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Contents

Summary of pages

This document comprises a front cover, an inside front cover, pages i and ii, pages 1 to 30, an inside back cover and a back cover.

Foreword

Publishing information

This Published Document is published by BSI and came into effect on 31 July 2008. It was prepared by Subcommittee B/525/10, *Bridges*, under the authority of Subcommittee B/525/2, *Structural use of concrete* and Technical Committee B/525, *Building and civil engineering*. A list of organizations represented on this committee can be obtained on request to its secretary.

Relationship with other publications

This Published Document gives non-contradictory complementary information for use in the UK with the Eurocode for concrete bridges, BS EN 1992-2 and its UK National Annex. Background is provided to some of the National Annex provisions where these differ from recommended values.

Presentational conventions

The provisions in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

The word "should" is used to express recommendations of this standard. The word "may" is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word "can" is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this standard. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

This Published Document is not to be regarded as a British Standard.

Introduction

When there is a need for guidance on a subject that is not covered by the Eurocode, a country can choose to publish documents that contain non-contradictory complementary information that supports the Eurocode. This Published Document provides such information and has been cited as a reference in the UK National Annex to BS EN 1992-2.

It is recommended that reference also be made to PD 6687:2006[1] which contains non-contradictory complementary information that supports BS EN 1992-1. Parts of PD 6687:2006 are relevant to the design of structures to BS EN 1992-2.

NOTE 1 Many of the clauses in BS EN 1992-1-1 are called up in BS EN 1992-2. Although the information provided in this Published Document relates to designs undertaken to BS EN 1992-2, for clarity, clause references are generally provided to the Eurocode part in which the relevant clause is given.

NOTE 2 References are also included to papers and publications that provide additional background or more detailed information associated with a specific issue. It should be noted that these papers and publications might also include material that is not fully in accordance with Eurocode requirements.

1 Scope

This Published Document contains non-contradictory complementary information for use with BS EN 1992-2 and its UK National Annex for the design of concrete structures. It does not cover assessment.

This Published Document gives:

a) background to the decisions made in the National Annex to BS EN 1992-2 for some of the Nationally Determined Parameters;

b) commentary on some specific subclauses from BS EN 1992-2;

c) commentary on the application to bridge structures of some specific subclauses of BS EN 1992-1-1 that are called up in BS EN 1992-2;

d) guidance on subjects not covered by BS EN 1992-2, but previously included in British Standards and other standards and codes of practice.

[1] PD 6687:2006 is being revised and is to be renumbered PD 6687-1.

2 General

2.1 Definition of National Authority [BS EN 1992-2:2005, Foreword]

Several clauses in BS EN 1992-2:2005 make reference to the "National Authority". In UK there is no single such body and, therefore, the NA to BS EN 1992-2:2005 defines the National Authority as "the body with a statutory responsibility for the safety of the structure". Such bodies will not necessarily have "national" coverage, for example in the case of a local Highway Authority. This definition aims to draw those aspects of the design for which reference has to be made to the "National Authority" within established processes of technical approval.

3 Basis of design

3.1 Actions and environmental influences [BS EN 1992-1-1:2004, 2.3.1]

BS EN 1992-1-1:2004, **2.3.1** allows thermal effects, settlements, and creep and shrinkage to be neglected at ultimate limit state provided they are not significant or because the ductility and rotation capacity of the elements affected are sufficient.

In assessing whether the ductility and rotation capacity of elements are sufficient it should be noted that there will be additional demand on the rotation capacity: a) when moment redistribution is carried out; and, b) to cater for differences between the stiffnesses assumed in an elastic design and the true element stiffnesses as the ultimate limit state is approached.

In the absence of a rigorous analysis, for solid sections of constant width, b, it may be assumed that the rotation capacity required to accommodate the differences between the stiffnesses assumed in an elastic design and the true element stiffnesses will not exceed 50% of the allowable plastic rotation capacity determined in accordance with BS EN 1992-1-1:2004, **5.6.3** (4) for sections where the area of main reinforcement exceeds $0.005bd$.

Generally, it will be reasonable to neglect thermal effects, settlements, and creep and shrinkage at ultimate limit state when moment redistribution is not carried out.

3.2 Partial factors for materials [BS EN 1992-1-1:2004, 2.4.2.4 (3) and Annex A]

BS EN 1992-1-1:2004, Annex A should only be used in conjunction with an execution specification that aligns fully with the special control and testing processes required by BS EN 1992-1-1:2004, Annex A and any additional special requirements of the National Authority.

4 Materials

4.1 Creep and shrinkage [BS EN 1992-1-1:2004, 3.1.4]

In the absence of more specific data, the relative humidity may be taken as 70% for the design of bridge structures in the UK.

4.2 Design compressive and tensile strengths [BS EN 1992-2:2005, 3.1.6 (101)P]

The value of α_{cc} in BS EN 1992-2:2005, Expression (3.15) is recommended to be 0.85 for bridges. This value is appropriate for calculations on bending and axial force in BS EN 1992-2:2005, **6.1**. However, there are other cases where a value of 1.0 is reasonable, such as for shear calculations, as the empirical model for shear is deemed to account for α_{cc}. Similarly, the membrane rules in BS EN 1992-2:2005, **6.109** already incorporate a 0.85 factor in the formula for direct compressive strength and, therefore, a value of 1.0 for α_{cc} is clearly justified. The NA to BS EN 1992-2:2005, therefore, identifies the clauses for which the value of α_{cc} should be taken as 0.85. For other clauses the value should be taken as 1.0.

The clauses for which the value of α_{cc} should be taken as 0.85 were determined to achieve consistency with the use of 0.85 for bending and axial force and 1.0 for shear. For example, α_{cc} is taken equal to 1.0 in the compression strut limit in BS EN 1992-1-1:2004, Expression (6.56), as this compression limit corresponds to that used in the variable angle shear truss model given in BS EN 1992-2:2005, **6.2** where α_{cc} is also taken as 1.0. Conversely, in BS EN 1992-1-1:2004, Expression (6.63), α_{cc} is given the value of 0.85 in the partially loaded area rules in order to give parity between these rules and those used for calculations of bending and axial force in BS EN 1992-2:2005, **6.1** when the entire cross-sectional area is loaded.

4.3 Confined concrete [BS EN 1992-1-1:2004, 3.1.9]

The general detailing requirements given in BS EN 1992-2:2005 should not be assumed to provide sufficient confinement for the model for increased concrete strength and strain capacity given in BS EN 1992-1-1:2004, **3.1.9** (2) to be used in the general design of elements for bending, axial force, shear and torsion.

The expressions provided for confined concrete may be used in checking triaxially confined nodes, in accordance with BS EN 1992-1-1:2004, **6.5.4** (6).

4.4 Ductility characteristic [BS EN 1992-2:2005, 3.2.4]

BS EN 1992-2:2005, **3.2.4** (101)P allows the classes of reinforcement that may be used in bridges to be defined. The recommended classes are Class B and Class C. The NA to BS EN 1992-2:2005 relaxes this requirement and allows the use of Class A fabric reinforcement provided it is not taken into account in the evaluation of the ultimate resistance.

The restriction on taking account of Class A fabric reinforcement in the evaluation of the ultimate resistance stems from the fact that the ultimate limit state deformation capacity of structures with Class A reinforcement can be very low compared with structures with Class B or Class C reinforcement [1], particularly for lightly reinforced elements. Therefore, its use as primary reinforcement for bridges is not allowed, including its use as shear reinforcement.

Class A fabric reinforcement may be used in the verification of serviceability criteria given in BS EN 1992-2:2005, Section 7 and the minimum reinforcement requirements given in BS EN 1992-2:2005, Section 9.

5 Durability and cover to reinforcement

5.1 Requirements for durability [BS EN 1992-1-1:2004, 4.3]

Half-joints should not be used in bridges unless there are adequate provisions for inspection and maintenance.

6 Structural analysis

6.1 Second order effects [BS EN 1992-1-1:2004, 5.1.4 (1)P]

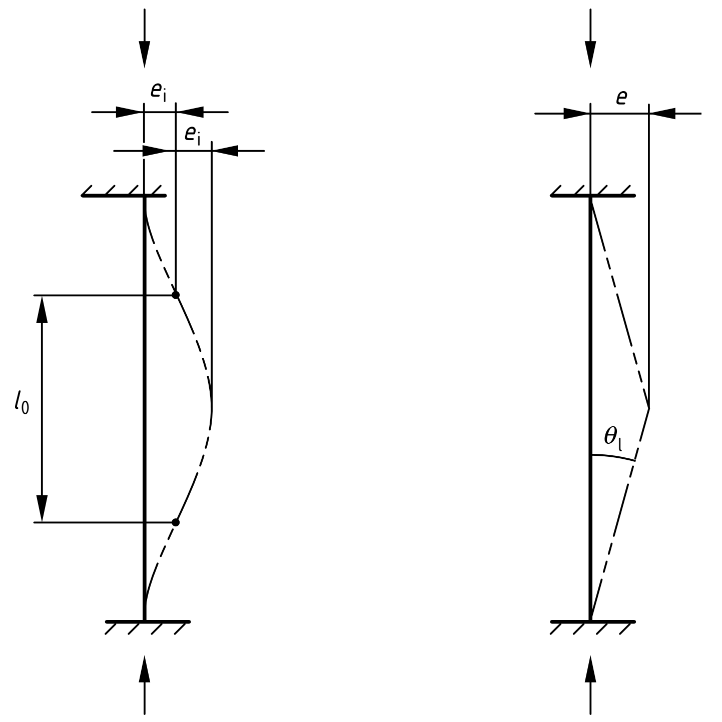
For externally prestressed members, second order effects between deviators, or other points where the tendon position in the section is fixed, need not be considered when the spacing between these points does not exceed 10 times section depth if the tendon force at the ultimate limit state is determined using BS EN 1992-1-1:2004, **5.10.8** (2). If non-linear analysis is used to determine the tendon force, second order effects should be considered.

6.2 Geometrical imperfections [BS EN 1992-1-1:2004, 5.2]

The disposition of imperfections used in analysis should reflect the behaviour and function of the structure and its elements. The shape of imperfection should be based on the anticipated mode of buckling of the member. For example, in the case of bridge piers, an overall lean imperfection should be used where buckling will be in a sway mode ("unbraced" conditions), while a local eccentricity within the member should be used where both ends of the member are held in position ("braced" conditions).

In using BS EN 1992-1-1:2004, Expression (5.2) the eccentricity, e_i, derived should be taken as the amplitude of imperfection over the half wavelength of buckling; Figure 1 shows the imperfection suitable for a pier rigidly built in for moment at each end. A lean imperfection should however be considered in the design of the positional restraints for braced members. Further guidance and background are given in Hendy and Smith [2].

Figure 1 **Imperfections for pier built in at both ends**



$l_0 = l/2$
$e = 2e_i = \theta_l l/2$

a) Sinusoidal imperfection b) Angular imperfection

6.3 Linear elastic analysis [BS EN 1992-1-1:2004, 5.4]

The use of fully cracked section properties to derive internal effects in members from indirect actions at the serviceability limit state is not permissible, as the formation of wide cracks implicit in this assumption cannot be accepted at the serviceability limit state. If cracking is considered in global analysis, the member stiffness used should be commensurate with the degree of cracking permitted at the serviceability limit state. This leads to the requirement in BS EN 1992-1-1:2004, **5.4** (3) that a "gradual evolution of cracking should be considered", which requires a non-linear analysis allowing for the effects of cracking and tension stiffening. Un-cracked global analysis in accordance with BS EN 1992-1-1:2004, **5.4** (2) may always be used as a conservative alternative to such considerations.

6.4 Linear elastic analysis with limited redistribution [BS EN 1992-1-1:2004, 5.5]

If a linear elastic analysis with limited redistribution is undertaken, shears and reactions used in the design should be taken as those either prior to redistribution or after redistribution, whichever is greater.

6.5 Plastic analysis [BS EN 1992-1-1:2004, 5.6]

BS EN 1992-2:2005, **5.6.1** (101)P allows the use of plastic analysis when permitted by National Authorities. Permission for the use of plastic analysis should therefore be sought from the relevant body on a project-specific basis, see **2.1** of this document. Typically this will be part of the technical approval process.

BS EN 1992-1-1:2005, **5.6.2** (5) allows the use of plastic methods to be extended to non-solid slabs if their response is similar to solid slabs. Plastic methods assume that structures have sufficient ductility for a complete collapse mechanism to form before any loss of section resistance occurs. Establishing whether such a condition is generally satisfied is not straightforward [3]. The assessment of whether the response of a non-solid slab is similar to a solid slab should include consideration of whether the non-solid slab will exhibit similar ductility to a solid slab. Particular caution is therefore necessary in cases where torsional hinges might form, because of their limited ductility.

6.6 Rotation capacity [BS EN 1992-1-1:2004, 5.6.3]

BS EN 1992-1-1:2004, **5.6.3** provides a method for verifying rotation capacity. The allowable rotation capacity may be determined from BS EN 1992-1-1:2004, Figure 5.6N, in which the variations in plastic rotation capacity with relative neutral axis depth, x_u/d, are plotted for Class B and Class C reinforcement and for different concrete strengths.

The curves plotted in BS EN 1992-1-1:2004, Figure 5.6N show an initial increase in rotation capacity with increasing neutral axis depth up to a peak value, beyond which the rotation capacity decreases. For neutral axis depths lower than that corresponding to the peak value, the rotation capacity is governed by reinforcement fracture. For higher neutral axis depths it is governed by concrete crushing.

Research has shown that rotation capacity might be subject to a size effect [1], whereby the rotation capacity of geometrically similar elements decreases with increasing size, and might also be lower for non-solid sections compared with rectangular beams and slabs [4]. Neither of these effects is explicitly dealt with in BS EN 1992-1-1:2004, Figure 5.6N.

The issue of size effect was recognized in BS 5400-4:1990, which placed a limit of 1.2 m on the maximum section depth for which moment redistribution was permitted. Therefore, it is recommended that moment redistribution is not used for sections deeper than 1.2 m unless a rigorous analysis of rotation capacity is undertaken.

The reduced rotation capacity of non-solid sections stems from the reduced compressive strain capacity of concrete compression flanges, as discussed in **7.1.1** of this document. This issue is also recognized in BS 5400-4:1990, with the expression for rotation capacity being dependent upon the relative neutral axis depth, x_u/d_e, taking d_e as equal to the effective depth for solid slabs or rectangular beams in a similar manner to BS EN 1992-2:2005, but, for non-solid sections, d_e is taken equal to the depth of the compression flange. This approach may be used conservatively in determining $\theta_{pl,d}$ in accordance with BS EN 1992-1-1:2004, Figure 5.6N. Alternatively, it is recommended that either moment redistribution is avoided or a rigorous analysis of rotation capacity is undertaken.

It will generally be reasonable to assume that the rotation capacity of large or non-solid sections will be sufficient for thermal effects, settlements, and creep and shrinkage to be neglected at the ultimate limit state, provided that no redistribution of moments is undertaken in the analysis. (See also **3.1** of this document.)

A comprehensive review of research on rotation capacity is given in fib Bulletin D'Information No. 242 [1].

6.7 Analysis of second order effects with axial load [BS EN 1992-1-1:2004, 5.8]

6.7.1 Slenderness and effective length of isolated members [BS EN 1992-1-1:2004, 5.8.3.2]

BS EN 1992-1-1:2004, Figure 5.7 of which shows effective lengths of isolated members does not cover typical bridge cases. The more usual bearing types for cantilever bridge piers are not included and cases a) to e) assume infinitely stiff end restraint. For these reasons, additional examples are given in Table 1 of this document.

The values given in Table 1 are based on the following assumptions:

a) rotational restraint is at least $4(EI)/l$ for cases 1, 2 and 4 to 6, and $8(EI)/l$ for case 7, where (EI) is the flexural rigidity of the column;

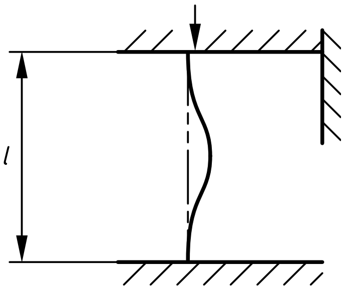
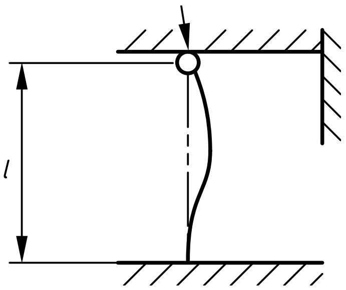
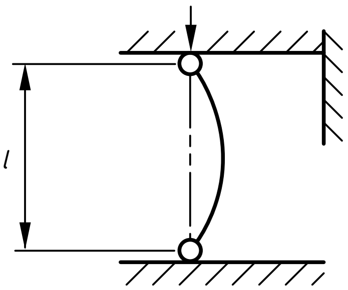
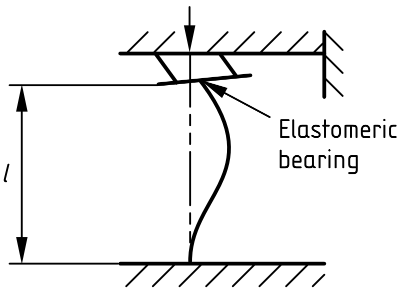
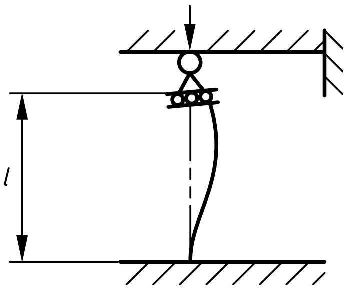
b) lateral and rotational rigidity of elastomeric bearings are negligible;

c) the height of bearing is negligible compared with that of the column.

Case 4 in Table 1 may also be used with roller bearings provided that the rollers are held in place by an effective means, such as racks.

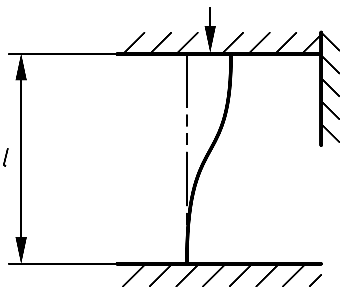
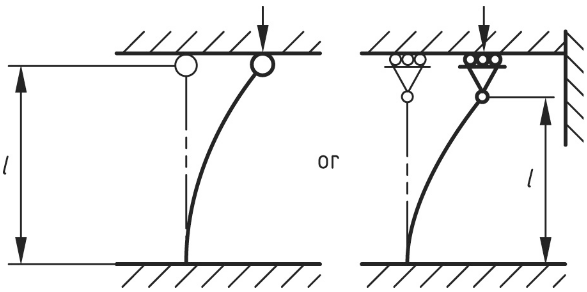
Where a more accurate evaluation of effective length is required, or where the end restraint stiffness is less than that given in item a) above, the effective length should derived from first principles (see Jackson [5]).

Table 1 **Effective height, l_0, for columns**

Case	Idealized column and buckling mode	Restraints			Effective height, l_0
		Location	Position	Rotation	
1		Top	Full	Full A)	$0.70l$
		Bottom	Full	Full A)	
2		Top	Full	None	$0.85l$
		Bottom	Full	Full A)	
3		Top	Full	None	$1.0l$
		Bottom	Full	None	
4		Top	None A)	None A)	$1.3l$
		Bottom	Full	Full A)	
5		Top	None	None	$1.4l$
		Bottom	Full	Full A)	

A) Assumed value (see **6.7.1**).

Table 1 **Effective height, l_0, for columns** (*continued*)

Case	Idealized column and buckling mode	Restraints			Effective height, l_0
		Location	Position	Rotation	
6		Top	None	Full [A)]	$1.5l$
		Bottom	Full	Full [A)]	
7		Top	None	None	$2.3l$
		Bottom	Full	Full [A)]	

A) Assumed value (see **6.7.1**).

6.7.2 Creep [BS EN 1992-1-1:2004, 5.8.4]

Because of the use of design ULS moment in the denominator of BS EN 1992-1-1:2004, Expression (5.19), it will not give full creep even for 100% permanent load. This, however, has been shown by Westerberg [6] to be safe for second order analysis.

7 Ultimate limit states

7.1 Bending with or without axial force [BS EN 1992-1-1:2004, 6.1]

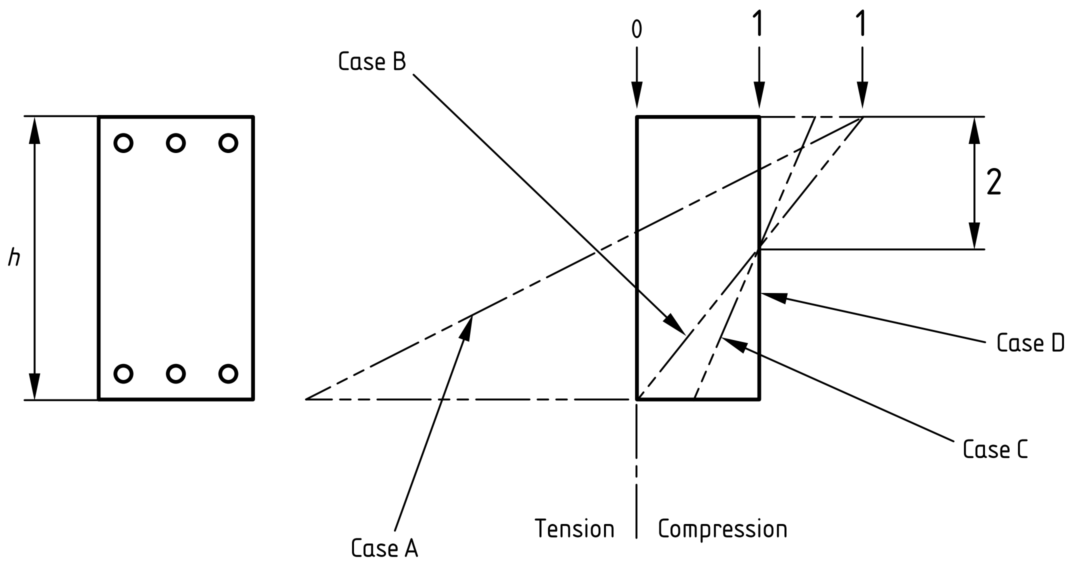
7.1.1 Strain distributions [BS EN 1992-1-1:2004, 6.1 (5) and 6.1 (6)]

In determining the bending and axial resistance of a section which is wholly in compression, the limiting strain in the concrete has to be reduced from the ultimate value, ε_{cu2} or ε_{cu3}, in accordance with BS EN 1992-1-1: 2004, **6.1** (6) and Figure 6.1. Similarly for wide flanges wholly in compression, the strain in the concrete should also be reduced in accordance with BS EN 1992-1-1:2004, **6.1** (6) and Figure 6.1, or in accordance with BS EN 1992-1-1:2004, **6.1** (5). Cases where the attached width of flange either side of the web is less than 3 times the flange depth can generally be assumed not to be wide.

The application of BS EN 1992-1-1:2004, **6.1** (6) and Figure 6.1 to a section wholly in compression, or to a wide flange wholly in compression, requires the strain in the concrete to be taken as either ε_{c2} at a distance of $(1 - \varepsilon_{c2}/\varepsilon_{cu2})h$ from the more compressive face, or ε_{c3} at a distance of $(1 - \varepsilon_{c3}/\varepsilon_{cu3})h$ from the more compressive face, depending upon the concrete stress-strain model used, as illustrated in Figure 2. Thus, when the neutral axis occurs at one face of the section, the limiting strain at the other face is equal to ε_{cu2} or ε_{cu3} (Case B in Figure 2). When the eccentricity of the loading is zero, so the section is in pure compression, the strain in the concrete throughout the section should be taken as ε_{c2} or ε_{c3} (Case D in Figure 2).

Case A in Figure 2 corresponds to a section that is not wholly in compression (i.e. the neutral axis lies within the depth of the section), in which case the limiting strain in the concrete is taken as equal to the ultimate value, ε_{cu2} or ε_{cu3}. Case C corresponds to a combination of axial load and bending that gives rise to a stress distribution between the limiting Cases B and D.

Figure 2 **Limiting strain distributions**



Case A Strain diagram for section with neutral axis within section

Case B Strain diagram for section with neutral axis at face of section

Case C Strain diagram for section fully in compression but with some bending

Case D Strain diagram for section in pure compression

1 ε_{c2} or ε_{c3}

2 $(1 - \varepsilon_{c2}/\varepsilon_{cu2})h$ or $(1 - \varepsilon_{c3}/\varepsilon_{cu3})h$

7.1.2 External prestressing strain between fixed points [BS EN 1992-2:2005, 6.1 (108)]

The wording in BS EN 1992-2:2005, **6.1** (108) differs from that in BS EN 1992-1-1:2004, **6.1** (8) to allow for the possibility of sliding caused by the difference in force on either side of “contact points” (i.e. deviators).

7.1.3 Robustness of prestressed elements [BS EN 1992-2:2005, 6.1 (109)]

One permissible way of preventing brittle fracture is to ensure that there is sufficient longitudinal reinforcement to compensate for the loss of resistance when the tensile strength of the concrete is lost due to cracking. This is achieved by providing a minimum area of reinforcement in accordance with BS EN 1992-2:2005, Expression (6.101a). This reinforcement is not additional to requirements for other effects and may be used in ultimate bending checks. Further guidance and background are given by Hendy and Smith [2].

The cracking moment forms the basis for the minimum steel. The NA to BS EN 1992-2:2005 recommends that it is calculated on the basis of the lower characteristic tensile strength rather than the mean value recommended in the BS EN 1992-2:2005. The use of the mean is considered unduly onerous and the lower characteristic value had been used in the UK for a number of years with no adverse effects.

7.2 Shear [BS EN 1992-1-1:2004, 6.2]

7.2.1 Evaluation of chord forces [BS EN 1992-1-1:2004, 6.2.1 (1)P]

For members with designed shear reinforcement, the forces in the tension and compression chords should be calculated from the truss model adopted and should be in equilibrium with the applied bending moment, axial force and shear force. The chord forces should not generally be assumed to be M_{Ed}/z as this could lead to significant error. If inclined chords are considered, the horizontal components of the chord forces may be obtained in the usual way for horizontal chords.

7.2.2 Shear enhancement [BS EN 1992-2:2005, 6.2.2 (101) and BS EN 1992-1-1:2004, 6.2.2 (6)]

BS EN 1992-1-1:2004, **6.2.2** (6) enables shear enhancement to be taken into account for a single load applied within $2d$ of the edge of a support, or centre of bearing where flexible bearings are used, by applying a reduction factor to this load. As acknowledged in PD 6687:2006, this approach is not suitable for cases with multiple, indirect or distributed loads. Therefore, in the NA to BS EN 1992-2 the expression for $C_{Rd,c}$ has been modified from the recommended value so that the effects of shear enhancement are taken into account through increasing the shear resistance of members near to supports. Modifying the expression for $C_{Rd,c}$ in this way makes the approach consistent with that used for punching shear resistance in BS EN 1992-1-1:2004, **6.4.4** (2) and with previous UK practice.

7.2.3 Shear resistance of uncracked prestressed members [BS EN 1992-1-1:2004, 6.2.2 (2)]

BS EN 1992-1-1:2004, **6.2.2** (2) limits the shear resistance of regions uncracked in bending and without designed shear reinforcement so that the tensile strength of the concrete is not exceeded.

Often the maximum principal tension will occur at the level of the centroid and BS EN 1992-1-1:2004, Expression (6.4) enables the principal tension to be checked at that level. Strictly, Expression (6.4) only applies to prismatic sections [2]. It is noted in BS EN 1992-1-1:2004, **6.2.2** (2), however, that the maximum principal tension might occur elsewhere in the section.

For prismatic sections, the requirement to limit the maximum principle tension to the tensile strength of the concrete at locations other than the centroidal axis may be checked using the following equation:

$$V_{\mathrm{Rd,c}} = \frac{Ib_{\mathrm{w}}}{S}\sqrt{(f_{\mathrm{ctd}})^2 + \sigma f_{\mathrm{ctd}}}$$

where the symbols are as defined in BS EN 1992-1-1, 2004, **6.2.2** (2), except that:

b_{w} is the width of the cross section at the location being checked, allowing for the presence of ducts, in accordance with BS EN 1992-1-1, 2004, **6.2.3** (6);

S is the first moment of area of the part of the section excluding any area below the location being checked, calculated about the centroidal axis of the whole section;

σ is the total direct stress in the section due to bending and axial load effects, and prestress effects, at the location being checked.

7.2.4 Members requiring design shear reinforcement [BS EN 1992-1-1:2004, 6.2.3]

7.2.4.1 Evaluation of inner lever arm [BS EN 1992-1-1:2004, 6.2.3 (1)]

In the design of members requiring shear reinforcement, BS EN 1992-1-1:2004, **6.2.3** (1) suggests that the lever arm, z, "normally" may be taken as $0.9d$. This value is not always appropriate, for example if: a) there is an axial force or prestress, or b) the width at the centroid is greater than the minimum cross-section width in compression, or c) the cross section has a tension flange but no compression flange (e.g. T-sections under reversed moment). In such cases the lever arm should be determined based on an analysis of the section under the applied actions and is generally the distance between the centroid of the more tensile chord and the more compressive chord.

7.2.4.2 Web crushing limits [BS EN 1992-2:2005, 6.2.3 (103)]

BS EN 1992-2:2005, **6.2.3** (103) can give very significant increases in the web crushing limit (the upper limit for shear) compared with past practice. This was investigated in the UK. The increases were found generally to be justified but there were some areas where there was insufficient evidence. In particular, the very high increases that can arise with inclined links did not appear to have been proved and the tests did not extend to sufficiently slender webs to cover all cases found in practice. Because of this, additional restrictions have been added in the NA to BS EN 1992-1-1 and the NA to BS EN 1992-2. Fuller details of the background are given by Jackson and Salim [7].

7.2.4.3 Design for smallest value of V_{Ed} in an increment [BS EN 1992-1-1:2004, 6.2.3 (5)]

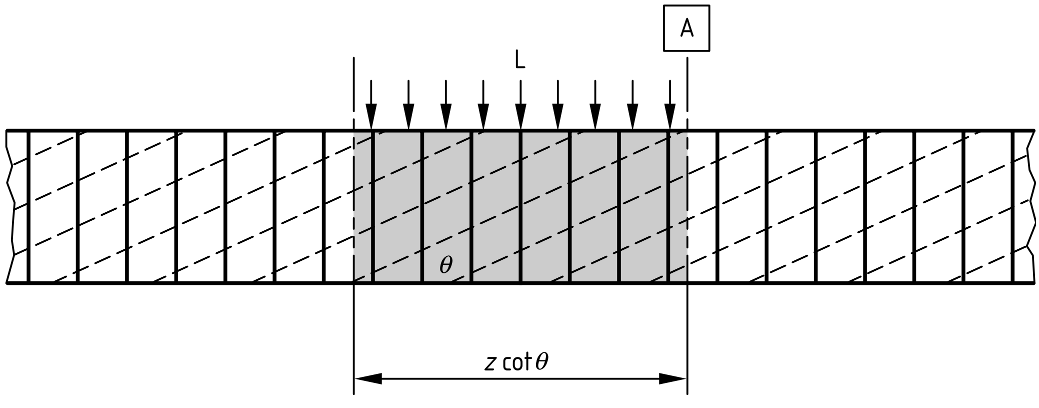
BS EN 1992-1-1:2004, **6.2.3** (5) allows shear reinforcement within an increment of length up to $z\cot\theta$ to be designed based on the smallest value of V_{Ed} in that increment provided that there is no discontinuity of V_{Ed}. However, no explicit explanation is given of what would constitute such a discontinuity.

The method for designing shear reinforcement based on a truss analogy is most clear for regions of constant shear force, where there is no load applied within an increment and the shear force at the left end of the increment is the same as at the right end of the increment. However, in loaded regions the shear force at the left end of an increment will be different to the shear force at the right end of that increment. It is also possible that the truss angle would be different at the left and right ends since a steeper angle could be required to prevent concrete crushing.

The presence of loading within an increment causes an increase in the shear force at one end of the length increment relative to the other end. However, the load in the length increment does not necessarily affect the forces in the shear reinforcement within that increment if the inclined struts transfer the load into the adjacent increment, as illustrated in Figure 3. Hence the shear reinforcement within the increment may be based on the minimum shear force in the increment, as long as the increment size is no greater than $z\cot\theta$ (see Figure 3). However, the truss angle for the increment and the increment size should be chosen to ensure that the maximum shear force in the increment does not exceed $V_{Rd,max}$. Furthermore, the lever arm, z, should be based on the minimum value within the length increment, which will typically correspond to the location of maximum force in the compression zone.

This approach is based on the assumption that there are no discontinuities of loading within a single increment, and might not be valid if the load increases within an increment or if there is a concentrated load within an increment. In these cases the increment size can be reduced to avoid any discontinuity, or the shear reinforcement can be based on the maximum shear force in the increment.

Figure 3 **Illustration of increment size for shear reinforcement design**



Links in shaded region based on the shear force at A

7.2.4.4 Additional tensile force [BS EN 1992-2:2005, 6.2.3 (107)]

Where unbonded tendons are used, the additional force required for shear will often be taken by bonded reinforcement. In cases where bonded reinforcement is not provided (as with segmental structures with no bonded secondary reinforcement) the force can be taken by the tendons themselves. However, because of the lack of bond, it is necessary to check that the force is adequate, and not just that there is sufficient steel to resist it. See also **10.2.1.3** in this document.

7.2.4.5 Segmental construction [BS EN 1992-2:2005, 6.2.3 (109)]

In applying BS EN 1992-2:2005, **6.2.3** (109), h_{red} should be taken as the depth of concrete in compression under the applied ultimate load. It will normally be greater than the depth used in a section analysis to determine the flexural resistance.

7.2.5 Shear between web and flange [BS EN 1992-1-1:2004, 6.2.4]

The rate of change of the flange forces can be underestimated if the effects of web shear on the flange forces are not included, particularly for compression flanges. The chord forces as determined for the design of the web reinforcement should therefore be used as the basis of the flange forces in BS EN 1992-1-1:2004, **6.2.4**.

For compression flanges, F_d may be calculated as follows:

$$F_d = \frac{b_{eff} - b_w}{2b_{eff}} F_{cd} \quad \text{when} \quad h_f > x_c$$

$$F_d = \frac{b_{eff} - b_w}{2} f_{cd} h_f \quad \text{when} \quad h_f \leqslant x_c$$

where:

x_c is the depth of the compression zone;

F_{cd} is the total force in the compression chord of the shear truss in the web.

For tension flanges, F_d may be calculated by determining the proportion of the total tension chord force, F_{td}, carried in each side of the flange.

7.2.6 Shear at the interface between concrete cast at different times [BS EN 1992-1-1:2004, 6.2.5]

Where a stepped distribution of transverse reinforcement is used in accordance with BS EN 1992-1-1:2004, **6.2.5** (3), the total resistance within any band of reinforcement should be not less than the total longitudinal shear in the same length and the longitudinal shear stress evaluated at any point should not exceed the resistance evaluated locally by more than 10%.

7.3 Punching [BS EN 1992-1-1:2004, 6.4]

7.3.1 Distribution of shear with eccentric support reaction

The expressions for W_1 in BS EN 1992-1-1:2004, **6.4.3** are derived for the basic perimeter, u_1, at a distance $2d$ from the load. Theoretically, they need adjustment for other perimeters at a distance r_i from the load. However, these expressions for W_1 may be used for perimeters inside the basic perimeter at $2d$, except in the design of bases in accordance with BS EN 1992-1-1:2004, **6.4.4**, because the punching rules were calibrated against tests basing W_1 on the u_1 perimeter.

For the design of bases in accordance with BS EN 1992-1-1:2004, **6.4.4**, the expressions for W_i have to be modified for the actual perimeter before being used in BS EN 1992-1-1:2004, Expression (6.51). Therefore, from BS EN 1992-1-1:2004, Expression (6.40) it follows that for a general perimeter at r_i with length u_i, the simplified expressions BS EN 1992-1-1:2004, Expressions (6.41) and (6.42) need adjustment as follows:

For a square column and a general perimeter at r_i with length u_i:

$$W_i = \frac{c_1^{\,2}}{2} + c_1 c_2 + 2c_2 r_i + 4r_i^{\,2} + \pi r_i c_1$$

For an internal circular column and a general perimeter at r_i with length u_i:

$$\beta = 1 + 0.6\pi \frac{e}{D + 2r_i}$$

7.3.2 Distribution of shear reinforcement [BS EN 1992-1-1:2004, 6.4.5]

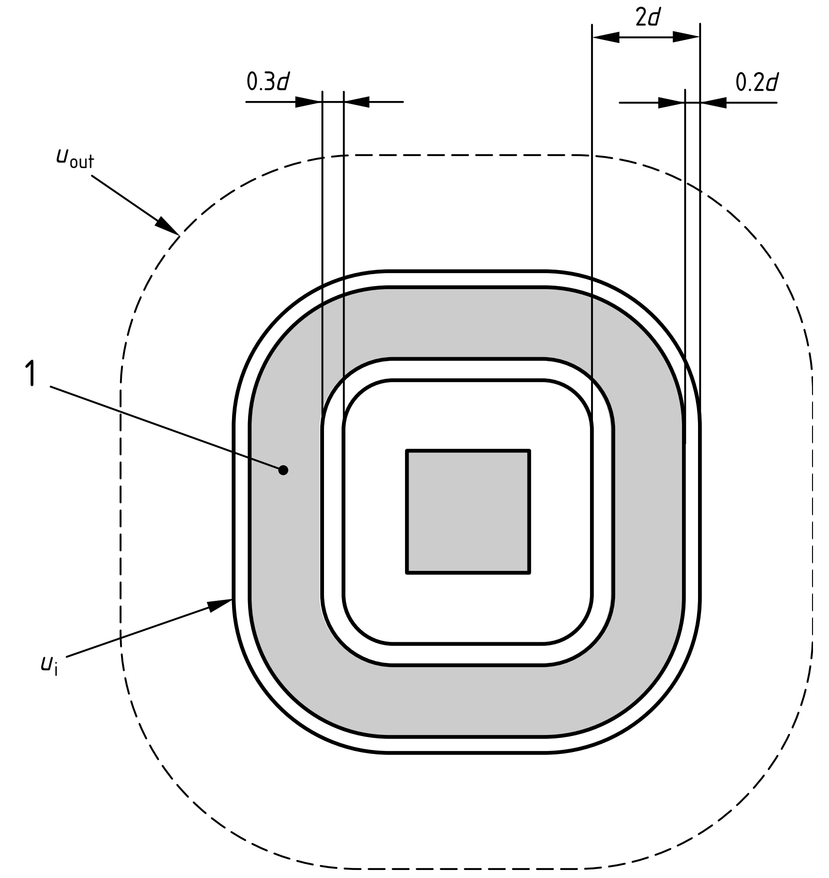
BS EN 1992-1-1:2004, Expression (6.52) has been presented assuming a constant area of shear reinforcement on each perimeter moving away from the loaded area as shown in BS EN 1992-1-1:2004, Figure 6.22. In cases where the reinforcement area varies on successive perimeters the required shear reinforcement may be determined by checking successive perimeters, u_i, between the basic control perimeter at $2d$ and the perimeter u_{out}, to ensure that shear reinforcement of area $\sum A_{sw}$ satisfies the following expression:

$$\sum A_{sw} = \frac{\left(v_{Ed} - 0.75 v_{Rd,c}\right) u_i d}{f_{ywd,ef} \sin\alpha}$$

where $\sum A_{sw}$ is the total shear reinforcement, shown in Figure 4, placed within an area enclosed between the control perimeter, u_i, chosen and one $2d$ inside it, except that shear reinforcement within a distance of $0.3d$ from the inner perimeter and $0.2d$ from the control perimeter should be ignored.

Further guidance and background are given by Hendy and Smith [2].

Figure 4 **Reinforcement $\sum A_{sw}$ to include in punching check**



Key

u_i is between u_1 and u_{out}

1 Reinforcement ΣA_{sw} to include for check on perimeter u_i

7.4 Design with strut and tie models [BS EN 1992-1-1:2004, 6.5]

7.4.1 Struts [BS EN 1992-1-1:2004, 6.5.2]

The compressive stress that a concrete strut can carry is strongly affected by its multi-axial state of stress. Transverse compression is beneficial while transverse tension reduces the concrete strut's compressive resistance. Therefore the two simplified and conservative limits are given in BS EN 1992-1-1:2004, **6.5.2**.

The limit given in BS EN 1992-1-1:2004, Expression (6.56) relates to a safe lower bound stress that can be assumed for all compression struts, provided that the strut and tie idealization does not depart significantly from elastic stress trajectories. It is the same limit for strut compressive stress used in the calculation of $V_{Rd,max}$. This limit does not distinguish between cracking running parallel to the strut and the more detrimental cracking skew to the strut, or between applied transverse tensile forces that are carried by reinforcement and those which arise purely from an elastic bulging of the struts between nodes. Further, it does not account for the actual magnitude of tensile strain, which is also relevant.

In reality, different limits apply in different situations, and this is acknowledged in BS EN 1992-1-1:2004, **6.5.2** (2) which allows a more rigorous approach to be used.

Further guidance on strut compressive limits is given by Hendy and Smith [2] and Schlaich, Shafer and Jennewein [8].

7.5 Partially loaded areas [BS EN 1992-1-1:2004, 6.7]

BS EN 1992-1-1:2004, **6.7** (1)P requires that partially loaded areas are designed accounting for both local concrete crushing and for transverse tensile forces. Reference is made to BS EN 1992-1-1:2004, **6.5** for the latter indicating that transverse tensile forces should be determined from a strut and tie model.

Transverse tensile forces can arise in the localized area near a concentrated load and also from any further spread of load outside this localized area. The strut and tie model used should account for both of these potential sources of transverse tensile forces. Further guidance and background are given by Hendy and Smith [2].

7.6 Fatigue [BS EN 1992-1-1:2004, 6.8]

7.6.1 Verification conditions [BS EN 1992-2:2005, 6.8.1 (102)]

Because of the high live load to dead load ratio, deck slabs are likely to be amongst the elements most affected by fatigue calculations. However, tests show that the actual stress ranges in the reinforcement in these are much lower than conventional elastic calculations suggest. Because of this, the NA to BS EN 1992-2:2005 identifies cases where fatigue assessment is not required and provides conservative rules.

7.6.2 Verification of concrete under compression or shear [BS EN 1992-2:2005, 6.8.7 (101)]

S-N curves needed to undertake a fatigue verification of concrete under compression or shear in accordance with BS EN 1992-2:2005, **6.8.7** (101), are unlikely to be available from National Authorities. In the absence of such data, the simplified approach given in BS EN 1992-2:2005, Annex NN may be used for railway bridges, but no such option exists for highway bridges. The fatigue verification may however be undertaken in accordance with BS EN 1992-1-1:2004, **6.8.7** (2).

7.6.3 Limiting stress range for reinforcement under tension [BS EN 1992-1-1:2004, 6.8.6 (1)]

The stress range given in BS EN 1992-1-1:2004, **6.8.6** (1) is applicable in all situations and to all loadings and so, effectively, has to be a non-propagating stress range. For specific types of loading, where the numbers of load cycles within the design life are limited, it is possible to devise higher deemed to satisfy limits and the NA to BS EN 1992-1-1:2004 allows these to be agreed with “appropriate authorities”. For the purposes of design to BS EN 1992-2:2005 the "appropriate authorities" should be taken to be the National Authority as defined in the NA to BS EN 1992-2:2005. For UK highway bridges, the values in Table 2 may be used for straight reinforcement. These are based on bars conforming to BS 4449. For bars not conforming to BS 4449, the rules for bars >16 mm diameter should be used for all sizes unless the ranges for bars ⩽16 mm diameter can be justified.

Table 2A **Limiting stress ranges – Longitudinal bending for unwelded reinforcing bars in road bridges**

Span	**Adjacent spans loaded**		**Alternate spans loaded**	
m	**Bars ⩽16 mm** MPa	**Bars >16 mm** MPa	**Bars ⩽16 mm** MPa	**Bars >16 mm** MPa
⩽3.5	150	115	210	160
5	125	95	175	135
10	110	85	175	135
20	110	85	140	110
30 to 50	90	70	110	85
100	115	90	135	105
⩾200	190	145	200	155

NOTE 1 Intermediate values may be obtained by linear interpolation.

Table 2B **Limiting stress ranges – Transverse bending for unwelded reinforcing bars in road bridges**

Span m	**Bars ⩽16 mm** MPa	**Bars >16 mm** MPa
⩽3.5	210	160
5	120	90
⩾10	70	55

NOTE 2 Intermediate values may be obtained by linear interpolation.

NOTE 3 Table 2 applies to slabs but need only be applied to those slabs which do not conform to the criteria given the NA to BS EN 1992-2:2005, ***6.8.1*** *(102).*

7.7 Shell elements [BS EN 1992-2:2005, Annex LL]

BS EN 1992-2:2005, Annex LL Clause 112 has been amended in the NA to BS EN 1992-2:2005 to allow the use of alternative realistic models instead of those given in BS EN 1992-2:2005, **6.109** and Annex F. For the design or verification of shell elements subject to bending alone (i.e. with zero membrane forces) the approaches given by Wood [9] Armer [10] and Denton and Burgoyne [11] may generally be used.

8 Serviceability limit states

8.1 Stress limitation [BS EN 1992-1-1:2004, 7.2]

8.1.1 Concrete in compression [BS EN 1992-2:2005, 7.2 (102)]

In areas exposed to environments of exposure class XD, XF or XS, it is recommended that the stress limit for concrete in compression specified in BS EN 1992-2:2005, **7.2** (102) should be applied.

8.1.2 Reinforcement [BS EN 1992-1-1:2005, 7.2 (5)]

Satisfying the stress limitations for reinforcement given in BS EN 1991-1-1:2004, **7.2** (5) does not remove the requirement to verify crack control and deflection control in accordance with BS EN 1992-2:2005, **7.3** and **7.4**, respectively.

8.2 Crack control [BS EN 1992-1-1:2004, 7.2]

8.2.1 Recommended values of w_{max} [BS EN 1992-2:2005, Table 7.101N]

The background to the changes in the NA to BS EN 1992-2:2005, **NA.2.2** and Table NA.2, from the recommended values in BS EN 1992-2:2005, Table 7.101N is as follows.

a) For XD or XS exposure, BS EN 1992-2:2005, Table 7.101N requires only decompression. While this will be satisfactory in the immediate vicinity of prestressing tendons, cracking could still occur in parts remote from the tendons. The NA to BS EN 1992-2:2005 has therefore given a crack width limit for these zones.

b) The decompression requirement is specified for durability of tendons. The recommended distance from the tendons where decompression has to be checked is given in BS EN 1992-2:2005 as 100 mm. Since 100 mm is likely to be greater than the cover required for durability, this introduces an anomaly. For example, only 50 mm of concrete in compression might be deemed adequate to protect the tendons, whereas 60 mm of concrete in compression plus 40 mm not in compression would not, which is clearly illogical. Changing the distance from the recommended value of 100 mm to the cover required for durability, $c_{min,dur}$, is more logical.

c) Because the exposure is defined at the surface but decompression is checked at the tendon, there was a lack of clarity when the surface with the most severe exposure classification is not the most tensile face.

d) It has been clarified that the exposure used is the worst the surface is exposed to, even if the cracking will arise at a different time.

e) It has been found that compliance with the recommended rules is unduly restrictive in the case of tops of the precast beams at the supports that have been made continuous for live load. The recommendations in the NA to BS EN 1992-2:2005 relax the rules in line with past practice as given in BD 57 [12] and BS EN 15050:2007, Annex D.

8.2.2 Concrete cover to be used in the evaluation of crack spacing and crack width [BS EN 1992-1-1:2004, 7.3.4]

In BS EN 1992-1-1:2004, Expression (7.11) the cover, c, should be taken as c_{nom}. The use of the resulting value of $s_{r,max}$ in BS EN 1992-1-1:2004, Expression (7.8) will then provide an estimation of the crack width at the surface of the concrete. In some situations, such as structures cast against the ground, c_{nom} will be significantly greater than the cover required for durability. Where there are no appearance requirements it is reasonable to determine the crack width at the cover required for durability and verify that it does not exceed the relevant maximum crack width.

This may be done by multiplying the crack width determined at the surface by $(c_{min,dur} + \Delta c_{dev})/c_{nom}$ to give the crack width at the cover required for durability, and verifying that it is not greater than w_{max}. This approach assumes that the crack width varies linearly from zero at the bar. Given the accuracy of crack calculation methods this simplification is considered reasonable.

NOTE It is planned that such an approach will be included in revisions to the National Annexes to BS EN 1992-1-1 and BS EN 1992-2.

8.2.3 Crack width due to restrained imposed deformation

BS EN 1992-2:2005 does not provide guidance on calculating crack widths due to early age restraint of imposed deformation, which can arise due to early thermal contraction and shrinkage. Such effects should be taken into account in design. Complementary guidance is provided in CIRIA Report C660 [13].

9 Detailing of reinforcement and prestressing tendons – General

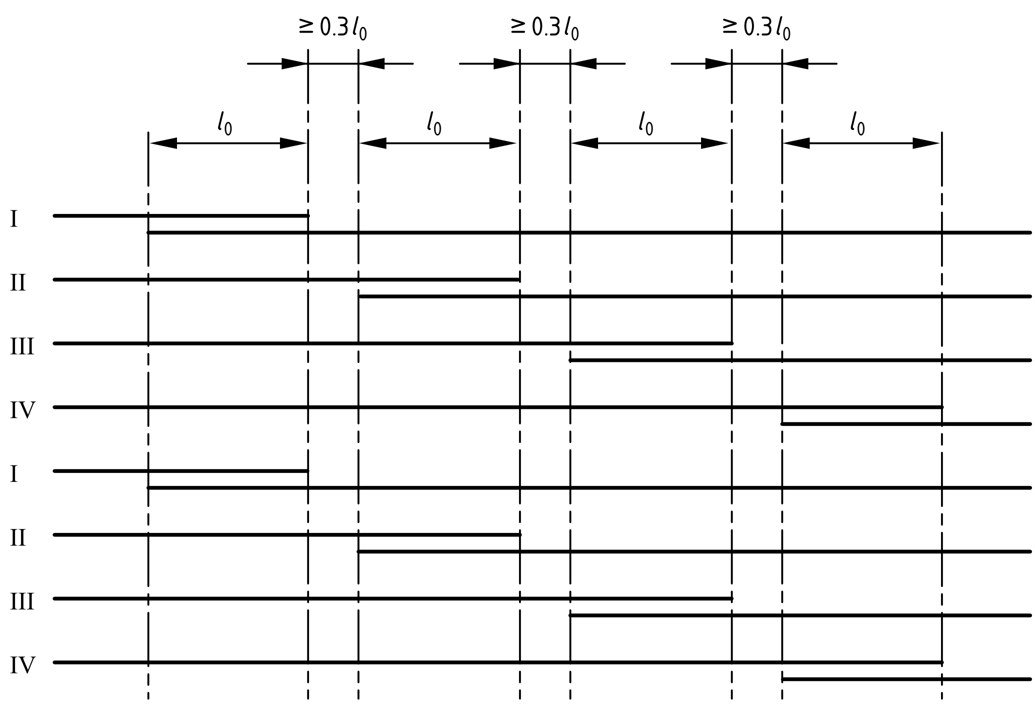
9.1 Laps [BS EN 1992-1-1:2004, 8.7]

BS EN 1992-1-1:2004, **8.7.2** (2) requires that laps should normally be staggered. This should be regarded as good practice but there might be situations where this requirement cannot be fulfilled. The definition of a staggered lap is effectively a lap where the longitudinal distance between it and adjacent laps is not less than $0.3l_0$.

Where the provisions of BS EN 1992-1-1:2004, **8.7.2** (3) and Figure 8.7 are met, BS EN 1992-1-1:2004 **8.7.2** (4) allows 100% of the bars in tension detailed in this way to be lapped when the bars are all in one layer. Different patterns of staggering will however lead to different required lap lengths through the parameter α_6. BS EN 1992-1-1:2004, Table 8.3 and Figure 8.8 introduce a definition of the percentage of laps in a section. This definition is used solely for determining α_6 and transverse reinforcement requirements in BS EN 1992-1-1:2004, **8.7.4**. It is not the same as the definition of the percentage of lapped bars used in BS EN 1992-1-1:2004, **8.7.2** (4), which effectively states that when all the bars in a layer are lapped in accordance with BS EN 1992-1-1:2004, Figure 8.7, 100% lapping of the reinforcement is achieved. Thus, if all the bars in a layer are staggered alternately in accordance with BS EN 1992-1-1:2004, Figure 8.7, this will constitute 50% lapping in a section in accordance with BS EN 1992-1-1:2004, Table 8.3. If the bars are in several layers, only 50% of bars may be lapped in accordance with BS EN 1992-1-1:2004, **8.7.2** (4). In this situation, this may be interpreted as a requirement not to have identical lapping arrangements overlying each other in each layer. This may be achieved by ensuring the staggering requirement of maintaining a distance of $0.3l_0$ between adjacent laps is maintained between layers as well as within layers. This requirement need not applied to bars in opposite faces of a beam or slab where the sign of the stress is different in each face. Further guidance and background are given by Hendy and Smith [2].

The intention of BS EN 1992-1-1:2004, **8.7.4.1** (2) is to permit the minimum transverse reinforcement already provided as links or distribution reinforcement to be used to satisfy the requirement for transverse reinforcement provision without any further justification when laps are quadruply staggered, see Figure 5. Therefore, the criterion that less than 25% of the reinforcement has to be lapped in one section may be taken as less than or equal to 25%.

Figure 5 **Illustration of quadruply staggered laps**



Where the diameter of the lapped bars is greater than or equal to 20 mm, BS EN 1992-1-1:2004 **8.7.4.1** (3) requires transverse reinforcement to be provided with a total area, ΣA_{st} , of not less than the area, A_s, of one lapped bar assuming that the lapped bar is fully stressed. The transverse reinforcement is required to be placed perpendicular to the direction of the lapped reinforcement at no more than 150 mm centres. For skew reinforcement with reinforcement ratio ρ_x at an angle ϕ from the perpendicular to the lapping bars, the effective reinforcement ratio transverse to the lapping bars should be taken as $\rho_x \cos^4 \phi$. This effective reinforcement ratio should be used in the calculation of ΣA_{st}.

In cases where transverse reinforcement is required by BS EN 1992-1-1:2004, **8.7.4.1** (3), that clause states that it should be placed between the lapped bars and the concrete surface. Clearly, this is only practical to apply in beams and one-way spanning slabs; the rules cannot conveniently be applied to two-way spanning slabs. In such circumstances, where it is impractical or impossible to comply with this requirement, the lapped bars should either be quadruply staggered or a value of α_3 of 1.0 should be used in lap length calculation.

9.2 Spacing of post-tension ducts [BS EN 1992-1-1:2004, 8.10.1.3]

In the absence of the provision of reinforcement between ducts to prevent splitting of the concrete, designed in accordance with the strut and tie rules given in BS EN 1992-2:2005, **6.5**, the minimum centre to centre duct spacings should be as given in Table 3. These spacings are as given in BS 5400-4:1990.

9.3 Anchorage zones of post-tensioned members [BS EN 1992-1-1:2004, 8.10.3]

The design of anchorage zones in post-tensioned members should include consideration of the following:

- the highly stressed compression concrete in the immediate vicinity of the anchorages;
- spalling at the loaded face;
- bursting stresses generated in the localized area of the anchorage;
- transverse tensile forces arising from any further spread of load outside this localized area.

Forces in reinforcement resulting from the above factors may be determined using the methods given in CIRIA Guide 1 [14] as a satisfactory method of compliance with BS EN 1992-2:2005, **8.10.3** (104) of. The design pretressing force should be taken as $\gamma_{p,unfav}P_{max}$.

BS EN 1992-2:2005, Annex J should be used with caution as, for bursting and spalling, it leads to minimum reinforcement requirements only and does not consider the dispersal of load beyond the primary prism. The bearing pressure behind the bearing plate should be verified in accordance with BS EN 1992-1-1:2004, **8.10.3** (3). Further guidance and background are given by Hendy and Smith [2].

Table 3 **Minimum spacing of post-tension ducts**

Radius of curvature of duct (m)	**Duct internal diameter** (mm)															
	19	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**	**130**	**140**	**150**	**160**	**170**
	Tendon force (kN)															
	296	**387**	**960**	**1337**	**1920**	**2640**	**3360**	**4320**	**5183**	**6019**	**7200**	**8640**	**9424**	**10336**	**11248**	**13200**
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
2	110	140	350	485	700	960										
4	55	70	175	245	350	480	610	785	940					Radii not normally used		
6	38	60	120	165	235	320	410	525	630	730	870	1045				
8			90	125	175	240	305	395	470	545	655	785	855	940		
10			80	100	140	195	245	315	375	440	525	630	685	750	815	
12						160	205	265	315	365	435	525	570	625	680	800
14						140	175	225	270	315	375	450	490	535	585	785
16							160	195	235	275	330	395	430	470	510	600
18								180	210	245	290	350	380	420	455	535
20									200	220	265	315	345	375	410	480
22											240	285	310	340	370	435
24												265	285	315	340	400
26												260	280	300	320	370
28																345
30																340
32																
34																
36																
38																
40	38	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340

NOTE 1 The tendon force shown is the maximum normally available for the given size of duct (taken as 80% of the characteristic strength of the tendon).

NOTE 2 Values less than 2 × duct internal diameter are not included.

9.4 Deviators [BS EN 1992-1-1:2004, 8.10.5]

Analysis to determine the forces in unbonded tendons is normally arranged to give a low estimate of tendon force because this is safe for the overall check of the structure. However, this means it is not safe to use it to design the deviators. Deviators, therefore, have to be checked for the ultimate limit state for the characteristic strength of the tendons. Although it might be argued a factor of γ_m should be applied to increase this, it is considered that other conservative features of the analysis make this unnecessary and a factor of 1.0 may be used. Where serviceability checks are required, the tendon forces before long term losses should be used.

In the absence of test results or other investigations justifying smaller values, the radius of curvature of tendons in the deviators should be not less than the values given in Table 4.

Table 4 **Minimum radius of curvature of tendons in the deviators**

Tendons		**Minimum radius of curvature**
Number of strands	**Diameter** mm	m
19	13	2.5
12	15	2.5
31	13	3.0
19	15	3.0
53	13	5.0
37	15	5.0

The limits given in Table 4 are based on recent UK practice and were originally taken from the SETRA document *External prestressing* [15].

10 Detailing of members and particular rules

10.1 Shear reinforcement and torsion reinforcement [BS EN 1992-1-1:2004, 9.2.2 and 9.2.3]

BS EN 1992-2:2005, **9.2.2** (101) and BS EN 1992-1-1:2004, **9.2.3** (1) give requirements for the angle between the shear reinforcement and the axis of the structural element and between the torsion reinforcement and the axis of the structural element, respectively. These requirements can be impractical for haunched elements and reinforcement links perpendicular to either surface of the element generally may be used.

10.2 Compression reinforcement of beams and columns [BS EN 1992-1-1:2004, 9.2.1.2 (3) and 9.5.3 (6)]

Where the design of a section has included the contribution of any longitudinal compression reinforcement in the resistance calculation, such longitudinal compression reinforcement should be effectively restrained by transverse reinforcement. Effective restraint may be achieved by satisfying all of the following conditions.

a) Links should be so arranged that every corner and alternate bar or group in an outer layer of compression reinforcement is held in place by a link anchored in accordance with BS EN 1992-1-1:2004, Figure 8.5 a) or b).

b) All other compression reinforcement should be within 150 mm of a bar held in place by a link.

c) The minimum size of the transverse reinforcement and links, where necessary, should be in accordance with BS EN 1992-1-1:2004, **9.5.3** (1).

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

10.3 Additional longitudinal reinforcement with unbonded tendons [BS EN 1992-1-1:2004, 9.2.1.3 (2)]

If unbonded tendons are used to resist the additional force ΔF_{td} it is necessary [as discussed in the comments on **6.2.4** (see **7.2.5** of this document)] to check that the tendon force is adequate. This can be done by considering equilibrium from first principles or by undertaking the flexural checks ignoring increases in prestress force or eccentricity in the distance a_l from the section considered.

10.4 Pile caps

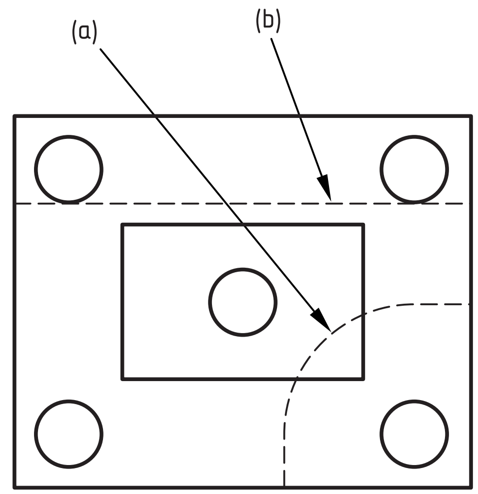
Where the distance between the edge of a pile and a pier is less than $2d$, some of the shear force in the pile cap will be transmitted directly between the pier and the pile via a strutting action. The basic punching perimeter cannot be constructed without encompassing a part of the support as shown in perimeter (a) in Figure 6. In such cases, it is recommended that the tension reinforcement is provided with a full anchorage beyond the line of the pile centres and that the shear design takes into consideration the following in addition to other verifications required by BS EN 1992-2:2005.

a) *Flexural shear on planes passing across the full width of the pile cap*. Flexural shear should be checked on planes passing across the full width of the pile cap, such as plane (b) in Figure 6. Shear enhancement should be taken into account by an increase to the concrete resistance and not by a reduction in the shear force, see **7.2.2** of this document. Where the spacing of the pile centres is less than or equal to 3 pile diameters, the short shear span enhancement may be applied over the whole section. Where the spacing is greater than this, the enhancement may only be applied on strips of width 3 pile diameters centred on each pile. The shear span, a_v, should be taken as the distance between the face of the column or wall and the nearer edge of the piles plus 20% of the pile diameter.

b) *Maximum permissible shear stress for punching.* The maximum permissible shear stress at the face of piles and piers should be checked in accordance with BS EN 1992-1-1:2004, Expression (6.53). The shear perimeter for corner piles should be the pile perimeter, or a perimeter passing partially around the pile and extending out to the free edges of the pile cap, whichever is less.

c) *Punching resistance of corner piles.* Corner piles should be checked for punching resistance at a $2d$ perimeter (without support enhancement) ignoring the presence of the pier or support and any vertical reinforcement within it.

Figure 6 **Corner pile within 2*d* of a column base**



Key

(a) $2d$ perimeter

(b) Flexural shear plane across cap

10.5 Requirements for voided slabs

10.5.1 General

The recommendations given in **10.5.2** to **10.5.5** should be used for the design of voided slab bridge decks cast in situ.

10.5.2 Transverse shear

The effects of cell distortion due to transverse shear should be considered. In particular:

a) the increased stresses in the transverse reinforcement and shear links due to cell distortion resulting from transverse shear should be calculated by an appropriate analysis (e.g. an analysis based on the assumption that the transverse section acts in a manner similar to a Vierendeel frame);

b) the resistance of the flanges and webs to the local moments produced by the transverse shear effects should be verified.

The top and bottom flanges should be designed as solid slabs, each to carry a part of the global transverse shear force proportional to the flange thickness.

10.5.3 Longitudinal shear

The longitudinal ribs between the voids should be designed as beams to resist the shear forces in the longitudinal direction, including any shear due to torsional effects.

10.5.4 Punching

Guidance on the punching of loads through a voided slab as a whole is given by Clark and Thorogood [16].

Punching of wheel loads through the top flange of decks with circular voids generally need only be considered for unusually thin flanges, typically those with void diameter to slab depth ratios of greater than 0.75 [17], [18]

10.5.5 Transverse reinforcement

In the absence of a detailed analysis, the minimum transverse reinforcement provided should be as follows [19]:

a) in the predominantly tensile flange either 1 500 mm^2/m or 1% of the minimum flange section, whichever is the lesser;

b) in the predominantly compressive flange either 1 000 mm^2/m or 0.7% of the minimum flange section, whichever is the lesser.

The spacing of the transverse reinforcement should not exceed twice the minimum flange thickness.

In skew voided slabs, it is preferable for the transverse steel to be placed perpendicular to the voids and the longitudinal steel to be placed parallel to the voids.

11 Additional rules for precast concrete elements and structures

11.1 Dynamic effects [BS EN 1992-1-1:2004, 10.2 (2)]

In the absence of more accurate analysis, dynamic factors of 0.8 and 1.2 may typically be used for precast elements for lifting and transport.

12 Additional rules for external prestressing

The following additional rules are recommended for the design of structures with external prestressing. The need for their application should be determined on a project-specific basis.

a) All external and unbonded tendons should be replaceable. Where the detailing does not enable tendons to be removed and replaced without damage to either the tendons or the structure, a method statement defining how the tendons can be replaced should be provided. A method statement defining how the structure can be demolished should also be provided.

b) Where it is necessary to restrict traffic on the bridge to replace the tendons, the extent of this restriction should be agreed with the client and defined in a method statement

c) Bridges should be checked to ensure that the removal either of any two tendons or of 25% of those at one section, whichever has the more onerous effect, will not lead to collapse at the ultimate limit state under the design ultimate permanent loads.

d) Where tendon restraints are widely spaced (typically at distances greater than $12h$) checks should be made to ensure the natural frequency of the free length of tendons is not resonant with that of the structure as a whole.

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