultant axial force in a member may act eccentrically due to vertical loads not being applied at the centre of the member or due to the action of horizontal forces. Such eccentricities shall be treated as indicated in 7.5.3 and 7.5.4.

The minimum moment in a direction at right-angles to the wall shall be taken as not less than that produced by considering the ultimate axial load per unit length acting at an eccentricity of 0.05 times the thickness of the wall.

7.5.3 Eccentricity in the plane of the wall or abutment

In the case of a single member this eccentricity can be calculated from statics alone. Where a horizontal force is resisted by several members, the amount allocated to each member shall be in proportion to its relative stiffness provided the resultant eccentricity in any individual member is not greater than one-third of the length of the member. Where a shear connection between vertical edges of adjacent members can withstand the calculated forces, an appropriate elastic analysis may be used.

7.5.4 Eccentricity at right-angles to walls or abutments

The load transmitted to a wall by a concrete deck may be assumed to act at one-third the depth of the bearing area from the loaded face. Where there is an in situ concrete deck on either side of the member the common bearing area may be assumed to be shared equally by each deck.

The resultant eccentricity of the total load on a member unrestrained in position at any level shall be calculated making full allowance for the eccentricity of all vertical loads and the overturning moments produced by any lateral forces above that level.

The resultant eccentricity of the total load on a member restrained in position at any level may be calculated on the assumption that immediately above a lateral support the resultant eccentricity of all the vertical loads above that level is zero.

7.5.5 Analysis of section Loads of a purely local nature (as at bearings or column bases) may be assumed to be immediately dispersed provided the local stress under the load does not exceed that given in 7.5.7. Where the resultant of all the axial loads act eccentrically in the plane of the member, the ultimate axial load per unit length of wall, n_w , shall be assessed on the basis of an elastic analysis assuming a linear distribution of load along the length of the member, assuming a tensile resistance of concrete of $0.12\sqrt{\frac{f_{cu}}{\gamma_{mc}}}$.

Consideration shall first be given to the axial force and bending in the plane of the wall to determine the distribution of tension and compression along the wall. The bending moment at right-angles to the wall shall then be considered and the section assessed for this moment and the compression or tension per unit length at various positions along the wall. Where the eccentricity of load in the plane of the member is zero, a uniform distribution of n_w may be assumed.

For members restrained in position, the axial load per unit length of member, n_w, due to ultimate loads shall be such that:

$$n_w \leq (0.675\ f_{cu}/\gamma_{mcw})(h-2e_x) \qquad \text{equation 36}$$

where

n_w is the maximum axial load per unit length of member due to ultimate loads;

h is the overall thickness of the section;

e_x is the resultant eccentricity of load at right-angles to the plane of the member (see 7.5.2) (minimum value 0.50h);

f_{cu} is the characteristic, or worst credible, concrete strength;

γ_{mcw} is a material partial safety factor which is taken as 2.25 if the characteristic concrete strength is used, and 1.80 if the worst credible strength is used.

7.5.6 Shear The resistance to shear forces in the plane of the member may be assumed to be adequate provided the horizontal shear force due to ultimate loads is less than either one-quarter of the vertical load, or the force to produce an average shear stress of 0.45 N/mm² over the whole cross-section of the member in the case of f_{cu} of at least 25 N/mm²; where f_{cu} is less than 25 N/mm², a figure of 0.3N/mm² should be used.

7.5.7 Bearing Bearing stresses due to ultimate loads of a purely local nature, as at girder bearings, shall be limited in accordance with 7.2.3.3.

7.5.8 Deflection of plain concrete walls or abutments
"Not applicable to assessment."

7.5.9 Shrinkage and temperature reinforcement
"Not applicable to assessment"

7.5.10 Stress limitations for serviceability limit state
"Not applicable to assessment."