Design Manual for Roads and Bridges

Highway Structures & Bridges
Inspection & Assessment

CS 455
The assessment of concrete highway bridges and structures

(formerly BD 44/15, BA 38/93, BA 40/93, BA 51/95 and BA 52/94)

Version 1.1.0

Summary
The use of this document enables the concrete highway bridges and structures to be assessed, providing key information that is required to manage risks and maintain a safe and operational network.

Application by Overseeing Organisations
Any specific requirements for Overseeing Organisations alternative or supplementary to those given in this document are given in National Application Annexes to this document.

Feedback and Enquiries
Users of this document are encouraged to raise any enquiries and/or provide feedback on the content and usage of this document to the dedicated National Highways team. The email address for all enquiries and feedback is: Standards_Enquiries@highwaysengland.co.uk

This is a controlled document.

Contents

Latest release notes

Document code	Version number	Date of publication of relevant change	Changes made to	Type of change
CS 455	1.**1**.0	February 2022	Core document	Incremental change to requirements

To introduce some minor technical and editorial changes resulting from feedback that clarify the applicability of some clauses and give supplementary advice and typos, applies to Eq. 5.7.1, Cl. 5.9, Cl. 8.17, Eq. 8.22f, Cl. 8.42.3 and Eq. 9.1a.

Previous versions

Document code	Version number	Date of publication of relevant change	Changes made to	Type of change
CS 455	1	January 2021		
CS 455	0	March 2020		

Foreword

Publishing information

This document is published by National Highways.

This document supersedes BD 44/15, BA 38/93, BA 40/93, BA 51/95 and BA 52/94, which are withdrawn.

Contractual and legal considerations

This document forms part of the works specification. It does not purport to include all the necessary provisions of a contract. Users are responsible for applying all appropriate documents applicable to their contract.

Introduction

Background

The use of this document enables concrete highway bridges and structures to be assessed, providing key information that is required to manage risks and maintain a safe and operational network.

The document is to be used in combination with CS 454 [Ref 2.N], which includes the key requirements for assessment of highway structures in general, while this document includes the additional requirements specific to concrete structures, including models for the assessment of resistance.

The latest update of this document has included editorial improvements in order to remove ambiguity, improve clarity, remove repetition, and condense the document to be focussed on the requirements.

The update has combined provisions in the previous documents BD 44/15, BA 38/93, BA 40/93, BA 51/95 and BA 52/94 as appropriate.

The main technical changes introduced in this update are:

1) alignment of assessment processes and basis of assessment with CS 454 [Ref 2.N];
2) incorporation of relevant material from BS 5400-4 [Ref 19.I];
3) SLS verification criteria are set out only for components that are required to be assessed at SLS, i.e. prestressed concrete;
4) further guidance on alternative methods for estimating worst credible strength to align with BS EN 1990 [Ref 6.I] and BS EN 13791 [Ref 2.I], and to improve safety;
5) simplification of the application of the shear resistance model;
6) increase in shear resistance near a support where there is a short anchorage length;
7) incorporation of strut-and-tie modelling;
8) SLS requirements for prestressed concrete are set out explicitly, and the assessment reports on the SLS class for prestressed elements;
9) shear resistance of sections uncracked in flexure reverts to the BD 44/95 approach that is justified empirically by testing;
10) new rules for the effect of cover on bond;
11) new appendix covering the assessment of concrete structures affected by steel corrosion, based on an updating of BA 51/95 and incorporating provisions from BA 38/93;
12) new appendix on the assessment of concrete structures affected by internal degradation of concrete, based on an updating of BA 52/94.

The document has also permitted some models from BS EN 1992-2 [Ref 3.N] to be used in specific assessment verifications.

When referring to the requirements of BS EN 1992-2 [Ref 3.N] in this document it is assumed that this reference also includes the relevant parts of BS EN 1992-1-1 [Ref 5.I] and the UK national annexes for both standards.

Assumptions made in the preparation of the document

The assumptions made in GG 101 [Ref 4.N] apply to this document.

Abbreviations and symbols

Abbreviations

Abbreviation	Definition
AAR	alkali aggregate reaction
ASR	alkali silica reaction
DEF	delayed ettringite formation
IDC	internal degradation of concrete
RH	relative humidity
SLS	serviceability limit state
ULS	ultimate limit state

Symbols

Symbol	Definition
A_c	area of concrete
A_{con}	contact area
A_e	area of fully anchored reinforcement per unit length crossing the shear plane
A_{ly}	area of effectively anchored longitudinal reinforcement in excess to that required to resist bending co-existent with the shear force
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 of a column
A_{s2}	area of reinforcement in the other face of a column
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_{su}	area of the untensioned steel
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_{con}	factor relating to the confining effect of links on bond of bars with low cover

Symbols (continued)

Symbol	Definition
a_v	distance from the section under consideration to the supporting member; distance from the boundary of the loaded area to the perimeter considered for punching shear
b	width or breadth of section; distance between void centres in voided slabs
b_a	average breadth of section excluding the compression flange
b_c	breadth of compression face
b_{col}	width of column
b_e	width of the edge zone of a slab
b_t	breadth of section at level of tension reinforcement
b_w	breadth of web or rib of a member
c	depth of cover concrete
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 from the more highly compressed face
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_0	depth to the horizontal reinforcement in the half joint
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 face other than the more highly compressed
E_{cm}	short-term modulus of elasticity for concrete
$(EI)_c$	flexural rigidity of the column cross-section
e_x	resultant eccentricity of load at right-angles to plane of wall
F	assessment tension force in a bar or group of bars
F_{bst}	tensile bursting force
F_{bt}	tensile force due to ultimate loads in a bar or group of bars
F_{ub}	ultimate anchorage capacity of tension reinforcement
f_{cav}	average compressive stress in the flexural compressive zone
f_{ci}	concrete strength at transfer
f_{ck}	characteristic cylinder strength of the concrete
f_{cu}	characteristic or worst credible concrete cube strength

Symbols (continued)

Symbol	Definition
f_{cui}	values of the in situ strengths obtained from testing a representative sample of cores.
f_{pe}	assessment prestress
f_{pt}	stress due to prestress
f_{pu}	characteristic or worst credible strength of prestressing tendons
f_s	stress in reinforcement in tension
f'_s	stress in reinforcement in compression
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_{ub}	average anchorage bond strength
f_{yi}	values of yield strength obtained from sample tests of the bar size and type being assessed
f_{yL}	characteristic or worst credible 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	depth of the flange
$h_{\max}$	larger dimension of section
$h_{\min}$	smaller dimension of section
h_{red}	depth of concrete in compression under the ultimate loads on precast segmental structures with unbonded prestressing only
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	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; reduction factor for the shear resistance of voided slabs employed, the nature of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete; reduction factor for the shear resistance of voided slabs
k_{cov}	cover factor
k_t	depends on the type of tendon
k_l	depends on the concrete bond across the shear plane
L_a	effective anchorage length
L_{rb}	slenderness limit for lateral stability verifications in beams
L_{rc}	slenderness limit for lateral stability verifications in cantilevers
L_s	length of shear plane

Symbols (continued)

Symbol	Definition
l	thickness of concrete member in the plane of a bent reinforcing bar
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_l	length of reinforcing bar measured inside the bend and bearing onto the concrete
l_{ra}	reanchorage length
l_t	transmission length
M	bending moment due to ultimate loads
$M^*_{\alpha i}$	moment resistance of a slab per unit width due to the i-direction reinforcement
M_{cr}	cracking moment at the section considered
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_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	moment resistance
M_{ux}	moment resistance in a short column assuming ultimate axial load and bending about the major axis only
M_{uy}	moment resistance 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; moments per unit width about the axes of a slab
M^*_n	moment resistance of a slab per unit width in the n direction
M_{xy}	twisting moment per unit width about the x,y axes of a slab
M_n	moment per unit width about an axis perpendicular to the n-direction in a slab
M^*_n	moment resistance of a slab per unit width in the n-direction
M_{nt}	twisting moment per unit length of a slab adjacent to the edge zone referred to axes perpendicular and parallel to the edge
M_o	moment necessary to produce zero stress in the concrete at the depth d
M_1	smaller initial end moment in a column due to ultimate loads (assumed negative if the column is bent in double curvature)
M_2	larger initial end moment in a column due to ultimate loads (assumed positive)
N	ultimate axial load at section considered; number of bars in a group
N^*_c	compressive resistance of a slab per unit width

Symbols (continued)

Symbol	Definition
N_i^*	tensile resistance per unit width of the i-layer of reinforcement in the direction of the reinforcement
N_x, N_y	in-plane axial forces in a slab per unit width
N_{xy}	in-plane shear force in a slab per unit width
N_u	axial resistance
N_{uz}	axial resistance of a column ignoring all bending
n	number of tests in the sample
n_t	number of strands or wires in a tendon
n_w	ultimate axial load per unit length of wall
p	effective perimeter
P_f	effective prestressing force after all losses
P_h	horizontal component of the prestressing force after all losses
P_k	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
R	support reaction
r	internal radius of bend
r_{ps}	radius of curvature of a tendon
s	sample standard deviation
s_v	spacing of links along the member
u_0	perimeter of the loaded area for a concentrated load
T	torque due to ultimate loads
T_u	ultimate torsional strength
t	breadth of bearing area over a support
$t_{0.01}$	parameter defining the 1% fractile on a one-sided region of a Student's t-distribution statistical curve
$t_{0.05}$	parameter defining the 5% fractile on a one-sided region of a Student's t-distribution statistical curve
V	shear force due to ultimate loads
V_c	shear resistance of concrete
V_c'	shear resistance of a solid slab
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_l	longitudinal shear force per unit length
V_{lu}	longitudinal shear force due to ultimate load

Symbols (continued)

Symbol	Definition
$V_{\max}$	maximum shear resistance based on concrete crushing
V_P	known coefficient of variation for the concrete strengths in the location being assessed
V_{Pc}	concrete component of the punching shear resistance
$V_{P\max}$	maximum punching shear resistance
V_{Pu}	punching shear resistance
V_p	shear capacity of precast prestressed section
V_{sv}	shear resistance due to links in a voided slab
V_t	flexural shear force per unit width at the edge of a slab
V_u	shear resistance
V_{uc}	component of shear resistance provided by concrete
V_{ue}	shear resistance of the edge zone of a slab
V_{us}	component of shear resistance provided by effective shear reinforcement
V_{ux}	shear resistance in the x-axis
V_{uy}	shear resistance in the y-axis
V_x	applied shear in the x-axis
V_y	applied shear in the x-axis
v	shear stress
v_1	ultimate shear stress in concrete
$v_{\max}$	maximum shear stress corresponding to concrete crushing
v_t	torsional shear stress
$v_{t\min}$	minimum ultimate torsional shear stress above which reinforcement is required
v_{tu}	ultimate torsional shear stress
x	neutral axis depth; distance form jack
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 to the member axis; factor to determine σ_{pb}
α_i	angle of the reinforcement in a slab from the x-axis
α_1	angle between the axis of the design moment and the direction of the tensile reinforcement
α_{cw}	coefficient taking into account of the state of the stress in the compression chord

Symbols (continued)

Symbol	Definition
α_n	coefficient as a function of column axial load
β	coefficient dependent on bar type
Γ	reduction factor for short anchorage
$\gamma_{f1}, \gamma_{f2}, \gamma_{f3}$	partial load factors
γ_{fL}	product of γ_{f1} and γ_{f2}
γ_m	partial safety factor for strength
γ_{mb}	partial safety factor for bond
γ_{mbs}	component of partial safety factor for bond allowing to the variation in bond strength
γ_{mc}	partial factor for concrete
γ_{mcw}	partial factor for plain concrete wall
γ_{ms}	partial factor for steel
γ_{mv}	partial factor applied to shear in concrete
ΔM	reduction in the hogging moment
$\Delta P_{(x)}$	loss of prestressing force at any distance x from the jack / along the curve from the tangent point
ϵ	strain
ϵ_s	strain in tension reinforcement
ϵ_p	prestrain in the tendon after all losses.
θ	angle between the concrete compression strut and the beam axis perpendicular to the shear force used in variable truss method; angle between the n-direction and the x-axis in a slab
θ_P	plastic rotation capacity
μ	coefficient of friction
μ_{joint}	effective coefficient of friction between joints in post-tensioned segmental construction.
ξ_s	depth factor
ρ	geometrical ratio of reinforcement, generally equal to $A_s/(bd)$
ρ_s	ratio of longitudinal reinforcement
$\sum A_{sv}$	area of shear reinforcement
$\sum bd$	area of the critical section for punching shear
$\sum \theta$	sum of the angular displacements over the distance x
σ	stress
σ_{ci}	maximum compressive stress at transfer
σ_{cpb}	total direct stress at the location of the prestressed section being checked due to bending and axial load effects, taken as positive in compression

Symbols (continued)

Symbol	Definition
σ_{cp}	mean compressive stress in the concrete due to the prestressing and axial loading
σ_{ct}	assessment tensile stress in the concrete at SLS in units of MPa
σ_{pb}	tensile stress in tendons at failure (of the section)
σ_{s2}	stress in the reinforcement in the other face, derived from Section 3 and taken as negative if tensile
υ	strength reduction factor for concrete cracked in shear
ϕ	size (nominal diameter) of bar or tendon
φ	diameter of the void in voided slabs
ψ	size factor for rotation capacity for reinforced concrete

Terms and definitions

Terms	Definitions
Assessment	the process of determining in terms of vehicle loading the load that an existing structure can carry with an acceptable probability that it does not suffer serious damage that can endanger any persons on or near the structure
Corbel	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.
Half joint	a joint in a beam or slab giving rotational freedom by means of a bearing or similar at or near mid depth
Plain concrete wall or abutment	vertical load bearing concrete member, the greatest lateral dimension of which is more than four times its least lateral dimension and which is assumed to be without reinforcement when considering strength
Reinforced concrete column	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
Reinforced wall	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
Transmission length in pretensioned 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.
Worst credible strength	worst value of that strength which the assessor realistically considers could be obtained in the structural element under consideration. The worst credible strength value can be greater or less than the characteristic strength of the material assumed at the design stage.

1. Scope

Aspects covered

1.1 This document shall be used for the assessment of existing concrete highway bridges and structures, and their structural elements, including those constructed from:

1) reinforced concrete;
2) prestressed concrete; or,
3) plain concrete.

NOTE 1 Requirements for cast in-situ, precast and composite concrete structures and elements are included.

NOTE 2 This document includes requirements for reinforced concrete constructed from lightweight aggregate concrete where the strength of the lightweight aggregate concrete exceeds 25 MPa. Lightweight aggregate concrete with strength below 25 MPa is not covered.

NOTE 3 This document does not include requirements for prestressed structures constructed of lightweight aggregate concrete.

NOTE 4 This document includes requirements for the assessment of prestressed structural elements using systems of pretensioning and post-tensioning, applied by means of internal or external tendons that are bonded or unbonded.

Implementation

1.2 This document shall be implemented forthwith on all schemes involving the assessment of concrete highway bridges and structures on the Overseeing Organisations' motorway and all-purpose trunk roads according to the implementation requirements of GG 101 [Ref 4.N].

Use of GG 101

1.3 The requirements contained in GG 101 [Ref 4.N] shall be followed in respect of activities covered by this document.

2. Assessment processes and basis of assessment

Assessment processes

2.1 The assessment processes described in CS 454 [Ref 2.N] shall be applied.

Basis of assessment

2.2 The basis of assessment shall align with the requirements of CS 454 [Ref 2.N].

Limit states

2.3 Concrete structures shall be assessed at the ULS.

2.3.1 The assessment at the ULS should include the verification of structural safety, regarding the collapse or failure of structural members or the entire structure as a result of material rupture or loss of stability.

2.4 The stresses in prestressed concrete elements shall be assessed and verified against limiting stresses at the SLS to determine the SLS class as defined in Section 8.

2.5 Where additional SLS assessment verifications are proposed, the methodology and criteria for the assessment verifications shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

2.6 Where there is a particular concern regarding the fatigue performance of the structure, the need for an assessment of fatigue shall be agreed with the Overseeing Organisation and subject to technical approval in accordance with CG 300 [Ref 7.N].

NOTE 1 Examples of particular concerns regarding the fatigue performance can include structures where it is known that there is welded reinforcement or corroded reinforcement.

NOTE 2 Appendix A includes guidance on assessment of corroded reinforcement.

NOTE 3 Methodologies for assessment of fatigue in steel structures are provided in CS 456 [Ref 26.I].

Assessment actions

2.7 The actions and partial factors for actions shall be defined in accordance with CS 454 [Ref 2.N].

2.8 The actions for creep, shrinkage and prestress shall be defined in accordance with this document.

2.9 The partial factors for creep, shrinkage and prestress shall be obtained from Table 2.9 with the exception that, for the moment resistance of bonded prestressing, the ULS partial factor is to be taken as equal to unity in all cases.

Table 2.9 Partial factors for creep, shrinkage and prestress

Action		γ_{fL} ULS	γ_{fL} SLS
Creep and Shrinkage		1.0	1.0
Prestress	when prestress has a beneficial effect	0.87	1.0
Prestress	when prestress has an adverse effect	1.15	1.0
Prestress	for calculating secondary effects of prestress in statically indeterminate structures	1.0	1.0

Assessment action effects

2.10 The assessment action effects shall be determined in accordance with CS 454 [Ref 2.N] and the requirements for structural analysis in this document.

NOTE General requirements for structural analysis are given in Section 4. Additional requirements for structural analysis for particular members are also stated in the relevant sections of this document.

2.10.1 The assessment action effects of creep and shrinkage of concrete, temperature difference and differential settlement actions may be omitted from the ULS assessment of reinforced and prestressed concrete structures unless either of the following applies:

1) there is a concern about the ductility of the structure, its components or materials; or,
2) the assessment is being carried out to investigate distress caused by movements or restraint to movements.

NOTE Creep and shrinkage affect the magnitude of the prestressing action, even when the action effects of creep and shrinkage are omitted.

Assessment of resistance

2.11 The resistance of concrete structures shall be determined in accordance with the principles of CS 454 [Ref 2.N] as implemented by the requirements of this document.

2.12 The effects of deterioration shall be included in the calculation of resistance.

2.12.1 The effects of deterioration in concrete structures should be assessed based on:

1) assessment models and parameters that directly account for the deterioration; and,
2) a condition factor, as defined in CS 454 [Ref 2.N], taken as equal to 1.0.

NOTE 1 Allowances for deterioration can include:

1) reductions in thickness or area to represent the loss of material;
2) changes to material properties such as strength, stiffness or ductility;
3) structural analysis models that represent the effects of deterioration on the structure behaviour; and,
4) structural resistance models that represent the effects of deterioration on the structural resistance.

NOTE 2 Section 3 includes guidance on estimating worst credible strengths.

2.12.2 The effects of reinforcement corrosion should be modelled according to Appendix A.

2.12.3 The effects of internal degradation of concrete should be modelled according to Appendix B.

2.13 Partial factors for material properties shall be taken from:

1) Table 2.13a at the ULS;
2) Table 2.13b for prestressed concrete at the SLS.

Table 2.13a Partial factors for material properties at the ULS

	Application		For use with characteristic strength	For use with worst credible strength
γ_{ms}	Reinforcement and prestressing tendons	All steel except grade 460	1.15	1.10 or 1.05 when measured steel depths are used.
		Grade 460 steel	1.05	1.05
γ_{mc}	Concrete		1.50	1.20
γ_{mv}	Shear in concrete		1.25	1.15
γ_{mb}	Bond		1.40	1.25
γ_{mcw}	Plain concrete wall		2.25	1.80

Note: The lower partial factors for use with worst credible strength allow for:

1) the reduction in uncertainty when information from the built structure is used to retrospectively estimate the strengths, relative to specifying a characteristic strength at the design stage; and,
2) the effect arising from the difference in concrete curing conditions between standard test cubes and concrete in the structure, so that the strength and uncertainty achieved in cube tests is higher than the in-situ concrete strength as measured in cores taken from the structure.

Table 2.13b Partial factors for SLS verification of prestressed concrete

	Application	For use with characteristic strength	For use with worst credible strength
γ_{mc}	Compression due to bending in concrete[1]	1.25	1.13
	Compression due to axial loads in concrete[2]	1.67	1.50
	Tension in pre-tensioned concrete.	1.25	1.13
	Tension in post-tensioned concrete.	1.55	1.40

Note 1: The factors for compression due to bending are based on a predominantly triangular stress distribution.
Note 2: The factors for compression due to axial loads are based on a predominantly rectangular stress distribution.
Note 3: The partial factors for use with characteristic strength at SLS are derived from BS 5400-4 [Ref 19.I] and are intended for use with the SLS verifications set out in Section 8. Where alternative SLS criteria are agreed, the partial factors need to be consistent with the agreed approach.
Note 4: The partial factors for use with worst credible strength at SLS are derived by reducing the partial factors for use with characteristic strength at SLS by 10%.

Table 2.13c Partial factors for SLS verification of reinforced concrete

	Application	For use with characteristic strength	For use with worst credible strength
γ_{mc}	Compression due to bending in concrete[1]	1.00	1.00
	Compression due to axial loads in concrete[2]	1.33	1.20
γ_{ms}	Tension and compression in reinforcement	1.00	1.00

Note 1: The factors for compression due to bending are based on a predominantly triangular stress distribution.
Note 2: The factors for compression due to axial loads are based on a predominantly rectangular stress distribution.
Note 3: The partial factors for use with characteristic strength at SLS are derived from BS 5400-4 [Ref 19.I] and are intended for use with the SLS verifications set out in BS 5400-4 [Ref 19.I] or CS 468 [Ref 1.N]. Where alternative SLS criteria are agreed, the partial factors need to be consistent with the agreed approach.
Note 4: The partial factors for use with worst credible strength at SLS are derived by reducing the partial factors for use with characteristic strength at SLS by 10%, but no less than unity.

2.14 Where additional SLS verifications are carried out, for example on non-prestressed concrete sections, the partial factors at SLS shall be consistent with the methods proposed for the verifications.

2.15 Where SLS verifications of reinforced concrete based on BS 5400-4 [Ref 19.I] or CS 468 [Ref 1.N] are proposed, the partial factors for materials shall be as set out in Table 2.13c.

Verification

2.16 The assessment of resistance shall be compared with the assessment action effects including the effects of γ_{f3} according to CS 454 [Ref 2.N].

3. Material properties

Concrete

3.1 Concrete cube strength shall be based on either:

1) the characteristic cube strength specified for the original design; or,
2) the worst credible in-situ concrete strength.

3.1.1 For structures designed to codes prior to the adoption of the term characteristic strength, the characteristic strength of concrete may be taken as the minimum 28 day works cube strength.

3.1.2 Characteristic concrete strength may be determined from construction records of standard 28-day cube results for concrete that is representative of the location being assessed.

3.1.3 Concrete from before 1939 should be assumed to have a strength no greater than 15 MPa, unless higher strengths are confirmed by testing.

3.1.4 The worst credible strength of concrete in the location of the structure being assessed may be estimated from a statistical analysis of the in-situ strengths obtained from testing a sample of cores.

NOTE 1 Some engineering judgement is involved in selecting an appropriate method of estimation, selecting the number and location of cores, considering whether outliers are genuine and agreeing a representative value for the worst credible strength.

NOTE 2 Concrete testing is most likely to be worthwhile for assessments that are sensitive to the value of concrete strength, for example where the load capacity of the structure is limited by compression or shear in concrete. Flexural resistance in lightly reinforced sections can be insensitive to concrete strength.

3.1.5 Where the worst credible strength for a location in the structure is determined based on core samples taken from the location, the number and location of cores should be selected to be sufficiently representative of each location being assessed.

3.1.6 The number of cores used to estimate worst credible strength for a location should be at least five.

NOTE The accuracy of the estimated strength, and the benefits that can be realised from testing, can be improved significantly by considering a larger number of cores, particularly for methods that are sensitive to the sample standard deviation of the core tests.

3.1.7 The number of cores used to determine worst credible strength for a whole structure should be not less than one core for each 50 m^3 of concrete.

3.1.8 Where a concrete worst credible strength for a location being assessed is estimated from core testing, the following methods may be used:

1) calculation of an estimate of the 5% quantile of the in-situ strength based on a known coefficient of variation for the location being assessed, according to Equation 3.1.8a;
2) calculation of an estimate of the 5% quantile of the in-situ strength based on the sample core test data, according to BS EN 13791 [Ref 2.I];
3) calculation of representative strength parameters using Bayesian analysis with a prior distribution based on strength parameters assumed in the original design and updated based on the data obtained in core testing;
4) for specific verifications of global or ductile failure mechanisms where an estimate of the mean in-situ concrete strength is justified, calculation of a lower bound estimate of the mean in-situ strength based on the sample core test data, according to Equation 3.1.8b.

Equation 3.1.8a Worst credible strength (estimate of 5% quantile) based on an assumed coefficient of variation in the location being assessed.

$$f_{cu} = \frac{\sum f_{cui}}{n}\left(1 - 1.64\left(\frac{1}{n}+1\right)^{0.5} V_P\right)$$

where:

f_{cui} are the values of the in-situ strengths obtained from testing a representative sample of cores, after correction for transverse reinforcement and the length:diameter ratio according to BS EN 12504-1 [Ref 22.I] and the UK National Annex

n is the number of core tests in the sample

V_P is the known coefficient of variation for the concrete strengths in the location being assessed. V_P for concrete strength in a location in a structure may be assumed to be 0.2, or a different value where justified (for example, a higher value can be needed where there is evidence of variable concrete quality within the location)

Equation 3.1.8b Worst credible strength (estimate of mean value) based only on sample core test data.

$$f_{cu} = \frac{\sum f_{cui}}{n} - \frac{t_{0.05}}{\sqrt{n}} s$$

where:

s is the sample standard deviation of the values of f_{cui}

$t_{0.05}$ is a parameter defining the 5% fractile on a one-sided region of a Student's t-distribution statistical curve, which is a function of the sample size n
and can be taken from Table 3.1.8 or standard statistical t-tables for $(n-1)$
degrees of freedom.

Table 3.1.8 Values for t0.05

n	$t_{0.05}$
5	2.13
6	2.02
7	1.94
8	1.89
9	1.86
10	1.83
11	1.81
12	1.80
13	1.78
14	1.77
15	1.76
16	1.75
17	1.75
18	1.74
19	1.73
20	1.73

NOTE 1 *The coefficient of variation for the strength of the concrete in the structure can be significantly larger than the coefficient of variation of contemporary cube tests, because it can depend on other factors including variable compaction quality and the possibility of concrete from multiple batches being used within the location being assessed.*

NOTE 2 *The sample standard deviation is based on the* $(n-1)$ *method.*

NOTE 3 *Methods of assessing the estimated in-situ strength at a location are given in BS EN 13791 [Ref 2.I].*

NOTE 4 *Equation 3.18a is consistent with Annex D of BS EN 1990 [Ref 6.I].*

NOTE 5 *The effect of the different curing conditions on the strength of standard test cubes relative to in-situ concrete is accounted for in the lower partial factor for concrete for use with worst credible strength. Worst credible strength is therefore based on in-situ strength, and the core values are not increased to convert back to an equivalent cube strength value corresponding to the curing conditions for standard test cubes.*

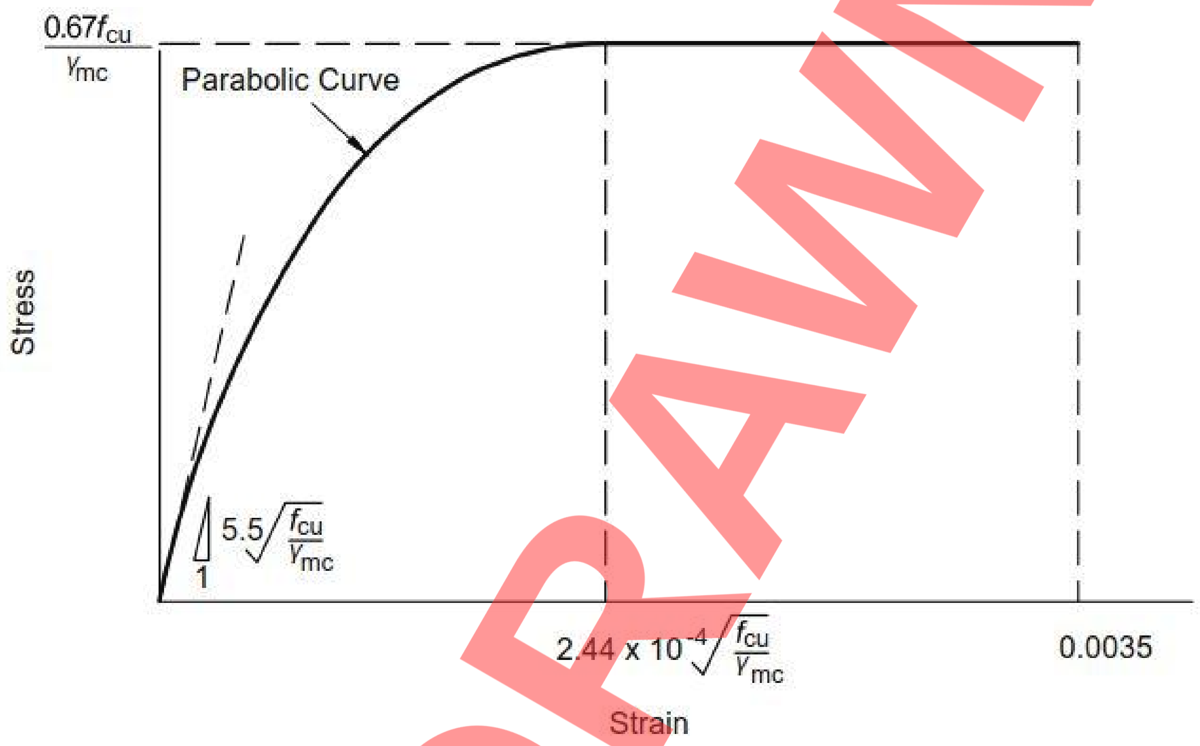
3.2 The method of determining the worst credible strength from core testing shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

3.3 Where the removal of cores is likely to reduce the load carrying capacity, the proposal for removal shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

3.4 The compressive stress-strain curve for concrete in flexure or compression shall be defined and combined with the value of γ_{mc} defined in Section 2.

3.4.1 The short term stress-strain curve for normal weight concrete in flexure or compression should be based on Figure 3.4.1.

Figure 3.4.1 Short term assessment stress-strain curve for normal weight concrete



NOTE 1 *In Figure 3.4.1, f_{cu} is in units of MPa.*

NOTE 2 *In Figure 3.4.1, The equation for the parabolic curve between $\epsilon = 0$ and $\epsilon = 2.44 \cdot 10^{-4}\sqrt{\frac{f_{cu}}{\gamma_m}}$ is described in Equation 3.4.1N2*

Equation 3.4.1N2 Parabolic stress strain curve for compression in concrete

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

where:

σ is the concrete compressive stress in units of MPa

ϵ is the concrete compressive strain.

3.5 The short-term modulus of elasticity for normal-weight concrete and lightweight aggregate concrete shall be taken from Equations 3.5a or 3.5b respectively, or based on test data.

Equation 3.5a Short term modulus of elasticity for normal weight concrete

$$E_{cm} = 20 + 0.27 f_{cu}$$

Equation 3.5b Short term modulus of elasticity for lightweight aggregate concrete

$$E_{cm} = \left(\frac{D_c}{2300}\right)^2 (20 + 0.27 f_{cu})$$

where:

D_c is the density of the lightweight aggregate concrete, in units of kg/m^3

NOTE 1 In Equations 3.5a and 3.5b, E_{cm} is in units of GPa, and f_{cu} is in units of MPa.

NOTE 2 Equation 3.5b is valid for values of D_c between 1400 and 2300 kg/m^3.

3.5.1 The effect of creep under permanent actions may be approximated by using half the short-term modulus of elasticity to analyse the effects of permanent actions.

3.5.2 To determine the effect of imposed deformations, calculate deflection and determine crack widths and stresses due to short and long term actions and imposed deformations, an intermediate value between the short-term modulus and half its value may be used.

3.6 The value of Poisson's ratio shall be taken as 0.2, or based on test data.

3.7 The value for the thermal coefficient of concrete shall be obtained from CS 454 [Ref 2.N] or based on test data.

Reinforcement and prestressing steel

3.8 Reinforcement and prestressing steel strengths shall be based on either:

1) characteristic yield strength or proof stress; or,
2) worst credible yield strength.

NOTE Where the worst credible strength is greater than the characteristic strength assumed for design, the stresses can be limited by the provided lap lengths and anchorage lengths, which might not be sufficient to develop yield.

3.8.1 For structures designed to codes prior to the adoption of the term characteristic strength, the characteristic yield strength may be taken as the guaranteed yield strength.

3.8.2 The characteristic yield strength of steel reinforcement produced before 1961 should be assumed to be not greater than 230 MPa unless higher strengths are confirmed by testing.

3.8.3 The characteristic yield strength of steel reinforcement produced after 1961 may be taken from the standards and specifications that were in use at the time of the design.

NOTE The strength of steel reinforcement after 1961 varied depending on the standards and specifications in use at the time and the grade of the steel.

3.8.4 The characteristic strength of prestressing tendons dating from before 1955 should be taken from documents of the period.

NOTE 1 The characteristic strength of prestressing tendons was first specified by the British Standards Institution in 1955.

NOTE 2 Information regarding prestressing from before 1955 can be obtained from the Institution of Structural Engineers 'First Report on Prestressed Concrete' ISE Prestressed [Ref 8.I].

3.8.5 The worst credible strength of reinforcement or prestressing steel of a particular size and type, and originating from a single batch, may be estimated from a statistical analysis of the strengths obtained from testing a representative sample of steel specimens extracted from the structure.

NOTE 1 The strengths of different size and types of steel component can have significantly different distributions.

NOTE 2 *As it is not usually known which reinforcement or prestressing steel originates from a single batch, there can be a need to investigate the possibility of multiple batches being used within the location being assessed. This can involve a larger number of specimens being tested and some engineering judgement to be used regarding the data analysis and the scope of application of the resulting strengths.*

3.8.6 Where the worst credible strength for reinforcement or prestressing steel is estimated from the strengths of a sample of test specimens, the number, location and size of the test specimens should be selected to be representative of the components being assessed.

NOTE *Testing of steel reinforcement and prestressing strand is covered by BS EN ISO 15630-1 [Ref 17.I] and BS EN ISO 15630-3 [Ref 16.I], respectively.*

3.8.7 Where the worst credible strength for reinforcement or prestressing steel is calculated based on sample testing, the value of the yield strength may be based on the lower bound to the mean yield strength according to Equation 3.8.7.

Equation 3.8.7 Worst credible strength of reinforcement or prestressing steel

$$f_y = \frac{\sum f_{yi}}{n} - t_{0.01}\frac{s}{\sqrt{n}}$$

where:

f_{yi} are the values of yield strength obtained from sample tests of the bar size and type being assessed

n is the number of tests in the sample

s is the sample standard deviation of the values of f_{yi}

$t_{0.01}$ is a parameter defining the 1% fractile on a one-sided region of a Student's t-distribution statistical curve, which is a function of the sample size n and can be taken from Table 3.8.7 or standard statistical t-tables for $(n-1)$ degrees of freedom

Table 3.8.7 Values for t0.01

n	$t_{0.01}$
5	3.75
6	3.36
7	3.14
8	3.00
9	2.90
10	2.82
11	2.76
12	2.72
13	2.68
14	2.65
15	2.62
16	2.60
17	2.58
18	2.57
19	2.55
20	2.54

NOTE 1 *Equation 3.8.7 provides a lower bound estimate of the mean yield strength of reinforcement or prestressing steel.*

NOTE 2 *The use of an estimate of the mean yield strength for the worst credible strength of reinforcement and prestressing steel is appropriate for ductile steel components that exhibit strain hardening, and for failure mechanisms that typically involve many bars.*

NOTE 3 *The sample standard deviation is based on the* $(n-1)$ *method.*

3.9 Where the removal of test samples of reinforcement or prestressing steel is proposed, an assessment of the effect of the removal of the test samples on the structure load-carrying capacity shall be carried out.

NOTE *The removal of prestressing steel for sampling purposes is likely to alter the stress distribution in the concrete section.*

3.10 Where the removal of test specimens from the structure is likely to reduce the load carrying capacity, the proposal for removal shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

NOTE *It is often impractical to extract test specimens from critical sections.*

3.11 In accessing reinforcement and prestressing steel for testing, methods of concrete removal that minimise risk of mechanical damage to the retained steel shall be used.

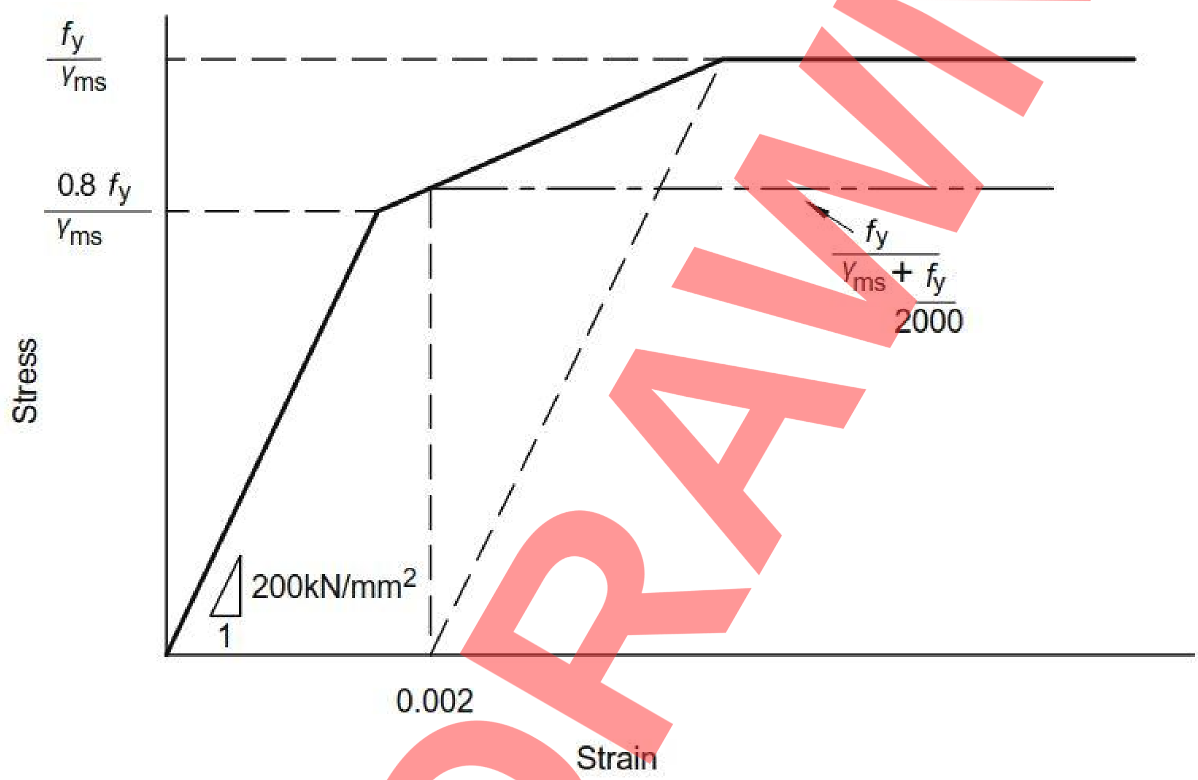
3.12 When cutting out the chosen test specimens, adjacent reinforcement and prestressing steel shall not be damaged.

3.13 The stress-strain curve for reinforcement shall be defined for assessment purposes, and combined with the value of γ_{ms} defined in Section 2.

3.13.1 The short-term stress-strain curve for reinforcement may be taken from:

1) Figure 3.13.1;
2) either of the models in BS EN 1992-2 [Ref 3.N] where the reinforcement is high-yield steel and is known to exceed or comply with the Class B ductility requirements of BS EN 1992-2 [Ref 3.N];
3) manufacturer's data; or,
4) test data.

Figure 3.13.1 Short-term assessment stress-strain curve for reinforcement



NOTE *The stress strain models in BS EN 1992-2 [Ref 3.N] for high-yield steel include a model with a rising branch to include the effects of strain-hardening.*

3.13.2 The modulus of elasticity for reinforcement should be taken as 200 GPa or based on test data.

3.14 The stress-strain curve for prestressing steel shall be defined for assessment purposes, and combined with the value of γ_{ms} defined in Section 2.

3.14.1 The short-term stress-strain curve for prestressing steel may be taken from:

1) Figure 3.14.1a for normal and low relaxation products;
2) Figure 3.14.1b for as-drawn wire and as-spun strand;
3) manufacturer's data; or,
4) test data.

Figure 3.14.1a Short-term assessment stress-strain curve for normal and low relaxation products

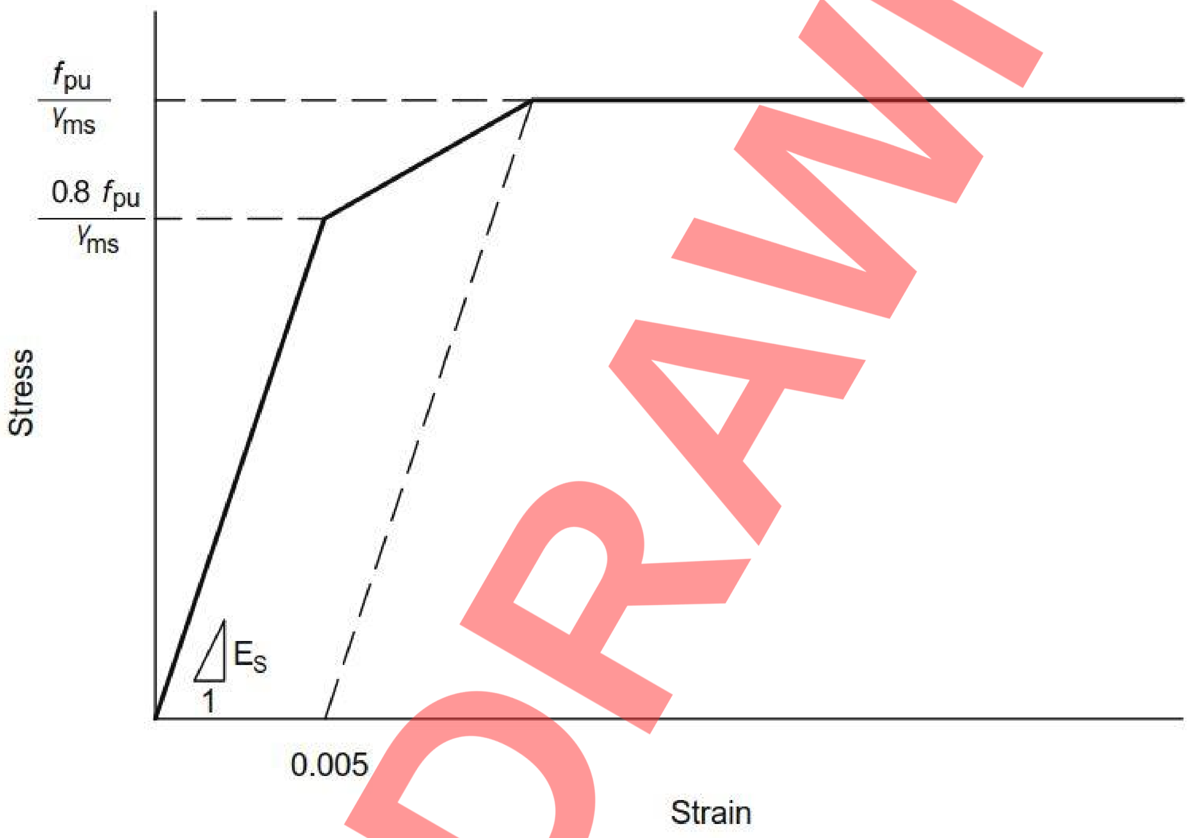
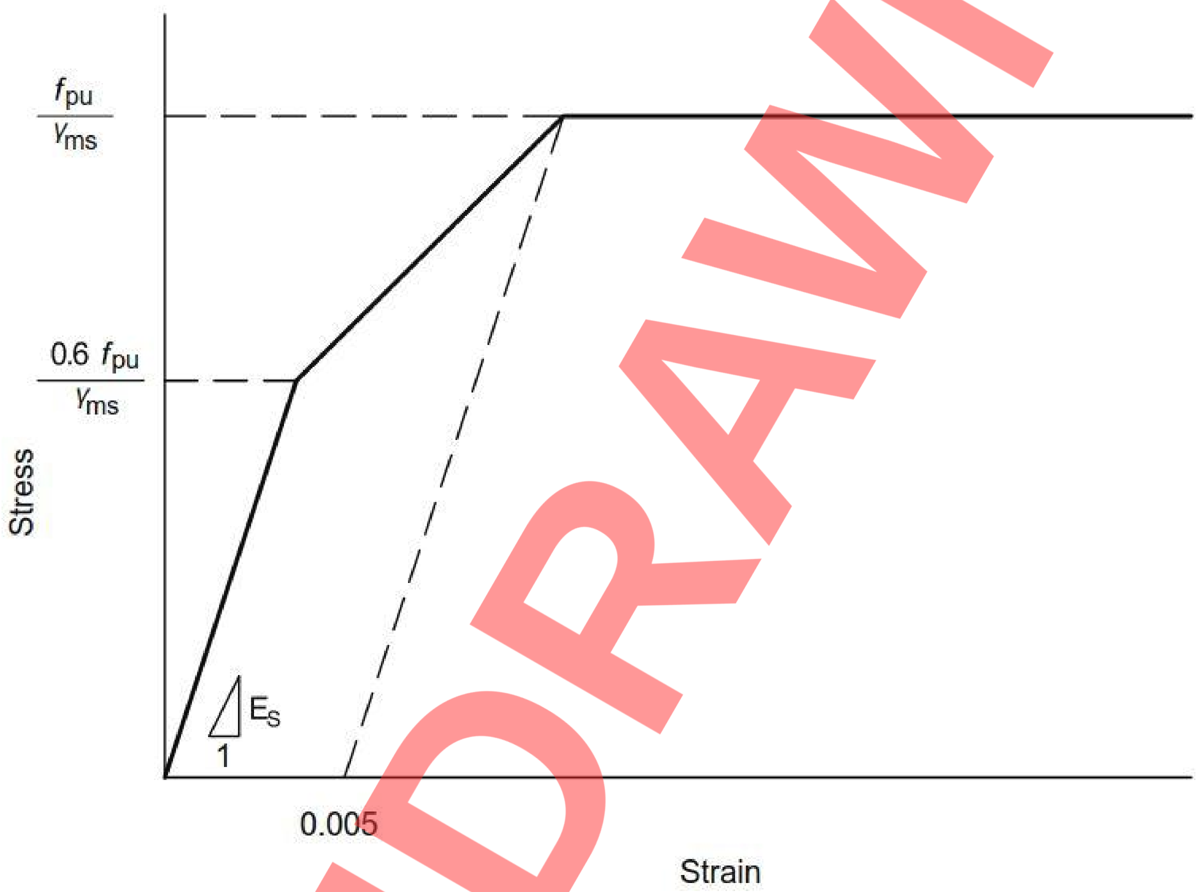


Figure 3.14.1b Short-term assessment stress-strain curve for as drawn wire and as spun strand



3.14.2 In Figure 3.14.1a, E_s should be taken as:

1) 200 GPa for wire and strand to BS 5896 [Ref 9.I] Sections 2 and 3; and,
2) 165 GPa for alloy bars to BS 4486 [Ref 14.I] and 19-wire strand to BS 4757 [Ref 15.I] Section 3.

3.14.3 In Figure 3.14.1b, E_s should be taken as:

1) 200 GPa for wire to BS 5896 [Ref 9.I]; and,
2) 175 GPa for 19-wire strand to BS 4757 [Ref 15.I] Section 2.

3.14.4 The modulus of elasticity for prestressing steel should be taken from Figure 3.14.1a for normal and low relaxation products, or from Figure 3.14.1b for as-drawn wire and as-spun strand, based on the tangent modulus at zero load, or based on manufacturer's data.

4. Structural analysis

Analysis method

4.1 The distribution and magnitude of action effects throughout the structure shall be determined using a structural analysis method that complies with the requirements of CS 454 [Ref 2.N] and the additional requirements of this section.

NOTE 1 Methods of analysis can include:

1) *elastic analysis;*
2) *elastic analysis with redistribution of moments;*
3) *analysis of compressive membrane action;*
4) *plastic analysis; and,*
5) *non-linear finite element analysis.*

NOTE 2 The requirements for each method are given in the following subsections.

Elastic analysis

4.2 When elastic analysis is used, the flexural stiffness constants shall be selected for the analysis model within representative limits that are consistent with the structural behaviour at the limit state being assessed.

4.2.1 When elastic analysis is used, the flexural stiffness constants may be based on any of:

1) uncracked concrete section: the entire concrete section, ignoring the presence of reinforcement;
2) uncracked gross transformed section: the entire concrete section, including the reinforcement transformed on the basis of modular ratio;
3) cracked transformed section: the area of the concrete section that is compression, together with the tensile reinforcement transformed on the basis of modular ratio; or,
4) any intermediate value between the values in 1), 2) and 3).

4.2.2 When elastic analysis is used, the axial and shear stiffness constants should be based on the uncracked concrete section.

4.2.3 When elastic analysis is used, the torsional stiffness constant maybe based on the:

1) uncracked concrete section; or,
2) for cracked sections, reduced to a lower value in proportion to the ratio of cracked and uncracked flexural stiffness.

4.2.4 Where the structural form is such that the behaviour can be characterised by components of bending and shear, without reliance on torsion for equilibrium, the structural analysis for ULS may be carried out assuming an idealised torsionless system of flexural members.

NOTE For the analysis of structures where the behaviour is judged to be strongly dependent on torsion or non-ductile failure mechanisms, a torsionless analysis can underestimate the resistance.

4.3 Where the assessment of the limit state being assessed is sensitive to the construction sequence, the analysis of composite structures shall include the effects of the construction sequence.

4.3.1 At the ULS, where the cross-section of composite members and the applied loading have increased by stages, the entire load may be assumed to act on the final cross-section.

4.4 The assessment of prestressed structures shall include the secondary effects of prestress.

Elastic analysis with redistribution of moments

4.5 Where the analysis includes redistribution of moments at ULS, the capacity of the structure to redistribute the moments shall be assessed.

4.5.1 Redistribution of moments obtained by elastic analysis may be carried out at ULS where any of the following apply:

1) there is sufficient rotation capacity at the sections where the elastic moments are redistributed from, based on Equation 4.5.1, except where structures contain external or unbonded tendons;
2) the requirements in BS EN 1992-1-1 [Ref 5.I] for linear elastic analysis with limited moment redistribution are satisfied; or,
3) the redistribution can be justified based on a special investigation.

Equation 4.5.1 Rotation capacity for reinforced concrete

$$\theta_P = \min\begin{cases} \frac{\psi}{d-d_c} \\ 0.008 + 0.035\left(0.5 - \frac{d_c}{d_e}\right) \end{cases}$$

where:

θ_P is the plastic rotation capacity, in radians (θ_P is not taken as less than zero)

ψ is taken as 0.6 times the diameter of the smallest tensile reinforcing bar for reinforced concrete, or 10 mm for prestressed concrete

d is the effective depth to the tension reinforcement

d_c is the depth in compression at the ULS

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

4.6 Where redistribution of moments is carried out, the changes in transverse moments, shear forces and reactions associated with the redistribution shall be analysed and assessed.

4.7 Where redistribution of moments is carried out, the shear forces and reactions shall be taken as the greater of the values before or after the redistribution.

4.8 Where linear analysis is performed, both primary and secondary effects of prestress shall be applied before any redistribution of moments is calculated.

Analysis of compressive membrane action

4.9 Where analysis of compressive membrane action is proposed, the analysis shall be carried out according to CD 360 [Ref 9.N].

Plastic analysis

4.10 Where plastic analysis is proposed for ULS assessment, the method shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

NOTE 1 Plastic analysis methods can include:

1) Equilibrium-based analyses that provide a conservative lower bound to the resistance of the structure (lower-bound analysis methods);
2) Collapse mechanism analyses that provide an upper bound to the resistance of the structure (upper-bound analysis methods).

NOTE 2 It can be unsafe to rely on upper-bound analysis as the primary assessment method, particularly where there is not certainty that the critical mechanism has been identified.

NOTE 3 The combination of lower bound and upper bound analyses can be a powerful way of confirming the accuracy of the estimated resistance. If there is a significant difference between lower bound and upper bound estimates, there is usually scope to refine the estimates through further optimisation.

4.11 Where upper-bound collapse mechanism analyses are proposed, the assessment shall determine the lowest value of the collapse load from an analysis of all possible failure mechanisms.

NOTE 1 Yield-line analysis is an example of an upper-bound collapse mechanism analysis for flexural assessment of slabs. Even though it is an upper-bound method, there is usually some conservatism in a fully optimised yield-line analysis, because the effects of compressive membrane action are generally neglected. Software that automatically considers all flexural mechanisms is widely available.

NOTE 2 Flexural yield-line analysis does not provide an assessment of shear resistance.

Non-linear analysis

4.12 Where non-linear analysis is proposed for ULS assessment, the method and the parameters assumed for the material model shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

4.13 Where there is uncertainty in the values of the parameters needed for non-linear analysis, the sensitivity of the analysis to the parameters shall be assessed.

4.14 Where a non-linear finite element analysis initially indicates that a structure or component is resisting loads at the ultimate limit state primarily due to tension in uncracked concrete, the sensitivity of the assessment to cracking shall be analysed, based on cautious assumptions for the tensile post-cracking behaviour, in order for the resistance following cracking to be assessed.

NOTE For some structures the assessed capacity can be significantly lower after the development of cracking. Cracks can develop for a variety of reasons, not solely due to the assessed loading, and in some cases can result in a collapse disproportionate to the original cause.

Effective spans and effective widths

4.15 The effective span of members shall be based on the requirements of CS 454 [Ref 2.N].

4.15.1 The effective span of continuous beams should be taken to the mid-points of the supports.

4.15.2 Where the hogging moment in a continuous beam is calculated based on a point support, and where the support is assumed to provide no restraint to rotation, the peak hogging moment may be reduced according to Equation 4.15.2.

Equation 4.15.2 Hogging moment reduction at supports

$$\Delta M = R\frac{t}{8}$$

where

ΔM	is the reduction in the hogging moment
R	is the support reaction
t	is the breadth of the bearing area over a support

4.15.3 The effective span of concrete cantilevers should be the length of the cantilever to the face of the support plus half the effective depth.

4.16 The effective width of flanges in the analysis shall depend on the limit state being assessed.

4.16.1 At the ULS, the full width of flanges may be assumed to be effective in the structural analysis.

4.16.2 At the SLS, the effective width of flanges may be either:

1) taken as the web width plus one tenth of the distance between points of zero moment, on each side of the web, but no greater than the full width of the flange; or,
2) based on a 3-D finite element analysis.

Slenderness for lateral stability verifications

4.17 Simply supported and continuous beams shall be analysed and assessed for lateral stability verifications when the clear distance between lateral restraints exceeds L_{Rb} in Equation 4.17.

Equation 4.17 Slenderness for lateral stability verifications in beams

$$L_{Rb} = 300\frac{b_c^2}{d}$$

where:

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

4.18 Cantilever beams shall be analysed and assessed for lateral stability verifications when the clear distance from the end of the cantilever to the face of the support exceeds L_{Rc} in Equation 4.18.

Equation 4.18 Slenderness for lateral stability verifications in cantilevers

$$L_{Rc} = 150\frac{b_c^2}{d}$$

where:

b_c is the breadth of the compression face at the end of the cantilever

4.19 Columns shall be analysed with a non-linear analysis where their slenderness exceeds the criterion in Equation 4.19.

Equation 4.19 Slenderness requiring non-linear analysis of columns

$$\frac{l_e}{h} > 60$$

NOTE Analysis of columns where $\frac{l_e}{h} \leq 60$ *is covered in Section 7.*

5. Reinforced concrete beams

Moment resistance of beams

5.1 The moment resistance of a cross section shall be calculated based on the following assumptions:

1) the strain distribution in the concrete and bonded reinforcement is based on the assumption that plane sections remain plane;
2) the compressive strain is limited to no greater than 0.0035;
3) the compressive stresses in the concrete are derived from either the compressive strain according to Section 3, or, where the compression zone is rectangular in cross section, a constant stress of $0.6f_{cu}/\gamma_{mc}$ in the compression zone;
4) the tensile stress in the concrete is zero; and,
5) the stresses in the reinforcement are derived from the strain in the reinforcement according to Section 3, but no greater than the stresses developed through bond, anchorage and bearing according to Section 9.

5.2 The moment resistance for beams shall be calculated from an analysis of the stresses in the cross-section based on a neutral axis position that satisfies the equilibrium of forces.

5.2.1 The effects of a net axial compressive force of less than $0.1f_{cu}A$ may be ignored.

5.2.2 The moment resistance may be calculated using Equation 5.2.2a for sections without compression reinforcement, or Equation 5.2.2c for sections with compression reinforcement, where all of the following apply:

1) there is zero net axial force;
2) the compression zone is rectangular in cross section; and,
3) the reinforcement has been detailed with sufficient anchorage to allow yield to be developed.

Equation 5.2.2a Moment resistance for beams without compression reinforcement)

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$$

where:

M_u	is the moment resistance
f_y	is the characteristic or worst credible strength of the reinforcement
γ_{ms}	is the partial factor for the reinforcement strength
A_s	is the area of tension reinforcement
z	is the lever arm, determined from Equation 5.2.2b, but no greater than $0.95d$
f_{cu}	is the characteristic or worst credible cube strength of the concrete
γ_{mc}	is the partial factor for the concrete strength
b	is the width of the section in compression at the level of the neutral axis
d	is the effective depth of the tension reinforcement from the extreme fibre in compression

Equation 5.2.2b Lever arm for beams without compression reinforcement

$$z = \left(1 - \frac{0.84\frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} bd}{\gamma_{mc}}}\right) d$$

Equation 5.2.2c Moment resistance for beams with compression reinforcement

$$M_u = \frac{0.6 f_{cu}}{\gamma_{mc}} bx(d - 0.5x) + f_s^{'} A_s^{'} \left(d - d^{'}\right)$$

where:

$A_s^{'}$	is the area of compression reinforcement
$d^{'}$	is the depth to the compression reinforcement from the extreme fibre in compression
$f_s^{'}$	is equal to $\frac{f_y}{\gamma_{ms} + \frac{f_y}{2000}}$
x	is the neutral axis depth, which may be determined from Equation 5.2.2d

Equation 5.2.2d Force equilibrium at ULS for beams with zero net axial force

$$\frac{f_y}{\gamma_{ms}} A_s = \frac{0.6 f_{cu}}{\gamma_{mc}} bx + f_s^{'} A_s^{'}$$

5.2.3 Where $d' > 0.429x$, the compression reinforcement should be ignored and the section is to be treated as singly reinforced.

5.2.4 For flanged beams where the flange is in compression and the neutral axis extends below the bottom of the flange, the resistance moment may be taken from Equation 5.2.4.

Equation 5.2.4 Moment of resistance for flanged beams

$$M_u = \min \begin{cases} \frac{f_y}{\gamma_{ms}} A_s \left(d - \frac{h_f}{2}\right) \\ \frac{0.6 f_{cu}}{\gamma_{mc}} b h_f \left(d - \frac{h_f}{2}\right) \end{cases}$$

where:

h_f	is the depth of the flange

5.3 Reinforcing bars in compression shall not be taken to develop the full compressive yield strength in the calculation of moment resistance unless the reinforcing bars are:

1) effectively restrained by links; or,
2) within 150 mm of a bar that is effectively restrained by links.

5.3.1 Compression bars may be taken to be effectively restrained where:

1) links are present that have a diameter at least one-quarter the size of the largest compression bar, at a spacing no greater than 12 times the size of the smallest compression bar; and,
2) every corner and alternate bar or group in an outer layer of reinforcement is restrained by a link passing round the bar and having an included angle of not more than 135°.

5.3.2 Where a bar in compression is not effectively restrained by links, the compressive strength for that bar should be either:

1) taken as zero; or,

2) reduced in proportion to the ratio of the actual link area to the minimum link area that is needed to effectively restrain the bar.

Shear resistance of beams

5.4 The shear resistance of beams shall be calculated according to the requirements in the following subsections:

1) shear resistance of beams without shear reinforcement; and,
2) shear resistance of beams with shear reinforcement.

5.4.1 In a haunched beam, the component of the flange force perpendicular to the longitudinal centroidal axis of the beam, calculated from an elastic section analysis under the relevant load case, may be subtracted from the applied shear force, and the resultant shear limited to the shear resistance ignoring the presence of the haunch.

5.4.2 Where significant axial compressive forces exist, the ultimate shear resistance may be enhanced by using the corresponding requirements for columns presented in Section 7.

5.5 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.

Shear resistance of beams without shear reinforcement

5.6 The assessment shear force shall not exceed:

1) $V_{\max}$ as defined in Equation 5.6a, anywhere;
2) V_{uc} as defined in Equation 5.6b, in regions more than $3d$ from a support; and,
3) V_u as defined in Equation 5.6c within $3d$ of a support.

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as
$\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement
$\rho_s = \frac{100 A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \begin{cases} \frac{3d}{a_v} \Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}} \xi_s (0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

NOTE 1 *The form of the Γ factor is derived from Denton et al 2007 [Ref 12.I] .*

NOTE 2 *For beams with $\Gamma < 1$ there is a step in the shear resistance at $a_v = 3d$. This is related to the difference in the resistance associated with:*

1) a shear failure mechanism at the support, limited by anchorage at the support; and,

2) a shear failure mechanism away from a support, not involving anchorage at the support.

NOTE 3 *The assessment of sections of beams with short anchorage lengths can be limited by the flexural resistance.*

5.6.1 For lightweight aggregate concrete, $V_{\max}$ should be taken as the value for normal weight concrete in Equation 5.6a multiplied by 0.8.

5.6.2 For lightweight aggregate concrete V_{uc} should be taken as the value for normal weight concrete in Equation 5.6b multiplied by 0.9.

5.6.3 For internal supports of continuous structures Γ may be taken as 1.0.

NOTE The assessment of internal supports with short anchorage lengths can be limited by the flexural resistance.

Shear resistance of beams with shear reinforcement

5.7 Where the provided shear reinforcement is not effective in resisting shear, the shear resistance shall be calculated using the requirements for beams without shear reinforcement.

NOTE Shear reinforcement can take the form of vertical links, inclined links or bent-up bars.

5.7.1 Shear reinforcement should be assessed to be effective in resisting shear if all of the following are satisfied:

1) the spacing of the legs of links, in the direction of the span and at right angles to it, does not exceed the effective depth, d ;
2) the minimum shear reinforcement criterion provided in Equation 5.7.1 is fulfilled;
3) the angle of the shear reinforcement from the longitudinal axis of the beam $\alpha \geq 30^o$; and,
4) the shear reinforcement satisfies the anchorage and bearing requirements of Section 9.

Equation 5.7.1 Minimum effective shear reinforcement

$$\frac{A_{sv}}{s_v b_w}(\sin\alpha + \cos\alpha)\left(\frac{f_{yv}}{\gamma_{ms}}\right) \geq 0.2\text{MPa}$$

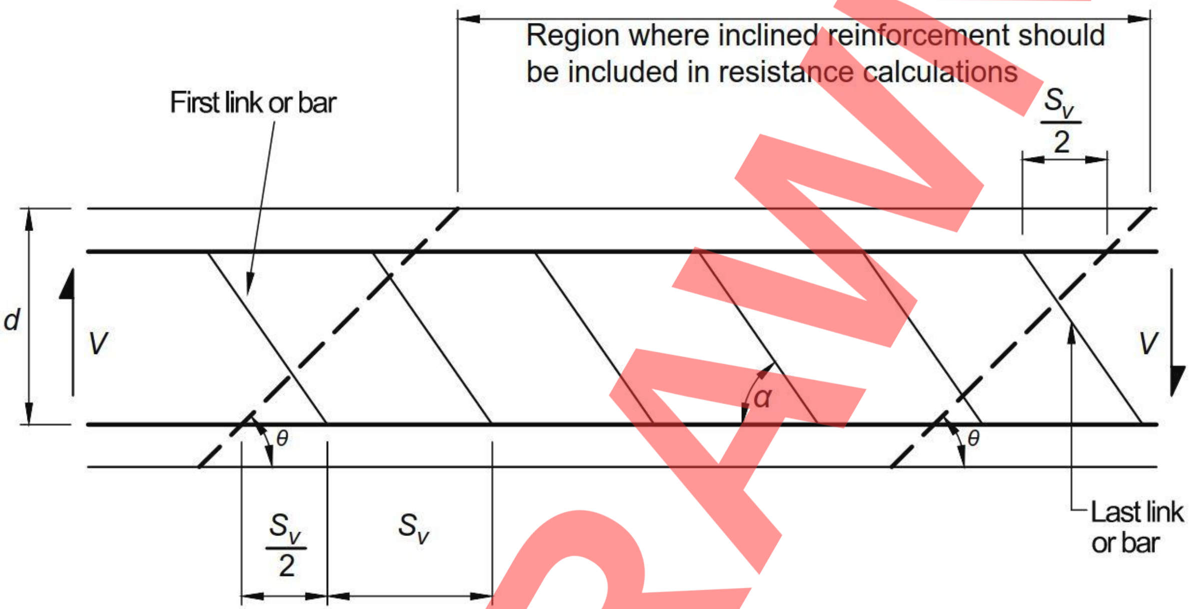
where

A_{sv}	is the cross-sectional area of shear reinforcement at a particular cross-section
α	is the angle of the shear reinforcement from the longitudinal axis of the beam
s_v	is the spacing of the shear reinforcement along the member
f_{yv}	is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 MPa
b_w	is the breadth of the cross-section

5.8 Where the provided shear reinforcement is effective in resisting shear, the shear resistance shall be calculated using the requirements for beams with shear reinforcement.

5.8.1 Where there is effective inclined shear reinforcement over part of a beam, the inclined shear reinforcement should be included in the calculation of shear resistance for sections within the region of the beam illustrated in Figure 5.8.1.

Figure 5.8.1 Inclined shear reinforcement



NOTE *In Figure 5.8.1, Angle θ is taken as 45º when using the additive approach and selected directly when using the variable angle truss approach.*

5.8.2 The shear resistance of sections that include effective shear reinforcement should be determined using one of the following methods:

1) additive approach;
2) variable angle truss approach;
3) strut and tie approach.

NOTE *The requirements for each approach are given in the respective subsections.*

Additive approach to shear resistance

5.9 When using the additive approach for calculating the shear resistance of a beam with effective shear reinforcement, the assessment shear force shall not exceed:

1) $V_{\max}$ as defined in Equation 5.6a, anywhere;
2) V_u as defined in Equation 5.9a, in regions more than $3d$ from a support;
3) V_u as defined in Equation 5.9c within $3d$ of a support.

Equation 5.9a Shear resistance more than 3d from a support

$$V_u = V_{uc} + V_{us}$$

where:

V_{uc} is calculated using Equation 5.6b

V_{us} is the component of shear resistance provided by effective shear reinforcement, as defined in Equation 5.9b

Equation 5.9b Component of shear resistance provided by effective shear reinforcement

$$V_{us} = d\frac{A_{sv}}{s_v}(\sin\alpha + \cos\alpha)\frac{f_{yv}}{\gamma_{ms}}$$

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

where:

Γ is the reduction factor to account for short anchorage lengths, as defined in Equation 5.6d

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

5.9.1 When using the additive approach in regions more than 3d from an end support, V_{us} should be taken as zero where equation 5.9.1 is not satisfied.

Equation 5.9.1 Longitudinal capacity of steel

$$A_s\frac{f_y}{\gamma_{ms}} \geq \frac{M}{z} + \frac{V - V_{uc}}{2}(1 - \cot\alpha)$$

Variable angle truss approach

5.10 Where the variable angle truss approach is used to assess the shear resistance of beams with effective shear reinforcement, the resistance shall be assessed using the requirements for the design shear resistance of members with designed shear reinforcement in BS EN 1992-2 [Ref 3.N].

5.10.1 The characteristic concrete cylinder strength used in BS EN 1992-2 [Ref 3.N] should be estimated from one of the following:

1) Equation 5.10.1;
2) the relationship between cube strength and cylinder strength in BS EN 206 [Ref 3.I] ; or,
3) derived from testing.

Equation 5.10.1 Approximate relationship between cylinder strength and cube strength

$$f_{ck} \approx 0.8 f_{cu}$$

where:

f_{ck} is the characteristic cylinder strength of the concrete

5.10.2 Where the variable angle truss approach is used, the partial factors for material properties should be taken from Section 2, except that γ_{ms} for shear reinforcement should not be less than 1.15.

NOTE *The background to the restriction on γ_{ms} for shear reinforcement can be found in Beeby & Jackson 2016 [Ref 11.I].*

5.10.3 When the variable angle truss approach is used, the truss angle should be varied within the limits permitted in BS EN 1992-2 [Ref 3.N] to find a solution where all of the following are simultaneously satisfied:

1) the assessment shear force does not exceed the maximum shear corresponding to concrete crushing;

2) the assessment shear force does not exceed the resistance corresponding to yielding of the shear reinforcement; and,
3) the tension in the longitudinal tension reinforcement due to the combination of bending and shear does not exceed the tensile resistance of the reinforcement, either in yielding or in bond.

5.10.4 The tension in the longitudinal tension reinforcement may be limited to the tensile resistance by either:

1) satisfying the corresponding shear force limit given in equation 5.10.4; or,
2) verifying that the tension reinforcement is capable of resisting the maximum bending moment in any section within a distance $\frac{z(\cot\theta - \cot a)}{2}$ in the direction of increasing moment from the section and provided with an effective anchorage in accordance with Section 9.

Equation 5.10.4 Shear resistance limit to account for tension in the longitudinal reinforcement

$$V_u < \frac{2\left(F_{ub} - \frac{M}{z}\right)}{\cot\theta - \cot\alpha}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

Strut and tie approach

5.11 Where the strut and tie approach is used to assess the shear resistance of parts of members with effective shear reinforcement, the resistance shall be assessed using the requirements for design based on strut and tie models in BS EN 1992-2 [Ref 3.N].

NOTE 1 Strut and tie models can be particularly useful for assessing discontinuity regions that have non-uniform geometry (e.g. half joints), or variable reinforcement arrangements.

NOTE 2 Further guidance on the use of strut and tie methods for half joints is provided in CS 466 [Ref 6.N].

NOTE 3 Strut and tie models provide lower bound solutions for the shear resistance. Adjustments to the arrangement of struts and ties can significantly improve the estimated resistance, particularly when the assessment is limited by a local deficiency.

5.11.1 The strut and tie approach should include the following steps:

1) idealising the part of the structure as a series of struts and ties;
2) analysing the forces in the struts and ties;
3) verifying the assessment stress limits are not exceeded; and,
4) adjusting or optimising the model if needed.

5.12 The form of the strut-and-tie model shall be derived from an approximation of the anticipated behaviour and principal stress directions at the ultimate limit state, including the effects of deterioration.

NOTE The form of the model can be influenced by: observations of cracking and deterioration; other analyses (e.g. non-linear finite element analysis, upper bound analyses); considerations of weaknesses in the design and detailing; and engineering judgement.

5.12.1 The location of the ties should align with the location of the existing reinforcing bars or groups of reinforcing bars that are assumed to contribute to the tie resistance.

5.12.2 Where multiple reinforcing bars are represented by a single tie member, all the contributing bars should lie within the geometry of the respective nodes at the ends of the tie member.

NOTE Increasing the size of the node and the connecting struts, where there is space to do so, can allow a greater number of bars to be included.

5.13 The struts and nodes shall be positioned to fit completely within the geometry of the existing concrete structure when drawn to scale.

5.13.1 Overlapping struts should be combined and replaced by a statically equivalent system of non-overlapping struts and nodes.

5.13.2 The struts and nodes should be located so that the angle between struts and ties that connect at a node exceeds 25 degrees.

NOTE *The minimum size of the struts and nodes is limited by the respective stress limitations.*

5.14 Where a node connects with a tie, the node shall be located to allow a sufficient anchorage length for the tie force to be developed and transmitted into the node.

5.14.1 The length of the bar within the node may be included in the anchorage length.

5.15 The strut and tie model shall be developed to satisfy the stress limits for the concrete struts and nodes given in BS EN 1992-2 [Ref 3.N] and to limit the forces in tie members to no greater than the resistance corresponding to either:

1) yielding of the fully anchored bars included in the tie member; or,
2) the anchorage force that can be developed in the bars included in the tie member at the node according to the anchorage, bond and bearing requirements of Section 9.

NOTE *The concrete cylinder strength in BS EN 1992-2 [Ref 3.N] is defined in the previous subsection on the variable angle truss approach.*

5.16 Where the strut and tie approach is used, the partial factors for material properties shall be taken from Section 2.

Torsion resistance in beams

Maximum shear and torsion limited by concrete crushing

5.17 The sum of the shear stresses resulting from shear force and torsion shall not exceed the maximum shear stress corresponding to crushing of the concrete, as in Equation 5.17a.

Equation 5.17a Maximum combined shear and torsional stresses

$$v + v_t \leq v_{\max}$$

where:

$v = \frac{V}{b_w d}$	is the shear stress from the applied shear force
v_t	is the torsional shear stress defined in Equation 5.17.b for box sections and Equation 5.17c for rectangular sections
$v_{\max} = \frac{V_{\max}}{b_w d}$	is the maximum shear stress corresponding to concrete crushing, with $V_{\max}$ defined by Equation 5.6a

Equation 5.17b Torsional shear stress in box sections

$$v_t = \frac{T}{2h_w A_o}$$

where:

T	is the torque due to ultimate loads
h_w	is the thickness of the thinnest wall
A_o	is the area enclosed by the median wall line

Equation 5.17c Torsional shear stress in rectangular sections

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

where:

$h_{\min}$	is the smaller dimension of the section
$h_{\max}$	is the larger dimension of the section

5.18 Where the assessment of torsional resistance includes the contribution of links that have a maximum centre-line dimension of less than 550mm, the torsional shear stress shall not exceed the value in Equation 5.18.

Equation 5.18 Maximum torsional shear stress in small sections with links

$$v_t \leq v_{\max} \frac{y_1}{550}$$

where:

y_1	is the maximum centre-line dimension of the links, in mm

Effective torsional reinforcement

5.19 Torsional resistance shall include the contribution of effective torsion reinforcement, comprising:

1) rectangular closed links; and,
2) longitudinal reinforcement.

5.20 Effective torsional reinforcement shall not include reinforcement that is required to resist coexistent shear and bending.

5.20.1 Effective torsional reinforcement should be calculated based on the area of reinforcement in excess of that required to resist shear or bending.

5.21 Reinforcement shall not be included in the effective torsional reinforcement if:

1) the bars are not anchored in accordance with section 9;
2) the spacing of the closed links exceeds $(x_1 y_1)/4$, where x_1 and y_1 are the link dimensions;
3) the spacing of closed links exceeds 16 times the longitudinal corner bar diameter; or,
4) the links do not enclose corner bars with a diameter greater than the diameter of the links.

Torsion resistance

5.22 Torsion resistance shall be calculated using one of the following approaches:

1) the approach for box sections, rectangular sections, and T,L and I sections in this document;
2) the approach for combined shear and torsion resistance provided in BS EN 1992-2 [Ref 3.N].

Box Sections

5.23 The torsional resistance for box sections shall be taken from the greater of the values calculated from:

1) Equation 5.23a; and,
2) Equation 5.23b.

Equation 5.23a Torsion resistance for box sections

$$T_u = 2A_o\sqrt{\left(\frac{\sum(A_{sL}f_y/\gamma_{ms})}{2(x_1+y_1)}\right)\left(\frac{A_{st}}{s_v}\frac{f_{yv}}{\gamma_{ms}}\right)}$$

where:

A_{st}	is the area of one leg of a closed link of a section included in the effective torsional reinforcement
$\sum(A_{sL}f_y/\gamma_{ms})$	is the tensile resistance of longitudinal reinforcement included in the effective torsional reinforcement
f_{yv}	is the characteristic, or worst credible, strength of the links, but not taken as greater than 500 MPa
f_y	is the characteristic, or worst credible, strength of the longitudinal reinforcement, but not taken as greater than 500 MPa
s_v	is the spacing of the links along the member
x_1	is the smaller centre-line dimension of a link
y_1	is the larger centre-line dimension of a link

Equation 5.23b Minimum torsion resistance for box sections

$$T_u \geq 2h_w A_o v_{t\,\min}$$

where:

$v_{t\,\min} = 0.082\sqrt{f_{cu}/\gamma_{mc}}$ in MPa, with f_{cu} in MPa

5.23.1 In areas subjected to simultaneous flexural compressive stress or axial compression, the value of $\sum(A_{sL}f_y/\gamma_{ms})$ in Equation 5.23a may be increased by the compressive force in the compression zone.

Rectangular sections

5.24 The torsional resistance for rectangular sections shall be taken as the greater of the values calculated from:

1) Equation 5.23a for an equivalent box section with $A_o = 0.8x_1y_1$; and,
2) Equation 5.24.

Equation 5.24 Minimum torsional resistance for rectangular sections

$$T_u \geq \frac{h_{\min}^2}{2}\left(h_{\max} - \frac{h_{\min}}{3}\right) v_{t\,\min}$$

T, L and I sections

5.25 The sectional torsional resistance of T, L and I sections shall be taken as the sum of the torsional resistances of the component rectangles.

5.26 The sectional torsional resistance of T, L and I sections shall not include any component rectangles that do not have torsional reinforcement.

5.27 A component rectangle shall not be treated as reinforced for torsion if its link reinforcement does not tie it to its adjacent rectangles.

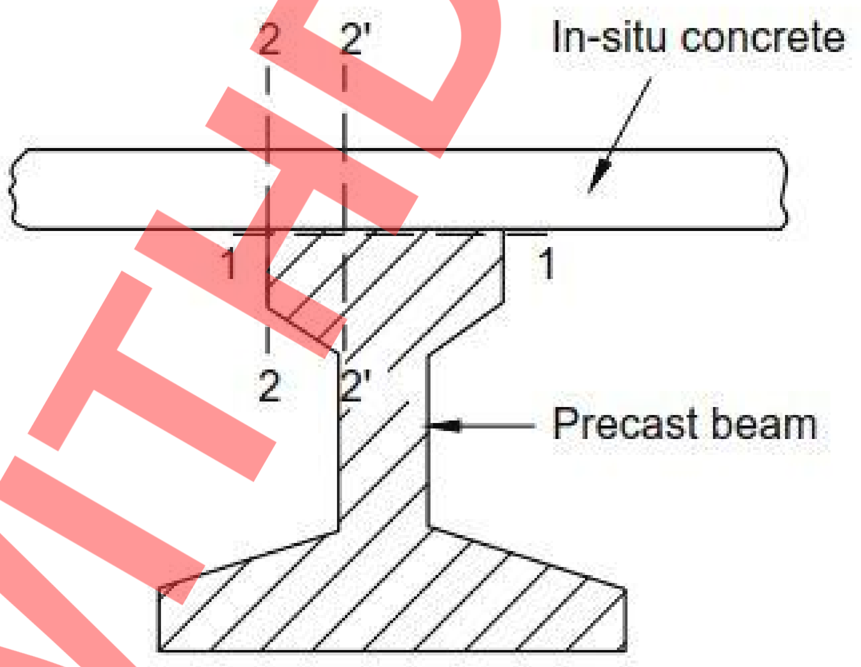
Longitudinal shear in beams

5.28 In beams, the longitudinal shear force per unit length, V_l , shall be assessed at:

1) any interface between a precast unit and in-situ concrete (e.g. plane 1-1 in Figure 5.28); and,
2) at any vertical planes critical in longitudinal shear (e.g. planes 2-2 and 2'-2' in Figure 5.28).

5.28.1 The longitudinal shear force in composite concrete members may be calculated with an elastic analysis of the composite concrete section, allowing for any differences in the stiffnesses of concrete in the precast and in-situ parts.

Figure 5.28.1 Potential shear planes



5.29 The longitudinal shear force per unit length, V_l shall not exceed the lesser of the values for V_{lu} given in Equation 5.29a and 5.29b.

Equation 5.29a Longitudinal shear resistance per unit length without effective reinforcement

$$V_{lu} = k_1 L_s \frac{f_{cu}}{\gamma_{mc}}$$

Equation 5.29b Longitudinal shear resistance per unit length with effective reinforcement

$$V_{lu} = \frac{v_1 L_s}{\gamma_{mv}} + \frac{0.8 A_e f_s}{\gamma_{ms}}$$

where:

k_1	is defined in Table 5.29a
f_{cu}	is the characteristic, or worst credible, strength of the weaker of the two concretes each side of the shear plane; but not greater than 45 N/mm²
L_s	is the breadth of the shear plane under consideration
v_1	is defined in Table 5.29a
A_e	is the area of reinforcement per unit length crossing the shear plane under consideration, including reinforcement resisting co-existent bending and vertical shear
f_s	is taken as f_y if the reinforcement crossing the shear plane is fully anchored, or reduced in proportion to the ratio of the anchorage available to that required for full anchorage; but f_s is not taken as greater than $10\frac{L_s}{A_e}$ MPa

Table 5.29a Parameters for longitudinal shear resistance

Interface surface type	v_1	k_1
Monolithic	$0.05 f_{cu}$ but not less than 1.13 MPa or greater than 1.56 MPa	0.24
Type 1	$0.04 f_{cu}$ but not less than 0.8 MPa or greater than 1.28 MPa	0.24
Type 2	$0.019 f_{cu}$ but not less than 0.38 MPa or greater than 0.63 MPa	0.14
Note: Interface surface types 1 and 2 are defined in Table 5.29b.		

Table 5.29b Interface surface type

Type 1	The contact surface of the concrete in the precast members was prepared as below: 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, to remove the outer mortar skin and expose the larger aggregate without disturbing it; or, 2) The surface skin and laitance were removed by sand blasting or the use of a needle gun.
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.

5.29.1 For lightweight aggregate concrete, v_1 and k_1 should be reduced by 25%.

5.29.2 In the absence of record drawings, site data or original design calculations, the type of surface should be assumed to be surface type 2.

5.30 For composite beam and slab construction, reinforcement crossing the shear plane shall not be included if its spacing exceeds either of the following:

1) six times the minimum thickness of the in-situ concrete flange; or,
2) 900 mm.

6. Reinforced concrete slabs

Moment resistance of slabs

6.1 The moment resistance of reinforced concrete slabs shall be verified in all directions of the slab based on the same assumptions as for the resistance moment of reinforced concrete beams, but taking account of the relative directions of the reinforcement and the contribution of twisting moments.

6.2 The moment resistance of slabs shall be verified to exceed the bending moment in all directions of the slab, in sagging and hogging, by satisfying Equation 6.2 for all directions of the n-axis as illustrated in Figure 6.2.1.

Equation 6.2 Verification of biaxial bending

$$\left|M_n^*\right| > |M_n|$$

where:

M_n^*	is the moment resistance per unit width in the n direction
M_n	is the moment per unit width in the n direction

6.2.1 The applied moment per unit width in the n direction, M_n should be calculated from the components of bending and twisting moments as defined in Equation 6.2.1.

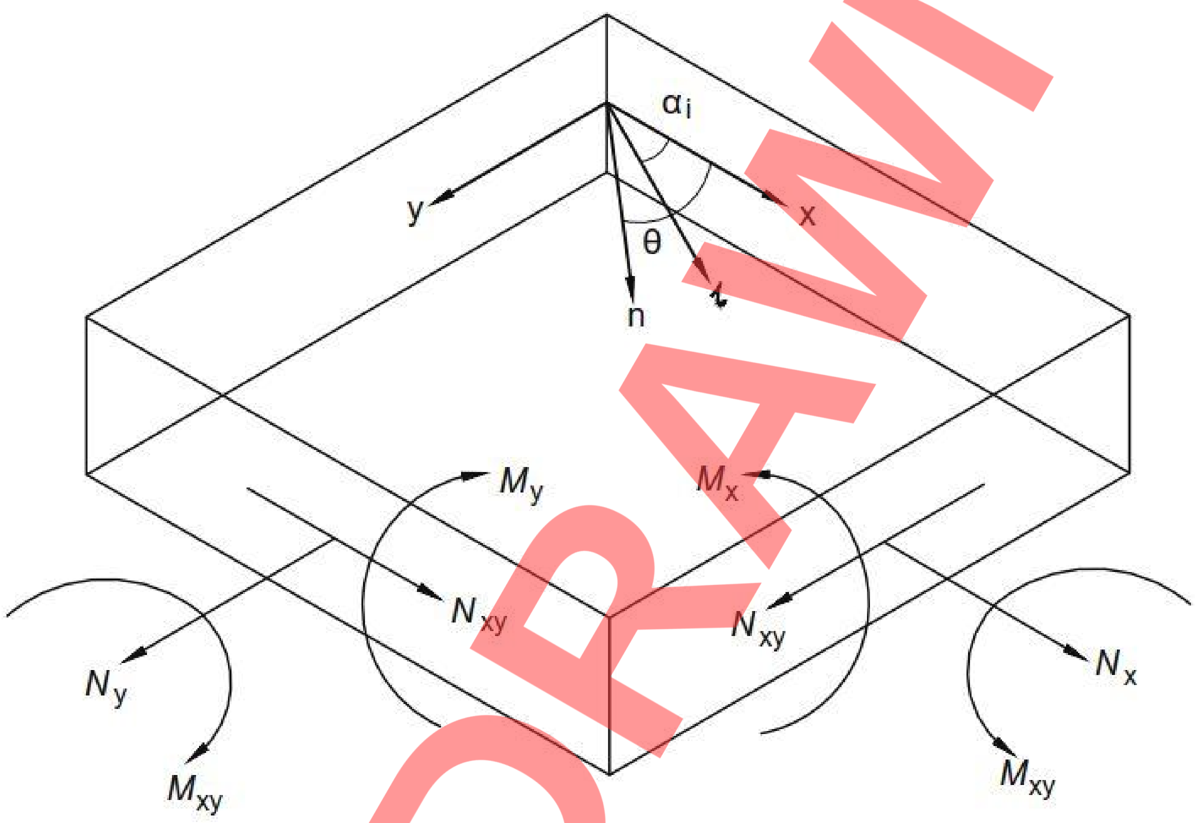
Equation 6.2.1 Moment in the n direction

$$M_n = M_x\cos^2\theta + M_y\sin^2\theta - 2M_{xy}\sin\theta\cos\theta$$

where:

M_x	is the assessment moment per unit width in the x direction as illustrated in Figure 6.2.1
M_y	is the assessment moment per unit width in the y direction as illustrated in Figure 6.2.1
M_{xy}	is the assessment twisting moment per unit width about the x axis and the y axis as illustrated in Figure 6.2.1
θ	is the angle between the x axis and the n axis, as illustrated in Figure 6.2.1
x, y	are orthogonal axes in the plane of the slab as illustrated in Figure 6.2.1

Figure 6.2.1 Forces and moments in a concrete slab



6.2.2 The resistance moment of the slab per unit width in the n direction, M_n^* , should be determined by one of the following methods:

1) a numerical analysis of the component of moment resistance in the n direction including the simultaneous contributions of all reinforcement layers, with the stress in each reinforcement layer determined based on the calculated strain in the direction of the reinforcement;
2) using Equation 6.2.2 to combine the components of moment resistance for each reinforcement layer, where the reinforcement is not heavily skewed; or,
3) using the sandwich model in BS EN 1992-2 [Ref 3.N].

Equation 6.2.2 Moment resistance in slabs

$$M_n^* = \sum_i M_{\alpha i}^* \cos^2(\alpha_i - \theta)$$

where:

α_i is the angle between the i-direction reinforcement and the x axis as illustrated in Figure 6. 2.1

$M_{\alpha i}^*$ is the moment of resistance per unit width of the slab due to the i-direction reinforcement alone, assuming the reinforcement in different directions acts independently

NOTE 1 For further guidance on the methods of analysing the effects and resistances in slabs in biaxial bending, refer to Denton et al 2011 [Ref 4.I].

NOTE 2 For skewed reinforcement, it can be unsafe to combine the components of bending resistance for each direction of reinforcement, neglecting the interaction between the different layers of reinforcement, *since this underestimates the depth of the compression zone. This effect is more pronounced for higher proportions of reinforcement. For further guidance, refer to Denton et al 2011 [Ref 4.I]*

6.2.3 Compression reinforcement in solid slabs should not be assumed to be fully effective in the resistance calculation unless secondary reinforcement is present that satisfies all of the following:

1) the area of secondary reinforcement is at least 0.12% of bd in the case of high strength reinforcement and 0.15% of bd in the case of mild steel reinforcement;
2) the diameter of the secondary bars is not less than one-quarter of the size of the main bars; and,
3) the spacing of the secondary reinforcement is no greater than 300mm.

6.2.4 Compression reinforcement with an area of greater than 1% of bd in a solid slab should not be assumed to be fully effective in the resistance calculation unless links are present that satisfy all of the following:

1) the link diameter exceeds 6 mm;
2) the link diameter exceeds one-quarter of the size of the largest compression bar;
3) the link spacing does not exceed twice the member thickness in either direction; and,
4) the link spacing does not exceed 16 times the compression bar size in the direction of the compressive force.

6.2.5 Where there is insufficient secondary reinforcement or links to allow the compression in the main reinforcement to be fully effective, a partially effective compressive strength of the bars may be assumed, calculated in proportion to the ratio of the provided secondary steel area or link area to the minimum area that would enable fully effective compression reinforcement to be assumed.

6.3 In voided slabs, the transverse flexural strength shall be calculated allowing for the effects of transverse shear.

6.3.1 An analysis based on the assumption that the transverse section acts as a Vierendeel frame may be used to account for the effects of transverse shear.

Resistance to in-plane forces in slabs

6.4 The resistance to in-plane forces in slabs shall be verified.

6.4.1 The resistance to in-plane forces may be deemed to be satisfied if both Equations 6.4.1a and 6.4.1b are satisfied.

Equation 6.4.1a Verification of yield due to inplane forces

$$\sum \left(N_i^* \cos^2\alpha_i - N_x\right) \sum \left(N_i^* \sin^2\alpha_i - N_y\right) \geq \sum \left(N_i^* \sin\alpha_i \cos\alpha_i + N_{xy}\right)$$

where:

N_i^*	is the tensile resistance per unit width of the i-layer of reinforcement in the direction of the reinforcement
α_i	is the angle of the reinforcement from the x-axis as illustrated in Figure 6.2.1
N_x	is the in-plane force in the x direction per unit width as illustrated in Figure 6.2.1
N_y	is the in-plane force in the y direction per unit width as illustrated in Figure 6.2.1
N_{xy}	is the in-plane shear force per unit width as illustrated in Figure 6.2.1

Equation 6.4.1b Verification of concrete crushing due to in-plane forces

$$\left(N_c^* + N_x\right)\left(N_c^* + N_y\right) \geq N_{xy}^2$$

where:

N_c^* is the compressive resistance per unit width of the concrete based on a stress of $0.6f_{cu}/\gamma_{ms}$

Shear resistance of slabs

General

6.5 The shear resistance for slabs shall be assessed in accordance with the shear capacity for beams as set out in Section 5, but with the following changes:

1) for solid slabs, the web width b_w is taken as the width of the part of the slab being assessed; and,
2) the shear resistance of a concrete slab more than $3d$ from a support V_{uc} is taken from Equation 6.5.

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

$$V_{uc} = \frac{0.27}{\gamma_{mv}}\xi_s\rho_s^{\frac{1}{3}}f_{cu}^{\frac{1}{3}}b_w d$$

NOTE 1 Shear resistance per unit width can be obtained by dividing all shear resistance formulae by b_w .

NOTE 2 All parameters are as defined in the corresponding parts of Section 5.

6.5.1 For lightweight aggregate concrete, V_{uc} should be taken as the value for normal weight concrete in Equation 6.5 multiplied by 0.9.

6.6 Shear reinforcement shall be assumed to be ineffective in slabs less than 200 mm thick.

Shear resistance of voided slabs

6.7 The shear resistance of the longitudinal ribs in voided slabs shall be assessed using the requirements for beams according to Section 5.

6.7.1 For voided slabs with circular voids, the longitudinal shear resistance may be calculated from Equation 6.7.1, if the following criteria are met:

1) φ/b is not greater than 0.8, where φ is the diameter of the void and b is the distance between void centres;
2) φ/h is no greater than 0.75, where h is the overall depth of the slab; and,
3) the thickness of the compression flange is not less than $0.35(h-\varphi)$.

Equation 6.7.1 Longitudinal shear resistance for voided slabs with circular voids

$$V_{cv} = KV_c'$$

where:

V_c' is the shear resistance of the solid slab ignoring the presence of voids

$K = 1 - \left\{0.4(\varphi/b) + 0.6(\varphi/b)^{2.5}\right\}$ is a variable reduction factor based on the structure geometry

6.8 The shear resistance of the longitudinal ribs in voided slabs shall exceed the assessment shear force, including shear due to torsional effects.

6.9 The top and bottom flanges, acting as solid slabs, shall be assessed to resist the global transverse shear in proportion to the flange thickness.

6.10 Each flange of a rectangular voided slab shall be assessed for punching effects due to concentrated loads acting on the flange.

Punching shear resistance

6.11 For concentrated loads and support reactions, the assessment shear force shall not exceed:

1) $V_{P\max}$ as defined in Equation 6.11a; or,
2) V_{Pu} as defined in Equation 6.11b.

Equation 6.11a Maximum punching shear resistance

$$V_{P\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\frac{f_{cu}}{\gamma_{mc}} d u_o$$

where:

u_o is the perimeter of the loaded area at the top surface of the concrete slab.

Equation 6.11b Punching shear resistance

$$V_{Pu} = \min \begin{cases} V_{Pc} + \sum A_{Psv} \sin\alpha\left(\frac{f_{yv}}{\gamma_{ms}}\right) \\ 1.4V_{Pc} + 0.3\sum A_{Psv} \sin\alpha\left(\frac{f_{yv}}{\gamma_{ms}}\right) \\ 2V_{Pc} \end{cases}$$

where:

V_{Pc} is the concrete component of the punching shear resistance calculated for the critical punching perimeter as defined in Table 6.11

$\sum A_{Psv}$ is the total area of effective shear reinforcement between the loaded area and the critical perimeter defined in Table 6.11, except for case (c)(ii), when it is the area of effective shear reinforcement within a distance d from the critical punching perimeter, on the other side from the loaded area

α is the inclination of the shear reinforcement to the plane of the slab

f_{yv} is taken as no greater than 500 MPa

Table 6.11 Parameters for shear in solid slabs under concentrated loads

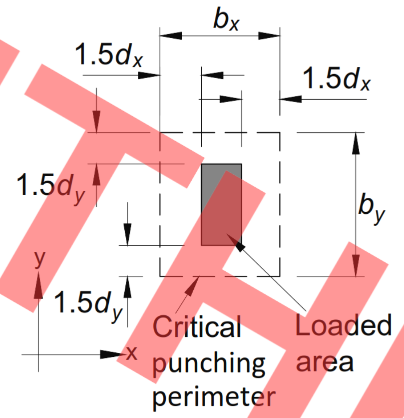
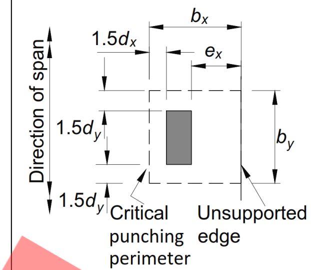
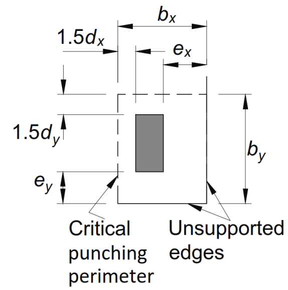
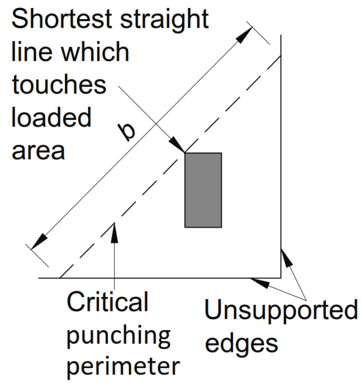
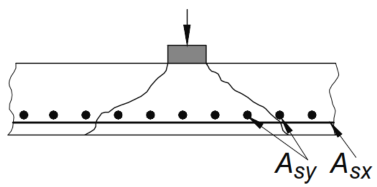
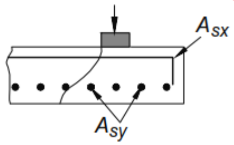
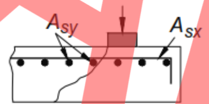
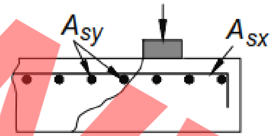
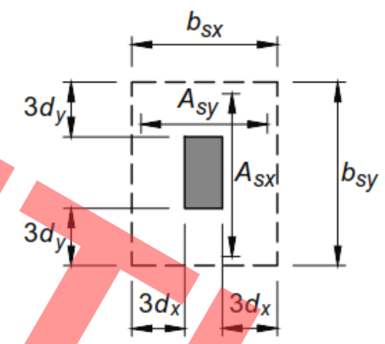
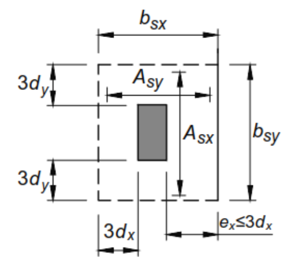
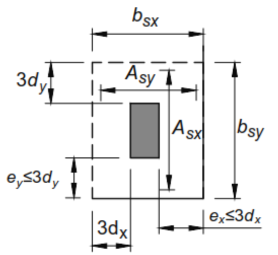
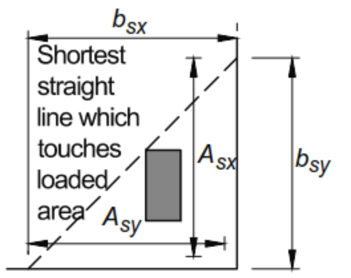
Case:	Load at middle of slab	Load at edge of slab	Load at corner of cantilever slab with square corners	
	Case (a)	Case (b)	Case (c)(i)	Case (c)(ii)
Critical punching perimeter for calculating shear resistance	 $V_{Pc} = \sum V_{uc}$ for 4 portions of the critical punching perimeter.	 $V_{Pc} = 0.8 \sum V_{uc}$ for 3 portions of the critical punching perimeter	 $V_{Pc} = 0.8 \sum V_{uc}$ for 2 portions of the critical punching perimeter	 $V_{Pc} = V_{uc}$ for the perimeter with parameters for ξ_s , ρ_s and d calculated as the average for the directions of the reinforcement.
Idealised mode of failure and illustration of longitudinal tension reinforcement contributing to resistance				

Table 6.11 Parameters for shear in solid slabs under concentrated loads (continued)

Parameters used to derive longitudinal reinforcement ratio ρ_s for each portion of the critical punching perimeter				
	$\rho_{sx} = \frac{A_{sx}}{b_{sy} \cdot d_x}$ $\rho_{sy} = \frac{A_{sy}}{b_{sx} \cdot d_y}$			
Note 1: The critical punching perimeter for calculating shear resistance is assumed to have squared corners for rectangular and circular loaded areas Note 2: A_{sx} and A_{sy} include only the effectively anchored tensile reinforcement Note 3: Only tension reinforcement is shown in the idealised modes of failure				

6.11.1 For lightweight aggregate concrete, the value of $V_{P\max}$ in Equation 6.11a should be multiplied by 0.8.

6.11.2 For a group of concentrated loads, adjacent loaded areas should be applied:

1) singly; and,
2) in combination.

6.11.3 Shear reinforcement should be assessed to be effective for punching shear if all of the following are satisfied:

1) the slab thickness exceeds 200 mm;
2) the area of shear reinforcement exceeds the criterion in Equation 6.11.3;
3) the angle of the shear reinforcement from the plane of the slab $\alpha \geq 30$ degrees;
4) the shear reinforcement satisfies the anchorage and bearing requirements of Section 9; and,
5) the longitudinal reinforcement satisfies the anchorage and bearing requirements of Section 9.

Equation 6.11.3 Criterion for effective shear reinforcement in slabs

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

where:

$\sum bd$ is the area of the critical perimeter in Figure 6.11

6.11.4 The ultimate punching shear resistance should be checked on perimeters progressively $0.75d$ from the critical perimeter, with the value of A_{sv} taken as the area of shear reinforcement between the perimeter and a perimeter $1.5d$ within the perimeter under consideration.

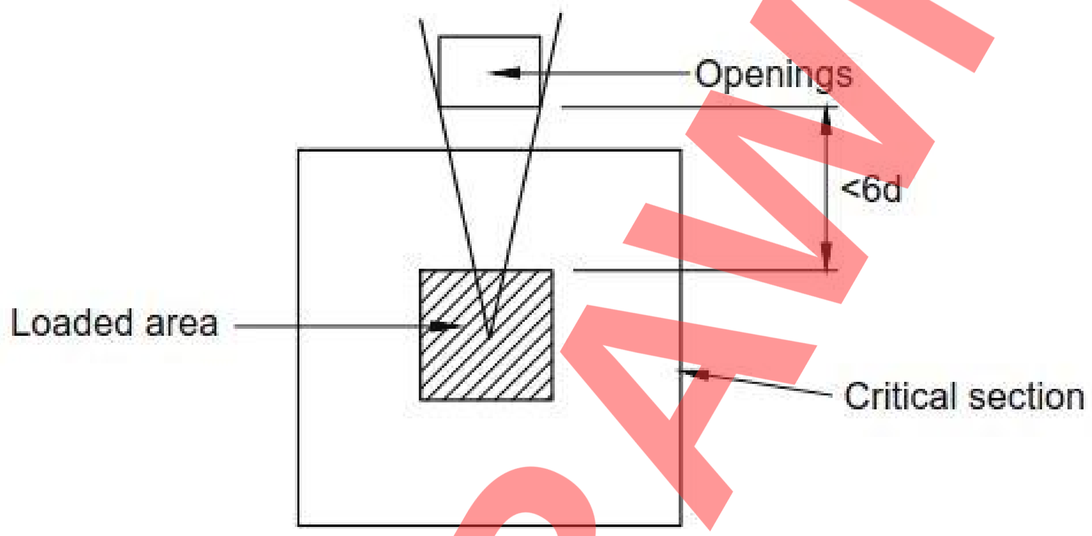
6.11.5 Where a part of a perimeter cannot extend $1.5d$ from the boundary of the loaded area, then the part perimeter should be taken as far from the loaded area as possible.

6.11.6 Where a part of a perimeter cannot physically extend $1.5d$ from the boundary of the loaded area, the value of V_{Pc} for that part should be increased by a factor $1.5d/a_v$.

NOTE a_v *is the distance from the boundary of the loaded area to the perimeter.*

6.11.7 Where openings in slabs and footings are located at a distance less than $6d$ from the edge of a concentrated load or reaction, then the part of the critical perimeter that is enclosed by radial projections of the openings to the centroid of the loaded area, as illustrated in Figure 6.11.7, should be assessed as ineffective.

Figure 6.11.7 Openings in slabs



6.11.8 Where a single opening is adjacent to the loaded area, its presence may be ignored if both of its dimensions are less than both of the following:

1) one quarter of the side of the loaded area; and,
2) one half of the slab depth.

Resistance of twisting effects in slabs

6.12 Interior regions of slabs to resist twisting moments shall be assessed in accordance with the requirements for moment resistance of slabs.

6.13 The shear resistance of unsupported slab edge zones shall exceed the combination of flexural shear and twisting effects as given in Equation 6.13.

Equation 6.13 Shear force in slab edge zones

$$V_{ue} \geq V_t b_e + M_{nt}$$

where:

V_{ue} is the shear resistance of the edge zone, determined using the requirements for shear resistance of beams in Section 5, with b_w taken as the width of the edge zone b_e

V_t is the flexural shear force per unit width at the edge acting on a vertical plane perpendicular to the edge

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

b_e is the width of the edge zone, taken as equal to the depth of the slab

7. Reinforced concrete columns, walls and bases

Moment and axial resistance of columns

General

7.1 Depending on the slenderness of the column, the column shall be assessed using the rules for:

1) slender columns; or,
2) short columns.

7.2 A column shall be assessed using the rules for short columns where its slenderness in each plane of buckling does not exceed:

1) the criterion in Equation 7.2a for normal concrete; or,
2) the criterion in Equation 7.2b for lightweight aggregate concrete.

Equation 7.2a Slenderness criterion for short columns

$$\frac{l_e}{h} < 12$$

Equation 7.2b Slenderness criterion for short columns of lightwegth aggregate concrete

$$\frac{l_e}{h} < 10$$

where:

l_e is the effective height in the plane of buckling under consideration, obtained from Table 7.2 or derived from first principles

h is the depth of the cross-section in the plane of buckling under consideration

Table 7.2 Effective heights in columns

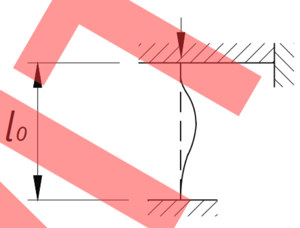
Idealised column and buckling mode	Restraints			Effective Height, l_e
	Location	Position restraint	Rotation restraint	
Case 1 	Top	Full	Full	$0.7l_o$
	Bottom	Full	Full	

Table 7.2 Effective heights in columns (continued)

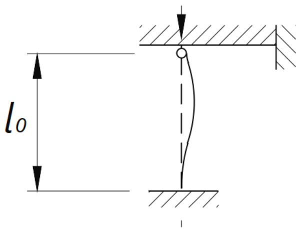
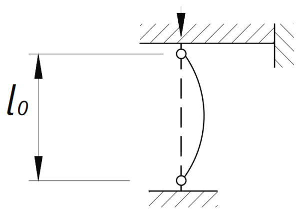
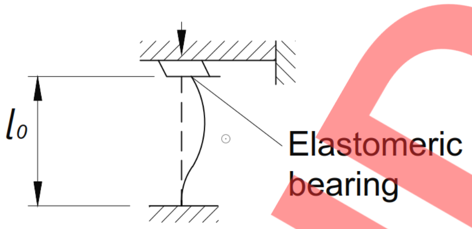
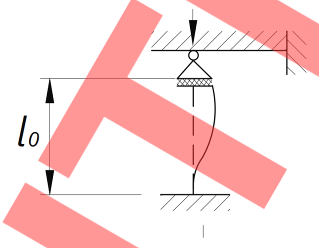
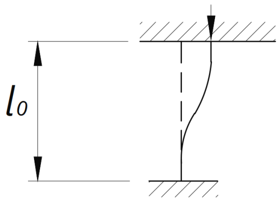
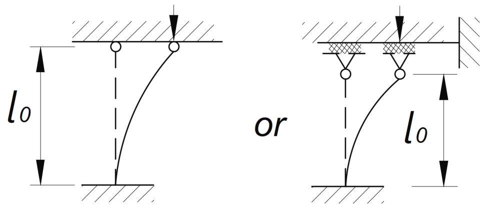
Case 2 	Top	Full	None	$0.85l_o$
	Bottom	Full	Full	
Case 3 	Top	Full	None	$1.0l_o$
	Bottom	Full	None	
Case 4 	Top	None	None	$1.3l_o$
	Bottom	Full	Full	
Case 5 	Top	None	None	$1.4l_o$
	Bottom	Full	Full	

Table 7.2 Effective heights in columns (continued)

Case 6 	Top	None	Full	$1.5l_o$
	Bottom	Full	Full	
Case 7 	Top	None	None	$2.3l_o$
	Bottom	Full	Full	

7.2.1 When applying Figure 7.2, the column end may be assumed to have fully fixed moment restraints when the rotational restraint exceeds:

1) $4(EI)_c/l_0$ for cases 1,2,4,5,6, where $(EI)_c$ is the flexural rigidity of the column cross-section;
2) $8(EI)_c/l_0$ for case 7.

7.2.2 Case 4 from Figure 7.2 should be used for columns that are restrained at the base and that have roller bearings at the top, provided the rollers are equipped with racks or other effective means to maintain them in position.

7.3 When first principles are used to derive l_0 , the accommodation of movements and the method of articulation shall be assessed accounting for:

1) the flexibility of the foundation;
2) the type of bearings; and,
3) the articulation system.

Assessment of short columns

7.4 The moment and axial resistance of a cross section of a column shall be calculated based on the following assumptions:

1) the strain distribution in the concrete and bonded reinforcement is based on the assumption that plane sections remain plane;
2) the compressive strain is limited to no greater than 0.0035 for unconfined concrete;
3) the compressive stresses in the concrete are derived from either the compressive strain according to Section 3, or, where the compression zone is rectangular in cross section, a constant stress of $0.6f_{cu}/\gamma_{mc}$ in the compression zone for unconfined concrete;
4) the tensile stress in the concrete is zero; and,
5) the stresses in the reinforcement are derived from the strain in the reinforcement according to Section 3, but no greater than the stresses that can be developed through bond, anchorage and bearing according to Section 9.

7.4.1 Equations 7.4.1a and 7.4.1b may be used to determine the resistance axial force and bending moment for rectangular columns.

Equation 7.4.1a Ultimate resistance axial load in columns

$$N_u = \left(\frac{0.6f_{cu}}{\gamma_{mc}}\right)bd_c + \left(\frac{f_y}{\gamma_{ms}}\right)A'_{sl} + \sigma_{s2}A_{s2}$$

Equation 7.4.1b Ultimate resistance moment in columns

$$M_u = \left(\frac{0.3f_{cu}}{\gamma_{mc}}\right)bd_c(h-d_c) + \left(\frac{f_y}{\gamma_{ms}}\right)A'_{sl}\left(\frac{h}{2}-d'\right) - \sigma_{s2}A_{s2}\left(\frac{h}{2}-d_2\right)$$

where:

N is the assessment axial load applied on the section considered

M is the assessment moment applied about the axis considered due to ultimate loads including the allowance for construction tolerance

N_u , M_u are the ultimate axial load and bending resistances 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 $2d'$

A'_{sl} is the area of compression reinforcement in the more highly compressed face

σ_{s2} is the stress in the reinforcement in the other face, derived from Section 3 and taken as negative if tensile

A_{s2} is the area of the reinforcement in the other face which can be considered as being:

1) in compression;
2) inactive; or,
3) in tension

as the result 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

NOTE *The M-N interaction curve can be plotted using Equation 7.4.1a and 7.4.1b for various d_c to demonstrate that the column section can resist the applied M-N combination.*

7.5 Where the confining effects of links are included, the model for assessing the confined concrete shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

7.6 Reinforcing bars in compression shall not be taken to develop the full compressive yield strength in the calculation of moment resistance unless the reinforcing bars are:

1) effectively restrained by links; or,
2) within 150 mm of a bar that is effectively restrained by links.

7.6.1 Compression bars may be taken to be effectively restrained where:

1) links are present that have a diameter at least one-quarter the size of the largest compression bar, at a spacing no greater than 12 times the size of the smallest compression bar; and,

2) every corner and alternate bar or group in an outer layer of reinforcement is restrained by a link passing round the bar and having an included angle of not more than 135°.

7.6.2 Where a bar in compression is not effectively restrained by links, the compressive strength for that bar should be either:

1) taken as zero; or,
2) reduced in proportion to the ratio of the actual link area to the minimum link area needed to effectively restrain the bar.

7.7 Short columns shall be assessed at the ultimate limit state for the applied axial forces and moments and accounting for additional moments caused by the actual eccentricity of the axial load arising from construction tolerances.

7.7.1 The actual eccentricity of the axial load may be:

1) determined by the inspection for assessment;
2) for bending about the minor axis, assumed to be 0.05 times the overall depth of the cross-section in the plane of bending, but not more than 20 mm; or,
3) for bending about the major axis, assumed to be 0.03 times the overall depth of the cross-section in the plane of bending, but not more than 20 mm.

7.7.2 Short columns subject to axial load and bending about both axes may be assessed using the criterion presented in Equation 7.7.2a.

Equation 7.7.2a Resistance criterion for short columns

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

where:

M_x , 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

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 assessment 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 assessment axial load, N

α_n is taken from Equation 7.7.2b, but within the range $1.0 \leq \alpha_n \leq 2.0$

Equation 7.7.2b Exponent for interation equation

$$\alpha_n = 0.67 + 1.66\frac{N_u}{N_{uz}}$$

where:

N_{uz} is the axial loading capacity of a column ignoring all bending, taken from Equation 7.7.2c

Equation 7.7.2c Ultimate axial capcity of short column ignoring bending

$$N_{uz} = 0.675\frac{f_{cu}}{\gamma_{mc}}A_c + f_y A_{sc}$$

where:

A_c	is the area of concrete
A_{sc}	is the total area of longitudinal reinforcement

Slender columns

7.8 The moment and axial resistance of slender columns shall be determined using the same rules as for short columns.

7.9 In the assessment of a slender column, additional moments, due to second order effects induced in the column by its deflection, shall be taken into account.

7.9.1 A slender column of constant cross-section bent about the minor axis (the y-y axis) should be assessed for its ultimate axial load, N , together with the moment M_{ty} given by Equation 7.9.1..

Equation 7.9.1 Total moment about the minor axis for slender column

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

where:

M_{iy}	is the initial assessment moment but not less than $0.02N$
h_x	is the overall depth of the cross-section in the plane of bending
K	is a coefficient taken as 1750 for normal concrete or 1200 for lightweight aggregate concrete
l_e	is the effective height either in the plane of bending or in the plane at right-angles, whichever is greater

7.9.2 For a column fixed in position at both ends where no transverse loads occur within its height, M_{ty} may be calculated based on a value of M_{iy} taken from Equation 7.9.2, but with M_{ty} not taken as less than M_2 .

Equation 7.9.2 Initial moment about the minor axis of a slender column

$$M_{iy} = 0.4M_1 + 0.6M_2$$

but with M_{iy} taken as not less than $0.4M_2$

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)

7.9.3 When the overall depth of the cross-section of a slender column, h_y , is less than three times the width, h_x , a slender column bent about the major axis (x-x axis) should be assessed for its ultimate axial load, N , together with M_{tx} given by Equation 7.9.3.

Equation 7.9.3 Total moment about the major axis for slender column

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

where:

M_{ix}	is the initial moment due to ultimate loads, but not less than $0.02N$
h_y	is the overall depth of the cross-section in the plane of bending

7.9.4 Where h_y is equal or greater than three times h_x , the column should be assessed as bi-axially loaded with the moment about the minor axis derived from the eccentricity for construction tolerances, as determined for short columns.

7.9.5 A slender column bent about both axes should 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 Equations 7.9.5a and 7.9.5b

Equation 7.9.5a Total moment about the major axis for slender column

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

Equation 7.9.5b Total moment about the minor axis for slender column

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

where:

M_{ix}	is the initial moment due to ultimate loads about the x-x axis, including the allowance for construction tolerances, as determined for short columns
M_{iy}	is the initial moment due to ultimate loads about the y-y axis, including the allowance for construction tolerances, as determined for short columns
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

Shear resistance of columns

7.10 The shear resistance of columns shall be assessed taking account of the effect of axial loading.

7.10.1 The shear resistance of a column subject to uniaxial shear should be assessed using the requirements for assessment of shear resistance of beams, but with the following changes and assumptions:

1) the concrete component of shear resistance V_{uc} is enhanced by the factor in Equation 7.10.1;
2) A_s is taken as the area of reinforcement which is in the half of the column opposite the extreme compression fibre;
3) d is taken as the distance from the extreme fibre with maximum compression to the centroid of the reinforcement included in A_s ; and,
4) b_w is taken as the column diameter.

Equation 7.10.1 Ultimate shear stress enhancement factor for columns

$$1 + \frac{0.15N}{A_c}$$

where:

N	is the ultimate axial load in Newtons, but not greater than $0.11 f_{cu} A_c$
A_c	is the area of the entire concrete section, in units of mm^2

7.11 A column subject to biaxial shear due to ultimate loads shall satisfy the criterion presented in Equation 7.11.

Equation 7.11 Criterion for column subject to biaxial shear

$$\left(\frac{V_x}{V_{ux}}\right) + \left(\frac{V_y}{V_{uy}}\right) \leq 1.0$$

where:

V_x and V_y	are the assessment shear forces in the x and y axes respectively
V_{ux} and V_{uy}	are the corresponding shear resistances in the x and y axes respectively, including the enhancement factor given in Equation 7.10.1

Bearing resistance of columns

7.12 Bearing stresses on columns due to local effects shall be assessed in accordance with Section 10.

Moments and axial resistance in reinforced concrete walls

General

7.13 A reinforced concrete wall shall be assessed using the rules for short walls or slender walls.

7.13.1 A reinforced concrete wall may be assessed as a short wall where the ratio of its effective height to its thickness does not exceed 12.

7.13.2 Short walls should be assessed using the rules for short columns.

7.13.3 Slender walls should be assessed using the rules for slender columns.

7.14 The effect of eccentric axial loading shall be included.

7.14.1 Where the actual load eccentricity has not been determined, the moment per unit length in the direction at right-angles to a wall should not be taken as 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.

7.15 Compression reinforcement in walls shall not be taken to develop the full compressive yield strength in the calculation of moment resistance unless the reinforcing bars are effectively restrained by secondary reinforcement.

7.15.1 Compression bars in walls should not be assumed to be fully effective in the resistance calculation unless secondary reinforcement is present that satisfies all of the following:

1) the area of secondary reinforcement is at least 0.12% of $b_t d$ in the case of high strength reinforcement and 0.15% of $b_t d$ in the case of mild steel reinforcement;
2) the diameter of the secondary bars is not less than one-quarter of the size of the main bars; and,
3) the spacing of the secondary reinforcement is no greater than 300 mm.

7.15.2 Compression reinforcement with an area of greater than 1% of bd in a wall should not be assumed to be fully effective in the resistance calculation unless links are present that satisfy all of the following:

1) the link diameter exceeds 6 mm;
2) the link diameter exceeds one-quarter of the size of the largest compression bar;
3) the link spacing does not exceed twice the member thickness in either direction; and,
4) the link spacing does not exceed 16 times the compression bar size in the direction of the compressive force.

7.15.3 Where the compression reinforcement is not fully effective, a partially effective compressive strength may be assumed, calculated in proportion to the ratio of the provided secondary steel area or link area to the minimum area needed for fully effective compression reinforcement.

Shear resistance in reinforced concrete walls

7.16 The shear resistance of walls shall be assessed taking account of the effect of axial loading.

7.16.1 The shear resistance of a wall subject to uniaxial shear should be assessed using the requirements for assessment of shear resistance of slabs, but with the concrete component of shear resistance V_{uc} enhanced by the factor in Equation 7.10.1.

7.17 A wall subject to biaxial shear due to ultimate loads shall satisfy Equation 7.11.

Moment resistance of bases

7.18 Moment resistance of bases shall be assessed according to the rules for slabs.

7.19 The assessment of the moment resistance of bases shall include reinforcement within a zone that is effective in resisting the bending effects from the column.

7.19.1 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 should be effective in resisting bending.

7.19.2 Where the width of the section considered is greater than $1.5(b_{col} + 3d)$, the total area of the effective reinforcement should be taken as the sum of the area of the reinforcement within a band of $(b_{col} + 3d)$ width centred on the column and the lesser of:

1) the actual area of reinforcement outside the band; and,
2) 50 % of the area of reinforcement within the band.

7.20 Pile caps shall be assessed using one of the following methods:

1) bending theory;
2) strut and tie analysis; or,
3) yield line analysis.

7.20.1 Where pile caps are assessed using strut-and-tie analysis, the effective area of reinforcement at a section should be taken as the lesser of:

1) the total area at the section; and,
2) 1.25 times the area of reinforcement in the strips linking the pile heads.

Shear resistance of bases

7.21 The shear resistance of bases and pile caps in the vicinity of a concentrated load shall be assessed under the more severe of the following two conditions:

1) 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, using the requirements for shear resistance of slabs according to Section 6;
2) punching shear around the loaded area, using the requirements for punching shear resistance in slabs according to Section 6.

7.21.1 Where shear resistance of a pile cap is governed by shear along vertical section extending across the full width of the cap, the enhancement of the shear resistance for sections close to supports should be applied to strips of width not greater than three times the pile diameter centred on each pile.

7.21.2 Where a_v is taken as the distance between the face of the column or wall and the nearer edge of the piles, a_v should be increased by 20% of the pile diameter.

7.21.3 Where shear resistance of a pile cap is governed by shear along vertical section extending across the full width of the cap, the allowable ultimate shear stress should be taken as the average over the whole section.

7.21.4 Where shear resistance of a pile cap is governed by punching shear around loaded areas and case (c)(ii) of Table 6.11 is used, the allowable ultimate shear stress may be enhanced over a width not greater than three times the pile diameter centred on the corner pile.

8. Prestressed concrete

External or unbonded prestressing

8.1 Where structures have external or unbonded prestressing, it shall be verified that there is sufficient resistance to prevent collapse under permanent loads at the ULS in both of the following circumstances:

1) failure of any two tendons at a cross section; and,
2) failure of 25% of the tendons at a cross section.

Accounting for deterioration in tendons

8.2 Assessment criteria for post-tensioned structures where investigations have identified areas with poorly grouted ducts or tendon corrosion shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

8.2.1 Tendons with poorly grouted ducts should be assessed to be unbonded.

NOTE 1 The decision on whether the grouting is sufficiently poor that reanchorage is not possible can involve some engineering judgement.

NOTE 2 Requirements for management of post-tensioned concrete bridges are given in CS 465 [Ref 10.I].

8.3 Prestressing bars and wires, including wires within strands and tendons, that have suffered sectional loss, which has resulted in them being unable to sustain their prestress force, shall be assumed to be ineffective for verifications at the ULS.

8.3.1 Where bonded post-tensioning tendons have complete grouting, and effective links are provided, locally ineffective wires and strands may be assumed to reanchor in accordance with Equation 8.3.1:

Equation 8.3.1 Reanchorage length

$$l_{ra} = l_t \cdot \sqrt{n_t}$$

where:

l_{ra}	is the reanchorage length
l_t	is the transmission length in accordance with Equation 8.35
n_t	is the number of wires or strands in a tendon

8.4 Where there is a risk of a loss of prestress in a member due to that tendon corrosion, calculations shall be performed taking upper and lower bound values for the effective prestress.

NOTE 1 The level of effective prestress can be informed by in-situ concrete stress measurement.

NOTE 2 Measuring in-situ concrete stresses can be subject to large errors, and is likely to require specialist advice.

NOTE 3 Spot checks on the level of remaining prestress in individual tendons can help informing the level of effective prestress.

Assessment prestress force

8.5 The assessment prestress force shall be calculated from:

1) the initial prestress force;
2) the losses in prestress; and,
3) the partial factor for prestress, as given in Section 2.

8.6 The initial prestress shall be estimated for assessment purposes.

8.6.1 The estimate for the initial prestress may be based on:

1) record drawings, available site data or original design calculations; or,
2) standards that were current at the time of design.

8.7 The losses in prestress shall include losses resulting from:

1) creep in concrete;
2) shrinkage of concrete;
3) relaxation in the prestressing steel;
4) the elastic deformation of the concrete;
5) movement of tendons at anchorages during transfer;
6) the effects of steam curing;
7) friction; and,
8) other causes of losses, in special circumstances.

Losses due to creep in concrete

8.8 Losses due to creep in concrete shall be calculated, or estimated based on experimental evidence.

8.8.1 The loss of prestress in bonded tendons due to creep of the concrete should be calculated based on:

1) the assumption that creep is proportional to stress in the concrete for stresses of up to one third of the cube strength at transfer; and,
2) the modulus of elasticity of the tendons given in Section 3.

8.8.2 The loss of prestress in unbonded tendons due to creep of the concrete should be calculated from:

1) the creep movement between anchors or other fixed points in the tendons; and,
2) the modulus of elasticity of the tendons given in Section 3.

8.8.3 The loss of prestress due to creep of concrete should be calculated using either:

1) Table 8.8.3; or,
2) BS EN 1992-2 [Ref 3.N].

Table 8.8.3 Creep strain of concrete

Tensioning system	Stress at transfer[2,5]	f_{ci} <40 MPa	f_{ci} ≥40 MPa
Pre-tensioning: transfer at between 3 days and 5 days after concreting	$\sigma_{ci} < 0.33 f_{ci}$	$48 \cdot 10^{-6} \left(\frac{40}{f_{ci}}\right) \sigma_c$	$48 \cdot 10^{-6} \sigma_c$
	$\sigma_{ci} \geq 0.5 f_{ci}$	$60 \cdot 10^{-6} \left(\frac{40}{f_{ci}}\right) \sigma_c$	$48 \cdot 10^{-6} \sigma_c$
Post-tensioning: transfer at between 7 days and 14 days after concreting	$\sigma_{ci} < 0.33 f_{ci}$	$36 \cdot 10^{-6} \left(\frac{40}{f_{ci}}\right) \sigma_c$	$36 \cdot 10^{-6} \sigma_c$
	$\sigma_{ci} \geq 0.5 f_{ci}$	$45 \cdot 10^{-6} \left(\frac{40}{f_{ci}}\right) \sigma_c$	$45 \cdot 10^{-6} \sigma_c$

Note 1: f_{ci} is the concrete strength at transfer in units of MPa.
Note 2: σ_{ci} is the maximum compressive stress at transfer anywhere in the section.
Note 3: σ_c is the concrete compressive stress in units of MPa.
Note 4: The values of creep strain are valid for both humid and dry exposure conditions.
Note 5: Interpolation can be used for intermediate stress levels at transfer.

NOTE *Creep losses in bonded construction are likely to vary over the length of a tendon and could be highest at critical sections, whereas creep losses in unbonded tendons are usually constant along their length.*

Losses due to shrinkage of concrete

8.9 Losses due to shrinkage of concrete shall be calculated, or estimated based on experimental evidence.

8.9.1 The loss of prestress in the tendons due to shrinkage of the concrete should be calculated from the modulus of elasticity for the tendons given in Section 3 and the values for concrete shrinkage strain given in either:

1) Table 8.9.1; or,
2) BS EN 1992-2 [Ref 3.N].

Table 8.9.1 Shrinkage strain of concrete

Tensioning system	Humid exposure (90% RH)	Normal exposure (70% RH)
Pretensioning	100×10^{-6}	300×10^{-6}
Post-tensioning: transfer at between 7 days and 14 days after concreting	70×10^{-6}	200×10^{-6}

Losses due to relaxation in the steel

8.10 Losses due to relaxation in the steel shall be calculated, estimated based on manufacturer's data or estimated based on experimental evidence.

8.10.1 Where tendons are not subject to high intensity lateral loads, or held at elevated temperatures, relaxation losses should be taken as the maximum relaxation after 1000 hours, subject to the jacking force at transfer.

NOTE *High intensity lateral loads can be applied by deviators in external tendons.*

Losses due to elastic deformation of concrete

8.11 Losses due to elastic deformation of concrete shall be calculated, or estimated based on experimental evidence.

8.11.1 Calculation of the immediate loss of force due to elastic deformation of the concrete should be based on the short-term elastic modulus of concrete given in Section 3, and the elastic modulus of tendons given in Section 3, or manufacturer's data.

8.11.2 Where tension is applied by means of a pre-tensioning system, the loss of prestress in the tendons at transfer due to elastic deformation of the concrete may be calculated on a modular ratio basis using the stress in the adjacent concrete.

8.11.3 Where tendons in post-tensioning systems were not stressed simultaneously, the loss of prestress in the tendons due to elastic deformation of the concrete should be calculated either:

1) 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; or,
2) based on the sequence of tensioning when the sequence of tensioning is known.

Loss of prestress at anchorages during transfer

8.12 Loss of prestress due to movement of the tendon at the anchorage during transfer in post-tensioned systems shall be estimated based on manufacturer's data.

Loss of prestress due to steam curing

8.13 Loss of prestress due to steam curing shall be estimated based on manufacturer's data.

NOTE *Where the `long-line' method of pre-tensioning was used there can be additional losses as a result of bond developed between the tendon and the concrete when the tendon was hot and relaxed.*

Loss of prestress due to friction

8.14 Loss of prestress due to friction shall be calculated, including the effects of:

1) friction in the duct due to unintended variation from the specified profile (wobble losses); and,
2) friction in the duct due to the curvature of the tendon.

NOTE 1 *Whether the desired duct profile was straight, curved or a combination of both, some unintended variations in the actual line of the duct can be expected, resulting in additional points of contact, and friction, between the tendon and the sides of the duct.*

NOTE 2 *The losses resulting from the friction due to unintended variations in profile can be known as wobble losses.*

8.14.1 Friction in the jack and anchorage may be neglected.

NOTE *Jacks are typically calibrated to give a specified force at the duct side of the anchorage.*

8.14.2 Wobble losses may be neglected for unbonded tendons that were free to move laterally at the time they were stressed.

8.14.3 Wobble losses due to unintended variation from the specified profile should be calculated using Equation 8.14.3:

Equation 8.14.3 Wobble losses

$$\Delta P_{(x)} = P_0\left(1 - e^{-Kx}\right)$$

where:

$\Delta P_{(x)}$ is the loss of prestressing force at a distance x from the jack

P_0 is the prestressing force in the tendon at the jacking end, before losses

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, in units of m^{-1}

x is the distance from the jack, in units of m

8.14.4 Where strong rigid sheaths or closely supported duct formers were used in construction, the value of K may be taken as 17×10^{-4} m^{-1}.

8.14.5 Where strong rigid sheaths or closely supported duct formers were not used in construction, the value of K should not be taken as less than 33×10^{-4} m^{-1}.

8.14.6 The value of K may be estimated from test data or manufacturer's data.

8.14.7 Where tendons are curved, the losses due to friction should be calculated using the sum of Equation 8.14.3 and Equation 8.14.7:

Equation 8.14.7 Friction due to tendon curvature

$$\Delta P_{(x)} = P_0\left(1 - e^{-\mu\sum\theta}\right)$$

where:

$\Delta P_{(x)}$ is the loss of prestressing force at any distance x along the curve from the tangent point

P_0 is the prestressing force in the tendon at the tangent point near the jacking end

μ is the coefficient of friction between the duct and the tendon taken from Table 8.14.7a and Table 8.14.7b, or derived by testing, or based on manufacturers' data

$\sum\theta$ is the sum of the angular displacements over the distance x

r_{ps} is the radius of curvature, which may be a variable function of x

Table 8.14.7a Coefficient of Friction for internal tendons

Steel moving on concrete	0.55
Steel moving on steel	0.30
Steel moving on lead	0.25
Greased coated monostrands moving on plastic sheaths	0.05

Table 8.14.7b Coefficient of Friction for external tendons

	Steel duct	HDPE duct
Lubricated strand	0.18	0.12
Lubricated wire	0.20	0.14
Non-lubricated strand	0.25	0.15
Non-lubricated wire	0.27	0.17

Assessment of prestressed sections at SLS

8.15 The assessment at the SLS shall determine the SLS class for prestressed elements of the structure, as defined in Table 8.15a.

Table 8.15a SLS classes for prestressed elements

SLS class	Tensile stress limits[1,2]	Compressive stress limits[3,4]	Consequence[5]
SLS Class 1	$\sigma_{ct} < 0$	$\sigma_c < 0.5\frac{f_{cu}}{\gamma_{mc}}$	The SLS loading is not expected to cause cracking.
SLS Class 2	$0 \leq \sigma_{ct} < \frac{0.56}{\gamma_{mc}}\sqrt{f_{cu}}$	$\sigma_c < 0.5\frac{f_{cu}}{\gamma_{mc}}$	The SLS loading is not expected to cause cracking.
SLS Class 3	The tensile stresses in the concrete do not satisfy SLS Class 2 but either of the following are satisfied: 1) hypothetical tensile stresses are assessed to be less than the equivalent limits given in Table 8.15b; or, 2) an assessment of crack widths demonstrates that crack widths satisfy SLS design requirements for durability.	$\sigma_c < 0.5\frac{f_{cu}}{\gamma_{mc}}$	Cracking is possible at SLS, but the cracking associated with the SLS loading is not expected to exceed durability criteria.
Does not meet SLS Class 1, 2 or 3			Cracking at SLS is expected to exceed durability criteria.

Note 1: σ_{ct} is the assessment tensile stress in the concrete at SLS in units of MPa calculated on a notionally uncracked section where plane sections are assumed to remain plane and the concrete is assumed to have linear elastic properties in tension and compression up to the limits.
Note 2: f_{cu} has units of MPa.
Note 3: γ_{mc} is the appropriate value for the partial factor for prestressed concrete verifications at SLS as given in Section 2.
Note 4: σ_c is the assessment compressive stress in the concrete at SLS due to prestress and permanent loads calculated as per Note 1.
Note 5: Cracking can be present even in structures assessed as SLS Class 1 or 2, for example due to a previous abnormal load.

Table 8.15b Hypothetical tensile stress limits for SLS Class 3

		Hypothetical tensile stress limits for a member of 400 mm depth[2]		
Prestressing type	Surface environment[1]	f_{cu} =30 MPa	f_{cu} =40 MPa	f_{cu} ≥50 MPa
Pre-tensioned tendons/ grouted post-tensioned tendons	Extreme	-	4.1	4.8
	Very severe	3.5	4.5	5.3
	Severe/ Moderate	4.1	5.5	6.3
Pre-tensioned tendons distributed in the tensile zone and positioned close to the tension faces of the concrete	Extreme	-	5.3	6.3
	Very severe	-	5.8	6.8
	Severe/ Moderate	-	6.8	7.8

Note 1: The surface environment is defined in Table 8.15c for the surface in tension.
Note 2: The hypothetical tensile stress limits are applicable for a member of 400 mm depth. For other depths, the stress limits should be multiplied by the depth factor in Table 8.15d.
Note 3: The hypothetical tensile stress limits are based on the analysis of a notionally uncracked section where plane sections are assumed to remain plane, and the concrete is assumed to have linear elastic properties in tension and compression up to the hypothetical stress limits.
Note 4: The hypothetical tensile stress limits are not applicable for unbonded tendons; prestressed structures containing exclusively unbonded tendons need not be checked for cracking, and those containing both bonded and unbonded tendon are treated as reinforced concrete sections in which the effect of prestress is an axial force and moment, and crack widths are calculated as reinforced concrete columns.
Note 5: The hypothetical tensile stress limits conservatively ignore the effect of additional tensile reinforcement. The effect of additional tensile reinforcement is given in BS 5400-4 [Ref 19.I].

Table 8.15c Surface environment

Environment	Description
Extreme	Concrete surfaces exposed to abrasive action by sea water or water with a pH<4.5
Very severe	Concrete surfaces directly affected by deicing salts or sea water spray
Severe	Concrete surfaces exposed to driving rain or alternate wetting and drying
Moderate	Concrete surfaces above ground level and fully sheltered against rain, deicing salts and sea water spray. Concrete surfaces permanently saturated by water with a pH>4.5

Table 8.15d Depth factor for hypothetical tensile stress limits

Depth of member (mm)	Depth factor
$\leq$ 200	1.1
400	1.0
600	0.9
800	0.8
$\geq$ 1000	0.7

8.15.1 Where a direct assessment of crack widths is proposed for the verification of SLS Class 3 the assessment should be carried out in accordance with the crack width requirements of BS 5400-4 [Ref 19.I].

8.16 The compressive stress at unreinforced contact joints at SLS shall be greater than the minimum values given in Table 8.16.

Table 8.16 Minimum SLS compressive stress at unreinforced contact joints

Joint type	Minimum compressive stress (MPa)
Cement mortar joints	1.5
All other contact joints	0

Moment resistance of prestressed sections at ULS

Analysis of cross-sections

8.17 The moment resistance of a cross section shall be calculated based on the following assumptions:

1) the changes in strain distribution in the concrete, bonded prestressing steel and reinforcement are based on the assumption that plane sections remain plane;
2) the strains due to prestress are included based on the calculated assessment prestress force after losses;
3) the compressive strain is limited to no greater than 0.0035;
4) the compressive stresses in the concrete are derived from either the compressive strain according to Section 3, or, where the compression zone is rectangular in cross section, a constant stress of $0.6\frac{f_{cu}}{\gamma_{mc}}$ in the compression zone;
5) the tensile stress in the concrete is zero;
6) the stresses in bonded prestressing steel and reinforcement are derived from the strain according to Section 3, but no greater than the stresses developed through bond, anchorage and bearing according to Section 9;

7) the additional strain developed in unbonded prestressing tendons at ULS is related to the deformation between anchor points; and,

8) the stress in unbonded prestressing tendons is derived from the total strain according to Section 3.

8.17.1 The additional strain in unbonded prestressing tendons related to the deformation between anchor points should be determined from either:

1) a non-linear analysis; or,

2) assuming the values for additional strain given in Table 8.17.1.

Table 8.17.1 Assumed additional strain developed in unbonded tendons

<table>
<tr><td>Tendon arrangements</td><td>Tendons lie within $0.1d$ of the soffit</td><td>Tendons do not lie within $0.1d$ of the soffit</td></tr>
<tr><td>Tendons do not extend more than $\frac{h}{2}$ beyond the supports</td><td>0.0005</td><td rowspan="3">0</td></tr>
<tr><td>Tendons are anchored more than $\frac{h}{2}$ beyond the supports and can slide over deviators</td><td>0</td></tr>
<tr><td>Tendons are anchored more than $\frac{h}{2}$ beyond the supports and it can be demonstrated that the tendons would not slide over deviators at ULS</td><td>0.0005</td></tr>
</table>

NOTE Some usually conservative assumptions, such as neglecting the tensile strength of concrete, can incorrectly increase the tendon strains in a non-linear analysis.

8.17.2 Unbonded tendons anchored within $h/2$ of a section should be ignored, except where the section is within $h/2$ of a simply supported end.

Assessment formulae

8.18 The moment resistance shall be calculated and reported for all critical cross-sections.

8.18.1 Where sections are rectangular, or flanged with the neutral axis within the flange, and in structures that contain only bonded prestressing, with no tendons in the compression zone at ULS, the moment resistance may be taken from Equation 8.18.1a.

Equation 8.18.1a Moment resistance of rectangular and flanged sections

$$M_u = \sigma_{pb} A_{ps} (d - 0.5x)$$

where:

M_u	is the moment resistance of the section
σ_{pb}	is the tensile stress in the tendons at failure of the element taken from Equation 8.18b
x	is the neutral axis depth at ULS taken from Equation 8.18c
d	is the effective depth of the tension reinforcement
A_{ps}	is the area of the prestressing tendons in the tension zone

Equation 8.18.1b Tendon stress at failure

$$\sigma_{pb} = \min \left\{ \begin{array}{l} \left(\alpha - \frac{f_{pu}A_{ps}}{f_{cu}bd}\right)\frac{f_{pu}}{\gamma_{ms}} \\ \frac{f_{pu}}{\gamma_{ms}} \end{array} \right.$$

where:

$$\alpha = \left\{ \begin{array}{ll} 1.3 & \text{for pre-tensioning} \\ 1.15 & \text{for post-tensioning} \end{array} \right.$$

f_{pu} is the ultimate strength of the tendon

Equation 8.18.1c Neutral axis depth

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

NOTE *Prestressing tendons and reinforcement in the compression zone at ULS are ignored in this method. An iterative approach can be needed where tendons lie near the neutral axis.*

Shear resistance of prestressed sections at the ULS

8.19 The shear resistance of prestressed members shall be assessed using one of the following approaches:

1) the additive approach based on the sum of the resistances of the concrete alone, V_c and of the shear reinforcement, V_s ; or,
2) using BS EN 1992-2 [Ref 3.N].

8.20 When using the additive approach, shear force in the section shall not exceed $V_{\max}$ as defined in Section 5, Equation 5.6a.

8.20.1 Where sections are uncracked in flexure, d in equation 5.6a may be taken to be d_t .

8.20.2 For sections with inclined tendons, the component of the assessment prestressing force normal to the longitudinal axis of the member may be added to $V_{\max}$.

8.20.3 In haunched sections, the component of the flange forces perpendicular to the longitudinal centroidal axis of the beam calculated from an elastic section analysis under the relevant load case may be subtracted from the applied shear force.

8.21 Within the transmission length of pre-tensioned members, the shear resistance of a section shall be taken as the greater of:

1) the shear resistance calculated as for a reinforced concrete section according to Section 5, 6 or 7, with the area of steel taken to be all passive reinforcement plus any rigid tendons; and,
2) the shear resistance calculated as for a prestressed concrete section, with the value of the prestress taken to vary linearly over the transmission length.

Concrete component of shear resistance

8.22 When the additive approach is used, the concrete component of shear resistance V_c shall be taken as

1) V_{co} as defined in Equation 8.22b when $M < M_{cr}$ according to Equation 8.22a;
2) the minimum of V_{co} as defined in Equation 8.22b and V_{cr} as defined in Equation 8.22c and 8.22d when $M \geq M_{cr}$.

Equation 8.22a Cracking moment

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

where:

f_{pt}	is the stress due to prestress only at the tensile fibre
y	is the distance to the tensile fibre from the centroid of the concrete section
I	is the second moment of area of the concrete section

Equation 8.22b Shear resistance of sections uncracked in flexure

$$V_{co} = 0.67bh\sqrt{f_t^2 + \sigma_{cp} f_t}$$

where:

f_t	is $0.32\sqrt{f_{cu}/\gamma_{mc}}$, and is taken as positive
b	is the breadth of the section, allowing for the presence of ducts. In flanged sections, the width of the web should be used
h	is the overall depth of the member
σ_{cp}	is the compressive stress in the longitudinal direction at the centroidal axis due to prestress and axial loads

Equation 8.22c Shear resistance of sections cracked in flexure (higher prestress)

$V_{cr} = V_{cr1}$ when $f_{pe} \geq 0.6 f_{pu}$

where:

V_{cr1}	is defined in Equation 8.22e, but not less than $0.12bd\sqrt{f_{cu}/\gamma_{mc}}$
d	is the distance from the compression fibre to the centroid of the tendons at the section considered
f_{pe}	is the assessment prestress

Equation 8.22d Shear resistance of sections cracked in flexure (lower prestress)

$V_{cr} = \max\begin{cases} V_{cr1} \\ V_{cr2} \end{cases}$ when $f_{pe} < 0.6 f_{pu}$

where:

V_{cr2}	is defined in Equation 8.22f, but not less than $0.12bd_s\sqrt{f_{cu}/\gamma_{mc}}$

Equation 8.22e Shear resistance of sections cracked in flexure

$$V_{cr1} = 0.045bd\sqrt{\frac{f_{cu}}{\gamma_{mc}}} + \frac{M_{cr}}{M/V - d/2}$$

where:

d	is the distance from the compression fibre to the centroid of the tendons at the section considered
M_{cr}	is the cracking moment
V, M	are the shear force and bending moment (both taken as positive) due to ultimate loads at the section considered

Equation 8.22f Shear resistance of sections cracked in flexure

$$V_{cr2} = \left(1 - 0.55\frac{f_{pe}}{f_{pu}}\right)V_{uc'} + \frac{M_0}{M/V - d_s/2}$$

where:

M_0	is the moment necessary to produce zero stress at the depth d , which should be taken from Equation 8.22g, but not greater than M_{cr}
f_{pe}	is the factored effective prestress
$V_{uc'}$	is taken as V_u obtained from Section 5, 6 or 7 but without including the contribution of the shear reinforcement V_{us} and with A_s 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

Equation 8.22g Moment for zero stress

$$M_0 = f_{pt}I/y$$

where:

f_{pt}	is the factored stress due to prestress only (including losses) at the depth d

8.22.1 The shear resistance of sections cracked in flexure should be checked for a scenario of maximum shear with coexistent bending effects, and also for maximum bending effects with coexistent shear.

8.22.2 Where both tensioned and untensioned steel are contained in A_s , f_{pe}/f_{pu} may be taken from Equation 8.22.2.

Equation 8.22.2 Prestress ratio

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

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 the untensioned steel
$f_{pu(t)}$	is the characteristic or the worst credible strength of the tensioned steel
$f_{yL(u)}$	is the characteristic or the worst credible strength of the untensioned steel

8.23 Where the position of a duct coincides with the position of maximum principal tensile stress, the value of the width shall be reduced by:

1) the duct width, for ungrouted ducts; or,
2) two-thirds of the diameter of the duct for fully grouted ducts.

8.24 For sections with inclined tendons, the component of the assessment prestressing force normal to the longitudinal axis of the member shall be added to V_{co} .

NOTE This component is positive when the resistance of the section is increased.

Shear reinforcement component of shear resistance

8.25 Where the provided shear reinforcement is not effective in resisting shear, the shear resistance shall be calculated using the requirements for sections without shear reinforcement.

8.25.1 Shear reinforcement should be taken as effective in resisting shear if all the following are satisfied:

1) the spacing of the legs of links, in the direction of the span and at right angles to it, does not exceed d_t or four times the thickness of the web;
2) the minimum shear reinforcement in Equation 5.7.1 is fulfilled;
3) the angle of the shear reinforcement from the longitudinal axis of the beam $\alpha \geq 30^o$;
4) the shear reinforcement satisfies the anchorage and bearing requirements of Section 9;
5) for structures containing only bonded reinforcement, the capacity of the longitudinal steel exceeds the criterion in Equation 8.25.1

Equation 8.25.1 Longitudinal capacity of steel

$$A_{st}\frac{f_{put}}{\gamma_{ms}} + A_{su}\frac{f_{yu}}{\gamma_{ms}} \geq \frac{M}{z} + \frac{V - V_{uc}}{2}(1 - \cot\alpha)$$

where:

d_t is the depth from the extreme compression fibre to either 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, whichever is greater

A_{st} is the area of tensioned steel

f_{put} is the characteristic strength or worst credible strength of the tensioned steel

A_{su} is the area of the untensioned steel

f_{yu} is the characteristic strength or worst credible strength of the untensioned steel

M and V are the coexisting assessment bending moment and shear force at the section under consideration

z is the lever arm which can be taken as $0.9d$

V_{uc} is the concrete component of the shear resistance of the section calculated using the rules for reinforced concrete in Section 5, 6 or 7, with A_s taken as the area of passive reinforcement plus any rigid bonded tendons

$\frac{V-V_{uc}}{2}$ may be taken as zero at internal supports of continuous beams

α is the angle of the shear reinforcement from the longitudinal axis of the beam

8.26 The component of shear resistance of vertical links shall be taken from Equation 8.26.

Equation 8.26 Component of shear resistance provided by effective shear reinforcement

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

where:

A_{sv} is the cross-sectional area of effective shear reinforcement at a particular cross-section

d_t is the depth from the extreme compression fibre to either 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, whichever is greater

f_{yv} is the yield strength of the shear reinforcement, taken as not greater than

1) 500 MPa for passive reinforcement;
2) for prestressed steel, the stress taken from the appropriate stress strain curve in Section 3, assuming a strain of 0.0045 above the prestrain, accounting for all losses and applying γ_{fp} .

8.26.1 Vertical prestress may be treated as prestrained reinforcement, with a design force not greater than that corresponding to a total strain of $0.0041 + \left(\frac{\epsilon_p}{1.15}\right)$, where ϵ_p is the prestrain in the tendon after all losses.

Assessment of shear in post-tensioned segmental construction

8.27 The shear force due to ultimate loads at joints which incorporate cast in situ concrete, dry-pack mortar or grout joint filler, shall not exceed V_{joint} in Equation 8.27:

Equation 8.27 Maximum shear force in segmental construction joints

$$V_{\text{joint}} = 0.7\gamma_{fL}P_h\mu_{\text{joint}}$$

where:

P_h is the horizontal component of the prestressing force after all losses

μ_{joint} is the effective coefficient of friction between joints in post-tensioned segmental construction. It can be taken as 0.7 for a smooth interface, and 1.4 for a roughened or castellated interface

8.28 Where segmental construction includes joints with only unbonded prestressing, within a distance h_{red} of a joint, the value of d_t in Equation 8.26 shall be taken from Equation 8.28.

Equation 8.28 Depth for segmental construction with unbonded prestressing

$$d_t = \min \left\{ \begin{array}{l} d_{t1} \\ d_{t2} \end{array} \right.$$

where

d_{t1} is the depth from the extreme compression fibre to either 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, whichever is greater

d_{t2} is the depth of concrete in compression under ultimate loads, but not less than half the overall section depth

8.29 The approach to the assessment of match-cast joints with shear keys shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

Torsion resistance of prestressed concrete

8.30 The requirements in Section 5 shall be used for the assessment of torsion in prestressed concrete sections.

8.31 When calculating v , in Equation 5.17a, d shall be taken as the depth to the tension reinforcement.

8.32 Where prestressing steel is used for the assessment of torsion, as either transverse torsional steel, or longitudinal torsional steel, the stress in the prestressing shall be taken from Equation 8.32, but no greater than f_{pu}/γ_{ms}

Equation 8.32 Stresses in prestressing steel used to resist torsion

$$f_{pe}+\frac{460}{\gamma_{ms}}$$

Prestressed concrete columns

8.33 The resistance of prestressed columns shall be assessed making allowance for the effects of prestressing except where the mean stress due to prestressing is lower than 2.5 MPa.

8.33.1 Where the mean stress in a concrete column due to prestressing is lower than 2.5 MPa, the resistance may be assessed using the provisions for reinforced concrete columns in Section 7.

Prestressed concrete tension members

8.34 The tensile strength of prestressed concrete tension members shall be based on the assessment strength of the prestressing tendons (f_{pu}/γ_{ms}) and the stress developed by any additional passive reinforcement.

8.34.1 The stress developed in the passive reinforcement may be assumed to be the assessment yield strength (f_y/γ_{ms}) .

NOTE The resulting strains in the prestressing and passive reinforcement can be calculated using the stress-strain relationships in Section 3, or manufacturers' data.

Transmission lengths

Transmission lengths in pre-tensioned members

8.35 Where the initial prestressing force in a tendon, is not greater than 75% of the characteristic strength of the tendon, and where the concrete strength at transfer is not less than 30 MPa, the transmission length, l_t , shall be taken from Equation 8.35.

Equation 8.35 Transmission length in pre-tensioned members

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

where:

f_{ci} is the concrete strength at transfer

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, based on manufacturers' data, or taken from Table 8.35.

Table 8.35 Transmission length coefficient for tendon type

Tendon Type	Value of k_t
Plain, indented and crimped wire with a total wave height less than 0.15ϕ	600
Crimped wire with a total wave height greater than or equal to 0.15ϕ	400
7-wire strand and super strand	240
7-wire drawn or compacted strand	360

8.36 Where the cover to pretensioned tendons is less than twice the nominal strand diameter, the effectiveness of the strand anchorage shall be reduced to take account of the effects of reduced cover and the presence or absence of enclosing links.

8.36.1 Where the cover to pretensioned tendons is less than twice the nominal strand diameter and the strand is enclosed by links, the transmission length should be divided by the cover factor, k_{cov} , defined in Section 9.

8.36.2 Where the cover to pretensioned tendons is less than twice the nominal strand diameter and the strand is not enclosed by links, the strand should not be assumed to be effective.

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

8.37.1 Where precast prestressed units, having pretensioned 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.

Assessment of details

Cover requirements

8.38 The cover to ducts shall not be less than 50 mm.

Assessment of End Blocks and Deviators

8.39 End blocks and deviators shall be assessed under the following situations:

1) end blocks, deviators and anchorages of unbonded tendons; and,
2) end blocks and anchorages of bonded tendons where signs of distress have been identified.

8.40 The bursting forces in end blocks, deviators or end regions of unbonded post-tensioned members shall be assessed at the ultimate limit state for a load equal to the characteristic strength of the tendon.

8.41 The bursting forces in end blocks or end regions of bonded post-tensioned members shall be assessed for the load effects in the tendon at the ULS.

8.42 Assessment of end blocks shall demonstrate:

1) sufficient resistance to contain bursting forces around individual anchorages;
2) overall equilibrium of the end block; and,
3) resistance to spalling from the loaded face around anchorages.

8.42.1 The assessment of end blocks should make explicit account of:

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 likely sequence of prestressing;
3) the shape of the end block relative to the shape of the member;
4) the layout of anchorages including asymmetry, group effects and edge distances;
5) influence of the support reaction; and,
6) forces due to curved or divergent tendons.

8.42.2 The bursting tensile force, F_{bst} , in an individual square end block that is loaded by a symmetrically placed anchorage or bearing plate, should be derived from Equation 8.42.2:

Equation 8.42.2 Bursting force

$$F_{bst} = P_k\left(0.32 - 0.3\frac{y_{po}}{y_o}\right)$$

where:

F_{bst}	is the tensile bursting force
P_k	is the load in the tendon
y_o	is half the side of the end block
y_{po}	is half the side of the loaded area

NOTE Equation 8.42.2 is valid for circular, square or rectangular anchor plates that are symmetrically positioned on the end face of a square or rectangular post-tensioned member.

8.42.3 The bursting tensile force, F_{bst} should be assumed to be distributed over a region extending from $0.2y_0$ to $2y_0$ from the loaded face of the end block.

8.42.4 Reinforcement in the bursting zone may be assumed to sustain the bursting force with a stress of f_y/γ_{ms} .

8.42.5 Where rectangular anchorage or bearing plates are present, the bursting tensile forces in the two principal directions should be calculated using Equation 8.42.2.

8.42.6 Where circular anchorage or bearing plates are present, the radius should be substituted for y_{po} in Equation 8.42.2.

8.42.7 Where groups of anchorages or bearing plates are present, the end blocks should be divided into a series of symmetrically loaded prisms, and each prism checked in accordance with Equation 8.42.2.

NOTE Specialist literature and advice can be consulted where end blocks have a different cross-sectional shape from the general cross-section in the beam.

8.43 Approaches that allow some portion of the bursting force to be carried by concrete in tension shall be subject to technical approval in accordance with CG 300 [Ref 7.N].

Interfaces

8.44 The interfaces between precast beams and in-situ slabs in composite construction shall be verified in accordance with the longitudinal shear requirements in Section 5.

External tendons

8.45 It shall be verified that external tendons have sufficient transverse restraint to avoid second order effects.

8.45.1 Tendons may be deemed to have sufficient transverse restraint if they are restrained relative to the concrete section at centres not exceeding 12 times the minimum depth of the beam between fixing points.

8.45.2 Where the spacing between points where the tendons are held in position laterally exceeds 12 metres, checks should be made to ensure that the first natural frequency of the tendons vibrating between fixing points is not in the range 0.8 to 1.2 times that of the bridge.

NOTE 1 Where external tendons are not adequately restrained within the concrete section, the deformation of the concrete between deviators can have a significant effect on the moment applied by the tendon to the concrete section.

NOTE 2 Inadequately restrained tendons can vibrate excessively and be susceptible to fatigue failure.

9. Bond, anchorage and bearing

Anchorage bond

9.1 The assessed tension or compression force in reinforcing bars at a section shall not exceed the anchorage resistance, provided by either:

1) the anchorage resistance F_{ub} over the effective anchorage length, as given in Equation 9.1a; or,
2) other effective anchorage arrangements verified by testing.

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub} p L_a$$

where:

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;

p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N) \sum (\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.

ϕ is the nominal bar diameter

L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

where:

k is taken as 1.0 for normal concrete, 0.8 for lightweight aggregate concrete, or 0.5 for lightweight aggregate concrete containing foamed slag

β is a coefficient dependent on bar type, and given in Table 9.1

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

γ_{mb} is a partial safety factor for bond given in Section 2

$k_{cov} = a_{con}\left(0.5 + \frac{c}{\phi}\right) \leq 1$ where $c > 0$

$k_{cov} = a_{con}\left(0.5 + \frac{c}{4\phi}\right)$ where $-0.5\phi \leq c \leq 0$

$k_{cov} = 0$ where $c < -0.5\phi$

a_{con} is taken as 0.65 for bars (or strand) that are enclosed by links. Where enclosing links are not present, it is taken as 0.40 for bars.

c is the cover from the surface of the bar to the surface of the concrete, but not taken as greater than half the clear spacing between bars

ϕ is the bar diameter, or effective strand diameter

Table 9.1 Bond coefficient depending on bar type

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

Note 1: Type 1 deformed bars include plain square twisted bars or plain chamfered square twisted bars, each with a pitch of twist not greater than 18 times the nominal diameter of the bar;
Note 2: Type 2 deformed bars include bars with transverse ribs with a substantial uniform spacing not greater than 0.8 times the nominal diameter of the bar (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 mm²/mm.

NOTE 1 *The bond model is based on theoretical consideration of a splitting mode of failure, which is supported for lower cover on the basis of experimental evidence.*

NOTE 2 *Bars with higher cover can have a higher bond strength governed by a pull-out failure mechanism. For further guidance, reference can be made to FIB Model Code [Ref 7.I].*

NOTE 3 *The derivation of the low cover reduction factor k_{cov} is based on a lower approximation to the mean of the available test data.*

NOTE 4 *Transmission lengths for prestressing strand are defined in Section 8.*

9.1.1 The bond strength in the length of reinforcement immediately over a bearing area may be enhanced by an additional 50%.

9.1.2 Where the analysis method includes an assumption of plastic behaviour at bond-critical sections, and the effective anchorage length is not sufficient to develop the yield strength, the bond strength should be reduced to account for the residual post-peak behaviour.

NOTE 1 *At higher values of bar slip, the force that can be resisted by anchorage bond can drop below the peak value to a lower residual value.*

NOTE 2 *The residual bond strength can vary depending on the bar type and cover, and in some cases can be assessed to be zero. For further guidance, reference can be made to FIB Model Code [Ref 7.I].*

Anchorage of links

9.2 A link shall be assumed to be fully anchored if either of the following are satisfied:

1) the link passes round another bar through an angle of 90° and continues beyond for a minimum length of eight times its own size;
2) the link passes round another bar through an angle of 150° and continues for a minimum length of four times its own size.

Laps and joints

9.3 The continuity of reinforcement laps and joints shall be assessed.

NOTE 1 *Continuity of reinforcement can be achieved using a variety of jointing methods including:*

1) *lapping bars;*
2) *butt welding;*
3) *sleeving;*
4) *parallel threading of bars and tapered threads; or,*
5) *other types of connection.*

NOTE 2 *Information on the design of laps and joints can be obtained from the design documentation (where available), by investigation (where carried out).*

9.3.1 Reinforcement joined by butt welding may be assumed to develop the full yield strength.

9.3.2 Unless lapping bars or butt welding has been used as reinforcement joining method, test evidence should be used to assess the strength and effectiveness of the joints.

9.4 Where bars are lapped, the strength of the lap shall be derived from the anchorage resistance of the smaller of the two bars over the provided lap length, reduced by the factor in Table 9.4 for bars in tension.

Table 9.4 Reduction factor for lapped bars in tension

Conditions	Reduction factor
The cover to the lapped bars from the top of the section as cast is less than twice the bar size.	1.4
The clear distance between the lap and another pair of lapped bars is less than 150 mm.	1.4
A corner bar is lapped and the cover to either face is less than twice the bar size.	1.4
The cover to the lapped bars from the top of the section as cast is less than twice the bar size, and the clear distance between the lap and another pair of lapped bars is less than 150 mm.	2.0
The cover to the lapped bars from the top of the section as cast is less than twice the bar size, and a corner bar is lapped with the cover to either face less than twice the bar size.	2.0

9.4.1 Where the lap length L is less than 15 times the size of the smaller of the two bars lapped, ϕ , the strength of the lap should be reduced by a factor $\frac{L}{15\phi}$.

Hooks and bends

9.5 The anchorage resistance of a hook or bend shall be determined from Equation 9.1a based on an effective anchorage length measured from the start of the bend to a point four times the bar size beyond the end of the bend.

9.6 The bearing stress inside a bend shall not exceed the value given by Equation 9.6a.

Equation 9.6a Ultimate bearing stress inside a bend for normal-weight aggregate concrete

$$\sigma_b \leq 5.63\left(\frac{a_b}{\phi}\right)^{\frac{1}{3}}\sqrt{\left(\frac{l}{l_1}\right)\frac{f_{cu}}{\gamma_{mc}}}$$

where:

σ_b is the bearing stress calculated from Equation 9.6b

a_b is taken as the centre-to-centre distance between bars or groups of bars perpendicular to the plane of the bend. Where the bar or group of bars is adjacent to the face of the member a_b is taken as no greater than the cover plus ϕ .

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

$\left(\frac{a_b}{\phi}\right)$ is taken as no greater than 8 unless verified by test evidence.

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

$\left(\frac{l}{l_1}\right)$ is taken as no greater than 3 unless verified by test evidence.

Equation 9.6b Bearing stress inside a bend

$$\sigma_b = \frac{F}{r\phi}$$

where:

F is the assessment tension force in a bar or group of bars

r is the internal radius of the bend

9.6.1 The limiting bearing stress given by Equation 9.6a should be reduced by the factor 0.67 for lightweight aggregate concrete.

9.6.2 Bearing stress verifications may be omitted for bends where:

1) the bar does not extend 4ϕ beyond the end of the bend;
2) the bar is assumed to be unstressed beyond a position 4ϕ from the end of the bend.

10. Specific structural elements and components

Infill concrete acting compositely with precast beams

10.1 In composite structures that comprise precast inverted T-beams and infill concrete, the component of shear resistance due to the infill concrete acting compositely with a beam shall be limited to no greater than $0.5V_c$, where V_c is the concrete component of shear resistance for the precast beam, or be confirmed through testing.

NOTE 1 The additive contributions of the precast beams and the concrete infill are based on the assumptions that there is transverse reinforcement placed through holes in the bottom of the webs of the units, and the units are infilled with concrete placed between and over the units to form a solid deck slab.

NOTE 2 The limit on the contribution of the infill is based on the scope of the information in TRL Project Report TRL PR/CE/130 [Ref 13.I].

10.1.1 The shear resistance of concrete infill that has no longitudinal reinforcement should be based on an effective depth $d = 0.9h$, where h is the average depth of the infill.

10.1.2 The shear resistance of the concrete infill should not include enhancement due to the effect of short shear spans.

NOTE Testing has shown that it can be unconservative to include enhancement of shear resistance due to the effect of short shear spans in the calculation of the shear resistance of the infill concrete as in The Concrete Bridge Development Group report TP5 [Ref 27.I].

Half joints and corbels

10.2 Half joints and corbels shall be assessed according to the requirements and guidance relating to structural assessment of half joints in CS 466 [Ref 6.N].

Freyssinet hinges

10.3 Freyssinet hinges shall be assessed according to CS 468 [Ref 1.N].

Deck hinges

10.4 Deck hinges shall be assessed according to the requirements relating to structural assessment in CS 467 [Ref 5.N].

Bearings

10.5 Bearings shall be assessed according to CS 459 [Ref 8.N].

10.6 Where there are no measures to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed $0.6 f_{cu}/\gamma_{mc}$.

10.7 Where measures have been provided to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed either of the following:

1) the value given in Equation 10.7a;
2) $1.5 f_{cu}/\gamma_{mc}$.

Equation 10.7a Ultimate bearing stress in contact area

$$f_{bc} = \frac{3(f_{cu}/\gamma_{mc})}{1 + 2\sqrt{A_{con}/A_{sup}}}$$

where:

A_{con} is the contact area

A_{sup} is the supporting area taken from Equation 10.7b

Equation 10.7b Supporting area for rectangular bearings

$$A_{sup} = (b_x + 2x)(b_y + 2y)$$

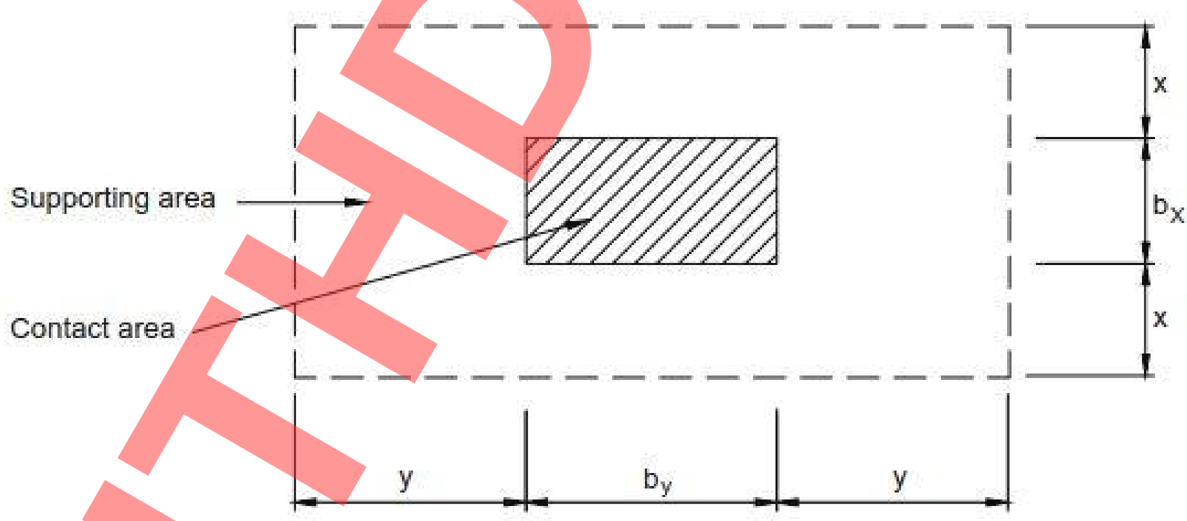
where:

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, as illustrated in Figure 10.7 but limited as below:

1) $x \leq b_x$
2) $y \leq b_y$

Figure 10.7 Bearing area for rectangular bearings



10.7.1 When the members are made of concretes of different strengths, the lower concrete strength should be assumed.

10.7.2 For lightweight aggregate concrete the ultimate bearing stresses due to ultimate loads should be limited to two-thirds of those for normal weight concrete.

11. Plain concrete walls and abutments

11.1 Plain concrete walls and abutments shall be assessed to resist the applied moments and forces.

NOTE The requirements for the assessment of plain concrete walls and abutments presented in this section are applicable for concrete walls and abutments with height not greater than five times their average thickness.

11.1.1 The axial force applied in a concrete wall or abutment may be calculated assuming that the beams and slabs transmitting forces into it are simply supported.

11.2 Where the resultant axial force in a member act eccentrically due to vertical loads not being applied at the centre of the member or due to the action of horizontal forces, the following eccentricities shall be taken into account in the assessment of plain concrete walls and abutments:

1) eccentricity in the plane of the wall or abutment;
2) eccentricity at right angles to walls or abutment.

11.2.1 Where the resultant of the axial loads act eccentrically in the plane of the member, linear distribution of load along the length of the member should be assumed.

11.2.2 Where the eccentricity of load in the plane of the member is zero, a uniform distribution of load may be assumed.

11.3 Tensile stresses shall not exceed the tensile strength.

11.3.1 The tensile strength of concrete should be taken as $0.12\sqrt{\frac{f_{cu}}{\gamma_{mc}}}$.

11.4 The effects of concentrated loads and bearings shall be assessed.

11.4.1 Concentrated loads may be assumed to be immediately dispersed if the local stress under the load does not exceed the bearing stress limits in Section 10.

11.5 For members restrained in position, the axial load per unit length of member, n_w , due to ultimate loads shall be limited according to Equation 11.5.

Equation 11.5 Axial limit load criterion for members restrained in position

$$n_w \leq \left(0.675\frac{f_{cu}}{\gamma_{mcw}}\right)(h - 2e_x)$$

where:

n_w is the 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, but not less than 0
$.05h$

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

11.6 The horizontal shear force in the plane of the member due to ultimate loads shall be either:

1) less than one quarter of the vertical load; or,
2) less than the force to produce an average shear stress over the whole cross-section of the member of:
    a) 0.45 MPa for $f_{cu} \leq$ 25 MPa;
    b) 0.30 MPa for $f_{cu} >$ 25 MPa.

12. Normative references

The following documents, in whole or in part, are normative references for this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Ref 1.N	Highways England. CS 468, 'Assessment of Freyssinet concrete hinges in highway structures'
Ref 2.N	Highways England. CS 454, 'Assessment of highway bridges and structures'
Ref 3.N	BSI. BS EN 1992-2, 'Eurocode 2. Design of concrete structures. Part 2: Concrete bridges. Design and detailing rules'
Ref 4.N	National Highways. GG 101, 'Introduction to the Design Manual for Roads and Bridges'
Ref 5.N	Highways England. CS 467, 'Risk management and structural assessment of concrete deck hinge structures'
Ref 6.N	Highways England. CS 466, 'Risk management and structural assessment of concrete half-joint deck structures'
Ref 7.N	Highways England. CG 300, 'Technical approval of highway structures'
Ref 8.N	Highways England. CS 459, 'The assessment of bridge substructures and retaining structures and buried structures'
Ref 9.N	Highways England. CD 360, 'Use of compressive membrane action in bridge decks'

13. Informative references

The following documents are informative references for this document and provide supporting information.

Ref 1.I	Applied Petrography Group. Eden, MA. SR2, 'A code of practice for the petrographic examination of concrete'
Ref 2.I	BSI. BS EN 13791, 'Assessment of in-situ compressive strength in structures and pre-cast concrete components'
Ref 3.I	BSI. BS EN 206, 'Concrete - specification, performance, production and conformity'
Ref 4.I	Bridge Design to Eurocodes – UK Implementation, ICE, London, 2011. Denton S, Shave J, Bennetts J, Hendy C. Denton et al 2011, 'Design of concrete slab elements in biaxial bending'
Ref 5.I	BSI. BS EN 1992-1-1, 'Eurocode 2: Design of concrete structures. General rules and rules for buildings'
Ref 6.I	BSI. BS EN 1990, 'Eurocode: Basis of structural design'
Ref 7.I	FIB - Fédération Internationale du Béton, 2010. The International Federation for Structural Concrete (FIB). FIB Model Code, 'FIB - Model Code for Concrete Structures'
Ref 8.I	Institution of Structural Engineers. ISE Prestressed, 'First Report on Prestressed Concrete'
Ref 9.I	BSI. BS 5896, 'High tensile steel wire and strand for the prestressing of concrete. Specification'
Ref 10.I	Highways England. CS 465, 'Management of post-tensioned concrete bridges'
Ref 11.I	Structures, Vol 5, Feb 2016 pp101-111. Beeby A, Jackson P. Beeby & Jackson 2016, 'Partial safety factor for reinforcement'
Ref 12.I	The Structural Engineer, 6 March 2007, pp30-37. Shave J. D., Ibell T.J., Denton S.R.. Denton et al 2007, 'Shear assessment of reinforced concrete bridges with short anchorage lengths'
Ref 13.I	Transport Research Laboratory. Cullington, DW, Hill, ME & Sykes, DP. TRL PR/CE/130, 'Shear capacity of pretensioned inverted T-beams with infill concrete.'
Ref 14.I	BSI. BS 4486, 'Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete'
Ref 15.I	BSI. BS 4757, 'Specification for nineteen-wire steel strand for prestressed concrete'
Ref 16.I	BSI. BS EN ISO 15630-3, 'Steel for the reinforcement and prestressing of concrete. Test methods. Prestressing steel'
Ref 17.I	BSI. BS EN ISO 15630-1, 'Steel for the reinforcement and prestressing of concrete. Test methods. Reinforcing bars, wire rod and wire'
Ref 18.I	BSI. BS 4449, 'Steel for the reinforcement of concrete. Weldable reinforcing steel. Bar, coil and decoiled product. Specification'
Ref 19.I	BSI. BS 5400-4, 'Steel, concrete and composite bridges. Code of practice for design of concrete bridges'
Ref 20.I	The Institution of Structural Engineers. ISE ASR 2010, 'Structural effects of alkali-silica reaction Technical guidance on the appraisal of existing structures – Addendum April 2010'

Ref 21.I	The Institution of Structural Engineers. ISE ASR 1992, 'Structural effects of alkali-silica reaction Technical guidance on the appraisal of existing structures – July 1992'
Ref 22.I	BSI. BS EN 12504-1, 'Testing concrete in structures. Cored specimens. Taking, examining and testing in compression.'
Ref 23.I	BSI. BS 1881-121, 'Testing concrete. Method for determination of static modulus of elasticity in compression'
Ref 24.I	BSI. BS EN 12390-3, 'Testing hardened concrete. Compressive strength of test specimens'
Ref 25.I	BSI. BS EN 12390-6, 'Testing hardened concrete. Tensile splitting strength of test specimens'
Ref 26.I	Highways England. CS 456, 'The assessment of steel highway bridges and structures'
Ref 27.I	The Concrete Bridge Development Group. Cullington DW, & Hill, ME. TP5, 'The effect of shear enhancement on the resistance of an infill deck'

Appendix A. Concrete structures affected by steel corrosion

A1 Nature and extent of corrosion

A1.1 Mechanisms of corrosion

Steel corrosion takes two main forms: local and general, with a variety of gradations between them.

Local corrosion takes place when there is considerable spatial variation in the distribution, at reinforcement level, of the chloride concentration and where conditions of moisture and oxygen supply are favourable. It is an electrochemical reaction and can lead to localised section loss by pitting corrosion. In some situations the corrosion product can be accommodated in the paste without disrupting the concrete cover. Because of this, the reaction can be difficult to detect visually.

General corrosion leads to much less local section loss but produces larger quantities of corrosion product, an expansive process which leads to spalling and rust staining. As the structural effects of the two types of corrosion are different, they are treated separately in this document. It should be noted, however, that the distinction is not completely clear cut. In particular, large amounts of local corrosion tend to lead to general corrosion. Consequently the presence of general corrosion should not be taken to indicate the absence of significant local corrosion.

A1.2 Corrosion of reinforcement

A1.2.1 General corrosion

General corrosion is normally apparent from rust staining, spalling and cracking. The major strength loss from general corrosion is due to the loss of bond and the spalling which it causes. The strength loss is not directly related to the reinforcement section loss and consequently it is not necessary to quantify the section loss due to general corrosion. However, if the strength of the structure is very sensitive to loss of steel area, it is desirable to expose some corroded bars to check that the steel is not suffering from significant loss of section.

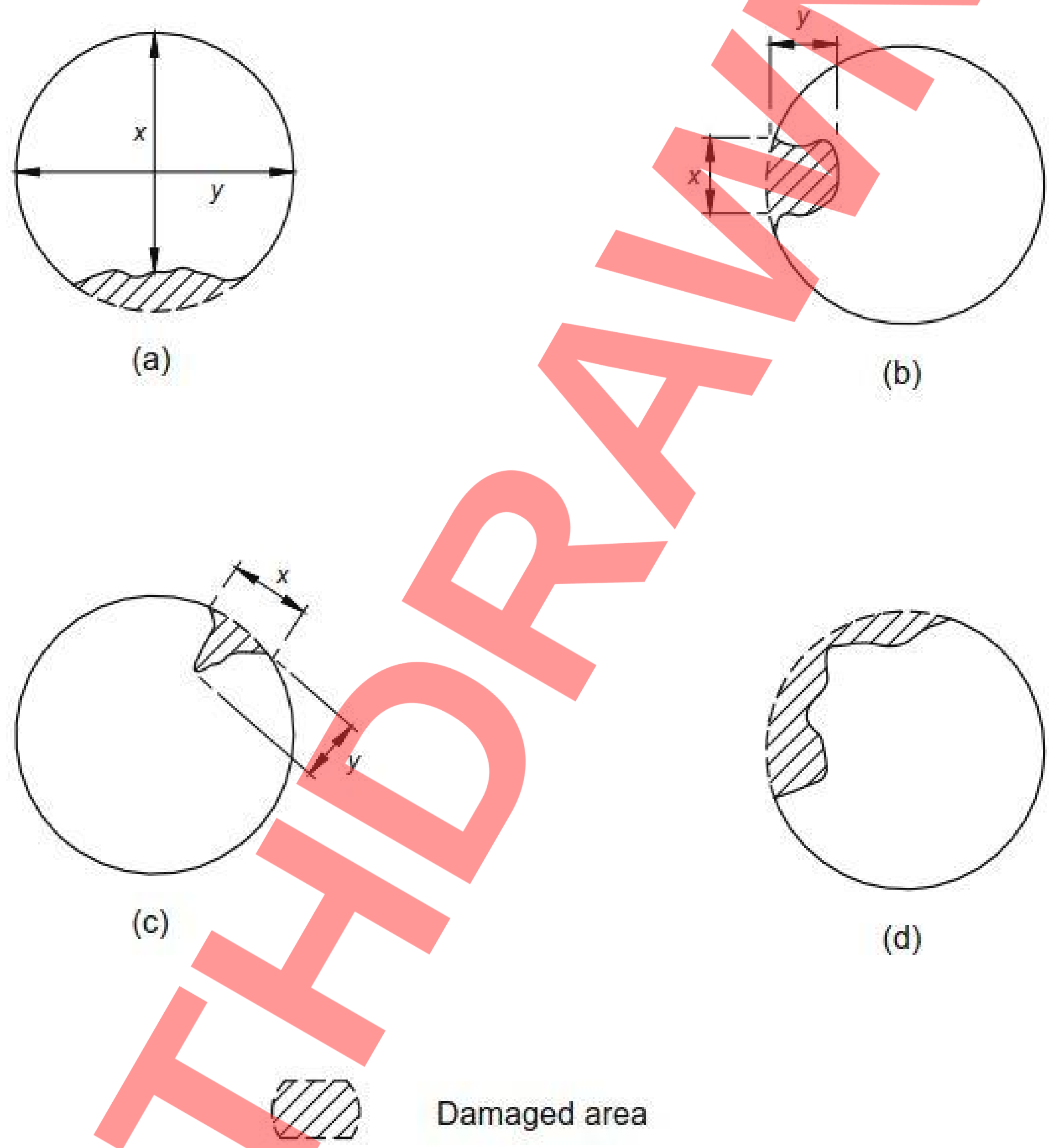
A1.2.2 Local corrosion

Local corrosion is difficult to detect. Indirect methods, such as half-cell potential, resistivity and chloride gradient readings, only give an indication of a risk of corrosion. Where these methods indicate a high risk and the structure is sensitive to local loss of section, some bars should be exposed to check if local corrosion is occurring. The section loss of the steel area should be estimated. Test results show that the loss of strength is not directly proportional to loss of section; it can be substantially less. For assessment purposes, the loss of strength may be assumed to be proportional to the estimated loss of area, or assessed by testing.

The effective cross-sectional area of corroded bars may be estimated using the following procedure:

1) To evaluate the loss of cross-sectional area for classification purposes the nominal and actual cross-section of the bar should be determined. Where drawings are available, the specified nominal diameter may be used to calculate the nominal area. In the absence of such information measurements at the undamaged and damaged sections of the bar should be taken to calculate the nominal and actual areas.
2) Prior to any measurements of the bar, all rust and loose concrete should be removed from a suitable length and the bar should then be cleaned down to bright metal. Areas of slight pitting which have a negligible effect on the surface may be ignored.
3) Where the nominal diameter is determined from the sample bar, two measurements of the width of the bar at 90 degrees to each other should be taken at an uncorroded section. The nominal area should then be based on the average of the measurements.
4) To calculate the actual cross-sectional area, measurements should be taken at the position of worst corrosion or mechanical damage on the sample length. The method of measurement of this area will depend on the shape of the area lost, as illustrated in Figure A.1.

Figure A.1 Measurements to determine cross sectional area of a corroded bar



Where the actual cross-section is as shown in Figure A.1(a) the actual cross sectional area may be taken as $A = \frac{\pi x y}{4}$ where x and y are measured at 90 degrees to each other and either x or y records the minimum width.

Figures A.1(b) and A.1(c) show a deeply pitted form of damage where measurements of the depth and width of the damaged area may be taken. From these measurements the loss of cross-sectional area can be determined.

Figure A1(d) illustrates an irregular boundary of damage which is not covered by Figures A.1(a), (b) or (c). Since measurements are difficult in this case an approximation may be made in evaluating the loss of cross-sectional area. Alternatively, corroded bars can be tested for strength. In this case, an uncorroded section of bar should also be tested. A relationship between estimated section loss of area and strength can then be established.

Although local corrosion may not reduce the static strength of bars as much as the loss of section suggests, it may have a disproportionately large effect on fatigue strength. Where bars subjected to significant live load stress range are suffering from local corrosion, the need for an assessment of fatigue strength and the methodology should be determined and recorded.

Local corrosion tends to be very variable throughout the length of the member. A random distribution of even quite severe local corrosion may have little effect on strength. However, a similar amount of corrosion concentrated at particular sections may have a large effect.

The assessment of corroded bars should include a sensitivity analysis to investigate how sensitive the assessment of the structure resistance is to the assumed degree of corrosion. This analysis should also be used to estimate the risk of further corrosion affecting the assessed capacity. The rate of corrosion should be estimated from the available data, including any previous investigations.

A1.3 Corrosion of prestressing tendons

Significant corrosion of pretensioned tendons, where the concrete is placed directly against them, is relatively rare and can be detected and quantified in the same way as that in reinforcement.

The corrosion of conventional ducted post-tensioned tendons is difficult to detect by direct methods. For methods of inspecting structures with grouted duct post-tensioning, see CS 465 [Ref 10.I]. It should be noted that the deflection of structures that are prestressed with bonded tendons is not sensitive to local corrosion. Because of this, deflection measurements, even in load tests, cannot be used as a means of detecting tendon corrosion except possibly in structures in which the tendons have become entirely unbonded.

Tendon force, and hence concrete stress, can be unaffected by loss of tendon area due to local corrosion until individual wires break. For this reason, and because bonded tendons can re-anchor at a certain distance from the section where local corrosion occurred, any measured loss of prestress attributed to loss of tendon area can underestimate of the loss of tendon strength. Allowance for this should be made in the assessment.

A2 Strength assessment

A2.1 General corrosion of reinforcement

Where general corrosion is very severe, sectional loss should be estimated as for local corrosion. In most cases, however, the loss of steel area can be insignificant. The loss of strength due to general corrosion is mainly due to loss of bond and also spalling. Tests show that the loss of bond strength is not significant until the point where longitudinal cracks form over the bars. General corrosion is most likely to occur, and most likely to cause cracks over the bars nearest the surface, in regions where cover is low. Tests show that bond strength is reduced in regions of low cover. For corroded bars where spalling is yet to occur, but longitudinal cracks have propagated over them, the bond stress should be reduced by 30%. Where spalling over corroded bars has reduced their cover, the bond strength should be reduced using the cover factor, k_{cov} , as provided in Section 9, in addition to the 30% allowance for corrosion.

When general corrosion becomes extensive, it can lead to spalling and delamination. Where the cover concrete has spalled or delaminated over a significant area, the structure should be assessed ignoring the cover concrete in those regions. The bond of bars which are in the plane of delamination should be ignored if they are exposed by more than half the bar diameter (since $k_{cov} = 0$, see Section 9).

A2.2 Local corrosion of reinforcement

Local corrosion can cause significant steel section loss. Unless test results indicate otherwise, it may be assumed that the strength loss is proportional to the estimated section area loss.

Individual elements can then be assessed for shear and flexural strength in accordance with this document, ignoring the steel area in both flexural and shear reinforcement that is assumed or known to be lost.

Local corrosion concentrates the area over which reinforcement yields. Because of this it effectively reduces the ductility of reinforcement. For this reason bars which are assumed to be suffering from local corrosion should not be assumed to be effective in plastic analyses, such as yield line analysis, unless tests on corroded bars show them to still have the ductility required by BS 4449 [Ref 18.I] for the equivalent grade of steel.

A2.3 Corrosion of prestressing tendons

In contrast to reinforcement where loss of bond and spalling determines the residual strength, it is the direct effect of tendon corrosion on the loss of steel section which is likely to be important when assessing prestressed structures. The prestressing force can be unaffected by local corrosion and consequently local failures in wires or strand may occur once the corroded tendon strength is reduced to the prestress force. For this reason, wires that are assumed to have lost more than 40% of their area locally should be assumed to be ineffective.

Fully grouted tendons, strands or wires that are ineffective locally, can reanchor and become effective elsewhere. In post-tensioned concrete construction, the anchorage length required can depend on the quality of grouting. Where the grouting is good and where the tendons are surrounded by effective links, the reanchorage length may be taken from Section 8, which gives a conservatively high estimate for the reanchorage length.

Where the grouting is poor, or where the links are not effective, assessments which depend on reanchorage should not be undertaken without special investigation.

Once the loss of effective tendon strength at a particular section has been estimated, the strength of the section can be calculated in the normal way.

A2.3.1 Slabs

The assessed carrying capacity of reinforced concrete slabs is not normally sensitive to localised corrosion. This is because their strength is a function of the average strength of reinforcement over significant widths of slab; they are not sensitive to the strength of individual bars. Unless there are reasons for believing that localised corrosion is concentrated at particular sections they do not normally require special consideration except when plastic analysis is used.

Because of the effects of membrane action, a phenomenon by which restrained reinforced concrete slabs fail at a load, which may be several times greater than simply supported slabs, the assessed carrying capacity of deck slabs in beam and slab bridges can be insensitive to reinforcement strength. Deck slabs should therefore not normally require special consideration unless there is evidence of delamination caused by general corrosion.

A2.4 Segmental construction

The tendons in segmental post-tensioned structures are particularly vulnerable to corrosion at the joints. This means that any tendon corrosion is likely to concentrate at these sections and consequently leads to a much greater risk of significant loss of strength than in monolithic structures. If the loss of tendon area at a particular section is known, the effect on the strength of the structure can be estimated from the loss of tendon area.

In bonded segmental structures, because the loss of effective prestress can be very localised, concentrated at joints, the overall stiffness of the structure is not significantly affected by tendon force loss. Deflection readings are not a good indicator of tendon force loss as there may be little effect on deflection until the structure is close to collapse.

A2.5 Future deterioration

In assessing the strength of a structure with corroded reinforcement or prestressing tendons, allowance should be made for possible future deterioration.

The assessment of corroded bars or tendons should include a sensitivity analysis to investigate how sensitive the assessment of the structure resistance is to the assumed degree of corrosion. This analysis should be used to estimate the risk of further corrosion affecting the assessed capacity. The rate of corrosion should be estimated from the available data, including any previous investigations and monitoring.

Where a structure has a significant level of reinforcement or tendon corrosion, or where there is a significant risk that further corrosion could result in the structure becoming substandard, remedial measures or monitoring should be proposed.

Appendix B. Structures affected by internal degradation of concrete

B1 Background

Concrete structures can be affected by a range of degradation mechanisms, including:

1) alkali aggregate reaction (AAR) (the most common being alkali silica reaction (ASR));
2) delayed ettringite formation (DEF);
3) thaumasite or other sulphate-associated degradation of concrete.

The degradation mechanisms affect the material properties of concrete, and can result in a progressive reduction in strength and stiffness over time. The degradation mechanisms are often collectively referred to as IDC (internal degradation of concrete). Sometimes in other documents thaumasite might not be included within the definition of IDC as the effects of thaumasite can be restricted to the cover zone, however this appendix covers the effects of all three degradation mechanism types.

The reduction in strength and stiffness can occur at different rates for different material properties, as illustrated in Table B.1. The relationships between the material properties that are usually assumed for concrete are therefore not valid for concrete affected by internal degradation. The resistance formulae for assessment of concrete structures in the main document are usually expressed in terms of concrete cube strength for convenience, however some of these resistance formulae can be invalid for concrete affected by internal degradation. For this reason the effect of internal degradation on the assessment of resistance is to be estimated based on the adjusted formulae as set out in this appendix with concrete material properties that are obtained from test data.

Table B.1 Typical lower bound residual mechanical properties for concrete affected by internal degradation as a percentage of values for unaffected concrete at 28 days

Free expansion	0.5 mm/m	1.0 mm/m	2.5 mm/m	5.0 mm/m	10.0 mm/m
Cube compression	100%	85%	80%	75%	70%
Uniaxial compression	95%	80%	60%	60%	*
Tension	85%	75%	55%	40%	*
Elastic modulus	100%	70%	50%	35%	30%
Note: Values in the table are based on ISE ASR 1992 [Ref 21.I] and ISE ASR 2010 [Ref 20.I].					

Where it is suspected that a structure is suffering from internal degradation of concrete, the presence and extent of degradation should be confirmed by investigation.

Internal degradation of concrete can be assumed to be present where material testing confirms that the worst credible strength values for tensile splitting and compressive cube strengths are in the ratio $\left(\frac{f_t}{f_{cu}}\right) \leq 0.06$ or where a petrographic examination of concrete samples positively identifies symptoms of internal degradation of concrete. Further guidance is provided in SR2 [Ref 1.I].

B2 Material properties

The material properties of concrete affected by internal degradation of concrete are to be obtained by testing. Worst credible strengths are to be determined for the concrete in its current condition based on the results of core testing. Worst credible strength values for tensile splitting strength f_t and cylinder strength f_c are to be established.

Compressive strength should be tested in accordance with BS EN 12390-3 [Ref 24.I] and extracted in accordance with BS EN 12504-1 [Ref 22.I] with tensile splitting carried out in accordance with BS EN 12390-6 [Ref 25.I]. For tensile testing, cores should be orientated similar to that found on site. Elastic modulus should be determined in accordance with BS 1881-121 [Ref 23.I].

B3 Assessment of resistance

The structure should be assessed using this document but f_{cu} should be replaced by:

1) $\left(\frac{f_t}{0.3}\right)^2$ for bond, torsion, cracking moment, transmission length;
2) $\frac{f_t}{0.11}$ for bearing stress inside bends and bearing stresses;
3) $\frac{f_c}{0.8}$ for other resistance models based on compression, flexure and shear behaviour.

Where the assessment of resistance is sensitive to the elastic modulus, the assessment should be carried out using the values for elastic modulus as determined from testing.

The effects of concrete delamination should be included explicitly in the resistance models, for example by including the actual cover in the calculation of bond strength.

Where the effects of internal degradation of concrete are modelled explicitly in the resistance models and in the values assumed for material properties, the condition factor is assumed to be 1.0 even for structures affected by internal degradation of concrete.