THE HIGHWAYS AGENCY

THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF THE ENVIRONMENT FOR
NORTHERN IRELAND

The Assessment of Concrete Highway Bridges and Structures

Summary: This Advice Note provides guidance on the assessment of concrete highway bridges and structures and accompanies BD 44 (DMRB 3.4.14). It supersedes BA 44/90.

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REGISTRATION OF AMENDMENTS

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VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 15

BA 44/96

THE ASSESSMENT OF CONCRETE HIGHWAY BRIDGES AND STRUCTURES

Contents

November 1996

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1. INTRODUCTION

General

1.1 This Advice Note provides guidance on the use of BD 44 (DMRB 3.4.14), The Assessment of Concrete Highway Bridges and Structures, and should be used in conjunction with the Standard.

1.2 The major part of this document is contained in Annex A which is set out in the form of a commentary on the Standard. It contains explanations for the main changes from the design code, BS 5400: Part 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 should be agreed with the Overseeing Organisation.

Scope

1.3 In view of the problems associated with the corrosion of tendons in grouted duct post-tensioning, all bridges with this form of construction are to be examined in accordance with BD 54, Post-tensioned Concrete Bridges; Prioritisation of Special Inspections, (DMRB 3.1.2). The condition of tendons in these structures should be investigated in accordance with BA 50, Post-tensioned Concrete Bridges; Planning, Organisation and Methods for Carrying Out Special Inspections, (DMRB 3.1.3).

1.4 Structures with corroded reinforcement or prestressing steel should be assessed in conjunction with BA 51, The Assessment of Concrete Structures Affected by Steel Corrosion, (DMRB 3.4.13). Structures suspected of suffering from Alkali-Silica Reaction (ASR) should be assessed in conjunction with BA 52, The Assessment of Structures affected by Alkali-Silica Reaction, (DMRB 3.4.10).

Implementation

1.5 This Advice Note should be used in all future assessments of structures or structural elements. It should also be taken into account in assessments currently at hand unless, in the opinion of the Overseeing Organisation, this would result in an unacceptable additional expense or delay.

2. USE OF ANNEX A - THE COMMENTARY

2.1 The format of the commentary is based on the clause numbering used in Appendix A of BD 44 (DMRB 3.4.14) To assist in identifying these clauses, section headings and subheadings have been retained. All other headings have been omitted for clarity.

2.2 Comments are given on those clauses where the changes from BS 5400: Part 4 are substantial or are not self evident. There are also comments on some of the clauses in BD44 which have been marked as "Not applicable to assessment". A number of these clauses relate to serviceability criteria and should only be included in an assessment on the direction of the Overseeing Organisation.

2.3 In the commentary there are many references to detailed aspects of BS 5400: Part 4. Therefore, to gain the maximum benefit from the commentary, a working knowledge of the code is necessary.

2.4 A list of references is included at the end of Annex A. Where an in-depth investigation of certain aspects of assessment is required these references should provide an additional source of background information.

2.5 References to BS 5400: Part 4 have been abbreviated to BS 5400 and should be taken as references to the document as implemented by BD 24 (DMRB 1.3.1).

2.6 Definitions of symbols used in the commentary are given in Appendix A of BD 44.

3. REFERENCES

1. Design Manual for Roads and Bridges (DMRB): HMSO

BD 44 The Assessment of Concrete Highway Bridges and Structures. (DMRB 3.4.14).

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

BD 54 Post-tensioned Concrete Bridges; Prioritisation of Special Inspections. (DMRB 3.1.2)

BA 38 Assessment of the 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 50 Post-tensioned Concrete Bridges; Planning, Organisation and Methods for Carrying Out Special Inspections. (DMRB 3.1.3)

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

BA 52 The Assessment of Structures affected by Alkali-Silica Reaction. (DMRB 3.4.10).

2. British Standards

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

4. ENQUIRIES

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ANNEX A - COMMENTARY ON APPENDIX OF DEPARTMENTAL STANDARD BD 44/95

2. DEFINITIONS AND SYMBOLS

2.1.3.1 (b) - Worst Credible Strength

The worst credible strength of reinforcement or prestressing tendons may be obtained by extracting and testing bar or tendon samples. However, it is generally impractical to extract samples from critical sections. In choosing lengths of bars for testing, the assessing engineer should ensure that the removal of samples will not reduce the carrying capacity of the element under consideration.

In order to avoid the mechanical damage to bars during removal of concrete, methods such as water-jetting may be preferred. Also, in cutting out the chosen samples of bars, care should be taken not to damage adjacent bars which may be highly stressed at the location in question.

In the case of concrete, methods of assessing the estimated in-situ concrete strength at a location are given in BS 6089[(4)]. A location is defined as a region where, in the engineer'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 in-situ concrete strength; e.g for cores it would be equal to the mean estimated in-situ concrete strength minus $(20/\sqrt{n})$% of the strength, where n is the number of cores tested. For n cores giving equivalent cube strengths of f_c .... f_{cn}.

$$\text{W.C.S} = \frac{\Sigma f_c}{100n}\left(100 - \frac{20}{\sqrt{n}}\right)$$

In applying this formula the engineer must be satisfied that the cores are representative of the location under consideration. The worst credible strength should be based on a minimum of three cores.

If the assessing engineer wishes to use a single worst credible concrete strength for the structure as a whole, rather than individual values at individual critical locations, it is suggested that he uses his judgement 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 m^3 of concrete. The worst credible strength should be taken as either the least of the individual values or derived in accordance with the formula above. If the concrete strength is being assessed by a method other than core testing (e.g by the internal fracture test) then a greater number of tests is required: Bungey gives guidance on the number of tests which are equivalent to one core test[(59)].

In the case of steel reinforcement or prestressing tendons, the worst credible strength may be taken as the lower bound to the in-situ concrete steel strength and based on a 99% confidence limit and a coefficient of variation which is representative of the steel. Alternatively, the worst credible strength may be based on the formula for concrete cores given above. In determining the steel strength, the engineer must be satisfied that the samples tested are representative of the location under consideration.

It should be noted that, when using a worst credible steel strength in excess of the characteristic strength, it is essential to check that the bar anchorages and laps are capable of developing the higher steel stresses.

The Engineer's knowledge of material strengths which were typical of the period of construction may in some instances aid his judgement of appropriate values for the structure under consideration.

Where the development of a failure mechanism would require the yielding of a considerable number of bars, then the assessing engineer should bear in mind that the worst credible strength used for the reinforcement should be that relating to the average strengths of the bars concerned.

Guidance on the investigation of concrete highway structures is given in BA 35 (DMRB 3.3)[60].

3. LIMIT STATE PHILOSOPHY

3.1 General

Assessment at the serviceability limit state is only necessary when specifically asked for by the Overseeing Organisation.

4. ASSESSMENT GENERAL

4.1.1 Serviceability Limit State

Where the Overseeing Organisation requires a serviceability limit state check, the criteria should generally be those specified in BS 5400. However, it may be possible to relax the serviceability criteria compared with the BS 5400 levels, in association with certain changes in the future management of the structure, e.g. increased frequency of inspection.

4.3.3.2 Where a serviceability limit state check is required, the values of γ_{mc} and γ_{ms} should, in general, be based on Table 4 of BS 5400: Part 4. Where worst credible strengths are used, the values of γ_{mc} may be reduced by 10% providing they are not taken as less than unity.

4.3.3.3 Ultimate Limit State

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

i. 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;

ii. γ_{m2} which takes account of possible weaknesses of the structure arising from any other cause.

In the case of 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 the Institution of Structural Engineers[1] and BS 8110: Part 2[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 [3]. This implies that, at the design stage, γ_{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[4] implies that a γ_m value of 1.2 should be applied to the mean estimated in-situ cube strength, whereas BS 8110: Part 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 would be extremely 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.

4.4.3 Analysis at Ultimate Limit State

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[5]. 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. These more accurate methods of analysis 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 has been given on the applications of such methods to bridges in reference[6].

ii. Non-linear analyses which are capable of predicting the behaviour of a structure at all stages up to collapse. At present most non-linear analyses are capable of predicting only flexural failures. Guidance on their use has been given in references 1 and 8.

iii. Methods which take account of restraints which are generally ignored in design; eg membrane action in top slabs of beam and slab decks[8].

4.7 Fatigue

For structures which are considered by the assessor to be fatigue prone, a fatigue assessment should be undertaken. 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 to once a year or as advised by the assessing engineer.

Compliance criteria for welded bars are covered by BS 5400: Part 10. The requirements for unwelded non-corroded bars given in BD 44, are based on a study carried out for the Department of Transport.

4.8.2 Analysis of Structure

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 should be considered. See also 4.4.3.

5. ASSESSMENT: REINFORCED CONCRETE

5.1.2.2 Durability

If a structure has a smaller cover than that given in 5.8.2 it does not necessarily mean that it is inadequate. However, it does suggest that frequent inspection is required.

5.1.4 Strength of Materials

See 2.1.3.1.

5.1.4.2 Strength of Concrete

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 28 day works cube strength.

5.2.4.3 Strength of Reinforcement

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.

5.2.2(d) Redistribution of Moments

Criterion (d) of BS 5400 has been omitted. The BS 5400 criterion (d) limited moment redistribution to members up to 1.2m deep, where as the available test data[9] on rotation capacity only cover members up to about 0.8m deep. Furthermore, it seems illogical to limit moment redistribution to members of a certain depth whilst permitting plastic methods to be applied to members of any depth. In view of the fact 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 to include also a specific limitation on depth.

5.3.1.3 Slenderness Limits for Beams

The design code gives two limits: one a function of b_c, and the other a function of b_c^2/d. According to Marshall[10], 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 Marshall's value by a partial safety factor of 1.5.

5.3.2.1 Analysis of Sections

The BS 5400 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.

The concrete stress strain curve in Figure 1 and the failure strain of .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 72 and 73.

5.3.2.2 Design Charts

The CP 110 charts include γ_{mc} value of 1.5, and γ_{ms} value of 1.15. When the reduced values of 1.20 for concrete and 1.10 or 1.05 for steel, (see 4.3.3.3) are used, the charts can still be used by adopting enhanced values of f_{cu} (by multiplying the actual value of f_{cu} by 1.5/1.2 = 1.25) and f_y (by multiplying the actual value of f_y by 1.15/1.10 = 1.05 or 1.15/1.05 = 1.10 as appropriate).

5.3.3.1 Shear Stress

The design limiting value of v in BS 5400 is given as $0.75\sqrt{f_{cu}}$ but not greater than 4.75 N/mm^2, which implies an upper limit on f_{cu} of 40N/mm^2. The partial safety factor included in the design value is 1.5 on f_{cu}; i.e. a nett value of $\sqrt{1.5} = 1.22$. Hence, the assessment value is $1.22 \times 0.75 \sqrt{(f_{cu}/\gamma_{mc})}$. The value $0.92\sqrt{f_{cu}}$ forms a lower bound to the test data[11]. Since the test data included concrete cube strengths up to 56 N/mm^2, an upper limit of $0.92 \sqrt{(56/\gamma_{mc})} \approx 7/ \sqrt{\gamma_{mc}}$ has been imposed.

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

5.3.3.2 Shear Reinforcement

Assessment clause 5.3.3.2 is a rearrangement of the BS 5400 design clause. Shear reinforcement at an angle to the member axis is treated in a general way. All of the shear can be resisted by bent-up bars, since Pederson[12] has demonstrated that such shear reinforcement is fully effective. Since test data are not available for $\propto < 30^o$, no attempt has been made to allow for shear reinforcement bent at such angle.

The upper limit of 480 N/mm^2 for the strength of shear reinforcement is the value which the Shear Study Group[14] found should be imposed in order to guarantee that the shear reinforcement would yield at collapse prior to crushing of the concrete.

The constant 0.27 in the BS 5400 expression for v_c has been reduced to 0.27 x 1.1/1.25 = 0.24, because the BS 5400 expression is actually the mean value divided by γ_m [13]. However, γ_m should be applied to the characteristic value, and the BS 5400 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 may be taken as 0.27.

The BS 5400 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[51].

The maximum spacing of links is specified as d because test data [15] show a reduction in shear strength at this spacing rather than the BS 5400 value of 0.75d.

The BS 5400 upper limit on f_{cu} of 40 N/mm^2 is not included because Clarke[13] has shown that it is not justified by test data collected for values up to 117 N/mm^2.

In line with BS 5400, the shear capacity in the assessment code is related to the area of longitudinal reinforcement in excess of that required to resist bending [39]. The treatment of 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:

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

However, when $\propto > 45^o$, V_u shall not be taken as greater than

$$V_u = \frac{2A_{lv} \left(f_y / \gamma_{ms}\right)}{1 - \cot \propto} + \xi_s v_c$$

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.

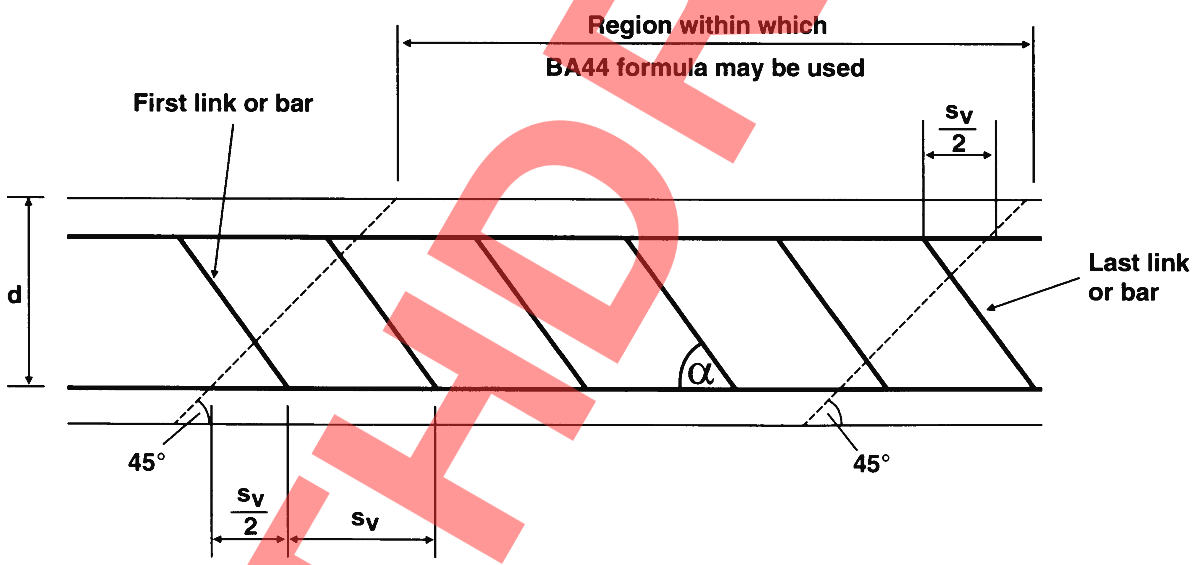

Figure 5.1 Inclined Shear Reinforcement

Shear has to be resisted by a combination of links and longitudinal reinforcement[39]. 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.

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

(1) The contribution to shear from the concrete has been discounted when calculating the additional longitudinal reinforcement requirement.

(2) In calculating the flexural tensile force which is co-existent with the longitudinal force associated with shear, the lever arm is limited to 0.9d

(3) 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 and 5.8.6.8 are not complied with.

BS 5400 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. The shear capacity of a member in which the shear reinforcement does not satisfy both the minimum area and the maximum spacing criteria should be taken as the concrete resistance alone.

5.3.3.3 Enhanced Shear Strength of Sections Close to Supports

The critical distance of a_v and the enhancement factor are both less conservative than the BS 5400 values. However the assessment values, which were proposed by Somerville[16], give a good lower bound fit to the test data.

Tests indicate that, where $a_v < d$ the load is transferred to the support by direct strut action and the ultimate shear strength of the concrete rises sharply. Sections less than d from the support are therefore not normally critical for shear. This has been reflected in the assessment code.

5.3.4.3 Stress and Reinforcement (Torsion)

v_{tmin} is 25% of the pure torsional strength without torsional reinforcement, and was chosen by American Concrete Institute Committee 438[17] 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/mm^2, as is required by BS 5400.

The test data[18] used to derive the expression for v_{tu} included cube strengths up to 57.9 N/mm^2. Hence, the upper limit or v_{tu} is given as $0.92 \sqrt{(57.9/\gamma_{mc})} = 7/\sqrt{\gamma_{mc}}$.

5.3.4.4 Treatment of Various Cross Sections

(a) - Box Sections

Equations 10 and 11 of BS 5400 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 10/11A is the general expression for torsional strength when the longitudinal and transverse steel do not necessarily make equal contributions to the torsional strength[19].

However, it should be noted that excessive torsional cracking could occur under service load conditions if the ratio of the first to second terms under the square root sign of equation 10/11A lies outside the range 2/3 to 3/2.

(b) - Rectangular Sections

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

(c) - T, L and I Sections

The assessing Engineer may choose any division of component rectangles compatible with the reinforcement in them. Hence, any unreinforced regions of a section may be ignored for torsional assessment purposes.

Provided that the sum of the torsional stiffnesses 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. However, excessive torsional cracking could occur under service load conditions in unreinforced regions of the section.

5.3.4.5 Detailing

The BS 5400 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 clause which relates to varying the ratio of link to longitudinal steel is now covered by equation 10/11A.

5.4.1 Moments and Shear Forces in Slabs

See 4.4.3.

5.4.2 Resistance Moments of Slabs

If a slab has n directions of reinforcement in a face, each of which is at angle $\propto_i$ to the x-axis (see Fig.5.2) and provides a moment of resistance M^*_i in its own direction, then it can resist the set of bending moments M_x and M_y, and the twisting moment, M_{xy}, if:

$$\Sigma|[M^*_i \cos^2\alpha_i)-M_x][\Sigma(M^*_i \sin^2\alpha_i)-M_y]| \geq [\Sigma(M^*_i \sin\alpha_i\cos\alpha_i) + M_{xy}]^2$$

The above expression is the general yield criterion for a slab element[20]. It is emphasised that the numerical values of M_i^* should be taken as positive for bottom (sagging) reinforcement, and negative for top (hogging) reinforcement. The well known Wood-Armer design equations are derived from this expression.

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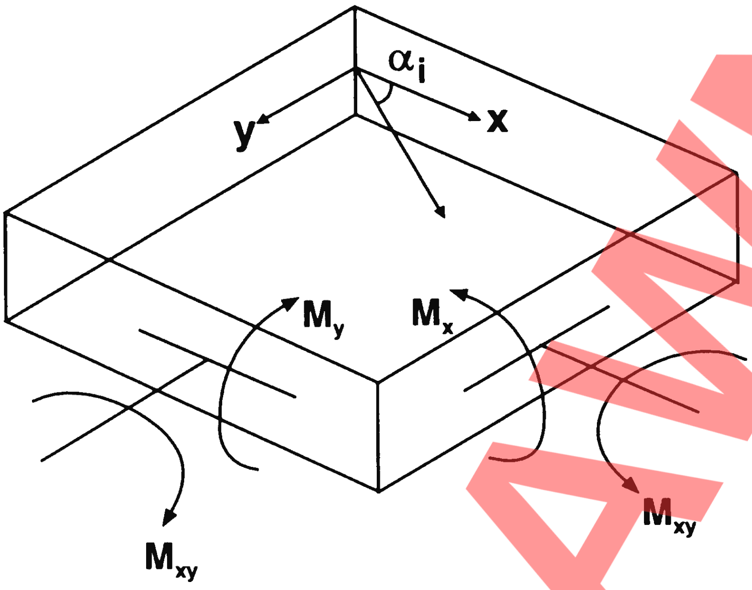

Figure 5.2 In-plane Forces

5.4.3 Resistance to In-Plane Forces

If a slab of overall thickness h has n directions of reinforcement, each of which is at angle $\propto_i$ 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:

$$[\Sigma(N^*_i \cos^2\alpha_i)-N_x][\Sigma(N^*_i \sin^2\alpha_i)-N_y] \geq [\Sigma(N^*_i \sin\alpha_i \cos\alpha_i) + N_{xy}] \text{ and}$$
$$(N_c + N_x)(N_c + N_y) \geq N^2_{xy}$$

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

The above expressions are the general yield criterion for a slab element subjected to in-plane forces[21].

5.4.4.1 Shear Stress in Solid Slabs: General

This clause is essentially a rearrangement of the BS 5400 clause. Reference has been made to 5.3.3.3 to permit short shear span enhancement for both slabs and beams.

5.4.4.2 Shear Stresses in Solid Slabs Under Concentrated Loads (Including Wheel Loads).

This clause is essentially a rearrangement of the BS 5400 clause. However, all shear reinforcement within the critical perimeter is considered effective.

The upper limit on f_{yv} have been increased to 480 N/mm^2 (see 5.3.3.2 comment).

Enhancement of v_c has been permitted for short shear spans.

The capacity reduction factor of 0.8 in Fig. 5A (b) and (c)(i) was introduced into building codes to allow for moment transfer at edge and corner columns[31].

5.4.4.3 Shear in Voided Slabs

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 shall not be greater than 0.7 where ϕ is the diameter of the void and b is the distance between void centres.

ii. ϕ/h shall not be greater than 0.65 where h is the overall depth of the slab.

iii. The thickness of the compression flange shall not be less than 0.4(h-ϕ).

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

$V_{cv} = KV_{c'}$ where

$V_{c'}$ is the shear resistance of the solid slab ignoring the presence of voids, calculated in accordance with BD44.

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

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

Voided slab bridge decks which do not comply with the dimensional criteria above or which are found to have an insufficient shear capacity when assessed in accordance with this Clause, should be referred to the Overseeing Organisation.

The BS 5400 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[6].

Guidance on the punching of loads through a voided slab as a whole is given in reference[22].

5.5.1.2 Effective Height of a Column

The effective heights in Table 11 relate to idealised situations. In design, an engineer 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 Engineer should then consult specialist literature[23-25].

5.5.1.3 Slenderness Limits for Columns

The BS 5400 slenderness limit of l_e/h of 40 was chosen because it was considered to be a practical upper limit[6]. However, the study of Cranston[26], on which the BS 5400 column clauses are based, included l_o/h values of up to 60. Hence, the latter limit, which was also in BS 5400:Part 4:1978, has been adopted in the assessment code. If this slenderness limit is exceeded a full non-linear analysis should be undertaken.

The BS 5400 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.

5.5.3.1 General (Short Columns)

The nominal eccentricities of BS 5400 may be replaced by the actual eccentricities when they are measured.

5.5.3.2 Analysis of Section

See comment on 5.3.2.1 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

See comment on 5.3.2.2

5.5.4 Short Columns Subject to Axial Load and either Bending about Major Axis or Biaxial Bending

See comment on 5.3.3.1.

The expression for $\propto_n$ gives the same values as Table 12 of BS 5400. It should be noted that the $\propto_n$ values are conservative[6]. 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 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.

5.5.5.1 General (Slender Columns)

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[25]. Hence, in some cases it may be preferable to carry out a full non-linear analysis[25-29].

A circular column subject to biaxial bending can be assessed for the resultant moment about a single axis.

5.5.6 Shear Resistance of Columns

The shear strength enhancement factor to allow for the axial load is that adopted in the ACI Code[30] and is less conservative than the BS 5400 factor. The requirements for calculating the shear capacity of a circular column are based on those of the ACI Code.

5.6.1.2 Limits to Slenderness

See comments on 5.5.1.2 and 5.5.1.3.

5.6.2 Forces and Moments in Reinforced Concrete Walls

See comment on 5.3.3.1.

5.6.4 Slender Reinforced Walls

See comment on 5.5.5.1.

5.6.5 Shear Resistance of Reinforced Walls

See comment on 5.5.6.

5.7.3.1 Resistance to Bending (Bases)

With regard to distributions of reinforcement, the assessment clause is a rearrangement of the BS 5400 design clause.

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

5.7.3.2(1) Shear

The short shear span enhancement factor is greater than the BS 5400 value (see comment on 5.3.3.3).

5.7.3.2(2)

The short shear span enhancement factor permitted for punching shear in 5.4.4.2 (see comment on 5.4.4.2) will often be beneficial when assessing pile caps.

Difficulties can arise in applying 5.4.4.2 and Figure 5A 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 8, 31 and 32.

5.7.3.3 Bond and Anchorage

The local bond clause of BS 5400 is not relevant: see comment on 5.8.6.2.

5.8.1.2 Accuracy of Position of Reinforcement

See comment on 4.3.3.3. Guidance on the measurement of the location of reinforcement by the use of covermeters is given in BA 23/86[60].

5.8.2 Concrete Cover to Reinforcement

The BS 5400 covers should, ideally, be present in all structures, although adequate durability may often be achieved with good quality and well compacted concrete with smaller covers. However, the presence of smaller covers suggests that the inspection frequency may need to be increased. If the cover is substantially less than the BS 5400 values bond strength could be reduced[58].

5.8.4.1 Minimum Area of Main Reinforcement

The minimum areas of tension reinforcement in a beam or slab specified in BS 5400 are intended to ensure that the reinforcement does not yield as soon as cracking occurs, and extremely wide cracks are thereby avoided. This may also be achieved by ensuring that the area of tension reinforcement is not less than $0.167\ b_a d(f_t/f_y)$[6] where f_t, 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 values can be obtained from this expression by assuming a value of 50 N/mm² for f_{cu}. If a section has less reinforcement than the specified minimum it may have adequate strength but could develop very wide cracks. It should therefore be inspected frequently to ensure that it remains durable.

The minimum number of longitudinal bars present in a column should be four in rectangular columns and six in circular columns. The BS 5400 minimum bar diameter of 12mm is intended to ensure a rigid cage for construction. This requirement is not relevant to assessment. The BS 5400 minimum steel areas for columns ensure that reinforcement yield does not occur under service load conditions[6]. Although it is not considered necessary to impose a minimum steel area for assessment purposes, the engineer should be aware that high service load stresses can occur in columns having less than the BS 5400 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 requirement of this clause may result in large cracks developing. More frequent inspections should be considered.

5.8.4.2 Minimum area of secondary reinforcement

Extremely 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 5.8.4.1.

(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;
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[6] whereas the minimum areas given in 5.8.4.1 merely ensure that the steel will not yield at first cracking[34].

In a solid slab or wall, the main reinforcement should 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. Failure to comply with the requirements of this clause may result in large cracks developing. More frequent inspections should be considered.

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. When there is less secondary reinforcement than the specified minimum, the compressive strength of the bars should be reduced in proportion to the ratio of the actual to specified minimum secondary steel areas.

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The BS 5400 reference to early thermal movement is not relevant to assessment.

See 5.8.4.1 for general comments on the implication of minimum steel areas not being provided.

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 resist compression. These minimum link requirements are intended to ensure restraint of compression bars so that their full compressive strength can be developed. When there is less link reinforcement than the specified minimum, the compressive strength of the bars should be reduced in proportion to the ratio of the actual to specified minimum link areas.

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.

In a beam, the spacing of links should not exceed the effective depth of the beam, nor should the lateral spacing of the individual legs of the links exceed this value. Links should enclose all tension reinforcement. The link spacing in beams is discussed in the comment on 5.3.3.2.

If the links do not enclose all of the tension reinforcement, then the dowel capacity of the reinforcement not enclosed cannot be relied on and the area of the reinforcement not enclosed should not be included in A_s when obtaining v_c from 5.3.3.2. However, if there are no links, then failure is deemed to occur immediately the shear stress attains v_c, and thus, there is no requirement to maintain the dowel component. It should also be noted that in the case of a slab or wall, transverse reinforcement outside the main reinforcement can also provide the necessary restraint to maintain the dowel component.

5.8.5 Maximum Areas of Reinforcement in Members

Maximum steel areas are specified in BS 5400 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 maxima (4% in beams, slab and walls, and 6% to 10% in columns) the concrete could be poorly compacted and particular attention should be given to inspection of such sections.

5.8.6.2 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[(2)]. Hence only anchorage bond need be considered.

5.8.6.3 Anchorage Bond

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

BS 8110 specifies a partial safety factor on the bond stress of 1.4. This partial safety factor allows for variations in both concrete strength and in 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 comment on 4.3.3.3). If it is assumed that γ_{mb} can be expressed as $\sqrt{(\gamma_{mc}\,\gamma_{mbs})}$, 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, γ_{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 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 because it is assumed that the code's covers, nominal link requirements and detailing clauses will be satisfied. In an assessment these various 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 58, 65-71.

5.8.6.4 Effective perimeter of a Bar or Group of Bars

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

5.8.6.5 Anchorage of Links

The requirements of 5.8.6.5 ensure that a link is fully anchored. A link which does not satisfy the requirements may be considered as partially anchored and, hence, as partially effective in resisting shear and providing restraint to compression reinforcement. It is not possible to make general recommendations on the degree of partial effectiveness, and, thus, each case has to be treated individually and agreed with the relevant Overseeing Organisations. However, the stress in the link should be limited to the value which would cause a bearing failure in the corner of link when calculated in accordance with 5.8.6.9.

5.8.6.7 Lap Lengths

The minimum lap lengths are those given in BS 8110 and are less conservative than the BS 5400 values.

The enhancement factors of 1.4 and 2.0 should be applied only to the calculated lap lengths of bars in tension. They should therefore not be applied to bars in compression nor to the minimum lap lengths specified in paragraph 1 of clause 5.8.6.7.

5.8.6.8 Hooks and Bends

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

5.8.6.9 Bearing Stress Inside Bends

The allowable bearing stress expression is based on tests by Soroushian[62] which show the BS 5400 requirement to be conservative. The maximum values of $l = 3l_i$ and $a_b/ø = 8$ represent the limits of the test evidence available.

5.8.7 Curtailment and Anchorage of Reinforcement

The BS 5400 requirements have been retained, with the exception that condition (b) has been made less conservative. The amended condition (b) is in agreement with test data [36,37]. The purpose of the BS 5400 conditions (a) to (c) is to guard against a reduction in shear strength at points of bar curtailment caused by the premature formation of flexural cracks at these points[38].

As an alternative to using the BS 5400 empirical curtailment rules, an assessment may be 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 [6,39,61,64].

5.8.8.1 Minimum Distance between Bars

Minimum bar spacings are specified in BS 5400 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 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

The main reason for limiting maximum bar spacings in design is to control crack widths. Assessment clause 5.8.8.2 is very similar to the BS 5400 clause but has been rearranged as a “crack width calculation” clause. The general spacing of 300mm and the limit for voided slabs in (d) of the BS 5400 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 recent research[34]. Similarly, a more appropriate tension stiffening formula has been added [40,41].

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

The BS 5400 clause requirements, which have been supplemented by BD 28 (DMRB 1.3),[63] are intended to control early thermal cracking. Such cracking is irrelevant to assessment because it occurs a few days after pouring. However, restrained members having less reinforcement that required by BD 28 (DMRB 1.3) are likely to exhibit wide early thermal cracks.

5.8.10 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. 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.

5.9.2 Durability (Lightweight Aggregate) See comment on 5.8.2.

5.9.4 Shear Resistance of Beams

BS 5400 applies a reduction factor of 0.8 to v_c for lightweight aggregate concrete. The higher values are consistent with test data[42].

5.9.10 Local Bond, Anchorage Bond and Laps

See comment on 5.8.6.2 regarding local bond. 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.[2]

6. ASSESSMENT: PRESTRESSED CONCRETE

6.1.1 Introduction

No references are made to Classes 1, 2 and 3 prestressed concrete since these Classes relate to criteria at the serviceability limit state. If an assessment under service load conditions is required, the relevant Organisation will specify the criteria (see clause 4.1.1 and its comment).

Grouted duct post-tensioning has exhibited tendon corrosion associated with the ingress of de-icing salts through inadequately grouted ducts. When inadequately grouted ducts or tendon corrosion are encountered in an assessment, the 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) 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 are provided, the re-anchorage length may be taken as the transmission length given in Cl 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 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 engineer the grouting is too poor to allow re-anchorage of tendons, the member should be treated as unbonded and assessed accordingly.
iv) In assessing the strength of a structure with corroded tendons, allowance should be made of possible future deterioration until the next Principal Inspection.

Further guidance in assessing structures with tendon corrosion is given in BA 51, The Assessment of Concrete Structures Affected by Steel Corrosion (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 relevant 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 using BD 44 (DMRB 3.4.14). If the grouting to the ducts is extremely poor, the tendons may have to be treated as unbonded.

When assessing a prestressed concrete bridge incorporating unbonded tendons, external tendons or lightweight aggregate concrete assessment criteria should be agreed with the relevant Overseeing Organisation. BD 58 (DMRB 1.3.9) and BA 58 (DMRB 1.3.10), The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing, may be used for the assessment of structures with external or unbonded tendons. Alternatively, with the agreement of the Overseeing Organisation, specialist literature on these topics may be used. The engineer is referred to: Reference 43 for unbonded tendons; Reference 44 for external tendons; and Reference 45 for lightweight aggregate concrete.

6.1.2.2 Durability

See comment on 5.1.2.2

6.1.4 Strength of Materials

See comment on 2.1.3.1

6.1.4.2 Strength of Concrete

See comment on 5.1.4.2.

6.1.4.3 Strength of Prestressing Tendons

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.

6.2.2 Redistribution of Moments

Item (2) of condition (a) is conservative for current U.K. tendons. However, it is not necessarily conservative for non-current U.K. tendons nor for non-U.K. tendons. For such tendons, information should be obtained on the tendon's ductility from past records. See also comment on 5.2.2.

6.3.2 Serviceability Limit State: Flexure

See comment on 4.1.1.

6.3.3.1 Section Analysis (Ultimate Limit State)

See comment on 5.3.2.1.

6.3.3.2 Design Charts

See comment on 5.3.2.2, and treat f_{pu} in the same way as f_y.

6.3.3.3 Assessment Formula

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

6.3.4.1 General (Shear Resistance of Beams)

The limit on the shear resistance is related to the area of longitudinal reinforcement in excess of that required to resist bending. See 5.3.3.2.

The assessment clause state 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 reinforced concrete clause (5.3.3) within the transmission zone, the area of bar tendons (e.g. Macalloy or Dividag bars) may be included in A_s, because their rigidity enables them to contribute to the shear resistance component due to dowel action.

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6.3.4.2 Sections Uncracked in Flexure

In view of the change in 6.3.4.1, which requires V_{cr} to be considered only for cracked sections, 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 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 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[46]. This factor has been reduced to 1.15 in the assessment code because variations in concrete quality are allowed for in γ_{mc}.

It should be noted that, for flanged beams, Equation 28 is an approximation[(6)]. For such beams it may be preferable to work from first principles.

6.3.4.3 Sections Cracked in Flexure

BS 5400 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[(47)].
Since no reference is made in the assessment code to classes of prestressed concrete the criterion for selecting Equation 29A or 30A has been made on the basis of the f_{pe}/f_{pu} ratio.

The BS 5400 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}.

Equations 29 and 30 have been further modified by including the d/2 term. This term was conservatively omitted from the BS 5400 equations[(47)].

Although it is illogical to ignore the vertical component of inclined tendons in cracked sections, there are insufficient test data to justify its inclusion.

6.3.4.4 Shear Reinforcement

Most of the comments on clause 5.3.3.2 are also applicable to 6.3.4.4. In particular the last paragraph of 5.3.3.2 regarding the absence and effectiveness of shear reinforcement is applicable to prestressed concrete as well as reinforced concrete.

The BS 5400 clause 6.3.4.4 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 requirement to relate shear capacity to the area of longitudinal reinforcement or prestressing steel has been moved to 6.3.4.1 (see first paragraph of the comment on 6.3.4.1).

The expression for V_s is not valid for a link spacing in excess of d_t. In such cases an analysis has to be preformed 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 and/or carry out non-linear analysis of the web.

6.3.4.5 Maximum Shear Force

Table 28 of BS 5400 has been replaced by an expression which gives very nearly the same values. See also the comment on 5.3.3.1.

6.3.5.2 Stresses and Reinforcement (Torsion)

The prestressing steel stresses given in BS 5400 are really stress increments(6). In the assessment code they have been replaced by total stresses, in line with the latest version of BS 5400: Part 4.

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

6.3.5.4 Other Assessment Methods

This clause permits the consideration of strength at various points rather than the consideration of cross-sectional strength. Possible methods for box girders are presented by Swan and Williams(48) and Maisel and Swan(49).

6.3.6 Longitudinal Shear

See comment on 5.3.5.

6.3.7 Deflection of Beams

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. The deflections of such beams may be determined using moment curvature relationships(50).

6.4 Slabs

See comment on 6.3.4.2 for the explanation of the f_t expression.

6.7.2.4 Loss of Prestress due to Shrinkage of the Concrete

The age factors of BS 5400 are not considered relevant to assessment.

6.7.2.5 Loss of Prestress due to Creep of the Concrete

The age factors of BS 5400 are not considered relevant to 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.

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6.7.4 Transmission Length in Pre-Tensioned Members

It is emphasised that data are not available on transmission lengths in weak concretes less than 28 N/mm^2. Hence, caution is advised if the concrete has deteriorated below 28 N/mm^2, even if the strength at transfer was in excess of this value.

6.7.5 End Blocks

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.

The restriction on the stress in anti-bursting reinforcement to that corresponding to a strain of 0.001 in BS 5400 is a serviceability criterion which has been omitted in the assessment code.

Detailed information on end block strength is given in Reference 52, whilst Reference 53 deals with end blocks in which the concrete is assumed to resist tension.

6.8.2.1 General (Cover to Prestressing Tendons)

See comment on 5.8.2.

6.8.3.3 Tendons in Ducts

The BS 5400 clause is not considered to be relevant to assessment except for curved ducts.

7. ASSESSMENT: PRECAST, COMPOSITE AND PLAIN CONCRETE CONSTRUCTION

7.1.2.3 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.

7.2.2 - Other Precast Members

The BS 5400 reference to precast components is not relevant to assessment.

7.2.3.1 - Concrete Corbels

Specialist literature[16,54] 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 to 1.0 in the assessment code. The latter value is within the range of applicability of the assessment method according to Somerville[16].

The BS 5400 minimum main steel percentage has been omitted since it is a serviceability requirement to prevent the rapid opening of cracks after initial cracking[54]. However, it should be noted that wide cracks are likely to occur if the main steel percentage is less than 0.4%.

BS 5400 requires the bearing area not to project beyond the straight portion of the main bars in order to prevent the cover concrete being sheared off. When assessing a corbel where such a projection does occur, the proportion of the bearing area which projects beyond the straight portion of the main bars should be ignored in all strength calculations.

The BS 5400 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 assessing Engineer should be aware that the absence of horizontal links could result in wide cracks and/or a brittle failure.

The BS 5400 requirement for a serviceability check is not considered relevant to assessment, unless specifically requested by the Overseeing Organisation.

7.2.3.3 Bearing Stresses

The allowable bearing stresses are higher than the BS 5400 values. However the bearing stress expression is that given in BS 8110 and is based on the latest CEB Formula[35].

The definition of A_{sup} has been modified to agree with the latest version of BS 5400: Part 4.

7.2.4.2 Halving Joint

The assessment clause dealing with half joints is more general than the BS 5400 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[55].

The BS 5400 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, The assessment of Reinforced Concrete Half-Joints, (DMRB 3.4.6) gives guidance on the assessment of half-joints at both the ultimate and serviceability limit states. As half-joints are prone to durability problems, it is advisable to check them at the serviceability limit state.

7.3.2.2 Sleeving

This clause has been abbreviated from BS 5400 clause, because it is envisaged that test data appropriate to the actual condition of use would be obtained.

7.3.2.3 Threading

See comment on 7.3.2.2.

7.3.2.4 Welding of Bars

The BS 5400 clause is assumed to refer to the locations of welded connections and is, thus, not relevant to assessment.

7.3.3 Other Types of Connections

The BS 5400 references to resin mortar and cement mortar joints are relevant only to construction and the serviceability limit state, and have, thus, been omitted from the assessment code.

They also preclude tension or shear transfer at a joint, whereas, generally, it is possible for such a joint to resist some tension and/or shear. Hence, it is suggested that the assessment of such joints should be based on relevant test data.

7.4.2.2 Vertical Shear

The BD 44 (DMRB 3.4.14) 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 currently being undertaken indicate 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

Fig. 8A is reproduced from the latest version of BS 5400.

The BS 5400 design values of k_1 implicity allow for a partial safety factor of 1.6. Hence, the assessment characteristic values are 1.6 times the BS 5400 design values, and the partial safety factor, γ_{ms}, is included in expression (a).

Test data[(56)] have shown that the BS 5400 design values of v_1 implicity 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. Hence, the assessment characteristic values have been obtained as follows.

<u>Monolithic</u>

Design values suggested by Hughes[(56)] have been incorporated in a slightly amended form in the latest version of BS 5400: Part 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.

<u>Surface type 1</u> BS 5400 values multiplied by 1.6.

<u>Surface Type 2</u> BS 5400 values multiplied by 1.25.

The partial safety factor for shear, γ_{mv}, (see 5.3.3.2) is applied to the characteristic v_1 values[56].

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 value of 0.7 when the BS 5400 value of γ_{ms} of 1.15 is applied.

The BS 5400 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[56]. Furthermore, in Reference 56 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 is also unclear. Hence, one should apply engineering judgement when considering this requirement and not automatically ignore reinforcement spaced at centres greater than the specified maximum. Some relevant data have been obtained in tests on PCDG T2 beams at TRRL. These beams behaved satisfactorily with links spaces at seven times the slab thickness, and there were indications that the 600mm limit on spacing was conservative.

7.4.3 Serviceability Limit State

If a serviceability limit state assessment is required by the relevant Overseeing Organisation, the following points should be considered:

a. There appears to be no test data to quantify the permissible enhancement in allowable compressive stress in 7.4.3.2 of BS 5400. However, for the case of a fully restrained flange, BS 8110 permits an enhancement of up to 50%[57].

b. The allowable flexural tensile stresses in table 32 of BS 5400 are very conservative and include a partial safety factor of about 2.5 on the lower bound to the experimental evidence[6]. For assessment, these stresses 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 should not be applied.

7.5.1 General (Plain Concrete Walls and Abutments)

BS 5400 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 engineer should consult clause 3.9.4 of BS 8110 which deals with unbraced and slender walls. In applying the latter clause for assessment purposes, the following modification should be made: replace the constant 0.3 in equations 43 to 46 of BS 8110 with ($0.675/\gamma_{mcw}$).

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7.5.5 Analysis of Section

The factor λ_w in BS 5400 and CP 110 has been replaced with a constant value of 0.3 in BS 8110. The latter value includes [57] an allowance for a partial safety factor of 2.25. Hence, in the assessment code, the factor 0.3 is replaced with (0.3 x 2.25/γ_{mcw}) where γ_{mcw} is either 2.25 (for use with the characteristic strength) or 2.25 x 1.2/1.5 = 1.80 (for use with the worst credible strength).

7.5.6 Shear

The background to this 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.

7.5.9 Shrinkage and Temperature Reinforcement

See comment on 5.8.9.

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