BA 61/96

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 Composite Highway Bridges and Structures

Summary: This Advice Note is intended to provide guidance of the assessment of existing composite highway bridges and structures.

REGISTRATION OF AMENDMENTS

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

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VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 17

BA 61/96

THE ASSESSMENT OF COMPOSITE HIGHWAY BRIDGES AND STRUCTURES

Contents

1. INTRODUCTION

General

1.1 This Advice Note provides guidance on the use of Standard BD 61 (DMRB 3.4.16), the Assessment of Composite Highway Bridges and Structures (referred to as 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 5: 1979, 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 Technical Approval Authority (TAA).

Scope

1.3 The Code of Practice for the design of composite bridges, BS 5400 Part 5: 1979 as implemented by BD 16 (DMRB 1.3) (hereafter referred to as the Design Code) is a design document. The Design Code makes certain assumptions and includes specific criteria that enable design to be simplified, since these aspects can be incorporated as required at the design stages. These include such considerations as restrictions on overall configurations and shape limitations to elements, assumptions for imperfections, tolerances and material properties and other simplifications that make the design approach reasonably simple and the structure acceptably efficient.

1.4 For these reasons where an existing bridge has to be assessed, the Design Code cannot be readily used. In some areas it may be unduly conservative, since more information can be ascertained for assessment than was assumed for design. In other areas it may be unsafe since some features in an existing older structure may be worse than assumed in new designs. Also for some forms of construction the Design Code will not be applicable, eg incidental shear connection.

1.5 Hence there is a need to supplement the Design Code with assessment amendments to cater for these aspects, that can be used as and when required. Any elements or sections of the structure that can be shown to safely comply with the Design Code and its inherent assumptions under the assessment loading can still be dealt with by the normal Design Code clauses.

1.6 Where new tentative methods, outside the scope of the design rules, are considered appropriate to assessment, these have generally been placed in the Advice Note. The Standard contains all other methods and this Advice Note should be used for further guidance. Methods of assessment that are either lengthy or will be rarely used are placed in Appendices A to C to Annex A of the Standard and Appendices A to I to Annex A of this Advice Note.

Implementation

1.7 This Advice Note should be used forthwith for all assessments, currently being prepared provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay progress. Its application to particular assessments should be confirmed with the Overseeing Organisation.

2. USE OF ANNEX A – THE COMMENTARY

2.1 The format of Annex A is based on the clause numbering used in Annex A of the Standard. To assist in identifying these clauses section headings and subheadings have been retained.

2.2 Comments are given on those clauses where the changes from BS 5400: Part 5: 1979 are substantial or are not self evident.

2.3 In the commentary there are many references to detailed aspects of BS 5400: Part 5: 1979. Therefore to gain the maximum benefit from the commentary a working knowledge of the Design Code is necessary.

2.4 A list of references is included in Appendix I 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 5: 1979 have been abbreviated to the Design Code and should be taken as references to the document as implemented by BD 16 (DMRB 1.3).

2.6 Definitions of symbols used in the commentary are given in Annex A of the Standard.

3. REFERENCES

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

Volume 1 Section 3 General Design

BD 9 Implementation of BS 5400: Part 10: 1980, Code of Practice for Fatigue (DMRB 1.3).

BD 13 Design of Steel Bridges. Use of BS 5400: Part 3: 1982. (DMRB 1.3).

BD 16 Design of Composite Bridges. Use of BS 5400: Part 5: 1979. (DMRB 1.3).

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

BD 37 Loads for Highway Bridges. (DMRB 1.3).

Volume 3 Section 4 Assessment

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

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

BD 56 The Assessment of Steel Highway Bridges and Structures. (DMRB 3.4.11).

BD 61 The Assessment of Composite Highway Bridges and Structures. (DMRB 3.4.16).

BA 44 The Assessment of Concrete Highway Bridges and Structures. (DMRB 3.4.15).

BA 56 The Assessment of Steel Highway Bridges and Structures. (DMRB 3.4.12).

2. British Standards (BS): BRITISH STANDARDS INSTITUTION

BS 5400: Part 3: 1982. Steel, Concrete and Composite Bridges. Code of Practice for Design of Steel Bridges. British Standards Institution, 1982.

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

BS 5400: Part 5: 1979. Steel, Concrete and Composite Bridges. Code of Practice for Design of Composite Bridges. British Standards Institution, 1979.

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

4. ENQUIRIES

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

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FOREWORD

This Advice Note gives guidance on procedures which satisfy the rules in Annex A of Standard BD 61[1] (hereafter referred to as the Standard) and contains two categories of alternative rules.

Category A These are simpler and more conservative rules which may sometimes be appropriate.

Category B These are less conservative rules which are new and not proven to the extent of giving results within the 'unsafe' tolerance of 5% (with respect to the safe tolerance required for design rules) adopted in formulating the assessment rules.

The inclusion of both sets of rules has implications both on the cost and of the technical accuracy of the assessment and when it is proposed to use the rules in Category A or B in assessment these should be listed in the Approval in Principle document (AIP hereafter). Sometimes when a structure has failed to comply with other procedures it will be advantageous to apply the rules in Category B. Unless these have been included in the AIP document the approval of the Technical Approval Authority (TAA hereafter) must then be obtained.

Some of the rules have been borrowed from other codes and have been modified.

Modifications have been made to these rules:

(i) to make the rules more compatible with BD 44[2] or BD 56[3];

(ii) to suit the special situations in bridges and older bridges in particular; or

(iii) to improve them, where improvements seem warranted.

For assessment of reinforced concrete bridges and for some steel bridges checks for many of the serviceability limit state (SLS hereafter) criteria are not essential to demonstrate the present adequacy of the bridge, but may be advisable to establish whether the bridge may be prone to higher than usual rates of deterioration and require more frequent maintenance.

Checks at SLS are more important for composite bridges than either steel or concrete bridges for reason given in **4.3.2**.

An important revision made is that γ_{f3} is now on the resistance side of the limit state expression throughout the Standard, so that the approach to the design of the reinforced concrete elements and the shear connectors is no longer the same as BD 44.

Methods of assessing filler beam bridges not complying with Chapter 8 of the Standard are included.

Reference to iron structures has been included wherever it is considered to be appropriate, and they are further considered in this Advice Note. Wrought iron and ductile iron have sufficient ductility to justify a plastic cross section analysis, but only wrought iron has sufficient ductility to satisfy the requirements of plastic global analysis. For cast iron and ductile iron no moment redistribution is permitted, other than the nominal allowance to correct for the inaccuracy of uncracked analysis. There are no iron structures known to have been designed compositely. Incidental strengthening of iron structures by metal devices is considered in Chapter 6 and by concrete or compacted fill in Chapter 8.

4 ASSESSMENT : GENERAL

4.1 Assessment Philosophy

4.1.1 General

When the statistical methods in Clause 4.3.3 of BD 56 enable the design strength to be derived directly from the test results without first deriving a characteristic value. To gain full benefit, BD 56 recommends at least 5 tests are included. However at least 3 samples should be used.

In estimating the predicted value, the material strength used should be the mean from the tests on the specimen or, when the material tests relate only to the batch of elements tested and not the individual specimens, the mean of those for the batch.

There are two statistical methods for deriving strengths in BD 56, one relating to tests on elements, which is an add-on clause to **4.3.3**, and one relating to tests on materials in Appendix H. The method of **4.3.3** is considered sufficient for the special requirements of composite construction.

4.3 Limit State Requirements

4.3.1 General

Comments made here relate only to the limiting criteria specified. Deflections are seldom by themselves critical, although

(a) they are taken into account in buildings because deflection of flat soffits above 20mm or span/500 is visible and may cause cracking of brittle finishes whereas L/350 deflections may cause cracking of less brittle finishes.[5] Neither of these criteria are routinely considered in bridge design and neither are appropriate to assessment except in unusual circumstances;

(b) a check on deflection provides an implicit guide to vibration, which in the Standard is more accurately taken into account by specific criteria;

(c) when the headroom below is insufficient the deflection may exacerbate the difficulties.

Deflection calculations however are needed for comparison with deflection measurements and it is mainly for this reason that the rules have been developed; more accurate procedures are available.[6].

4.3.2 Serviceability Limit State

Checks at the serviceability limit state (hereafter SLS) are more important for composite bridges than either steel or concrete bridges for the following reasons:

(i) being lighter and more flexible than concrete bridges and normally of shorter span than steel bridges they are more prone to vibration problems than either concrete or steel bridges;

(ii) the most common type shear connectors, studs, exhibit unusual fatigue behaviour as discussed in **5.3.2.1**, which needs to be taken into account, albeit approximately;

(iii) compact cross sections sometimes have very high shape factors, which have the effect of increasing the criticality of the performance at SLS;

(iv) the SLS condition relative to the ultimate limit state (hereafter ULS) conditions is more critical in assessment than in design due to the less conservative assumptions at ULS;

(v) moment redistribution is now permitted at ULS but not at SLS which further increases the criticality of the SLS condition.

(a) The shear check for structural steel at SLS is not necessary for sections assessed elastically at ULS in which no moments have been redistributed. For other situations the SLS condition tends to be more critical than in initial design (for the reason given above) and so needs to be checked.

An important relaxation in the rules, which is not appropriate to design, is that the stress check at SLS disregards both lateral torsional and local buckling. For composite beams it is considered to be adequately taken into account at ULS.

(b) There is no firm evidence to differentiate between the stress limit in the reinforcement of $0.75f_{ry}$ in BS 5400: Part 4[7] and that of $0.80f_{ry}$ in BS 8110 and for assessment the higher of the two values has been adopted. It should be noted however that the limit in BS 8110 only relates to the validity of the expression for the determination of crack widths, whereas in BS 5400: Part 4 it also applies to consideration of fatigue. There are stress limits in BD 44 relating to fatigue in reinforcement which are very restrictive in relation to unpropped composite beams as the stress in the reinforcement is mainly due to live loads. It is considered the relaxation is justifiable here because there is no evidence of fatigue failures in the reinforcement of composite bridges. The fatigue in the reinforcement of composite beams is considered in Appendix E.

(c) The limiting crack width has been set at 0.4mm for the slab of a composite beam, which compares with the limit of 0.25mm for reinforced concrete construction in a non-aggressive environment in BS 5400 Part 4. The higher limit in the assessment of composite beams is justified because:

(i) In studies on the relationship between crack widths and corrosion in reinforced concrete beams the conclusions indicated that corrosion was mainly determined by the quality of the concrete, which was significantly more important even than the cover. No relationship between corrosion and crack width could be made over a wide range of crack widths and the limit of 0.4mm is not close to the range of crack widths at which corrosion might be expected to increase. These conclusions would appear to be unaffected were these findings to be expressed in terms of the crack angle (crack width/cover to longitudinal steel), which is now recognised as more relevant to corrosion than the crack width. Beeby[8] has investigated what aspect of the concrete quality was most important; he found there was little to choose between the water/cement ratio and the strength, both of which were of rather greater importance than the cement content (better correlation still would be expected with permeability or porosity determined by mercury vapour diffusion).

(ii) It is generally easier to satisfy crack width criteria in reinforced concrete bridge beams and slabs than in composite beams since most of the flexural resistance is provided by large reinforcing bars, mainly within the width of the rib (and mainly confined by stirrup reinforcement). The large total bar perimeter and excess area of reinforcement over the minimum required for crack control and the appreciable amount of reinforcement (provided for other reasons) in the slab enables small crack widths to be achieved at little cost to the design. In contrast in composite beams the crack width is the determining factor in selecting the longitudinal slab reinforcement at support locations in many designs, so there is an incentive to ensure that the criteria are not unduly conservative. The choice of a less conservative limiting crack width is also influenced by the fact that a composite beam is not wholly reliant upon the strength of the reinforcement (as in a reinforced concrete beam) and that about half the reinforcement is in the bottom of the slab, so slight corrosion to the top reinforcement would not endanger the structure (which is generally protected by waterproofing).

(d) Limiting criteria for vibration are given in Ref 9, as modified by Appendix D.

(e) The limit to the load on the shear connector at SLS, of 55% of the nominal static strength, was specified with a view to limiting the slip under the passage of heavy vehicles. For the majority of minor roads, when the risk of fatigue failure of the shear connectors is low, some increase is clearly warranted. In the new method the shear connector strength depends upon the fatigue history, unlike the method in the Design Code in which there is no such dependence. For this reason no checks are required on the slip, though it should be noted that the limiting criterion specified in this Clause was adopted when selecting the connector stiffness used in developing some of the deemed-to-satisfy criteria in the Standard.

5 ASSESSMENT OF THE SUPERSTRUCTURE FOR THE SERVICEABILITY LIMIT STATE

5.1 Analysis of Structure

5.1.1 Distribution of bending moments and vertical shear forces.

5.1.1.1 General

The requirements of **5.1.1** are quite different from those in the Design Code but the clause structure has been retained as far as possible. As a result **5.1.1.1** contains both the general statements and the simplified method of designing continuous composite beams. The simplified method could be regarded as the general method, since it is appropriate to simply supported and continuous beams (see below). It is deemed to be a Category A method for the purposes of this Advice Note.

For simply supported spans the global analysis is governed by considerations of equilibrium, so bending moments and vertical shear forces are independent of whether or not shear lag is taken into account.

For continuous construction a simplified procedure is given in **5.1.1.1** whereby the concrete at supports is assumed to be uncracked and shear lag is neglected. This procedure however overestimates the support moments, and aspects of the construction failing the assessment criteria must be repeated using the more accurate method of analysis in **5.1.1.2(a)**.

The simplified procedure in **5.1.1.1** underestimates the mid-span moments and this is taken into account by the correction in **5.1.1.2(b)**. It should be noted however that in spans of substantially uniform depth, the mid-span region under nominal assessment live loading is seldom critical and the construction can generally be assumed to have sufficient capacity for negative moment redistribution (moment redistributed from the support to the mid-span) to compensate for the inaccuracy in the analysis.

The degree of inaccuracy of the representation of the method in **5.1.1.1** for continuous beams depends on the situation. It may accurately represent the situation in prestressed composite bridges but it poorly represents the situation in some older bridges in which there may be little or no reinforcement in the slab. It is also inappropriate where there is a low grade fill with some compression resistance, which may sometimes be taken into account (see Chapter 8). The method is appropriate to the rare circumstances in which the determination of a reliable estimate of the effective breadth is impossible.

5.1.1.2 Continuous Beams

The check in the Design Code to determine whether or not the cross section is cracked invariably shows that the cross section is cracked. Whilst accepting that there may be situations, in lightly loaded short span beams and in composite cross members in longer span bridges, where the concrete is uncracked, for purposes of assessment the cracked condition may generally be assumed. The use of the cracked section properties should therefore be regarded as a feature of the assessment method. Frequently no cracks will be observable, but their absence is not considered sufficient justification for the assumption of uncracked conditions.

The use of the same effective breadth in both the global and cross section analysis will give a more economic assessment of the support cross section than the procedure in the Design Code. The procedure in the Design Code has been justified, to the required degree of accuracy for the Standard, by finite element studies. However for b/l ratios of 0.125 or less the designer has the option of using the full breadth for the global analysis. Intermediate effective breadths between the theoretical and unity give a more economical solution for b/l ratios less than 0.125, so the assessor may choose to group cross sections in a beam (for this purpose a continuous beam over many spans may be considered to be a single beam). At greater b/l ratios the assumption of the full breath in the global analysis is considered too conservative for assessment generally, but there is no objection to its adoption on theoretical grounds. For example it might be appropriate in short span structures (say of 12m span or less). In each span variation is only required between the support and mid-span cross section.

It should be noted that the assumption of a mean cracked length of 15% gives a good indication of the distribution of moments for cracked lengths of between 10-25% of the span and is virtually of general applicability.

The method in **5.1.1.2(b)** differs from the general method in **5.1.1.1** in that shear lag is taken into account.

5.2 Analysis of Sections

5.2.1 General

See also comments under **5.1.1.2**.

Assumptions regarding the method of construction are now considered; these affect **5.2.5.2** to **5.2.5.4** and also the global analysis.

It is necessary for the engineer to assess the worst credible construction sequence, which should take into account design and construction procedures current when the bridge was designed and constructed. Occasionally the special nature of the construction may suggest the use of special procedures and these should be taken into account where appropriate.

For simply supported composite beams the greatest flexural resistance is obtained with propped construction, whereby the beam is propped until the slab has hardened. For reasons unrelated to composite action the size of the steel beam is generally larger than the minimum achievable with propped construction and normally sufficient resistance is obtained by using unpropped construction, when the slab is cast on the beam in the unpropped condition. When the method of construction is unknown it is required to make safe assumptions, but which do not result in the structure being overstressed. Unfortunately, no single set of assumptions can be made which can be regarded as generally safe.

If propped construction is assumed when unpropped construction was used, it is likely that the shear connection and possibly the slab will be found to be overstressed, whereas the steel section will have a large reserve capacity.

If unpropped construction is assumed when propped construction was used, it is likely that the shear connection will be found to have a large reserve capacity but the steel section will be overstressed.

Where it is suspected this situation has been encountered on completing the assessment the structure should be re-examined for evidence of the sequence of construction and possible signs of distress in areas which the calculations suggest have been overstressed. The subsequent procedure shall then be agreed with the TAA.

However it is noted that the usual assumptions in composite bridge design in 1963[11] were that the steel carried the weight of the steel section and the weight of the wet concrete, and the composite section carried the applied load (which included the surfacing). This corresponds to present day practice except in regards to the practice (which was common by 1970) of concreting the part of slab at mid span before that at the support.

It is suggested therefore that for bridges constructed since 1960 the unpropped condition should be assumed, but that the casting of the support section last should only be assumed when it can be justified. Otherwise it is advisable to make the assumption that the slab over the entire length of the beam was placed in one pour. The worst case scenario, that the support sections were cast first, is considered unjustifiable.

For internal beams of spans up to at least 40m the stresses in the concrete from live load sufficiently exceed those from superimposed dead load that the short term concrete properties may be assumed throughout. In other situations the engineer should make a sensible assessment in differentiating between short term and long term cross section properties, taking into account the lesser sensitivity of the global analysis to the precise properties assumed than in the cross section analysis, which reduces as the ratio of the area of concrete to that of the steel beam increases.

In deriving the cross section properties of the cracked cross section it should be noted that the elastic modulus of reinforcement in BS 5400 Part 4 is 200,000 N/mm^2 and the elastic modulus for steel in BS 5400 Part 3 is 205,000 N/mm^2, consequently the modular ratio of the reinforcement should be taken as 1.025.

5.2.2 Analysis

Vertical shear was required to be assessed at SLS in the Design Code presumably in order to satisfy the requirements of Clause 4.2.2 of BD 56. However for steel sections the rules in BD 56 impose no restrictions on the webs at SLS, so the mention of a check on vertical shear in the Design Code is omitted from the assessment.

5.2.3 Effective breadth of concrete flange

5.2.3.1 General

Finite element studies on composite beams have indicated that shear lag may be neglected, within the accuracy required for assessment, for b/l ratios of 0.05 or less.

5.2.3.2 Effective cracked flange

The effective cracked flange factor is a new term, but the effective breadth obtained is the same as given by the method in the Design Code. The use of an effective breadth for cracked sections wider than that for uncracked sections is attributable to the shear stiffness of a cracked slab being greater than the longitudinal stiffness[12]. It is moreover supported by evidence from tests.

5.2.3.3 Width over which slab reinforcement is effective

The common situation in skew bridges, in which the slab reinforcement is not parallel to the beam, is now taken into account. Theoretically for strength calculations (as in this paragraph) the angle in the expression should be squared, but reflecting the lack of research on flexural resistances of composite beams and reinforced concrete beams with the slab reinforcement so arranged, and the need therefore to be conservative, the angle has been taken to the power of four. This has the benefit to the assessor that the same power is used when calculating the cross section properties for determining the crack width (when it is theoretically correct), so that the same cross section properties may sometimes be used for purposes of both strength and crack width control calculations. It is noted that in BS 5400 Part 4 non-parallel reinforcement is taken into account in the calculation of crack widths, but not of flexural resistance.

This is because in reinforced concrete T-beams most of the main top tensile reinforcement is contained within the stirrups and therefore must lie within the width of the web, and so be parallel to the beam.

5.2.4 Deck slab forming flanges of composite beams

5.2.4.3 Co-existent stresses

Co-existent stresses in the deck slab need to be taken into account particularly in bridges with skew. Appendix A of BD 56 strictly applies only to the steel flange of box girders.

5.2.5.4 Slab casting sequence

See comments under **5.2.1** and **12.1**.

5.2.5 Steel section

5.2.5.1 Propped construction

In propped construction it is likely that the beams were propped at discrete locations rather than continuously, so that the beam would have spanned between the props. The propped state may be disregarded for the purposes of assessment, unless it is evident that excessive prop settlement has occurred. To do otherwise would be often impractical as details of the propping may be unavailable, and further most composite beams, even non-compact ones, have some capacity for stress redistribution within the cross-section. Even if they do not it is most unlikely that the stresses would be sufficiently high to invalidate any aspect of the assessment. See also comments under **5.2.1**.

5.2.6 Assessment of cracking in concrete

Reinforcement in the slab controls the development of cracking resulting from differential thermal strains during setting and subsequently, shrinkage on the loss of moisture and flexural strains. Whilst the cracking itself does not invalidate the strength assessment it may:

(a) affect the susceptibility of the reinforcement to corrosion and also, when the crack passes through the full depth of the slab (which is likely only in the deeper beams);

(b) when cracking passes through the full depth of the slab it is accommodated by slip of the shear connectors which, although taken into account in the methods of analysis, does not otherwise necessarily occur (see **5.3.2.1(b)(ii)**).

When the slab is protected by a waterproof membrane, water ingress cannot occur, so depending on the quality of the waterproofing and its maintenance cracking may be unimportant. The concrete in edge beams however may be more exposed and so too is the soffit, particularly for wide spacings of main beams.

If there is evidence of significant cracking or corrosion, stress and crack width calculations may help in diagnosing the cause, and the likely crack width should be assessed where a significant increase in the loading is likely as a result of new industry or re-routing of traffic.

5.3 Longitudinal Shear

5.3.1 General

The distribution of vertical shear from which the longitudinal shear force is calculated is affected less by the global analysis than the distribution of bending moments and so a global analysis using cracked section properties at supports may be used.

Studies on continuous beams of different spans, with medium and relatively high amounts of slab reinforcement and short and long term concrete properties, indicated that global analyses employing the cracked cross section properties required significantly fewer shear connectors at the supports and in total than analyses employing uncracked cross section properties throughout. Taking into account the effect of tension stiffening in the concrete (see Appendix A), in both the global analysis and cross section check, cracked analysis is considered to be potentially unsafe(13). Consequently the method in the Design Code is retained, with a modification to the effective breadth in beams simply supported one end and fixed or continuous the other, when the analyses suggested that the method of the Design Code overestimated the effective breadth.

Due to the doubt over the assessment of the shrinkage effects expressed later, and the uncertainty as to whether differential settlement will occur, these effects are only included in the load combinations which includes temperature.

Attention is drawn to the fact that the correct equation for determining longitudinal shear is

$$q = \frac{d}{dx}\left(\frac{MAy}{I}\right) = \frac{VAy}{I} + M\frac{d}{dx}\left(\frac{Ay}{I}\right)$$

with the second term being zero for beams of uniform section. The second term can increase (eg, haunched sections at supports) or decrease (eg, on ‘fish-bellied’ girders) the value of q.

Where the section changes suddenly (eg, at a splice), this can usually be ignored if the changes in neutral axis level or second moment of area are small(24), noting that an increase on I due to the steel section changing is usually accompanied by an increase in y. If the change in longitudinal force ΔQ is significant, then ½ ΔQ may be applied each side of the change in section over a length of l_{ss} (see **5.4.2.3**).

5.3.2 Shear Connectors

5.3.2.1 Nominal strength of shear connections embedded in normal density concrete

(a) Nominal initial mean static strength

The term ‘nominal’ to describe the strength is intended to indicate that these strengths are not necessarily related to routine testing. They are nevertheless considered to accurately represent the strength of the shear connectors.

The nominal strengths in the Design Code are the mean values from tests, not characteristic values as sometimes assumed, and this is now clear in the terminology. More recent research using improved test specimens and reanalysis of earlier research has resulted in an increase in the predicted strength of stud shear connectors of about 10%(4)(14), which assumes the weld collar having a height of 0.28d(15). The significance of this feature was not appreciated until 1981 and, as the current programme of bridge assessment extends only to structures built before the end of 1985, it is considered inevitable that many shear connectors will have lower weld collars. In recognition of the impracticality of inspecting more than a small sample of shear connectors on any bridge, this increase has not been taken into account in table 5.1. The strengths in table 5.1 have however been adjusted to reflect the variation in strengths suggested by the more recent work.

Furthermore, recent research[16] has established that longitudinal cracks weaken the shear connectors, so the use of strengths somewhat lower than the values now being used for design makes a nominal adjustment for this effect. In the very occasional case of severe cracking, or construction joints being encountered on the line of shear connectors, the reduction in strength indicated in Ref 16 should be taken into account.

Channels and bar shear connectors have not been subject to the same intensity of research as stud shear connectors and adjustments to these values have been made solely on the basis of the strengths of studs.

(b) Nominal present mean static strength

The nominal present mean static strength is the nominal initial mean static strength reduced to take account of the progressive damage suffered by the shear connection since the bridge was constructed[17][18].

Shear connectors exhibit the unusual characteristic that the static strength is reduced in proportion to the equivalent number of fatigue vehicles which have passed. Other materials suffer similar degradation[19], but in general materials can be subject to a significant and sometimes very large number of fatigue cycles before the degradation commences.

Combining Oehlers' recommendations with those of BS 5400 Part 10 one obtains

$$P_{am} = P_{im}\left[1 - \frac{1}{1320}\sum_{i=1}^{n}(P_{ri}/P_{im})^{m}\right] \quad \text{when } N_a \leq 10^7 \qquad (5.2a)$$

Where

n is the total number of stress cycles which may be of varying amplitude (this is the 'n' in BS 5400 Part 10). Due to the high power of m it is permissable in practical situations to take n as N_a, the equivalent number of SFV's.

P_{ri} is the range of the shear connector force for the i^{th} reservoir.

This method is found to be conservative compared to the present method for taking into account fatigue in shear connectors in BS 5400 Part 10. Taking into account a nominal increase in strength to reflect the absence of stud shear connector failures two modifications have been introduced. One is a correction factor k and the other the correction for low stress fatigue from BS 5400 Part 10.

The following expression is proposed for the usual circumstances in which P_{ri}/kP_{im} is less than 0.133.

$$P_{am} = P_{im}\left[1 - \frac{1}{23.3}\sum_{i=1}^{n}(P_{ri}/kP_{im})^{m+2}\right] \qquad (5.2b)$$

Where

n and P_{ri} are as equation 5.2(a),

k is considered below.

If P_{ri}/kP_{im} is greater than 0.133, equation 5.2(a) should be used but with P_{ri}/kP_{im} in place of P_{ri}/P_{im}.

Including only the peak to peak reservoirs the expression 5.2(b) simplifies to expression 5.2 of the Standard.

Whilst it has not been shown that the above approach applies to types of shear connectors other than welded studs, it has been extended to other types of shear connectors on the assumption that it is applicable to shear connectors part of which are flexible and part rigid and to rigid types on the assumption that their performance in fatigue is at least equivalent to that of the studs. Where the shear connectors are not welded, methods relating to the procedures in BS 5400 Part 10[20] are acceptable as an alternative.

There is however as yet no evidence, despite unforeseen increases in HGV loading, that any fatigue failures of shear connectors have occurred in bridges, either for road or rail traffic. The new fatigue criterion will generally be found to be less restrictive than the rules in the Design Code for most composite bridges as they are a relatively new form of construction, there being few composite bridges constructed in the UK before 1950. The clause however may be found to be more conservative than the rules in the Design Code which are nearer the end of their fatigue design life than any are believed to be at present.

Should a structure fail the assessment criterion the measures proposed are:-

(i) conduct an inspection to determine if the bond has been broken.

(ii) when the bond has not been broken, a situation considered to exist in the majority of composite bridges (but see **5.2.6(b)**), and subject to TAA approval, P_{ri} may be reduced by a factor of 1.15 for stud shear connectors and 1.25 for other connectors[18]. This is the factor k in Equation 5.2 of the Standard.

(iii) take into account other forms of incidental shear connection or other incidental strengthening as may be appropriate to the bridge.

(iv) when the bond has been broken, friction may be used in assessing the effect of imposed load on shear connectors, and a coefficient of friction of 0.40 is suggested.

(v) review the latest literature, because this is a subject on which research can be expected to continue, as the cumulative damage law for shear connectors of equation 5.2 differs from that for most other materials, for which the resistance reduces significantly only towards the end of the design life.

A possible rule for calculating N_a is that the number of years in the following periods should be multiplied by[21].

1925 to 1950	by 1/5
1950 to 1965	by 1/3
1965 to 1975	by 1/2
1975 to 1990	by 2/3

Due to the high power of m (5.1) it is not necessary to consider all reservoirs. For a fatigue reservoir of a depth of 40% of the peak to peak range the effect for an equivalent number of occurrences is only 2 - 3% of the peak to peak values. It is therefore permissible to neglect fatigue reservoirs more shallow than this depth.

The new approach would, if so required, enable the remaining life of the structure to be determined.

For assessment, checks on the fatigue endurance of shear connectors in accordance with BS5400 Part 10 are not required.

(c) Strength of connectors not included in table 5.1

See comments in **5.3.2.4**.

(d) Strength of connectors not complying with 5.3.3.3

Shear connectors not satisfying **5.3.3.3** are essentially of two types:-

(i) there are connectors similar in form to complying connectors, except that they lack the capacity to resist uplift, which were assumed in the design to be shear connectors;

(ii) there are shallow devices projecting into the slab from the beam which were not considered to be shear connectors in the original design.

In the Standard the two types are considered by the same approach (see **5.3.3.8**), except that with the very shallow connectors larger safety factors are required and that without other devices to resist the uplift they offer little shear resistance at ULS.

5.3.2.4 Tests on shear connectors

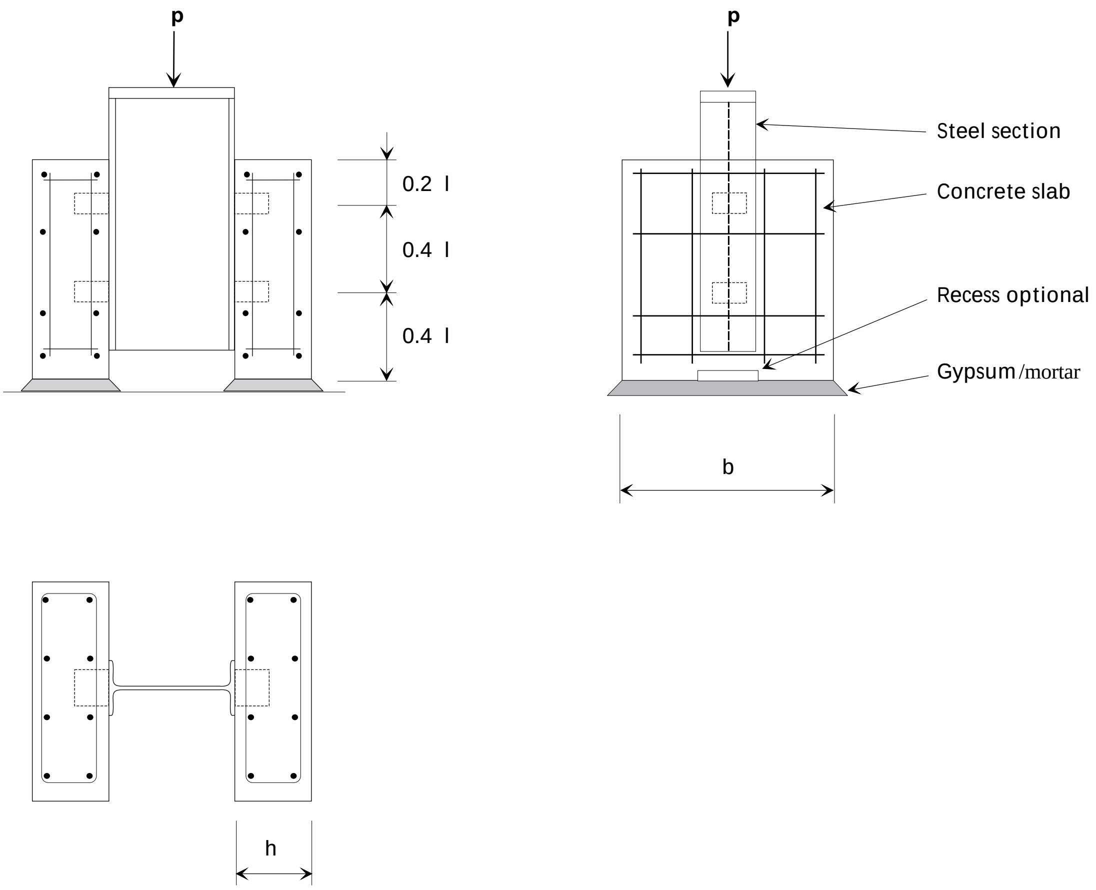
(a) Nominal initial mean static strengths

When tests are performed the initial mean static strength is taken as the lowest test result of three specimens or the average of a group of five or more. Clearly five will generally be preferable when new testing is undertaken, but existing test data is likely to relate to only three specimens.

(b) Details of tests

The standard push test in the Design Code is reproduced in the Standard. Larger test specimens are now specified in BS 5950 Part 3.1 and in Eurocode 4, Part 1.

Eurocode 4 includes a push test which is suitable for shear connectors lacking uplift capacity and this is included here in figure 5.6. It is not included in the Standard as it may need to be modified for use with shallow incidental shear connectors. The type of modification which may be necessary is in the form of additional horizontal restraints. Until such time as such modifications have been incorporated in the method the intended use of the method shall be included in the AIP document together with any preliminary tests proposed, or they shall be subsequently agreed with the TAA.



Notes

(1) Reinforcement as in beams assessed.

(2) Steel section to have flanges representative of those to which the shear connectors are attached in the beam.

(3) $l = s/0.4$ where s = connector spacing.

(4) b = width of connector plus 500mm, or any dimension up to the minimum effective breadth.

(5) h = greater of 150mm or height of connector plus 50mm or any dimension up to the minimum depth of the slab.

(6) Where there is a haunch this should be represented.

Figure 5.6 Test specimen for specific push test

5.3.3 Assessment of shear connection

5.3.3.5 Assessment resistance of shear connectors

The design longitudinal shear resistance of a connector P_s, disregarding γ_{f3}, is

$$P_{am}/\gamma_{slip}$$

where γ_{slip} is a reduction factor to ensure that slip is not excessive, which in **4.1.2** is assigned a value of 1.375.

The maximum value P_{am} permitted by equation 5.2 is $0.82P_{im}$, so that the maximum value of P_s is $0.82P_{im}/1.375$ or $0.60P_{im}$, in which 0.60 is the figure in **4.3.2(e)**. It only applies when the risk from fatigue is low, as when the fatigue risk is high the first criterion in equation 5.2 gives a lower value of P_{am}.

5.3.3.6 Shear connector spacing and longitudinal shear resistance

For the purposes of assessment the rules for calculating the connector forces have been developed to cover any spacing of shear connector groups as follows[22]:-

(1) not greater than 1000mm nor span/20
(2) exceeding 1000mm but less than span/8
(3) exceeding span/8

The VAy/I method is used for (1) and may be used for (2), but a more accurate result is obtained for (2) using the approach required for (3).

(1) is the normal situation for which, in the Design Code, q/q_r is 1.0 at the end of a span and 1.10 elsewhere, with the maximum values maintained over 10% of the span. For assessment these restrictions have been relaxed by taking advantage of the flexibility of shear connectors and the facts that at internal supports (see figure 5.7):

(i) where the spans are similar the shear connectors under dead load are undeflected (due to symmetry) and so unloaded, and;

(ii) the horizontal forces in the shear connectors at the ends are redistributed as a result of the shear deflection of the steel beam.

The values of q/q_r take into account variation in the ratios of adjacent span lengths up to 1.50, with a reasonable allowance for distributed live load effects, but not concentrated loads. If account has been taken of cracking the values in table 5.2 would be appreciably greater.

Table 5.2 was determined from finite element analyses of bridges with spans between 10 and 25 metres, and Class a shear connectors were assumed to be studs with a stiffness of 200kN/mm[13][23] whereas Class b and c connectors were assumed to be rigid. The stiffness of the shear connection increases as the number of shear connectors increases, which reduces the redistribution of longitudinal shear along the span in continuous beams. In simply supported beams the redistribution due to flexibility is small and may be adverse, but there will be some benefit from shrinkage, so there is a small net benefit. Class a shear connectors allow up to 20 number 19mm studs per metre, which have a stiffness per metre run of 4000kN/mm (or 4000MN/m). In fact the benefit from shear connector flexibility reduces only slowly with increase in stiffness and if the stiffnesses specified for class a connectors were doubled, q/q_r values would be mid way between those for class a and b shear connectors in table 5.2.

Shear connectors in beams in which the steel beam to slab depth ratio is less than 2.2 are excluded from class a due to the increased tendency of large tensile forces developing in the shear connectors with reduction in the depth of the steel beam from end effects in simply supported beams and from shear deflection in continuous beams.

For (2) q/q_r must not exceed unity and the shear connector force needs to be estimated for the distribution of the shear flow mid way between adjacent groups of shear connectors such that the force in the ith shear connector is $P_{si} \geq \Sigma q_{ri}\, d_x$, as shown in figure 5.8.

For connector spacings exceeding span/8 the method in (2) may not provide an accurate estimate of the connector forces for concentrated loads (though it has been shown to give a good indication of the shear connector forces under uniform loading up to a spacing exceeding span/4). For this situation an analysis is required in which the steel beam and concrete slab are modelled as separate members connected rigidly at the position of the shear connectors. For beams of normal span there is little advantage with such connector spacings of including the effect of connector flexibility. Furthermore for most situations in which these conditions apply the connectors are likely to be rigid (see **5.3.3.8**).

5.3.3.7 Uplift on shear connectors

In deriving the uplift forces on the shear connectors in unstiffened girders from differential bending of adjacent girders it is acceptable to disregard the stiffness of the web, so that the slab/beam connection is assumed to be pinned in the transverse direction. With stiffened webs however, particularly in the deeper beams, the effect of the rigidity of the connection may be significant and therefore should be evaluated.

Shear connectors may also subject to significant direct tension due to slab tie-down forces, particularly severe in the corners of some skew slabs. Another common situation which causes uplift in shear connectors occurs where there are composite cross girders which do not extend outside the external main girders, ie, the cantilever is a concrete slab. Parapet impact combined with accidental wheel loading can cause significant uplift in shear connectors on the crossbeam near its junction with the main beam.

5.3.3.8 Incidental shear connection

The method for assessing the strength of incidental shear connectors is derived from the method for bar connectors in Eurocode 4. The method gives appreciably lower strengths than those in table 5.1 for bar connectors and an additional safety factor γ_b introduced for incidental shear connectors which has the values:

1.25 for bolt heads, plate ends and other vertical surfaces
2.00 for rivet heads

These values have been derived from an assessment of the relative efficiency of these devices compared to shear connectors satisfying the requirements of **5.3.3.7**.

Attention is drawn to the increased stresses in many bolt groups resulting from their action as shear connectors, and frequently this action would appear to increase the shear in half the bolts and reduce it in the other half (compare figures 5.9a and 5.9b). Possible actions then are:

(i) assess the spare capacity in the bolts and take this as the limiting capacity of the group of incidental shear connectors;

(ii) assess the bolts taking into account the extra force from the incidental shear connection;

(iii) disregard the incidental action on the grounds that bolt failure of this kind has never been attributed to incidental action;

(iv) take full advantage of the incidental shear connection and ignore its possible adverse effects for the reasons given under (iii).

The procedure adopted should be included in the AIP document or otherwise agreed with the TAA.

Consideration has been given to specifying an upper limit to the increase in the resistance permissible with incidental shear connections. For tall devices, which may be effective as shear connectors but for their lack of capacity to resist uplift forces, and which were probably designed as shear connectors, no upper limit is imposed. Provisional limitations to the increase in the capacity of the steel beam permissible for bolts and other vertical bearing surfaces of 40%, and for rivet heads of 33%, are to be used until more accurate values can be justified.

For tall devices formed of plate which are clearly not rigid it is proposed that P_{im} given by equation 5.6 is multiplied by a factor k which is the ratio of the area of plate which under the design pressure (P_{am}/A_1) deflects by no more than 0.2mm to the total area of the plate.

In checks assuming devices act as incidental shear connectors unpropped construction is appropriate, so these devices carry no longitudinal shear under the self weight of the structure. They may however be assumed to be effective in carrying longitudinal shear from the superimposed dead load.

(a) Provision of devices to resist uplift

In riveted or bolted construction, composite action can be ensured by replacing some of the rivets or bolts by long bolts[25] or other suitable devices, as shown in figure 5.10. This would provide a nominal uplift capacity at positions where the analysis showed this to be most effective. Each scheme needs to be assessed on its merits, taking into account that the passage of a concentrated load will tend to cause uplift in the shear connectors at all positions. The frequency of the position of restraint to uplift therefore must ensure:

(i) the capacity of the slab to carry the load to where the uplift is resisted;

(ii) that significant vertical separation between the slab and the beam cannot occur.

For this purpose a maximum separation of 0.5mm disregarding the loading on the slab is suggested.

Where the whole of the deck is replaced, the replacement of rivets or bolts can be done on a more systematic basis.

For heavily trafficked bridges there is advantage to be gained by providing devices to resist uplift which do not themselves carry longitudinal shear, since the longitudinal shear significantly reduces their capacity to resist uplift (see **5.3.3.7**). Fewer devices would then be needed.

Where the girders are of cast or wrought iron, careful consideration should be given to the selection of connectors; there may be reasons for preferring mechanical fixings (as in figure 5.10) to welded fixings in this situation.

(b) Buckle plates and troughs

The undulation of the concrete surface in contact with the steelwork in construction using buckle plates and troughs constitutes a potential shear connection. The connectors between these features and the steel beam should be capable of carrying the longitudinal shear force calculated assuming full interaction.

5.3.3.8.3 Testing

When appreciable dependence is being placed upon the strength of the incidental shear connection (here defined as increasing the flexural strength of beam by 25% or more) field tests are required to demonstrate the existence of flexural stiffening effects. It may be assumed for the purpose of interpreting the results that 50% of the increase in flexural stiffness is attributable to increase in strength, or 75% when 75% or more of composite action is confirmed by strain profiles[46] (See also **5.5.3**).

Whilst accidental composite action may often appear to have withstood the test of time it may reduce with time due to slip or corrosion. Therefore, when compliance with the assessment criteria is dependent on accidental shear connectors, the bridge should be tested periodically under identical live loading to determine whether or not the stiffness of the bridge is diminishing. Where such a testing programme is adopted and finishes, furniture and parapets are replaced, tests should be conducted before and after the replacement.

5.3.3.9 Partial interaction

For short span beams with Class a connectors two effects combine which improve the overall efficiency of composite beams.

(i) the deflection of flexible shear connectors not only redistributes but also significantly reduces the longitudinal force, but the reduction in the moment of resistance of the composite section in bridges of normal span designed for full interaction (as in the Design Code) is too small to justify consideration;

(ii) continuous beams with elastic shear connectors possess the ability to redistribute the longitudinal shear force along the shear connection in such a way as to reduce the maximum shear flow in bridges of normal spans designed for full interaction (as in the Standard)[13][26]. The amount of redistribution is significant even when the shear connectors remain elastic, but in continuous short span beams without point loads a substantially uniform distribution of shear flow occurs, and a virtually uniform distribution is obtained with minimal inelasticity in the shear connection.

The procedure adopted is to assess the moments and forces using the short term modulus of elasticity and then to reduce the modulus of elasticity such that the force in the slab is not less than “p” times the value obtained using the short term modulus. The entire cross section is then checked for this slab stiffness.

5.3.3.10 Modification of horizontal force for concentrated load

The method in **5.3.3.9** is very conservative for point loads close to the supports and very low values of “p” may be obtained analytically. Unfortunately, for point loads at mid-span the method of estimating the effective breadth in composite construction underestimates the true effective breadth, and strictly Appendix A of BS 5400 Part 3 should be used in this situation. Equation 5.11 has therefore been modified to take account of the fact that there are likely to be other point loads nearer mid-span, for which the method presented would be otherwise be unsafe.

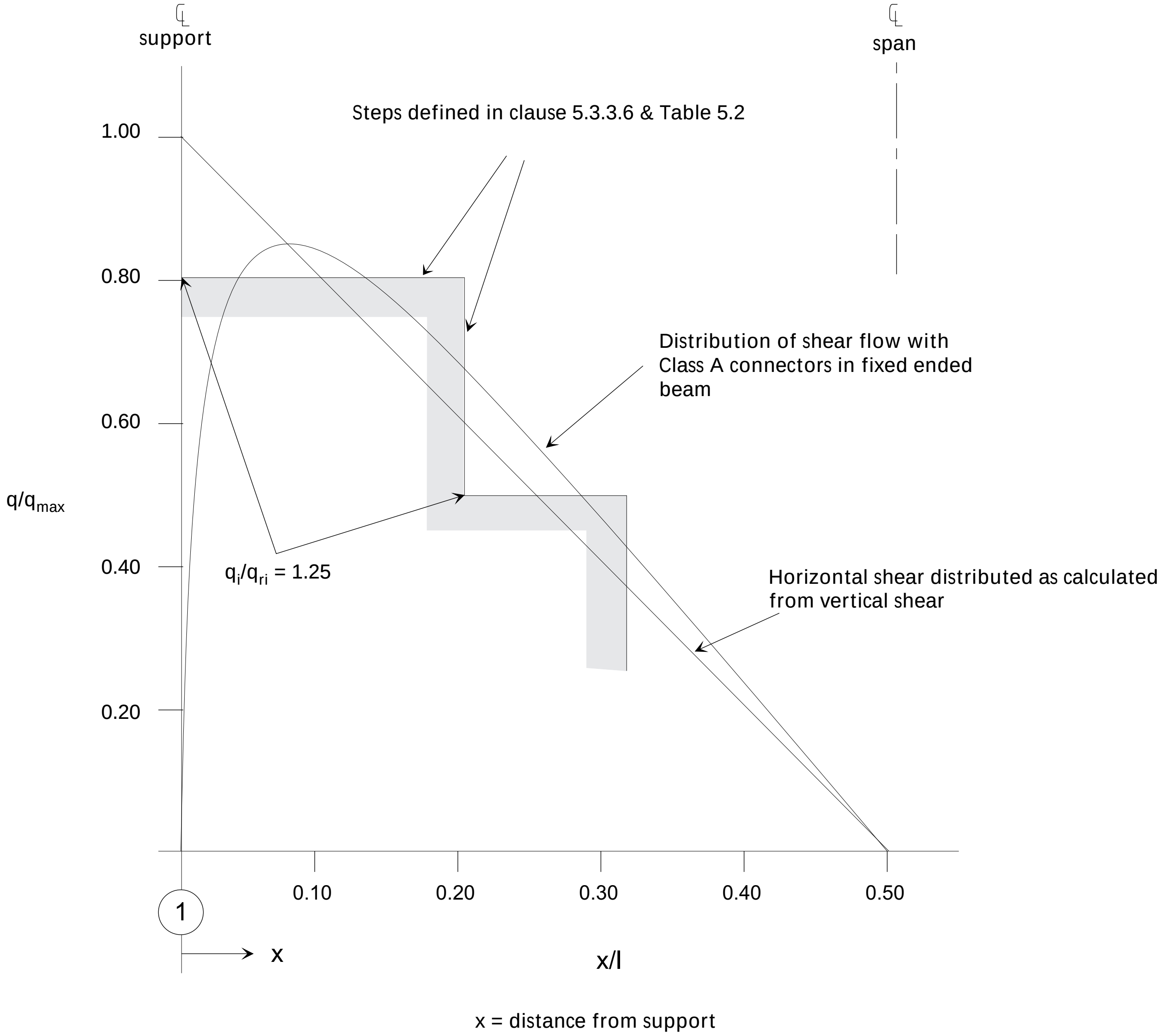


Figure 5.7 Shear flow normalised on shear force

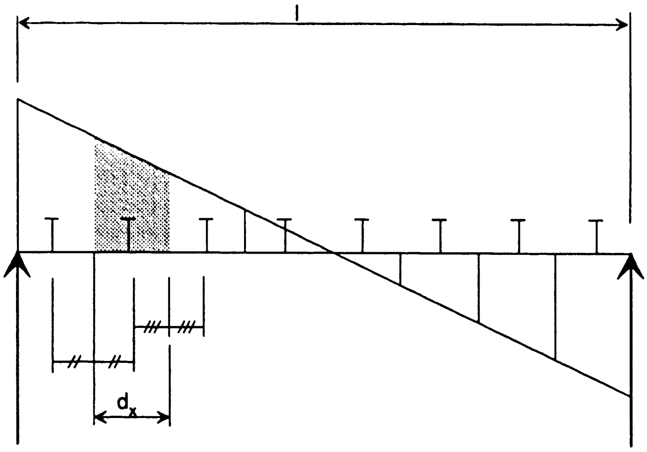


Figure 5.8 Calculation of shear connector force when S⩾Span/8

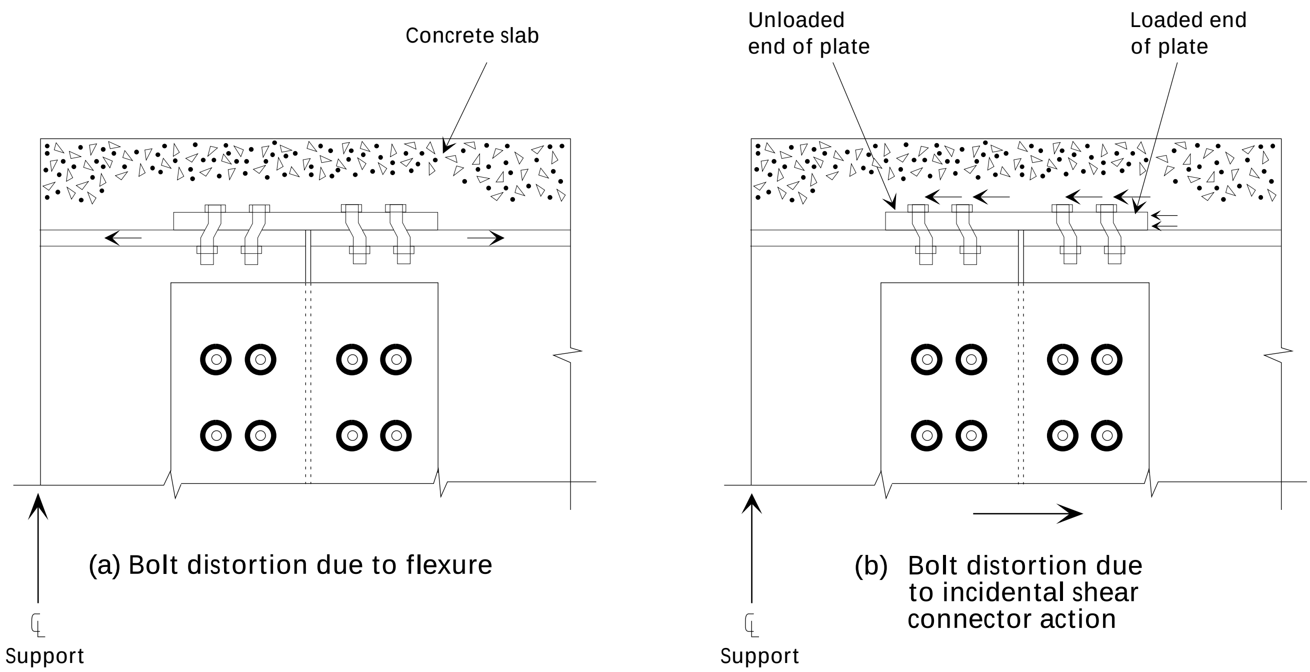


Figure 5.9 Interaction of bolt forces from flexure and incidental shear connector action

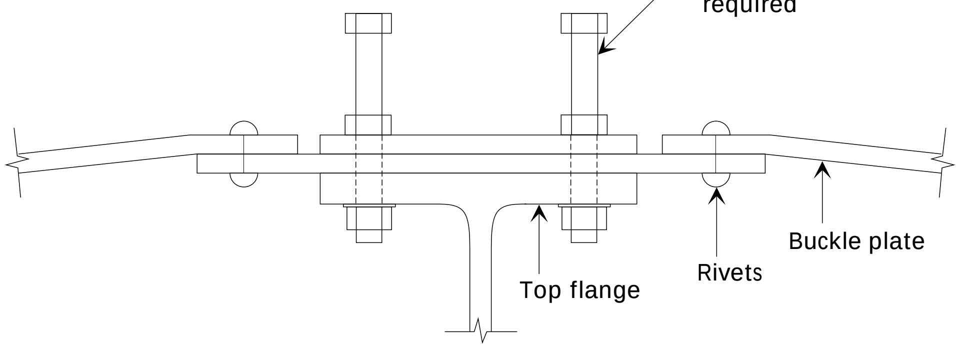


Figure 5.10 Typical device to resist uplift

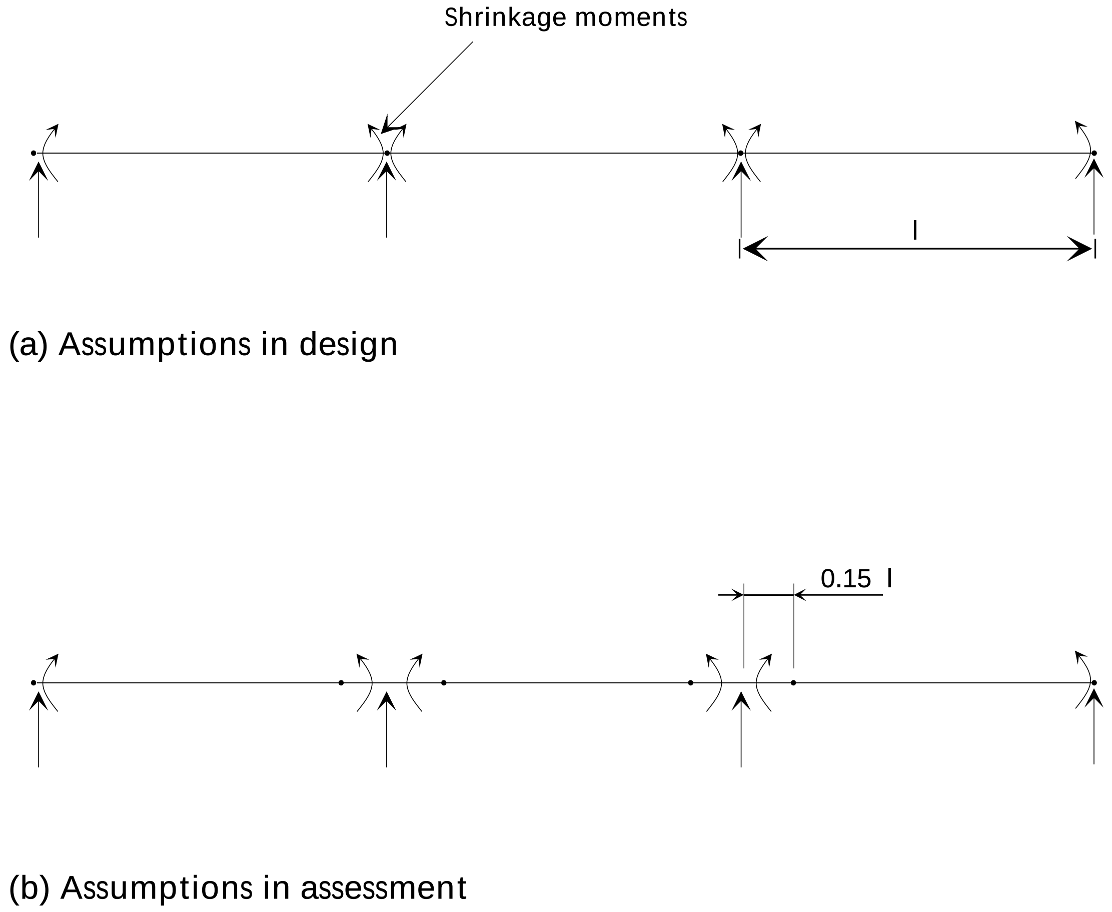


Figure 5.11 Application of shrinkage moments

5.4 Temperature Effects, Shrinkage Modified by Creep and Differential Settlement

5.4.1 General

Whilst there is no justification for completely eliminating temperature effects and shrinkage modified by creep in assessment, their effect can be modified such that they are less onerous than in the design situation. The justification for this is in part that the design situation specified is very conservative, having to take account of the worst likely situation (which is usually more severe than the actual situation) and in part the very few occurrences of damage to structures attributed to thermal and shrinkage effects.

Measures taken to relax the severity of the measure in this Clause are as follows:

(i) include differential settlement and shrinkage only in the load combination which includes temperature, so these effects are excluded for the combination with the highest load factors on dead and live load, which tends to be critical;

(ii) assume that concrete cracked in tension is unable to carry thermal and shrinkage strains, and that the moment from these effects are applied to the beam at 0.15l[10] from the supports, as shown in figure 5.11 (see **5.4.2.1**). In fixed ended beams this reduces the support moment due to shrinkage by 30%; and

(iii) in the calculation of L_s in equation 5.1, reduce the concentrated loading from temperature and shrinkage at simply supported ends of span by 20% for class a connectors and 10% for class b connectors (see **5.4.2.2**.).

5.4.2 Temperature Effects

5.4.2.3 Longitudinal shear

There is a problem inherited from CP117 Part 2 in relation to the K values which appear unconservative, but modifications based upon more accurate theory are overconservative. This is that the present requirements allow for only 1.67 shear connectors per metre, whereas in practice at the ends of beams there are likely to be 10 to 25 connectors/metre[13]. For shrinkage such a reduction could be attributed to early age effects, but if this was the case then the shrinkage force would be many times too high. Such a justification for the use of the specified K values for shrinkage is not applicable to their use for temperature, for which they can only be justified on the grounds that:

(i) Differential temperature movements in the sense which increases forces in connectors at the free end are generally much less than the shrinkage movements (which occur in the opposite direction).

(ii) Differential temperature movements which occur in the sense which increase moments at the penultimate support are unloading the end shear connectors, so due to inelastic effects, the stiffness is lower.

5.4.4 Differential Settlement

Following the principles in Ref 27, that the normally smaller SLS effects can be disregarded for assessment unless they are unusually large, differential settlements should be disregarded unless they significantly affect the calculation. As a guide for assessment, the effects need only be considered when the flexural moments exceed 10% of the sum of the moments from dead and live loads, or alternatively where there are physical signs that settlement has occurred. Such signs are excessive cracking, relative settlement between columns, abutments and the carriageway, and bearing distortion. Sometimes, when excessive cracking has occurred and excessive drying shrinkage is considered unlikely, support settlement is the most likely cause. Attempts should then be made to estimate the magnitude of differential settlement as it may have a significantly adverse effect on the stress state in the bridge.

5.5 Deflections

5.5.1 General

See comments in **4.3.1**.

5.5.2 Elastic Deflection

The importance of shear deflection of the webs of steel beams is disregarded in conventional steelwork calculations, but its effect has been shown to be very significant in continuous beams. For typical continuous composite beams with a span to steel beam depth ratios of 20 and 27 the shear deflection in cracked analyses increases the deflection by 22% and 13%, and it is very much higher both for lower span/depth ratios and if the uncracked, or partially cracked, state of the concrete is taken into account. In simply supported beams the effect is still significant, increasing the deflection typically by 25% at an l/d ratio of 10, 9% at an l/d ratio of 20 and 6% at an l/d ratio of 6. The shear deflection is generally more significant in composite beams than steel beams because the shear stress in the webs is higher.

However, deflections of all bridges tend to be significantly less than theory predicts, which is due, inter alia, to unintended composite action, unintended lateral distribution (eg, from erection bracing which has not been removed) and from bearing restraint (the effect of which is often large). Unless advantage is taken of such unintended composite action, the shear deflection may be disregarded.

Unintended composite action includes not only the effect of incidental shear connectors but also the stiffening effects of the carriageway and the parapets. This is why in **5.3.3.8.3** of this Advice Note only a proportion of the increased stiffness found in tests may be attributed to incidental shear connectors. However the stiffening from the carriageway and parapet are significantly enhanced by the incidental shear connection, so these effects may be indicative of its occurrence.

What is unknown, when incidental shear connection is justified by testing, is how close is the condition to the failure of the shear connection. For this reason special care should be taken to ensure structures with incidental shear connectors are not overloaded, either during the test or as a result of change of traffic usage.

6 ASSESSMENT OF SUPERSTRUCTURE FOR THE ULTIMATE LIMIT STATE

6.1 Analysis of Structure

6.1.1 General

Shrinkage, temperature and differential settlement effects may be disregarded at the ULS except for the most slender cross sections.

6.1.3 Composite action

Composite action from incidental connectors at ULS is only permitted where separation is taken into account and when the connectors can resist the bending due to separation. Separation at ULS is most likely to occur under heavy axle loads, and adjacent to any cross sections at which inelastic rotation has occurred. It is however sufficient for the purposes of the calculation to assume elastic conditions throughout and model the structure as discussed in **5.3.3.6**, but using an iterative procedure in which connections are removed when the deflection exceeds 0.50mm, which with adjustment to the resistance could be relaxed to a significant proportion of the depth of the connectors, or when the moment exceeds their flexural capacity.

Lift off is unlikely to occur over appreciable lengths of beam, even under the effect of concentrated loads. This is because away from the immediate vicinity of these loads the self weight of the slab will ensure the slab is in contact with the beam. For this reason shallow incidental shear connectors may be assumed to provide lateral restraint to the top flange at ULS, even though composite action may be inappropriate for the design of the cross section.

6.1.4 Distribution of bending moments and vertical shear forces

6.1.4.2 Redistribution of support moments in principal longitudinal members

In Eurocodes 3 and 4 there are four classes of cross section which compare with two in BS5400 Part 3, described as compact and non-compact. The finer cross section classification permits a more economical calculation procedure to be defined for many cross sections, and BD 56 has followed Eurocode 3 in dividing compact cross sections into those suitable and those unsuitable for plastic analysis.

Plastic global analysis is not used in the Standard for two reasons, as follows:

(1) the rules necessary for plastic analysis of composite beams have onerous restrictions, which severely restrict their usefulness in the assessment of UK bridges. One is effectively a requirement that most of the loading is uniformly distributed and there are others restricting the ratios of adjacent span lengths. Of these the former is the most restrictive in application to UK bridges. Furthermore the rules were derived for beams of uniform cross section, although this is not a limitation on the method.

(2) The method in assuming unlimited redistribution of support moments results in the ULS becoming less critical than the SLS, to the extent that the assessment relies upon the SLS check for satisfactory performance of the bridge, rather than in ensuring against severe infringements of the SLS condition - the most appropriate approach for assessment.

In Eurocode 4 the cross section classes determine a number of aspects of the calculation, but they have been adopted in the Standard purely to define the amount of redistribution permitted, and, to avoid confusion with the compact and non-compact definitions in BD 56, they are described as “cross section redistribution classes”. The values in the table for cracked analysis are intended (in Eurocode 4) to give comparable results to those for uncracked analysis; it is likely however that in the larger bridges, when the strength of the steel section by itself is high in comparison to that of the composite cross section, that the uncracked analysis, after moment redistribution, will result in lower support moments. Therefore as an interim measure until more precise guidance can be offered the amount of redistribution made when spans exceed 30m has been reduced, and when spans exceed 45m cracked analysis is required.

The cross sections are defined by the more critical of the web and compression flange cross section slenderness. For assessment, having redistributed the moments, one has the option for the more compact cross sections of designing them elastically or plastically **(see 6.2.2**), and for this reason the cross section classes are defined both in terms of the elastic and plastic stress distribution. The limits for the elastic redistribution have been derived such that they approximately agree with those for the plastic redistribution in table 6.4, which differentiates between cross section classes 1, 2 and 3 only in terms of the plastic cross section, and so the values in table 6.3 are an approximation to those in table 6.4, which may be used instead, even if an elastic cross section check is intended. However for plastic assessment of the cross section table 6.3 may not be used.

The use of an elastic cross-section check for cross-sections at which the degree of moment redistribution ensures plasticity within the cross-section effectively introduces a degree of partial shear connection equivalent to that for full interaction and an even lower degree where **5.3.3.9** applies.

Moment redistribution is restricted to beams of substantially uniform cross-section throughout each span. The reason for this restriction is that a unit moment at supports produces a greater support rotation in a beam with a reduced mid span cross-section than in a beam of uniform cross-section over its length. The degree of moment redistribution is related to the hinge rotation capacity and upon there being spare flexural capacity at the more critical cross-sections in adjacent spans. For spans in which the cross-sections are non-uniform the permitted redistribution at a support could be taken as k times the value in table 6.1 where:

$$k = \frac{\textit{end rotation of beam B under unit moment at that support}}{\textit{end rotation of beam A under unit moment at that support}}$$

where beam A is the actual beam, but which may be considered fixed at the remote end of the span,
beam B is a beam with the second moment of area of the support cross-section uniform across the span and with the same fixity at the remote end as beam A.

However k may be taken as unity where the second moment of area of the cracked support cross-section does not exceed the second moment of area at the mid span cross-section.

6.1.4.3 Redistribution of span moment in principal longitudinal members

In assessment of bridges for HB vehicles there may be some benefit to be gained for redistributing mid-span moment to the support. For beams in which the cross section varies substantially the correction in **6.1.4.2** does not apply but, where the support sections are stiffer, the permitted degrees of redistribution may be applied to the span section. When this is done the ratio of adjacent span lengths should satisfy the requirements for plastic analysis in Ref 28.

6.1.5 Temperature effects, shrinkage modified by creep and differential settlement

Where there are no obvious construction defects and no appreciable transverse loads on the girder then the requirement from BD 56 that shrinkage should be included when

$$\lambda_{LT} \geq 45\sqrt{355/\sigma_y}$$

is onerous and in most circumstances may be relaxed to:

$$\lambda_{LT} \geq 70\sqrt{355/\sigma_y}$$

and may be disregarded in bridges more than 15 years old entirely if the condition is considered good.

See also comments under **6.1**.

6.1.6 Shear resistance of composite beams

As the web may have been weakened by corrosion it is advantageous if the contributions of the slab to the shear resistance is taken into account. Tests show that for shear, composite beams may be designed disregarding the interaction of shear and moment. However the tests were confined mainly to beams in which the ratio of the slab depth to the girder depth was rather greater than in bridges with steel girder depths greater than 800mm, and this is taken into account in the assessment rules. This assumes concrete deck thicknesses of about 230mm.[29]

The rule for shear interpolates between a condition of no moment-shear interaction for a steel beam with a depth of 800mm and the normal design situation in BS5400 Part 3 in a steel beam with a depth of 1600mm.

An alternative procedure is allowed whereby the shear in the steel beam is reduced by the shear carried by the slab in a finite element analysis in which the slab is modelled by plate elements. It is suggested that cracking in the slab can be adequately taken into account by reducing the stiffness of the slab over a length of 0.15l from the supports to two thirds of that of the cracked slab disregarding tension stiffening. This method is not permitted when the moment redistribution taken exceeds that permitted for a beam in cross section redistribution class 2. For this method the moment/shear interaction expressions of BD 56 must be used.

The coincidental direct tension in the slab reduces its shear strength and must be taken into account. The shear strength is first estimated using BD 44, and the reduction due to the tension should be based on the latest edition of AASHTO[30].

6.2 Analysis of Sections

6.2.2 Bending resistance of compact sections

The coefficient in the concrete term is increased from 0.60 and 0.67. The earlier value was based on a concrete stress block of $0.40f_{cu}$, whereas $0.45f_{cu}$ is used in column design in the Design Code and in the more accurate stress block of BS 8110. It is noted that, based on studies by Stark [31], Eurocode 4 uses a higher concrete strength than Eurocode 2.

Lateral-torsional buckling is disregarded in the assessment of composite beams with compact cross sections when the shear connectors have adequate capacity to resist uplift forces, when U-frame action exists.

Compact beams with incidental shear connectors however are subject to lateral torsional instability and they should be checked in accordance with Appendix B.

Whether the composite cross section or the steel cross section alone is employed in the check depends upon criteria given in **6.1.3** of the Standard. With shallow devices extra ties may be required if it is required to assume the entire cross section is composite.

6.2.3 Bending resistance of non-compact sections

When the steel section comprises a UB, UC or similar section with equal flanges of strength not exceeding 355 N/mm^2 and certain other conditions (unrelated to the cross section slenderness) are satisfied, lateral torsional effects can be disregarded. The conditions were derived by consideration of Appendix B, but taking into account work by Weston, Nethercot and Crisfield[32] which shows that lateral torsional buckling can be disregarded for λ_{LT} values up to 60, which includes all UB sections not exceeding 355 N/mm^2. It will be found that for a very few cross sections Appendix B gives slightly lower strengths than the method of **6.2.3.3**, even when all the conditions on **6.2.3.3** are satisfied.[33]

6.3 Longitudinal Shear

6.3.1 General

A check is required on the strength of the shear connectors at ULS on spans where cross sections are assumed in the assessment to be plastic[46], but no check is required on spans where the beam is assumed to be elastic throughout. Shear connector will also have to be checked at ULS if stresses are redistributed from the web or tension flange or if there is uplift in the connectors. When cross sections are designed plastically, the compression force in the concrete in the sagging moment region and the tensile force in the reinforcement in the adjacent hogging moment region (which act in the same direction) are combined, and a check is made that there are sufficient shear connectors between the two sections to resist this force.

When stud shear connectors or other flexible shear connectors are used, there is sufficient redistribution of the longitudinal force between the shear connectors to obviate the need to check the distribution of longitudinal shear under this condition. When the shear connectors are rigid a different situation exists and the condition could be somewhat more critical. However, there is no known evidence suggesting that a beam designed for the elastic longitudinal shear distribution at SLS performs poorly at ULS, so no check is required.

The longitudinal shear force from a concentrated load is uniform between the load and the adjacent supports, both in the plastic and elastic design of the longitudinal shear connection, so it is unlikely to be critical at ULS provided the yield moment is not exceeded. No check at the load is then required. If, however, the yield moment is exceeded, due to the absence of a recognised intermediate state of plasticity, a plastic hinge must be assumed under the point load. This is likely to produce a more critical condition for the check on the longitudinal shear resistance.

The effective breadth used in determining the longitudinal shear may now be taken as the quarter span values, except for very wide slabs.

The plastic capacity provides an upper limit to the longitudinal shear force calculated elastically. This situation is only likely to be realised where there are deep edge beams integral with the slab.

6.3.3 Transverse reinforcement

6.3.3.1 Definition and general requirements

The rules for transverse reinforcement in the Design Code require broadly similar amounts of transverse reinforcement to the rules in Eurocode 2 (for reinforced concrete) which is used in design to Eurocode 4[34]. The areas of transverse reinforcement required by these codes are very much higher than those required by BS 8100[35], in which the minimum transverse reinforcement in the slabs of T beams is 0.15%, which is placed near the top surface. The same quantity of reinforcement is required for mild steel, high yield steel, smooth bars and deformed bars. The amount specified is the same as in CP 114[36], the code of practice used in designing concrete buildings in the 1960's (and for minor structures until more recently), but it had been increased to 0.30% (with similar lack of qualification) in CP 110[37], issued in 1972. The drafting committee of BS 8110, which replaced CP 110 in 1985, chose to revert to the earlier values on the practical consideration that no problems were known to have resulted from this aspect of construction in building structures designed to CP 114.

The satisfactory performance of building structures designed to CP 114 and BS 8110 is attributed to the fact that the cracking of the slab parallel to the face of the beam is never sufficient to significantly reduce the longitudinal shear resistance of the beam below its uncracked strength, which is so high, at least with good concrete, as to be able to carry the longitudinal shear without assistance from the reinforcement. The rules in the Design Code and Eurocode 2[38] however are based on tests in which large cracks had been induced along the sides of the beams (by high slab loading), a degree of cracking only to be expected when the slab was on the point of failure. BS 8110, therefore, effectively recognises the practicality of a situation that is likely to occur in bridges. BS5400 Part 4 requires the same minimum amount of transverse reinforcement as BS 8110, but, on account of considerations unrelated to the strength of the shear connection, slabs designed to BS5400 Part 4 usually have much more reinforcement than the minimum requirement for transverse reinforcement.

When shear connectors are grouped such that the spacing exceeds span/8 the bursting stresses on the concrete are likely to be higher than when the connectors are more uniformly distributed. Strictly successive stress blocks, as used for end block design[39] need to be considered to ensure that splitting does not occur. This approach does not work well for shear connections as the transverse tensile stress on the bearing side of a loaded area is greatest over a distance of 0.2d to 2.0d[7], and for 19mm studs this zone is about 0 to 50mm in front of the bearing face of the stud. There is however no requirement for transverse reinforcement over this distance, but reinforcement is nevertheless likely within 100mm of the bearing face.

For this reason the alternative approach has been adopted, of preventing splitting in the general vicinity of the shear connectors by requiring that the longitudinal shear due to the group is carried over a limited distance on the compression side of the group of shear connectors, using the normal design expression of the Standard.

The indirect tensile forces from incidental shear connectors are lower than those from other shear connectors. The requirement for longitudinal shear resistance carried locally is therefore reduced.

8 CASED BEAMS AND FILLER BEAM CONSTRUCTION

8.1 Scope

8.1.1 Introduction

There is a problem in the assessment of filler beams and cased beams that, if plastic cross section analysis is to be permitted at ULS, the longitudinal shear strength of the steel/concrete interface is likely to be exceeded.

However, for filler beams the situation is somewhat different as tests have been conducted on construction without transverse reinforcement, or with reinforcement inadequate to satisfy the requirements of **8.3.2**, to indicate the plastic design of the composite cross section at ULS is achievable. In a form of filler beam construction developed in France [(41)], which includes special details and transverse prestressing, no check is required on the local bond stress at either SLS or ULS. However until such time as longitudinal shear in filler beams has been researched a check on the longitudinal shear on planes of type 6 and 7 in figure 8.3 is required as a safeguard against possible failure modes not yet observed.

8.1.2 Cased beams

Cased beams exist which do not comply with the requirements of the Standard, and reduced interface shears are appropriate for less efficient casings and partial casings which do not have efficient confining reinforcement and may be entirely unreinforced. These are types B, C and D in figure 8.2. Permissible interface bond strengths at SLS for these beams are as follows (N/mm^2):

A.	(As in Standard)	$0.10 \sqrt{f_{cu}}$ but $\not> 0.70$N/mm^2
B.	20mm $\leq$ cover $\leq$ 50mm	$0.07 \sqrt{f_{cu}}$ but $\not> 0.40$N/mm^2
C.		$0.06 \sqrt{f_{cu}}$ but $\not> 0.35$N/mm^2
D.	Cover $\leq$ 50mm	$0.10 \sqrt{f_{cu}}$ but $\not> 0.50$N/mm^2

For cased beams of types A and C the checks required are as follows. Partial shear connection calculations should assume plastic methods of cross section design and a ductile shear connection. The equilibrium method is the method of Clause 5.5.2 of Ref 28 or the 'plastic theory' method of Clause 6.2.1.2 of Ref 4. The linear interpolation method is the more conservative method of Clause 6.2.1.2 of Ref 4. In the bond strength expression f_{cu} is the worst credible cube strength in N/mm^2 and the longitudinal shear stress on the shear connection at ULS for both methods is taken as not more than 0.50N/mm^2.

For spans exceeding 12m the bond stress at SLS should not exceed the permissible bond strength given above nor 0.5N/mm^2. At ULS the moment of resistance should be taken as the yield moment.

For spans between 6m and 9m the permissible bond strengths should not be exceeded at SLS. At ULS the moment of resistance should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the linear interpolation method.

For spans less than 6m the permissible bond strength at SLS should not be exceeded. At ULS the moment of resistance should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the equilibrium method. The following conditions apply:

(i) Where the moment of resistance is taken as the yield moment the design strength of the metal should not be taken greater than $275N/mm^2$.

(ii) The moment of resistance for cased beams type B should not be taken greater than the yield moment.

(iii) The moment of resistance for beams type D should not be taken greater than the lower of the moment at bond failure and the moment of resistance calculated in accordance with chapter 6 of the Standard assuming full composite action. A check at SLS is not required.

(iv) The moment of resistance for beams of types A, B and C may be assumed to be the yield moment where this is sufficient to satisfy the assessment loading.

(v) These rules do not apply to cased beams where the soffit of the slab is above the top flange of the steel beam by more than 25mm.

(vi) In beams with abnormal concrete depths above the metal beam the depth of concrete used in calculation should be restricted such that the elastic neutral axis lies within the depth of the steel section.

(vii) As an alternative to (vi) the full depth of concrete may be assumed and the metal beam concentrated at the centroid of the steel section.

8.1.3 Complying filler beams

This section considers only filler beam construction in which the transverse reinforcement is adequate to resist the moments obtained from orthotropic plate or conventional grillage analysis. For this situation it is recommended that orthotropic plate analysis is adopted for analysis, but grillage representations, allowing for the different stiffness transversely and longitudinally, are an acceptable alternative. The transverse distribution rules of BA 16 are inappropriate for this form of construction.

The method in **8.3.2** of the Standard, which gives transverse distribution rules for 45 units of HB loading, should now be regarded as a Category A alternative (see Foreword). However providing the conditions in **8.3.2(b)** and **(d)** are satisfied:

(i) the transverse hogging moment may be taken as 10% of the maximum sagging moment, and

(ii) it may be assumed that there is a linear reduction in the transverse sagging moment in the 2m side strips as specified in the Standard.

Filler beams may be assessed assuming the principles in **8.1.2**, but the stress on the shear connection at ULS taken as 1.4 times the values for cased beams type A and:

For spans exceeding 8m the permissible bond strength should not be exceeded at SLS and the flexural capacity at ULS should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the linear interpolation method.

For spans less than 8m the permissible bond strengths at SLS should not be exceeded at SLS and the flexural capacity at ULS should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the equilibrium method.

Alternatively for beams satisfying **8.3.2(c)** of the Standard the moment of resistance may be assumed to be the full plastic moment of resistance without a check on the bond stress at ULS, provided the yield stress of the metal beam used in the calculation is not greater than $275N/mm^2$ and provided **8.1.2(vi)** is satisfied or provided there is not more than 75mm of concrete above the top flange of the steel section. Where of these conditions only **8.3.2(c)** of the Standard is not satisfied the moment of resistance may be taken as the yield moment.

8.1.4 Non-complying filler beams

For beams not complying with **8.3.2** of the Standard, providing there is no evidence of excessive corrosion, fretting action or cracking (in the case of cemented materials) sufficient to adversely affect the achievement of composite action, Clauses **8.1, 8.3, 8.4, 8.5.1** and **8.5.2** apply.

In filler beams without transverse reinforcement, the lateral distribution of load is greater than that suggested by grillage analysis with pinned transverse members. To reproduce the actual distribution the flexural stiffness of beams should be based on the composite cross section, and the torsional stiffness of internal beams taken as that of the rectangle of concrete horizontally between mid span of adjacent infill elements and vertically between the flanges of the steel beam. Suitable grillages for analyzing filler beam decks are shown in figure 8.4, where the member properties to be assumed are shown in Table 8.1. In using the grillage in figure 8.4(b) the torsion per unit width is to be taken as the sum of the torsions per unit width in the two directions. The two grillages give similar distributions of bending moments and torsions, but that in figure 8.4(b) allows lateral distribution of reactions, whereas in the grillage in figure 8.4(a) allows none (as the method of BD 16).

Table 8.1 Cross Section Properties for Global Analysis of Non-complying Filler Beams

	Lateral Distribution of Reactions: Disregarded	Lateral Distribution of Reactions: Included
Main beams		
I	$I_{composite}$	$I_{composite}$
J	J_{steel}	J_{steel}
A*	$A_{composite}$ or ∞	$A_{composite}$ or ∞
Intermediate beams		
I	-	0
J	-	$J_{concrete}$
A*	-	$A_{concrete}$ or ∞
Transverse beams		
I	0	$I_{concrete}$
J	J	$J_{concrete}$
A*	$A_{concrete}$ or ∞	$A_{concrete}$ or ∞

* Analysis is insensitive to A

The grillage in figure 8.4(b) should only be used where the abutments are sufficient to resist the torsions, a condition which may be assumed providing.

(a) The abutments are in good condition and not sufficiently cracked so as to relieve torsion moments, and

(b) the slab should project a distance equal to the depth of the metal beam past the end of the beam, or

(c) the concrete in the deck should be monolithic with a substantial abutment beam of depth not less than 50% greater than the depth of the filler deck, or

(d) the concrete in the deck should be monolithic with a concrete abutment.

In assessing the results of the analysis, the flexural resistance may be assumed to be the same as for complying filler beams providing the effective concrete is restricted for torsion to that within a depth above the steel beam soffit of 1.5 times the depth between the flanges. For flexure **8.1.2(vi)** applies (the alternative in **8.1.2(vii)** does not apply). The torsional strength of the concrete may be taken as

$$\frac{0.41(f_{cu}/\gamma_{mc})^{0.5}}{\gamma_{f3}}$$

Dense brickwork filler beams with the mortar fully bonded to the bricks and the steel beams shall be assessed in accordance with above provisions except the bond stress in **8.5.1** shall be taken not greater than $0.3N/mm^2$, the resistance of attachments shall not be taken greater than 60% of the value in **5.3.3.8.1** and moment of resistance should not be taken in excess of the yield moment.

When the brickwork is not bonded to the steel beams, similar provisions apply except that the bond shall be taken not greater than $0.3N/mm^2$ and the resistance of attachments shall be taken no greater than 40% of the values in **5.3.3.8.1**.

8.1.5 Vertical shear resistance

For cased beams and filler beams the shear resistance of cemented material up to 250mm above the level of the steelwork, and for a width on either side as shown in figure 8.5, may be added to that of the steelwork assuming a shear strength of concrete v_c based on the concrete strength f_{cu} as given in BD 44. Strictly this value applies only when there is a small amount of longitudinal reinforcement and for this purpose the steelwork is deemed to be effective as reinforcement. The shear assumed to be carried by the concrete should not exceed 15% of the total shear in cased beams and 30% of the total shear in filler beams.

8.1.6 Procedure when longitudinal shear resistance is inadequate

When the longitudinal shear exceeds the permissible shear stress, composite action should be disregarded and all beams with fill on both sides should be considered to be compact, irrespective of the cross section slenderness.

8.1.7 Punching shear resistance

The punching shear resistance to a wheel load may be assessed assuming the load is replaced by two strip loads, each of which has the same width and centroid of the part of the load which would be carried by statics to the supporting beam. The shear may be assumed to be carried over a width equal to the loaded width plus a_v, assuming a concrete strength of $2v_c d/a_v$, where a_v is the distance from the strip to the face of the web of the neighbouring metal beam and d is the depth from the surface of the concrete to the lower web/flange intersection of the metal beam.

For a wheel load on a bay adjacent to an edge beam the resistance should be taken as 70% of that for an internal bay of similar dimensions, unless it can be shown that the horizontal thrust resulting from the arching action shown in Fig 8.6 can be adequately resisted.

8.1.8 Effect of end restraints and of finishings and infill material not satisfying BD 44

Where tests with vehicles of weight not less than 70% of the assessment vehicle suggest there are significant incidental strengthening effects under four passages of the vehicle, an increase in strength may be taken into account as follows

(i) Effect of end restraint

The effect of end restraint from friction in resisting the resolved longitudinal and transverse forces (if any are assumed) may be taken into account calculated from the dead loads above the level of the soffit, including that in the abutment beam and a coefficient of friction of:

0.35	for concrete on masonry or masonry on masonry
0.50	for concrete cast on concrete with an unprepared surface
0.60	for concrete cast on concrete with a prepared surface, or monolithic concrete of strength not exceeding 20N/mm^2
0.75	for monolithic concrete of strength exceeding 20N/mm^2

It should be noted that where the abutments are thicker than the slab there may be significant end moments from continuity with the support, but this should be discounted due to the likely loss of this effect when the concrete cracks, unless there is flexural reinforcement present satisfying BD 44.

(ii) Effect of finishings

The contribution of the concrete which would not normally be regarded of structural quality, masonry and well compacted (cohesive or weakly cemented cohesive) material, between and above the steelwork, may be taken into account in the assessment of the effect of live loads of filler beam decks, where it can be shown that the material is in contact with the full depth of the web or on flat rough concrete surfaces of construction satisfying **8.1.2**, **8.1.3** or **8.1.4**. An initial elastic cross section analysis is required in which such material may be taken into account by assuming

α_e = 15 where there is at least 150mm of concrete above the top flange of the steel beam, or otherwise

α_e = 30.

The strain at the surface of the carriageway determines the method by which the finishings may be taken into account in carrying the live load moments as follows:

Multiple passage of vehicles (as at SLS)	Single passage of vehicles (as at ULS)	Method of taking finishings into account
≤ 350 με	≤ 500 με	by the analysis just described
≤ 700 με	≤ 1000 με	by increasing the elastic section modulus of the steel by $(h_c + h_s)/h_s$

In the calculations the dead load should be assumed to be carried by the bare steel section, but the superimposed dead load may be carried on a composite cross section satisfying **8.1.2**, **8.1.3** or **8.1.4**.

The calculation may be used for any purposes for which the results are more economical than the methods of **8.1.2**, **8.1.3** and **8.1.4**.

In no case should the strengthening effect be taken greater than that suggested by the measured deflection (or preferably strain) readings nor greater than:

2.5 times the strength of the bare steel section, or

1.7 times the strength of a composite section which satisfies **8.1.2**, **8.1.3** or **8.1.4**.

In constructions in which shear is critical the effect of finishings should only be taken into account when approved.

It is difficult to offer guidance in regard to construction in which it is suspected that the parapets may carry a significant portion of the bending moment

- A load test might damage the parapet.
- When the deck is less strong than the theory suggests brittle failure could result.
- Where the parapet is well connected to the deck it could sometimes contribute to the strength.
- Substantial parapets can span independently of the deck, thereby removing heavy dead loads from the deck.

Some benefits from the finishings, typically 5 to 10%, is likely when the cross sections are designed plastically, but asphalt or concrete wearing courses may be sufficiently strong to induce bond failure in weaker layers beneath. The top 75mm of the finishings should be disregarded.

(iii) Effect of infill material not satisfying BD 44

Suggestions for inclusion at the ULS of the effects of infill material not satisfying BD 44 are presented in Appendix H, which may be used when approved.

(iv) Combined effects of end restraint, finishes and infill material not satisfying BD 44.

End restraints and either finishes or, where permitted, infill material not satisfying BD 44 may be considered to act simultaneously, but account should be taken of the fact that end moments may reduce deflections disproportionately to their increase in resistance to the load. The effect of finishes not satisfying Appendix H and infill material not satisfying BD 44 may not be combined.

(v) Testing

As an alternative to (i) to (iv), where due to non uniformity of material, doubt as to its quality or where these procedures are believed to underestimate the strength, other agreed procedures are permitted. A procedure such as that in the second sentence of **5.3.3.8.3** above may be valid, but the increase in strength attributable to increase in stiffness needs to be established.

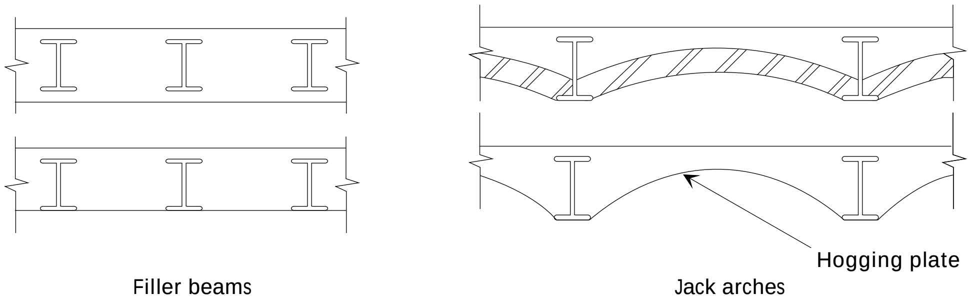
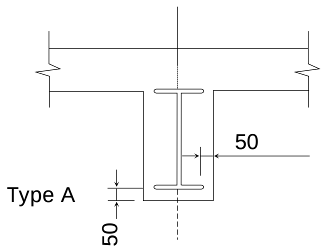
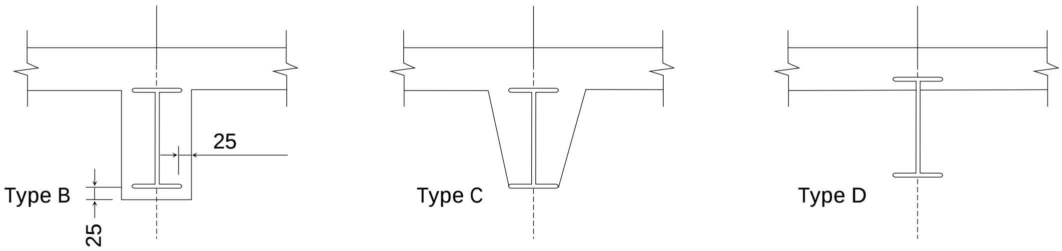


Figure 8.1 Filler beams and jack arches



(a) Type of cased beam considered in the standard



(b) Other types of cased beams occasionally encountered

Figure 8.2 Cased and partially cased beams

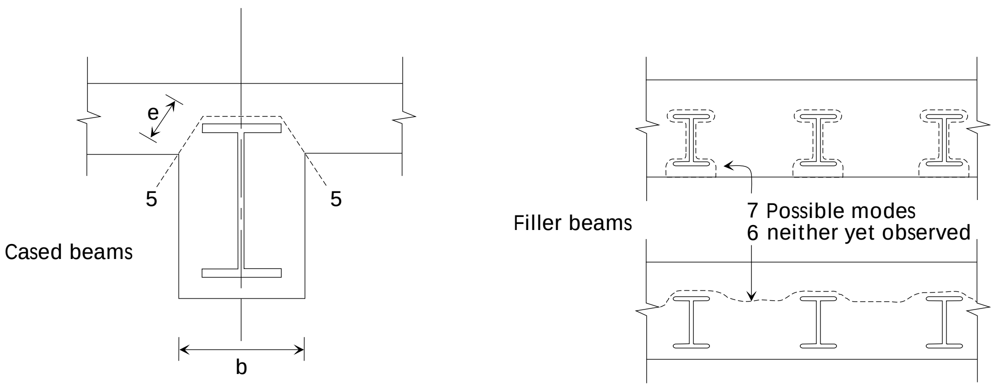
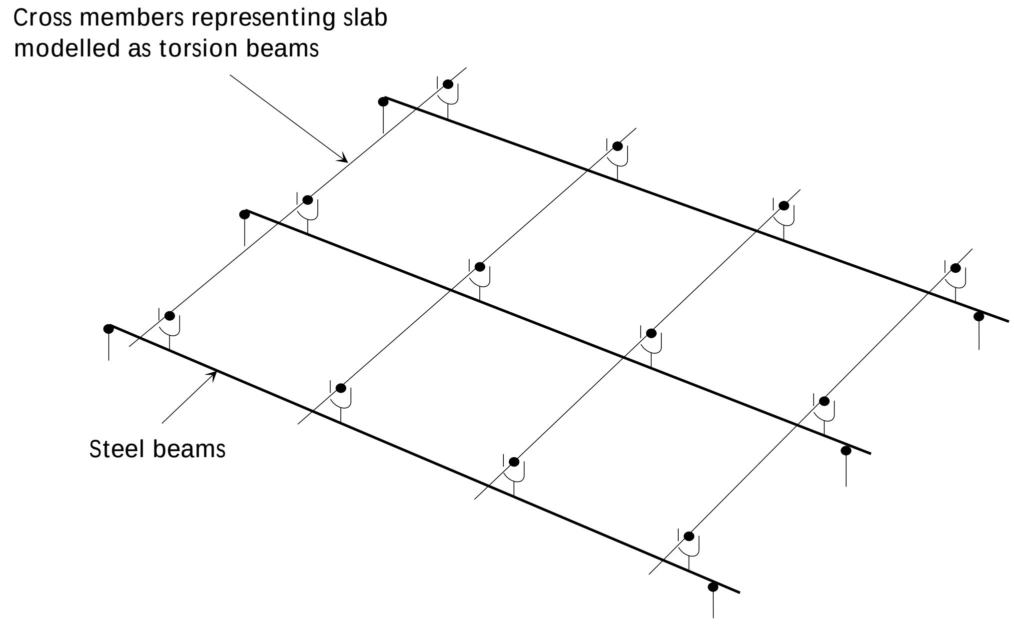
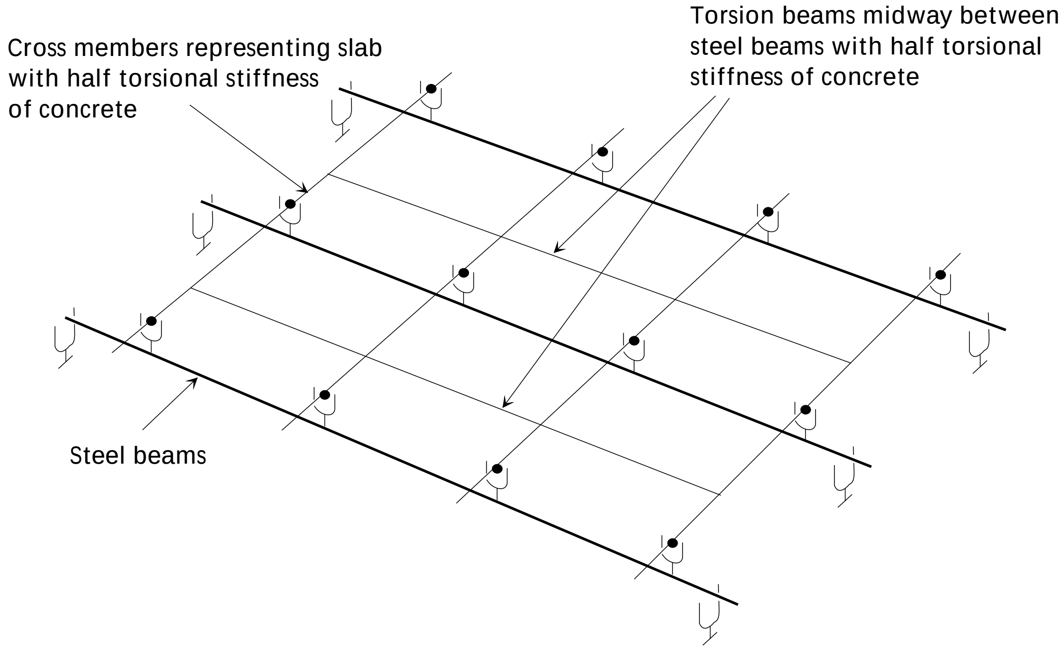


Figure 8.3 Failure modes in longitudinal shear



(a) Model for static distribution of beam reactions



(b) Model for transverse distribution of beam reactions

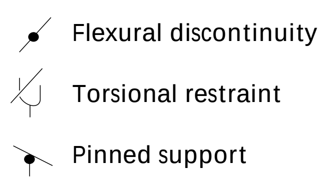


Figure 8.4 Grillages for the analysis of non-complying filler beams

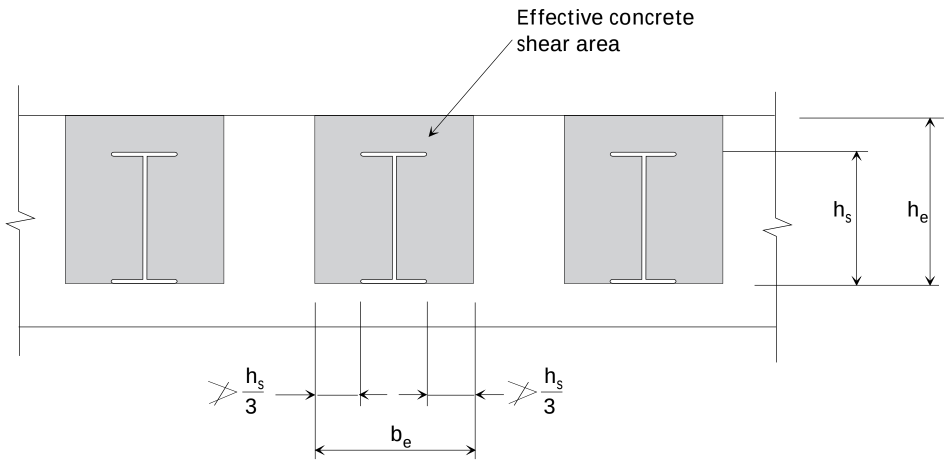


Figure 8.5 Effective concrete shear area of filler beams

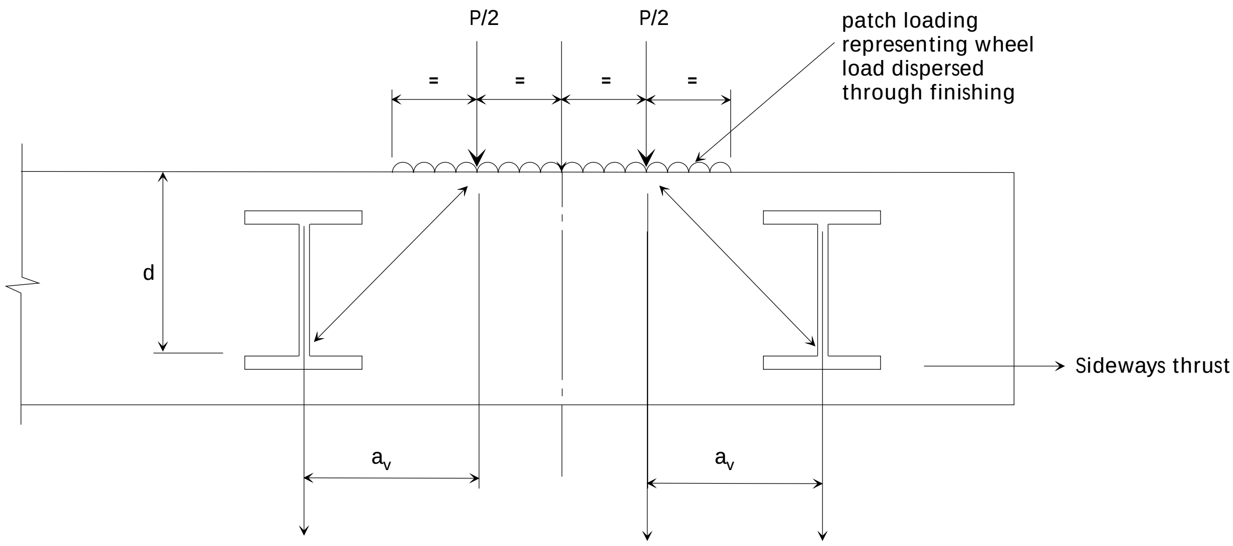


Figure 8.6 Raking strut representation of punching shear

11 COMPOSITE COLUMNS

The Standard is not as comprehensive in its treatment of composite columns as the Design Code and occasionally recourse will be necessary to the Design Code, when the strength and safety factors should be adjusted to agree with the Standard. In particular the Standard does not consider filled tubes subject to bending.

The method for designing cased columns in the Design Code is more elaborate and more conservative than the method in Eurocode 4 and it is seldom used. A method has been produced[(43)] which is simpler than either method and which has been calibrated to give column strengths agreeing with strengths obtained using the Design Code to the following tolerances:

at normal eccentricities of column load (e_x/h_x and $e_y/h_y \leq$ 0.60 for uniaxial bending)	5% unsafe to 10% safe
for biaxial bending and at higher eccentricities in uniaxial bending	10% unsafe to 25% safe

This is the basis of the present assessment procedure.

The method is considered to be generally conservative compared to the method in EC4.

14 JACK ARCH AND TROUGH CONSTRUCTION

14.1 Jack arch construction

The rules for the assessment of jack arches are the same as those for non- complying filler beams subject to the following modifications:

For the assessment of jack arches with concrete infills Chapter 7 of BA 16 permits the use of Clause 8 of the Standard, but with the simple transverse distribution rules in Chapter 2 of BA 16. Alternatively a global analysis may be performed in which the depth of the transverse members is taken as the thickness at the crown of the arch plus one fifth of the vertical distance between the crown and the soffit of the steel section(40).

Jack arches may otherwise be assessed as filler beams, but the effective perimeter assumed to provide bond should be restricted to the upper surface of the top flange and the web where the infill is at least 50mm thick. Also the bond strength at SLS and the stress on the shear connection at ULS reduced by a factor of 1.40 (or some other agreed factor), and the option of sometimes using the plastic moment of resistance does not apply.

The shear resistance of jack arches may be assumed to be the same as for filler beams in **8.1.5**.

The effect of finishings may be taken into account by the method in **8.1.7**. When the girders are of cast iron the values of α_e should be halved.

In hogging plate construction, which is the more modern form of jack arch construction (in which the beams are more likely to be of mild steel than cast iron), the horizontal shear resisted between the hogging plate and the fill should be disregarded, except where the fill is of concrete when incidental shear connection from rivet (or bolt) heads may be taken into account.

Tie rods are often present in jack arches to resist the arch thrust along the sides of the bridge. The adequacy of these should be established.

14.2 Trough Construction

Smooth troughs with ‘open’ profiles have substantially lower incidental strengthening than jack arch or filler beam construction, so no alternative assessment procedures to those in BA 16(2) are provided. However where there are rivet or bolt heads penetrating concrete overlays the rules for incidental shear connectors in **5.3.3.8** may be used, and, where the surfaces of the troughs are sufficiently steep that these devices resist uplift, consideration may be given to reducing values of γ_b.

APPENDIX A
CRACK WIDTHS IN COMPOSITE BEAMS

A method is given for assessing the crack widths in composite beams which is less conservative than BS5400 Part 4, which in the Design Code must be used for assessing crack widths in composite bridges by applying the rules for T beams, for which the crack width is:

$$w = 3a_{cr}\,\varepsilon_m$$

Two situations are taken into account in the Standard, the first being flexural cracking in a well designed beam. An allowance is made for tension stiffening, which is reduced not only by the reinforcement but also by contact with the top flange of the steel beam[(44)]. The expression does not take into account the situation in which the top flange is debonded, which is confined to laboratory conditions. The bonded situation is considered more appropriate to design.

The last term in the tension stiffening expression is a nominal allowance for

(i) the effects of shrinkage, which are not otherwise taken into account,

(ii) the effect of stress attracted to the top flange by the stiffness of the uncracked concrete between the cracks, and

(iii) shear deflection.

The effect of reinforcement which is not parallel to the beam is taken into account by an expression similar to that in BS5400 Part 4.

The other situation (equation A.2) relates to beams in which the reinforcement is appreciably less than that in bridges designed to BS5400 Part 4. It was derived following a study of crack widths in Austrian composite bridges[(45)].

APPENDIX B
LATERAL-TORSIONAL BUCKLING OF COMPOSITE BEAMS WITH SLAB AND GIRDERS TIED LATERALLY AND ROTATIONALLY

The method of assessing the effects of lateral torsional buckling is derived from the method of Eurocode 4, but it has been modified[33]:

(i) to give better agreement with the unbraced case when lateral bracing is added to the compression flange between the support and the point of contraflexure, and

(ii) by combining it with the Perry Robertson expression of BS5400 Part 3, Appendix G[47].

In both respects the method is less conservative than the method of the draft EC4 and is believed to be more accurate.

From the assumption made in the derivation of equation B.1 it is reasonable to assume that it may also be used for cross sections which have steel members asymmetrical about the x axis and are in redistribution class 3, providing:

(i) $b_{ft}t_{ft} + A_r/1.025 \geq b_{fc}t_{fc}$

(ii) $b_{ft}t_{ft} \geq 0.60 b_{fc}t_{fc}$

(iii) $f_{ry}/\gamma_{mr} \geq \sigma_y/\gamma_m$

It is also reasonable that requirements 7 and 8 in **B.2.2** are replaced by conditions (c), (d), (e) (which is 8), (f), (h), (j) and (k) in **6.2.3.3** and providing the steel strength does not exceed 355 N/mm^2.

APPENDIX C
LATERAL-TORSIONAL BUCKLING OF COMPOSITE BEAMS WITH SLABS AND GIRDERS TIED LATERALLY BUT NOT ROTATIONALLY

C.1 General

C.1.1 Introduction

C.1.1.1 Scope

Even when U-frame action is not present, a significant increase in the flexural resistance of beams is obtained from lateral restraints to the tension flange, because the centre of rotation in lateral torsional buckling is lifted to the level of the shear connection.

This topic has been extensively researched at the University of Manchester over a number of years, from which has been produced a design method published in Appendix G of BS 5950 Part 1[(48)]. The method has been translated into the terms used in BS5400 Part 3 and methods have been devised (in **C.1.1.2**) for employing it in assessing composite beams.

Two situations are taken into account:

(1) the situation in which the beam has devices which act as incidental shear connectors at SLS, but which are ineffective at ULS. In this case the method in this Appendix assumes that flexure at ULS is resisted solely by the steel beam, and

(2) the situation in which the beam has deep devices which are similar to shear connectors, except for the absence of resistance to uplift. In this case the full composite section is assumed to carry the loading at ULS.

BS 5950 Part 1 does not specify the maximum spacing of lateral restraint and instead requires that lengths between lateral restraints are checked as unrestrained non-composite beams. Limited research has established that the method is tolerant to widely spaced restraints and conservatively it is suggested there should be restraints at the third span positions.

APPENDIX D
VIBRATION OF COMPOSITE BRIDGES

D.1 Traffic Vibration

Vibration of vehicular bridges due to traffic is not a problem, partly due to the considerable mass of vehicular bridges and partly due to the insensitivity of seated passengers to vibration. For foot and cycle track bridges the problem is addressed in Appendix B of BS5400 Part 2, as implemented by BD 37 [(49)].

Vibration is not normally a problem in older bridges as they are generally heavier than modern bridges, although on some of the more heavily trafficked it is possible that a degree of loosening of parts of the structure has occurred. In this case vibration of those parts is possible. The main concern is with footbridges. Foot and cycle track bridges need not be checked if the span is less than 35m for simply supported spans or 40m if continuous one or both ends. Highway bridges need not be checked if the span is less than 55m for simply supported spans or 65m if continuous one or both ends. Foot and cycle track bridges with rolled steel beams with non-structural parapets are particularly prone to vibration problems.

Nevertheless checks should be made on any bridge about which there has been public complaint.

D.2 Aerodynamic Vibration

Aerodynamic criteria are given in BD 49[(9)]. However composite highway bridges need not be checked for aerodynamic effects when two of the three following criteria are satisfied:

(a) the deck widths do not exceed 12m if not more than 10m above ground level, or otherwise 15m,

(b) the edge details satisfy Clause 2.1.3.2 of BD 37 and the overhang should not be less than $0.75d_4$,

(c) both the following are satisfied:

(i) the super elevation of the line between the top of the facias on either side of the bridge does not exceed 1/40, and

(ii) the incident wind within ±45° of the normal to the span is not consistently inclined to the horizontal (on account of the local topography).

Footbridges need not be checked for spans less than 35m when condition (b) above and (c)(ii) are satisfied; otherwise they need not be checked for spans less than 30m.

Further, vortex shedding need not be considered for any composite bridge of span less than 70m.

These limits are rather greater than the limits advisable for steel bridges as composite decks are very stiff torsionally and therefore are not particularly susceptible to the more dramatic form of vibrational instability. Cable suspension bridges pose particular problems, but are not considered here.

D.3 Cantilevered Footways

Footways cantilevered from vehicular bridges may pose particular problems and should be checked when the length of the cantilever from the primary beams exceed 10m. Sometimes there is a problem with shorter cantilevers, but this is due to interaction with the transverse bending.

D.4 Calculation Procedures

Generally the bridge design codes quoted above should be sufficient, but the information may be supplemented by the procedure in chapter 7.6 of reference 50 and by the use of computer programs, many of which give the frequencies and shapes of the main modes of vibrations, although with some this facility may require the choice of particular, and more powerful, types of elements.

There is a problem in continuous bridges of whether the exciting energy is sufficient to excite adjacent spans, as when this is the case the effective stiffness of continuous beams reduces to a value similar to that of simply supported beams. Some account of this is taken in the method of ref 50. More sophisticated use of computers is permissible to determine the effect of particular excitations, but then the problem is encountered of how best to represent the excitation.

D.5 Damping Values

It is acceptable to assume a logarithmic decrement of 0.04 for composite bridges, as suggested in BD 37[(49)].

APPENDIX E
FATIGUE IN REINFORCEMENT OF COMPOSITE BEAMS

There are two methods of considering fatigue in reinforcement in BD 44. In the first method, which relates to local combination 1 at SLS under HA loading, the permissible stress ranges for unwelded non-corroded reinforcement under imposed loading are given. In shorter span composite bridges these criteria may be critical(21). In this situation the alternative method which refers to BS5400 Part 10(20) should be used.

Corrosion and manufacturers markings can significantly reduce the resistance to a greater extent than the loss of bar area might suggest. Such conditions are not specifically taken into account, but could be assumed to be taken into account in the general conservatism of the first method. For composite beams in which the reinforcement is corroded, BA 38(52) should be used in estimating the remaining fatigue life.

APPENDIX F
REVISIONS IN THE ASSESSMENT RULES SUITABLE FOR DESIGN

This appendix considers the appropriateness of the technical revision only. Limited improvements in the structure of the Design Code have been made which are not mentioned.

All reference to "iron" and to "most credible strength" etc are inappropriate to design.

References to BD 44, BD 56 and BD 21 are also inappropriate to a design standard.

The separate identity of partial safety factors, desirable for assessment, is not necessary for design.

4.1.2 Verification of Structural Adequacy. The use of γ_{mt} in assessing test data is appropriate to design but the term "worst credible strength" is inappropriate. However consideration should be given to revising table 4.2 to give characteristic values.

The consistent use of expression 2b throughout the Standard is appropriate for adoption in the Design Code.

4.3.1 The last paragraph concerns incidental shear connection and is inappropriate to design.

4.3.2 (b) The coefficient 0.80 is inappropriate to design.

(e) This revision is considered acceptable for the design situation.

5.1.1.1 and 5.1.1.2 These revisions are considered appropriate to design, although the effect of considering very short span beams to have cracked supports should be evaluated.

5.2.1 Revision is appropriate to design.

5.2.3.1 Revision is considered appropriate to design.

5.2.3.2 Revision is appropriate to design.

5.2.3.3 Revision is appropriate to design, but further research is needed to establish whether the power of the cosine of the angle, could be reduced to 2.

5.2.5.3 Revision inappropriate to design.

5.2.6.2 and 5.2.6.3 Revision inappropriate to design.

5.3.1 Revisions considered appropriate to design.

5.3.2.1 (b) Revision is considered acceptable for design, but the effect of equation 5.2 near the end of the design life needs to be assessed.

(d) Non-complying shear connectors are inappropriate to design.

5.3.2.4 (a) The use of the mean for five or more specimens is appropriate to design.

Table 5.1 The strength of stud shear connectors could be increased by about 10%. For the other types of shear connectors considered the strength quoted could be used.

5.3.3 The revisions in this section are considered inappropriate to design.

5.4 Excepting the revision concerning the effective breadths is 5.4.2.1 the revisions in this chapter are inappropriate to design; however

(1) the effect of shrinkage should be reduced and

(2) the problems noted with the shear connector stiffness in equation 5.12 need to be addressed.

6.1.3 Revision inappropriate to design.

6.1.4.2 Revisions appropriate to design, however it would be better if BS5400 Part 3 introduced four cross section classifications and then they could be described as “cross-section classes”, rather than cross-section redistribution classes”.

6.1.5 Revision appropriate to design.

6.1.6 Improved version of revision may be appropriate for design.

6.2.1 Revision appropriate to design.

6.3.3 All revisions appropriate to design except 6.3.3.8.

6.3.4 Revision appropriate to design.

8. Revision inappropriate to design, except check on yield stress at serviceability.

9. Revision inappropriate to design.

10. Revision inappropriate to design.

11. Revision appropriate to design, but greater scope of former method requires that it is retained in an appendix.

Appendix A Inappropriate to design, but equation A.1 and A.3 could be used as an interim measure.

Appendix B Appropriate to design, but needs to be modified for U-frame action.

Appendix C Inappropriate for design.

APPENDIX G
SIGNIFICANCE OF NOMINAL CONSIDERATIONS

There are a number of situations in which dimensional and other criteria specified in the Standard may be infringed without significantly affecting the structural performance, but which in some may significantly affect its rate of deterioration after the assessment. Other criteria may not significantly affect the performance of many structures. Some aspects of the construction which are clearly satisfactory need only be checked by calculation if there is a likelihood of them becoming affected by a change of use.

The measures to be taken depend upon the category of the infringement as follows:

Category A: Where these criteria are infringed by 20% or more an inspection shall ensure that the concrete has not spalled locally as a result of corrosion.

Category B: Where these criteria are infringed the strength may be lower than assumed in the Standard. When infringed the effect on the strength shall be assessed.

Category C: As Category B, but assess only when infringements are greater than 5% (or 7% when they affect areas).

Category D: As Category B, but assess only when infringements are greater than 35%.

The various dimensional criteria of the Standard are categorised as follows:

Clause	Category	Remarks
5.2.6.3	A	Check also concrete flange for cracks
figures 5.1, 5.2	C	Except for stud dimensions and widths of shear connectors which are category B and edge distances which are category D
5.3.3.2	C	
5.3.3.3(a)	C	
5.3.3.3(b)	D	
5.3.3.6(1)	B	
6.2.3.3	B	
6.3.3.1(b)	D	
figure 6.2	D	
6.3.3.7	D	

8.7.2	D	
8.8	A for 50mm dimension D for 600mm dimension	
9.6.4	B	If no evidence of honeycombing disregard this limit
11.1.2.2	C	
11.3.1	B	
11.3.4	B	
11.3.9	C	

APPENDIX H
SUGGESTED PROPERTIES OF INFILL MATERIAL NOT SATISFYING BD 44 FOR THE ASSESSMENT OF FILLER BEAMS

It is proposed that:

The local bond strength should not be taken greater than the greater of:

(i) for cemented material 1/40 times the compressive strength determined from large diameter cores or large undisturbed specimens when practical, but not greater than 0.50 N/mm^2.

for uncemented material the square root of the product of:

1/40 of the mean of the confined and unconfined compressive strength, and

the shear strength forecast by the principles of soil mechanics.

(ii) The flexural strength should not exceed the strength of the steel cross-section by more than:

35% for cemented materials, or

20% for uncemented materials.

In calculating the resistance of the composite cross section, the part of the fill in tension shall be disregarded and, to provide an additional margin of safety, the depth of the fill in excess of 200mm above the steelwork shall be reduced by:

30% for cemented materials, or

40% for uncemented materials.

Any material below the soffit of the steelwork should be neglected as this is liable to spall.

The approach has not been checked for situations in which the ratio of the depth of fill to the span is such that arching action in the fill occurs in the direction of the span. For this situation, the fill within the depth of the steelwork is to be considered as acting compositely with it, and the fill above is to be considered as spanning by arching action and anchored at the ends by friction against the composite section, or by lateral resistance from material of at least similar compressive strength and compaction.

APPENDIX I
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