
**VOLUME 3 HIGHWAY STRUCTURES:
INSPECTION AND
MAINTENANCE**

SECTION 4 ASSESSMENT

PART 14

BD 44/15

**THE ASSESSMENT OF CONCRETE HIGHWAY
BRIDGES AND STRUCTURES**

SUMMARY

This Standard gives requirements and guidance for the assessment of existing concrete highway bridges and structures on motorways and other trunk roads. This is a revised document to be incorporated into Design Manual for Roads and Bridges. It supersedes and replaces BD 44/95 and BA 44/96.

INSTRUCTIONS FOR USE

1. Remove existing contents pages for Volume 3 and insert new contents pages for Volume 3 dated August 2015.
2. Remove BD 44/95 and BA 44/96 from Volume 3, Section 4 which are superseded by this Standard and archive as appropriate.
3. Insert BD 44/15 into Volume 3, Section 4, Part 14.
4. Archive this sheet as appropriate.

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HIGHWAYS ENGLAND

BD 44/15
Volume 3, Section 4,
Part 14



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THE DEPARTMENT FOR REGIONAL DEVELOPMENT
NORTHERN IRELAND

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**THE ASSESSMENT OF CONCRETE
HIGHWAY BRIDGES AND STRUCTURES**

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NOT FOR CONSTRUCTION

1 INTRODUCTION

Background

- 1.1 This Standard gives requirements and guidance for the assessment of existing concrete highway bridges and structures and their structural elements, and must be used in conjunction with BD 21 (DMRB 3.4.3), BD 37 (DMRB 1.3), BD 86 (DMRB 3.4.19) and other relevant documents. It supersedes and replaces BD 44/95 and BA 44/96.
- 1.2 Appendix A of this Standard contains the relevant assessment clauses which have been presented in the same format as the design clauses in BS 5400-4. The assessment clauses have been specifically developed to suit assessment conditions and therefore must not be used in design or construction of new structures. It is noted that while BS 5400-4 has been withdrawn and replaced by BS EN 1992-2 (the Eurocode for concrete bridges) for design purposes, the Eurocodes do not as yet include provisions for assessment. The commentary is contained alongside the assessment clauses. It contains explanations for the main changes from BS 5400-4, and gives advice on the interpretation of the assessment requirements. Also included are comments and references which provide additional information appropriate to special situations. Where such situations arise, any special method of analysis or variation of criteria proposed for an assessment must be agreed with the Overseeing Organisation.
- 1.3 In Appendix A of this Standard, any reference to characteristic strength must be to characteristic strength or to worst credible strength as appropriate.
- 1.4 The major changes to the Standard compared with the earlier version are as follows:
 - a) A single BD with mandatory clauses including commentary/guidance replacing the previous BD and BA 44.
 - b) Incorporation of the requirements of BD 24 (DMRB 1.3.1), IAN 4/96 and IAN 5/96, which are withdrawn, and updating of superseded references in a number of clauses.
 - c) Updated guidance for calculating the worst credible strength in line with the approaches of BS EN 13791 and BS 6089.
 - d) Explicitly added requirements for SLS (Serviceability Limit State) checks for prestressed structures and clarified loads and partial factors to be used when SLS checks are to be performed.
 - e) A reduced value of the partial safety factor to be applied to grade 460 reinforcing steel at the ultimate limit state.
 - f) Updated methodology for assessing the shear capacity at support of beams with theoretically inadequately anchored longitudinal reinforcement.
 - g) Added provisions for the assessment of structures with external and/or unbonded prestressing.
 - h) Clarifications made to a number of clauses and alternative approaches permitted which incorporate provisions of the Eurocode for concrete structures (BS EN 1992-1-1 and BS EN 1992-2).

Scope

- 1.5 This Standard gives requirements for the assessment of existing concrete highway bridges and structures on trunk roads and motorways.

Mutual Recognition

- 1.6 The requirements and guidance in this Standard only cover the assessment of existing structures and structural elements and are given on the basis that any construction and maintenance of concrete highway bridges and structures will be carried out using the Specification for Highway Works (SHW, Vol.1). However, products conforming to equivalent standards and specifications of other member states of the European Economic Area and tests undertaken in other member states may be acceptable in accordance with the terms of the 104 and 105 series of clauses of that Specification.

Implementation

- 1.7 This Standard must be used forthwith on all projects for the assessment of concrete highway bridges and structures on motorway and all-purpose trunk roads (and on all roads in Northern Ireland). The requirements must be applied to assessment already in progress provided that, in the opinion of the Overseeing Organisation, its use would not result in significant additional expense or delay progress (in which case the decision must be recorded in accordance with the procedure required by the Overseeing Organisation).

2 ASSESSMENT OF STRENGTH

General

- 2.1 A key objective of this Standard is to produce a more realistic assessment of the strength of a concrete element than can be produced using the design code. This is in part achieved by taking advantage of the information available to an assessor, such as material strength, which can only be predicted at the design stage.
- 2.2 Many of the criteria given in the design code are based on experimental evidence which in some cases have been either conservatively interpreted for use in design or updated by later evidence allowing a less conservative interpretation. For assessment purposes such criteria have been reviewed and amended where appropriate.
- 2.3 An important feature of BS 5400-4 is the application of the partial safety factor for material strength γ_m to the characteristic values. This approach is retained in Appendix A but the concept of worst credible strength with a reduced value of γ_m is introduced as an alternative.

Worst Credible Strength

- 2.4 The term *worst credible strength* is used in this Standard to allow for the actual material strengths of the structural elements being used for assessment. Worst credible strength can be defined as the worst value of that strength which the assessor, based on experience and knowledge of the material, realistically considers could be obtained in the structural element under consideration. This value may be greater or less than the characteristic strength of the material assumed at the design stage. Since this value eliminates some of the uncertainties associated with the use of characteristic strengths, reductions may be made in the partial safety factor for material γ_m .
- 2.5 The method of determining the worst credible strength must be agreed with the Overseeing Organisation.
- 2.6 Worst credible strengths should be used in the following circumstances:
 - a) when an initial assessment using characteristic values has shown an element of structure to be incapable of carrying the full assessment loading of BD 21 (DMRB 3.4.3).
 - b) if a structure has suffered damage or deterioration in such a way that the actual strengths are known or thought to be less than the assumed characteristic values.
 - c) where no information exists on the characteristic values used in design and it is not possible or appropriate to adopt an assumed value from BD 21 3.4(DMRB.3).
- 2.7 Worst credible strengths should generally be derived from tests on concrete cores or steel samples. However, it would be desirable to undertake assessments, or initial assessments, without undertaking the material tests required to determine worst credible strengths. Advice on the strengths to be assumed in the absence of definite information is given in BD 21 (DMRB 3.4.3). Where concrete strength is specified in terms of 28 day minimum cube strength, this should be considered as being equal to characteristic cube strength. Where concrete is specified in terms of prescribed concrete mixes these may be used to estimate strengths which should also be considered as equivalent to characteristic cube strengths. In estimating the strength of prescribed mixes, it is important to use judgement based on contemporary or older information as there has been a progressive increase in the strength of concrete with similar proportions due to increases in the reactivity of cement.
- 2.8 If construction records include standard 28 day cube results for concrete which is representative of the critical areas, these may be used. They should be processed in the same way that core results are processed

to obtain worst credible values, see 2.10 below. Because of the difference between wet cured cubes and in situ concrete, the uncertainty is higher than for in situ core tests so the representative value obtained from records should be treated as a characteristic, rather than a worst credible value. Caution will be needed in using the results of non-standard cube tests. Older age tests and tests on cubes stored with the structure may be used. However, because the early age strength gain of cements varies significantly, extrapolation of early age cube results is not normally reliable. Similarly, extreme caution is required in using cube results at ages substantially greater than 28 days as they may show increases in strength which are not achieved in the real structure.

- 2.9 Where assessment using the above approaches does not give the required strength, consideration should be given to taking samples and using worst credible strength values instead. However, before undertaking the tests, the sensitivity of the assessed strength to assumed material strength should be investigated. In particular, the strength of lightly reinforced members is often very insensitive to the strength of concrete.
- 2.10 When samples are required, worst credible strength for concrete must be derived from tests carried out on cores. Cores are destructive and cannot normally be taken at the critical (most highly stressed) locations of an element, hence a reliable and realistic method based on the worst credible strengths data from adjacent locations should be used to obtain worst credible strengths in the critical locations. To assist in determining the statistically most reliable results of core tests, an integrated programme of testing which may include destructive, semi-destructive (e.g. near surface tests) and non-destructive tests will be necessary for each element. The assessor should use judgement in selecting the locations and numbers of samples for such tests. Methods of assessing the estimated in-situ concrete strength at a location are given in BS EN 13791 and BS 6089. A location is a region where, in the assessor's judgement, there is no more than the normal random variation in concrete strength. Information on the accuracy of the assessed value is also given. The worst credible strength at a location may be taken as the lower bound to the estimated mean in situ concrete strength; e.g. for n concrete cores giving equivalent cube strengths of $f_{c1} \dots f_{cn}$ and a sample standard deviation of s , and for a confidence level of 90% (corresponding to a fractile of 0.05 on a one-sided region of a Student's t-distribution statistical curve), the worst credible strength is equal to $[(\sum f_{ci}/n) - t_{0.05}(s/\sqrt{n})]$, where the value of $t_{0.05}$ is a function of the number of cores n and can be taken from BS 6089 or standard statistical t-tables. In applying this formula, the assessor must be satisfied that the cores are representative of the location under consideration. The formula only accounts for errors in estimating the mean strength of concrete in the location considered. In practice, due to variations in water content, curing conditions and compaction, the strength of concrete can vary significantly over short distances. It is therefore necessary to use caution in interpreting the results of tests. When either the appearance of the concrete or the test results themselves suggest the variation of strength is significantly greater than the formula assumes it is necessary to use judgement based on the actual variation or on the minimum value recorded to obtain a more realistic estimate. The worst credible strength must be based on a minimum of three cores.
- 2.11 If the assessor wishes to use a single worst credible concrete strength for the structure as a whole, rather than individual values at individual critical locations, it will be necessary to determine the number and location of cores required to produce a representative value for the in situ concrete strength. The sampling rate should however not be less than one core for each 50 m³ of concrete. The worst credible strength may be taken as either the least of the individual values or derived in accordance with the approach described above.
- 2.12 When samples are required for reinforcement or prestressing tendons and bars, the worst credible strength should be obtained by testing samples taken from the element being assessed. However, it is often impractical to extract samples from critical sections and the assessor should use judgement in selecting the locations and numbers of samples for such tests. Removal of prestressing steel for sampling purposes will alter the stress distribution in the concrete section, and this must be allowed for in the assessment calculations. In choosing lengths of bars for testing, the assessor must check that the removal of samples will not significantly reduce the load carrying capacity of the element under consideration. Testing of steel reinforcement and prestressing strand should generally be carried out to the requirements of BS EN ISO 15630-1 and BS EN ISO 15630-3,

respectively. In accessing tendons and bars for testing, methods of concrete removal that minimise risk of mechanical damage to bars must be used. Also, when cutting out the chosen samples of bars, adjacent bars which may be highly stressed must not be damaged.

- 2.13 In the case of steel reinforcement or prestressing tendons, the worst credible strength may be taken as the lower bound to the mean in situ steel strength based on a 98% confidence level (corresponding to a fractile of 0.01 on a one-sided region of a Student's t-distribution statistical curve) calculated with the formula for concrete cores given above and the corresponding value of $t_{0.01}$ taken from standard statistical t-tables. In determining the steel strength, the assessor must be satisfied that the samples tested are representative of the location under consideration.
- 2.14 When using a worst credible steel strength in excess of the characteristic strength, the assessor must check that the bar anchorages and laps are capable of developing the higher steel stresses. The assessor's knowledge of material strengths which were typical of the period of construction may in some instances aid consideration of appropriate values for the structure under consideration.
- 2.15 For non-conforming details, or details not covered by the code, laboratory testing can be used to determine behaviour and strength. In some cases, test data may already be available.

Partial Safety Factor for Materials

- 2.16 The values of γ_m for concrete and reinforcing or prestressing steel are given in Appendix A Table 4A. The values for use with the characteristic strength may be different from those for use with worst credible strengths. To enable the correct value of γ_m to be used, all limiting criteria have been expressed as formulae with γ_m stated explicitly rather than as tabulated values.

Application of Reliability Techniques

- 2.17 As part of the assessment process, reliability techniques may be used subject to the agreement of the Overseeing Organisation. The full application of reliability techniques to bridge assessment requires specialist knowledge and experience. Reference to BD 79 (DMRB 3.4.18) should be made for further guidance.

Deterioration and Condition Factors in BD 21 (DMRB 3.4.3)

- 2.18 Specific guidance on assessment of structures which have deteriorated due to Alkali Silica Reaction (ASR) and reinforcement corrosion is given in BA 52 (DMRB 3.4.10) and BA 51 (DMRB 3.4.13), respectively. Reinforcement corrosion can also have a disproportionate effect on fatigue life and this is considered in BA 38 (DMRB 3.4.5).
- 2.19 While the application of the condition factor F_c in BD 21 (DMRB 3.4.3) is not affected in principle by the requirements of this Standard, the estimated values of F_c must not allow for any deficiencies of the materials in a structure which are separately allowed for by using worst credible strengths or by making other allowances. Where possible, specific allowances for deterioration should be made directly, such as loss of material strength or of material in assessment calculations, and a condition factor of 1.0 used.

WITHDRAWN

3 LIMIT STATES

General

- 3.1 Assessments must be carried out at the ultimate limit state in accordance with BD 21 (DMRB 3.4.3). Checks at the serviceability limit state must be carried out only in the limited number of cases required by this Standard, or when explicitly required by the Overseeing Organisation.
- 3.2 Serviceability assessment may be useful for investigating the significance of visible signs of deterioration, such as excessive cracking, considering factors such as deflection, fatigue and durability.

Ultimate Limit State (ULS)

- 3.3 The assessment must check that collapse or failure of a structural member will not occur under the assessment loads as a result of rupture of one or more critical sections, by overturning or by buckling caused by elastic or plastic instability, having due regard to the effects of sway when appropriate.
- 3.4 The effects of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state.

Serviceability Limit State (SLS)

- 3.5 The need for a serviceability limit state assessment and the necessary criteria for that assessment must be agreed with the Overseeing Organisation. Unless otherwise agreed with the Overseeing Organisation, the only SLS check required is to determine that the stresses in prestressed concrete do not exceed the limits given in 6.3.2 of Appendix A.
- 3.6 Where the Overseeing Organisation requires other serviceability limit state checks, the criteria should generally be those specified in Appendix A.

WITHDRAWN

4 USE OF APPENDIX A AND BS 5400-4:1990

- 4.1 Appendix A is based on BS 5400-4 and retains the terminology and clause numbering of that document. In cases where the BS 5400-4 clauses are not required for assessment, the numbers and headings of those clauses have been included to retain the structure of the document but the words “Not applicable to assessment” have been added in italics. Advisory clauses have been provided with a suffix ‘A’.
- 4.2 Wherever possible, tabulated values in BS 5400-4 have been replaced by formulae in which γ_m is stated explicitly. Modified tables, figures and equations retain their BS 5400-4 numbers.
- 4.3 Comments are given on those clauses where the changes from BS 5400-4 are substantial or are not self evident. There are also comments on some of the clauses in Appendix A which have been marked as “Not applicable to assessment”. A number of these relate to serviceability criteria which should only be included in an assessment on the direction of the Overseeing Organisation.
- 4.4 When, in Appendix A, reference is made to other clauses within Appendix A, the referenced clause number is given without any suffix. When, in Appendix A, reference is made to Chapters 1 to 5 of BD44, the referenced clause numbers are provided with the suffix ‘B’.

WITHDRAWN

5 REFERENCES

5.1 Normative References

Design Manual for Roads and Bridges (DMRB)

DMRB Volume 1: Section 3: General design

BD 37 Loads for Highway Bridges
(DMRB 1.3)

DMRB Volume 3: Section 4: Assessment

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

BD 86 The Assessment of Highway Bridges and Structures for the Effects of Special Types General Order (STGO) and Special Order (SO) Vehicles
(DMRB 3.4.19)

British Standards (BS): BRITISH STANDARDS INSTITUTION

BS 5400-4:1990. Steel, concrete and composite bridges. Code of practice for design of concrete bridges

5.2 Informative References

Design Manual for Roads and Bridges (DMRB) and Manual of Contract Documents for Highway Works (MCHW) : HMSO

DMRB Volume 3: Section 4: Assessment

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

BA 51 The Assessment of Concrete Structures Affected by Steel Corrosion
(DMRB 3.4.13)

BA 52 The Assessment of Concrete Structures Affected by Alkali Silica Reaction
(DMRB 3.4.10)

BD 79 The Management of Sub-standard Highway Structures
(DMRB 3.4.18)

MCHW Volume 1 – Specification for Highway Works

British Standards (BS): BRITISH STANDARDS INSTITUTION

BS 6089:2010. Assessment to in situ compressive strength in structures and precast concrete components – Complementary guidance to that given in BS EN 13791

BS EN 1992-1-1:2004. Eurocode 2. Design of concrete structures – Part 1-1: General rules and rules for buildings (incorporating amendments 1 and 2, 2011)

BS EN 1992-2:2005. Eurocode 2. Design of concrete structures – Part 2: Concrete bridges – design and detailing rules (incorporating amendment 1, 2010)

BS EN 13791:2007. Assessment of in situ compressive strength in structures and precast concrete components

BS EN ISO 15630-1:2010. Steel for the reinforcement and prestressing of concrete. Test methods. Reinforcing bars, wire rod and wire

BS EN ISO 15630-3:2010. Steel for the reinforcement and prestressing of concrete. Test methods. Prestressing steel

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ANNEX A: REFERENCES

WITHDRAWN

1. SCOPE

1.1 Refer to 1.5B.

2. DEFINITIONS AND SYMBOLS

2.1 Definitions

2.1.1 General. For the purposes of this Standard the definitions given in BD 21 (DMRB 3.4.3) apply. All formulae are based on SI units in Newtons and millimetres unless otherwise stated.

2.1.2 Partial load factors. For the sake of clarity the factors that together comprise the partial safety factor for loads are restated as follows. Assessment loads, Q_A^* , are obtained by multiplying the nominal loads, Q_k^* , by γ_{fl} , the partial safety factor for loads. γ_{fl} is a function of two individual factors, γ_{f1} and γ_{f2} , which take account of the following:

γ_{f1} possible unfavourable deviations of the loads from their nominal values;

γ_{f2} reduced probability that various loadings acting together will all attain their nominal values simultaneously.

The relevant values of the function γ_{fl} ($= \gamma_{f1} \gamma_{f2}$) are given in BD 21 (DMRB 3.4.3), BD 37 (DMRB 1.3), BD 86 (DMRB 3.4.19) and BD 48 (DMRB 3.4.7) as appropriate.

The assessment load effects, S_A^* , are obtained from the assessment loads by the relation:

$$S_A^* = \gamma_{f3} (\text{effects of } Q_A^*) = \gamma_{f3} (\text{effects of } \gamma_{fl} Q_k^*)$$

where

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

The values of γ_{f3} are given in 4.2.

2.1.3 Materials

2.1.3.1 Strength. Material strengths are expressed in terms of the cube strength of concrete, f_{cu} , the yield or proof strength of reinforcement, f_y or the breaking stress of prestressing tendon, f_{pu} . The material strengths used may be either:

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

(b) Worst credible strength, refer to 2.4B to 2.15B.

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

2.2 Symbols

The symbols used in Appendix A are as follows:

A_c	area of concrete
A_{con}	contact area
A_e	area of fully anchored reinforcement per unit length crossing the shear plane
A_{lv}	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_{sup}	supporting area
A_{sv}	cross-sectional area of the legs of a link
A_t	area of transverse reinforcement
a'	distance from compression face to a point at which the crack width is being calculated
a_b	centre-to-centre distance between bars
a_{cr}	distance from the point (crack) considered to the surface of nearest longitudinal bar
a_v	distance from the section under consideration to the supporting member; 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
$(EI)_c$	flexural rigidity of the column cross-section
e_x	resultant eccentricity of load at right-angles to plane of wall
F_{bst}	tensile bursting force
F_{bt}	tensile force due to ultimate loads in a bar or group of bars
F_{ub}	ultimate anchorage capacity of tension reinforcement
$F_{ub\ max}$	anchorage capacity required for fully anchored behaviour
f	stress

f_{cav}	average compressive stress in the flexural compressive zone
f_{ci}	concrete strength at (initial) 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
σ_{cp}	mean compressive stress in the concrete due to the prestressing and axial loading
f_{cu}	characteristic or worst credible concrete cube strength
f_{pb}	tensile stress in tendons at (beam) failure
f_{pe}	effective prestress (in tendon)
f_{pt}	stress due to prestress
f_{pu}	characteristic or worst credible strength of prestressing tendons
f'_s	stress in reinforcement in compression, taken as equal to $f_y/(\gamma_{ms} + f_y/2000)$
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_{yL}	characteristic or worst credible strength of longitudinal reinforcement
f_{yv}	characteristic or worst credible strength of shear reinforcement
h	overall depth (thickness) of section (in plane of bending)
h_f	thickness of flange
h_{max}	larger dimension of section
h_{min}	smaller dimension of section
h_{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	factor depending on the type of duct or sheath used; reduction factor for the shear resistance of voided slabs
k	constant (with appropriate subscripts)
k_t	depends on the type of tendon
k_l	depends on the concrete bond across the shear plane
L_s	length of shear plane
l_e	effective height of a column or wall
l_{ex}	effective height for bending about the major axis
l_{ey}	effective height for bending about the minor axis
l_o	clear height of column between end restraints
l	thickness of concrete member in the plane of a bent reinforcing bar
l_l	length of reinforcing bar measured inside the bend and bearing onto the concrete
l_t	transmission length
M	bending moment due to ultimate loads
$M_{\alpha,i}^*$	moment of resistance of a slab due to the i -direction reinforcement to the x -axis
M_{cr}	cracking moment at the section considered
M_g	moment due to permanent loads
M_i	maximum initial moment in a column due to ultimate loads
M_{ix}	initial moment about the major axis of a slender column due to ultimate loads
M_{iy}	initial moment about the minor axis of a slender column due to ultimate loads
M_q	moment due to live loads
M_{tx}	total moment about the major axis of a slender column due to ultimate loads
M_{ty}	total moment about the minor axis of a slender column due to ultimate loads
M_u	ultimate resistance moment

M_{ux}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the major axis only
M_{uy}	ultimate moment capacity in a short column assuming ultimate axial load and bending about the minor axis only
M_x, M_y	moments about the major and minor axes of a short column due to ultimate loads; moments about the axes of a slab
M_x, M_y	moments of resistance of a slab about the axes in-plane
M_{xy}	torsional moment about the axes of a slab
M_{xy}^*	torsional moment of resistance of a slab
M_n	moment about an axis perpendicular to the n-direction in a slab
M_n	moment of resistance of a slab about an axis perpendicular to 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	ultimate resistance of a slab in compression (per unit length)
N_i^*	ultimate tensile resistance of a slab due to the i-reinforcement (per unit length)
N_x, N_y	in-plane axial forces in a slab
N_{xy}	in-plane shear force in a slab
N_u	ultimate resistance axial load
N_{uz}	axial loading capacity of a column ignoring all bending
n_w	ultimate axial load per unit length of wall
P_f	effective prestressing force after all losses
P_h	horizontal component of the prestressing force after all losses
P_k	basic load in tendon
P_o	prestressing force in the tendon at the jacking end (or at tangent point near jacking end)
P_x	prestressing force at distance x from jack
Q_A^*	assessment load
Q_k^*	nominal load
R_A^*	Assessment reaction at support
r	internal radius of bend
r_{ps}	radius of curvature of a tendon
S	first moment of area of the part of the section excluding any area below the location being checked, calculated about the centroidal axis of the whole section
S_A^*	assessment load effects
s_v	spacing of links along the member
u_0	perimeter of the loaded area for a concentrated load
t	breadth of bearing area over a support
T	torque due to ultimate loads
T_u	ultimate torsional strength
V	shear force due to ultimate loads
V_c	ultimate shear resistance of concrete
V_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_p	shear capacity of precast prestressed section
V_l	longitudinal shear force due to ultimate load

V_s	shear resistance of shear reinforcement
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	ultimate shear resistance of section; ultimate punching shear capacity of a slab
V_{ux}	ultimate shear capacity of a section for the x-x axis
V_{uy}	ultimate shear capacity of a section for the y-y axis
V_x	applied shear due to ultimate loads for the x-x axis
V_y	applied shear due to ultimate loads for the y-y axis
v	shear stress
v_u	ultimate shear stress in concrete (for half joints)
v_c	ultimate shear stress in concrete
v_l	ultimate longitudinal shear stress per unit area of contact surface
v_t	torsional shear stress
v_{tmin}	minimum ultimate torsional shear stress above which reinforcement is required
v_{tu}	ultimate torsional shear stress
x	neutral axis depth; distance from jack
x_l	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_l	larger centre-line dimension of a link
z	lever arm
α	inclination of shear reinforcement to the member axis; factor to determine f_{pb}
α_l	angle between the axis of the design moment and the direction of the tensile reinforcement
α_2	angle of friction at the joint
α_{cw}	coefficient taking into account of the state of the stress in the compression chord
α_n	coefficient as a function of column axial load
α_s	factor that takes account of the increased bond strength due to transverse pressure
β	coefficient dependent on bar type
$\gamma_{f1}, \gamma_{f2}, \gamma_{f3}$	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 safety for concrete
γ_{mcw}	partial safety for plain concrete wall
γ_{ms}	partial safety factor for steel
γ_{mv}	partial safety factor applied to v_c
ϵ	strain
ϵ_m	average strain
ϵ_s	strain in tension reinforcement
ϵ_l	strain at level considered
μ	coefficient of friction
ξ_s	depth factor
ρ	geometrical ratio of reinforcement, generally equal to A_s/bd
ρ_{net}	ρ for a flange
ΣA_{sv}	area of shear reinforcement
Σbd	area of the critical section for punching shear
ϕ	size (nominal diameter) of bar or tendon
φ	diameter of the void in voided slabs
Γ	reduction factor for short anchorage

κ	shear enhancement factor for short shear spans
σ	stress, pressure
σ_r	stress level for fatigue life
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
v	strength reduction factor for concrete cracked in shear

3. LIMIT STATE PHILOSOPHY

3.1 **General.** Refer to 3.1B to 3.6B.

4. ASSESSMENT: GENERAL

4.1 **Limit state requirements.** Refer to 3.1B to 3.6B.

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

4.2.1 Loads. The nominal values of loads are given in BD 21 (DMRB 3.4.3), BD 37 (DMRB 1.3), BD 86 (DMRB 3.4.19) and BD 48 (DMRB 3.4.7) as appropriate. However, Type HB loading to BD 37 (DMRB 1.3) must only be used when required by the Overseeing Organisation, as Special Type General Order (STGO) and Special Order (SO) vehicles and associated SV and SOV load models to BD 86 (DMRB 3.4.19) are now the load models for abnormal loading. Creep and shrinkage of concrete and prestress (including secondary effects in statically indeterminate structures) must be regarded as permanent loads. Collision loading to BD 48 (DMRB 3.4.7) must be considered only when required by the Overseeing Organisation.

4.2.2 Serviceability Limit State. For serviceability limit state checks, the Assessment Live Loading (ALL) derived from type HA loading or Authorised Weight (AW) vehicles to BD 21 (DMRB 3.4.3) and the SV/SOV load models (or STGO/SO vehicles) must be used, together with the other non-vehicular loads defined in BD 37 (DMRB 1.3); they must be applied and have the load factors and combinations in accordance with BD 86 (DMRB 3.4.19) and BD 37 (DMRB 1.3) as appropriate, with the γ_{fl} factor equal, for load combinations 1 to 5 of BD 37 (DMRB 1.3), to 1.0 for all SV/SOV load models (or STGO/SO vehicles) and associated ALL (or AW vehicles), in accordance with 2.7 of BD 86 (DMRB 3.4.19). When it has been agreed with the Overseeing Organisation that Type HB loading has to be included, the ALL (or AW vehicles) and Type HB loading must be used, together with the other non-vehicular loads defined in BD 37 (DMRB 1.3); they must be applied and have the load factors and combinations in accordance with BD 37 (DMRB 1.3).

The value of γ_{fl} for creep and shrinkage of concrete and for prestress (including secondary effect in statically indeterminate structures) must be taken as 1.0. The value of γ_{f3} must be taken as 1.0.

For the limitations given in 4.1.1.1 b) of BS 5400-4 for prestressed concrete, all members must be checked as being in class 2 for load combinations 1 to 5 of BD 37 (DMRB 1.3) except that, for load combination 1, live loading can be ignored for lightly trafficked highway bridges (e.g. accommodation bridges, bridleway bridges and foot/cycle bridges) and railway bridges where the live loading is controlled. For load combination 1, live loading must comprise ALL or AW vehicles only; however, for transverse cantilever slabs, transversely and two-way spanning slabs and central reserves (including skew slabs with significant transverse action), the loading must comprise AW vehicles only, applied as a single vehicle or convoy of vehicles in accordance with Annex D of BD 21 (DMRB 3.4.3). Single wheel loads need not to be considered except for cantilever slabs and the top flanges in beam-and-slab, voided slabs and box-beam construction. Cracking type checks for prestressed concrete are considered satisfied when the requirements of 6.3.2 are met; cracking type checks do not need to routinely be performed for bridges of prestressed concrete construction where all tendons in a particular section are external and unbonded except that, for segmental structures with precast elements, the stresses at unreinforced contact joints must comply with the requirements of 6.3.2.

If cracking type checks in reinforced concrete are required by the Overseeing Organisation, crack widths must be calculated in accordance with 5.8.8.2, and for the limitations given in 4.1.1.1 a) of BS 5400-4 for reinforced concrete, only load combination 1 of BD 37 (DMRB 1.3) needs to be considered. Live loading must comprise ALL or AW vehicles only; however, for transverse cantilever slabs, transversely and two-way spanning slabs and central reserves (including skew slabs with significant transverse action), the loading must comprise AW vehicles only, applied as a single vehicle or convoy of vehicles in accordance with Annex D of BD 21 (DMRB 3.4.3). Single wheel loads need not to be considered except for cantilever slabs and the top flanges in beam-and-slab, voided slabs and box-beam construction.

For the stress limitations given in 4.1.1.3 of BS 5400-4 for prestressed concrete and, where required by the Overseeing Organisation, for reinforced concrete, checks must be performed for load combinations 1 to 5 of BD 37 (DMRB 1.3). For prestressed concrete, stress limitations checks are considered satisfied when the compressive stress limits of 6.3.2 are met.

4.2.2A Serviceability Limit State

As the tendons in externally prestressed structures do not rely on concrete for corrosion protection, and as the problem of high stress fluctuations in tendon associated with cracks in bonded construction are not relevant to unbonded prestressing, the serviceability cracking criteria have been relaxed in the case of external unbonded prestressing. In longer span prestressed girder concrete bridges, temperature effects often give greater stresses than live loads and should not be overlooked, especially if required to investigate the cause of observed cracking.

4.2.3 Ultimate Limit State. For ultimate limit state checks, the ALL (or AW vehicles) and the SV/SOV load models (or STGO/SO vehicles) must be used, together with the other non-vehicular loads defined in BD 37 (DMRB 1.3) and, when required by the Overseeing Organisation, with the loads in BD 48 (DMRB 3.4.7); they must be applied and have the load factors and combinations in accordance with BD 21 (DMRB 3.4.3), BD 86 (DMRB 3.4.19), BD 37 (DMRB 1.3) and BD 48 (DMRB 3.4.7) as appropriate, with the γ_{fL} factor equal, for load combinations 2 and 3 of BD 37 (DMRB 1.3), to 1.0 for all SV/SOV load models (or STGO/SO vehicles), in accordance with 2.9 of BD 86 (DMRB 3.4.19). When it has been agreed with the Overseeing Organisation that Type HB loading has to be included, the ALL (or AW vehicles) and Type HB loading must be used, together with the other non-vehicular loads defined in BD 37 (DMRB 1.3) and, when required by the Overseeing Organisation, with the loads in BD 48 (DMRB 3.4.7); they must be applied and have the load factors and combinations in accordance with BD 21 (DMRB 3.4.3), BD 37 (DMRB 1.3) and BD 48 (DMRB 3.4.7) as appropriate. In calculating the resistance of members to flexure, vertical shear and torsion, γ_{fL} for the prestressing force must be taken as 1.15 where it adversely affects the resistance and 0.87 in other cases, except that, when no unbonded prestressing is present, the factor γ_{fL} must be applied only for calculating the resistance of members to vertical shear and torsion. In calculating secondary effects in statically indeterminate structures, γ_{fL} for the prestressing force may be taken as 1.0. The value of γ_{fB} must be taken as 1.10 in accordance with BD 21 (DMRB 3.4.3) except that where plastic upper bound methods are used for the analysis of the structure (e.g. yield-line analysis), γ_{fB} must be taken as 1.15. For other non-elastic methods, the factor may be taken as 1.10.

4.2.4 Deflection. When required by the Overseeing Organisation, deflection must be calculated for the most unfavourable distribution of loading for the member (or strip of slab) and may be derived from an elastic analysis of the structure. The partial safety factors are those of the serviceability limit states.

4.3 Properties of materials

4.3.1 General. Either the characteristic strength, or the worst credible strength (see 2.1.3.1), may be used for a material strength. In general, in analysing a structure to determine load effects, the material properties appropriate to the characteristic, or worst credible, strength must be used, irrespective of the limit state being considered.

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

- (a) Serviceability limit state: values are given in 4.3.1 of BS 5400-4
- (b) Ultimate limit state: values are given in 4.3.2

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

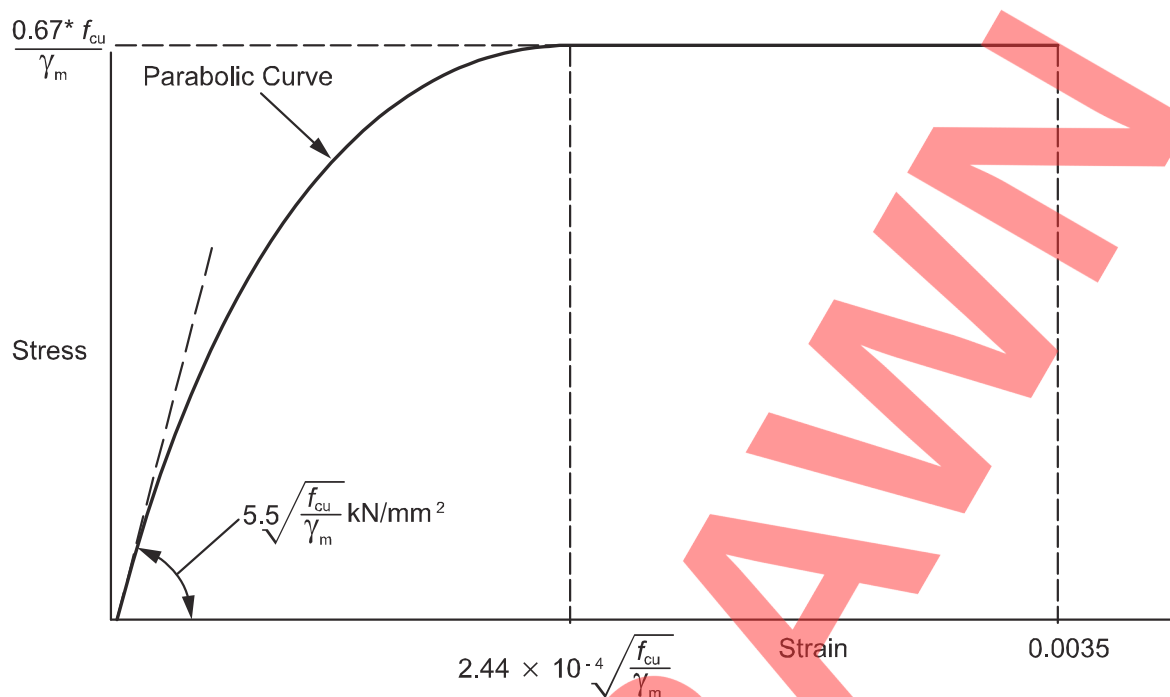
4.3.2 Material properties

4.3.2.1 Concrete. In assessing the strength of sections at the ultimate limit state, the assessment stress-strain curve for normal weight concrete may be taken from Figure 1, using the value of γ_{mc} for concrete given in 4.3.3.3.

The modulus of elasticity to be used for elastic analysis must be appropriate to the cube strength of the concrete, and, in the absence of test data, the short term value must be taken as $(20 + 0.27 f_{cu})$ kN/mm² with f_{cu} in N/mm² units. The effect of creep under long term loading may be allowed for by using half of the short term modulus of elasticity. To determine the effects of imposed deformations, for the calculation of deflections and to determine crack widths and stresses due to the effects of long and short term loading and imposed deformations, an appropriate intermediate value between the two above may be used.

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

Poisson's ratio may be taken as 0.2. The value for the coefficient of thermal expansion may be taken from Table 4.3 of BD 21 (DMRB 3.4.3).



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

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

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

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

where

f is the stress

ε is the strain

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

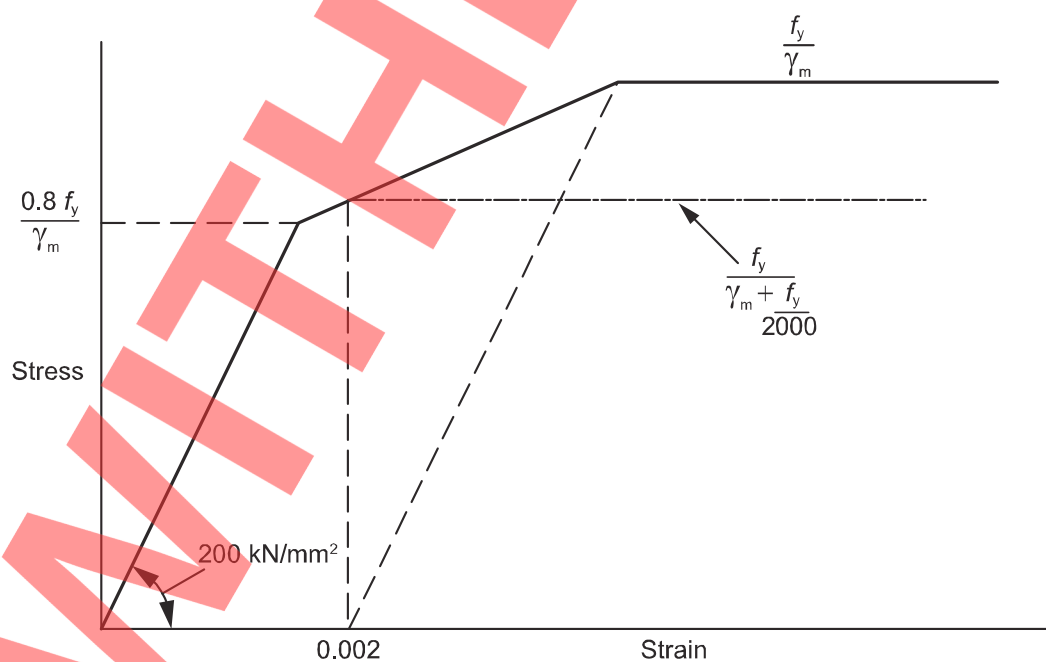


Figure 2 Short term assessment stress-strain curve for reinforcement

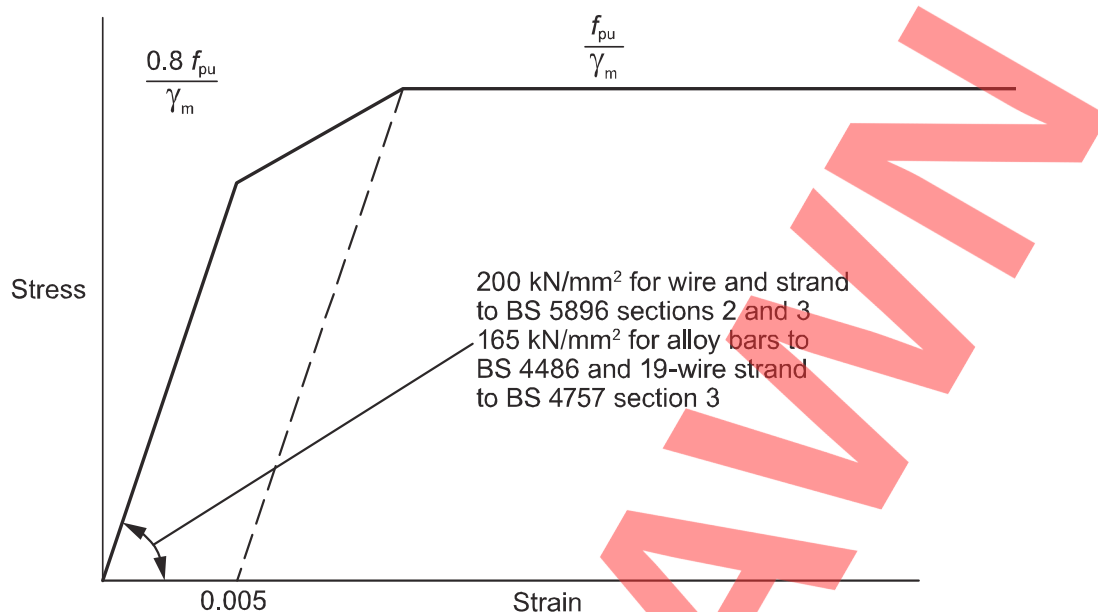


Figure 3 Short term assessment stress-strain curve for normal and low relaxation products

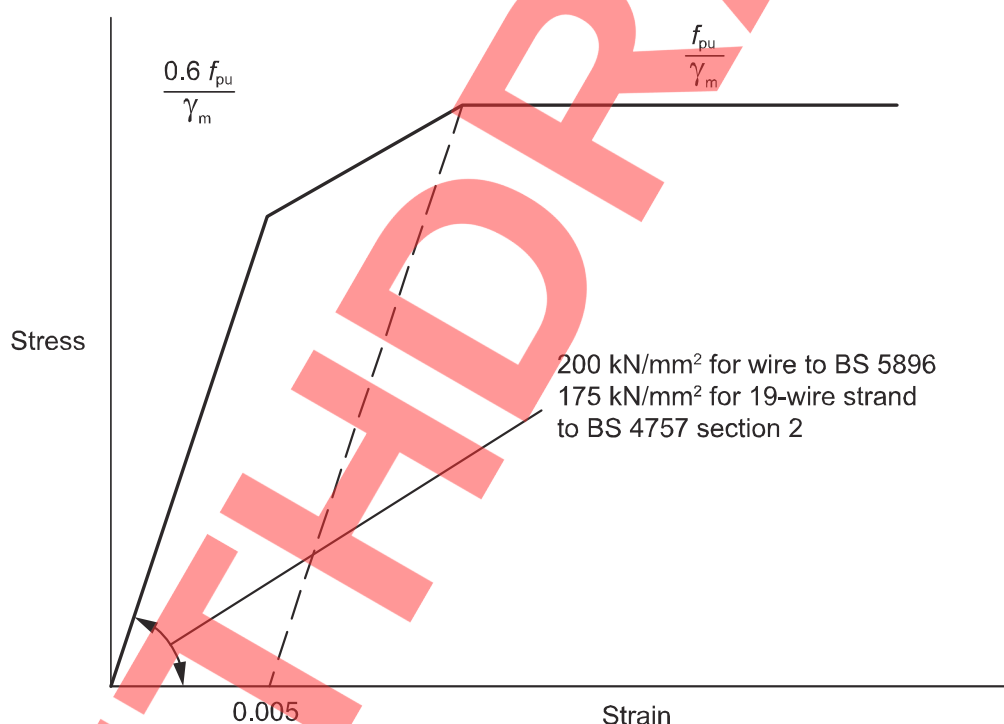


Figure 4 Short term assessment stress-strain curve for 'as drawn' wire and 'as spun' strand

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

- (a) for reinforcement, from Figure 2, using the value of γ_{ms} given in 4.3.3 or, for steels with sufficient ductility, from the stress-strain diagram with the inclined top branch in 3.2.7 of BS EN 1992-1-1, using the value of γ_{ms} given in 4.3.3;
- (b) for prestressing steel; from Figure 3 or Figure 4, using the value of γ_{ms} given in 4.3.3.

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

For reinforcement, the modulus of elasticity may be taken as 200 kN/mm². For prestressing steel, the modulus of elasticity may be taken from Figure 3 or Figure 4 as the appropriate tangent modulus at zero load.

4.3.2.2A Reinforcement and prestressing steel

The stress-strain diagram with the inclined top branch of BS EN 1992-1-1 should only be used, at the ultimate limit state, for reinforcing steels whose properties are not inferior to those corresponding to class B of Annex C of BS EN 1992-1-1.

4.3.3 Values of γ_m

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

4.3.3.2 Serviceability Limit State. Where a serviceability limit state check is required, the values of γ_{mc} and γ_{ms} to be applied to the characteristic stresses defined in 2.1.3.2 and 4.3.1 must be based on Table 4 of BS 5400-4 unless worst credible strengths are used, when the values of γ_{mc} may be reduced by 10% provided they are not taken as less than unity.

4.3.3.3 Ultimate Limit State. For both reinforced concrete and prestressed concrete, the values of γ_m applied to either the characteristic strengths or worst credible strengths are summarised in Table 4A; these replace the γ_m values for reinforced and prestressed concrete given in BD 21 (DMRB 3.4.3).

Table 4A Values of γ_m at the Ultimate Limit State.

Application	Symbol	Value for use with:	
		Characteristic strength	Worst credible strength
Reinforcement and prestressing tendons	γ_{ms}	1.15 ¹	1.10 ²
Concrete	γ_{mc}	1.50	1.20
Shear in concrete	γ_{mv}	1.25	1.15
Bond	γ_{mb}	1.40	1.25
Plain concrete wall	γ_{mcw}	2.25	1.80

¹ May be reduced to 1.05 for grade 460 steel.

² May be reduced to 1.05 for grade 460 steel or for any steel grade if measured steel depths are used in addition to the worst credible steel strength.

4.3.3.3A Ultimate Limit State

The partial safety factor γ_m is composed of two sub-factors:

- γ_{m1} which takes account of possible reductions in the strength of the material in the structure as a whole as compared with the characteristic value deduced from control specimens;
- γ_{m2} which takes account of possible weaknesses of the structure arising from any other cause.

In the case of steel, BS 8110-1 suggests that the γ_{ms} value of 1.15 used in BS 5400-4 and in earlier issues of BS 8110-1 is unjustifiably conservative. The γ_{ms} value of 1.15 used here may therefore be reduced to 1.05 for grade 460 steel. If the worst credible strength has been determined by testing samples of bars or tendons extracted from the structure, then γ_{m1} could be taken as 1.0. Furthermore, if measured effective depths are used in calculations, γ_{m2} could be reduced from its value used in design. The actual design value of γ_{m2} is not known, but both reference ⁽¹⁾ and BS 8110-2 suggest that γ_m could be reduced from its design value of 1.15 to 1.05 for assessment. Hence, a γ_m value of 1.10 has been adopted for use with the worst credible steel strengths and 1.05 when measured steel depths are also used.

In the case of concrete, γ_{m1} is often taken to be $1/0.8 = 1.25$ ⁽²⁾. This implies that, at the design stage, $\gamma_{m2} = 1.5/1.25 = 1.2$. If the worst credible concrete strength has been determined then, in an assessment, γ_{m1} can be taken as 1.0. Hence, $\gamma_m = \gamma_{m2}$. It is emphasised that γ_{m2} has to allow for any future deterioration of the concrete due to, for example, chemical attack, weathering, shrinkage and thermal movements. Hence, γ_{m2} could take a value between 1.2 for new concrete (i.e. the value implied in design) and 1.0 for old concrete which is not expected to deteriorate further. BS 6089 implies that a γ_m value of 1.2 should be applied to the mean estimated in situ cube strength, whereas BS 8110-2 states that a value not less than 1.05 should be applied to the worst credible strength. The latter value is considered to be rather low, and hence, allowing for the fact that it is difficult to determine accurately the worst credible strength for concrete, the higher value of 1.20 has been adopted for both new and old concrete.

If concrete strengths obtained from original construction records are used, the γ_{mc} values used should be those from Table 4A for characteristic strength. This is because, although some of the uncertainty about the potential strength of the concrete is eliminated, the actual strength of concrete is greatly influenced by the curing conditions (temperature and humidity). BS cured specimens used for concrete control are typically some 20% stronger than concrete in structure. If steel strengths from original construction records are used, the factors for characteristic strength may be used.

4.3.3.4 Fatigue. When applying 4.7, the values of γ_{ms} applied to a reinforcement stress range is 1.00.

4.4 Analysis of structure

4.4.1 General. The requirements of methods of analysis appropriate to the determination of the distribution of forces and deformations which are to be used in ascertaining that the limit state criteria are satisfied are described in BS 5400-1. Where a member is continuous over a support which is considered to provide no restraint to rotation and the analysis is undertaken assuming a span equal to the distance between the centres of point supports, the bending moment at the support may be reduced by an amount equal to $R_A^*t/8$, with R_A^* being the assessment reaction at support and t being the breadth of the bearing area.

4.4.1A General

The permitted reduction in bending moment over supports (moment rounding) is that adopted by the Eurocodes (5.3.2.2 of BS EN 1992-2).

4.4.2 Analysis for Serviceability Limit State

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

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

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

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

Axial, torsional and shearing stiffness constants, when required by the method of analysis, should be based on the concrete section and used with (a) or (b). In assessment, however, it is often beneficial to use cracked properties, and these should be used with reduced torsional properties to achieve compatibility with (c). For slabs (where torsion and flexure are not really separate effects) it is appropriate to reduce the torsional stiffness in proportion to the ratio of the average cracked to uncracked flexural stiffness for the two directions.

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

4.4.2.1A General

BS 5400-4 allows cracked or uncracked properties to be used. In assessment intermediate properties may be used.

4.4.2.2 Methods of analysis and their requirements. The method of analysis must take account of all the significant aspects of behaviour of a structure governing its response to loads and imposed deformations.

4.4.3 Analysis for Ultimate Limit State. Elastic methods can be used to determine the distribution of forces and deformations throughout the structure. Stiffness constants can be based on any of those listed in 4.4.2.1. The torsional stiffness may be reduced where appropriate in accordance with 5.3.4.2. Other constants may also be adjusted to give some allowance for redistribution where this will give a more realistic representation of behaviour.

Non-linear and plastic methods of analysis may be used with the agreement of the Overseeing Organisation.

4.4.3A Analysis for Ultimate Limit State

A wide variety of analytical approaches can be used to assess concrete bridges, ranging from simple static load distributions, through conventional elastic analyses to sophisticated non-linear analyses. It should be borne in mind that static analysis and conventional linear elastic analyses normally give safe lower bound solutions for ultimate strength. It is therefore often appropriate to use the approach of “progressive screening”. That is, starting with simple conservative approaches and progressing to more realistic approaches until either adequate strength is proved or the structure is found to be genuinely inadequate.

Concrete bridges are generally designed by performing elastic analyses using uncracked section properties, and ensuring that individual sections can resist the elastic stress resultants. The elastic analysis is not intended to predict the actual behaviour of the structure being designed, but is used merely because it results in a set of stress resultants which are in equilibrium and, hence, provides a safe design ⁽³⁾. However, in assessment, one is attempting to predict the actual behaviour of an existing structure. Although an elastic analysis would give a conservative assessment, there is scope for more accurate analysis, because the section properties are fully defined.

Elastic analysis can be made more realistic by using reduced stiffness for element properties corresponding to load effects which would otherwise exceed section strengths. This is commonly done for torsion. In particular, if analysis of simply supported skew slab structures suggests that top steel is required when not provided or is overstressed in the obtuse corners, this can be avoided by the use of torsionless analysis. Also, because many earlier concrete beam and slab bridges were designed using static load distribution approaches, with little allowance for either torsion or global transverse moments, it is often advantageous to analyse them using reduced transverse and torsional stiffnesses. In principle, provided ductility is adequate, any elastic analysis is a safe lower bound solution whatever section properties are used.

Although elastic analysis is safe, it is often advantageous to use more realistic analyses. These include:

- i. Upper bound collapse analyses which predict the collapse load of the complete structure, as opposed to checking discrete critical sections. It is emphasised that experience of these methods is necessary in order that the critical collapse mechanism can be identified. Guidance on the applications of such methods to bridges can be found in references ⁽⁴⁻⁸⁾.
- ii. Non-linear analyses which are capable of predicting the behaviour of a structure at all stages up to collapse. Most non-linear analyses are capable of predicting only flexural failures. Guidance on their use can be found in references ⁽¹⁾ and ⁽⁹⁾.
- iii. Methods which take account of restraints which are generally ignored in design, e.g. membrane action in top slabs of beam and slab decks ⁽¹⁰⁾. Guidance on the use of compressive membrane action for assessment of bridge deck slabs is given in BD 81 (DMRB 3.4.20).

4.5 Analysis of section

4.5.1 Serviceability Limit State. An elastic analysis must be carried out. In-plane shear flexibility in concrete flanges (shear lag effects) must be allowed for. This may be done by taking an effective width of flange (see 5.3.1.2).

4.5.2 Ultimate Limit State. The strength of critical sections must be assessed in accordance with clauses 5, 6 or 7 to satisfy the requirements of 4.1. In-plane shear flexibility in concrete flanges (shear lag effects) may be ignored.

4.6 Deflection. When deflection checks are required by the Overseeing Organisation, the material properties, stiffness constants and calculations of deflections may be based on the information given in 4.3.2.1 and/or in Appendix A of BS 5400-4.

4.7 Fatigue. When the assessor considers a structure to be fatigue prone, the effect of repeated live loading on the fatigue strength of a bridge must be assessed. For reinforcing bars that have been subjected to welding, details of compliance criteria are given in BS 5400-10 as implemented by BD 9 (DMRB 1.3).

For unwelded non-corroded reinforcement the fatigue life must be determined in accordance with BS 5400-10 as implemented by BD 9 (DMRB 1.3), using the following parameters for the σ_r - N relationship:

$$\begin{array}{lll} \text{bars} \leq 16\text{mm diameter;} & m = 9 & k_2 = 0.75 \times 10^{27} \\ \text{bars} > 16\text{mm diameter;} & m = 9 & k_2 = 0.07 \times 10^{27} \end{array}$$

The effective stress range to be used in fatigue assessment should be obtained by adding 60% of the range from zero stress to maximum compressive stress to that part of the range from zero stress to maximum tensile stress.

However, where the stress range under load combination 1 of the Assessment Live Loading at the serviceability limit state of 4.2.2 is less than the value given below, no further fatigue check is required.

Span	bars $\leq$ 16mm dia	bars $>$ 16mm dia
less than 3.5m	280 N/mm ²	220 N/mm ²
3.5m - 5.0m	250 N/mm ²	190 N/mm ²
5.0m - 10.0m	195 N/mm ²	150 N/mm ²
10.0m - 200m	155 N/mm ²	120 N/mm ²
200m and greater	250 N/mm ²	190 N/mm ²

Provided the following requirements are met, the local effects of wheel loads applied directly to a slab spanning between beams or webs need not be checked for fatigue:

1. The clear span to overall depth ratio of the slab does not exceed 18.
2. The slab acts compositely with its supporting beams or webs.
3. Either (a) the slab acts compositely with transverse diaphragms or webs or (b) the width of the slab perpendicular to its span exceeds three times its clear span.
4. The slab does not contain welded reinforcement or reinforcement couplers.

4.7A Fatigue

The requirements for unwelded non-corroded bars given in 4.7 are based on a study carried out for the Department of Transport. Many deck slabs would fail fatigue checks. However, extensive testing has shown that the stress range due to wheel loads experienced by the reinforcement in most cases is substantially less than that predicted by conventional analysis. Accordingly, checks are not required for local effects provided the conditions in 4.7 above are met.

Guidance on determining the fatigue life of corroded reinforcement is given in BA 38 (DMRB 3.4.5). Guidance on the fatigue strength of tack welded reinforcing bars is given in BA 40 (DMRB 1.3.4).

Failure to satisfy the fatigue requirements should not necessitate immediate remedial action. Management of the structure may however be affected; for instance inspection frequency of the affected elements may be increased as advised by the assessor.

4.8 Combined global and local effects

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

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

In order to take advantage of the beneficial effects of membrane action, methods of analysis which take account of in-plane as well as flexural effects may be considered. See also 4.4.3.

4.8.2A Analysis of structure

Guidance on the use of compressive membrane action for the assessment of bridge deck slabs is given in BD 81 (DMRB 3.4.20).

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

(a) Serviceability Limit State

- (1) For reinforced concrete elements, if a crack width check is required by the Overseeing Organisation, the total crack width due to combined global and local effects will be determined in accordance with 5.8.8.2.

- (2) For prestressed concrete elements, coexistent stresses, acting in the direction of prestress, may be added algebraically in checking the stress limitations.
- (b) **Ultimate Limit State:** The resistance of the section to the combination of local and global effects must be checked using the assumptions given in 5.3.2.1 or 6.3.3.1 as appropriate allowing for any axial force. However, in the case of a deck slab, the resistance to combined global and local effects is deemed to be satisfactory if the axial force from the global effects is checked separately from the resistance to local moments.

5. ASSESSMENT: REINFORCED CONCRETE

5.1 General

5.1.1 Introduction. This clause gives methods of assessment which will in general ensure that, for reinforced concrete structures, the requirements set out in 4.1 are met.

5.1.2 Limit state assessment of reinforced concrete. Clause 5 follows the limit state philosophy set out in 3.1B to 3.6B.

5.1.3 Loads. In clause 5 the assessment load effects (see 2.1) for the ultimate and serviceability limit states are referred to as 'ultimate loads' and 'service loads' respectively. The values of the loads to be used in assessment are derived from 4.2. When analysing sections, the terms 'strength', 'resistance' and 'capacity' are used to describe the assessment resistance of the section (see also BD 21, DMRB 3.4.3).

5.1.4 Strength of materials

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

5.1.4.2 Strength of concrete. Assessment may be based on either the specified characteristic cube strength, or the worst credible cube strength assessed as the lower bound to the estimated in situ cube strength determined in accordance with 2.4B to 2.11B. For structures designed to codes prior to the adoption of the term characteristic strength, the concrete strength was specified in terms of the minimum 28 day works cube strength. For the purpose of assessment, the characteristic strength of concrete may be taken as the minimum 28 day works cube strength.

5.1.4.3 Strength of reinforcement. Assessment must be based on either the specified characteristic yield or proof stress, or the worst credible yield or proof stress assessed from tests on reinforcement samples extracted from the structure. For structures designed to codes prior to the adoption of the term characteristic strength, the reinforcement strength was specified in terms of the guaranteed yield strength. For the purpose of assessment, the characteristic strength of reinforcement may be taken as the guaranteed yield strength.

5.2 Structures and structural frames

5.2.1 Analysis of structures. Structures must be analysed in accordance with the requirements of 4.4.

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

- (a) Checks are made to ensure that adequate rotation capacity exists at sections where moments are reduced, making reference to appropriate test data. In the absence of a special investigation, the plastic rotation capacity may be taken as the lesser of:

- 1) $0.008 + 0.035 (0.5 - d_c / d_e)$
- or
- 2) $\frac{0.6\phi}{d - d_c}$ but not less than 0

where

- d_c is the calculated depth of concrete in compression at the ultimate limit state;
- d_e is the effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange;
- ϕ is the diameter of the smallest tensile reinforcing bar;
- d is the effective depth to tension reinforcement.

- (b) Proper account is taken of changes in transverse moments and transverse shears consequent on redistribution of longitudinal moments.
- (c) Shears and reactions used in assessment are taken as those calculated either prior to, or after redistribution, whichever are the greater.

As an alternative to the approach in the previous paragraph, a linear elastic analysis with limited moment redistribution under the ultimate limit states may be carried out, without explicit checks done on the rotation capacity, in accordance with 5.5 of BS EN 1992-2 and the accompanying 5.5 of BS EN 1992-1-1, provided the conditions and limitations contained therein are met.

5.2.2A Redistribution of moments

Criterion (d) of BS 5400-4 has been omitted. The BS 5400-4 criterion (d) limited moment redistribution to members up to 1.2m deep, whereas the available test data ⁽¹¹⁾ on rotation capacity only cover members up to about 0.8m deep. In view of the fact that BS 5400-4 limits moment redistribution to members of a certain depth whilst permitting plastic methods to be applied to members of any depth, and given that criterion (a) requires either a special investigation or the adoption of conservative formulae, 1) and 2), for rotation capacity, it is not considered necessary by the Overseeing Organisation to include also a specific limitation on depth.

In BS EN 1992-2 and BS EN 1992-1-1 a clear distinction is made between the conditions and limitations for performing a linear analysis with limited moment redistribution and plastic analysis. Provided the requirements in 5.5 of BS EN 1992-2 and 5.5 of BS EN 1992-1-1 are met, ultimate limit state analysis with limited moment redistribution may be performed without explicitly checking the rotation capacity. Further restrictions on ductility and rotation capacity are generally necessary when a plastic analysis is performed.

5.3 Beams

5.3.1 General

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

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

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

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

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

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

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

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

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

5.3.1.3 Slenderness limits for beams. A simply-supported or continuous beam is considered to have adequate lateral stability when the clear distance between lateral restraints does not exceed $300 b_c^2/d$, where d is the effective depth to tension reinforcement and b_c is the breadth of the compression face of the beam midway between restraints.

A cantilever with lateral restraint provided only at the support is considered to have adequate lateral stability when the clear distance from the end of the cantilever to the face of the support does not exceed $150 b_c^2/d$.

Beams outside these limits, require additional analysis and calculation to demonstrate their stability.

5.3.1.3A Slenderness limits for beams

BS 5400-4 gives two limits: one a function of b_c , and the other a function of b_c^2/d . According to reference ⁽¹²⁾, the first limit is not a major parameter, and the second limit is conservative. The assessment limits on b_c^2/d are obtained by dividing the value in reference ⁽¹²⁾ by a partial safety factor of 1.5.

5.3.2 Resistance moment of beams

5.3.2.1 Analysis of sections. When analysing a cross-section to determine its ultimate moment of resistance, the following assumptions must be made:

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

- (b) The stresses in the concrete in compression are either derived from the stress-strain curve in Figure 1 with the appropriate value of γ_{mc} given in 4.3.3.3 or, in the case of rectangular sections and in flanged, ribbed and voided sections where the neutral axis lies within the flange, the compressive stress may be taken as equal to $0.6 f_{cu}/\gamma_{mc}$ over the whole compression zone. In both cases the strain at the outermost compression fibre at failure is taken as 0.0035.
- (c) The tensile strength of concrete is ignored.
- (d) The stresses in the reinforcement are derived from 4.3.2.2. The values of γ_{ms} are given in 4.3.3.3.

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

5.3.2.1A Analysis of sections

The concrete stress strain curve in Figure 1 and the failure strain of 0.0035 are appropriate to unbound concrete. Higher failure stresses and strains are achieved when the concrete is laterally restrained by helical binding or, to a lesser extent, by conventional links. If the ultimate strength of a member is governed by failure of the concrete compression zone and if the member marginally fails an assessment using the unbound stress-strain curve, it would be advisable to allow for the enhancing effects of links or helical binding. Appropriate guidance can be obtained from references ⁽¹³⁾ and ⁽¹⁴⁾.

The BS 5400-4 requirement to check the steel strain and the provision of an alternative method of analysis are not relevant to assessment. However, it should be remembered that if the section is over-reinforced it could fail in a brittle mode with little warning.

5.3.2.2 Design charts. "Not applicable to assessment"

5.3.2.3 Assessment formulae. Provided that the amount of redistribution of the elastic ultimate moments has been less than 10%, the formulae below may be used to calculate the ultimate moment of resistance of a solid slab or rectangular beam, or of a flanged beam, ribbed slab or voided slab when the neutral axis lies within the flange and the flange is in compression. For flanged beams where the flange is in tension the formulae below may be used without limitation on the neutral axis depth. For sections without compression reinforcement the ultimate moment of resistance may be taken as the lesser of the values obtained from Equations 1 and 2. Equations 3 and 4 may be used for sections with compression reinforcement. A rectangular stress block of maximum depth $0.5d$ and a uniform compression stress of $0.6f_{cu}/\gamma_{mc}$ has been assumed.

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

$$M_u = (0.225 f_{cu} / \gamma_{mc}) b d^2 \quad \text{Equation 2}$$

$$M_u = (0.6 f_{cu} / \gamma_{mc}) b x (d - 0.5x) + f'_s A'_s (d - d') \quad \text{Equation 3}$$

$$(f_y / \gamma_{ms}) A_s = (0.6 f_{cu} / \gamma_{mc}) b x + f'_s A'_s \quad \text{Equation 4}$$

where

- M_u is the ultimate resistance moment;
 A_s is the area of tension reinforcement;
 A'_s is the area of compression reinforcement;
 b is the width of the section in compression, at the level of the neutral axis;

d is the effective depth to the tension reinforcement;
 d' is the depth to the compression reinforcement;
 f_y is the characteristic, or worst credible strength, of the reinforcement;
 f'_s is equal to $f_y/(\gamma_{ms} + f_y/2000)$;
 x is the neutral axis depth;
 z is the lever arm;
 f_{cu} is the characteristic, or worst credible strength, of the concrete;
 γ_{mc}, γ_{ms} are the material partial safety factors given in 4.3.3.3.

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

$$z = \left[1 - \frac{0.84(f_y / \gamma_{ms}) A_s}{(f_{cu} / \gamma_{mc}) b d} \right] d \quad \text{Equation 5}$$

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

When using Equations 3 and 4 for sections with compression reinforcement the neutral axis depth x must first be calculated from Equation 4.

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

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

For flanged beams, where the flange is in compression, but where the neutral axis extends below the bottom of the flange, the ultimate resistance moment may be taken as the lesser of the values given by Equations 6 and 7, where h_f is the thickness of the flange:

$$M_u = (f_y / \gamma_{ms}) A_s \left(d - \frac{h_f}{2} \right) \quad \text{Equation 6}$$

$$M_u = (0.6 f_{cu} / \gamma_{mc}) b h_f \left(d - \frac{h_f}{2} \right) \quad \text{Equation 7}$$

5.3.2.3A Assessment formulae

BS 5400-4 has a limitation on the compressive stress in steel which it inherited from CP110. This has not been justified by tests and was abandoned in BS 8110-1 and does not appear in BS EN 1992-1-1. Therefore, the restriction on steel compression has been removed.

5.3.3 Shear resistance of beams

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

$$v = \frac{V}{b_w d} \quad \text{Equation 8}$$

where

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

Except when the section is assessed in accordance with 5.3.3.5, the shear stress, v , must not exceed:

$$0.36 (0.7 - f_{cu} / 250) f_{cu} / \gamma_{mc}$$

(where γ_{mc} is the partial safety factor for concrete given in 4.3.3.3) whatever shear reinforcement is provided.

In some situations where significant axial compressive forces exist, they may enhance shear capacity (resistance). This may be allowed for in assessment e.g. by using the corresponding requirement for columns in 5.5.6.

In a haunched beam, 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 algebraically from the applied shear force.

5.3.3.1A Shear stress

The upper limit to shear stress in BS 5400-4 is known to be conservative and the increased limit here is based on plasticity theory ⁽⁵⁾, which is the basis of the varying angle truss approach of BS EN 1992-1-1 given as an alternative method given in 5.3.3.5. However, the maximum shear limit of $0.36 (0.7 - f_{cu} / 250) f_{cu} / \gamma_{mc}$ is more conservative than that in 5.3.3.5, so sections which do not satisfy this maximum to 5.3.3.1 may still have adequate strength to 5.3.3.5 if they have sufficient transverse and longitudinal reinforcement.

The sub-clauses in BS 5400-4 are written for prismatic beams and are generally conservative for haunched ones. This sub-clause allows advantage to be taken of the vertical component of flange forces in these. If three dimensional finite element models are used for box beams, the web shears from the computer model will already have the flange forces subtracted.

5.3.3.2 Shear capacity

Sections without shear reinforcement: In the absence of shear reinforcement, the ultimate shear resistance V_u of a section is given by:

$$V_u = \xi_s v_c b_w d$$

where the depth factor ξ_s is given by:

$$\xi_s = \left(\frac{500}{d} \right)^{0.25} \text{ but not less than } 0.7.$$

The ultimate shear stress in concrete v_c is:

$$v_c = \frac{0.24}{\gamma_{mv}} \left(\frac{100 A_s}{b_w d} \right)^{1/3} (f_{cu})^{1/3} \text{ (but see 5.3.3.2A)}$$

where the term $\left(\frac{100 A_s}{b_w d} \right)$ should not be taken less than 0.15 or greater than 3.0, and the partial safety factor for shear γ_{mv} is taken from Table 4A.

The term A_s is the area of longitudinal tension reinforcement that continues at least a distance equal to the effective depth, d , beyond the section being considered. At supports the area of longitudinal tension reinforcement that continues up to the support may be used, appropriately reduced in accordance with 5.8.7 when there is insufficient anchorage at

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

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

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

where

A_{sv} is the cross-sectional area of shear reinforcement at a particular cross-section;
 s_v is the spacing of the shear reinforcement along the member;
 α is the inclination of the shear reinforcement to the axis of the member;
 f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 N/mm²;
 γ_{ms} is the material partial safety factor for steel given in 4.3.3.3;
 b_w is the breadth of the cross-section.

Where effective vertical links are present, V_u must be taken as:

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

where the parameters ξ_s , v_c , b_w and d are as defined above.

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

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

where M and V are the co-existent ultimate bending moment and shear force at the section under consideration, z is the lever arm, which may be taken as $0.9d$ or calculated from Equation 5, whichever the greater (but not to be taken as greater than $0.95d$), and A_s is the effectively anchored longitudinal reinforcement in the tensile zone, as defined in the formula for calculating v_c in the previous section. However, within an individual sagging or hogging region, the longitudinal tension force must not be taken as more than $M_{\max}/z$, where $M_{\max}$ is the maximum ultimate moment within that region.

Inclined links or bent-up bars must be assumed to form the tension members of one or more single systems of lattice girders in which the concrete forms the compression members. The maximum stress in any link or bar must be taken as $\frac{f_{yv}}{\gamma_{ms}}$. Bent-up bars must be checked for anchorage (see 5.8.6.3) and bearing (see 5.8.6.8 and 5.8.6.9).

The treatment of shear reinforcement at an angle to the member axis (inclined links or bent up bars) as members of a lattice is conservative when there is a long length of inclined links or bent up bars. In such situations, the ultimate shear resistance indicated in Figure 5.1 may be taken as:

$$V_u = \xi_s v_c b_w d + A_{sv} (\sin \alpha + \cos \alpha) \left(\frac{f_{yv} d}{\gamma_{ms} s_v} \right)$$

However, when $\alpha > 45^\circ$, V_u must not be taken as greater than

$$V_u = \frac{2A_{lv}(f_y\gamma_{ms})}{1 - \cot \alpha} + \xi_s v_c b_w d$$

where A_{lv} is the area of effectively anchored longitudinal reinforcement in the tensile zone in excess of that required to resist that bending moment which is co-existent with the shear force under consideration.

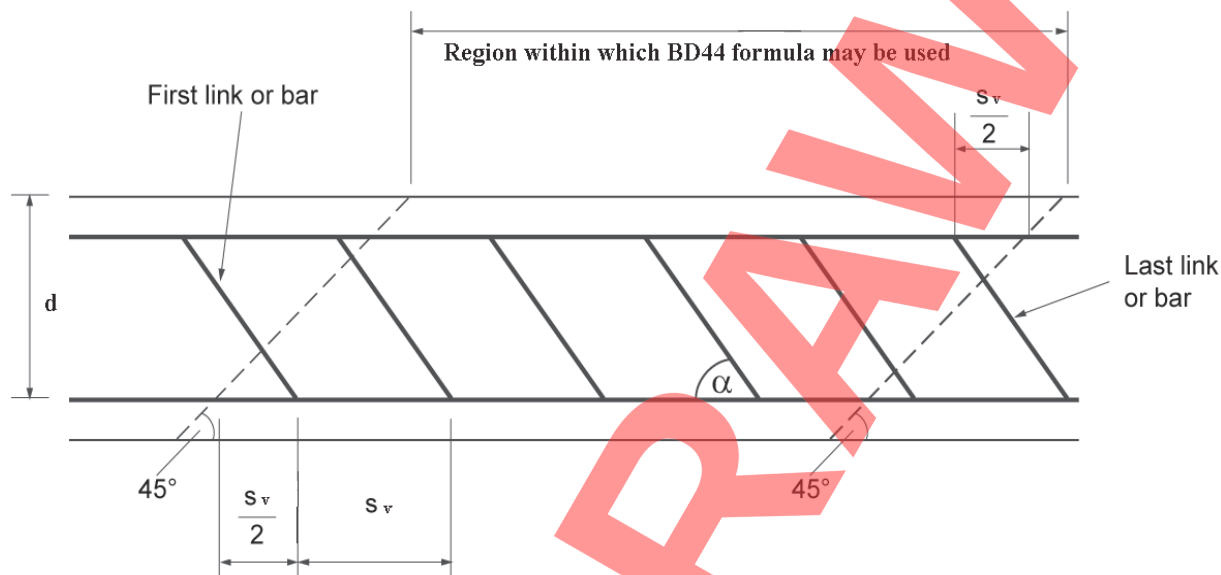


Figure 5.1 Inclined shear reinforcement

Shear has to be resisted by a combination of links and longitudinal reinforcement. Hence, part of the longitudinal tension reinforcement in a section is used to resist shear and is not available to resist the co-existing moment. If under a specified assessment loading there is sufficient longitudinal tension reinforcement to resist the bending moment but insufficient excess reinforcement to resist the co-existing shear force, then the section is incapable of carrying the specified assessment loading. However, the section would be capable of resisting a smaller loading which would induce a smaller bending moment, and, thus, result in a greater excess of reinforcement to resist the smaller co-existing shear force. Hence the section should be checked under progressively smaller assessment loadings until the combined bending moment and co-existing shear force can be resisted.

An alternative method for determining the shear capacity based on varying angle truss approach is given in 5.3.3.5.

5.3.3.2A Shear capacity

Assessment sub-clause 5.3.3.2 is a rearrangement of the BS 5400-4 design sub-clause. It covers sections both with and without shear reinforcement (either in the form of links or bent-up bars). All of the shear can be resisted by bent-up bars, since it has been demonstrated⁽¹⁵⁾ that such shear reinforcement is fully effective. However, since test data are not available for $\alpha < 30^\circ$, no attempt has been made to allow for shear reinforcement bent at such angle.

The upper limit of 460 N/mm² for the strength of shear reinforcement in BS 5400-4 is merely what was the standard grade at the time of publication. 480 N/mm² is the value which was found⁽¹⁶⁾ should be imposed in order to guarantee that the shear reinforcement would yield at collapse prior to crushing of the concrete. However, the 480 N/mm² limit is the actual reinforcement strength, so increasing the upper limit of the unfactored strength to 500 N/mm² is justified, as the actual value for use in assessment would always be reduced to values lower than 480 N/mm² after the material partial safety factor for steel is applied.

The constant 0.27 in the BS 5400-4 expression for v_c has been reduced to $0.27 \times 1.1/1.25 = 0.24$, because the BS 5400-4 expression is actually the mean value divided by γ_m ⁽¹⁷⁾. However, γ_m should be applied to the characteristic value, and the BS 5400-4 expression actually implies a γ_m value in the range 1.0 to 1.1 applied to the characteristic shear strength. Hence, the correction detailed above has been carried out. However, where a substantial volume of concrete would be involved in the shear failure, as in the case of slabs, the constant can be taken as 0.27 (see 5.4.4.1).

The BS 5400-4 requirement to over-design links to resist an additional shear stress of 0.4 N/mm^2 has been omitted from the assessment code. It is understood that its introduction was to allow for a possible reduction in shear capacity under fatigue loading. In general, it is not considered necessary to make such an allowance in an assessment. However, when it is known or suspected that links have been tack welded to main steel it would be advisable to include the additional shear stress, since significant reductions in fatigue strength can occur as a result of tack welding ⁽¹⁸⁾. See also 4.7 and BA 40 (DMRB 1.3.4).

The maximum spacing of links is specified as d because test data ⁽¹⁹⁾ show a reduction in shear strength at this spacing rather than the BS 5400-4 value of $0.75d$.

The BS 5400-4 upper limit on f_{cu} of 40 N/mm^2 is not included because it has been shown ⁽⁴⁾ that it is not justified by test data collected for values up to 117 N/mm^2 . More recent work ⁽²⁰⁾ suggests this may not be valid for concretes made with some aggregates, notably limestone. Special considerations (tests or evidence of satisfactory performance of the mix with the types of aggregates used) should therefore be made before using values of f_{cu} above 60 N/mm^2 for the calculation of the shear capacity.

The BS 5400-4 requirement for additional longitudinal reinforcement has been relaxed in the following respects:

- (1) The contribution to shear resistance from the concrete has been taken into account when calculating the additional longitudinal reinforcement requirement.
- (2) An upper limit of $M_{\max}/z$ on the total longitudinal force to be resisted is laid down.

It should be noted that the stress in bent-up bars may have to be limited to less than f_{yv}/γ_{ms} if the anchorage or bearing stress requirements of 5.8.6.3, 5.8.6.8 and 5.8.6.9 are not complied with.

BS 5400-4 requires beams to have a minimum area of shear links. In assessment it is recognised that some beams designed to previous codes may have no shear links or, less than the minimum, but are still capable of resisting shear. Therefore the requirement to provide minimum shear links has been removed. However, it should be remembered that a beam without links could fail in a brittle mode with little warning. When the shear reinforcement in a member does not satisfy either the minimum area or the maximum spacing criteria, the shear capacity of the member should be taken as that resulting from the contribution of the concrete resistance alone, disregarding the contribution from the shear reinforcement.

5.3.3.3 Enhanced shear strength of sections close to supports. For sections within a distance $a_v < 3d$ of a support, an enhancement to the shear strength of 5.3.3.2 may be allowed where the term

$$\xi_s v_c b_w d$$

is replaced by

$$\kappa \xi_s v_c b_w d$$

and where the shear capacity V_u in 5.3.3.2, for both cases with and without effective shear reinforcement, is multiplied by the factor Γ , giving, for sections without effective shear reinforcement, a shear capacity of

$$V_u = \Gamma \kappa \xi_s v_c b_w d$$

and, for section with effective shear reinforcement, a shear capacity of

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

In the above formulae, for the purpose of calculating v_c the full area of tensile reinforcement that continues up to the support must be used.

The term κ is a shear enhancement factor for short shear spans which may be applied if the main reinforcement continues to the support. For sections within a distance $a_v < 3d$ of a support, the value of κ is given by:

$$\kappa = \frac{3d}{a_v}$$

where the distance a_v is measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support.

The term Γ is equal to:

$$\Gamma = \sqrt{\frac{\alpha_s F_{ub}}{F_{ub \max}}} \quad (\text{to be taken not greater than unity})$$

where α_s is a factor that takes into account the increased bond strength due to bearing pressure in the support region, given by

$$\alpha_s = 1.0 + \frac{5.3 \sigma}{f_{cu}} \quad (\text{to be taken not greater than 2.6})$$

and σ is the bearing pressure on the reinforcement due to the ultimate loads (that may be taken as equal to the reaction at support divided by the bearing area), F_{ub} is the total ultimate anchorage force in the tension reinforcement at the front face of the support (calculated in accordance with 5.8.6.3) and $F_{ub \max}$ is given by

$$F_{ub \max} = 6 \xi_s v_c b_w d$$

In the formula for Γ above, $\alpha_s F_{ub}$ and $F_{ub \max}$ are not to be taken as greater than $A_s f_y / \gamma_{ms}$. For internal supports of continuous structures Γ may be taken as 1.0.

Where this sub-clause gives a lower shear strength than that given by 5.3.3.2 the greater value should be used.

Notwithstanding the permitted shear enhancement above, the shear stress upper limit of 5.3.3.1 must not be exceeded, whatever shear reinforcement is present.

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

5.3.3.3A Enhanced shear strength of sections close to supports

Two factors are introduced. The factor κ is the shear enhancement factor which is a modification of the enhancement defined in 5.3.3.3 of BS 5400-4. This factor is less conservative than the BS 5400-4 values and the length over which shear enhancement is effective is increased to $3d$ in line with the proposal in ⁽²¹⁾, which gives a good lower bound fit to the test data.

BS 5400-4 requires a 20ϕ diameter anchorage to allow short shear span enhancement to be used. However, tests ⁽²²⁾ show that stirrups can contribute to the shear capacity and shear enhancement can occur at short shear spans, even at short anchorage lengths. The revised assessment rules introduce a reduction factor Γ in the equation for V_u . This method ⁽²²⁾, derived by application of the variable angle truss approach described in 5.3.3.5 and consistent with BS EN 1992-1-1, gives more accurate predicted shear capacities than the previous version of BD 44 and has been verified against a large number of test data.

Sections less than d from the support are not normally critical for shear, as tests indicate that where $a_v < d$ the load is transferred to the support by direct strut action, and the ultimate shear strength rises sharply.

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

5.3.3.5 Alternative method. As an alternative to the method given in 5.3.3.2, shear capacity of members with links may be assessed using the varying angle truss approach adopted by the Eurocodes (BS EN 1992-1-1 and BS EN 1992-2). The shear strength V_u is, for elements with vertical links, the lesser of:

$$\frac{A_{sv}}{s_v} z \frac{f_{yv}}{\gamma_{ms}} \cot \theta$$

and

$$\alpha_{cw} b_w z \nu (1 - 0.5 \cos \alpha) \frac{0.8 f_{cu}}{\gamma_{mc}} / (\cot \theta + \tan \theta)$$

and, for elements with inclined links, the lesser of

$$\frac{A_{sv}}{s} z \frac{f_{yv}}{\gamma_{ms}} (\cot \theta + \cot \alpha) \sin \alpha$$

and

$$\alpha_{cw} b_w z \nu (1 - 0.5 \cos \alpha) \frac{0.8 f_{cu}}{\gamma_{mc}} (\cot \theta + \cot \alpha) / (1 + \cot^2 \theta)$$

In these equations:

z is the lever arm and may be taken as $0.9d$ for members without axial force;

θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force;

α_{cw} is a coefficient taking into account of the state of the stress in the compression chord, taken as 1.0 for non-prestressed structures. Where the section is subjected to an external applied load, an enhanced value in accordance with 6.3.4.7 may be used;

ν is a strength reduction factor for concrete cracked in shear, taken as $0.6 \left(1 - \frac{0.8 f_{cu}}{250} \right)$;

α is the inclination of the shear reinforcement to the axis of the member.

In the above, the angle θ may be chosen within the limits of $1.0 \leq \cot \theta \leq 2.5$ to provide the maximum shear capacity, except in elements where shear co-exist with externally applied tension, where $\cot \theta$ must be taken as 1.25. The other symbols are as in 5.3.3.1 and 5.3.3.2. In addition, the maximum shear force, in Newtons, must not be greater than $200b_w^2$ at sections more than d from the edge of a support with b_w in mm.

If this approach is used, the flexural reinforcement at any section must be capable of resisting the maximum bending moment in any section within a distance $z(\cot \theta - \cot \alpha)/2$ in the direction of increasing moment from the section and be provided with effective anchorage in accordance with 5.8.6.3. However, at a simply-supported end, the bond stress in the length of reinforcement immediately over the bearing may be taken as 1.5 times that given in 5.8.6.3. Where this requirement is not satisfied, the shear capacity V_u is limited to:

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

In this equation, F_{ub} is the maximum force that can be developed in the main tension reinforcement at the section considered, that is the sum of the forces in the bars in the section limited to either the available bond strength to 5.8.6.3 (with the modification for simply supported ends noted above) or to $A_s f_y / \gamma_{ms}$ whichever is less, and M is the co-existing moment acting at the section.

5.3.3.5A Alternative method

The traditional shear checking approach adds the concrete contribution to a link contribution calculated assuming a 45° truss. An alternative approach, which is arguably theoretically more correct and is based on BS EN 1992-1-1, is to assume the entire shear force to be taken by the truss but allow the truss angle to be varied to give a higher shear force. The equations given can then be optimised with respect to the angle θ to determine the best lower bound that does not exceed the relevant limits. A flatter truss angle gives a greater assessed strength for a given shear reinforcement area. However, it also gives a higher compressive stress in the concrete “struts” and a requirement to continue the main flexural reinforcement further beyond the section where it is no longer required according to purely flexural calculations.

The method can give greater benefit for links than the conventional approach, so can give higher strength. For convenience, separate equations are given for vertical links and for inclined links although the former can be derived from the latter. When using either pair of equations for members without curtailments, the angle of the struts θ will be determined first. For members with low shear stresses this will be the flattest allowed ($\tan^{-1} 0.4$). With higher shear stresses it will be such that the crushing equations (the ones including f_{cu}) give the actual shear force. The shear strength is then given by the other equation (the web crushing limit) that when the provision of links is such that the angle does not have to be much flatter than 45° can give shear strengths significantly above the BD 44 limit given in 5.3.3.1. This is particularly significant with high concrete strengths ⁽²³⁾.

The same approach can be used for members with curtailments and with short anchorage. However, the angle of the struts may need then also be limited by the curtailments.

5.3.3.6 Other approaches. With the approval of the Overseeing Organisation, methods employing plasticity theory may be used for the assessment of the shear capacity of concrete beams.

5.3.3.6A Other approaches

Methods employing the lower-bound and upper bound theorem of plasticity theory, including strut and tie methods and collapse mechanisms have been successfully applied to problems of shear in concrete, as described in reference ⁽⁵⁾. A generalised upper-bound model has been developed ⁽⁶⁾ for the shear capacity of beam-and-slab concrete bridges. However, these methods should be applied with care and following approval from the Overseeing Organisation.

5.3.3.7 Assessment of deck hinges and half joint structures. The assessment of half joints is covered in 7.2.4.2 of this Standard.

5.3.3.7A Assessment of deck hinges and half joint structures. Guidance on the assessment of bridges containing deck hinges is given in BA 93 (DMRB 3.1.5).

5.3.4 Torsion

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

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

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

Where the torsional shear stress, v_t , exceeds the value v_{tmin} given below, torsion reinforcement must be present:

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

The sum of the shear stresses resulting from shear force and torsion ($v + v_t$) must not exceed the value of the ultimate shear stress, v_{tu} , nor, in the case of small sections ($y_1 < 550\text{mm}$), shall the torsional shear stress, v_t , exceed $v_{tu} y_1 / 550$, where y_1 is the larger centre-line dimension of a link and

$$v_{tu} = 0.36 (0.7 - f_{cu} / 250) f_{cu} / \gamma_{mc}$$

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

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

As an alternative approach, combined shear and torsion may be assessed by considering the resulting shear flow in accordance with 5.3.3.5 and the corresponding approach for torsion in 6.3 of BS EN 1992-2 and 6.3 of BS EN 1992-1-1.

5.3.4.3A Stresses and reinforcement

$v_{\min}$ is 25% of the pure torsional strength without torsional reinforcement, and was chosen ⁽²⁴⁾ as the torque below which a significant reduction in shear or flexural strength of a member does not occur. Hence, there is no need to limit f_{cu} to 40N/mm² as required by BS 5400-4. The upper limit has been brought into line with that for shear.

5.3.4.4 Treatment of various cross-sections

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

$$T_u = 2 A_o \sqrt{\left(\frac{\sum A_{sL} (f_{yL} / \gamma_{ms})}{2(x_l + y_l)} \right) \left(\frac{A_{st} (f_{yv} / \gamma_{ms})}{s_v} \right)} \quad \text{Equation 10/11}$$

and

$$T_u = 2h_w A_o v_{\min}$$

where

h_w is the thickness of the thinnest wall;
 A_o is the area enclosed by the median wall line;
 A_{st} is the area of one leg of a closed link of a section;
 A_{sL} is the area of one bar of longitudinal reinforcement;
 f_{yv} is the characteristic, or worst credible, strength of the links;
 f_{yL} is the characteristic, or worst credible, strength of the longitudinal reinforcement;
 s_v is the spacing of the links along the member;
 x_l is the smaller centre-line dimension of a link;
 y_l is the larger centre-line dimension of a link;
 f_{yv} and f_{yL} must not be taken as greater than 500 N/mm².

In addition, the torsional shear stress calculated from:

$$v_t = \frac{T}{2 h_w A_o} \quad \text{Equation 9}$$

must satisfy the requirements of 5.3.4.3, where T is the torque due to ultimate loads.

(b) Rectangular sections: The ultimate torsional resistance must be taken as the greater of the value calculated from Equation 10/11 (with A_o taken as $0.8 x_l y_l$), and

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

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

In addition, the torsional shear stress calculated from:

$$v_t = \frac{2T}{h_{\min}^2 \left(h_{\max} - \frac{h_{\min}}{3} \right)} \quad \text{Equation 9(a)}$$

must satisfy the requirements of 5.3.4.3.

- (c) T, L and I sections: Such sections must be divided into component rectangles for purposes of torsional assessment. Any division into component rectangles may be chosen which is compatible with the torsional reinforcement present. Hence, any unreinforced regions of a section may be ignored for torsional assessment purposes. A component rectangle must be treated as reinforced for torsion only if its link reinforcement ties it to its adjacent rectangles.

The ultimate torsional resistance of each component rectangle must then be determined using 5.3.4.4(b), and the sectional torsional resistance taken as the sum of the torsional resistances of the component rectangles. In addition, the torsional shear stress in each component rectangle must be calculated from Equation 9(a) and must satisfy the requirements of 5.3.4.3.

Provided that the sum of the torsional resistances of the chosen component rectangles exceeds the torque due to assessment loading at the ultimate limit state, it can be assumed that the section has adequate torsional strength.

5.3.4.4A Treatment of various cross-sections

- (a) – Box sections

Equations 10 and 11 of BS 5400-4 are derived by considering a space truss model and imposing the restriction that the longitudinal and transverse steel contributions to torsional strength are equal. Equation 10/11 is the general expression for torsional strength when the longitudinal and transverse steel do not necessarily make equal contributions to the torsional strength ⁽²⁵⁾.

It should be noted that excessive torsional cracking could occur under service load conditions if the ratio of the first to second term under the square root sign of Equation 10/11 lies outside the range 2/3 to 3/2.

- (b) – Rectangular sections

Equation 10(a) of BS 5400-4 is identical to Equation 10 of BS 5400-4 if $A_o = 0.8 x_1 y_1$. Hence, Equation 10(a) is not used in the assessment code.

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

In areas subjected to simultaneous flexural compressive stress, the value of A_{sL} used in Equation 10/11 may be notionally increased by:

$$f_{cav} \text{ (area of section subject to flexural compression)} / (f_{yL} / \gamma_{ms})$$

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

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

5.3.4.5A Detailing

The BS 5400-4 link spacing limit of 300mm is intended to control cracking at the serviceability limit state and has been omitted from the assessment code. The last paragraph of the BS 5400-4 sub-clause, which relates to varying the ratio of link to longitudinal steel, is now covered by Equation 10/11.

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

5.3.6 Deflection in beams. If required by the Overseeing Organisation, deflections may be calculated in accordance with 4.2.4 and 4.6.

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

5.4 Slabs

5.4.1 Moments and shear forces in slabs. Moments and shear forces in slab bridges and in the top slabs of beam and slab, voided slab and box beam bridges may be obtained from a general elastic analysis, or such particular elastic analyses as those due to Westergaard or Pucher. Strength enhancement due to compressive membrane action can be taken into account, in accordance to the provisions of BD 81 (DMRB 3.4.20).

Non-linear methods may also be used. Alternatively, Johansen's yield line method may be used to obtain the slab strength directly.

The effective spans must be in accordance with 5.3.1.1.

5.4.1A Moments and shear forces in slabs

Guidance on the application of yield line methods of analysis for concrete slabs is given in references ⁽⁵⁾ and ⁽²⁶⁾.

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

The following formulae give the general equations which allow for any number of sets of skewed reinforcement in a slab. For the sign convention shown in Figure 5.2, the applied moment, M_n , about an axis perpendicular to the n-direction is:

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

The corresponding moments of resistance for positive (sagging) and negative (hogging) bending should be calculated separately, based on reinforcement in the bottom and top faces respectively. For a slab with j directions of reinforcement in a face, the moment of resistance, M_n^* , is given by:

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

$$\begin{aligned} M_x^* &= \sum (M_{\alpha,i}^* \cos^2 \alpha_i) \\ M_y^* &= \sum (M_{\alpha,i}^* \sin^2 \alpha_i) \\ M_{xy}^* &= \sum (M_{\alpha,i}^* \sin \alpha_i \cos \alpha_i) \end{aligned}$$

where α_i is the angle between the i -direction reinforcement and the x axis and $M_{\alpha,i}^*$ is the moment of resistance of the slab due to the i -direction reinforcement alone. This assumes the reinforcement in different directions acts

independently. Care is needed to ensure the correct sign convention is used including for M_{xy} as different sign conventions are sometimes used.

The slab has adequate capacity provided that, for all values of θ , M_n lies between the values of M_n^* calculated for sagging and hogging. The limiting case when M_n equals M_n^* for only a single value of θ arises when:

$$(M_x^* - M_x)(M_y^* - M_y) = (M_{xy}^* - M_{xy})^2$$

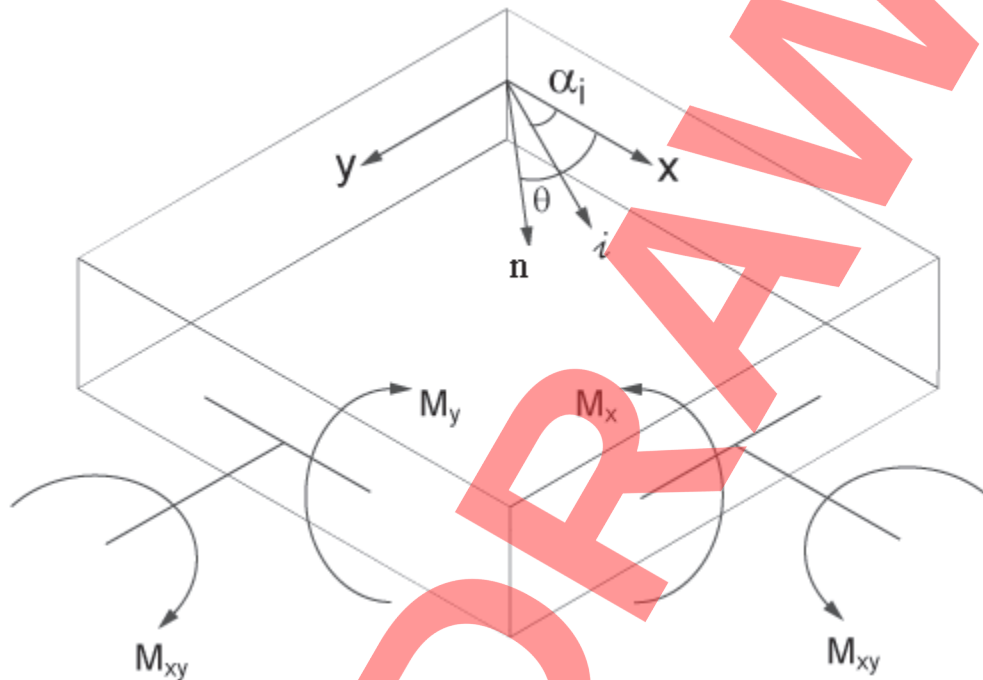


Figure 5.2 Forces in a concrete slab

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

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

If a slab of overall thickness h has n directions of reinforcement, each of which is at angle α_i to the x -axis (see Fig. 5.2) and provides a resistive tensile force in its own direction of N_i^* , then it can resist the set of in-plane forces N_x, N_y, N_{xy} if:

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

and

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

where $N_c = 0.6h f_{cu} / \gamma_{mc}$.

5.4.3A Resistance to in-plane forces

The above expressions are the general yield criterion for a slab element subjected to in-plane forces ⁽⁵⁾.

5.4.4 Shear resistance of slabs

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

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

Equation 12

where

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

The value of v must not exceed the appropriate maximum value given in 5.3.3.1 for beams.

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

- (a) b_w must be replaced with b in all equations;
- (b) shear reinforcement must not be considered as effective in slabs less than 200mm thick;
- (c) the constant 0.24 in the equation for v_c must be increased to 0.27.

	(a) Load at middle of slab	(b) Load at edge of slab	(c) Load at corner of cantilever slab	
Critical section for calculating shear resistance V_c (Critical sections (a), (b) and (c) (i) are assumed to have squared corners for rectangular and circular loaded areas)				(i) Shortest straight line which touches loaded area (ii) Shortest straight line which touches loaded area
Idealized mode of failure (only tension reinforcement shown)				Unreinforced edges Critical section
Parameters used to derive V_c from Table 8 for each portion of critical section NOTE A_s should include only tensile reinforcement which is effectively anchored.				The ratio of reinforcement should be taken as the average of the two ratios of reinforcement in the two directions $\left[\left(\frac{A_{sx}}{b_sx} + \frac{A_{sy}}{b_sy} \right) / 2 \right] V_c b \left[(d_x + d_y) / 2 \right]$
Shear resistance V_c at critical section	$\Sigma \xi_s V_c b d$ for 4 critical portions	$0.8 \Sigma \xi_s V_c b d$ for 3 critical portions	$0.8 \Sigma \xi_s V_c b d$ for 2 critical portions	

Figure 5 Parameters for shear in solid slabs under concentrated loads

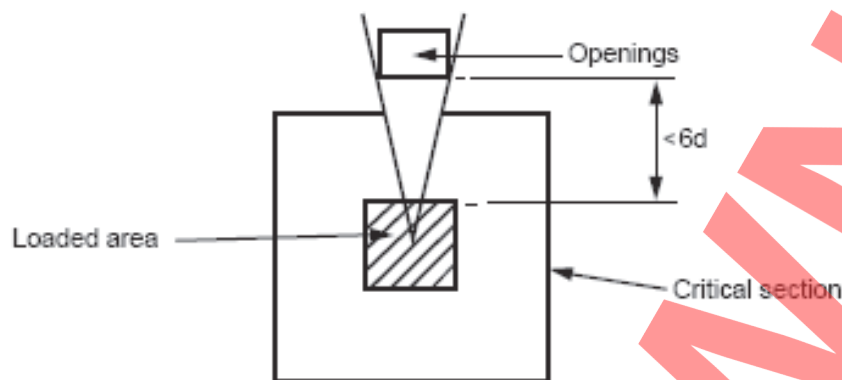


Figure 6 Openings in slabs

5.4.4.2 Shear stresses in solid slabs under concentrated loads (including wheel loads). The maximum shear capacity of a solid slab under a concentrated load must not exceed:

$$0.36 (0.7 - f_{cu} / 250) f_{cu} d u_0 / \gamma_{mc}$$

where u_0 is the perimeter of the loaded area. In this case the dispersal of wheel loads allowed in BD 37 (DMRB 1.3.14) must be taken to the top surface of the concrete slab only and not down to the neutral axis.

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

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

$$V_u = V_c + \sum A_{sv} \sin \alpha (f_{yv} / \gamma_{ms}) \leq 1.6 V_c \quad \text{Equation 13}$$

If this gives $V_u > 1.6V_c$ then the following equation must be used:

$$V_u = 1.4 V_c + \sum A_{sv} \sin \alpha (f_{yv} / \gamma_{ms}) / 3.5 \leq 2.0 V_c$$

In these equations:

- f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 N/mm^2 ;
- γ_{ms} is the material partial safety factor for steel given in 4.3.3;
- α is the inclination of the shear reinforcement to the plane of the slab;
- V_c is the shear resistance of the concrete;
- $\sum A_{sv}$ is the area of shear reinforcement within the area between the loaded area and the critical perimeter, except for case (c)(ii) of Figure 5 when it is the area of shear reinforcement within a distance from the load equal to the effective depth.

However, shear reinforcement must be considered effective only if $\sum A_{sv} \sin \alpha (f_{yv} / \gamma_{ms}) \geq 0.2 \sum bd$, where $\sum bd$ is the area of the critical section;

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

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

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

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

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

5.4.4.2A Shear stresses in solid slabs under concentrated loads (including wheel loads)

All shear reinforcement within the critical perimeter is considered effective. The upper limit to the shear force has been made more conservative and the effectiveness of large areas of links has been reduced in line with BS 8110-1. The upper limit on f_{yv} has been increased to 500 N/mm^2 (see 5.3.3.2A).

The reduction factor of 0.8 in Figure 5 (b) and (c)(i) was introduced into building codes to allow for moment transfer at edge and corner columns ⁽²⁷⁾.

Enhancement of v_c has been permitted for short shear spans.

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

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

The longitudinal shear resistance of a circular voided slab may be calculated in accordance with the formulae below provided that the following criteria are met:

- i. ϕ/b is not greater than 0.8, where ϕ is the diameter of the void and b is the distance between void centres.
- ii. ϕ/h is no greater than 0.75 where h is the overall depth of the slab.
- iii. The thickness of the compression flange is not be less than $0.35(h-\phi)$.

The shear capacity of a circular voided slab, V_{cv} , can then be derived from:

$$V_{cv} = K V_c$$

where

V_c is the shear resistance of the solid slab ignoring the presence of voids, calculated in accordance with 5.4.4.2.

K is a variable reduction factor based on the structure's geometry and may be taken as:

$$K = 1 - \{0.4(\phi/b) + 0.6(\phi/b)^{2.5}\}$$

Alternatively, the web between the voids may be assessed in accordance with 5.3.3.5.

5.4.4.3A Shear in voided slabs

The BS 5400-4 requirement to include shear due to torsion when checking the flanges has been omitted because the torsional shear flow in a flange is perpendicular to the flexural shear flow. Alternative methods of checking flanges for transverse effects, which are based on Vierendeel action, are available ⁽⁴⁾.

The formulae for the longitudinal shear resistance of a circular voided slab are based on reference ⁽²⁸⁾. Guidance on the punching of loads through a voided slab as a whole is given in reference ⁽²⁹⁾.

5.4.5 Deflection of slabs. If required by the Overseeing Organisation, deflections may be calculated in accordance with 4.2.4 and 4.6.

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

5.4.7 Torsion in slabs

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

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

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

5.5 Columns

5.5.1 General

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

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

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

Otherwise it must be considered as slender.

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

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

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

Case 4 from Table 11 may be used for columns which are restrained at the base and which have roller bearings at the top, provided the rollers are equipped with racks or other effective means to maintain them in position.

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

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

5.5.1.2A Effective height of column

The effective heights in Table 11 relate to idealised situations. In design, the designer can compare the actual bearing condition with the idealised conditions of Table 11 and choose a conservative effective height. This approach is also applicable to assessment. However, in assessment, it may be necessary to make a more accurate estimate of the effective height in order to prove the adequacy of a particular column. The assessor should then consult specialist literature⁽³⁰⁻³²⁾. Reference should also be made to 5.8.3.2 of BS EN 1992-1-1.

5.5.1.3 Slenderness limits for columns. In each plane of buckling, the ratio l_e/h must not exceed 60. If this slenderness limit is exceeded a full non-linear analysis should be undertaken.

5.5.1.3A Slenderness limits for columns

The BS 5400-4 slenderness limit of l_e/h of 40 was chosen because it was considered to be a practical upper limit⁽⁴⁾. However, the study⁽³³⁾ on which the BS 5400-4 column sub-clauses are based, included l_e/h values of up to 60. Hence, the latter limit, which was also in BS 5400-4:1978, has been adopted in the assessment code.

The BS 5400-4 limit on l_e/h of 30 for a column not restrained in position at one end is intended to control service load lateral displacements. It has been omitted from the assessment code which is concerned predominantly with ultimate rather than service load behaviour.

When a full non-linear analysis is required, it may be performed in accordance with the method described in 5.8.6 of BS EN 1992-1-1, using the stress-strain relationships and material properties for concrete and steel defined in 4.3.2, with the effect of creep allowed for by multiplying all strain values in the concrete stress-strain diagram by a factor of 2.

5.5.1.4 Assessment of strength. Sub-clauses 5.5.2 to 5.5.8 give methods for assessing the strength of columns at the ultimate limit state, which are based on a number of assumptions. These methods may be used provided the assumptions are realized for the case being considered and the effective height is determined accurately.

In addition, for columns subject to applied bending moments, crack widths may need to be calculated at the serviceability limit state if required by the Overseeing Organisation (see 4.1).

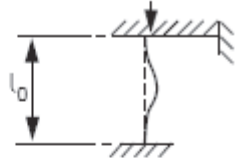
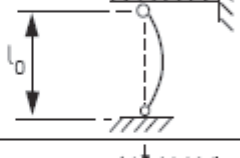
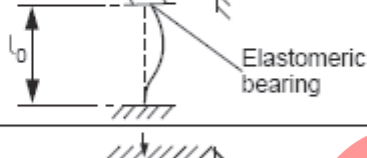
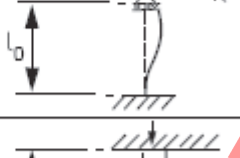
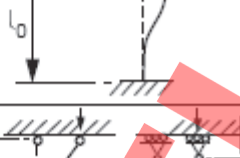
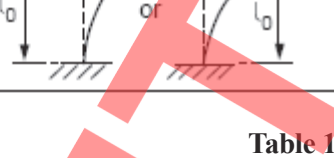
Case	Idealised column and buckling mode	Restraints			Effective Height, l_e
		Location	Position	Rotation	
1		Top	Full	Full	0.70 l_0
		Bottom	Full	Full	
2		Top	Full	None	0.85 l_0
		Bottom	Full	Full	
3		Top	Full	None	1.0 l_0
		Bottom	Full	None	
4		Top	None	None	1.3 l_0
		Bottom	Full	Full	
5		Top	None	None	1.4 l_0
		Bottom	Full	Full	
6		Top	None	Full	1.5 l_0
		Bottom	Full	Full	
7		Top	None	None	2.3 l_0
		Bottom	Full	Full	

Table 11 Effective height, l_e , for columns

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

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

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

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

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

- (a) The strain distribution in the concrete in compression and the compressive and tensile strains in the reinforcement are derived from the assumption that plane sections remain plane.
- (b) The stresses in the concrete in compression are either derived from the stress-strain curve in Figure 1 with the appropriate value of γ_{mc} from 4.3.3.3, or taken as equal to $0.6 f_{cu}/\gamma_{mc}$ over the whole compression zone where this is rectangular or circular. In both cases, the concrete strain at the outermost fibre at failure is taken as 0.0035.
- (c) The tensile strength of the concrete is ignored.
- (d) The stresses in the reinforcement are derived from 4.3.2.2 with the appropriate value of γ_{ms} from 4.3.3.3.

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

5.5.3.2A Analysis of sections

See comment on 5.3.2.1A regarding the enhancement of concrete strength and failure strain arising from restraining links or helical binding.

5.5.3.3 Design charts for rectangular and circular columns. "Not applicable to assessment"

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

$$N_u = (0.6 f_{cu} / \gamma_{mc}) b d_c + (f_y / \gamma_{ms}) A'_{sl} + f_{s2} A_{s2} \quad \text{Equation 14}$$

$$M_u = (0.3 f_{cu} / \gamma_{mc}) b d_c (h - d_c) + (f_y / \gamma_{ms}) A'_{sl} \left(\frac{h}{2} - d' \right) - f_{s2} A_{s2} \left(\frac{h}{2} - d_2 \right) \quad \text{Equation 15}$$

where

N is the ultimate axial load applied on the section considered;

M is the moment applied about the axis considered due to ultimate loads including the allowance for construction tolerance (see 5.5.3.1);
 N_u, M_u are the ultimate axial load and bending capacities of the section for the particular value of d_c assumed;
 f_{cu} is the characteristic, or worst credible, cube strength of the concrete;
 b is the breadth of the section;
 d_c is the depth of concrete in compression assumed subject to a minimum value of $2d'$;
 A'_{sl} is the area of compression reinforcement in the more highly compressed face;
 f_{s2} is the stress in the reinforcement in the other face, derived from Figure 2 and taken as negative if tensile;
 A_{s2} is the area of reinforcement in the other face which may be considered as being:
 (1) in compression,
 (2) inactive, or
 (3) in tension,
 as the resultant eccentricity of load increases and d_c decreases from h to $2d'$;
 h is the overall depth of the section in the plane of bending;
 d' is the depth from the surface to the reinforcement in the more highly compressed face;
 d_2 is the depth from the surface to the reinforcement in the other face;
 f_y is the characteristic or worst credible strength of reinforcement.

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

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

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

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

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 (see preceding paragraph);
 M_{ux} is the ultimate moment capacity about the major x-x axis assuming an ultimate axial load capacity, N_u , not less than the value of the ultimate axial load, N ;
 M_{uy} is the ultimate moment capacity about the minor y-y axis assuming an ultimate axial load capacity, N_u , not less than the value of the ultimate axial load, N ;
 α_n is given by:

$$\alpha_n = 0.67 + 1.66 N_u / N_{uz} \quad \text{but not } < 1.0 \text{ and not } > 2.0$$

where

N_{uz} is the axial loading capacity of a column ignoring all bending, taken as:

$$N_{uz} = (0.675 f_{cu} / \gamma_{mc}) A_c + f_y A_{sc} \quad \text{Equation 17}$$

f_{cu} and f_y are as defined in 5.5.3.4

A_c is the area of concrete

A_{sc} is the total area of longitudinal reinforcement.

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

5.5.4A Short columns subject to axial load and either bending about the major axis or biaxial bending

The expression for α_n gives the same values as Table 12 of BS 5400-4. It should be noted that the α_n values are conservative ⁽⁴⁾. Hence, a column which is apparently inadequate when using Equation 16 could, possibly, be shown to be adequate if assessed in accordance with 5.5.3.2.

The BS 5400-4 sub-clause states that Equation 16 is also applicable to circular columns. However, a circular column subject to biaxial bending can be assessed for the resultant moment about a single axis.

5.5.5 Slender columns

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

5.5.5.1A General

The additional moment approach to allowing for lateral column deflection of 5.5.5.2 to 5.5.5.4 can be very conservative for certain end restraint conditions ⁽³²⁾. Hence, in some cases it may be preferable to carry out a full non-linear analysis ⁽³²⁻³⁶⁾, which may be performed in accordance with the method described in 5.8.6 of BS EN 1992-1-1, using the stress-strain relationships and material properties for concrete and steel defined in 4.3.2, with the effect of creep allowed for by multiplying all strain values in the concrete stress-strain diagram by a factor of 2. A circular column subject to biaxial bending can be assessed for the resultant moment about a single axis.

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

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

where

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

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

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

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

$$M_{iy} = 0.4 M_1 + 0.6 M_2 \quad \text{Equation 19}$$

where

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

M_2 is the larger initial end moment due to ultimate loads (assumed positive).

M_{iy} must not be taken as less than $0.4 M_2$ or such that M_{iy} is less than M_2 .

5.5.5.3 Slender columns bent about a major axis. When the overall depth of its cross-section, h_y , is less than three times the width, h_x , a slender column bent about the major x-x axis must be assessed for its ultimate axial load, N , together with the moment M_{ix} given by:

$$M_{tx} = M_{ix} + \frac{N h_y}{1750} \left(\frac{l_e}{h_x} \right)^2 \left(1 - \frac{0.0035 l_e}{h_x} \right) \quad \text{Equation 20}$$

where

l_e , h_x are as defined in 5.5.5.2;

M_{ix} is the initial moment due to ultimate loads, but not less than that corresponding to the allowance for construction tolerances as given in 5.5.4;

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

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

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

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

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

where

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

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

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

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

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

5.5.6 Shear resistance of columns. A column subject to uniaxial shear due to ultimate loads must be assessed in accordance with 5.3.3 except that the ultimate shear stress, $\xi_s v_c$, may be multiplied by:

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

where

N is the ultimate axial load (in Newtons) $< 0.11f_{cu} A_c$
 A_c is the area of the entire concrete section (in mm²)

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

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

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

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

5.5.6A Shear resistance of columns

The shear strength enhancement factor to allow for the axial load is that adopted in BS EN 1992-1-1 and is less conservative than the BS 5400-4 factor. The requirements for calculating the shear capacity of a circular column are based on those in reference ⁽³⁷⁾.

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

5.5.8 Bearing on columns. Bearing stresses due to ultimate loads of a purely local nature, as at girder bearings, must be limited in accordance with 7.2.3.3.

5.6 Reinforced concrete walls

5.6.1 General

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

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

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

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

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

5.6.1.2A Limits to slenderness

See comments in 5.5.1.2A and 5.5.1.3A.

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

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

Where the axial load is non-uniform, consideration must be given to deep beam effects and the distribution of axial loads per unit length of wall. It will generally be necessary to consider the maximum and minimum ratios of moment to axial load in assessing a wall.

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

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

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

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

5.6.4A Slender reinforced walls

See comment in 5.5.5.1A.

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

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

where

N is the ultimate load (in Newtons) $< 0.11f_{cu}A_c$;
 A_c is the area of entire concrete section (in mm²).

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

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

where

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

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

5.6.5A Shear resistance of reinforced walls

See comment on 5.5.6.

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

5.6.7 Crack control in reinforced walls. If required, flexural crack widths in walls subject to bending must be calculated in accordance with 5.8.8.2.

5.7 Bases

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

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

- (a) Where the base is axially loaded, the reactions to ultimate loads are uniformly distributed per unit area or per pile.
- (b) Where the base is eccentrically loaded, the reactions vary linearly across the base. For columns and walls restrained in direction at the base, the moment transferred to the base must be obtained from 5.5.

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

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

5.7.3 Assessment of bases

5.7.3.1 Resistance to bending. Bases must be assessed in accordance with 5.4, and must be capable of resisting the total moments and shears at the sections considered.

Where the width of the section considered is less than or equal to $1.5(b_{\text{col}} + 3d)$, where b_{col} is the width of the column and d is the effective depth to the tension reinforcement of the base, all reinforcement crossing the section may be considered to be effective in resisting bending. For greater widths, all reinforcement within a band of width $(b_{\text{col}} + 3d)$ centred on the column may be considered to be effective and the area of effective reinforcement outside this band should be taken as the lesser of:

- (a) the actual area of reinforcement outside the band, and
- (b) 50 % of the area of reinforcement within the band.

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

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

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

An alternative method of analysis for a slab base is yield line theory.

5.7.3.1A Resistance to bending

When assessing pile caps by truss analogy using strut and tie systems, further guidance on sizing of struts, ties and nodes can be found in BS EN 1992-1-1.

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

- (a) Shear along a vertical section extending across the full width of the base, at a distance equal to the effective depth from the face of the loaded area, where the requirements of 5.4.4.1 apply.
- (b) Punching shear around the loaded area, where the requirements of 5.4.4.2 apply.

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

- (1) Shear along any vertical section extending across the full width of the cap. The requirements of 5.4.4.1 apply, except that the enhancement of the shear resistance for sections close to supports (see 5.3.3.3) must be applied only to strips of width not greater than twice the pile diameter centred on each pile. Where a_v is taken as the distance between the face of the column or wall and the nearer edge of the piles, it must be increased by 20% of the pile diameter. In applying 5.4.4.1, the allowable ultimate shear stress must be taken as the average over the whole section.
- (2) Punching shear around loaded areas, where the requirements of 5.4.4.2 apply. When considering case (c)(ii) of Figure 5, the allowable ultimate shear stress may be enhanced in accordance with 5.3.3.3, over a width

not greater than twice the pile diameter centred on the corner pile. The short shear span enhancement factor permitted for punching shear in 5.4.4.2 will often be beneficial when assessing pile caps.

5.7.3.2A Shear

With reference to the note (1) on the shear strength of pile caps, the short shear span enhancement factor is greater than the BS 5400-4 value (see comment on 5.3.3.3). BS 8110-1 allows shear enhancement over a much greater width than BS 5400-4. This rule is a relaxation of the BS 5400-4 rule but is still more conservative than BS 8110-1.

Difficulties can arise in applying 5.4.4.2 and Figure 5 to the assessment of certain pile caps (e.g. circular pile caps with circumferential and radial bars). It is not possible to give general recommendations to cover all such situations, and it is necessary to consider the actual punching shear failure surfaces which could occur. Useful information is given in references ⁽⁹⁾, ⁽²⁷⁾ and ⁽³⁸⁾.

5.7.3.3 Bond and anchorage. The requirements of 5.8.6 apply to reinforcement in bases.

5.7.3.3A Bond and anchorage

The local bond sub-clause of BS 5400-4 is not relevant: see 5.8.6.2.

5.7.4 Deflection of bases. The deflection of bases need not be considered.

5.7.5 Crack control in bases. If required, crack widths may be calculated in accordance with 5.8.8.2 taking into account the type of base and treatment of assessment (see 5.7.3.1).

5.8 Considerations of details

5.8.1 Constructional details

5.8.1.1 Size of members. *“Not applicable to assessment”*

5.8.1.2 Accuracy of position of reinforcement. When the reduced material partial safety factor for steel of 1.05 given in 4.3.3.3 is adopted for the worst credible strengths of steel reinforcement other than grade 460, then covers and effective depths must be measured.

5.8.1.2A Accuracy of position of reinforcement

Guidance on the measurement of the location of reinforcement can be found in reference ⁽³⁹⁾.

5.8.1.3 Construction joints. *“Not applicable to assessment”*

5.8.1.4 Movement joints. *“Not applicable to assessment”*

5.8.2 Concrete cover to reinforcement. ‘Nominal’ cover is that dimension used in design and indicated on the drawings. In accordance with the provisions of BS 5400-4, the actual cover may be up to 5mm less than the nominal cover.

The nominal cover indicated on the drawings should be not less than the size of the bar or maximum aggregate size, plus 5mm; in the case of a bundle of bars (see 5.8.3.1 and 5.8.8.1), it should be equal to or greater than the size of a single bar of equivalent area plus 5mm.

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

If the nominal cover is less than the values defined above, the bond strength of the reinforcing steel could be reduced and the assessor must exercise appropriate engineering judgment in evaluating its contribution to the load capacity.

5.8.2A Concrete cover to reinforcement

Adequate cover to reinforcement should ideally be present in all concrete structures designed to standards containing appropriate provisions for durability, and the cover generally varies with the concrete grade and with the particular condition of exposure. When the cover is less than the size of the bar, bond strength could be reduced ⁽⁴⁰⁾, and reinforcement corrosion is most likely to occur. Guidance on the assessment of concrete structures affected by steel corrosion and with low cover is given in BA 51 (DMRB 3.4.13).

5.8.3 Reinforcement: general considerations

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

- (1) the bundle is restrained by links;
- (2) the bars in a bundle terminate at different points spaced at least 40 times the bar size apart except for bundles stopping at a support;
- (3) bars in pairs or bundles of three may be lapped one bar at a time, but the laps must be so staggered that in any cross-section there are no more than four bars in a bundle.

5.8.3.2 Bar schedule dimensions. *“Not applicable to assessment”*

5.8.4 Minimum area of reinforcement in members

5.8.4.1 Minimum area of main reinforcement. *“Not applicable to assessment”*

5.8.4.1A Minimum area of main reinforcement

The minimum areas of tension reinforcement in a beam or slab specified in BS 5400-4 are intended to ensure that the reinforcement does not yield as soon as cracking occurs, and wide cracks are thereby avoided. This may also be achieved by ensuring that the area of tension reinforcement is not less than $0.167 b_a d (f_t / f_y)$ ⁽⁴⁾ where f_t is the flexural tensile strength of the concrete, which may, in the absence of other information, be taken as $0.556 \sqrt{f_{cu}}$. The BS 5400-4 values can be obtained from this expression by assuming a value of 50 N/mm² for f_{cu} . Although it is not considered necessary to impose a minimum steel area in a beam or slab for assessment purposes, the assessor should be aware that if a section has less reinforcement than the specified minimum it may have adequate strength but could develop wide cracks.

The minimum number of longitudinal bars present in a column should be four in rectangular columns and six in circular columns. The BS 5400-4 minimum bar diameter of 12mm is intended to ensure a rigid cage for construction. This requirement is not relevant to assessment. The BS 5400-4 minimum steel areas for columns ensure that reinforcement yield does not occur under service load conditions ⁽⁴⁾. Although it is not considered necessary to impose a minimum steel area for assessment purposes, the assessor should be aware that high service load stresses can occur in columns having less than the BS 5400-4 minimum steel areas.

A wall should not be considered as a reinforced concrete wall unless the percentage of vertical reinforcement provided is at least 0.4% of the gross cross-sectional area of the concrete. This vertical reinforcement may be in one or two layers. Failure to comply with the above requirement may result in large cracks developing.

5.8.4.2 Minimum area of secondary reinforcement

In a solid slab or wall, the main reinforcement may be considered able to resist compression if the area of secondary reinforcement restraining the main 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. The diameter of the secondary bars should not be less than one-quarter of the size of the main bars and the spacing should not exceed 300mm.

When there is less secondary reinforcement than the specified minimum, the compressive strength of the bars must be reduced in proportion to the ratio of the actual to specified minimum secondary steel areas, or alternatively the reinforcement acting in compression omitted altogether from the respective strength calculations.

5.8.4.2A Minimum area of secondary reinforcement

Wide cracks may develop if the following minimum amounts of reinforcement are not present:

- (1) In the predominantly tensile area of a solid slab or wall, the minimum area of secondary reinforcement should be not less than that given in the first paragraph of 5.8.4.1A.
- (2) In beams where the depth of the side face exceeds 600mm, longitudinal reinforcement should be present having an area of at least 0.05% of $b_t d$ on each face with a spacing not exceeding 300mm, where:

 b_t is the breadth of the section at the level of tension reinforcement;
 d is the effective depth to tension reinforcement.
- (3) In a voided slab, the amount of transverse reinforcement, expressed as a percentage of the minimum flange cross-sectional area, should be at least 0.6% in the case of high strength steel and 1% in the case of mild steel. These minimum areas are intended to prevent the first crack from immediately passing through the flange thickness ⁽⁴⁾ whereas the minimum areas given in 5.8.4.1 merely ensure that the steel will not yield at first cracking ⁽⁴¹⁾.

The purpose of the minimum amount of secondary steel in beams and slabs with compression reinforcement is to restrain the latter reinforcement so that its full compressive strength can be developed. It should be noted that the secondary reinforcement has to be arranged outside the main reinforcement for it to be considered as having the potential to restrain the main reinforcement effectively.

The BS 5400-4 reference to early shrinkage and thermal cracking is not relevant to assessment.

5.8.4.3 Minimum area of links

When, in a beam or column, part or all of the main reinforcement is required to resist compression, links or ties at least one-quarter the size of the largest compression bar should be present at a maximum spacing of 12 times the size of the smallest compression bar. Links should be so arranged that every corner and alternate bar or group in an outer layer of reinforcement is supported by a link passing round the bar and having an included angle of not more than 135°. All other bars or groups within a compression zone should be within 150mm of a restrained bar in order to be considered effective in resisting compression.

For circular columns, where the longitudinal reinforcement is located round the periphery of a circle, adequate lateral support is provided by a circular tie passing round the bars or groups.

When the percentage of reinforcement required to resist compression in the compression face of a wall or slab exceeds 1%, links at least 6 mm or one-quarter of the size of the largest compression bar, whichever is the greater, should be present through the thickness of the member. The spacing of these links should not exceed twice the member thickness in either of the two principal directions of the member and be not greater than 16 times the bar size in the direction of the compressive force.

When in a beam, column, wall or slab there is less link reinforcement than that specified in the above paragraphs, the compressive strength of the bars must be reduced in proportion to the ratio of the actual to specified minimum link areas, or alternatively the reinforcement acting in compression omitted altogether from the respective strength calculations.

5.8.4.3A Minimum area of links

These minimum link requirements are intended to ensure restraint of compression bars so that their full compressive strength can be developed.

As discussed in 5.3.3.2 and 5.3.3.2A, for links to be effective in a beam their spacing should not exceed the effective depth of the beam, nor should the lateral spacing of the individual legs of the links exceed this value.

5.8.5 Maximum areas of reinforcement in members. *“Not applicable to assessment”*

5.8.5A Maximum areas of reinforcement in members

Maximum steel areas are specified in BS 5400-4 to ensure that concrete can be placed and compacted easily. These maxima are not directly relevant to assessment. However where the steel areas exceed the BS 5400-4 maxima (4% in beams, slab and walls, and 6% to 10% in columns) the concrete could be poorly compacted.

5.8.6 Bond, anchorage and bearing

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

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

Type 2 A bar with transverse ribs with a substantial uniform spacing not greater than 0.8ϕ (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 $0.15 \text{ mm}^2/\text{mm}$ where ϕ is the size (nominal diameter) of the bar.

5.8.6.2 Local bond. *“Not applicable to assessment”*

5.8.6.2A Local bond

Local bond stress is not considered applicable in assessment provided that at both sides of any cross section, the force in each bar is developed by an appropriate embedment length or other end anchorage. Hence only anchorage bond need to be considered.

5.8.6.3 Anchorage bond. To prevent bond failure the tension or compression in any bar at any section due to ultimate loads must be developed on each side of the section by an appropriate embedment length or other end anchorage. The anchorage bond stress, assumed to be constant over the effective anchorage length, taken as the force

in the bar divided by the product of the effective anchorage length and the effective perimeter of the bar or group of bars (see 5.8.6.4), must not exceed the value $\beta \sqrt{f_{cu}} / \gamma_{mb}$ where:

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

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

Table 15 Values of bond coefficient β

5.8.6.3A Anchorage bond

The allowable ultimate anchorage bond stress expression is that given in BS 8110-1 and gives values almost identical to the BS 5400-4 Table 15 values. It should be noted that values have been included for fabric.

BS 8110-1 specifies a partial safety factor on the bond stress of 1.4. This partial safety factor allows for variations in both concrete strength and bond strength (when the concrete strength is constant). If the worst credible concrete strength is used it is reasonable to reduce the partial safety factor (see 4.3.3.3A). If it is assumed that γ_{mb} can be expressed as $\sqrt{(\gamma_{mc} \gamma_{mbs})}$, where γ_{mc} allows for the variation in concrete strength and γ_{mbs} allows for the variation in bond strength, then with γ_{mc} and γ_{mb} equal to their design values of 1.5 and 1.4, respectively, $\gamma_{mbs} = 1.31$. Hence, if γ_{mc} is equal to its assessment value of 1.20 when using the worst credible concrete strength (see 4.3.3.3), $\gamma_{mb} = \sqrt{(1.20 \times 1.31)} = 1.25$.

The BS 5400-4 allowable ultimate anchorage bond stresses in Table 15 are functions of only concrete strength, bar type and whether the bar is in tension or compression. Hence, the bond failure mechanism is grossly simplified by BS 5400-4 because it is assumed that the code's covers, nominal link requirements and detailing sub-clauses will be satisfied. In an assessment these various sub-clauses are often not satisfied and it may be necessary to express allowable ultimate anchorage bond stresses in terms of the additional variables of cover, bar diameter and spacing, quantity and arrangement of restraining reinforcement, lateral pressure applied by external loads or reactions and location of bar within the member. Further guidance on these aspects can be obtained from references (40) and (42-48). Reference should also be made to 8.4, 8.5 and 8.6 of BS EN 1992-1-1. Methods of analysis to evaluate the post-slip behaviour of partially anchored bars should be agreed with the Overseeing Organisation.

5.8.6.4 Effective perimeter of a bar or group of bars. The effective perimeter of a bar may be taken as 3.14 times the nominal size. The effective perimeter of a group of bars (see 5.8.3.1) must be taken as the sum of the effective perimeters of the individual bars multiplied by $(1.2-0.2N)$, where $N (\leq 4)$ is the number of bars in the group.

5.8.6.4A Effective perimeter of a bar or group of bars

The multiplier $(1.2-0.2N)$ gives the same values as Table 16 of BS 5400-4. Test data do not appear to be available for more than 4 bars in a group.

5.8.6.5 Anchorage of links. A link may be considered to be fully anchored if it passes round another bar through an angle of 90° and continues beyond for a minimum length of eight times its own size, or through 150° and continues for a minimum length of four times its own size. Where full anchorage of links is not achieved, its effective size must be taken as the equivalent bar diameter that the anchorage provides.

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

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

The strength of joints using the methods given in (c) and (d) and any other method not listed must be verified by test evidence (see 7.3.2.1).

5.8.6.7 Lap lengths. When bars are lapped, the strength of the lap must be derived from 5.8.6.3 from the anchorage bond of the smaller of the two bars lapped. The lap strength as calculated above must be reduced for bars in tension by a factor of 1.4 if any of the following conditions apply:

- (a) the cover to the lapped bars from the top of the section as cast is less than twice the bar size;
- (b) the clear distance between the lap and another pair of lapped bars is less than 150 mm;
- (c) a corner bar is lapped and the cover to either face is less than twice the bar size.

Where conditions (a) and (b) or conditions (a) and (c) apply the lap strength must be reduced by a factor of 2.0.

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

The factors of 1.4 and 2.0 should be applied only to the calculated lap strengths of bars in tension.

5.8.6.8 Hooks and bends. Hooks, bends and other reinforcement anchorages should be of such form, dimension and arrangement as to avoid overstressing the concrete.

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

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

The radius of the bend must not be less than twice the radius of the test bend guaranteed by the manufacturer of the bar. However, it will be sufficient to ensure that the bearing stress at the mid-point of the curve does not exceed the value given in 5.8.6.9.

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

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

5.8.6.8A Hooks and bends

An additional paragraph has been added to the BS 5400-4 sub-clause to clarify the anchorage value of hooks and bends which do not satisfy the BS 5400-4 requirements.

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

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

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

where

- F_{bt} is the tensile force due to ultimate loads in a bar or group of bars;
 r is the internal radius of the bend;
 ϕ is the size of the bar or, in a bundle, the size of a bar of equivalent area.

The stress must not exceed

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

where

- a_b for a particular bar or group of bars in contact must be taken as the centre-to-centre distance between bars or groups of bars perpendicular to the plane of the bend; for a bar or group of bars adjacent to the face of the member, a_b must be taken as the cover plus ϕ . The ratio of a_b/ϕ must not exceed 8.
 l_1 is the length of the bar measured inside the bend and bearing on to the concrete.
 l is the thickness of concrete member in the plane of the bend, but not greater than $3l_1$.

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

5.8.6.9A Bearing stress inside bends

The allowable bearing stress expression is based on tests ⁽⁴⁹⁾ which show the BS 5400-4 requirement to be conservative. The maximum values of $l = 3l_1$ and $a_b/\phi = 8$ represent the limits of the test evidence available.

5.8.7 Curtailment and anchorage of reinforcement. Curtailment lengths and anchorages of bars must be assessed either by rigorous analysis at the curtailment or anchorage point for the worst load case in accordance with 5.8.6.3. Where the actual anchorage length of a bar is less than the full anchorage length required from 5.8.6.3, its effective area may be reduced in proportion to the ratio between the actual anchorage length and the full anchorage length.

5.8.7A Curtailment and anchorage of reinforcement

The assessment is based on a rigorous analysis of the forces at the curtailment point for the worst load case. In carrying out such an analysis the actual bending moment distributions need to be considered, and it is also essential to take account of the fact that the tension reinforcement has to resist tensile forces which arise from both the bending moment and the shear force at the section under consideration ^(4, 50-51). This is particularly relevant when the alternative approach of 5.3.3.5 based on BS EN 1992-1-1 is used.

5.8.8 Spacing of reinforcement

5.8.8.1 Minimum distance between bars. “Not applicable to assessment”

5.8.8.1A Minimum distance between bars

Minimum bar spacings are specified in BS 5400-4 to aid placing and compacting of concrete. These minima are not directly relevant to assessment. However, in sections where the bar spacings are less than the BS 5400-4 minima, the concrete could be poorly compacted and particular attention should be given to inspection of such sections.

5.8.8.2 Maximum distance between bars in tension. When required by the Overseeing Organisation as part of serviceability limit state checks, crack widths under the specified loads must be calculated in accordance with the following:

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

$$\text{Crack width} = \frac{3 a_{cr} \varepsilon_m}{1 + 2(a_{cr} - c)/(h - d_c)} \quad \text{Equation 24}$$

where

a_{cr} is the distance from the point (crack) considered to the surface of the nearest bar which controls the crack width;

c_{nom} is the nominal cover to the outermost reinforcement (see 5.8.2);

c is the effective cover to the reinforcement which controls the width of the cracks under consideration and must be taken as the lesser of (a) actual cover to this reinforcement and (b) perpendicular distance from this reinforcement to a surface at a distance c_{nom} from the outermost bars;

d_c is the depth of the concrete in compression (if $d_c = 0$ the crack widths must be calculated using Equation 26);

h is the overall depth of the section;

ε_m is the calculated strain at the level where cracking is being considered, allowing for the stiffening effect of the concrete in the tension zone; a negative value of ε_m indicates that the section is uncracked. The value of ε_m must be obtained from the equation:

$$\varepsilon_m = \varepsilon_1 - \left[\frac{3.8 b_t h (a' - d_c)}{\varepsilon_s A_s (h - d_c)} \right] \left[\left(1 - \frac{M_q}{M_g} \right) \times 10^{-9} \right] \quad \text{Equation 25}$$

but not greater than ε_1 .

where

- ϵ_1 is the calculated strain at the level where cracking is being considered, ignoring the stiffening effect of the concrete in the tension zone;
- b_t is the width of the section at the level of the centroid of the tension steel;
- a' is the distance from the compression face to the point at which the crack width is being calculated;
- M_g is the moment at the section considered due to permanent loads;
- M_q is the moment at the section considered due to live loads;
- ϵ_s is the calculated strain at the centroid of reinforcement, ignoring the stiffening effect of the concrete in the tension zone;
- A_s is the area of tension reinforcement.

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

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

where

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

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

$$\text{Crack width} = 3 a_{cr} \epsilon_m \quad \text{Equation 26}$$

where ϵ_m is obtained from Equation 25.

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

$$\text{Crack width} = 1.2 \epsilon_m (h_f / \rho_{net}) \sqrt{c/\phi} \quad \text{Equation 26A}$$

where

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

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

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

5.8.8.2A Maximum distance between bars in tension

The main reason for limiting maximum bar spacings in design is to control crack widths. Assessment sub-clause 5.8.8.2 is very similar to the BS 5400-4 sub-clause but has been rearranged as a “crack width calculation” sub-clause. The general spacing of 300mm and the limit for voided slabs in (d) of the BS 5400-4 sub-clause have been omitted because they are not directly relevant to assessment.

In (b), Equation 26 is not appropriate for calculating crack widths in voided slabs subjected to transverse bending. An additional Equation, 26A, has been added for these situations, which is based on ⁽⁴¹⁾. Similarly, a more appropriate tension stiffening formula has been added ^(52, 53).

The approaches given in (c) are conservative. A more accurate method is to base calculations on the strains due to the combined global and local effects.

5.8.9 Shrinkage and temperature reinforcement. *“Not applicable to assessment”*

5.8.10 Arrangement of reinforcement in skew slabs. *“Not applicable to assessment”*

5.8.10A Arrangement of reinforcement in skew slabs

Many existing skew slab bridges have very small amounts of transverse reinforcement compared with the amounts required to comply with current design standards. However, a small amount of transverse reinforcement does not necessarily imply that the bridge is inadequate. When required, yield line theory can often be used to demonstrate that such a bridge has adequate strength, although such decks may suffer from serviceability problems due to premature yielding of the transverse reinforcement, and may warrant more frequent inspection and maintenance. In using yield line theory it is not always possible to state in advance which will be the critical collapse mechanism, particularly for continuous decks. Hence, the assessing engineer will need to consult specialist literature. Guidance on the application of yield line methods of analysis for concrete slabs is given in references ⁽⁵⁾ and ⁽²⁶⁾.

5.9 Additional considerations in the use of lightweight aggregate concrete

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

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

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

5.9.2 Durability. *“Not applicable to assessment”*

5.9.3 Strength of concrete. See 5.1.4.2.

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

5.9.4A Shear resistance of beams

BS 5400-4 applies a reduction factor of 0.8 to v_c for lightweight aggregate concrete. The higher values in this Standard are consistent with test data ⁽⁵⁴⁾.

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

5.9.6 Deflection of beams. Where required, deflection of lightweight aggregate concrete beams may be calculated using a value of the modulus of elasticity of concrete as described in 4.3.2.1.

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

5.9.8 Deflection of slabs. Where required, deflection of lightweight aggregate concrete slabs may be calculated using a value of the modulus of elasticity of concrete as described in 4.3.2.1.

5.9.9 Columns

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

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

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

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

In lightweight aggregate concrete members containing foamed slag however, bond stresses must not exceed 50% of those given in 5.8.6.3 for reinforcement that was in a horizontal position during casting.

5.9.10A Local bond, anchorage bond and laps

Anchorage bond stresses for mild steel bars embedded in lightweight aggregate concrete have been increased from 50% to 80% of those for normal weight aggregate concrete (see BS 8110-2).

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

6. ASSESSMENT: PRESTRESSED CONCRETE

6.1 General

6.1.1 Introduction. This clause gives methods of assessment which will in general assure that, for prestressed concrete construction, the requirements set out in 4.1 are met. In certain cases the assumptions made in this clause may be inappropriate and the assessor may adopt, with the approval of the Overseeing Organisation, a more suitable method having regard to the nature of the structure in question.

Assessment criteria for post-tensioned structures where inadequately grouted ducts and/or tendon corrosion is encountered must be agreed with the Overseeing Organisation.

This clause covers prestressed concrete construction using external and/or unbonded tendons. It does not cover prestressed concrete construction using lightweight aggregate concrete.

6.1.1A Introduction

Bonded prestressing is prestressing where, in the finished structure, continuous bond is provided between the prestressing elements and the concrete section. This is always the case in pre-tensioned structures, except for zones where debonding sleeves are present. In post-tensioned structures, bond between the prestressing elements and the concrete is generally provided by the provision of grout in ducts after stressing. Where no continuous bond is provided the prestressing is termed unbonded. The term external prestressing is applied to that class of unbonded prestressed structures where some or all of the prestressing is unbonded and outside the concrete section, and where the load is transferred to the concrete through end anchorages and deviators. Unbonded internal prestressing is where unbonded prestressing elements are used in ducts which lie within the concrete section.

When tendon corrosion is encountered in an assessment (which is often associated with the ingress of de-icing salts through inadequately grouted ducts), the normal rules for prestressed concrete should be modified by taking into account the following:

- i) Local failure of wires or strands may occur when the tendon strength is reduced to the prestressing force. Hence, wires which have suffered sectional loss which has resulted in them being unable to sustain their prestress force (typically a 40% section loss) should be considered ineffective. The strength of a section at the ultimate limit state should be based on the remaining cross-sectional area of the effective wires only.
- ii) In bonded post-tensioning, tendons, strands or wires which are ineffective locally can re-anchor and become fully effective elsewhere. The anchorage length will depend on the quality of the grouting in the ducts. Where the grouting is good and where nominal links to BS 5400-4 are provided, the re-anchorage length may be taken as the transmission length given in 6.7.4 multiplied by the square root of the number of strands in the tendon.
- iii) Where there is evidence of extensive inadequate grouting or where the BS5400-4 minimum link requirement is not met, assessments which depend on re-anchorage of tendons should not be undertaken without special investigation. Where in the opinion of the assessor the grouting is too poor to allow re-anchorage of tendons, the member should be treated as unbonded and assessed accordingly.
- iv) In external and/or unbonded prestressing, failure of a tendon at any position makes it ineffective over its entire length. Hence structures with unbonded prestressing may be vulnerable to a disproportionate collapse, which is particularly true for continuous bridge decks where localised failure in one span could result in progressive collapse of the spans. Structures with external and/or unbonded prestressing should be checked to ensure that failure of either any two tendons or 25% of those at one section, whichever has the more onerous effect, would not lead to collapse at the ultimate limit states under permanent loads.

- v) In assessing the strength of a structure with corroded tendons, there is a need to consider the possibility of further deterioration. Management strategies for keeping damaged, deteriorating and substandard structures in service will need information on sensitivity of the load assessment to further loss of prestress. Information on possible failure modes is essential for the design of monitoring schemes.

Further guidance in assessing structures with tendon corrosion is given in BA 51 (DMRB 3.4.13).

If the extent of tendon corrosion cannot be directly measured by observing damage to steel, overall levels of prestress in a member can be determined from concrete stress measurements. Before adopting this approach, the agreement of the Overseeing Organisation should be obtained. Specialist advice should also be sought. As there is a possibility of significant errors in determining the level of prestress, spot checks on levels of remaining prestress in individual tendons should also be made. In calculations, it will always be necessary to assume a value of effective prestress to a greater accuracy than is actually known. Calculations should therefore be performed using an upper and lower bound to the estimated effective prestress. In practice, the lower bound will normally be critical for assessments. Having estimated the effective level of prestress in a structure, the flexural, shear and torsional strength can be assessed.

Additional requirements and modifications to the BS 5400-4 provisions are introduced throughout clause 6 specifically for the assessment of prestressed concrete structures with external and/or unbonded tendons. Specialist literature on this topic may also need to be consulted ^(55, 56).

Guidance on prestressed lightweight concrete can be found in reference ⁽⁵⁷⁾.

6.1.2 Limit state assessment of prestressed concrete

6.1.2.1 Basis of assessment. Clause 6 follows the limit state philosophy set out in 3.1B to 3.6B.

6.1.2.2 Durability. *“Not applicable to assessment”*

6.1.2.3 Other limit states and considerations. Clause 6 does not specify special requirements for vibration or other limit states.

6.1.3 Loads. In clause 6 the assessment load effects (see 2.1) for the ultimate and serviceability limit states are referred to as ‘ultimate loads’ and ‘service loads’ respectively.

The values of the ultimate loads and service loads to be used in assessment are derived from 4.2.

In clause 6, when analysing sections, the terms ‘strength’, ‘resistance’ and ‘capacity’ are used to describe the assessment resistance of the section (see BD 21, DMRB 3.4.3).

Consideration must be given, at both ultimate and serviceability limit states as appropriate, to the construction sequence and to the secondary effects due to prestress.

6.1.4 Strength of materials

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

The assessment strengths of concrete and prestressing tendons are given by f_{cu}/γ_{mc} and f_{pu}/γ_{ms} , respectively, where γ_{mc} and γ_{ms} are the appropriate material partial safety factors given in 4.3.3.3.

6.1.4.2 Strength of concrete. Assessment may be based on either the specified characteristic cube strength, or the worst credible cube strength determined in accordance with 2.4B to 2.11B. For structures designed to codes prior to the adoption of the term characteristic strength, the concrete strength was specified in terms of the minimum 28 day works cube strength. For the purpose of assessment, the characteristic strength of concrete may be taken as the minimum 28 day works cube strength.

6.1.4.3 Strength of prestressing tendons. Assessment may be based on either the specified characteristic strength, or the worst credible strength assessed from tests on tendon samples extracted from the structure. For structures designed to codes prior to the adoption of the term characteristic strength, the tendon strength was specified in terms of minimum ultimate strength. For the purpose of assessment, the characteristic strength of tendons may be taken as the minimum ultimate strength.

6.2 Structures and structural frames

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

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

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

- (a) Appropriate checks are made to ensure that adequate rotation capacity exists at sections where moments are reduced, making reference to appropriate test data. For structures which do not contain external and/or unbonded tendons, in the absence of a special investigation the plastic rotation capacity may be taken as the lesser of:

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

or

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

but not less than 0

where

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

- (b) Proper account is taken of changes in transverse moments and transverse shears consequent on redistribution of longitudinal moments.
- (c) Shears and reactions used in assessment are taken as those calculated either prior to or after redistribution, whichever are the greater.

6.2.2A Redistribution of moments

When a linear analysis is performed, both primary and secondary effects of prestressing should be applied before any redistribution of moments is calculated.

Item (2) in 6.2.2 was derived from the ultimate elongation and gauge length specified in British Standards for various types of prestressing steels and from data provided by manufacturers of prestressing steels. For tendons installed before the 1970's or for tendons not specified to British Standards, information should be obtained on the tendon's ductility from past records.

6.3 Beams

6.3.1 General

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

6.3.1.2 Slender beams. *“Not applicable to assessment”*

6.3.2 Serviceability Limit State: flexure. The tensile stress calculated on the gross concrete section under the loading given in 4.2 must be checked against the class 2 values from BS 5400-4, i.e. it must not exceed the design flexural tensile strength of $(0.56/\gamma_{mc})(f_{cu})^{1/2}$. The compressive stress must be limited to $0.5(f_{cu}/\gamma_{mc})$. The calculated tensile stress at unreinforced contact joints in segmental structures with precast elements must not exceed 0.0 N/mm^2 , i.e. no tensile stress is allowed, except for cement mortar joints, where the stress must be in compression throughout and not less than 1.5 N/mm^2 . The values of the partial factors must be taken from 4.3.3.2. In the stress analysis it may be assumed that plane sections remain plane and elastic behaviour exists for the concrete up to the compressive and tensile stress limits given above.

In situations where, subject to the agreement of the Overseeing Organisation, serviceability limit state cracking checks are undertaken against the class 3 values of BS 5400-4, elastic behaviour is deemed to exist up to the compressive stress of $0.5(f_{cu}/\gamma_{mc})$ and the hypothetical tensile stress at the maximum size of cracks defined in 4.1.1.1 of BS 5400-4, and the cracking checks must be performed in accordance with 6.3.2.4 a) 3) of BS 5400-4. However, prestressed structures containing external and/or unbonded prestressing must be treated as reinforced concrete sections in which the axial force and moment due to prestress is considered as an applied load, and the maximum crack widths, calculated as for reinforced concrete columns (see 4.2.2), must be less than the crack widths given in Table 1 of BS 5400-4.

When assessment is required for other serviceability conditions, the necessary criteria must be agreed with the Overseeing Organisation.

6.3.2A Serviceability Limit State: flexure

The partial safety factors to be taken are those from Table 4 of BS 5400-4, where distinction is made between pre-tensioned and post-tensioned members in tension. Note that 4.3.3.2 allows a reduction of 10% for γ_{mc} when worst credible strengths are used, with the caveat that γ_{mc} is not to be taken as less than unity.

For prestressed concrete, class 2 stress limit (as defined by 4.1.1.1 of BS 5400-4) aims to avoid cracking and is dependent on the tensile strength of concrete. Failure to comply with serviceability criteria will not always require remedial action, and further consideration by the assessor should be given to the actual consequences of SLS check failures before this is recommended. In particular, it is unlikely to be justified to take remedial action for non-compliance with class 2 tensile stress limits if there are no signs of distress and the class 3 tensile stress limits are complied with.

6.3.3 Ultimate Limit State: flexure

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

- (a) The strain distribution in the concrete in compression is derived from the assumption that plane sections remain plane.
- (b) The stresses in the concrete in compression are derived either from the stress-strain curve given in Figure 1 with the appropriate value of γ_{mc} given in 4.3.3.3 or, in the case of rectangular sections or flanged sections with the neutral axis in the flange, the compressive stress may be taken as equal to $0.6 f_{cu}/\gamma_{mc}$ over the whole compression zone; in both cases the strain at the outermost compression fibre is taken as 0.0035.
- (c) The tensile strength of concrete is ignored.
- (d) The strains in bonded prestressing tendons and in any additional reinforcement, whether in tension or compression, are derived from the assumption that plane sections remain plane. In addition, the tendon will have an initial strain due to prestress after all losses. When unbonded prestressing is used, the initial strain of the tendons due to prestress after all losses must be multiplied by the appropriate value of γ_{fl} given in 4.2.3.
- (e) The stresses in bonded prestressing tendons, whether initially tensioned or untensioned, and in additional reinforcement, are derived either from the appropriate stress-strain curve in Figures 2, 3 and 4 or, when available, manufacturers' stress-strain curves. The values of γ_{ms} are given in 4.3.3.3. An empirical approach for obtaining the stress in the tendons at failure for structures containing only bonded prestressing is given in 6.3.3.3.
- (f) The strain in unbonded tendons must be assumed not to increase above the initial value due to prestress after all losses including γ_{fl} except that either:
 - (i) In slabs and beams, the strain in the mid-span region of cables which are within $0.1d$ of the soffit at mid-span and which do not extend beyond the supports may be taken to increase by 0.0005, with no additional calculation;
 - (ii) The strain in the tendons at failure may be calculated from a non-linear analysis of the structure. If this is done checks must be made to ensure that conventional 'conservative' assumptions, such as ignoring the tensile strength of concrete, do not have the effect of increasing the tendon strain and hence the ultimate strength.
- (g) In structures with unbonded tendons, tendons and reinforcing bars which are anchored within a distance equal to $h/2$ of the section being considered must be ignored. However, within $h/2$ of a simply supported end, all prestress which is anchored beyond the centre line of the support and all reinforcement which is positively assessed as being anchored in accordance with 5.8.7 may be considered effective.

6.3.3.1A Section analysis

The concrete stress strain curve in Figure 1 and the failure strain of 0.0035 are appropriate to unbound concrete. Higher failure stresses and strains are achieved when the concrete is laterally restrained by helical binding or, to a lesser extent, by conventional links. If the ultimate strength of a member is governed by failure of the concrete compression zone and if the member marginally fails an assessment using the unbound stress-strain curve, it would

be advisable to allow for the enhancing effects of links or helical binding. Appropriate guidance can be obtained from references ⁽¹³⁾ and ⁽¹⁴⁾. See also comment 5.3.2.1A.

As the ultimate strength of structures with unbonded prestressing is dependant on the prestress force, allowance has to be made for the actual prestress force present being less than which is assumed in design. This may be, for instance, a result of the jacking force being less than intended or the prestress loss being greater than calculated. A partial safety factor γ_{fl} of 0.87 is therefore applied to the prestress force due to the unbonded tendons.

Since the strain in unbonded tendons at the ultimate limit state is unlikely to be sufficient to cause yield, failure is likely to be through crushing of the concrete. However, the overall behaviour of the structure should remain ductile with extensive cracking and excessive deflections being apparent before yield.

In bonded prestressed construction plane sections, including their prestressing tendons, remain plane. This assumption depends on the tendons remaining bonded to the concrete and is therefore not valid with unbonded prestressing. The increase in steel strain in unbonded prestressing at failure is less than for bonded tendons and usually not sufficient to reach yield. Sub-clause 6.3.3.1(f) introduces a simple but conservative rule to estimate the steel strain, and hence the stress at failure. As an alternative a more rigorous non-linear analysis may be used.

6.3.3.2 Design charts. "Not applicable in assessment"

6.3.3.3 Assessment formula. For structures containing only bonded prestressing, in the absence of an analysis based on the assumptions given in 6.3.3.1, the resistance moment of a rectangular beam, or of a flanged beam in which the neutral axis lies within the flange, may be obtained from Equation 27:

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

where

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

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

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

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

The neutral axis depth, x , may be calculated from:

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

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

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

6.3.3.3A Assessment formula

The tabulated values of f_{pb} and x given in Table 27 of BS 5400-4 have been replaced by two equations which give the same numerical values as Table 27 when the BS 5400-4 design values of γ_{mc} and γ_{ms} are adopted.

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

6.3.4 Shear resistance of beams

6.3.4.1 General. Calculations for shear are only required for the ultimate limit state. At any section the ultimate shear resistance is the sum of the resistances of the concrete alone V_c (see 6.3.4.2 and 6.3.4.3) and of the shear reinforcement V_s (see 6.3.4.4).

In structures containing only bonded prestressing, for vertical links to be effective the tensile capacity of the longitudinal steel at a section must be greater than

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

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

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

where $A_{s(t)}$, $f_{pu(t)}$, $A_{s(u)}$ and $f_{yL(u)}$ are as defined in 6.3.4.3 and γ_{ms} is the partial safety factor for the tendons given in 4.3.3.3. However within an individual sagging or hogging region, such longitudinal force must not be taken as more than M_{max}/z where M_{max} is the maximum ultimate moment within that region.

In structures containing unbonded prestressing, compliance with the requirements of 6.3.3.1(g) will ensure that vertical links at the section under consideration are effective.

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

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

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

- 5.3.3 except that in determining the area A_s , the area of tendons must be ignored unless the tendons are rigid bars; and
- 6.3.4.2 to 6.3.4.4, using the appropriate value of prestress at the section considered, assuming a linear variation of prestress over the transmission length.

In a haunched beam, 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 algebraically from the applied shear force.

6.3.4.1A General

The design rules for shear in beams given in BS 5400-4 are empirical and based on test results on bonded tendons. Research has shown ⁽⁵⁸⁾ that they can also be safely be used to calculate the shear strength of prestressed concrete beams with unbonded or external tendons.

The design for shear is based on a 45 degrees truss analogy and thus implies a greater force in the tension chord than would be expected by simple bending theory. The limit on the shear resistance for bonded prestressing is related to the area of longitudinal reinforcement in excess of that required to resist bending. However, in the case of unbonded prestressing it is necessary to ensure that the tendon force rather than tendon strength is available, and this is guaranteed by checking that the tendons extend sufficiently beyond the section at which they are required.

The assessment sub-clause states that V_c may be taken as V_{co} when the applied moment does not exceed M_{cr} . This is because the section will not be flexurally cracked and, hence, the V_{cr} calculation is not appropriate.

When using the provisions in 5.3.3 within the transmission zone, the area of bar tendons may be included in A_s , because their rigidity enables them to contribute to the shear resistance component due to dowel action.

The sub-clauses in BS 5400-4 are written for prismatic beams and are generally conservative for haunched ones. This sub-clause allows advantage to be taken of the vertical component of flange forces in these. If three dimensional finite element models are used for box beams, the web shears from the computer model will already have the flange forces subtracted.

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

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

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

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

The value of V_{co} at the height of the cross section where f_t is maximum is given by:

$$V_{co} = \frac{Ib}{S} \sqrt{f_t^2 + \sigma_{cpb} f_t} \quad \text{Equation 28}$$

where

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

- σ_{cpb} is the total direct stress at the location of the section being checked, due to bending and axial load effects after all losses have occurred, taken as positive in compression and multiplied by the appropriate value of γ_{fl} (see 4.2.3);
- b is the breadth of the member at the location of the section being checked, allowing for the presence of the ducts (where the position of a duct coincides with the position of maximum principal tensile stress, e.g. at or near the junction of flange and web near a support, the value of b must be reduced by the full diameter of the duct if ungrouted and by half of the diameter if grouted);
- I is the second moment of area;
- S is the first moment of area of the part of the section excluding any area below the location being checked, calculated about the centroidal axis of the whole section.

In many cases the maximum principal stress occurs at the level of the centroidal axis. However, when this is not the case (i.e. cross sections where the width varies over the height) the minimum value of the shear resistance V_{co} must be found by calculating it at various axis positions in the cross section.

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

6.3.4.2A Sections uncracked in flexure

The equation for the cracking moment has been included in 6.3.4.2. The terms $0.49\sqrt{f_{cu}/\gamma_{mc}}$ and $0.32\sqrt{f_{cu}/\gamma_{mc}}$ replace the BS 5400-4 terms $0.37\sqrt{f_{cu}}$ and $0.24\sqrt{f_{cu}}$, respectively. The latter values include a partial safety factor of 1.5 applied to f_{cu} , which has been replaced in the assessment code by the general value, γ_{mc} . The BS 5400-4 values also include an additional safety factor of 1.25 to allow for strength reductions caused by shrinkage cracking, repeated loading and variations in concrete quality⁽⁵⁹⁾. This factor has been reduced to 1.15 in the assessment code because variations in concrete quality are allowed for in γ_{mc} .

6.3.4.3 Sections cracked in flexure. The ultimate shear resistance of a section cracked in flexure, V_{cr} , must be calculated using Equation 29 when the factored effective prestress, f_{pe} , exceeds $0.6 f_{pu}$. When f_{pe} is less than $0.6 f_{pu}$, the shear strength must be calculated using Equation 29 or Equation 30, whichever is greater.

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

$$\text{but } \geq 0.12bd\sqrt{f_{cu}/\gamma_{mc}}$$

Equation 29

$$V_{cr} = (1 - 0.55 f_{pe}/f_{pu})v_c bd_s + M_o/(M/V - d_s/2)$$

$$\text{but } \geq 0.12bd_s\sqrt{f_{cu}/\gamma_{mc}}$$

Equation 30

where

- d is the distance from the extreme compression fibre to the centroid of the tendons at the section considered;
- γ_{mc} is the partial safety factor for concrete given in 4.3.3.3;
- M_{cr} is the cracking moment defined in 6.3.4.2;
- V, M are the shear force and bending moment (both taken as positive) at the section considered due to ultimate loads;
- M_o is the moment necessary to produce zero stress in the concrete at the depth d , which is equal to $M_o = f_{pt} I/y$ but not greater than M_{cr} in which f_{pt} is the stress due to prestress only at the depth d , distance y from the centroid of the concrete section which has a second moment of area I ; the value of f_{pt} must be derived from the prestressing forces after all losses have occurred, multiplied by the appropriate value of γ_{fl} (see 4.2.3);

- f_{pe} is the factored effective prestress which is equal to the effective prestress after all losses have occurred, multiplied by the appropriate value of γ_{fl} (see 4.2.3);
- v_c is obtained from 5.3.3.2; it may be adjusted to allow for short shear span enhancement in accordance with 5.3.3.3.
- A_s (required to obtain v_c) must be taken as the actual area of steel in the tension zone, irrespective of its characteristic strength;
- d_s is the distance from the compression face to the centroid of the steel area, A_s .

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

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

where

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

For sections cracked in flexure and with inclined tendons, the component of prestressing force normal to the longitudinal axis of the member must be ignored. However, in a haunched section the component of prestressing force (multiplied by the appropriate value of γ_{fl}) normal to the (inclined) longitudinal centroidal axis of the member may be considered. This component should not be taken as greater than it would be if the tendons were parallel to the flange which is the tension flange ignoring the effect of prestress.

6.3.4.3A Sections cracked in flexure

BS 5400-4 gives different design equations for classes 1 and 2 (Equation 29) and class 3 (Equation 30). However, it is actually the f_{pe}/f_{pu} ratio which determines which equation should be used ⁽⁶⁰⁾.

The BS 5400-4 Equation 29 and the lower limiting values for V_{cr} contain a partial safety factor of 1.5 applied to f_{cu} . The equivalent terms in the assessment code have been modified and f_{cu} replaced by f_{cu}/γ_{mc} . This has led to some anomalies in assessment since simplifications made in the equations result in there being a step change between the values given by the two equations and the value of f_{pe} is not accurately known. Since there is no reason why Equation 29 should be unsafe for lower f_{pe}/f_{pu} ratios, the use of either equation is allowed.

Equations 29 and 30 have been further modified by including the $d/2$ term. This term was conservatively omitted from the BS 5400-4 equations ⁽⁶⁰⁾. M_0 (used in Equation 30) is the moment for zero stress at tendon level whereas M_{cr} (used in Equation 29) is the moment for cracking at the tension face allowing for some tensile strength of concrete. In the derivation ⁽⁶⁰⁾ it was assumed d was not much less than h . For sections with the tendon remote from the tension face, M_0 can be very large leading to unsafe results. A maximum of M_{cr} is imposed to avoid this.

Equation 30 was originally based on an interpolation between a modified Equation 29 and the expressions used for reinforced concrete. The short shear span enhancement is now allowed for the latter.

Although it may seem illogical to ignore the vertical component of inclined tendons in cracked sections, the rules are empirical. They were derived for sections with horizontal tendons and tests suggest they could be unsafe if the vertical component of prestress is included when they are applied to sections with inclined tendons. However, it is safe to include the component of prestress perpendicular to the longitudinal centroidal axis when this results from haunching rather than from draped cables.

6.3.4.4 Shear reinforcement. Types of shear reinforcement and the criterion for the amount of shear reinforcement required to be considered effective are defined in 6.3.4.1 and 5.3.3.2.

Vertical prestress may be treated as prestrained reinforcement, with a design force not greater than that corresponding to a total strain of:

$$0.0041 + (\text{estimated prestrain after all losses})/1.15$$

Links must be considered as effective only if their spacing both along a beam and laterally does not exceed d_t , nor four times the web thickness for flanged beams, where d_t is defined below.

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

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

where

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

f_{yv} Is the yield strength of the shear reinforcement. This must be taken as not greater than 500N/mm² unless it is prestressed in which case the stress in the steel may be taken as the appropriate value from Figure 3 at a concrete strain of 0.0045, that is a steel strain of 0.0045 greater than the prestrain after all losses including allowance for γ_{fp} of 0.87.

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

In general, sections within a distance d from the support need not be assessed for shear provided the shear reinforcement calculated for the section at a distance d is continued up to the support.

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

6.3.4.4A Shear reinforcement

Most of the comments on 5.3.3.2 are also applicable to 6.3.4.4.

BS 5400-4 does not give rules for vertical prestress. It has largely gone out of favour but bridges with vertical prestress exist. Vertical prestress can be considered either as controlling the maximum principal tensile stress or as acting as prestrained reinforcement. The former approach can only be used in the uncracked in flexure check whilst the latter generally gives a greater strength. Hence, only the latter approach is given here. The rule limits the strain in addition to the prestrain to that corresponding to the strain required according to the code to reach yield in the highest grade unstressed reinforcement it allows.

The BS 5400-4 requires the link spacing to be reduced when the shear force is large. This requirement has been omitted from the assessment code because there is no logical reason to justify its inclusion. It is understood that the requirement to limit the link spacing to four times the web thickness is related to web buckling. If the web cannot buckle (because, for example, it is surrounded by in-situ infill concrete) then this spacing requirement may be relaxed.

The expression for V_s is not valid for a link spacing in excess of d_t . In such cases an analysis would need to be performed which considers various possible shear failure planes. The requirement to limit spacing to four times the

web thickness is considered to be related to web stability of flanged beams. Where greater link spacings occur, it is advisable to ignore the V_s term, or carry out non-linear analysis of the web.

The requirement to relate shear capacity to the area of longitudinal reinforcement or prestressing steel has been moved to 6.3.4.1.

6.3.4.5 Maximum shear force. The shear force, V , due to ultimate loads, must not exceed the stress $0.36 (0.7 - f_{cu}/250)f_{cu}/\gamma_{mc}$, multiplied by bd_s , where b is as defined in 6.3.4.2, d_s is as defined in 6.3.4.3 and γ_{mc} is the partial safety factor for concrete given in 4.3.3.3.

Where the section is uncracked in flexure according to 6.3.4.2, d_s may be taken as equal to d_t as defined in 6.3.4.4 and, where the component of the prestressing force (multiplied by the appropriate value of γ_{fl}) normal to the longitudinal axis of the member increases the shear resistance of the section due to the presence of inclined tendons, it may be subtracted from the applied shear force V .

In haunched beams the component of flange force perpendicular to the longitudinal centroidal axis may be subtracted from the flange force.

6.3.4.5A Maximum shear force

The upper limit to shear stress in BS 5400-4 is known to be conservative and the increased limit here is based on plasticity theory⁽⁵⁾, which is the basis of the varying angle truss approach of BS EN 1992-1-1 given as an alternative method given in 5.3.3.5/6.3.4.7. However, the maximum shear limit of $0.36 (0.7 - f_{cu} / 250)f_{cu} / \gamma_{mc}$ is more conservative than that in 6.3.4.7, so sections which do not satisfy this maximum to 6.3.4.5 may still have adequate strength to 6.3.4.7 if they have sufficient transverse and longitudinal reinforcement.

The definition of d used has been changed from BS 5400-4 because this is over-conservative for sections with the tendons remote from the tension face.

The rule is known to be conservative for some cases and higher forces may be allowed by use of 6.3.4.7.

6.3.4.6 Segmental construction. The shear checks for post-tensioned segmental structures are generally performed in the same way as for non-segmental structures except that, at joints with cast in situ concrete, dry-pack mortar or grout joint filler the shear force due to ultimate loads must not be greater than:

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

where

- γ_{fl} is the partial safety factor for the prestressing force, to be taken as 0.87;
- P_h is the horizontal component of the prestressing force after all losses;
- α_2 is the angle of friction at the joint. The value for $\tan \alpha_2$ can be taken as 0.7 for a smooth interface and 1.4 for a roughened or castellated interface. If there is any doubt regarding the type of interface, $\tan \alpha_2$ must be taken as 0.7.

In the case of segmental construction with precast elements and unbonded prestressing only, the shear resistance V_s of the vertical links within a distance h_{red} from both edges of the joint (but not greater than the segment length) must be calculated with the formula for V_s in 6.3.4.4 by either using d_t or h_{red} , whichever the lesser, where h_{red} may be taken as the depth of concrete in compression under the ultimate loads, but not less than $0.5h$, with h being the overall section depth.

The method of assessment of match cast joints with shear keys must be agreed with the Overseeing Organisation.

6.3.4.6A Segmental construction

For segmental construction with no bonded prestressing, in accordance with 6.2.3 of BS EN 1992-2 it is assumed that, as the applied load increases, the joint opens and the force in the tension chord remains unchanged, so the depth of concrete section available for the flow of the web compression field is reduced to a value of h_{red} , which can be taken as the depth of concrete in compression under the ultimate loads, in accordance with PD 6687-2.

6.3.4.7 Alternative method. As an alternative to the method given in 6.3.4.1 to 6.3.4.5, beams may be assessed using the varying angle truss approach described in 5.3.3.5. In this case the vertical component of prestress may be deducted algebraically from the applied shear force and α_{cw} may be taken equal to:

$$\begin{aligned} 1 + 2 \sigma_{cp}/f_{cu} & \quad \text{for } 0 < \sigma_{cp} < 0.125 f_{cu} \\ 1.25 & \quad \text{for } 0.125 f_{cu} < \sigma_{cp} < 0.25 f_{cu} \\ 2.5 (1 - 2 \sigma_{cp}/f_{cu}) & \quad \text{for } \sigma_{cp} > 0.25 f_{cu} \end{aligned}$$

where σ_{cp} is the mean compressive stress in the concrete due to the prestressing and axial loading, obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not to be calculated at a distance less than $0.5d \cot \theta$ from the edge of the support. In the case of straight tendons, a high level of prestress ($\sigma_{cp}/f_{cu} > 0.25$) and thin webs, if the tension and the compression chords are able to carry the whole prestressing force and blocks are provided at the extremity of beams to disperse the prestressing force, it may be assumed that the prestressing force is distributed between the chords. In these circumstances, the compression field due to shear only should be considered in the web ($\alpha_{cw}=1$).

In the case of segmental construction with precast elements and unbonded prestressing only, the shear resistance from the varying angle truss approach in 5.3.3.5 within a distance $h_{red} \cot \theta$ from both edges of the joint (but not greater than the segment length) must be evaluated by assuming a value of the angle θ derived from the residual depth h_{red} , in accordance with the provisions of 6.2.3 of BS EN 1992-2, where h_{red} is taken as the depth of concrete in compression under the ultimate loads, but not less than $0.5h$, with h being the overall section depth.

6.3.4.7A Alternative method

As the BD 44 approach to link design in prestressed sections is relatively economic, the varying angle truss approach is less likely to be advantageous in terms of assessing the adequacy of links. However, it can be advantageous when the link area is relatively large. It can also give significant increases in web crushing strength compared with the normal BD 44 limit in 6.3.4.5.

6.3.4.8 Other approaches. With the approval of the Overseeing Organisation, methods employing plasticity theory may be used for the assessment of the shear capacity of concrete beams.

6.3.4.8A Other approaches

The ultimate shear capacity of prestressed concrete structures can be assessed by analytical methods based on the lower bound and upper bound models of plasticity theory, which may prove very useful in situations not explicitly covered by codes. However, these methods should be applied with care and under the supervision of the Overseeing Organisation. See also 5.3.3.7A. An upper bound model for assessing the shear capacity of prestressed concrete bridge beams post-tensioned transversely together to form a deck is presented in references (7) and (8).

6.3.5 Torsional resistance of beams

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

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

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

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

In calculating $(v + v_t)$ for comparison with v_{tu} (see 5.3.4.3), v must be calculated from Equation 8 of 5.3.3.1 regardless of whether 6.3.4.2 or 6.3.4.3 is critical in shear.

6.3.5.2A Stresses and reinforcement

The BS 5400-4 increase in v_{tu} for concrete grades above 40 is already allowed for in the expression for v_{tu} (see 5.3.4.3A).

6.3.5.3 Segmental construction. *“Not applicable to assessment”*

6.3.5.4 Other assessment methods. Alternative methods of assessing members subjected to combined bending, shear and torsion may be used provided that it can be shown that they satisfy the ultimate limit state requirements. Box sections may be assessed by checking webs and flanges in accordance with 6.3.4 for the sum of shear forces calculated for direct shear and torsion.

6.3.5.4A Other assessment methods

This sub-clause permits the consideration of strength at various points rather than the consideration of cross-sectional strength. Possible methods for box girders are presented in references ⁽⁶¹⁾ and ⁽⁶²⁾.

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

6.3.7 Deflection of beam. If required by the Overseeing Organisation, deflections may be calculated in accordance with 4.2.4 and 4.6 taking appropriately into account the level of prestress.

6.3.7A Deflection of beam

The deflection of a beam which is uncracked under the assessment service load may be determined using an elastic analysis based on the concrete section properties and a modulus of elasticity which allows for creep, if appropriate. Beams with low levels of prestress may be cracked under the assessment service load, and the deflections of such beams may be determined using moment curvature relationships ⁽⁶³⁾.

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

The assessment of shear must be in accordance with 6.3.4. In the treatment of shear stresses under concentrated loads, the ultimate shear resistance of a section uncracked in flexure, V_{co} , may be taken as corresponding to the occurrence of a maximum principal tensile stress of $f_t = 0.32 \sqrt{f_{cu} / \gamma_{mc}}$ at the centroidal axis around the critical section which is assumed as a perimeter $h/2$ from the loaded area. The shear resistance of any shear reinforcement, V_s , must be assessed in accordance with 6.3.4.4.

6.4A Slabs

See 6.3.4.2A for the explanation of the expression for f_t .

6.5 Columns

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

6.6 Tension members

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

6.7 Prestressing requirements

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

6.7.2 Loss of prestress, other than friction losses

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

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

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

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

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

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

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

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

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

6.7.2.4 Loss of prestress due to shrinkage of the concrete. The loss of prestress in the tendons due to shrinkage of the concrete may be calculated from the modulus of elasticity for the tendons given in 4.3.2.2, assuming the values for shrinkage per unit length given in Table 29. For other ages of concrete at transfer, for other conditions of exposure, or for massive structures, some adjustment to these figures will be necessary, in which case reference can be made to Appendix C of BS 5400-4 or specialist literature.

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

Table 29 Shrinkage of concrete

6.7.2.4A Loss of prestress due to shrinkage of the concrete

The age factors of BS5400-4 are not considered relevant to assessment.

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

For pre-tensioning at between 3 days and 5 days after concreting and for humid or dry conditions of exposure where the cube strength at transfer was greater than 40 N/mm², the creep of the concrete per unit length must be taken as 48 x 10⁻⁶ per N/mm². For lower values of cube strength at transfer the creep per unit length must be taken as 48 x 10⁻⁶ x 40/f_{ci} per N/mm², where f_{ci} is the concrete strength at transfer, see 6.7.4.

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

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

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

6.7.2.5A Loss of prestress due to creep of the concrete

The age factors of BS 5400-4 are not considered relevant to assessment. In unbonded tendons, although the lack of bond will not alter the average loss due to creep, as the creep losses are constant over the length of a beam, losses at critical sections are likely to be smaller than with bonded construction. This may be significant in shorter spans where the permanent compressive stresses for live load are high.

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

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

6.7.3 Loss of prestress due to friction

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

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

6.7.3.2 Friction in the jack and anchorage. Jacks are generally calibrated to give a specified force at the duct side of the anchorage. Hence, the friction loss in the jack and anchorage should not be of concern in assessment.

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

$$P_x = P_o e^{-Kx}$$

Equation 31

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

where

P_o is the prestressing force in the tendon at the jacking end;
 e is the base of Napierian logarithms (2.718);
 K is the constant depending on the type of duct, or sheath employed, the nature of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete.

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

The effect may be ignored in sections of unbonded tendon which are free to move laterally at the time they are stressed.

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

With external unbonded prestressing wobble losses are eliminated.

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

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

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

Equation 32

where

P_o is the prestressing force in the tendon at the tangent point near the jacking end;
 r_{ps} is the radius of curvature

In the above equation:

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

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

Values of μ for internal tendons may be taken as:

0.55 for steel moving on concrete;
 0.30 for steel moving on steel;
 0.25 for steel moving on lead;
 0.05 for greased coated monostrands moving on plastic sheaths

For external tendons, in the absence of more exact data, the values of μ may be taken from the following table:

COEFFICIENT OF FRICTION μ	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

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

6.7.3.4A Friction in the duct due to curvature of the tendon

The use of unducted systems or ducts with greased or waxed strands means that prestressing systems used in external and unbonded prestressing have lower coefficients of friction than bonded systems.

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

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

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

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

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

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

where

- f_{ci} is the concrete strength at transfer (in N/mm²) which must be assessed from contract record drawings, available site data or original design calculations. In the absence of such information, the likely nominal value must be assessed from the standards current at the time of the design;
- l_t is the transmission length (in mm);
- ϕ is the nominal diameter of the tendon (in mm);
- k_t is a coefficient dependent on the type of tendon, to be taken as:
 - 600 for plain, indented and crimped wire with a total wave height less than 0.15 ϕ ;
 - 400 for crimped wire with a total wave height greater than or equal to 0.15 ϕ ;
 - 240 for 7-wire standard and super strand;
 - 360 for 7-wire drawn or compacted strand.

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

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

6.7.4A Transmission length in pre-tensioned members

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

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

The transmission lengths of the tendons towards the top of a unit may be greater than those at the bottom. The sudden release of tendons may also cause a considerable increase in the transmission lengths.

It is emphasised that data are not available on transmission lengths in weak concretes less than 28 N/mm². Hence, caution is advised if the concrete has deteriorated below 28 N/mm², even if the strength at transfer was in excess of this value.

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

For unbonded construction, end blocks, anchors and deviators must be assessed at the ultimate limit state for a load equal to the characteristic strength of the tendon. If serviceability checks are required, as for flexural cracking in deviator beams, the design load in the tendons must be taken as the tendon load before long term losses.

For bonded construction, the critical condition for end blocks normally arises during construction. Hence, if there are no signs of distress, it will not normally be necessary to assess the end blocks.

When it is necessary to consider the strength of end blocks, the following aspects must be considered:

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

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

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

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

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

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

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

where

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

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

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

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

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

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

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

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

6.7.5A End blocks and deviators

The strength of structures with unbonded prestressing is dependent on the anchors and, where they are used, the deviators. This is particularly significant as the failure mode of anchors and deviators may be brittle. Although members with unbonded prestressing are conservatively designed on the basis of a lower bound estimate of tendon force, anchors and deviators should be conservatively assessed for the unfactored tendon strength.

Tendon jacking loads which have to be considered in end block design are not relevant to assessment.

The equation for F_{bst} gives the same values as Table 30 of BS 5400-4.

The restriction on the stress in anti-bursting reinforcement to that corresponding to a strain of 0.001 in BS 5400-4 is a serviceability criterion which has been omitted.

Detailed information on end block strength is given in ⁽⁶⁴⁾, whilst ⁽⁶⁵⁾ deals with end blocks in which the concrete is assumed to resist tension.

6.8 Considerations of details

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

6.8.2 Cover to prestressing tendons

6.8.2.1 General. The requirements given in 6.8.2.2 and 6.8.2.3 concerning cover to reinforcement, other than those for curved ducts, are those which are currently considered to be necessary for the safe transmission of bond forces.

6.8.2.1A General

Adequate cover to reinforcement should ideally be present in all concrete structures designed to standards containing appropriate provisions for durability, and the cover generally varies with the concrete grade and with the particular condition of exposure. When the cover is less than the values resulting from the requirements in 6.8.2.2 and 6.8.2.3, bond strength could be reduced and reinforcement corrosion is most likely to occur. Guidance on the assessment of concrete structures affected by steel corrosion and with low cover is given in BA 51 (DMRB 3.4.13). Reference may also be made to the provisions in 4.4.1.2 of BS EN 1992-1-1 for the minimum cover required for bond.

6.8.2.2 Pre-tensioned tendons. The requirements of 5.8.2 concerning cover to reinforcement may be taken to be applicable, except that the nominal cover should not be less than one and a half the size of the tendon or maximum aggregate size, plus 5mm. The ends of individual pre-tensioned tendons do not normally require concrete cover.

6.8.2.3 Tendons in ducts. The cover to any duct should be not less than 50mm. Requirements for the cover to curved tendons in ducts are given in Appendix D of BS 5400-4.

6.8.3 Spacing of prestressing tendons

6.8.3.1 General. *“Not applicable to assessment”*

6.8.3.2 Pre-tensioned tendons. *“Not applicable to assessment”*

6.8.3.3 Tendons in ducts. Requirements for the spacing of curved tendons in ducts are given in Appendix D of BS 5400-4.

6.8.4 Longitudinal reinforcement in prestressed concrete beams

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

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

6.8.6 Shock loading. *“Not applicable to assessment”*

6.8.7 Deflected tendons. *“Not applicable to assessment”*

6.8.8 External tendons. To avoid second order effects due to beam deflections between points where tendons are fixed, it must be checked that external tendons are restrained transversely relative to the concrete section at centres not exceeding 12 times the minimum depth of the beam between the fixing points. If the spacing between points where the tendons are held in position laterally exceeds 12 metres, check must 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.

6.8.8A External tendons

If 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. In addition, inadequately restrained tendons may vibrate excessively and be susceptible to fatigue failure. Compliance with the requirements of 6.8.8 ensures that tendons are adequately restrained and no second order effect need to be accounted for.

7. ASSESSMENT: PRECAST, COMPOSITE AND PLAIN CONCRETE CONSTRUCTION

7.1 General

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

Criteria for assessment of bridges containing shear key decks or Freyssinet hinges must be agreed with the Overseeing Organisation.

7.1.1A Introduction

Guidance on shear key decks and Freyssinet concrete hinges in bridges is given in BE 23 (DMRB 1.3) and BE 5 (DMRB 1.3), respectively.

7.1.2 Limit state assessment

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

7.1.2.2 Handling stresses. *“Not applicable to assessment”*

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

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

7.1.2.3A Connections and joints

No reference is made to movement joints in the assessment code. However, it should be remembered that the lack of adequate joints can lead to concentrated cracking.

7.2 Precast concrete construction

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

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

7.2.3 Supports for precast members

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

The adequacy of the main tension reinforcement in a corbel must be assessed on the assumption that the corbel behaves as a simple strut and tie system; with due allowance made for horizontal forces. The tensile force which the main reinforcement can develop may be limited by any one of the following: the yield of the reinforcement; the anchorage of the reinforcement in the supporting member and the anchorage at the front face of the corbel. It should be noted that wide cracks are likely to occur if the main steel percentage ρ is less than 0.4%.

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

7.2.3.1A Concrete corbels

Specialist literature ^(21, 66) should be consulted when the depth at the outer edge of the bearing is less than one-half of the depth at the face of the supporting member.

The limiting value of a_v/d for a corbel has been extended from the value of 0.6 in BS 5400-4 to 1.0 in the assessment code. The latter value is within the range of applicability of the assessment method according to reference ⁽²¹⁾.

When assessing corbels using strut and tie systems, further guidance on sizing of struts, ties and nodes can be found in BS EN 1992-1-1.

The BS 5400-4 minimum main steel percentage has been omitted since it is a serviceability requirement to prevent the rapid opening of cracks after initial cracking ⁽⁶⁶⁾.

The BS 5400-4 requirement for minimum horizontal links is to ensure a ductile failure and to control the widths of diagonal cracks. These criteria are not considered relevant to assessment and, thus, the minimum requirement has been omitted in the assessment code. However, the assessor should be aware that the absence of horizontal links could result in wide cracks and/or a brittle failure.

The BS 5400-4 requirement for a serviceability check is not considered relevant to assessment, unless specifically requested by the Overseeing Organisation.

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

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

Where suitable measures have been taken to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas and additional binding reinforcement in the ends of the members, higher bearing stresses may be acceptable, and bearing stresses due to ultimate loads must then be limited to:

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

where

A_{con} is the contact area;
 A_{sup} is the supporting area.

For rectangular bearings (see Figure 6a):

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

where

b_x, b_y are the dimensions of the bearing in the x, y directions respectively;
 x, y are the dimensions from the boundary of the contact area to the boundary of the support area.

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

Higher bearing stresses due to ultimate loads must be used only where justified by tests, e.g. concrete hinges.

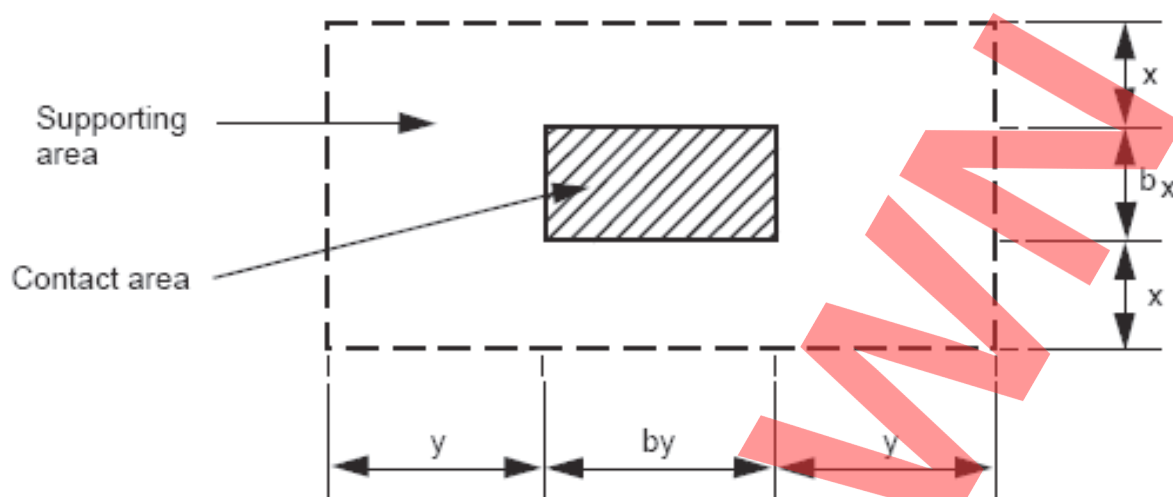


Figure 6a Bearing areas

7.2.3.3A Bearing stresses

The allowable bearing stresses are higher than the BS 5400-4 values. However the bearing stress expression is that given in BS 8110-1 ⁽⁶⁷⁾.

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

- (a) sliding bearings are present; or
- (b) suitable lateral reinforcement is present in the top of the supporting member; and
- (c) continuity reinforcement is present to tie together the ends of the supported members.

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

7.2.4 Joints between precast members

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

7.2.4.2 Half joint. For the type of joint shown in Figure 7(a), the maximum vertical ultimate load, F_v , must not exceed $v_u b d_o$ where

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

The capacity of an half joint may be determined by considering the two following strut and tie systems and summing the capacities of the two systems.

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

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

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

7.2.4.2A Half joint

The assessment sub-clause dealing with half joints is more general than the BS 5400-4 sub-clause in that it permits two strut and tie systems to be assessed and the load capacities of the two systems added. This approach has been shown to predict adequately failure loads ⁽⁶⁸⁾.

The BS 5400-4 requirement to limit the shear stress to $4v_c$, with v_c calculated for the full beam section, was intended to prevent over-reinforcement of the joint and, hence, to ensure a ductile failure. It is more logical, in terms of shear capacity, to adopt the maximum allowable shear stress given in 5.3.3.1.

BA 39 (DMRB 3.4.6) gives additional guidance on the assessment of half joints at both the ultimate and serviceability limit states. Further guidance on sizing of struts, ties and nodes can be found in BS EN 1992-1-1.

Other feasible strut and tie systems may also be appropriate, particularly where there are reinforcement detailing shortfalls, and assessors may consider other reference sources and specialist literature. Reference should be made to the Overseeing Organisation for the latest advice on assessing structures containing half joints.

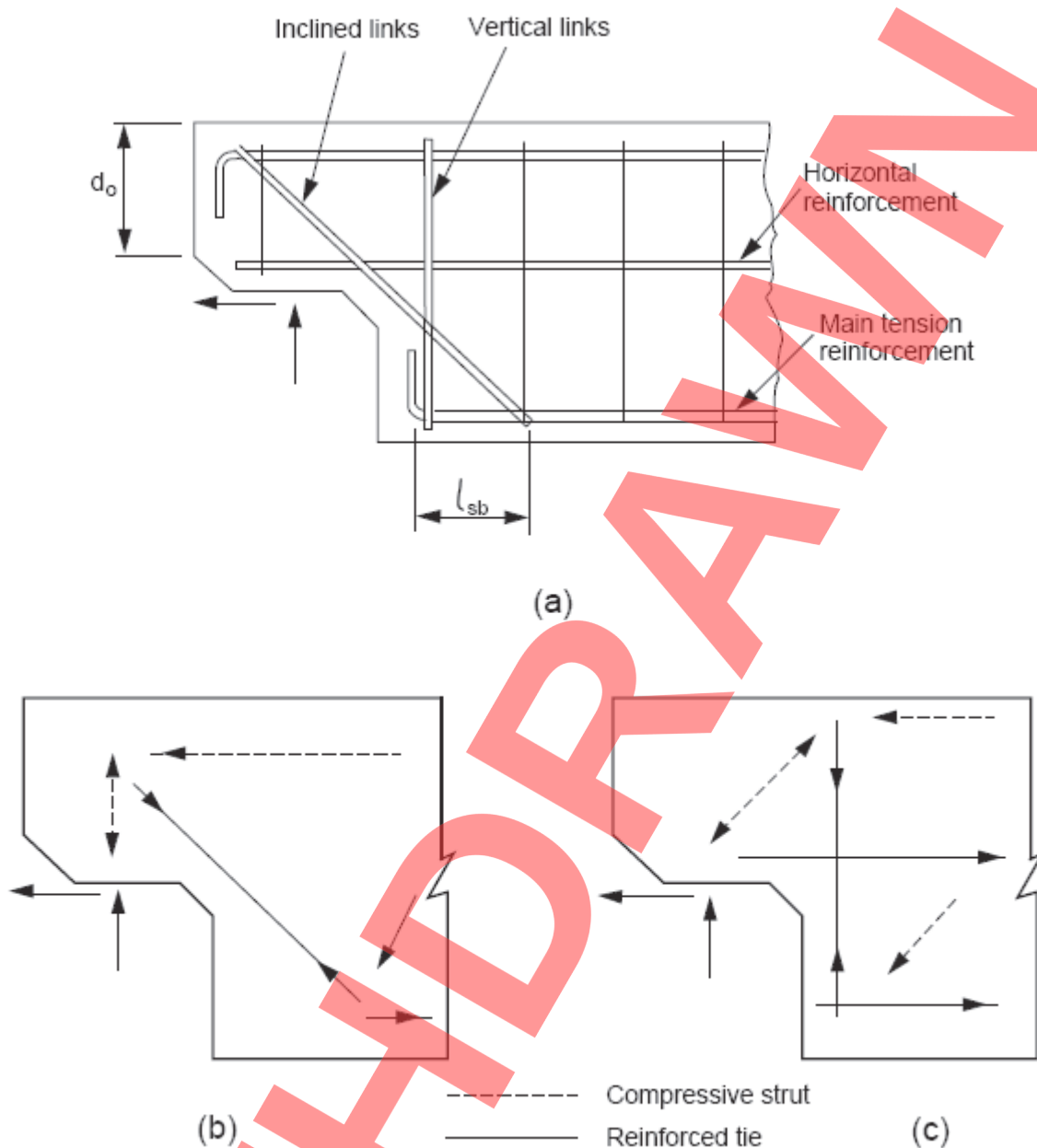


Figure 7 Half joint

7.3 Structural connections between units

7.3.1 General

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

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

7.3.1.3 Considerations affecting design details. *“Not applicable to assessment”*

7.3.1.4 Factors affecting design and construction. *“Not applicable to assessment”*

7.3.2 Continuity of reinforcement

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

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

The strength of the joints in (c) and (d) and any other method not listed must be assessed on the basis of test evidence, including behaviour under fatigue conditions where relevant. For methods (c) and (d), in tests on an assembly consisting of the size, grade and profile of the reinforcing bar used and the type of connection used, the permanent elongation after loading to 60% of the characteristic yield strength of the reinforcement must not exceed 0.1mm, and the ultimate tensile strength of the joined bar must exceed its characteristic yield strength by at least the percentage specified in the British Standard the bar was specified to. Where supported by satisfactory test evidence, the strength of the joint may be based on the specified characteristic yield strength of the joined bars divided in each case by the appropriate γ_{ms} factor.

7.3.2.2 Sleeving. The following three principal types of sleeve jointing were acceptable under BS 5400-4 and are considered adequate for assessment purposes, provided that appropriate test data are available:

- (a) grout or resin filled sleeves;
- (b) sleeves that mechanically align the square-sawn ends of two bars to allow the transmission of compressive forces only;
- (c) sleeves that are mechanically swaged to the bars.

7.3.2.2A Sleeving

This sub-clause has been abbreviated from BS 5400-4 sub-clause, because it is envisaged that test data appropriate to the actual condition of use would be obtained.

7.3.2.3 Threading. The following methods for joining threaded bars were acceptable under BS 5400-4 and are considered adequate for assessment purposes:

- (a) Parallel threaded ends of bars are joined by a coupler having left and right-hand threads.
- (b) One set of bars is welded to a steel plate that is drilled to receive the threaded ends of the second set of bars, which are fixed to the plate by means of nuts.
- (c) Threaded anchors cast into a precast unit to receive the threaded ends of reinforcement.
- (d) Taper threaded bars joined by the use of internally taper threaded couplers.

7.3.2.3A Threading

See 7.3.2.2A.

7.3.2.4 Welding of bars. “Not applicable to assessment”

7.3.2.4A Welding of bars

The BS 5400-4 sub-clause is assumed to refer to the locations of welded connections and is, thus, not relevant to assessment.

7.3.3 Other types of connection. The load carrying capacity of any other type of connection at the ultimate limit state must be justified by test evidence.

Under service loads, no tensile stresses are permitted for resin adhesive joints and resin mortar joints; for cement mortar joints, the stresses in the joint must be compressive throughout the joint and not less than 1.5 N/mm².

7.3.3A Other types of connection

For resisting shear and flexure, suitable connections are generally only those types which are made by prestressing across the joint.

7.4 Composite concrete construction

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

In general, the analysis and assessment of composite concrete structures and members must be in accordance with clause 5 or 6, modified where appropriate by 7.4.2 and 7.4.3. Particular attention must be given in the assessment to the effect of the method of construction and whether or not props were used. The relative stiffness of members must be based on the concrete, gross transformed or net transformed section properties as described in 4.4.2.1; if the concrete strengths in the two components of the composite member differ by more than 10 N/mm², allowance for this must be made in assessing stiffness and stresses.

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

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

7.4.2 Ultimate Limit State

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

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

- (a) For I, M, T, U and box beam precast prestressed concrete units with an in situ reinforced concrete top slab cast over the precast units (including pseudo box construction), the shear resistance may be based on either of the following:
 - (1) the precast unit acting alone assessed in accordance with 6.3.4;
 - (2) the composite section assessed in accordance with 6.3.4. In this case, section properties must be based on those of the composite section, with due allowance for the different grades of concrete where appropriate.
- (b) For inverted T beam precast prestressed concrete units with transverse reinforcement placed through holes in the bottom of the webs of the units, completely infilled with concrete placed between and over the units to form a solid deck slab, the shear resistance may be taken as the sum of V_i and V_p where:

V_i is the shear capacity of the infill concrete assessed in accordance with 5.3.3 with the breadth taken as the distance between adjacent precast webs and the depth as the mean depth of infill concrete, or the mean effective depth to the longitudinal reinforcement where this is provided in the infill section. Where there is no longitudinal reinforcement in the infill, d may be taken as 0.9 times the average depth of the infill. V_i should not be taken as greater than half of the concrete component of the shear capacity of the prestressed beam (i.e. not greater than $0.5V_{co}$ nor $0.5V_{cr}$ of the precast prestressed section, whichever is applicable);

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

7.4.2.2A Vertical shear

The BS 5400-4 rules for shear in infill concrete decks are conservative as they do not allow for redistribution of shear between the in situ and precast sections. Tests ⁽⁶⁹⁾ have shown that the shear capacity of an infill concrete deck can be taken as the sum of the infill concrete section, V_i , and the precast concrete section, V_p .

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

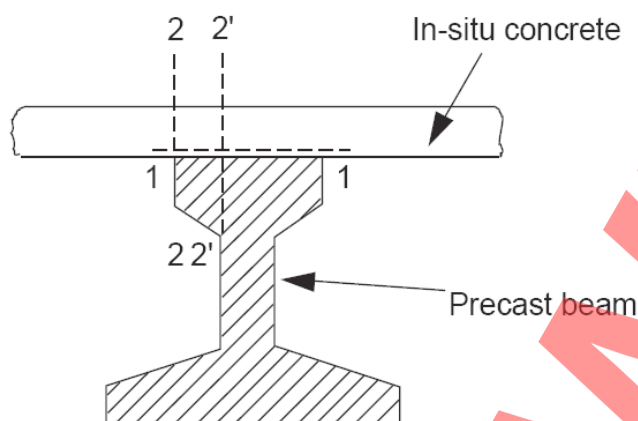


Figure 8 Potential shear planes.

V_1 must not exceed the lesser of the following:

- (a) $(k_1 f_{cu} / \gamma_{mc}) L_s$
- (b) $(v_1 / \gamma_{mv}) L_s + (0.8 A_e f_s / \gamma_{ms})$, where

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

f_{cu} is the characteristic, or worst credible, strength of the weaker of the two concretes each side of the shear plane; but must not be taken as $>45 \text{ N/mm}^2$

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

L_s is the breadth of the shear plane under consideration;

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

- (1) For monolithic construction: $0.05 f_{cu}$ but not less than 1.13 N/mm^2 and not greater than 1.56 N/mm^2 ;
- (2) For surface type 1: $0.04 f_{cu}$ but not less than 0.8 N/mm^2 and not greater than 1.28 N/mm^2 ;
- (3) For surface type 2: $0.019 f_{cu}$ but not less than 0.38 N/mm^2 and not greater than 0.63 N/mm^2 .

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

γ_{mv} is the material partial safety factor for shear given in 4.3.3.3;

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

f_s is the stress at the ultimate limit state in the steel reinforcement of area A_e . The stress may be assumed to be the characteristic, or worst credible, strength, f_y , if the reinforcement A_e is fully anchored (see 5.8.6); otherwise f_s must be taken as a fraction of f_y in proportion to the ratio of the anchorage available to that required by 5.8.6; the value of f_s should be such that $A_e f_s / b$ is not greater than 10 N/mm^2 , where b is the width of the interface under consideration.

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

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

- (a) six times the minimum thickness of the in-situ concrete flange;
- (b) 900 mm.

The types of surface are defined as follows:

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

- (1) When the concrete had set but not hardened the surface was sprayed with a fine spray of water or brushed with a stiff brush, just sufficient to remove the outer mortar skin and expose the larger aggregate without disturbing it.
- (2) The surface skin and laitance were removed by sand blasting or the use of a needle gun, but not by hacking.

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

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

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

7.4.2.3A Longitudinal shear

The BS 5400-4 design values of k_1 implicitly allow for a partial safety factor of 1.6. Hence, the assessment characteristic values are 1.6 times the BS 5400-4 design values, and the partial safety factor, γ_{ms} , is included in expression (a).

Test data ⁽⁷⁰⁾ have shown that the BS 5400-4 design values of v_1 implicitly allow for partial safety factors of 1.25, 1.6 and 2.0 for type 2, type 1 and monolithic surfaces, respectively. Furthermore, the values for monolithic concrete have been shown to be less dependent on concrete strength than implied by BS 5400-4. Hence, the assessment characteristic values have been obtained as follows.

Monolithic

Design values suggested in reference ⁽⁷⁰⁾ were incorporated in a slightly amended form in BS 5400-4. These values incorporate a partial safety factor of 1.25 and have thus been multiplied by 1.25 to give the characteristic values for assessment.

Surface type 1

BS 5400-4 values multiplied by 1.6.

Surface type 2

BS 5400-4 values multiplied by 1.25.

The partial safety factor for shear γ_{mv} is applied to the characteristic v_1 values ⁽⁷⁰⁾.

The steel stress definition has been modified to permit partially anchored reinforcement to contribute to the longitudinal shear capacity.

The factor 0.8 in expression (b) reduces to the BS 5400-4 value of 0.7 when the BS 5400-4 value of γ_{ms} of 1.15 is applied.

The BS 5400-4 minimum steel requirement of 0.15% has not been included because its origin is unclear. However, one should be aware that brittle longitudinal shear failures can occur with small amounts of reinforcement⁽⁷⁰⁾. Furthermore, in reference⁽⁷⁰⁾ it is observed that the stirrups act as ties across the interface. If the form of construction consists of a slab cast on to the top of beams with no reinforcement crossing the interface, then there will be nothing to provide a tie if the tensile resistance of the concrete across the interface is destroyed by the effects of repeated loading. In such a case, it would be prudent not to treat the member as acting compositely. However, if the in-situ slab encases the top flange of the beam then mechanical interlock between the precast and in-situ concretes may provide adequate tie action and the interface shear strength could be based on the concrete interface shear resistance above.

The reason for the maximum steel spacing requirement in BS 5400-4 is also unclear. Hence, the assessor should apply engineering judgement when considering this requirement and not automatically ignore reinforcement spaced at centres greater than the specified maximum.

7.4.3 Serviceability Limit State. When a serviceability limit state assessment for composite concrete structures is required by the Overseeing Organisation, it must be based on 7.4.3 of BS 5400-4, with the following amendments:

- a. Any reference, in 7.4.3.1 to 7.4.3.5 of BS 5400-4, to clauses and sub-clauses of BS 5400-4 is to be intended as a reference to the corresponding clauses and sub-clauses in this Standard.
- b. The maximum compressive stress limit referred to in 7.4.3.2 of BS 5400-4 can be taken as equal to $0.625 (f_{cu}/\gamma_{mc})$.
- c. The allowable flexural tensile stresses in Table 32 of BS 5400-4 may be increased by multiplying by a factor not exceeding 2.5. However in such circumstances, the 50% increase permitted in 7.4.3.3 of BS 5400-4 should not be applied.
- d. The flexural and hypothetical tensile stress limits referred to in the last paragraph of 7.4.3.3 of BS 5400-4 are to be taken, respectively, as the class 2 tensile stress limit and the class 3 hypothetical tensile stress limit in 6.3.2.

7.4.3A Serviceability Limit State

There appears to be no test data to quantify the permissible enhancement in allowable compressive stress in 7.4.3.2 of BS 5400-4. However, for the case of a fully restrained flange, an enhancement of up to 50%⁽⁶⁷⁾ was permitted in BS 8110-1.

The allowable flexural tensile stresses in Table 32 of BS 5400-4 are very conservative and include a partial safety factor of about 2.5 on the lower bound to the experimental evidence⁽⁴⁾.

7.5 Plain concrete walls and abutments

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

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

7.5.1A General

BS 5400-4 restricts its application to braced plain concrete walls with a slenderness ratio not exceeding 5, since it is considered that this ratio reflects current practice. When assessing an unbraced and/or a more slender wall, the assessor may consult 3.9.4 of BS 8110-1 which deals with unbraced and slender walls. In applying the latter sub-clause for assessment purposes, the constant 0.3 in Equations 43 to 46 of BS 8110-1 should be replaced with $(0.675/\gamma_{mcw})$, with γ_{mcw} as given in 4.3.3.3.

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

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

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

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

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

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

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

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

7.5.5 Analysis of section. Loads of a purely local nature (as at bearings or column bases) may be assumed to be immediately dispersed provided the local stress under the load does not exceed that given in 7.5.7. Where the resultant of all the axial loads act eccentrically in the plane of the member, the ultimate axial load per unit length of wall, n_w , must be assessed on the basis of an elastic analysis assuming a linear distribution of load along the length

of the member, assuming a tensile resistance of concrete of $0.12 \sqrt{\frac{f_{cu}}{\gamma_{mc}}}$. Consider first the axial force and bending in

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

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

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

where

- n_w is the maximum axial load per unit length of member due to ultimate loads;
- h is the overall thickness of the section;
- e_x is the resultant eccentricity of load at right-angles to the plane of the member (see 7.5.2) (minimum value 0.05h);
- 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.

7.5.5A Analysis of section

The factor γ_w in BS 5400-4 and CP 110 has been replaced with a constant value of 0.3 in BS 8110-1. The latter value includes ⁽⁶⁷⁾ an allowance for a partial safety factor of 2.25. Hence, in the assessment code, the factor 0.3 is replaced with $(0.3 \times 2.25 / \gamma_{mcw})$ where γ_{mcw} is either 2.25 (for use with the characteristic strength) or $2.25 \times 1.2 / 1.5 = 1.80$ (for use with the worst credible strength).

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

7.5.6A Shear

The background to this sub-clause, which originated in CP 110, is unclear. In particular the value of the inherent partial safety factor is not known. Hence, the shear stresses should be considered as assessment values and no partial safety factor applied to them.

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

7.5.8 Deflection of plain concrete walls or abutments. *“Not applicable to assessment”*

7.5.9 Shrinkage and temperature reinforcement. *“Not applicable to assessment”*

7.5.10 Stress limitations for Serviceability Limit State. *“Not applicable to assessment”*

ANNEX A: REFERENCES

A.1 NORMATIVE REFERENCES

Design Manual for Roads and Bridges (DMRB): HMSO

DMRB Volume 1: Section 3: General design

BD 9 Implementation of BS 5400-10: 1980 – Code of Practice for Fatigue (DMRB 1.3)

BD 37 Loads for Highway Bridges (DMRB 1.3)

DMRB Volume 3: Section 4: Assessment

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

BD 48 The Assessment and Strengthening of Highway Bridge Supports (DMRB 3.4.7)

BD 86 The Assessment of Highway Bridges and Structures for the Effects of Special Types General Order (STGO) and Special Order (SO) Vehicles (DMRB 3.4.19)

British Standards (BS): BRITISH STANDARDS INSTITUTION

BS 5400-1:1988. Steel, concrete and composite bridges. General statement

BS 5400-4:1990. Steel, concrete and composite bridges. Code of practice for design of concrete bridges

BS 5400-10:1980. Steel, concrete and composite bridges. Code of practice for fatigue

A.2 INFORMATIVE REFERENCES

Design Manual for Roads and Bridges (DMRB): HMSO

DMRB Volume 1: Section 3: General design

BA 40 Tack Welding of Reinforcing Bars (DMRB 1.3.4)

BE 23 Shear key decks (DMRB 1.3)

BE 5 Rules for the Design and Use of Freyssinet Concrete Hinges in Highway Structures (DMRB 1.3)

DMRB Volume 3: Section 1: Inspection

BA 93 Structural Assessment of Bridges with Deck Hinges (DMRB 3.1.5)

DMRB Volume 3: Section 4: Assessment

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

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

BA 51 The Assessment of Concrete Structures Affected by Steel Corrosion (DMRB 3.4.13)

BD 81 Use of Compressive Membrane Action in Bridge Decks (DMRB 3.4.20)

British Standards (BS): BRITISH STANDARDS INSTITUTION

BS 4757:1971. Specification for nineteen-wire steel strand for prestressed concrete

BS 4486:1980. Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete

BS 5400-4:1978. Steel, concrete and composite bridges. Code of practice for design of concrete bridges

BS 5896:1980. Specification for high tensile steel wire and strand for the prestressing of concrete

BS 6089:2010. Assessment to in situ compressive strength in structures and precast concrete components – Complementary guidance to that given in BS EN 13791

BS 8110-1:1997. Structural use of concrete. Part 1. Code of practice for design and construction (incorporating amendments 1 to 4, 2007)

BS 8110-2:1985. Structural use of concrete. Part 2. Code of practice for special circumstances (incorporating amendments 1 to 3, 2005)

BS EN 1992-1-1:2004. Eurocode 2. Design of concrete structures – Part 1-1: General rules and rules for buildings (incorporating amendments 1 and 2, 2011)

BS EN 1992-2:2005. Eurocode 2. Design of concrete structures – Part 2: Concrete bridges – design and detailing rules (incorporating amendment 1, 2010)

CP110. Parts 2 and 3:1972. British Standards code of practice - Use of concrete

PD 6687-2:2008. Recommendations for the design of structures to BS EN 1992-2:2005

Other Documents

1. INSTITUTION OF STRUCTURAL ENGINEERS. Appraisal of existing structures. October 2010.
2. THE CONCRETE SOCIETY. Concrete core testing for strength. Technical Report No. 11, 1987.
3. CLARK, L. A. Elastic analysis of concrete bridges and the ultimate limit state. The Highway Engineer, October 1977, pp. 22-24.

4. CLARK, L. A. Concrete bridge design to BS 5400. Construction Press, 1983, pp. 186. (Including supplement incorporating 1984 revisions, 1985, pp. 31).
5. NIELSEN, M. P. and HOANG, L. C. Limit analysis and concrete plasticity. Prentice-Hall, 2011, 816 pp.
6. IBELL, T. J., MORLEY, C. T. and MIDDLETON, C. R. Shear assessment of concrete beam-and-slab bridges. Proceedings of the Institution of Civil Engineers, Structures and Buildings 128 (3), 1998, pp. 386-391.
7. VALERIO, P., IBELL, T. J. and DARBY, A. P. Shear assessment of prestressed concrete bridges. Proceedings of the Institution of Civil Engineers, Bridge Engineering 164 (4), 2011, pp. 195-210.
8. VALERIO, P. Realistic shear assessment and novel strengthening of existing concrete bridges. PhD thesis, University of Bath, 2009, 235 pp.
9. COPE, R. J. and RAO, P. V. A two-stage procedure for the non-linear analysis of slab bridges. Proceedings of the Institution of Civil Engineers, Vol. 75, Part 2, December 1983, pp. 671-688.
10. COPE, R. J. and CLARK, L. A. Concrete slabs: analysis and design. Elsevier Applied Science, 1984, 507 pp.
11. SIVIERO, E. Rotation capacity of monodimensional members in structural concrete. Istituto Universitario di Architettura di Venezia, Istituto di Scienza delle Costruzioni, 1973, pp. 17.
12. MARSHALL, W. T. A survey of the problems of lateral instability in reinforced concrete beams. Proceedings of the Institution of Civil Engineers, Vol. 43, July 1969, pp. 397-406.
13. KENT, D. C. and PARK, R. Flexural members with confined concrete. Proceedings of the American Society of Civil Engineers, Vol. 97, No. ST7, July 1971, pp. 1969-1990.
14. CHAN, W. W. L. The ultimate strength and deformation of plastic hinges in reinforced concrete frameworks. Magazine on Concrete Research, Vol. 7, No. 21, November 1955, pp. 121-132.
15. PEDERSON, C. Shear in beams with bent-up bars. Final Report of the International Association for Bridge and Structural Engineering Colloquium on Plasticity in Reinforced Concrete, Copenhagen, 1979, pp. 79-86.
16. SHEAR STUDY GROUP. The shear strength of reinforced concrete beams. Institution of Structural Engineers, London, 1969, pp. 170.
17. CORGI, J. L. Shear capacity of high strength concrete beams. Concrete, Vol. 21, No. 3, March 1987, pp. 24-26.
18. BURTON, K. T. and HOGNESTAD, E. Fatigue tests of reinforcing bars – tack welding of stirrups. Proceedings of the American Concrete Institute, Vol. 64, No. 5, May 1967, pp. 244-252.
19. BAKER, A. L. L., YU, C. W. and REGAN, P. E. Explanatory note on the proposed Unified Code Clause on shear in reinforced concrete beams with special reference to the Report of the Shear Study Group. The Structural Engineer, Vol. 47, No. 7 July 1969, pp. 285-293.
20. CONCRETE BRIDGE DEVELOPMENT GROUP. Testing of high strength concrete beams with limestone aggregate. Technical Paper no 7, 2003.

21. SOMERVILLE, G. The behaviour and design of reinforced concrete corbels. Cement and Concrete Association, Technical Report 42.472, August 1972, pp. 12.
22. SHAVE, J. D., IBELL, T. J. and DENTON, S. R. Shear assessment of reinforced concrete bridges with short anchorage lengths. The Structural Engineer, Vol. 85, No. 5, March 2007, pp. 30-36.
23. JACKSON, P. and SALIM, S. Web crushing in EN 1992. The Structural Engineer, Vol. 84, No. 23, December 2006, pp. 50-57.
24. AMERICAN CONCRETE INSTITUTE COMMITTEE 438. Tentative recommendations for the design of reinforced concrete members to resist torsion. Proceedings of the American Concrete Institute, Vol. 66, No. 1, January 1969, pp. 1-8.
25. LAMPERT, P. Torsion and bending in reinforced and prestressed concrete members. Proceedings of the Institution of Civil Engineers, Vol. 50, December 1971, pp. 487-505.
26. KENNEDY, G. and GOODCHILD, C. Practical yield line design. British Cement Association, 2003, 171 pp.
27. REGAN, P. E. Behaviour of reinforced concrete flat slabs. CIRIA Report 89, London 1981, pp. 89.
28. CULLINGTON, D. W. and HILL, M. E. Assessment of reinforced concrete slab bridge decks with circular voids. Bridge Management 4, Thomas Telford Publishing, 2000.
29. CLARK, L. A. and THOROGOOD, P. Flexural and punching shear strengths of concrete beams and slabs. Transport and Road Research Laboratory, Contractor Report 60, 1987, pp. 33.
30. LEE, D. J. The theory and practice of bearings and expansion joints for bridges. Cement and Concrete Association, Publication 12.036, 1971, pp. 65.
31. TIMOSHENKO, S. P. Theory of elastic stability. McGraw-Hill Book Co., New York and London, 1936, pp. 518.
32. JACKSON, P. A. Slender bridge piers and the effective height provisions of BS 5400: Part 4. Cement and Concrete Association, Technical Report 42.461, June 1985, pp. 18.
33. CRANSTON, W. B. Analysis and design of reinforced concrete columns. Cement and Concrete Association, Research Report 41.020, 1972, pp. 54.
34. CRANSTON, W. B. A computer method for the analysis of restrained columns. Cement and Concrete Association, Technical Report 42.402, April 1967, pp. 20.
35. MENEGOTTO, M. and PINTO, P. E. Slender RC compressed members in biaxial bending. Proceedings of the American Society of Civil Engineers, Vol. 103, No. ST3, March 1977, pp. 587-605.
36. CRANSTON, W. B. Computer analysis of columns. Instability and plastic collapse of steel structures, Proceedings of The Michael R. Horne Conference. Editor: L. J. Morris. St. Albans, Granada Publishing Ltd., 1983, pp. 243-252.
37. AMERICAN CONCRETE INSTITUTE. Building code requirements for reinforced concrete (ACI 318-83), 1983, pp. 111.

38. REGAN, P. E. and BRAESTRUP, M. W. Punching shear in reinforced concrete. CEB Bulletin No. 168, January 1985, pp. 232.
39. INSPECTION MANUAL FOR HIGHWAY STRUCTURES. The Stationery Office, 2007.
40. REYNOLDS, G. C. Bond strength of deformed bars in tension. Cement and Concrete Association, Technical Report 42.548, May 1982, pp. 23.
41. ODUYEMI, T. O. S. and CLARK, L. A. Prediction of crack widths in circular-voided reinforced concrete slabs subjects to transverse bending. Magazine of Concrete Research, Vol. 39, No. 140, September 1987, pp. 124-132.
42. SNOWDON, L. C. Classifying reinforcing bars for bond strength. Building Research Station Current Paper 36/70, November 1970.
43. NAVARATNARAJAH, V, and SPEARE, P. R. S. An experimental study of the effects of lateral pressure on the transfer bond of reinforcing bars with variable cover. Proceedings of the Institution of Civil Engineers, Vol. 81, Part 2, December 1986, pp. 697-715.
44. MORITA, S. and FUJI, S. Bond capacity of deformed bars due to splitting of surrounding concrete. Bond in Concrete, Applied Science Publishers, 1982, pp. 331-341.
45. ROBINS, P. J. and STANDISH, I. G. Effect of lateral pressure on bond of reinforcing bars in concrete. Bond in Concrete, Applied Science Publishers, 1982, pp. 262-272.
46. ROBINS, P J and STANDISH, I G. The influence of lateral pressure upon anchorage bond. Magazine of Concrete Research, Vol. 36, No. 129, December 1984, pp. 195-202.
47. JIRSA, J, O'BREEN, J E, LUKE, J J and HAMAD, B S. Effect of casting position on bond. In: Bond in Concrete, Applied Science Publishers, 1982, pp. 300-307.
48. CAIRNS, J. An analysis of the ultimate strength of lapped joints of compression reinforcement. Magazine of Concrete Research, Vol. 31, No. 106, March 1979, pp. 1927.
49. SOROUSHIAN, P, OBASEKI, K and ROJAS, M. Bearing Strength and Stiffness of Concrete Under Reinforcing Bars. ACI Materials Journal, May-June 1987.
50. CLARK, L. A. Longitudinal shear reinforcement in beams. Concrete, Vol. 18, No.2, February 1984, pp. 22-23.
51. REGAN, P. E. Concrete Society Current Practice Sheet No. 105, Shear, Concrete, Vol. 19, No. 11, November 1985, pp.25.
52. ODUYEMI, T. O. S. and CLARK, L. A. Tension stiffening in transverse sections of circular-voided concrete slabs. Magazine of Concrete Research, Vol. 39. No. 138, March 1987, pp. 35.41.
53. ODUYEMI, T. O. S. Stiffnesses and cracking of voided slab bridges. PhD Thesis, University of Birmingham, September 1986.
54. CLARKE J. L. Shear strength of lightweight aggregate concrete. Concrete, Vol. 21, No. 4, April 1987, pp. 12-13.

55. ANDREW, A. E. Unbonded tendons in post-tensioned construction. Thomas Telford, 1987.
56. VIRLOGEUX, M. External prestressing of concrete. FIP Notes, 1987/2, pp. 16-20.
57. SWAMY, R. N. Prestressed lightweight concrete. Developments in prestressed Concrete – 1, Sawko, F. (Ed.) Applied Science Publishers, 1978, pp. 149-191.
58. CLARK, L. A. and TOMS, S. Shear Strength of Beam with Unbonded or External Tendons. Proceedings of FIP Symposium, London, 1996.
59. HAWKINS, N. M. The shear provision of AS CA35-SAA Code for prestressed concrete. Civil Engineering Transactions, Institute of Engineers, Australia, Vol. CE6, September 1964, pp. 103-116.
60. REYNOLDS, G. C., CLARKE, J. L. and TAYLOR, H. P. J. Shear provisions for prestressed concrete in the Unified Code, CP 110:1972. Cement and Concrete Association, Technical Report 42.500, October 1974, pp. 16.
61. SWAN, R. A. and WILLIAMS, A. Combined loading tests on model prestressed concrete box beams with steel mesh. Cement and Concrete Association, Technical Report 42.485, October 1973, pp. 43.
62. MAISEL, B. I. and SWAN, R. A. The design of concrete box spine-beam bridges. CIRIA Report 52, November 1974, pp. 52.
63. BEEBY, A. W., KEYDER, E. and TAYLOR, H. P. J. Cracking and deformations of partially prestressed concrete beams. Cement and Concrete Association, Technical Report 42.465. January 1972, pp. 26.
64. CLARK, J. L. A guide to the design of anchor blocks for post-tensioned prestressed concrete members. CIRIA Guide 1, June 1976, pp. 34.
65. ZIELINSKI, J. and ROWE, R. E. An investigation of the stress distribution in the anchorage zones of post-tensioned concrete members. Cement and Concrete Association, Research report 9, September 1960, pp. 32.
66. KRIZ, L. B. and RATHS, C. H. Connections in precast concrete structures: strength of corbel brackets. Journal of the Prestressed Concrete Institute. Vol. 10, No. 1, February 1965, pp. 16-61.
67. ROWE, R. E. et al. Handbook to British Standard BS 8110: 1985. The Structural Use of Concrete. Viewpoint Publication, 1987, pp. 206.
68. CLARK, L. A. and THOROGOOD, P. Serviceability behaviour of reinforced concrete half-joints. Transport and Road Research Laboratory, Contractor Report 70, 1988.
69. CULLINGTON, D. W., HILL, M. E. and SYKES, D. P. Shear capacity of pretensioned inverted T-beams with infill concrete. Transport Research Laboratory, Project Report PR/CE/130/95, 1995.
70. HUGHES G. Longitudinal shear in composite concrete bridge beams. Part 2: Experimental investigation and recommendations. Transport and Road Research Laboratory, Contractor Report 52, 1987, pp. 97.